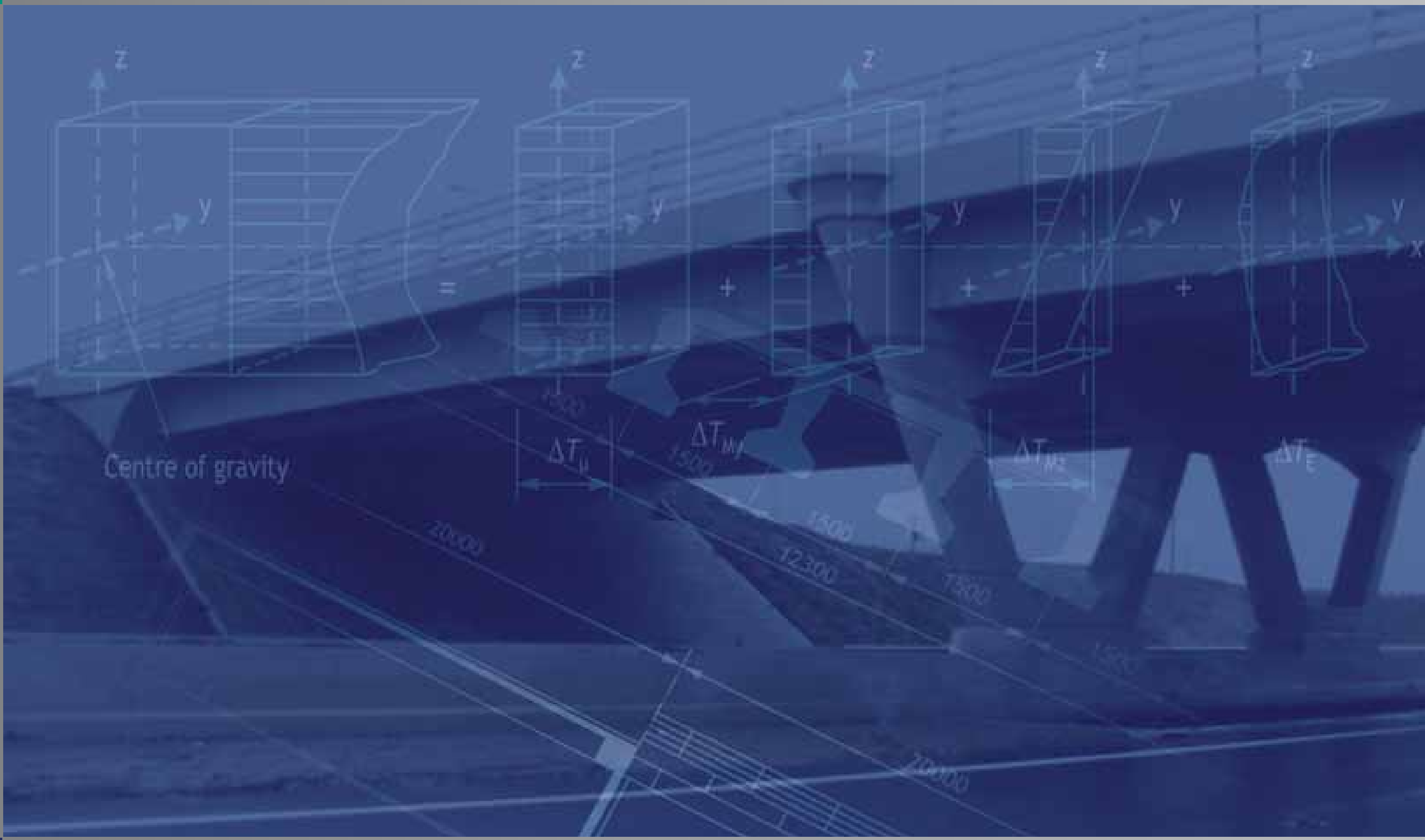
A **cement** and **concrete** industry publication

Integral Concrete Bridges to Eurocode 2

Commentary and a worked example of a two span bridge

Technical Guide No. 13



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Contents

Contents

1. Introduction

This document describes the design of a concrete bridge to the relevant European Standards, applied in accordance with their UK National Annexes.

The design example considered is a two-span integral bridge comprising pretensioned precast concrete beams with a reinforced concrete in-situ deck. However, in a departure from the approach taken in BS 5400, BS EN 1992-2:2005 generally treats prestressed and reinforced concrete in the same way. As a result, some of the guidance given in this document is equally valid in the design of a reinforced concrete or post-tensioned bridge structure.

The chapters that follow cover the design process in detail and also act as a commentary for the design calculations given in Appendix A. The reference numbers given in Appendix A correspond to the relevant chapter and section number of the commentary.

2. Scheme design

The design is for a two-span integral bridge, with each span having a length of 20.00 m. The bridge carries a 7.30 m wide carriageway with 2.00 m wide footways on either side. The superstructure consists of eight standard precast, pretensioned concrete Y-beams with a 160 mm deep in-situ reinforced concrete deck slab cast on ribbed permanent glassfibre reinforced concrete (GRC) formwork.

There are in-situ diaphragms at the abutments and pier. A relatively large gap between the precast beams is used at the pier. This makes detailing of the in-situ diaphragm easier and reduces the live load hogging moment for which the precast beams have to be designed. On the other hand, the precast beams need to be supported on temporary supports until the in-situ concrete diaphragm has cured. For this reason the in-situ section is kept short enough to enable the falsework to be supported directly off the pile cap.

The superstructure is made integral with the substructure. The foundations for the bridge consist of precast concrete piles with in-situ pile caps. The pile caps at the abutments are integral with the end diaphragms, while the pier wall is rigidly fixed to both its pile cap and the central diaphragm, avoiding the need for bearings altogether and simplifying the construction.

The integral abutments are small and the piles relatively flexible in order to avoid excessive reactions resulting from thermal expansion of the deck. However, there is still sufficient fill behind the abutment diaphragms to resist longitudinal acceleration and braking forces.

Each abutment has a reinforced concrete run-on slab, which spans the fill immediately behind the abutment. This is to prevent traffic compacting the material that is partially disturbed by the thermal movement of the bridge. Relative movement between the bridge structure and the highway pavement can be absorbed either by local deformation of the pavement or by a compressible joint at the end of the run-on slabs.

Figure 2.1 gives a diagrammatic representation of the bridge design.

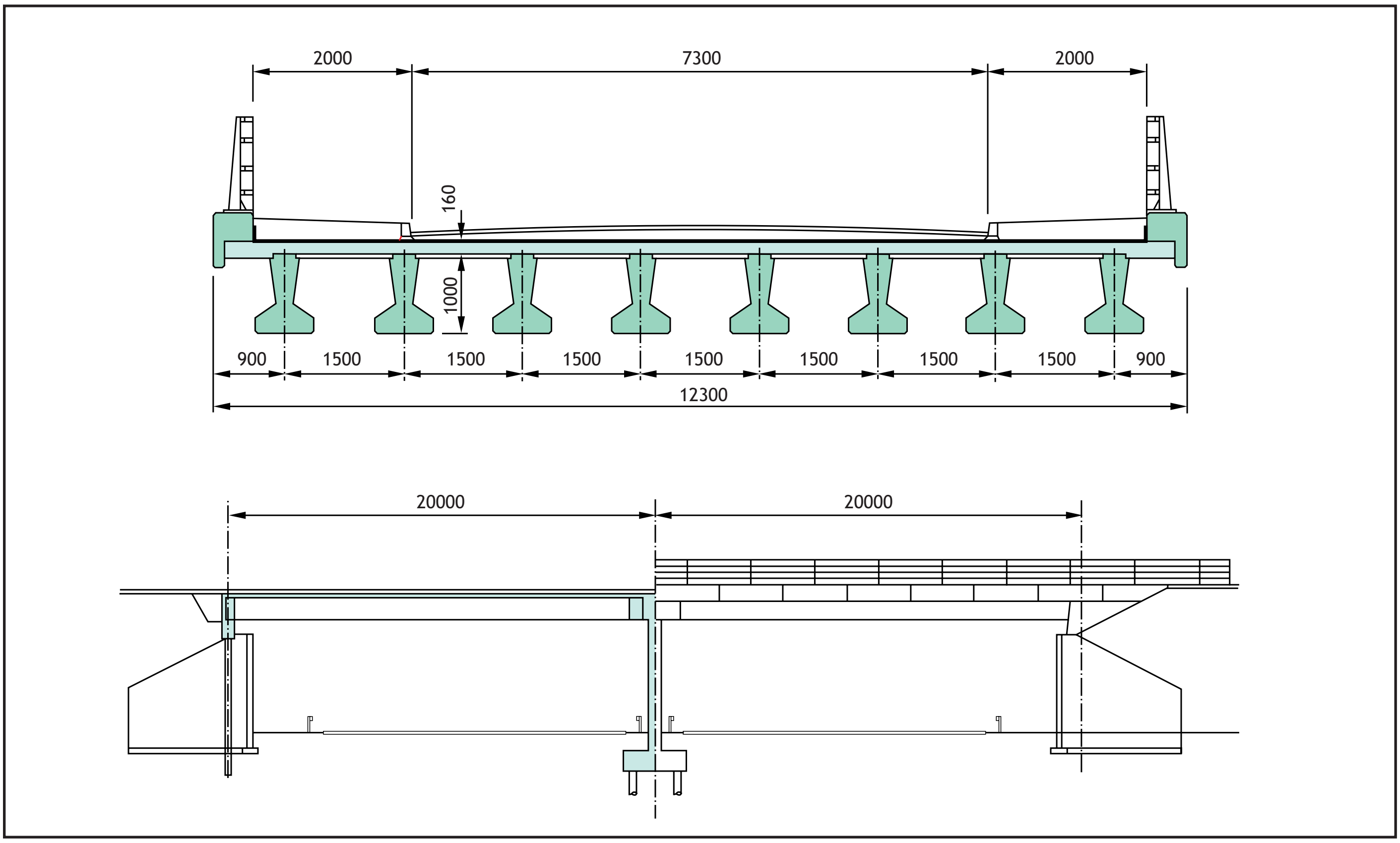


Figure 2.1
General arrangement of the bridge.

3. Schedule of design standards

The following list contains the relevant European and complementary British Standards that are used for the design of the bridge. The Eurocodes are to be used in accordance with their UK National Annexes. Where a normative part of the Eurocode allows for a choice to be made at the national level, the UK National Annex may overrule the recommended value, method or application rule. Throughout this document reference to a Eurocode implies reference to its respective UK National Annex as well.

Eurocode 0	**Basis of structural design**
BS EN 1990:2002 *Incorporating Amendment No. 1* National Annex to BS EN 1990:2002, BSI 2004, *Incorporating National Amendment No. 1*	Eurocode – Basis of structural design Includes Annex A2 – Application for bridges Includes Nationally Determined Parameters for Bridges
Eurocode 1	**Actions on structures**
BS EN 1991-1-1:2002 *Incorporating Corrigendum No. 1* National Annex to BS EN 1991-1-1:2002, BSI, 2005	Eurocode 1: Actions on structures – Part 1-1: General actions – Densities, self-weight, imposed loads for buildings
BS EN 1991-1-5:2004 *Incorporating Corrigendum No. 1* National Annex to BS EN 1991-1-5:2003, BSI, 2007	Eurocode 1: Actions on structures – Part 1-5: General actions – Thermal actions
BS EN 1991-1-7:2006 National Annex to BS EN 1991-1-7:2006, BSI, 2008	Eurocode 1: Actions on structures – Part 1-7: General actions – Accidental actions
BS EN 1991-2:2003 *Incorporating Corrigendum No. 1* National Annex to BS EN 1991-2:2003 *Incorporating Corrigendum No. 1*, BSI, 2008	Eurocode 1: Actions on structures – Part 2: Traffic loads on bridges
Eurocode 2	**Design of concrete structures**
BS EN 1992-1-1:2004 *Incorporating Corrigendum January 2008* National Annex to BS EN 1992-1-1:2004, BSI, 2005 *Incorporating National Amendment No. 1*	Eurocode 2: Design of concrete structures – Part 1-1: General rules and rules for buildings
BS EN 1992-2:2005 National Annex to BS EN 1992-2:2005, BSI, 2007	Eurocode 2: Design of concrete structures – Part 2: Concrete bridges – Design and detailing rules
Eurocode 7	
BS EN 1997-1:2004 *Incorporating Corrigendum February 2009* National Annex to BS EN 1997-1:2004, BSI, 2007	Eurocode 7: Geotechnical design – Part 1: General rules
BS EN 1997-2:2007 National Annex to BS EN 1997-2:2007, BSI 2009	Eurocode 7: Geotechnical design – Part 2: Ground investigation and testing

Other European standards	
BS EN 206-1:2000	Concrete – Part 1: Specification, performance, production and conformity
BS EN 13670: 2009	Execution of concrete structures
BS EN 10080: 2005	Steel for the reinforcement of concrete
BS EN 15050: 2007	Precast concrete bridge elements
Other reference standards	
BS 4449:2005	Steel for the reinforcement of concrete – Weldable reinforcing steel – Bar, coil and decoiled product – Specification
BS 5400	BS 5400-2 Steel, concrete and composite bridges – Specification for loads BS 5400-4, Steel, concrete and composite bridges - Code of practice for design of concrete bridges
BS 5896:1980 *Incorporating Amendment No. 1*	Specification for high tensile steel wire and strand for the prestressing of concrete
BS 8500-1:2006 *Incorporating Amendment No. 1*	Concrete – Complementary British Standard to BS EN 206-1 – Part 1: Method of specifying and guidance for the specifier
IAN 95/07	Revised guidance regarding the use of BS 8500 (2006) for the design and construction of structures using concrete
BA 36/90	The use of permanent formwork
PD 6694-1	Recommendations for the design of structures subject to traffic loading to BS EN 1997-1: 2004
PD 6688-1-7:2009	Recommendations for the design of structures to BS EN 1991-1-7
PD 6687-2:2008	Recommendations for the design of structures to BS EN 1992-2:2005
Reports	
CIRIA Report C660	Early age thermal crack control in concrete (2007)

Note:

The standards referenced in this publication and as used for the design examples are those relevant at the time of production. Those undertaking further designs should be vigilant for any subsequent queries with respect to, or amendments to, the Eurocode text and equations.

4. Structural model and analysis

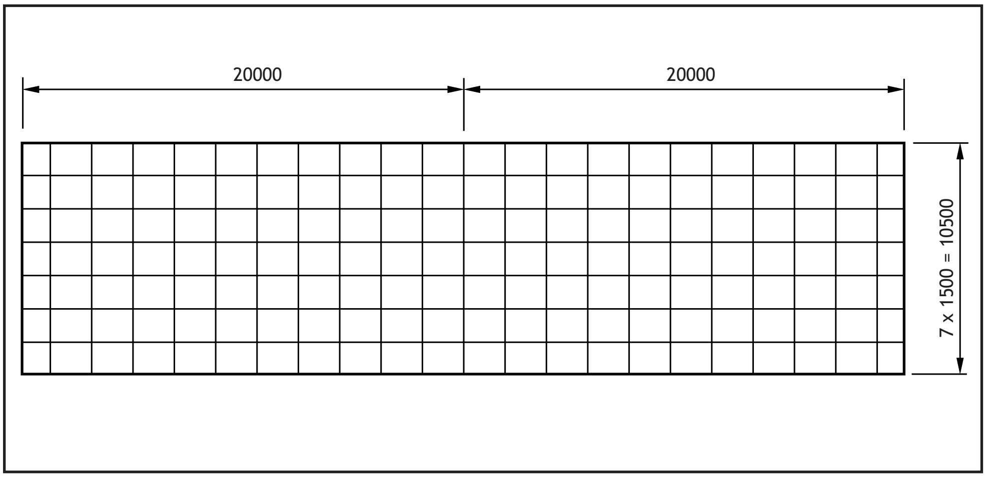
The global analysis of the bridge is carried out using a grillage model of the deck with the stiffness of the piers and abutments represented by rotational springs. The model is shown in Figure 4.1. It has eight longitudinal members at 1.50 m centres, each representing a precast beam and its associated width of slab, and transverse members at 1.85 m centres. Additional nodes are included either side of the central pier, at the location where the diaphragm ends, so that stress resultants for these critical locations can easily be obtained.

The rotational stiffness of the springs at the pier is calculated from the stiffness of the pier wall assuming fixity at pile cap level. The stiffness of the springs at the abutments is calculated assuming fixity at a pile depth of 5 m. The pile size, and hence stiffness, will have to be assumed at this stage but the relative flexibility of the piles compared to the bridge deck means that the results are not too dependent on the assumptions made.

Alternatively, the superstructure and substructure could be modelled together in a single 3D model. However, the practicalities of the design process mean that they are often modelled separately.

The bridge will be built by placing the beams on temporary supports and then installing formwork for the in-situ concrete. Permanent formwork, supported by the beams, will be used for the deck slab concrete while that for the diaphragm over the pier will be cast using temporary formwork supported off the ground. This means that all the dead load in the main part of the bridge (except for the string course which is assumed to be cast separately) is applied before the in-situ concrete has cured. Since the grillage model represents the complete bridge, with a structurally effective slab, it is more correct to analyse for this part of the dead weight using a simple line beam analysis. Note that the stresses from this weight (including the slab) are taken by the beam section alone, rather than the composite section. However, this is only significant for the serviceability limit state (SLS) calculations. The strain discontinuity between precast and in-situ concrete, which results from the former being loaded before the latter has cured, is not sufficient to be worth considering in ultimate limit state (ULS) calculations.

Figure 4.1
Grillage model of bridge deck.



The weight of the in-situ concrete over the pier is supported by temporary works until after it is structurally effective. It is therefore correct to apply this to the computer model. The temporary works also support the precast beams, taking the weight of these and all the load they support until they are released. The approach taken to analyse the structure for dead load was therefore as follows:

- Weight of beams and slab applied to simply supported line beam model supported at prop positions. The section properties used in the model are those of the Y4 beam only.
- Weight of in-situ concrete over pier applied to the grillage model of the completed structure, which uses composite section properties.
- The effect of removing the props supporting the diaphragm formwork is modelled by applying the reaction from the line beam analysis described in item 1 above as a load on the grillage model.

The prestress is also applied to the simply supported precast beam. Strictly speaking, creep of the concrete means that over time a certain proportion of both the prestressing force and the dead load carried by the beam alone will be taken over by the composite section. Modelling this redistribution of forces over time requires a rather involved, step-by-step analysis, the results of which are of questionable accuracy. However, as the effect is relatively minor, it is often neglected for relatively short structures such as this. This is the approach that has been taken in the body of this design example. The reader can refer to BS EN 1992-2 Annex KK for different methods of calculating the long-term creep redistribution effects.

The flexural stiffness of the concrete members used in the grillage analysis was generally calculated assuming uncracked cross-sections. However, BS EN 15050 Annex D recommends that cracked section properties be used for the reinforced concrete diaphragm over the pier. This contradicts BS EN 1992-1-1, clause 5.4(2) which states that for the determination of action effects, uncracked section properties should be used, and clause 5.4(3) which states that for thermal, settlement and shrinkage effects cracked section properties can be used at ULS, but that at SLS a gradual evolution of cracking, including tension stiffening, should be used.

Although the normative clauses of BS EN 1992-1-1 take precedence over the Informative Annex of BS EN 15050, it is expected that for relatively short-span structures such as those being considered here, many designers will follow the simplified guidance given in BS EN 15050. Therefore, this is the approach that has been taken in this example.

In calculating the cracked section properties it was assumed that the longitudinal reinforcement in the in-situ slab comprised 20 mm diameter bars at 150 mm centres top and bottom. Although the reinforcement finally chosen for the slab differed from this slightly, the effect will be negligible.

The torsion constant was set to zero in the model, meaning that the beams were assumed to carry no torsional moment. This is conservative for structures where torsion plays only a minor part in the behaviour of a bridge, and simplifies the calculations as torsional effects do not have to be considered. This is particularly beneficial for beam-and-slab bridges such as that considered here, as the rules for torsion can be hard to apply to thin webs(1).

The section properties used are given in Table 4.1.

Table 4.1
Section properties.

	Section height, h (mm)	Centroid height, y (mm)	Area, A_c (mm^2)	2nd moment of area, I (mm^4)
Y4 beam	1000	400	410 191	3.82×10^{10}
Y4 beam with 1.5 m wide slab[a]	1160	638	630 732	1.05×10^{11}
Diaphragm at pier[b]	1160	–	–	3.26×10^{10}
In-situ slab (per 1.85 m width)	160	80	296 000	6.31×10^{8}

a. Composite section is transformed into an equivalent class C50/60 section
b. Diaphragm properties are calculated for a cracked section, as per BS EN 15050

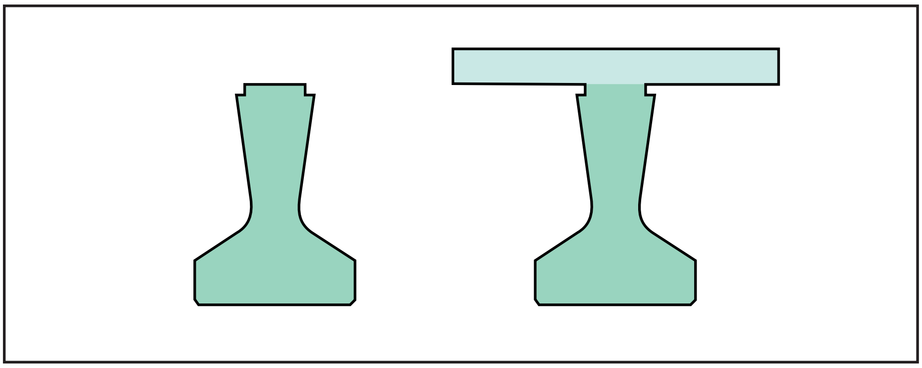


Figure 4.2
Y4 beam alone (left) and composite section comprising Y4 beam and in-situ slab.

The global grillage model is not sufficiently refined to provide the local component of transverse moments due to wheel loads on the region of slab between the longitudinal beams. These can be calculated separately either by using influence charts(2) or a small-scale finite-element model with plate bending elements. The latter approach will be used here. The calculated global and local components of the transverse moments in the deck slab can then be superposed to give the total design values.

5. Materials

5.1 Concrete

Assumed properties of concrete are given in BS EN 1992-1-1, clause 3.1. BS EN 1992 uses cylinder strengths throughout but Table 3.1 gives the corresponding cube strengths, allowing the continued use of cubes as control specimens.

The concrete strength class used in this design is C50/60 for the precast beams and C35/45 for the in-situ deck slab, diaphragms, pier wall, pile cap and precast piles.

5.2 Prestressing steel

The proposed European standard for prestressing steel is EN 10138 and it is often referred to in BS EN 1992-1-1. EN 10138, however, has since been voted down, though it is likely that it will be rewritten and published at a future date. In the meantime, BS 5896: 1980 has been amended to cover those products currently on the market for which no specification would otherwise exist.

5.3 Reinforcement

The European standard for reinforcing steel for concrete is BS EN 10080. However, BS EN 10080 does not define steel grades and rather inconveniently leaves it to the designer to specify its properties. In the UK this void is filled by BS 4449:2005 which specifies the required properties for standardised grades.

5.4 Cover

The exposure classes are specified in section 4 of BS EN 1992-1-1 and BS EN 1992-2. The example bridge is assumed to be passing over a carriageway, and so this is classified as XD3 (exposed to spray containing chlorides). The bridge soffit is more than 5 m above the carriageway and so according to the National Annex to BS EN 1992-2 does not have to be classified as XD3, though it does not explicitly specify what it should be classified as. XD1 (exposed to airborne chlorides) would appear most suitable. If instead it passed over a railway line, fresh water or other non-highway obstruction, the pier wall and underside of the bridge could be classified XC3 (exposed to carbonation but not chlorides).

The top of the deck is protected by waterproofing and so BS EN 1992-2 allows this to be classified as XC3.

The minimum cover for durability is a factor of exposure class and concrete class and type. The National Annex to BS EN 1992-1-1 specifies that minimum cover requirements should be taken from BS 8500-1, rather than Tables 4.3N, 4.4N and 4.5N of BS EN 1992-1-1.

The nominal cover which is specified for the design is this minimum cover plus an allowance for deviation (Δc_{dev}). The National Annex to BS EN 1992-1-1 states that this should be taken as 10 mm in most cases. Where production is subjected to a quality assurance system where an accurate monitoring system is in place and non-conforming elements are rejected, BS EN 1992-1-1 allows this to be reduced to 10 mm $\geq \Delta c_{dev} \geq$ 0 mm. This may be applied to precast elements when such a system is in place, which will normally be the case.

Additional guidance regarding the allowance for deviation is given in the Highways Agency's Interim Advice Note 95/07 *Revised Guidance regarding the use of BS 8500(2006) for the design and construction of structures using concrete*. The advice note recommends that Δc_{dev} generally be taken as 15 mm, but that this may be reduced to 10 mm for the deck slab (a lower fixing tolerance being necessary and appropriate for thinner sections) and 5 mm for factory-controlled precast concrete, where the accurate fixing is easier to achieve and monitor. These values have been used in Appendix A.

6. Actions (loading)

6.1 Permanent actions

The Eurocodes refer to 'actions' rather than loads. Permanent actions include self-weight of the structure, superimposed dead load including surfacing and any soil weight, hydrostatic effects, creep and shrinkage, settlement and prestressing.

6.1.1 Self-weight

The nominal weights of various materials are given in BS EN 1991-1-1 Annex A.

As the thickness of surfacing has a high probability of variation, particularly as a result of future resurfacing, the nominal thickness of the surfacing is increased by a factor in accordance with clause 5.2.3(3) of the National Annex.

In Appendix A, sheet A6.1, 'Self-weight 1a' represents the dead load of the beam and slab, which is carried by the precast beam alone, acting as a simply supported beam between temporary props during construction. The resulting peak moments and shears are readily calculated by hand, based on a span length between the props of 19 m, and a total beam length of 19.5 m.

'Self-weight 1b' represents the reaction from the line beam model that is then applied to the grillage model.

6.1.2 Differential settlement

A maximum total settlement of 20 mm has been assumed. The resulting differential displacement of the supports is considered in the most unfavourable arrangements. This settlement is that which takes place after the bridge deck has been made continuous. Settlement taking place before the deck is made continuous need not be considered as the simply supported beams are statically determinate structures. It is assumed that half of the 'final' ±20 mm differential displacement will occur before the bridge is first opened for traffic. On the other hand the long-term load effects are reduced by half, taking into account the creep of the concrete. Differential settlement normally only needs to be considered in the serviceability limit states.

6.1.3 Differential shrinkage

Because the deck slab is cast once the beams have already undergone some shrinkage, it shrinks more than the beams once they have been made composite. This causes tension within the deck slab, compression within the beams, and an overall sagging within the deck.

Calculation of the shrinkage strains is covered later, in section 7. Once these are known, the resulting moments and axial forces can be calculated. The moments should be included in the grillage model, as they alter the load distribution within the structure, while the axial forces will need to be taken into account when checking stress limits.

Differential shrinkage need only be considered in the serviceability limit states.

Shrinkage will also cause a shortening of the bridge as a whole. The resulting axial restraint force due to the fixed supports could be calculated using a plane frame model with appropriate support stiffnesses, and should be for long-span bridges or those with stiff integral abutments. However, the bridge being considered here is relatively short, which limits the total shrinkage, and has deliberately been designed with small abutments and relatively flexible piles so as to limit the axial restraint, as discussed in section 2 above. Therefore, the axial restraint force has been neglected.

6.2 Variable actions

Variable actions include wind, thermal and construction loads as well as traffic loads. Wind loading on relatively heavy short-span bridges is not a critical loadcase, and so does not need to be considered. For brevity, loads arising during the construction process will not be considered in this example.

6.2.1 Temperature effects

Daily and seasonal fluctuations in shade air temperature, solar radiation, etc. cause changes in the temperature of a bridge superstructure, which in turn cause movement of that structure. Depending on the restraint conditions of the bridge, this movement can lead to stresses within the structure. This effect can be divided into three components: the uniform temperature component which causes expansion or contraction of the deck, the temperature difference component which leads to curvature of the bridge and the non-linear temperature component, which causes local stresses within the structure.

The first stage in determining the uniform temperature component is to determine the minimum and maximum shade air temperatures for the location where the bridge is to be built. For the UK these can be determined from the maps of isotherms given in Figures NA.1 and NA.2 of the National Annex to BS EN 1991-1-5, with an adjustment being made for the effect of altitude.

The minimum and maximum shade temperatures are then converted into minimum and maximum uniform bridge temperatures, $T_{e,min}$ and $T_{e,max}$. There are two main factors that will affect these temperatures, namely the type of bridge construction and the thickness of the deck surfacing. BS EN 1991-1-5, Figure 6.1 allows the conversion from shade temperature to uniform bridge temperature for three basic types of bridge construction: steel, concrete deck with steel beams, and concrete. Adjustment for varying surfacing can then be made using the values given in Table NA.1 of the National Annex.

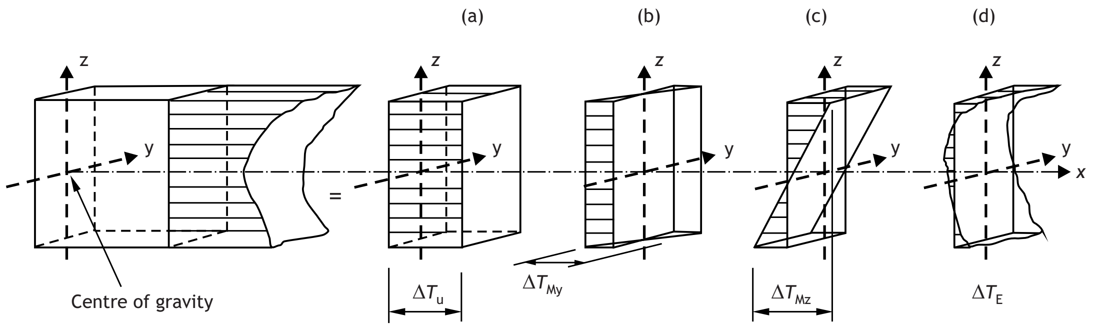
The maximum contraction that the bridge will experience will depend on the difference between the minimum uniform bridge temperature and the uniform bridge temperature at the time when the bridge is first made continuous with its abutments, T_0. Similarly, the maximum expansion depends on the difference between the maximum uniform bridge temperature and that at the time the bridge is made continuous. As the temperature at the time of construction cannot be known in advance, the NA to BS EN 1991-1-5, suggests that T_0 be taken as 20°C when considering contraction and 0°C for expansion. Alternatively, maximum and minimum temperatures under which the bridge can be made continuous can be specified. However, it must be remembered that T_0 is the uniform bridge temperature and so will depend on the average air temperature over a period of time (the length of which will depend on the thermal mass of the structure), rather than the instantaneous air temperature.

The maximum calculated displacement for this bridge is 14 mm. The force required to generate relatively small movements of this magnitude is negligible, especially when it is remembered that these extreme values occur slowly as a result of the seasonal change rather than the daily cycle between night and day. Furthermore, the abutments have deliberately been kept small to reduce the required force. Therefore it is usual to ignore these movements when analysing the deck, though they must be included in the design of the foundations.

As well as uniform temperature changes, which cause uniform changes in length, variations in temperature through the thickness of the deck must be considered. Both positive (heating) and negative (cooling) temperature differences are considered. The values for temperature difference distributions given in Figure 6.2c of BS EN 1992-1-1 assume a depth of surfacing of 100 mm. Values of temperature distribution for other thicknesses of surfacing are given in Annex B.

Each of these temperature profiles can be broken down into three parts as shown in Figure 6.1: a uniform temperature through the whole section, a temperature profile that varies linearly through the depth of the section and a non-linear temperature profile. The temperature profiles given in BS EN 1991-1-5 are plotted on arbitrary datums and so the uniform temperature components of the profiles need not be considered; instead that calculated earlier is used. The linearly varying temperature profile will cause the bridge deck to try to curve. In a simply supported structure this curvature can occur freely and so will cause no stress within the structure. However, in a statically indeterminate structure such as in this design example the restraints at the abutments and the pier will resist this curvature, leading to internal forces being set up within the structure. Therefore the linearly varying temperature component has to be included in the global model. The non-linear temperature component is in equilibrium with itself, in that globally it does not induce either a change in length or curvature of the structure. Therefore it does not need to be included in the global analysis model. However, the non-linear temperature component does induce local stresses within the structure which should be included in the SLS checks.

In order to calculate these three components the deck is first assumed to be fully restrained at the ends. The stresses in the deck are equal to the temperature change multiplied by the coefficient of thermal expansion, α, and the elastic modulus of the concrete. Summation of these stresses over the whole cross-section gives an axial force, which combined with its eccentricity produces a moment. These forces can then easily be converted into a uniform stress and linearly varying stress across the *transformed* section (so as to allow for the varying material properties). Normally it will then be beneficial to convert the linearly varying stress into a linearly varying temperature for input into the grillage model.



(a) A uniform temperature component, ΔT_u

(b) A linearly varying temperature difference component about the z–z axis, ΔT_{MY}

(c) A linearly varying temperature difference component about the y–y axis, ΔT_{MZ}

(d) A non-linear temperature difference component, ΔT_E. This results in a system of self-equilibrated stresses which produce no net load effect on the element

Figure 6.1
Diagrammatic representation of constituent components of a temperature profile.

The residual stresses are the stresses resulting from the temperature change remaining after the uniform and linearly varying stresses are subtracted from the original temperature distribution.

6.3 Traffic loads

6.3.1 Notional lanes

Before traffic loading can be applied to the model of the structure, the carriageway must be divided into notional lanes as specified in BS EN 1991-2, Table 4.1. For widths of 6.0 m and above, the carriageway is divided into an integer number of 3.0 m wide lanes. Any excess width is known as the 'remaining area'.

For the analysis of the bridge, the positioning of the notional lanes does not have to correspond to the position of the actual lane markings on the bridge. Instead, the lanes and the remaining area are positioned so as to create the most severe load effects for each element being considered. Similarly, the numbering of the notional lanes is not related to their position. Instead, the lane producing the most unfavourable effect on the element being considered is 'lane 1', that producing the second most unfavourable effect is 'lane 2', and so on.

6.3.2 Load Model 1

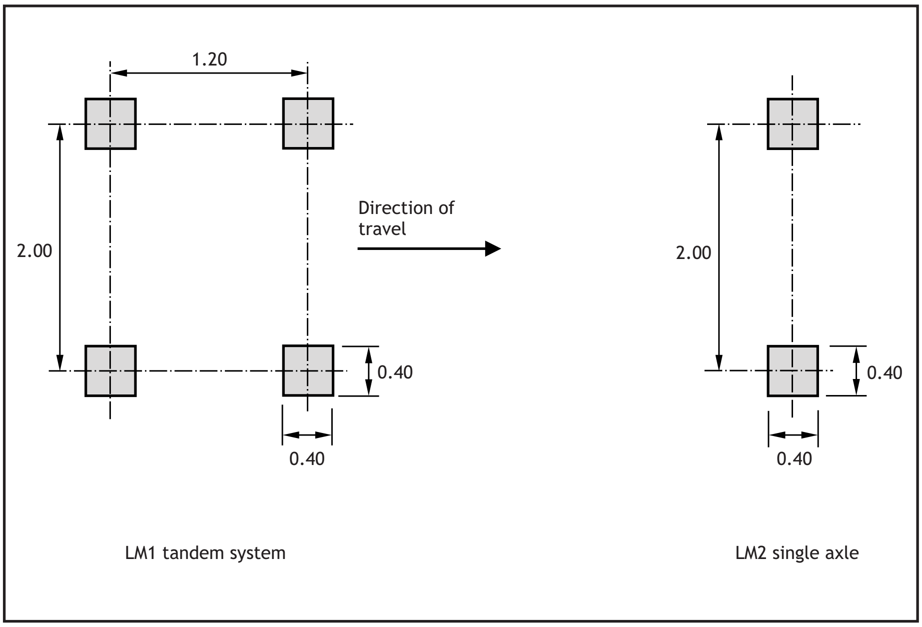
Normal traffic in BS EN 1991-2 is represented by 'Load Model 1' (LM1), which is the equivalent of HA loading in BD 5400-2. For each lane, LM1 consists of two parts:

- A double-axle loading, referred to as the tandem system, or TS. Each axle has a weight of $\alpha_Q Q_k$, where α_Q is a nationally determined adjustment factor. For spans greater than 10 m, the two axles can be combined into one according to BS EN 1992-2 clause 4.3.2(6)b. Although this is always conservative, the National Annex states that this should not be done and it has not been done here.
- A uniformly distributed load (UDL) having a weight per square metre of $\alpha_q q_k$, where α_q is a nationally determined adjustment factor.

Only one tandem system is applied to each lane, symmetrically around the centreline of the lane[†] and in the position that causes the most severe effect on the element being considered. The tandem systems and UDLs should only be applied in the unfavourable parts of the influence surface, both longitudinally and transversely. The nationally determined adjustment factors for the UDL have been set so that a UDL of 5.5 kN/m^2 is applied to all lanes and the remaining area, irrespective of the number of nominal lanes, simplifying the input of loading into the analysis model. In contrast to BS 5400-2, the magnitude of this load pressure does not vary with loaded length.

[†] For global analysis only. For local effects the tandem system can be positioned anywhere within the lane, but no closer than 0.50 m to the TS in the adjacent lane.

Figure 6.2
LM1 tandem system and LM2 single-axle load.



6.3.3 Load Model 2

Load Model 2 (LM2) is a single-axle load with a weight of 400 kN, and is not combined with other traffic models. LM2 is only significant for shorter members (predominantly those less than 7 m) where it can produce more severe load effects than LM1. It therefore needs to be considered for local checks of the in-situ slab, but not global design of the structure in this case. In BS EN 1991-2 wheel contact shapes are different for LM1 and LM2, which could result in one wheel shape being critical for the reinforcement in one direction but the other wheel shape critical for the reinforcement in the other. However, the UK National Annex changes them to both be 0.4 m square. The contact area is larger than in BS 5400-2 and the distribution through surfacing as well as concrete slab is at 45°. This can make them less severe relative to BS 5400-2 than might be anticipated from their actual values.

6.3.4 Load Model 3

Load Model 3 represents abnormal vehicles. Section 3 of the National Annex to BS EN 1991-2 defines a series of load models to be used in the design of UK road bridges to represent the effect of vehicles that are in accordance with Special Types (General Order) (STGO) Regulations, i.e. generally vehicles with gross weights of between 44 and 150 t[†]. Three models are defined, namely SV80, SV100 and SV196, representing STGO vehicles with maximum gross weights of 80 t, 100 t and 150 t (196 t gross train weight) respectively. The SV vehicle can be placed anywhere on the carriageway, either wholly within one notional lane or straddling two adjacent lanes. The SV loading should be combined with a reduced value of Load Model 1, known as the ‘frequent’ value. It should be noted that LM1 loading in the same lane(s) as the SV vehicle should include the Tandem System axles; this is in marked contrast to the combined HB and HA loading of BS 5400-2, where the HA Knife Edge Load is omitted from the lane(s) in which the HB vehicle is positioned. The lanes should be defined such that the most severe load effect is produced – that is, the lane in which the SV vehicle is placed does not have to be lane 1.

[†] Additional load models to represent Special Order Vehicles with a gross weight in excess of 150 t have also been developed and can be found in the National Annex to BS EN 1992-1. These may be required to be taken into account on certain routes.

6.3.5 Crowd and footpath loading

Load Model 4 represents crowd loading and so is especially applicable to bridges in urban areas. It need only be considered when expressly demanded. The loading consists of a UDL of 5 kN/m^2 and is applied to the central reserve as well as the carriageway.

While BS EN 1991-2 suggests a uniformly distributed load of 5.0 kN/m^2 for road bridge footways and a variable UDL dependent on loaded length for foot and cycle bridges, the National Annex specifies the variable UDL in both cases. The footpath loading should only be applied where it causes unfavourable effects – it does not always need to be applied to both spans, or even both footpaths. When combined with LM1 the intensity is reduced to 3.0 kN/m^2.

6.3.6 Other traffic loading

Accidental loading can be ignored in the design of this deck as the deck is uniform, and the accidental loading on the footways is less severe than the traffic loading on the carriageway. Accidental loading will have to be considered in the design of the parapet string courses and the pier. The horizontal forces applied to the structure due to acceleration and braking are small enough to be ignored in the design of the bridge deck. They will have to be included in the design of the abutment foundations.

6.3.7 Groups of traffic loads

The ways in which these traffic load models can be combined are specified in BS EN 1991-2, clause 4.5, and Table NA.3 of the National Annex to BS EN 1991-2 and are summarised in Appendix A, sheet A6.5 for the load models being considered. The characteristic values of the loads are those values that have been calculated in the preceding sections of Appendix A. The frequent values are found by multiplying the characteristic values by the frequent load factor, ψ_1. The significance of these values will be explained in section 6.4 below. *These load groups are mutually exclusive.*

6.4 Combinations of actions

There are three combinations of actions that must be considered at the SLS:

- the characteristic combination, which can be considered the most severe loading to which the structure should be subjected
- the frequent combination, which is the most severe load case to which the structure should be subjected on a regular basis
- the quasi-permanent loadcase, or the loading to which the structure is subjected most of the time.

These three combinations are shown in Appendix A, sheet A6.6. E_d represents the design values of the effects of the actions. G_k represents the characteristic values of the permanent actions, which include shrinkage, creep and settlement as well as dead weight. P represents the representative (not characteristic) values of the prestressing actions while Q_k represents the characteristic values of the variable actions. Depending on the combination being considered, the characteristic values of the variable actions may be used directly or multiplied by one of three factors; ψ_0, the combination factor, ψ_1, the frequent factor; or ψ_2, the quasi-permanent factor.

The characteristic combination is generally associated with irreversible SLS criteria. This takes the full characteristic value of one variable action (the *leading variable action*) and combines it with the combination value of the remaining variable actions. For most short- to medium-span bridges, the critical characteristic combination will have a traffic load group as the leading action.

The frequent combination is used for evaluating undesirable but reversible criteria. As such, it takes a reduced value for the variable actions, accepting that this loading will occasionally be exceeded.

The quasi-permanent load combination is used to evaluate long-term effects and those that are concerned with aesthetics and durability. For bridges, this load group does not include traffic loads but does include thermal effects (at a reduced value) as the duration of these tends to be significant.

Four ultimate limit states are defined in BS EN 1990, namely EQU, STR, GEO and FAT. These are defined as follows:

EQU	Loss of static equilibrium of the structure or any part of it when considered as a rigid body.
STR	Internal failure or excessive deformation of the structure or structural member.
GEO	Failure or excessive deformation of the ground where the strengths of soil are significant.
FAT	Fatigue failure of the structure or structural members.

In the design of the bridge deck we are mainly concerned with the STR limit state (fatigue of the reinforcement and tendons will have to be covered, but the FAT limit state is defined in the individual material codes (BS EN 1992 to BS EN 1999) rather than BS EN 1990, and so will be covered later). BS EN 1990 gives two approaches for considering the STR limit state. However, the UK National Annex to the bridges annex of BS EN 1990 specifies that only one of these (Equation 6.10 rather than 6.10a and 6.10b) should be considered (Table NA.A.2.4(B)).

Table NA.A.2.4(B) also gives the partial factors to be used for bridges with the STR limit state. There are two partial factors for each load type: γ_{sup} when the effect is unfavourable and γ_{inf} when favourable. The characteristic values of all permanent actions from one source should be multiplied by a single partial factor, either γ_{sup} or γ_{inf} – alternate spans with different partial factors do not need to be considered. In contrast, variable actions are only applied where unfavourable.

It is usual at ULS to neglect the effects of temperature, settlement, creep and shrinkage – this is permitted in BS EN 1992-1-1, clause 2.3 provided the deck over the support has sufficient ductility and rotational capacity. This is easily satisfied for most bridge decks but may not be for heavily over-reinforced sections, which must be checked.

7. Creep and shrinkage

Creep and shrinkage cause a number of effects that have to be considered at various stages in the design of prestressed concrete structures. Perhaps the most significant is the loss of prestress that results from creep and shrinkage and the effect that this will have on the SLS criteria.

Differential shrinkage between the precast beams and the in-situ slab will cause both axial forces (for both statically determinate and indeterminate structures) and moments (for statically indeterminate structures only) to be set up within the structure. The magnitude of these loads has been calculated in Appendix A, sheets A6.1-A6.6, using the shrinkage parameters calculated on sheet A7.1.

Creep will also lead to a redistribution of forces within the structure over time – at the time of construction all of the prestressing force and the dead load of the beam and in-situ slab is carried by the precast beams alone. With time, creep will result in some of these loads being carried by the composite beam-and-slab section, changing the stress distributions within the sections. However, the calculation of this effect is complicated and the results are of questionable accuracy so it is common to neglect this effect for simple structures, as has been done with this design.

BS EN 1992-1-1, clause 3.1.4 and Annex B describe the creep and shrinkage prediction models, with clause 3.1.4 giving the final values and Annex B allowing the development of the two effects over time to be calculated. The model for creep calculates a creep coefficient, $\varphi(t, t_0)$, from which the creep can be calculated using the relationship:

$$\varepsilon_{cc}(t, t_0) = \varphi(t, t_0)\, \sigma_c / E_c$$

where

$\varepsilon_{cc}(t, t_0)$ = creep strain at time t

σ_c = compressive stress

E_c = tangent modulus

The creep coefficient is itself the product of two coefficients, φ_0 and $\beta_c(t, t_0)$. The *notional creep coefficient*, φ_0, is the long-term creep coefficient of the section, and allows for the physical and chemical properties of the section, such as section geometry, concrete strength and the maturity of the concrete when first loaded. $\beta_c(t, t_0)$ describes the development of creep with time after loading.

In the calculation of the shrinkage of a section, a notable difference from previous practice is that the total shrinkage strain is decomposed into a drying shrinkage component and an autogenous shrinkage component. Previous models recognised external drying as the main driving mechanism behind shrinkage. With increasing concrete strength, internal chemical drying (i.e. loss of moisture due to the hydration of the hardening concrete) plays an increasingly significant role. Therefore autogenous shrinkage is particularly important for high-strength concrete and high-performance concrete. Since autogenous shrinkage is independent of the size of the concrete member and the relative humidity of the ambient environment, the shrinkage in bulk concrete members exposed to humid environments is dominated by autogenous shrinkage.

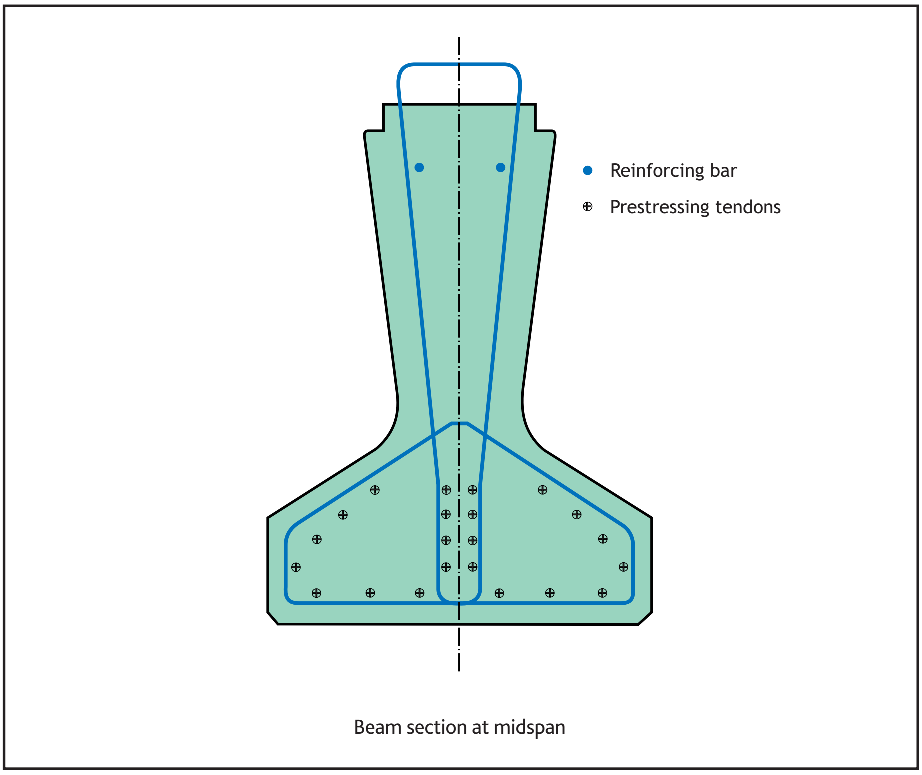
The expected error of the prediction is an important but often overlooked attribute of creep and shrinkage models. The designer should appreciate that creep and shrinkage are among the most uncertain mechanical properties of concrete. Prediction models only reflect the mean tendencies observed in highly scattered experimental data. The coefficient of variation for creep and shrinkage models is around 20–30%. While for smaller bridges it is usually adequate to consider the expected mean values of creep and shrinkage, it may be important for deformation-sensitive structures to take into consideration the consequences of a potential deviation from the expected mean.

For high-strength concrete (that with a strength class greater than C50/60), the creep and shrinkage models presented in BS EN 1992-1-1 are still applicable as they are the updated versions of the widely accepted and well-established models of the CEB-FIP Model Code 1990. They have been updated in order to take into account the particular characteristics of high-strength concrete and high-performance concrete and are valid for concrete strength up to 120 MPa. It should be noted that the National Annex to BS EN 1992-2 forbids the use of the modifications to Annex B presented in BS EN 1992-2 but allows the use of the Annex B presented in BS EN 1992-1-1.

8. Prestressing

The prestressing tendon layout, and the resulting calculation of prestressing losses and secondary moments in the global model, were initially based on an assumed layout based on previous experience. After the initial run of the global analysis, the layout was modified to rectify any deficits in the results, and the calculations revised. Several iterations of this process may be required before the final tendon layout is arrived at, but this process is made relatively painless through the use of spreadsheets and suitable analytical programs. The calculations shown in Appendix A, sheets A8.1 and A8.2 reflect the final tendon layout, shown in Figure 8.1 below. The tendon positions were selected from the standard positions available for Y4 beams. The two top reinforcing bars provided at the ends of the beam provide crack control at transfer (see section 9.6). Also shown is the shear reinforcement.

Figure 8.1
Beam section showing strand positions.



8.1 Stress limits and loss of prestress

Requirements for prestressing and the calculation of the effective prestressing force are given in BS EN 1992-1-1, clause 5.10. The applied prestress in the strands during tensioning should not exceed 80% of the characteristic tensile strength or 90% of the characteristic 0.1% proof stress, whichever is the lesser. This gives a limit of 1422 MPa in this case.

The stress in the strands immediately after transfer is also limited – this time to the lesser of 75% of the characteristic tensile strength or 85% of the characteristic 0.1% proof stress. Losses of prestress should be taken into account. For pretensioned concrete elements these losses are due to the elastic shortening of the concrete element, relaxation of the prestressing steel and shrinkage of the concrete which takes place before transfer. As the loss of prestress due to elastic shortening is not equal for all the tendons, depending instead on where in the section they are positioned within the section, strictly speaking the stress after transfer should be calculated for the tendon closest to the centroid. However, for reasonably closely grouped tendons it is standard practice to calculate the average stress, as has been done in Appendix A, sheets A8.1 and A8.2.

In the calculations carried out, the convention of calculating elastic shortening at transfer but not allowing for subsequent increases in tendon forces due to applied load is followed. BS EN 1992-1-1 does not require this and some computer approaches would include the increase in tendon force. Allowing for this effect would recover around 28 MPa of prestress loss and increase the concrete compressive stress at decompression by 0.5 MPa. The conservative approximation of ignoring this gives some allowance for the failure to consider creep redistribution of the prestress between the beam and deck slab.

After transfer, further losses will occur due to continuing relaxation of the tendons and shortening of the concrete element due to creep and shrinkage.

BS EN 1992-1-1, clause 3.1.4(4) states that if the compressive stress at the time of loading exceeds 0.45 times the characteristic strength at the time of loading, non-linear creep has to be considered. In the design considered in Appendix A, the stress at the level of the centroid of the tendons at midspan of the beams (loss of prestress at the ends of the beams not being a concern) is 14.7 MPa. This is very slightly higher than $0.45f_{ck}(t_0)$ = 14.4 MPa. However, the resulting increase in creep would be small, and would only occur for a very short time as the concrete is rapidly increasing in strength at the age of transfer (less than one day), and so the effect of non-linear creep may justifiably be neglected in this instance.

Partial factors are not applied to the prestressing force for the SLS checks. However, BS EN 1992-1-1, clause 5.10.9 requires use of the characteristic value of the prestressing force for SLS checks. Two multiplying factors are defined in order to obtain the lower and upper characteristic values. The recommended values are overruled in the UK National Annex and both values should be taken as 1.0. Therefore, the mean value of the calculated effective prestressing force is used for the SLS checks and clause 5.10.9 has no practical relevance in the UK.

8.2 Transmission length

The transmission length of a tendon is the length over which the prestressing force is fully transmitted to the concrete. It is assumed that the transfer of stress from the tendon to the concrete is via a constant bond stress, f_{bpt}, such that there is a linear transfer of prestress from the tendon to the concrete beam. This bond stress is calculated allowing for the type of tendon (indented or multi-wire strands), tensile strength of the concrete at the time of release and the bond conditions – this last allows for the poor bond that can occur with top-cast bars in deep sections.

For tendons with a circular cross-section that are released gradually at transfer, the transmission length, l_{pt}, is simply the tendon's initial prestressing force, F_{pm0}, divided by the product of the bond stress and the circumference of the tendon, i.e.:

$$\text{Transmission length, } l_{pt} = \frac{F_{mp0}}{\pi\phi f_{bpt}} = \frac{\left(\pi \frac{\phi^2}{4}\right)\sigma_{pm0}}{\pi\phi f_{bpt}} = 0.25\phi\,\sigma_{pm0}/f_{bpt}$$

where
σ_{pm0} is the tendon stress at transfer.

If the tendons are released rapidly (not done in bridge beam construction) then the sudden transfer of stress across the tendon–concrete interface can damage the bond and so increase the transmission length. In these situations Eurocode 2 increases the predicted transmission length by 25% (coefficient α_1). Conversely, when the tendon is a multi-wire strand, as is normal for bridges, the surface area of the tendon is increased, allowing the prestressing force to be transmitted in a shorter length. This is recognised by replacing the factor of 0.25 in the above equation with 0.19 – coefficient α_2.

For design purposes the transmission length is taken as being either 20% higher or 20% lower than the calculated value, whichever is more onerous in the design situation being considered.

9. Global design at SLS

9.1 Introduction

The design of most prestressed concrete structures will be limited by serviceability criteria. There are three checks that must be carried out: decompression, crack widths and stress limits.

For most situations[†], the decompression limit is checked for the frequent load combination, and requires that all concrete within a certain distance of the tendons remains in compression. Table NA.1 of the National Annex to BS EN 1992-2 specifies that this distance is equal to the minimum cover required for durability. Parts of the prestressed beam outside this limit may go into tension, but should be checked against a crack width limit of 0.2 mm.

Finally, stress limits, both in the concrete and tendons, must be checked under the characteristic load combination.

Unlike the approach taken by BS 5400 for prestressed concrete, where uncracked section properties are used, even when the tensile stresses exceed the tensile strength of the concrete, these checks are carried out using cracked section analysis where the flexural tensile stress limit of $f_{ct,eff}$ is exceeded, in accordance with clause 7.1(2).

The checks required for reinforced concrete sections (e.g. the diaphragms) differ slightly. Obviously, there is no decompression limit. Instead, a maximum crack width of 0.3 mm must be complied with. This is checked under the quasi-permanent combination of loads. As for prestressed concrete sections, stresses in both the concrete and the reinforcement must be checked under the characteristic load combination.

A summary of the critical sections and checks is given below.

Table 9.1
Critical sections and checks for prestressed beams.

Section	Likely to be critical for:	Location	Load combination	Critical time period
Midspan	ULS	–	Full ULS	Long term
	Decompression	Bottom of beam	Frequent	Long term
	Stress limit	Top of beam Slab Tendons	Characteristic	At opening and long term
End of diaphragm/end of transmission length	ULS	–	Full ULS	Long term
	Crack width	Slab (RC crack limit)	Quasi-permanent	At opening
	Stress limit	Bottom of beam Slab reinforcement	Characteristic	At opening
Over pier	ULS	–	Full ULS	–
	Crack width	Slab (RC crack limit)	Quasi-permanent	At opening
	Stress limit	Bottom of diaphragm Slab reinforcement	Characteristic	At opening
End of transmission length (and end of debonding, if any) at transfer of prestress	Crack width	Top of beam (prestressed crack limit)	Frequent	At transfer
	Stress limit	Bottom of beam Tendons	Characteristic	At transfer

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[†] Table NA.1 of the National Annex to BS EN 1992-2 specifies that sections not exposed to chlorides need only be checked for decompression under the less severe quasi-permanent load combination and for a crack width of 0.2 mm under the frequent load combination. This will allow reduced amounts of prestress to be used where chloride exposure does not occur, such as rail bridges that do not pass over roads or salt water.

9.2 Analysis results

The moments on the bridge due to the SLS loadcase combinations are shown in Appendix A, Table A9.1. Three critical regions are considered: midspan, the end of the transmission length in the precast beams near the central pier (pier B) and the RC diaphragm over the central pier.

For simplicity, the end of the transmission length has been assumed to coincide with the end of the central diaphragm – both regions that could prove critical. The end of the transmission length is actually a short distance beyond the end of the diaphragm, where the hogging moments due to applied loads are lower. Combining the hogging moments taken from the grillage model for the end of the diaphragm with the hogging prestressing moment for the simply supported beam model at the end of the transmission length is therefore conservative, and reduces the number of critical sections that must be checked.

The results in Appendix A, Table A9.1 show only the moments for the critical beams in the bridge; this was generally the central two bridge beams, apart from the quasi-permanent load case, where the edge beams were critical.

As will be shown later, no debonding of the tendons was necessary to meet transfer requirements, and so the prestressing is constant along the length of the beams (apart from in the transmission zones). This largely prevents the beams at the abutments being critical. However, lighter reinforcement would normally be used in the slab here than at the pier, which would have to be checked – something not done in this instance for brevity, but the calculations would follow the same method as used at the pier.

9.3 Decompression

Cracked sections should be considered if the tensile flexural stress exceeds $f_{ct,eff}$ (BS EN 1992-1-1, clause 7.1(2)). The use of cracked section analysis at SLS has some technical advantages over the BS 5400-4 approach of assuming uncracked sections. It does make the calculations more involved but this is generally not a major problem as the cracked section analysis can be done by computer. However, it does mean the prestress cannot be designed directly; as with crack width checks in RC, it is necessary to assume a design and then check it. It is therefore convenient to begin by carrying out the decompression checks – for typical beams with tendons near the tension face, the resulting analysis will be for an uncracked section, allowing the effect of variations in prestressing to be quickly evaluated using hand calculations or a simple spreadsheet.

Appendix A, Table A9.3 shows that the concrete located the minimum cover distance required for durability, $c_{min,dur}$ (30 mm) below the lowest tendon stays in compression at midspan.

If the prestressed concrete beams had tendons near the top of the section, decompression of these near the pier as a result of the hogging moment would also have to be checked.

9.4 Stress limits

The SLS stress limits are $0.6f_{ck}$ for concrete, $0.8f_{yk}$ for reinforcement and $0.75f_{pk}$ for prestressing tendons. For this design, these limits equate to:

- Concrete compressive stress limit = 0.6 x 50 = 30 MPa for the prestressed beam
  = 0.6 x 35 = 21 MPa for the in-situ slab
- Reinforcement stress limit = 0.8 x 500 = 400 MPa
- Prestressing tendon stress limit = 0.75 x 1860 = 1395 MPa

These stress limits are checked for the characteristic combination of loads. A cracked section analysis will generally be used, but clause 7.1(2) of BS EN 1992-1-1 allows uncracked sections to be used provided the peak tensile stress does not exceed the effective tensile strength of the section; in effect, this is the equivalent of checking to class 2 with BS 5400-4. Advantage is taken of this clause to check the stress limits of the beams at midspan, for which the effective tensile strength is 4.1 MPa.

As discussed previously, creep, and the resulting redistribution of load from the beam to the composite section over time, have been neglected in this analysis. This redistribution would have the effect of reducing the prestress on the beam, and hence increasing the tensile stresses at the soffit of the beam, while increasing the compressive stress on the in-situ slab. However, the calculated stress values are far enough from the limiting values such that the neglect of creep is not significant.

It has not been common practice when carrying out stress limit calculations by hand to consider the change in stress in the tendons due to the change in curvature of the section under the applied loads. This is slightly inconsistent, given that the effects of elastic shortening are considered when calculating the prestress losses. The change in stress in the tendons due to the applied loads has been calculated in this instance. This is not to imply that the Eurocodes require this, but simply to demonstrate that the error in neglecting this effect is relatively small – 8% in this case.

In the hogging region over the pier, the tensile stresses exceed the tensile strength of the concrete, so the stresses must be calculated based on cracked sections. For the diaphragm directly over the pier, which is one uniform mass of unprestressed concrete, the calculation is straightforward to carry out by hand, or using software such as SAM(3). The reinforcement used in this check was 20 mm bars at 150 mm centres in the top of the in-situ slab, and 12 mm bars at 150 mm centres in the bottom of the slab; this is the steel required to provide sufficient bending resistance over the pier, as calculated later in the ULS checks.

At the end of the diaphragm, allowance must be made for the stress distribution that results from the slab being cast after the pretensioned precast concrete beam has been installed. This can be achieved using a section analysis program such as SAM by applying initial strains to the portion of the section representing the precast beam. These initial strains would be obtained from an analysis of the prestressed concrete beam alone subject to prestress, self-weight and slab deadweight loads alone.

9.5 Crack width limitation

Crack widths need to be checked for the hogging regions, where the in-situ slab will crack. As the in-situ slab is reinforced rather than prestressed concrete, the limiting crack width is 0.3 mm under the quasi-permanent load combination. The reinforcement in the in-situ slab is 20 mm diameter bars at 150 mm centres in the top and 12 mm bars at 150 mm centres in the bottom.

The rules given in BS EN 1992-1-1, clause 7.3.4 for the calculation of crack width look somewhat complicated, containing a large number of parameters to allow for the range of variables that affect cracking. However, the rules are relatively straightforward to apply, and the procedure will be simplified once section analysis software has been updated to comply with the Eurocodes.

As an alternative to the calculations presented in Appendix A, sheet A9.5, it is often possible to use instead clause 7.3.3 which gives a simplified method for the sizing of reinforcement to limit crack widths. The simplified rules assume that the total area of reinforcement required by the section to meet ULS requirements has been calculated, and then places a limit on either the maximum bar diameter or maximum bar spacing that can be employed if the crack widths are not to exceed the specified limit.

If clause 7.3.3 were used to check compliance with the crack width limits at the pier, then the following procedure would be used:

i) ULS bending checks require a minimum of 4200 mm^2 per 1.5 m width of bridge (see later).
ii) If this area of steel is assumed to be placed at mid-depth within the in-situ slab, then the stress within the steel under quasi-permanent loading (–652 kNm), calculated using SAM for a cracked section, would be 151 MPa.
iii) Using Table 7.3 from clause 7.3.3, then for a steel stress of 151 MPa and a maximum crack width of 0.3 mm, the reinforcement should be spaced at centres of 300 mm or less, with the bar size being chosen such that the required area of steel is provided (i.e. 40 mm bars at 300 mm centres, or smaller bars at a smaller spacing).
iv) Alternatively, using Table 3.2, a maximum bar diameter of 32 mm is obtained. However, this value must then be modified using Equations 7.6N and 7.7N, which allow for the stress distribution within the section. This modification factor increases the maximum bar diameter to 50 mm, giving a bar spacing of 700 mm. It is clear that expecting bars at such a large spacing to control cracking to any significant degree is unrealistic.

In both cases, the resulting bar spacing would be superseded by clause 9.3.1.1, which specifies the maximum spacing of bars in slabs. For principal reinforcement in areas of maximum moment the bar spacing must not exceed 250 mm.

The actual reinforcement layout exceeds the requirements of either table by a considerable margin.

Although the notes to Tables 7.2 and 7.3 list the assumptions made in terms of material properties, cover etc. when deriving these tables, it is expected that they will be used even for situations that differ significantly from these values. However, there is an anomaly in that due to the low covers (25 mm) used to derive the tables, the simplified method can be more economic that the full rules.

Only the hogging region near the pier has been checked, as the moments at the abutments are much less. However, lighter slab reinforcement would usually be used away from the central pier, in which case the crack widths at the abutments should also be checked.

9.6 Transfer

At transfer, the critical section for the precast beam is at the end of the transmission zone. At this section the moment due to self-weight is negligible, and the prestressing force is causing the beam to hog and so placing the top face of the beam in tension. Under BS 5400, a tensile stress of 1 MPa was permitted in the extreme fibre at transfer. To comply with this limit, it was often necessary to place two or more tendons near the top face of the prestressed concrete beam and also to debond a number of the lower tendons close to the ends of the beams.

The decompression rule in BS EN 1992-1-1 applies at transfer. However, in contrast with previous practice, this limit on decompression only applies close to the tendon (within a distance equal to the cover required for durability). Therefore, if tendons were placed near the top of the section, then the region around them at the top of the section would have to stay in compression. However, if the tendons are omitted, then there is no requirement for the upper part of the section to remain in compression; instead, it only has to comply with the crack width limit of 0.2 mm specified in Table NA.1 of the National Annex to BS EN 1992-2.

Therefore it was decided to use two 12 mm bars in the top of the precast beam to control the crack widths (see Figure 8.1). While the material cost of this would be lower than the strand it replaced, if only because they would only have to be provided in the ends of the beams, it is not clear that it would actually represent a saving because the practicalities of precasting mean placing of strand is more convenient. However, in this particular case, it was found that using this approach made it possible to eliminate the debonding, which would result in further cost savings.

An alternative approach would be to treat the section as uncracked and limit the tensile stress at the top fibre to that given by clause 7.1(2). However, if this is done, it is still necessary to check decompression around the tendons. It is also necessary to use the tensile strength at the age of transfer. An estimate for this can be obtained from clause 3.1.2(9) although the note suggests this relationship is approximate and further justification would be needed to use the full value, typically 3 MPa.

The crack widths are calculated using the same method as was used in calculating the crack widths in the in-situ slab in the hogging region – the only difference being that the limiting crack width for prestressed concrete sections is 0.2 mm as opposed to 0.3 mm for reinforced concrete sections.

The maximum stresses at transfer must also be checked. The limiting tendon stress of the lesser of 75% of the characteristic tensile strength or 85% of the characteristic 0.1% proof stress has already been checked in section 8 above.

Clause 5.10.2.2 of BS EN 1992-1-1 places a limit on the maximum concrete stress at transfer of 60% of the characteristic concrete strength at the time of transfer. This may be increased to 70% if *it can be justified by tests or experience that longitudinal cracking is prevented*. It is expected that precast concrete manufacturers will utilise this increase.

It should be noted that BS EN 1992-1-1 explicitly states that the maximum transfer stress is derived from a *characteristic* value of concrete strength; this is a departure from previous practice, where under BS 5400 a mean value derived from a small number of cubes was commonly used. It is expected that in practice, rather than casting enough cubes to derive an accurate characteristic strength each time, precasters will use a small number of cubes in conjunction with historic records of the concrete variability that they achieve.

The peak concrete stress should be calculated using a cracked section where appropriate – where the tensile stress calculated on an uncracked section exceeds that given by clause 7.1(2). For economic reasons it is preferable for the prestressing force to have as large an eccentricity as possible, as this will reduce the required amount of prestress. The degree of eccentricity possible will normally be limited by the allowable compressive stress at transfer.

10. Global design at ULS

10.1 Analysis results

The ULS design forces obtained from the analysis are shown in Appendix A, Tables A10.1 and A10.2 for three critical locations: midspan, the edge of the diaphragm near the central pier and the diaphragm over the central pier.

The effects of shrinkage, temperature, differential settlement and secondary effects due to prestress are normally neglected at the ULS. This is because none of these loads results in a net loading on the structure as a whole, and so a sufficiently ductile structure (such as a properly detailed reinforced or prestressed concrete bridge) can accommodate these loads through redistribution without any loss in strength. Similarly, load distribution between the precast beam alone and the composite section is neglected, as the plastic behaviour of the concrete and steel means that at the ultimate limit all load will redistribute to be carried by the composite section.

10.2 Flexure

ULS flexural analysis to BS EN 1992-1-1 is based on the standard assumptions that:

- plane sections remain plane
- the strain in the reinforcement or bonded tendons is the same as that in the surrounding concrete
- the tensile strength of concrete is ignored.

As such, the methods used to calculate flexural capacity are very similar in principle to those used in other codes, barring differences in the stress–strain curves used to model the behaviour of the concrete, reinforcement and tendons.

Two stress–strain curves are available for reinforcement (Figure 10.1). Both are bilinear and identical up until first yield. Beyond this point, the first has a top branch that remains at a constant stress and assumes an infinite strain limit. The second has an inclined top branch that continues to increase in stress with increasing strain, but has a finite strain limit. The first model will usually be used for hand calculations, being easier to use, while the slightly higher moment resistance available with the second will probably be taken advantage of in analysis programs.

Two stress–strain curves are also available for prestressing tendons. They are similar in principle to those for reinforcement, one having a horizontal top branch, the other a sloping branch.

There are also two main stress blocks for concrete: a parabolic-rectangular stress block and a simpler rectangular stress block that is more suitable for hand calculations (Figure 10.2). This rectangular stress block is not applied over the full compression zone; instead it is applied over the top 80% of the compression zone for concrete strengths less than or equal to 50 MPa, and over a further reduced area for higher-strength concretes. This latter stress block is used in the calculations presented in Appendix A.

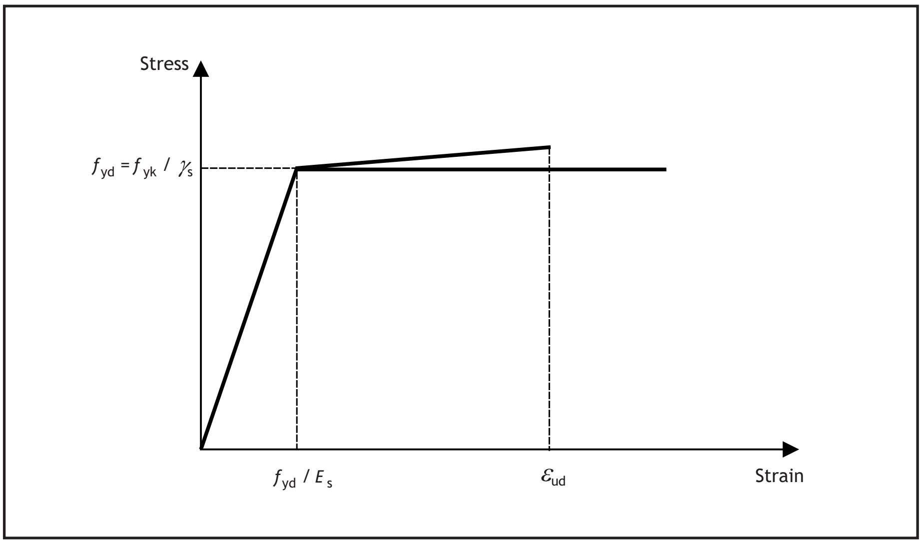
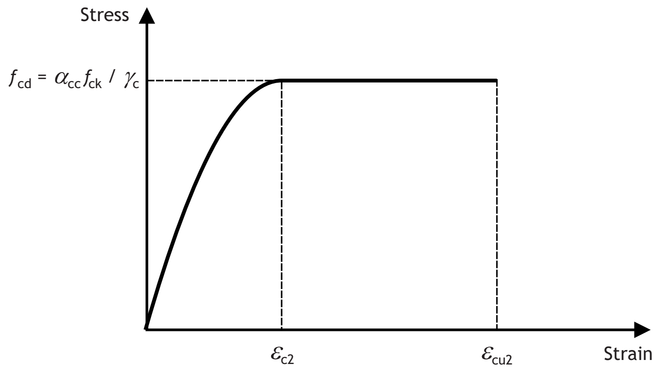
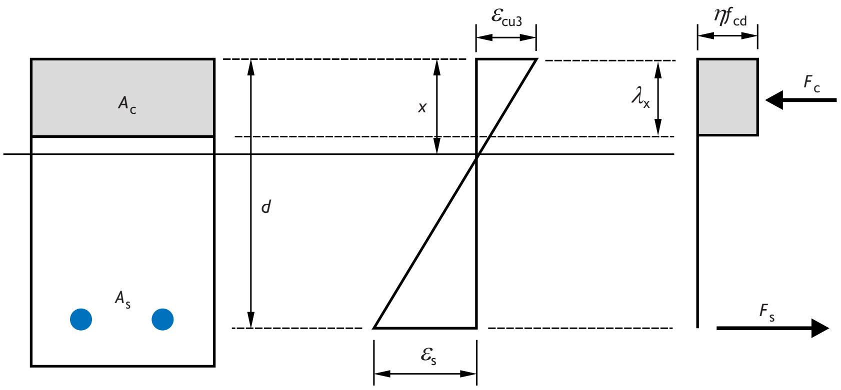


Figure 10.1
Alternative stress–strain curves for reinforcing steel – those for prestressing tendons are similar.



(a) Parabolic-rectangular stress–strain curve for concrete



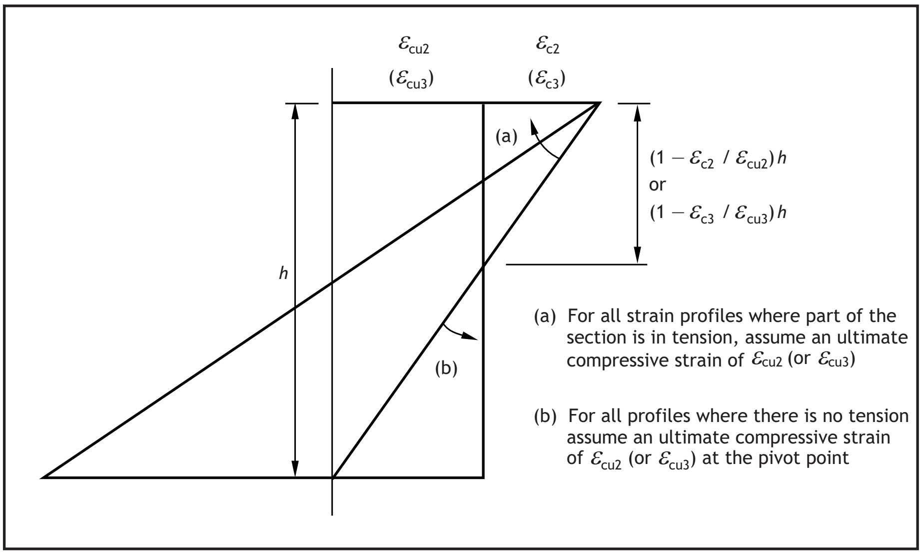
(b) Alternative rectangular stress distribution

Figure 10.2
(a) Parabolic-rectangular stress–strain curve for concrete; (b) alternative rectangular stress distribution.

Unlike BS 5400-2, BS EN 1992-1-1 does not provide the simple design formulae for calculating the ultimate moment capacities, but similar expressions are easy to derive. For heavily reinforced sections, or those subject to a significant axial force or prestressing, a check must be made that the reinforcement or tendons yield.

BS EN 1992-1-1 specifies two strain limits for the concrete: ε_{cu2} (or ε_{cu3} if the rectangular stress distribution is used) when the section is predominantly in bending; and a lower strain, ε_{c2} (or ε_{c3} if the rectangular stress distribution is used) if the loading is mainly axial, with the section being entirely in compression. The resulting possible strain distributions at failure are shown in Figure 10.3.

Figure 10.3
Possible strain distributions in the ultimate limit state.



10.2.1 Minimum reinforcement requirements

It is necessary to provide a minimum area of reinforcement in all reinforced concrete sections to ensure that the reinforcement in the section does not immediately yield when a crack forms, which would result in a small number of wide cracks forming and lead to brittle failure of the member. The minimum area is calculated according to clause 9.3.1.1 of BS EN 1992-1-1, and effectively calculates how much reinforcement is required to carry the stress that was carried by the tensile strength of the concrete prior to it cracking.

In prestressed concrete members this minimum reinforcement requirement does not have to be met provided that under the characteristic combination of actions the maximum value of the tensile stress on the section does not exceed the mean tensile strength of the concrete; in other words, provided that the concrete is (nominally) uncracked, there is no need to provide reinforcement to limit crack growth. However, clause 5.10.1(5(P)) of BS EN 1991-1-1 does require that “brittle failure of the member caused by failure of prestressing tendons shall be avoided”.

In clause 6.1(109), BS EN 1992-2 provides three methods for ensuring that compliance with this requirement is met. One method is to agree an appropriate inspection regime for the tendons with the relevant national authority, so as to prevent failure of the tendons due to deterioration (109(c)). This method is suitable for external tendons, which can readily be inspected.

An alternative method (109(a)) verifies that the structure has adequate load capacity even if some of the area of prestressing tendons is lost.

The third method (109(b)) checks that there is sufficient reinforcement to provide ductility in the event of cracking due to loss of prestress, which will allow redistribution of the load (provided that the structure has some degree of redundancy). Clause 6.1(110) states that for pretensioned members any tendons with a cover at least twice the minimum permissible cover can be included in the minimum steel area. However, the National Annex amends this to any tendon with at least minimum cover (i.e. any tendon) as there is no justification for the requirement for twice the cover.

10.2.2 Continuity reinforcement

Over time, an additional bending moment will arise over the pier due to creep. This moment will be sagging if prestress prevails over permanent load, hogging in the opposite case. The evaluation of this effect could be carried out by means of suitable creep and shrinkage calculations, and the resulting moment included in the analysis of the structure. However, the accuracy of such calculations is poor, due to uncertainties regarding the properties of the concrete and the precise timing of the construction sequence.

The alternative is to ignore these effects and simply ensure that there is adequate continuity steel in the bottom flange of the beams to provide crack control over the central pier in the event that creep due to prestress dominates. BS EN 15050 requires that the minimum area of continuity steel be calculated to clause 7.3.2 of BS EN 1992-1-1.

10.3 Shear

There are three types of shear check that must be carried out: global shear resistance of the composite beam and slab sections (and diaphragms), shear failure between the web and flanges of the composite section and shear failure at the interface between the beam and slab.

10.3.1 Global shear

Unlike BS 5400, BS EN 1992-1-1 uses the same method to calculate the shear capacity of both reinforced and prestressed concrete sections.

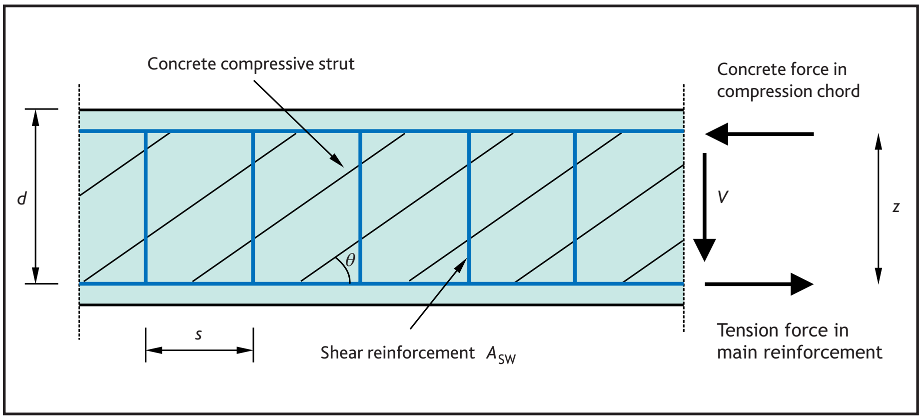
The calculation of the shear resistance of beams not requiring shear reinforcement is similar to that of BS 5400 for reinforced concrete sections, in that it takes into account the effect of concrete class, reinforcement ratio and effective depth of the section.

However, it differs in that an allowance is made for the enhanced shear capacity of sections subjected to compressive forces. As well as being utilised for the analysis of prestressed concrete sections, this can be used to advantage in the calculation of the shear capacity of columns and other reinforced concrete sections subject to axial forces.

Beams not requiring design shear reinforcement must still be provided with the minimum area of shear reinforcement specified in BS EN 1992-1-1, clause 9.2.2, but slabs and other structures capable of transverse redistribution of loads are spared this requirement.

When shear reinforcement is necessary, the variable-angle truss method is used to calculate the required area of reinforcement. This method is slightly more complicated than the fixed-angle truss method used by BS 5400 but can be more economical (see Figure 10.4).

Figure 10.4
Variable-angle truss model.



BS EN 1991-1-1 states that the lever arm between the tension and compression chords, z, can normally be taken as 0.9*d* for reinforced concrete without axial force. This value can be used when calculating the shear resistance of the diaphragm over the central pier.

When there is an axial force or prestress, the value of *z* is likely to differ from this, and so must be calculated more accurately. Guidance is given in PD 6687-2:2008, clause 6.2.4.1, but for this slightly unusual situation where the prestressing tendons are in the compression zone it is easiest to use a program such as SAM(3) to calculate the centroid of the concrete compression force. The lever arm, *z*, is then the distance between this and the centroid of the tension reinforcement.

The shear capacity, V_{Rd}, is given by the smaller of:

(Taken from BS EN 1992-1-1)
(clause 6.2.3, Equation 6.8)

$$V_{Rd} = \frac{A_{SW}}{s} z f_{ywd} \cot \theta$$

$$V_{Rd} = \frac{\alpha_{cw} b_w z \nu_1 f_{cd}}{(\cot \theta + \tan \theta)} = \frac{1}{2} \alpha_{cw} b_w z \nu_1 f_{cd} \sin 2\theta$$

(clause 6.2.3, Equation 6.9)

Equation 6.9 calculates the shear force at which the concrete compressive struts will crush, for any given angle of strut. Equation 6.8 calculates the shear force at which the shear reinforcement will yield, again for any given strut angle. For a fixed size of beam, a steeper strut angle will give a higher maximum shear capacity, as it increases the load required to cause crushing of the concrete, but at the expense of a correspondingly higher required area of shear reinforcement.

As presented, the two equations are not particularly useful due to the number of variables, and must first be rearranged into a form more suited to the required objective. Normally, the engineer will want to calculate the minimum area of shear reinforcement that will allow the section to carry the design load. This is found by first calculating the shallowest angle for the concrete struts that just avoids compressive failure of the concrete under the design shear force; this can be obtained by rearranging Equation 6.9, but is not permitted to be less than 21.8°. Once found, the strut angle can then be used in Equation 6.8 to obtain the required area of shear reinforcement; this is the route taken in the calculations shown in Appendix A, sheet A10.7.

Occasionally, the controlling requirement may be to minimise the depth of the section, or the width of the web. In this case, the strut angle should be set to its steepest permissible value (45°) and Equation 6.9 (the expression for crushing of the concrete) is then used to find the minimum value for the inner lever arm, z, and hence the required effective depth, d (from the approximation $z = 0.9d$). The necessary area of shear reinforcement can then be found using Equation 6.8.

The third possible option is determining the maximum shear capacity of a beam for which the depth and shear reinforcement has already been specified. In this case all variables are known apart from the angle of the struts. To find this, Equations 6.8 and 6.9 are equated and solved for the angle θ (within the limit set). Once found, it can be used in either equation to determine the shear capacity.

When calculating the shear capacity of a section, it should be noted that the value of α_{cc} used to calculate the design compressive strength of the concrete should be taken as 1.00, rather than the value of 0.85 used in flexural or axial calculations. This is because the shear formulae were derived from test results on this basis. Those situations where α_{cc} should be taken as 0.85 and those where it should be taken as 1.00 are defined in clause 3.1.6(101)P of the National Annex to BS EN 1992-2.

10.3.2 Shear failure between flange and web

Where a cross-section has thin flanges there is a risk of failure at the interface between the web and flanges. If the shear stress across the interface exceeds 40% of the design tensile strength of the concrete then transverse reinforcement must be provided across the interface to provide shear reinforcement.

10.3.3 Interface shear

Just as a shear failure could occur at the interface between the flanges and the web of a section, a shear failure can also occur at the interface between the precast beam and in-situ slab, where the tensile strength between the old and new concrete is lower than that of mass concrete.

10.4 Fatigue

Carrying out full fatigue checks on the reinforcement can be avoided by using clause 6.8.6 of BS EN 1992-1-1, whereby if the stress range of the reinforcement due to the live load component of the *frequent load combination* does not exceed a certain value then the fatigue resistance is deemed adequate. The stress range limits in BS EN 1992-1-1 are very low. The National Annex does not change these values but does allow other values to be specified by the 'appropriate authorities'. For UK highway bridges the higher values given in Table 2 of PD 6687-2:2008 may be used, which should avoid the need for detailed calculations in most cases.

For the bridge being considered in this publication the current stress range limits mean that the reinforcement over the pier would have to be increased by around 80% if detailed checks were to be avoided. However, when full fatigue checks are carried out, the current reinforcement is shown to be adequate.

These more detailed checks are carried out using the 'damage equivalent stress range' method, following the guidance given in Annex NN of BS EN 1992-2. The stress ranges are calculated using 'Fatigue Load Model 3', which represents a four-axle vehicle with an all-up weight of 48 t. Annex NN further increases this weight to 84 t for intermediate supports and 67 t for other areas. For reinforced concrete elements this load model can be applied on its own, the resulting forces found and hence the stress range in the reinforcement found. However, for prestressed concrete structures the load model should be applied in conjunction with the load combination given in clause 6.8.3 of BS EN 1992-1-1 (effectively the frequent load combination without traffic loads). This is necessary as the magnitude of the dead and superimposed loads will determine whether the section is cracked, which in turn will affect the tendon stresses.

The resulting stress range is then multiplied by a series of correction factors which allow for site-specific factors such as traffic volume and design life of the structure in order to obtain the damage equivalent stress range. When calculating one of these factors, $\lambda_{s,2}$, using Equation NN.103, it should be noted that N_{obs}, the number of lorries in the slow lane per year, should be entered in units of a million.

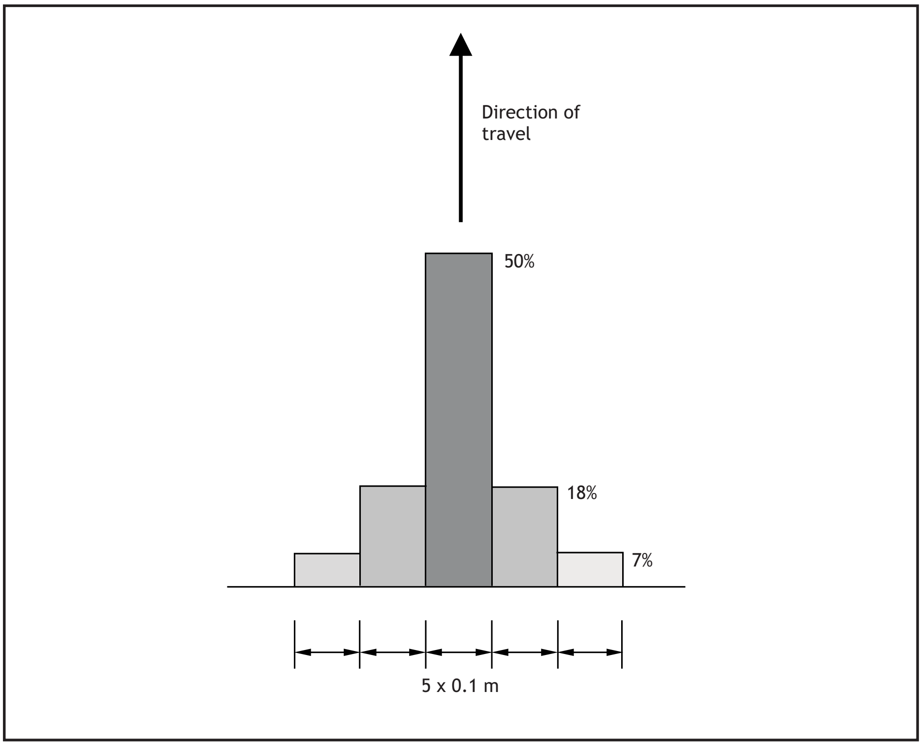
The fatigue resistance of the reinforcement or tendon is deemed adequate if the damage equivalent stress range is less than the factored permissible stress range at N^* cycles, which can be obtained from Tables 6.3N and 6.4N of BS EN 1992-1-1.

Clause 6.8.1(102) of the NA to BS EN 1992-2 relieves the designer of the need to carry out fatigue verification checks for the local effect of wheel load on slabs spanning between beams or webs, provided certain criteria are met.

In addition to the fatigue calculations presented in appendix A, the fatigue resistance of the interface reinforcement would also have to be checked where the link spacing increases to 150 mm and 300 mm. Similarly, the links in the pier diaphragm and the transverse reinforcement in the in-situ slab would also need checking.

When considering the transverse reinforcement in the slab, the transverse position of Fatigue Load Model 3 would have a significant effect. In such situations, a statistical distribution of the transverse location of the load model around its most critical position should be taken into account (Figure 10.5). The distribution to be used is given in BS EN 1991-2 clause 4.6.1(5) – although in practice it will often be sufficient to assume that all the vehicles pass in the worst position, and only refine the analysis if the fatigue resistance proves inadequate, especially for longer elements, where the variation of ±0.25 m will be fairly insignificant.

Figure 10.5
Statistical distribution of vehicles around critical position.

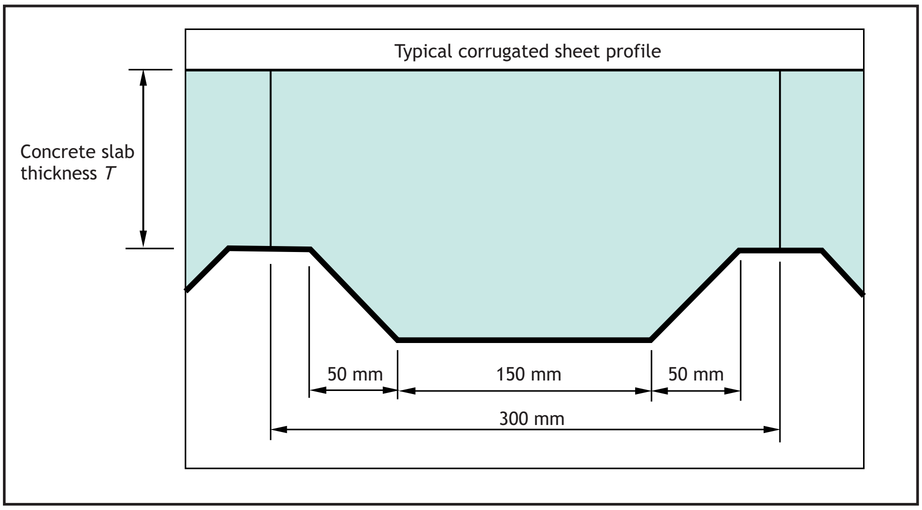


11. In-situ slab in the transverse direction

The in-situ slab is cast on ribbed glassfibre reinforced concrete (GRC) permanent formwork. The clear span between the beams (1.2 m) is near the limit of what can be achieved with profiled GRC formwork. If the precast beams were placed at larger centres, GRP formwork with encapsulated metal stiffeners could be used.

The profile of the GRC formwork is shown in Figure 11.1. In order to provide sufficient space for the four layers of reinforcement within a relatively thin slab, the bottom layer of transverse steel will be placed at alternate 200 and 100 mm centres, so that it can be positioned above the downstands in the formwork, and hence can be placed lower while still maintaining sufficient cover. The bottom face longitudinal steel will be fixed with the required 50 mm cover to the top of the ribs, and the bottom face transverse steel placed below this. In the top face, the transverse steel will be the outer layer.

Figure 11.1
Assumed profile of the permanent GRC formwork.



11.1 Loading and analysis

Transverse loading on the slab will be a combination of global and local effects. The global moments and shears can be obtained from the grillage model used for the design of the longitudinal elements, but this model is too coarse to obtain the local effects. For these, a finite-element model representing a region of slab between adjacent beams was used. The results of this analysis were then added to the results of the global analysis to obtain the design forces.

There are three wheel loads that have to be considered for local effects: a wheel from the tandem system associated with Load Model 1, a wheel from the single axle of Load Model 2, or one from the Abnormal Vehicles associated with Load Model 3. These are summarised in Table 11.1. Although the LM2 wheel is the heaviest and so produces the most severe local effects, it is only considered on its own, resulting in global effects that are less than with LM1 and LM3. It is not therefore obvious which will be critical, and so all must be considered.

Table 11.1
Wheel loads from BS EN 1991-2 and National Annex.

Load model	Contact area (length x width, mm)	Design load (kN)
LM1	400 x 400	150
LM2	400 x 400	200
LM3 (SV196)	350 x 350	99

Load spread is 1:1 through the surfacing and down to the level of the centroid of the slab (BS EN 1991-2, clause 4.3.6). Since the analysis will use uncracked section properties, this is taken as mid-depth. The minimum surfacing thickness is 150 mm, giving a spread of 230 mm in each direction.

The results are summarised in Table 11.2.

Table 11.2
Peak transverse stress resultants for the in-situ slab.

	ULS	SLS	
		Characteristic	Quasi-permanent
Sagging moment (kNm/m)	65	50	4
Hogging moment (kNm/m)	34	26	10

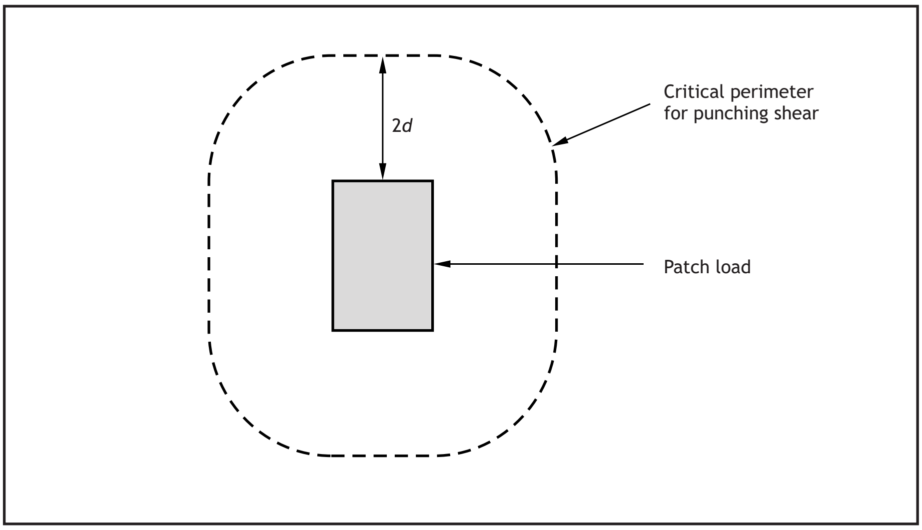
11.2 ULS and SLS verification

The principles used in the design of the longitudinal elements apply to the transverse design of the in-situ slab.

Additionally, a check has to be made of the punching resistance of the slab under wheel loads. This is carried out on a critical perimeter 2*d* from the edge of the loaded area. Unlike BS 5400, this edge distance is maintained around the corners of the loaded area, leading to the critical perimeter having curved corners, as shown in Figure 11.2.

BS EN 1992-1-1 simplifies the calculation of the shear perimeter by using an effective depth that is the average of that in the two directions. Similarly, the punching shear resistance of the slab is calculated for the average of the slab properties in the two directions.

Figure 11.2
Control perimeter for punching shear.



12. Design of the pier

12.1 Loading

The forces on the pier due to loading on the bridge deck can be obtained from the support reactions on the grillage model of the deck. These are the forces used for the main ULS and SLS calculations.

However, the effects of accidental vehicle impact with the pier must also be considered. BS EN 1991-2 suggests that impact loads should be modelled using a static equivalent impact force of 1000 kN in the direction of vehicle travel or 500 kN perpendicular to this. The National Annex requires that the significantly higher loadings specified in the NA of BS EN 1991-1-7 be used instead. These are summarised in Table 12.1 for highway bridges over motorways, trunk and principal roads. Both the main and residual load components should be considered together, but loads need only be considered in one direction at a time.

Table 12.1
Accidental impact loads on bridge piers over motorways, trunk and principal roads (BS EN 1991-1-7 NA).

	Force F_{dx} (parallel to carriageway, kN)	Force F_{dy} (perpendicular to carriageway, kN)	Height of application on bridge support
Main load component	1500	750	At the most severe point between 0.75 m and 1.5 m above carriageway
Residual load component	750	375	At the most severe point between 1.0 m and 3.0 m above carriageway

Note: Impact forces to be applied over a height of 0.5 m and a width of 1.5 m or the member width, whichever is less.

The pier design being considered in this publication extends most of the width of the deck (11.25 m) and is 600 m thick. It is therefore apparent that the collision loads perpendicular to the direction of flow of the traffic under the bridge are more likely to be critical. The moments and shears resulting from the accidental impact loads were evaluated using a simple frame model in which the pier was assumed to be fully fixed at the base and pinned at the top.

These accidental loads are to be considered in combination with loads due to traffic on the bridge, which can be obtained from the grillage model of the deck. These should be taken into account as accompanying actions at their frequent values (BS EN 1990, Annex A2 - Application for bridges, A2.2.5). All actions considered in the accidental design situation, whether accidental, accompanying traffic actions or permanent loads, should be used with a partial load factor γ of 1.0.

12.2 Ultimate limit state – persistent and accidental

BS EN 1992-1-1 specifies different material partial factors for use in ULS and accidental situations. Therefore it is not immediately apparent which will prove critical, and so the pier must be designed to one set of loads (persistent in this case) and checked against the other.

Imperfection in the construction of the pier means that axial loads can produce additional moments. This is taken into account in the analysis in two ways. First, it is assumed that the axial load is applied with an eccentricity of 1/30 of the depth of the section (but not less than 20 mm). Second, the column is assumed to deviate slightly from the vertical, while the axial force remains vertical. The additional moment produced by the axial force as a result of these assumptions must be added to the moment obtained from the grillage model. Tall, thin piers may buckle under axial loading. The likelihood of this is related to the *slenderness* of the member. Provided the slenderness of an axially loaded member is below a limiting value, as it is in this example, then buckling does not need to be considered. Otherwise, methods for including second-order effects are given in BS EN 1992-1-1, sections 5.8.6 to 5.8.8.

12.3 Serviceability limit state

The pier must be checked for stress limits under the characteristic load combination and crack widths under quasi-permanent loads. The additional moments due to geometric imperfections of the pier do not need to be considered. The calculations are similar to those carried out for the central diaphragm in section 9 above, and so are not repeated in section 12 of Appendix A.

13. Piled foundation design

As indicated earlier, the scheme design allowed for the foundations for the bridge to consist of precast concrete piles with in-situ pile caps.

The characteristic actions for the pier and abutment loading are obtained from grillage analysis and result in vertical, horizontal forces and overturning moments applied to the foundations.

BS EN 1997-1 presents three design approaches in section 2.4.7.3.4, which define the partial factors that are to be applied to the actions or effects of actions, soil parameters and resistances. This is overridden by the National Annex that specifies that only Design Approach 1 may be used and which requires that two combinations of partial factors be considered.

The ground conditions were taken as consisting of made ground, overlying river terrace gravels upon stiff clay, with a highest water table 2.25 m below ground level. The made ground was deemed too variable thus relying on the gravel and clay for foundation support.

The pile resistance (base resistance and shaft resistance) was assessed from the design material parameters modified by correction factors (dependent upon the number of ground tests).

This led to pier piles of 31.5 m and abutment piles 27.5 m.

14. Conclusions

All the key calculations in the design of the superstructure of a two-span integral beam-and-slab bridge and its piled foundations have been presented in Appendix A. There still remains a number of items that need considering, including:

- design of the string course, including the effects of accidental impact with the parapets and the resulting forces transmitted to the in-situ slab
- forces on the bridge due to impact with the deck by vehicles passing under the bridge
- design of the abutments, including the run-on slab
- detailing of the cill beam.

References

1 HENDY, C.R. and SMITH, D.A. *Designers' Guide to BS EN 1992 Eurocode 2: Design of concrete structures. Part 2: concrete bridges*. Thomas Telford, London, 2007.

2 PUCHER, A. *Influence Surface of Elastic Plates*. Springer Verlag, Wien and New York, 1964.

3 *SAM-LEAP5, version 5.20f*, Bestech Systems Ltd, 2007.

CONCRETE BRIDGE DEVELOPMENT GROUP

Project details

CBDG Design Example
Material properties and cover

Chapter Ref. 5
Clause Ref. 5.1–5.4
Sheet No. A5.1

BS EN 1992-1-1

CONCRETE

From BS EN 1992-1-1, Table 3.1:

Table A5.1

3.1.2 (3)
3.1.2 (9)
3.1.3 (2)

Class			C50/60	C35/45
Characteristic cylinder strength	f_{ck}	MPa	50	35
Characteristic strength at transfer		MPa	32*	-
Mean value of axial tensile strength	f_{ctm}	MPa	4.1	3.2
Secant modulus of elasticity	E_{cm}	GPa	37	34

* Typically derived from a small number of cube tests and the precaster's historical records

BS 5896

PRESTRESSING STEEL

Table A5.2

Grade			1860
Tensile strength	f_{pk}	MPa	1860
0.1% proof stress	$f_{p0.1k}$	MPa	1580
Modulus of elasticity	E_p	GPa	195
Elongation at maximum force	e_{uk}	%	3.5
Relaxation class			Class 2

BS EN 10080/
BS 4449

REINFORCING STEEL

Table A5.3

Grade			B500B
Yield strength	f_{yk}	MPa	500
Modulus of elasticity	E_s	GPa	200
Elongation at maximum force	e_{uk}	%	5.0

BS EN 1992-1-1
4.4.1 and BS 8500

CONCRETE COVER TO REINFORCEMENT

Table A5.4

Element	Surface	Concrete class	Exposure class	c_{min} mm	Δc_{dev} mm	c_{nom} mm
Slab	Deck	C35/45	XC3	35	10	45
	Soffit	C35/45	XD1	40	10	50
Beam		C50/60	XD1	30	5	35
Diaphragm	Deck	C35/45	XC3	35	10	45
	Soffit/side	C35/45	XD1	40	15	55
Pier wall		C35/45	XD3	50	15	65
Stringcourse		C35/45	XD3	50	15	65

CONCRETE BRIDGE DEVELOPMENT GROUP

Project details

CBDG Design Example
Permanent actions

Chapter Ref. 6
Clause Ref. 6.1.1–6.1.3
Sheet No. A6.1

BS EN 1991-1-1

DEAD LOAD

The following nominal weights are used in the calculations:

Reference	Material	Weight
Annex A	Concrete	24.0 kN/m^3
Annex A	Reinforced/prestressed concrete	25.0 kN/m^3
Annex A	Hot rolled asphalt	23.0 kN/m^3
	Parapet	0.5 kN/m

Self-weight 1a (dead load)

Weight of beam and slab = 0.680 m^3 x 25.0 kN = 17.0 kN/m per beam
(volume allows for transverse ribs of permanent formwork)

Resulting moment on beam = unit weight x (span length)^2/8
= 17.0 x 19.0^2/8 = 767 kNm

Self-weight 1b (dead load)

Reaction on diaphragm from beam = unit weight x total length of beam/2
= 17.0 x 19.5/2 = 166 kN

Self-weight 2 (super-imposed dead)

Weight of string course and parapet = 0.23 m^3 x 25.0 kN + 0.5 kN
= 6.3 kN/m

Weight of footway (270 mm concrete) = 0.270 m x 24.0 kN
= 6.5 kN/m^2

5.2.3(3)

Self-weight 3 (surfacing)

Weight of surfacing (average 120 mm thick) = 0.120 m x 23.0 kN x 1.55
= 4.3 kN/m^2

DIFFERENTIAL SETTLEMENT

The following settlement differential between the supports will be considered in the most unfavourable arrangement:

Settlement at opening = ± 10 mm
Total settlement = ± 20 mm

BS EN 1992-1-1

DIFFERENTIAL SHRINKAGE

Differential shrinkage values calculated using shrinkage values derived on calculation sheets A7.1 and A7.2

Table A6.1

	Shrinkage at:		
	Construction ($\mu\varepsilon$)	Opening for traffic ($\mu\varepsilon$)	Long term ($\mu\varepsilon$)
Precast beam	109	237	379
In-situ slab	0	143	286

The effect of concrete relaxation is taken into account by using an effective elastic modulus which is taken as half the value of the tangent modulus.

CONCRETE BRIDGE DEVELOPMENT GROUP

Project details

CBDG Design Example
Permanent and variable actions

Chapter Ref. 6
Clause Ref. 6.1.3–6.2
Sheet No. A6.2

Shrinkage differential at time zero (when bridge is opened for traffic):

Difference in shrinkage, Δz = (143 - 0) - (237 - 109) = 15 $\mu\varepsilon$

M = stress due to shrinkage Δz x area of deck x lever arm about centroid of composite section

= $\Delta z\ (E_c/2)\ A_{c,\ \text{in situ}}$ x lever arm

= $15 \times 10^{-6} \times 17.85 \times 10^3 \times 240000 \times (1160 - 638 - 160/2)$ = 28 kNm

N = $M/(y_{slab} - y_{beam})$

= $28 \times 10^6/(1080 - 400)$ = 41 kN

Shrinkage differential after long time:

Δz = $(286 - 0) - (379 - 109)$ = 16 $\mu\varepsilon$

M = $19 \times 10^{-6} \times 17.85 \times 10^3 \times 240000 \times (1160 - 638 - 160/2)$ = 36 kNm

N = $36 \times 10^6/(1080 - 400)$ = 53 kN

BS EN 1991-1-5

VARIABLE ACTIONS

6.1

TEMPERATURE

6.1.3

UNIFORM TEMPERATURE COMPONENT

Bridge deck type = Type 3 (concrete deck)
Average surfacing thickness = 166 mm
Altitude ≈ Sea level

Table A6.2

	Shade air temperature (NA 2.5)	Adjusted for altitude (Annex A)	Uniform bridge temperature, T_e (Fig 6.1)	T_e adjusted to allow for surfacing thickness (Table NA. 1)
Minimum	-13	-13	-6	-5
Maximum	34	34	36	35

Initial temperature, T_0 = more severe of 0°C or 20°C

Therefore:

NA.2.21

Maximum contraction range, $\Delta T_{N,con}$ = $T_0 - T_{e,min}$

= $20 - (-5)$ = 25°C

Maximum expansion range, $\Delta T_{N,exp}$ = $T_{e,max} - T_0$

= $35 - 0$ = 35°C

The coefficient of thermal expansion for concrete, α, is 1.00×10^{-5}/°C

Maximum contraction = $\alpha\ \Delta T_{N,con}\ L$ (where L = total length of bridge)

BS EN 1992-1-1

= $1.00 \times 10^{-5} \times 25 \times 40000$ = 10 mm

3.1.3(5)

Maximum expansion = $1.00 \times 10^{-5} \times 35 \times 40000$ = 14 mm

CONCRETE BRIDGE DEVELOPMENT GROUP

Project details

CBDG Design Example
Variable actions

Chapter Ref. 6
Clause Ref. 6.2
Sheet No. A6.3

BS EN 1991-1-5
6.1.4 and NA.2.9

TEMPERATURE DIFFERENCE COMPONENT

Temperature profiles

Extrapolating from Table B.3 (BS EN 1991-1-5) for a depth of beam and slab of 1.160 m and an average depth of surfacing of 166 mm, the heating and cooling temperature difference distributions are:

Table A6.3

Heating		Cooling	
Depth (m)	Temp. (°C)	Depth (m)	Temp. (°C)
0.000	9.2	0.000	-5.8
0.150	2.4	0.232	-0.8
0.400	0.0	0.432	0.0
0.894	0.0	0.728	0.0
1.160	1.8	0.928	-1.6
		1.160	-6.1

The resulting restrained forces can be calculated using SAM(3) or other similar programs:

Table A6.4

	Moment, M (kNm)	Axial force, N (kN)
Heating	183	554
Cooling	-15	-626

The uniform temperature components, ΔT_U, and linearly varying temperature difference component, ΔT_M, of the temperature difference profile can be calculated from the relaxing forces as follows:

$$\Delta T_U = (N/A_c)/(E\alpha) \quad = \quad (N/0.631)/(37 \times 10^6 \times 1 \times 10^{-5})$$
$$\Delta T_M = (Mh/I)/(E\alpha) \quad = \quad (M \times 1.16/1.05 \times 10^{-1})/(37 \times 10^6 \times 1 \times 10^{-5})$$

Giving:

Table A6.5

	ΔT_U (°C)	ΔT_M (°C)
Heating	2.4	5.5
Cooling	-2.7	-0.4

BS EN 1991-1-5
6.1.4.2(1)

The uniform temperature component, ΔT_U, has already been incorporated into the earlier calculation of the uniform bridge temperature component, and so can be neglected here.

The linearly varying temperature component will induce curvature of the bridge deck, which will cause moments and shear forces to be set up in the bridge deck. Therefore it should be included in the global model.

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CONCRETE BRIDGE DEVELOPMENT GROUP

Project details

CBDG Design Example
Variable actions

Chapter Ref. 6

Clause Ref. 6.2, 6.3.1 –6.3.3

Sheet No. A6.4

Non-linear temperature difference component

There is an additional non-linear temperature difference component, ΔT_E. This component is self-equilibrating, so does not need to be included in the global model. However, it sets up local stresses within the deck that must be included in SLS calculations. The stresses are those resulting from the difference between the original temperature distribution and the uniform and linearly varying temperatures.

Table A6.6

Heating			Cooling		
Depth (m)	ΔT_E (°C)	σ_E (MPa)	Depth (m)	ΔT_E (°C)	σ_E (MPa)
0.000	4.4	1.49	0.000	-2.9	-0.98
0.150	-1.7	-0.59	0.160	0.5	0.16
0.160	-1.8	-0.61	0.160	0.5	0.17
0.160	-1.8	-0.66	0.232	2.0	0.74
0.400	-3.0	-1.09	0.432	2.7	1.01
0.894	-0.6	-0.23	0.728	2.6	0.96
1.160	2.4	0.90	0.928	0.9	0.34
			1.160	-3.6	-1.35

Compressive stresses are positive

TRAFFIC LOADS

BS EN 1991-2
4.2.3

Divisions of the carriageway into notional lanes

Carriageway width w = 7.30 m

Width of notional lanes w_1 = 3.00 m

Number of notional lanes n_1 = 2

Width of remaining area w_r = 1.30 m

4.3.2

LOAD MODEL 1

Table A6.7

	Tandem system			UDL system		
	Q_{ik} (kN)	α_{Qi}	$\alpha_{Qi}\,Q_{ik}$ (kN)	q_{ik} (or q_{rk}) (kN/m^2)	α_{qi} (or α_{qr})	$\alpha_{qi}\,q_{ik}$ (kN/m^2)
Lane number 1	300	1.00	300	9.0	0.61	5.5
Lane number 2	200	1.00	200	2.5	2.20	5.5
Remaining area				2.5	2.20	5.5

4.3.3

LOAD MODEL 2

Axle weight Q_{ak} = 400 kN

Adjustment factor β_Q = α_{Q1} = 1.00

Applied axle weight $\beta_Q Q_{ak}$ = 400 kN

CONCRETE BRIDGE DEVELOPMENT GROUP

Project details

CBDG Design Example
Variable actions - traffic loads

Chapter Ref. 6
Clause Ref. 6.3.4–6.3.7
Sheet No. A6.5

BS EN 1991-2
4.3.4 and NA.3.1

LOAD MODEL 3 - Model SV196

Table A6.8

Basic axle load (kN)	Dynamic amplification factor	Design axle weight (kN)
100	1.20	120
165	1.12	185
180	1.10	198

4.3.5

LOAD MODEL 4 - Not considered

5.3.2.1(1) and NA.2.11.2

FOOTPATH LOADING

Uniformly distributed load on footway, q_{fk} = $2.0 + 120/(L + 10)$ kN/m^2

But $2.5\ \text{kN/m}^2 \leq q_{fk} \leq 5.0\ \text{kN/m}^2$

Where L is the loaded length in metres. For this bridge the loaded length will be 20 m when considering mid-span sagging and 40 m when considering hogging over the central pier. Therefore:

Uniformly distributed load on footway, q_{fk} = 5 kN/m^2 when one span loaded
= 4.4 kN/m^2 when both spans loaded

4.7

ACCIDENTAL LOADING - Not considered

4.4

HORIZONTAL LOADING - Not considered

4.5

GROUPS OF TRAFFIC LOADS

Table NA.3

Characteristic values

Table A6.9

Load system		Carriageway			Footway
		LM1 (TS and UDL)	LM2 (Single axle)	LM3 (Special vehicles)	UDL
Groups of loads	gr1a	Characteristic			0.6 x Characteristic
	gr1b		Characteristic		
	gr3				Characteristic
	gr5	Frequent		Characteristic	

Table 4.4b

Frequent values

Table A6.10

Load system		Carriageway		Footway
		LM1 (TS and UDL)	LM2 (Single axle)	UDL
Groups of loads	gr1a	Frequent		
	gr1b		Frequent	
	gr3			Frequent

CONCRETE BRIDGE DEVELOPMENT GROUP

Project details

CBDG Design Example
Combinations of actions

Chapter Ref. 6
Clause Ref. 6.4
Sheet No. A6.6

BS EN 1990

COMBINATIONS OF ACTIONS

SERVICEABILITY LIMIT STATE

6.5.3 There are four relevant combinations of actions to be considered – three for SLS and one for ULS. The three SLS combinations are:

Characteristic combination: $E_d = E\,(\Sigma G_{k,j} + P + Q_{k,1} + \Sigma \psi_{0,i} Q_{k,i})$

Frequent combination: $E_d = E\,(\Sigma G_{k,j} + P + \psi_{1,1} Q_{k,1} + \Sigma \psi_{2,i} Q_{k,i})$

Quasi-permanent combination: $E_d = E\,(\Sigma G_{k,j} + P + \Sigma \psi_{2,i} Q_{k,i})$

where $j \geq 1$; $i > 1$

The combination factors are give in Table NA.A.2.1 of the National Annex to the BS EN 1990 bridges annex. These are summarised below for the variable loads being considered here.

Table A6.11

Action	Group of loads	Load component	ψ_0	ψ_1	ψ_2
Thermal actions			0.60	0.60	0.50
Traffic loads	gr1a	TS	0.75	0.75	0.00
		UDL	0.75	0.75	0.00
		Pedestrian loads	0.40	0.40	0.00
	gr3	Pedestrian loads	0.00	0.40	0.00
	gr5	Vertical forces from SV vehicles	0.00	0.00	0.00

ULTIMATE LIMIT STATE

6.4.3.2 The ULS combination to be considered for “STR” limit state is:

Equ. 6.10 $E_d = E\,(\Sigma \gamma_{G,j} G_{k,j} + \gamma_P P + \gamma_{Q,1} Q_{k,1} + \Sigma \gamma_{Q,i} \psi_{0,i} Q_{k,i}) \quad j \geq 1;\ i > 1$

Table NA.A.2.4 (B)

Table A6.12

Group of loads	γ_{sup}	γ_{inf}	ψ_0
Dead load	1.35	0.95	-
Superimposed DL	1.20	0.95	-
Temp. diff.	1.50	0.00	0.00*
Prestress	1.10**	0.90**	-
Traffic groups (gr1a, gr3, gr4 and gr5)	1.35	0.00	N/A***

* The value of ψ_0 for thermal actions of 0.60 may be reduced to 0 in most cases when considering Ultimate Limit States (NA to BS EN 1990, Table NA.A.2.1)
** From the National Annex to BS EN 1992-1-1, clause 2.4.2.2
*** For a bridge of this type, traffic loads will always be the leading action.

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Project details

CBDG Design Example
Creep and shrinkage properties

Chapter Ref. 7
Clause Ref. 7
Sheet No. A7.1

BS EN 1992-1-1	CREEP AND SHRINKAGE MODEL PARAMETERS FOR PRECAST BEAM					
3.1.4 & Annex B	Mean compressive cylinder strength			f_{cm}	=	$f_{ck} + 8$
						58 MPa
	Cross-section area of concrete member			A	=	410191 mm^2
	Perimeter in contact with the atmosphere			u	=	3128 mm
Equ. B.6	Notional size of member	h_0	=	$2A_c/u$	=	262 mm
	Relative humidity of ambient environment			RH	=	75%
	Equivalent concrete age at release of prestress			$t_{0,eq}$	=	1 day
	Concrete age at construction			t_1	=	30 days
	Concrete age when bridge is opened for traffic			t_2	=	180 days
	CREEP MODEL					
		α_1	=	$(35/58)^{0.7}$	=	0.70
Equ. B.8c	Coefficients allowing for concrete strength	α_2	=	$(35/58)^{0.2}$	=	0.90
		α_3	=	$(35/58)^{0.5}$	=	0.78
Equ. B.3	Factor allowing for relative humidity			φ_{RH}	=	1.15
Equ. B.4	Factor allowing for concrete strength			$\beta(f_{cm})$	=	2.21
Equ. B.9	Modification to t_0 to allow for type of cement			t_0	=	4 days
Equ. B.5	Factor allowing for concrete age at loading			$\beta(t_0)$	=	0.70
Equ. B.8	Coefficient dependent on relative humidity			β_H	=	647
Equ. B.2	Notional creep coefficient	φ_0	=	$\varphi_{RH}\beta(f_{cm})\beta(t_0)$	=	1.15 x 2.21 x 0.70
					=	1.79
	SHRINKAGE MODEL					
	Age of concrete at beginning of drying shrinkage			t_s	=	1 day
Equ. 3.12	Final value of the autogenous shrinkage strain			$\varepsilon_{ca}(\infty)$	=	100 με
Equ. B.12	Factor allowing for relative humidity			β_{RH}	=	0.90
Table 3.3	Coefficient depending on notional size			k_h	=	0.79
Equ. B.11	Basic drying shrinkage strain			$\varepsilon_{cd,0}$	=	354 με
Equ. 3.9	Final value of the drying shrinkage strain			$\varepsilon_{cd,\infty}$	=	279 με

DEVELOPMENT OF CREEP AND SHRINKAGE WITH TIME FOR PRECAST BEAM

Table A7.1

			At stress transfer	At construction	At opening for traffic	Long term
	t	day	1	31	181	∞
	$t-t_s$	day	0	30	180	∞
	$t-t_0$	day	0	30	180	∞
Equ. B.7	$\beta_c(t,t_0)$	-	0.00	0.39	0.63	1.00
Equ. 3.13	$\beta_{as}(t)$	-	0.18	0.67	0.93	1.00
Equ. 3.10	$\beta_{ds}(t,t_s)$	-	0.00	0.15	0.51	1.00
Equ. B.1	Creep coefficient, $\varphi(t, t_0)$	-	0.00	0.70	1.13	1.79
Equ. 3.11	Autogenous shrinkage strain, $\varepsilon_{ca}(t)$	με	18	67	93	100
Equ. 3.9	Drying shrinkage strain, $\varepsilon_{cd}(t)$	με	0	42	143	279
	Total shrinkage strain, $\varepsilon_{cs}(t) = \varepsilon_{cs}(t) + \varepsilon_{cd}(t)$	με	18	109	237	379

CONCRETE BRIDGE DEVELOPMENT GROUP

Project details

CBDG Design Example
Creep and shrinkage properties

Chapter Ref. 7
Clause Ref. 7
Sheet No. A7.2

BS EN 1992-1-1

SHRINKAGE MODEL PARAMETERS FOR IN-SITU SLAB

Ref.	Parameter	Symbol		Value
3.1.4 & Annex B	Characteristic compressive cylinder strength	f_{ck}	=	35 MPa
Table 3.1	Mean compressive cylinder strength	f_{cm}	=	43 MPa
	Cross-section area of concrete member	A	=	1500 x 160
			=	240000 mm^2
	Perimeter in contact with the atmosphere	u	=	1500 mm
Equ. 6	Notional size of member	h_0	=	2 x 240000/1500
			=	320 mm
	Relative humidity of ambient environment[1]	RH	=	75%

SHRINKAGE MODEL

Ref.	Parameter	Symbol		Value
Equ. 3.12	Final value of the autogenous shrinkage strain	ε_{ca}	=	63 με
Equ. B.12	Factor allowing for relative humidity	β_{RH}	=	0.90
Table 3.3	Coefficient depending on notional size	k_h	=	0.75
Equ. B.11	Basic drying shrinkage strain	$\varepsilon_{cd,0}$	=	300 με
Equ. 3.9	Final value of the drying shrinkage strain	ε_{cd}	=	224 με

DEVELOPMENT OF SHRINKAGE WITH TIME FOR IN-SITU SLAB

Table A7.2

Ref.		Units	At opening for traffic	Long term
	t	day	150	∞
	t-t_s	day	144	∞
Equ. 3.13	$\beta_{as}(t)$	-	0.91	1.00
Equ. 3.10	$\beta_{ds}(t,t_s)$	-	0.39	1.00
Equ. 3.11	Autogenous shrinkage strain	με	57	63
Equ. 3.9	Drying shrinkage strain	με	86	224
	Total shrinkage strain	με	143	286

[1] In the absence of site specific data such as used here, PD 6687-2:2008 for BS EN 1992-2 suggests that the relative humidity can be taken as 70% for bridges within the UK.

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Project details

CBDG Design Example
Applied prestressing force and prestressing losses

Chapter Ref. 8
Clause Ref. 8.1
Sheet No. A8.1

BS EN 1992-1-1

RESULTANT APPLIED PRESTRESSING FORCE

5.10.2.1

Allowed maximum applied stress in tendon $\sigma_{p,max}$ = 1422 MPa

Table A8.1

5.10.2.1

No. of strands	Height mm	Area per strand mm^2	Applied stress MPa	Force per strand kN	Total axial force kN	Moment about soffit kNm
6	60	150	1422	213	1280	77
4	110	150	1422	213	853	94
4	160	150	1422	213	853	137
4	210	150	1422	213	853	179
4	260	150	1422	213	853	222
22	← SUM →				4693	708
Height of resultant axial force =					151 mm	

ECCENTRICITY OF PRESTRESSING FORCE AND RESULTANT MOMENT

Table A8.2

Section	CoG mm	Eccentricity mm	Force, F kN	Moment, M kNm
Beam	400	249	4693	1169

LOSSES OF PRESTRESS

Area of concrete member A_c = 410191 mm^2

Section modulus for centroid of steel Z_c = $I/(y-133)$

= $3.82 \times 10^{10}/(400 - 133)$

= 1.43×10^{8} mm^3

Sheet A5.1

Secant modulus of elasticity of concrete E_{cm} = 37000 MPa

3.1.4(2)

Tangent modulus of elasticity of concrete E_c = $1.05 \times E_{cm}$

Modulus of elasticity of prestressing steel E_p = 195000 MPa

Loss of prestress due to instantaneous deformation

Equivalent concrete age at release of prestress $t_{eq,p}$ = 1 day

Equ. 3.5

Modulus of elasticity at release of prestress $E_{cm}(t)$ = 28601 MPa

Loss of prestress $\Delta\sigma_E{}^*$ = $(F/A_c + M/Z_c) \times (E_p/E_{cm}(t))$

= 130 MPa

Loss of prestress due to creep

Note: Before construction the concrete stress, σ_c, is approximated as $F/A_c + M/Z_c$. After construction the concrete stress, σ_c, is taken as F/A_c as the true stress state under permanent loads is not known.

Equ. 3.6

Loss of prestress at construction, $\Delta\sigma_{Cr}(t_1)$ = $\varphi(t_1)\sigma_c E_p/E_c$

= $0.70 \times 20.2 \times 195000/(1.05 \times 37000)$

= 67 MPa

Loss of prestress at opening for traffic, $\Delta\sigma_{Cr}(t_2)$ = $\Delta\sigma_{Cr}(t_1) + (\varphi(t_2)) - \varphi(t_1))\sigma_c E_p/E_c$

= $71 + (1.13 - 0.70) \times 11.4 \times (195000/38850)$

= 90 MPa

Loss of prestress after long time, $\Delta\sigma_{Cr}(t_\infty)$ = $71 + (1.79 - 0.70) \times 11.4 \times (195000/38850)$

= 126 MPa

CONCRETE BRIDGE DEVELOPMENT GROUP

Project details

CBDG Design Example
Losses of prestress and
transmission length

Chapter Ref. 8
Clause Ref. 81–8.2
Sheet No. A8.2

BS EN 1992-1-1

Loss of prestress due to shrinkage

Stress loss at stress transfer $\Delta\sigma_S = \varepsilon_{cs}(t_0)\,E_p$
$= 18 \times 10^{-6} \times 195000 = 4$ MPa

Stress loss at construction $\Delta\sigma_S = 109 \times 10^{-6} \times 195000 = 21$ MPa

Stress loss at opening for traffic $\Delta\sigma_S = 237 \times 10^{-6} \times 195000 = 46$ MPa

Stress loss after long time $\Delta\sigma_S = 379 \times 10^{-6} \times 195000 = 74$ MPa

3.3.2 — Loss of prestress due to relaxation

3.3.2(6) — Nominal relaxation at 1000 hours $\rho_{1000} = 2.5\%$

Ratio of initial prestress to tensile strength
$\mu = 1422/1860 = 0.76$

Equ. 3.29 — Relaxation loss at stress transfer $\Delta\sigma_R = 0.66\rho_{1000}\, e^{9.1\mu}\, (t/1000)^{0.75\,(1-\mu)} \times 10^{-5} \times \sigma_{pi}$
$= 0.66 \times 2.5\, e^{9.1 \times 0.76}\, (24/1000)^{0.75(1-0.76)} \times 10^{-5} \times 1422$
$= 13$ MPa

Relaxation loss at construction $\Delta\sigma_R = 23$ MPa

Relaxation loss at opening for traffic $\Delta\sigma_R = 32$ MPa

Relaxation loss after long time $\Delta\sigma_R = 74$ MPa

EFFECTIVE PRESTRESS IN TENDON $\sigma_{p,eff} = \sigma_p - \Delta\sigma_E - \Delta\sigma_{Cr} - \Delta\sigma_S - \Delta\sigma_R$

At stress transfer $\sigma_{p,eff} = 1422 - 114 - 0 - 4 - 13$
$= 1276$ MPa (90%)

At construction $\sigma_{p,eff} = 1180$ MPa (83%)

At opening for traffic $\sigma_{p,eff} = 1123$ MPa (79%)

Long term $\sigma_{p,eff} = 1019$ MPa (72%)

5.10.3(2) — Allowed maximum stress in tendon after transfer
$\sigma_{pm0} = \min\{0.75f_{pk};\ 0.85f_{p0,1k}\}$
$= \min\{0.75 \times 1860;\ 0.85 \times 1580\}$
$= 1343$ MPa OK

8.10.2.2 — TRANSMISSION LENGTH

Initial prestress (after stress transfer) $\sigma_{pm0} = 1276$ MPa

Coefficient allowing for tendon type $\eta_{p1} = 3.2$ (for 7-wire strands)

Coefficient allowing for bond condition $\eta_1 = 1.0$ (good bond conditions)

Measured value of concrete compressive strength at transfer $f_{ck} = 32$ MPa

Table 3.1 — Mean value of the concrete tensile strength $f_{ctm} = 3.00$ MPa

Equ. 3.16 — Design tensile strength at stress transfer $f_{ctd}(t) = 1.40$ MPa

Equ. 8.15 — Bond stress $f_{bpt} = \eta_{p1}\eta_1 f_{ctd}(t) = 3.2 \times 1.0 \times 1.52$
$= 4.48$ MPa

Nominal diameter of tendon $\phi = 15.7$ mm

Coefficient allowing for speed of release $\alpha_1 = 1.00$

Coefficient allowing for tendon type $\alpha_2 = 0.19$

Equ. 8.16 — Basic value of transmission length $l_{pt} = 1.00 \times 0.19 \times 15.7 \times 1292/4.48$
$= 849$ mm

Equ. 8.17 — Design value of transmission length (lower) $l_{pt1} = 0.8 \times 992 = 680$ mm

Equ. 8.18 — Design value of transmission length (upper) $l_{pt2} = 1.2 \times 992 = 1019$ mm

CONCRETE BRIDGE DEVELOPMENT GROUP

Project details

CBDG Design Example
Serviceability limit state combinations of actions

Chapter Ref. 9
Clause Ref. 9.1–9.2
Sheet No. A9.1

Table A9.1

Moments			Proportion of total		ψ factor			$M_{AB,MID}$ (kNm)	$M_{AB,TRA}$ (kNm)		M_B (kNm)
			Opening	Long term	ψ_0	ψ_1	ψ_2	Beam 4	Edge beam	Beam 4	Beam 4
Self-weight 1a	Simply supported beam action		-	-	-	-	-	767	86	86	-
Self-weight 1b	Effect of reactions from simply supported beam		-	-	-	-	-	12	28	28	-150
Self-weight 2 (super-imposed dead weight)			-	-	-	-	-	0	-314	0	0
Self-weight 3 (surfacing)			-	-	-	-	-	225	0	-158	-252
Prestressing		SSLBM action	0.79	0.72	-	-	-	-1169	-1169	-1169	-
Differential settlement			0.50	0.50	-	-	-	-137	-409	-409	-441
			0.50	0.50	-	-	-	137	409	409	441
Differential shrinkage			0.78	1.00	-	-	-	17	5	5	-27
Temperature difference		Heating	-	-	0.60	0.60	0.50	91	173	173	183
		Cooling	-	-	0.60	0.60	0.50	-8	-16	-16	-17
Traffic loads	gr1a	LM1 UDL AB	-	-	0.75	0.75	0.00	306	-	-100	-205
		LM1 UDL BC	-	-	0.75	0.75	0.00	-36	-	-89	-96
		LM1 TS AB	-	-	0.75	0.75	0.00	1039	-	-269	-453
		Footways	-	-	0.40	0.40	0.00	0	-	0	0
	gr5	LM3 SV196	-	-	0.00	0.00	0.00	1142	-	-452	-694
	NB! Frequent value →	Co-existing LM1	-	-	0.00	0.00	0.00	409	-	-221	-344
Characterisic combinations	Load case 1	SSLBM (beam only)	-	-	-	-	-	-157	-	-838	0
	gr5, heating, at opening	CGM (beam + slab)	-	-	-	-	-	1924	-	-899	-1572
	Load case 2	SSLBM (beam only)	-	-	-	-	-	-75	-	-756	-
	gr5, heating, long term	CGM (beam + slab)	-	-	-	-	-	1928	-	-899	-
	Load case 3	SSLBM (beam only)	-	-	-	-	-	-157	-	-838	0
	gr5, cooling, at opening	CGM (beam + slab)	-	-	-	-	-	1865	-	-1013	-1692
	Load case 4	SSLBM (beam only)	-	-	-	-	-	-75	-	-756	-
	gr5, cooling, long term	CGM (beam + slab)	-	-	-	-	-	1868	-	-1012	-
Frequent combinations	Load case 6	SSLBM (beam only)	-	-	-	-	-	-75	-	-	-
	gr1a, heating, long term	CGM (beam + slab)	-	-	-	-	-	1376	-	-	-
	Load case 7	SSLBM (beam only)	-	-	-	-	-	-75	-	-	-
	gr1a, cooling, long term	CGM (beam + slab)	-	-	-	-	-	1327	-	-	-
Quasi-permanent combination	Load case 10	SSLBM (beam only)	-	-	-	-	-	-	-838	-	0
	Cooling, at opening	CGM (beam + slab)	-	-	-	-	-	-	-495	-	-652

SSLMB action = Moments resulting from the Simply Supported Line Beam Model
CGM = Moments resulting from the grillage model
$M_{AB,MID}$ = Moment at mid span
$M_{AB,TRA}$ = Moment at end of transmission length at central pier end of span
M_B = Moment over central pier

The appropriate values of $M + f\psi$ in the upper portion of the table are used to obtain the combination results.
e.g. Frequent combination, load case 6:

E_d = $E\,(\Sigma G_{kj} + P + \psi_{1,1}Q_{k,1} + \Sigma\psi_{2,i}Q_{k,i})$
Moment on simply supported precast beam (SSLBM) = 767 + 0.72 x (-1169) = -75
Moment on continuous bridge (CGM) = 12 + 225 + 0.5. x 137 + 1.0 x 17 + 0.5 x 91 + 0.75 x 306 + 0.75 x 1039 + 0.40 x 0 = 1376

Table A9.2

Axial forces			$f(t)$		ψ factor			$N_{AB,MID}$	$N_{AB,TRA}$	N_B
			Opening	Long term	ψ_0	ψ_1	ψ_2	(kN)	(kN)	(kN)
Prestressing (full applied value)		SSLBM (beam only)	0.79	0.72	-	-	-	4693	4693	0
Prestressing (at opening)		SSLBM (beam only)	-	-	-	-	-	3707	3707	0
Prestressing (long term)		SSLBM (beam only)	-	-	-	-	-	3379	3379	0

CONCRETE BRIDGE DEVELOPMENT GROUP

Project details

CBDG Design Example
Decompression check

Chapter Ref. 9
Clause Ref. 9.3
Sheet No. A9.2

DECOMPRESSION CHECK AT MIDSPAN

Table A9.3

		Beam		Slab	
		30 mm below lowest tendon	Top	Bottom	Top
				E_{C35}/E_{C60} =	0.92
Cross-section area (beam only)	mm^2	4.10×10^5	4.10×10^5		
Section modulus (beam only)	mm^3	-1.01×10^8	6.36×10^7		
Cross-section area (beam + slab)	mm^2	6.31×10^5	6.31×10^5	6.31×10^5	6.31×10^5
Section modulus (beam + slab)	mm^3	-1.70×10^8	2.90×10^8	2.90×10^8	2.01×10^8
Midspan - frequent load combination, long term, heating					
Normal force (kN) on SSLBM	3379	8.2	8.2		
Moment (kNm) on SSLBM	-75	0.7	-1.2		
Moment (kNm) on CGM	1376	-8.1	4.7	4.4	6.3
Non-linear temp. diff. component[2]		0.5	-0.3	-0.3	0.7
Diff. shrinkage local component[3]		0.1	0.1	-0.2	-0.2
Total Normal Stress (MPa)		**1.5**	**11.6**	**3.8**	**6.8**
Midspan - frequent load combination, long term, cooling					
Normal force (kN) on SSLBM	3379	8.2	8.2		
Moment (kNm) on SSLBM	-75	0.7	-1.2		
Moment (kNm) on CGM	1327	-7.8	4.6	4.2	6.1
Non-linear temp. diff. component		-0.7	0.1	0.1	-0.5
Diff. shrinkage local component		0.1	0.1	-0.2	-0.2
Total Normal Stress (MPa)		**0.6**	**11.9**	**4.1**	**5.4**

Notes:

1. Compression is positive. Sagging moments positive
2. Values from Table A6.6, multiplied by appropriate value of ψ_2
3. This is the component due to the axial force resulting from the shrinkage. Value calculated by dividing the force calculated in section 6 by the cross-sectional area of the member. The component due to the resulting moment is included in the results of the grillage analysis.

All locations remain in compression - OK

CONCRETE BRIDGE DEVELOPMENT GROUP

Project details

CBDG Design Example
Stress limitation

Chapter Ref. 9
Clause Ref. 9.4
Sheet No. A9.3

BS EN 1992-1-1
7.2

MID SPAN STRESS

Table A9.4

		Beam		Slab	Tendon
		Bottom	Top	Top	Bottom
				E_{C35}/E_{C50} = 0.92	E_p/E_{C50} = 5.27
Cross-section area (beam only)	mm^2	4.10×10^5	4.10×10^5	-	4.10×10^5
Section modulus (beam only)	mm^3	-9.55×10^7	6.36×10^7	-	-1.12×10^8
Cross-section area (beam + slab)	mm^2	6.31×10^5	6.31×10^5	6.31×10^5	6.31×10^5
Section modulus (beam + slab)	mm^3	-1.65×10^8	2.90×10^8	2.01×10^8	-1.82×10^8
Limiting stress		**-4.1**	**30**	**21**	**-1395**
Characteristic load combination, at opening, heating					
Initial prestress		-	-	-	-1163
Normal force on SSLBM	3707	9.0	9.0	0.0	-
Prestressing moment on SSLBM	-924	9.7	-14.5	0.0	-
Dead load moment on SSLBM	767	-8.0	12.1	0.0	-36
Moment on CGM	1924	-11.7	6.6	8.8	-56
Non-linear temp. diff. component[2]	-	0.5	-0.4	0.9	-
Diff. shrinkage local component[3]	-	0.1	0.1	-0.2	-
Total Normal Stress (MPa)	-	**-0.4**	**12.9**	**9.5**	**-1255**
Characteristic load combination, at opening, cooling					
Initial prestress		-	-	-	-1163
Normal force on SSLBM	3707	9.0	9.0	0.0	-
Prestressing moment on SSLBM	-924	9.7	-14.5	0.0	-
Dead load moment on SSLBM	767	-8.0	12.1	0.0	-36
Moment on CGM	1865	-11.3	6.4	8.5	-54
Non-linear temp. diff. component	-	-0.8	0.1	-0.6	-
Diff. shrinkage local component	-	0.1	0.1	-0.2	-
Total Normal Stress (MPa)	**-**	**-1.3**	**13.2**	**7.7**	**-1253**
Characteristic load combination, long term, heating					
Normal force (kN) on SSLBM	3379	8.2	8.2	0.0	
Total moment (kNm) on SSLBM	-75	0.8	-1.2	0.0	
Moment (kNm) on CGM	1928	-11.7	6.7	8.8	
Non-linear temp. diff. component	-	0.5	-0.4	0.9	
Diff. shrinkage local component	-	0.1	0.1	-0.2	
Total Normal Stress (MPa)	**-**	**-2.0**	**13.4**	**9.5**	
Characteristic load combination, long term, cooling					
Normal force (kN) on SSLBM	3379	8.2	8.2	0.0	
Total moment (kNm) on SSLBM	-75	0.8	-1.2	0.0	
Moment (kNm) on CGM	1868	-11.3	6.4	8.5	
Non-linear temp. diff. component	-	-0.8	0.1	-0.6	
Diff. shrinkage local component	-	0.1	0.1	-0.2	
Total Normal Stress (MPa)	**-**	**-3.0**	**13.7**	**7.7**	

Notes:

1. Compression is positive. Sagging moments positive
2. Values from Table A6.6, multiplied by appropriate value of ψ_0
3. See note 3 on previous sheet

All stresses less than limiting value – OK

CONCRETE BRIDGE DEVELOPMENT GROUP

Project details

CBDG Design Example
Stress limitation

Chapter Ref. 9
Clause Ref. 9.4
Sheet No. A9.4

DIAPHRAGM OVER PIER

- The stress is checked at the bottom fibre of the beam and topmost reinforcement bar position.
- The stresses are calculated using cracked section analysis.

Table A9.5

	Normal stress (MPa)	
	Concrete	Reinforcement
Limiting stress	21.0	-400
Characteristic load combination, at opening, heating		
From SSLBM action	0.0	0.0
From CGM action	11.6	-356
Non-linear temp. diff. component	0.5	-
Diff. shrinkage local component	0.0	-
Total	**12.1**	**-356**
Characteristic load combination, at opening, cooling		
From SSLBM action	0.0	0.0
From CGM action	12.5	-385
Non-linear temp. diff. component	-0.8	-
Diff. shrinkage local component	0.0	-
Total	**11.7**	**-385**
All stresses less than limiting value – OK		

END OF DIAPHRAGM

- The stress is checked at the bottom fibre of the beam and topmost reinforcement bar position.
- The stresses are calculated using cracked sections. To follow the effects of the phased construction using cracked section analysis is not practical by manual calculation. The calculation may be carried out using a suitable design software with a capability of entering an initial stress field. SAM[3] was used to calculate the values in this example.
- The effect of the local component of the differential shrinkage is neglected for the reinforcement.

Table A9.6

	Normal stress (MPa)	
	Beam	Reinforcement
Limiting stress	30	-400
Characteristic load combination, at opening, heating		
From SSLBM and CGM action	27.3	-163
Non-linear temp. diff. component	0.5	-
Diff. shrinkage local component	0.1	-
Total	**27.9**	**-163**
Characteristic load combination, at opening, cooling		
From SSLBM and CGM action	28.5	-187
Non-linear temp. diff. component	-0.8	-
Diff. shrinkage local component	0.1	-
Total	**27.8**	**-187**
All stresses less than limiting value – OK		

CONCRETE BRIDGE DEVELOPMENT GROUP	Project details CBDG Design Example Crack widths	Chapter Ref.	9
		Clause Ref.	9.5
		Sheet No.	A9.5

BS EN 1992-1-1

DIAPHRAGM OVER PIER

The following is calculated for a 1500 mm width.

SLS quasi-permanent flexural moment M = -652 kNm

Note: Self-equilibrating stresses may be ignored (BS EN 1992-2 N.A, Table NA.1)

Reinforcement in slab Top row: B20/150, Bottom row: B12/150

7.3.4(2) Difference between the mean strain in the reinforcement and the concrete

Modulus of elasticity of steel E_s = 200000 MPa

3.1.3 Secant modulus of elasticity of concrete E_{cm} = 34000 MPa

$\alpha_e = E_s/E_{cm}$ α_e = 5.9

Area of reinforcing steel (10no. B20 & 10no. B12) A_s = 4273 mm^2

7.3.2(3) & Fig. 7.1 Effective depth of concrete in tension $h_{c,ef}$ = min {2.5(h - d); (h - x)/3; h/2}

= min {2.5 x (1160 - 1081); (1160 - 175)/3; 1160/2} = 198 mm

Effective area of concrete in tension $A_{c,eff}$ = 297000 mm^2

7.3.4(2) Effective reinforcement ratio $\rho_{p,eff}$ = $A_s/A_{c,eff}$ = 0.0144

Factor dependent on the duration of the load k_t = 0.4

Mean value of the concrete tensile strength f_{ctm} = 3.20 MPa

Stress in the tension reinforcement assuming cracked section (calculated using SAM) σ_s = 149 MPa

Equ. 7.9 Mean strain in reinforcement minus mean strain in concrete:

$$\varepsilon_{sm} - \varepsilon_{cm} = [\sigma_s - k_t\,(f_{ct,eff}/\rho_{p,eff})(1 + \alpha_e\rho_{p,eff})]/E_s \geq 0.6\sigma_s/E_s = 447 \text{ microstrain}$$

7.3.4(3) Maximum crack spacing

Factor allowing for reinforcement bond properties k_1 = 0.8 for high bond bars

Factor allowing for strain distribution k_2 = 0.5 for bending

k_3 = 3.4

k_4 = 0.425

Equ. 7.12 Equivalent bar diameter ϕ_{eq} = $(20^2 + 12^2)/(20 + 12)$

= 17.0 mm

Concrete cover c = 45 mm

Equ. 7.11 Maximum crack spacing

$$s_{r,max} = k_3c + k_1k_2k_4\phi/\rho_{p,eff} = 3.4 \times 45 + 0.8 \times 0.5 \times 0.425 \times 20/0.0144$$

= 354 mm

7.3.4(1) Crack width

Equ. 7.8 Calculated crack width w_k = $s_{r,max}\,(\varepsilon_{sm} - \varepsilon_{cm})$ = $389 \times 456 \times 10^{-6}$

= 0.16 mm

The limiting crack width, w_{max}, is 0.3 mm. $0.16 < 0.3$ ∴OK

Calculation of the crack widths at the end of the diaphragm are similar to those above, once allowance has been made for the different concrete cross-section (beam and slab, rather than solid diaphragm). The quasi-permanent flexural moment leads to a calculated crack width of 0.11 mm for the top face of the slab and 0.10 mm for the soffit.

CONCRETE BRIDGE DEVELOPMENT GROUP	Project details CBDG Design Example Transfer	Chapter Ref.	9
		Clause Ref.	9.6
		Sheet No.	A9.6

BS EN 1992-1-1	PRESTRESSING FORCES AT TRANSFER		
Values from sheet A8.1 adjusted to allow for prestress loss	Normal force from prestressing	N	= 4693 x 91% = 4271 kN
	Bending moment, M, at end of transfer zone		= Prestressing moment + dead load moment
			= -1169 kNm x 91% + 66 kNm = -998 kNm
	CRACK CONTROL		
	Crack control will be provided by two 20 mm diameter bars positioned 100 mm below top face of beam.		
7.3.4(2)	Difference between the mean strain in the reinforcement and the concrete		
	Equivalent concrete age at transfer of prestress	$t_{eq,p}$	= 1 day
	Modulus of elasticity of steel	E_s	= 200000 MPa
	Measured value of concrete compressive strength at transfer	f_{ck}	= 32.00 MPa
3.1.3	Secant modulus of elasticity of concrete at transfer	E_{cm}	= 32400 MPa
	$\alpha_e = E_s/E_{cm}$	α_e	= 6.2
	Area of reinforcing steel (2 x 12 mm diameter bars)	A_s	= 226 mm^2
7.3.2(3) & Fig. 7.1	Effective depth of concrete in tension	$h_{c,ef}$	= 106 mm
	Effective area of concrete in tension	$A_{c,eff}$	= 34269 mm^2
7.3.4(2)	Effective reinforcement ratio	$\rho_{p,eff}$	= 0.0066
	Factor dependent on the duration of the load	k_t	= 0.4
Table 3.1	Mean value of the concrete tensile strength	f_{ctm}	= 3.00 MPa
	Stress in the tension reinforcement assuming cracked section*	σ_s	= 57 MPa
Equ. 7.9	Mean strain in reinforcement minus mean strain in concrete	$\varepsilon_{sm} - \varepsilon_{cm}$	= 171 με
7.3.4(3)	Maximum crack spacing		
	Factor allowing for reinforcement bond properties	k_1	= 0.8
	Factor allowing for strain distribution	k_2	= 0.5
		k_3	= 3.4
		k_4	= 0.425
	Bar diameter	ϕ	= 12.0 mm
	Concrete cover	c	= 90 mm
Equ. 7.11	Maximum crack spacing	$s_{r,max}$	= 615 mm
7.3.4(1)	Crack width		
Equ. 7.8	Calculated crack width	w_k	= $s_{r,max}(\varepsilon_{sm} - \varepsilon_{cm})$ = 0.11 mm
	The limiting crack width, w_{max}, is 0.2 mm.		
	CONCRETE STRESS AT TRANSFER		
3.1.2	Concrete strength at transfer of prestress	$f_{ck}(t)$	= 32 MPa
5.10.2.2	Permitted concrete stress at transfer	0.7 $f_{ck}(t)$	= 22.4 MPa
	Peak concrete stress at transfer of prestress*	σ_c	= 22.3 MPa
		22.3 < 22.4 ∴OK	

*From cracked section analysis, calculated using SAM

cbdg CONCRETE BRIDGE DEVELOPMENT GROUP	Project details CBDG Design Example ULS moments and ULS shear	Chapter Ref. 10 Clause Ref. 10.1 Sheet No. A10.1

ULS MOMENTS

Table A10.1

			Partial factors		$M_{AB,MID}$ (kNm)	$M_{AB,DIA}$ (kNm)	M_B (kNm)
			γ_{sup}	γ_{inf}			
Self-weight 1a		Simply supported beam action	1.35	0.95	767	86	0
Self-weight 1b		Effect of reactions from simply supported beams	1.35	0.95	12	28	-150
Self-weight 2 (super-imposed dead weight)			1.20	0.95	0	0	0
Self-weight 3 (surfacing)			1.20	0.95	225	-158	-252
Traffic loads	gr1a	LM1 UDL AB	1.35	0.00	306	-100	-205
		LM1 UDL BC	1.35	0.00	-36	-89	-96
		LM1 TS AB	1.35	0.00	1039	-269	-453
		Footways	1.35	0.00	0	0	0
	gr5	LM3 SV196 No.1.	1.35	0.00	1142	-452	-694
	NB! Frequent value →	Co-existing LM1	1.35	0.00	409	-221	-344
ULS loadcase combination					**3416**	**-990**	**-1906**

e.g. $M_{AB,MID} = 1.35 \times 767 + 1.35 \times 12 + 1.20 \times 0 + 1.20 \times 225 + 1.35 \times 1142 + 1.35 \times 409 = 3416$

ULS SHEAR

Table A10.2

			Partial factors		V_A (kN)	$V_{AB,DIA}$ (kN)	$V_{AB,DIA + 3m}$ (kN)	V_B (kN)
			γ_{sup}	γ_{inf}				
Self-weight 1		SSLBM action	1.35	0.95	167	167	115	0
		CGM action (reactions from SSLBM)	1.35	0.95	0	0	0	215
Self-weight 2 (super-imposed dead weight)			1.20	0.95	0	0	0	0
Self-weight 3 (surfacing)			1.20	0.95	59	73	54	82
Traffic loads	gr1a	LM1 UDL AB	1.35	0.00	76	83		93
		LM1 UDL BC	1.35	0.00	0	0		0
		LM1 TS AB	1.35	0.00	116	153		153
		Footways	1.35	0.00	0	0		0
	gr5	LM3 SV196	1.35	0.00	390	391	168	464
	NB! Frequent value →	Co-existing LM1	1.35	0.00	62	41	32	134
ULS loadcase combination		**CGM (beam + slab)**			**906**	**896**	**490**	**1196**

CONCRETE BRIDGE DEVELOPMENT GROUP

Project details	Chapter Ref. 10 Clause Ref. 10.2 Sheet No. A10.2
CBDG Design Example ULS bending check	

BS EN 1992-1-1 6.1

DIAPHRAGM OVER PIER

From sheet A10.1 — Design flexural moment M_{Ed} = -1906 kNm per 1.5 m width of bridge

Use rectangular stress block (3.1.7)

3.2.7 — Steel force at design ultimate yield $F_s = A_s f_{yk}/\gamma_S = 500A_s/1.15$

3.1.7 and 3.1.6 — Depth of concrete compression block $\lambda x = F_s/(b\eta f_{cd}) = F_s/(b\eta \alpha_{CC} f_{ck}/\gamma_C)$

$= F_s/(1500 \times 1.0 \times 0.85 \times 35/1.5)$

Lever arm $z = d - (\lambda x/2) = d - F_s/(2b\eta f_{cd})$

Assume centroid of reinforcement is at mid depth of in-situ slab:

Effective depth $d = 1160 - 160/2 = 1080$ mm

From the above equations : $M_{Rd} = F_s z = F_s d - F_s^2/(2b\eta f_{cd}) \geq M_{Ed}$

$1080F_s - F_s^2/(2 \times 1500 \times 1.0 \times 0.85 \times 35/1.5) \geq 1906 \times 10^6$

Which gives a required steel force of: $F_s \geq 1816$ kN

Required area of steel $A_s = 1816000 \times 1.15/500$

$= 4177$ mm^2

(Note: d_c is 61 mm, steel clearly yields)

This area of reinforcement can be achieved using B20 bars at 150 centres in the top layer of the slab and B12 bars at 150 mm centres in the lower layer:

Total area $= \{(20/2)^2/150 + (12/2)^2/150\} \times \pi \times 1500$

$= 4273$ mm^2

END OF DIAPHRAGM

Design flexural moment M_{Ed} = -990 kNm

Method is as above, except the width of the concrete compression zone, *b*, is the width of the base of the Y4 beam (750 mm) and the concrete in compression is class C50/60. This gives:

Steel force at design ultimate yield $F_s \geq 936$ kN

Required area of steel $A_s = 936000 \times 1.15/500$

$= 2153$ mm^2 $<$ 4273 mm^2

	Project details	
CONCRETE BRIDGE DEVELOPMENT GROUP	CBDG Design Example ULS bending check	Chapter Ref. 10 Clause Ref. 10.2 Sheet No. A10.3

BS EN 1992-1-1

MID-SPAN

Design flexural moment M_{Ed} = 3416 kNm

3.3.6(7) Use tendon stress-strain model with horizontal top branch

Total tendon force at yield F_P = $A_p f_{p0.1k}/\gamma_S$ = 22 x 150 x 1580/1.15
= 4534 kN

Use rectangular concrete stress block. Assume tendons beyond yield then check.

Depth of concrete compression block λx = 4534000/(1500 x 0.85 x 35/1.5)
= 152 mm (i.e. within in-situ slab)

Depth to highest tendon d = 1160 - 260 = 900 mm

Concrete strain at level of highest tendons = ε_{cu3} (d - x)/x
= 0.0035 (900 - 152/0.8)/(152/0.8)
= 0.013

Prestrain in tendons = $\sigma_{p,eff}\ \gamma_{P,fav}/E_P$ = 1019 x 0.9/195000
= 0.005

Minimum strain in tendons = 0.013 + 0.005 = 0.018

Yield strain of tendons = $(f_{p0.1k}/\gamma_S)/E_p$ = (1580/1.15)/195000
= 0.007 ∴ all tendons yield

If the tendons were not yielding, the stress in the tendons at failure of the section should be calculated by strain compatibility.

Effective depth to centroid of tendons d = 1160 - 151 = 1009 mm

Moment capacity M_{Rd} = F_P (d - λx/2)
= 4534000 (1009 - 152/2)
= 4230 kNm

M_{Rd} = 4230 kNm > M_{Ed} = 3416 kNm OK

CONCRETE BRIDGE DEVELOPMENT GROUP

Project details CBDG Design Example Minimum reinforcement requirements	Chapter Ref.	10
	Clause Ref.	10.2.1
	Sheet No.	A10.4

BS EN 1992-1-1

MINIMUM REINFORCEMENT REQUIREMENTS

Reinforcement in slab

Minimum area of steel according to BS EN 1992-1-1, 9.3.1.1(1)

f_{ctm} = 3.2 MPa

f_{yk} = 500 MPa

b_t = 1500 mm

d = 1080 mm

Equ. 9.1N

$A_{s,min}$ = $0.26\ (f_{ctm}/f_{yk})\ b_t d$

but not less than $0.0013 b_t d$

$A_{s,min}$ = $0.26 \times (3.2/500) \times 1500 \times 1080$

= $2696\ \text{mm}^2$

This is less than that provided near the supports. Away from the supports the in-situ slab is always in compression and so this requirement does not apply.

Prestressing tendons

BS EN 1992-2 & NA, 6.1(109)

Tensile strength of concrete f_{ctx} = $f_{ctk,0.05}$

= 2.9 MPa

Cracking bending moment, ignoring the effect of prestressing M_{rep} = $\dfrac{f_{ctx} I}{y}$

= 477 kNm

BS EN 1992-2, 6.9(110) and Equ. 6.101a

Actual area of reinforcement, including tendons A_s = $3300\ \text{mm}^2$

Lever arm at ULS, based on steel area A_s z_s = 951 mm

Minimum area of reinforcement $A_{s,min}$ = $\dfrac{M_{rep}}{z_s\ f_{p0.1k}}$

= $317\ \text{mm}^2$

$A_s >> A_{s,min}$ Therefore, the requirement of clause 5.10.1(5(P)) that brittle failure of the member due to failure of prestressing tendons can be considered to have been avoided.

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CONCRETE BRIDGE DEVELOPMENT GROUP

Project details
CBDG Design Example
Continuity reinforcement

Chapter Ref. 10
Clause Ref. 10.2.2
Sheet No. A10.5

BS EN 1992-2

Continuity reinforcement over pier
According to BS EN 15050 D.4 and BS EN 1992-2, 7.3.2

Area of concrete beam in tension before formation of first crack, A_{ct} = area below centroid of uncracked composite section
= 2.72×10^5 mm^2

Max value of permitted stress in reinf. immediately after cracking, σ_s = 240 MPa
(This value is taken from Table 7.2N for a crack width limit of 0.3 mm (BS EN 1992-2 NA, Table NA.1) and assuming 16 mm diameter bars are used)

k = 0.65
Equ. 7.2 k_c = 0.4 (as $\sigma_c = 0$)

Equ. 7.1 Minimum amount of bottom continuity steel $A_{s,min}$ = $k_c\, k\, f_{ctm}\, A_{ct}/\sigma_s$
= $0.4 \times 0.65 \times 3.2 \times 2.72 \times 10^5/240$
= 942 mm^2

5 No. B16 bars are required in the bottom flange to provide adequate continuity steel.

CONCRETE BRIDGE DEVELOPMENT GROUP

Project details

CBDG Design Example
ULS shear check

Chapter Ref. 10
Clause Ref. 10.3, 10.3.1
Sheet No. A10.6

END OF DIAPHRAGM AT PIER B

From sheet A10.1 — Design shear force, V_{Ed} = 896 kN

Co-existing moment, M_{Ed} = -990 kNm

Co-existing normal force, N_{Ed} = (distance from end of beam/transmission length, l_{pt2}) x $\gamma_{P,fav}$ x long term prestress force

(allowing for reduced prestress value within transmission region

= (600/1274) x 0.90 x (0.72 x 4693)

= 1432 kN

Tension reinforcement: B20 at 150 mm centres in top of slab,
B12 at 150 mm centres in bottom

Shear reinforcement: B12 links, spacing to be decided

Section parameters

Smallest width of cross-section in tensile area b_w = 216 mm

Effective depth of reinforcement d = 1080 mm

Area of tensile reinforcement A_{sl} = 615 mm^2

Cross-sectional area of shear reinforcement A_{sw} = 226 mm^2

BS EN 1992-2, 6.2.2 — Shear capacity without design shear reinforcement

$C_{Rd,c} = 0.18/\gamma_C = 0.12$

Allowance for size effect $k = 1 + (200/d)^{0.5} \leq 2.0$

$= 1 + (200/1080)^{0.5} = 1.43$

Reinforcement ratio $\rho_l = A_{sl}/(b_w d) \leq 0.02$

$= 615/(216 \times 1080) = 0.003$

$k_1 = 0.15$

Equ. 6.3N — Minimum shear strength v_{min} = 0.42 MPa

Equ. 3.15 — Design compressive strength $f_{cd} = 50 \times 1.0/1.5 = 33$ MPa

Mean compressive stress $\sigma_{cp} = N_{Ed}/A_c < 0.2\, f_{cd}$

$= 1432 \times 10^3/410191 = 3.5$ MPa

Equ. 6.2 — Shear resistance $V_{Rd,c} = [C_{Rd,c}k(100\rho_l f_{ck})^{1/3} + k_1\sigma_{cp}]\, b_w d$

$= [0.12 \times 1.43 \times (100 \times 0.003 \times 50)^{1/3} + 0.15 \times 3.5] \times 216 \times 1080$

= 221 kN

$V_{Rd,c} < V_{Ed}$. Therefore shear reinforcement is required

BS EN 1992-2, 6.2.3 — Shear capacity with calculated shear reinforcement

Find the smallest angle of the concrete compressive struts before concrete crushing occurs using equation 6.9:

Equ. 6.9 — $V_{Rd,max} = 0.5\alpha_{cw} b_w z v_1 f_{cd} \sin 2\theta \geq V_{Ed}$

Rearrange to give concrete strut angle $\theta = 0.5\sin^{-1}\left(\dfrac{2V_{Ed}}{\alpha_{cw} b_w z v_1 f_{cd}}\right)$

where: Allowance for state of stress in compression chord, α_{cw}

$= 1 + \sigma_{cp}/f_{cd}$ for $0 < \sigma_{cp} \leq 0.25 f_{cd}$

$= 1 + 2.3/33 = 1.07$

CONCRETE BRIDGE DEVELOPMENT GROUP

Project details

CBDG Design Example
ULS shear check

Chapter Ref. 10
Clause Ref. 10.3, 10.3.1
Sheet No. A10.7

BS EN 1992-1-1 Inner lever arm* $z = 936$ mm

Equ. 6.6N Reduction factor for concrete cracked in shear $\nu_1 = 0.6(1 - f_{ck}/250) = 0.6(1 - 50/250)$
$= 0.48$

* From cracked section analysis, calculated using SAM

Giving: $\theta = 0.5\sin^{-1}[2 \times 896 \times 10^3/(1.07 \times 216 \times 936 \times 0.48 \times 33)]$
$= 16°$

However, the angle of the concrete strut is limited to $1 \leq \cot\theta \leq 2.5$, or $22° \leq \theta \leq 45°$. If $\theta > 45°$, then the maximum shear capacity of the section has been exceeded, and either the depth of the beam or the concrete class should be increased. If $\theta < 22°$ then assume $\theta = 22°$ for the calculation of the shear reinforcement.

Rearrange equation 6.8 to obtain the required area of reinforcement:

Equ. 6.8 $A_{sw}/s = V_{Ed}\tan\theta/(zf_{ywd})$
$= 896000 \times \tan 22°/(936 \times 435)$
$= 0.89$ mm²/mm

Giving maximum spacing for the links of: $s = 226/0.89 = 254.00$ mm

Therefore, place links at 250 mm centres at ends of beams (unless interface shear proves more critical). This spacing could be reduced further from the supports, where the shear force is less.

9.2.2 Required minimum amount of shear reinforcement

Equ. 9.5 Minimum shear reinforcement ratio $\rho_{w,min} = 0.08 \times f_{ck}^{0.5}/f_{yk} = 0.08 \times 50^{0.5}/500$
$= 0.00113$

Equ. 9.6 Maximum spacing of shear links $s_{l,max} = 0.75d\,(1 + \cot\alpha) = 0.75 \times 1080 \times (1 + \cot 90°)$
$= 810 \text{ mm} > 225 \text{ mm}$

Equ. 9.4 Minimum area of links for a spacing of 225 mm
$A_{sw,min} = \rho_{w,min} s b_w \sin\alpha = 0.00113 \times 225 \times 216 \times \sin 90°$
$= 55 \text{ mm}^2 < 226 \text{ mm}^2$

DIAPHRAGM AT PIER B

Design shear force, V_{Ed} = 1196 kN
Co-existing normal force, N_{Ed} = 0 kN
Tension reinforcement: B20 at 150 mm centres in top of slab,
B12 at 150 mm centres in bottom
Shear reinforcement: 4 x B12 legs, spacing to be decided

Section parameters

Smallest width of cross-section in tensile area $b_w = 750$ mm
Effective depth of reinforcement $d = 1092$ mm
Area of concrete cross-section $A_c = 870000$ mm²
Area of tensile reinforcement $A_{sl} = 2136$ mm²
Cross-sectional area of shear reinforcement $A_{sw} = 452$ mm²

CONCRETE BRIDGE DEVELOPMENT GROUP

Project details

CBDG Design Example
ULS shear check

Chapter Ref. 10
Clause Ref. 10.3, 10.3.1
Sheet No. A10.8

BS EN 1992-2, 6.2.3

Shear capacity with calculated shear reinforcement

From equation 6.9: $\theta = 0.5\sin^{-1}\left(\dfrac{2V_{Ed}}{\alpha_{cw}\, b_w\, z v_1\, f_{cd}}\right)$

where:

Allowance for state of stress $\alpha_{cw} = 1.00$ (for non-prestressed elements)

Inner lever arm $z \approx 0.9d$ (when no axial force)

$= 983$ mm

Equ. 6.6N

Reduction factor for concrete cracked in shear $v_1 = 0.6[1 - 35/250]$

$= 0.52$

Giving: $\theta = 0.5\sin^{-1}[2 \times 1196 \times 10^3/(1.00 \times 750 \times 983 \times 0.52 \times 23)]$

$= 8°$

Take the minimum permissible value of θ of 22°

From equation 6.8: $A_{sw}/s = V_{Ed}\tan\theta/(zf_{ywd})$

$= 1.13$ mm^2/mm

Maximum spacing of links $s = A_{sw}/1.13 = 401$ mm

Therefore, place links at 400 mm centres within the diaphragm.

9.2.2

Required minimum amount of shear reinforcement

Equ. 9.5 — Minimum shear reinforcement ratio $\rho_{w,min} = 0.00095$

Equ. 9.6 — Maximum spacing of shear links $s_{l,max} = 819$ mm

Equ. 9.4 — Minimum area of links for a spacing of 400 mm $A_{sw,min} = 285$ mm^2

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Project details

CBDG Design Example
ULS shear check

Chapter Ref. 10
Clause Ref. 10.3, 10.3.2
Sheet No. A10.9

SHEAR CHECK BETWEEN WEB AND FLANGES AT END OF DIAPHRAGM AT PIER B

Design shear force, V_{Ed}: 693 kN
(Average shear force within a 3 m long section)

BS EN 1992-2,
6.2.4(103)

Design value of the shear stress

Total width of flange b = 1500 mm
Total width of web at junction with flange b_w = 285 mm
Thickness of flange at junction with web h_f = 160 mm
Lever arm of composite section z = $0.9d$ = 983 mm

Total shear force transmitted from web to flange = V_{Ed}/z = 693000/983
= 705 N/mm length

Proportion of shear force remaining within width of web = b_w/b = 0.19
∴ proportion transferred to flange across interface on either side of web = $(1-b_w/b)/2$
= 0.41

Longitudinal shear stress across junction = $(V_{Ed}/z) \times (1-b_w/b)/2 \times (1/h_f)$
= 705 x 0.41/160
= 1.81 MPa

BS EN 1992-1-1
6.2.4(6)

Permissible shear stress = $0.4f_{ctd}$ = $0.4\,\alpha_{ct}\,f_{ctk,0.05}/\gamma_C$
= 0.4 x 1.0 x 2.2/1.5 = 0.59 MPa

The longitudinal shear stress exceeds the permissible value, and so transverse reinforcement is required.

BS EN 1992-1-1
Equ. 6.6

Required amount of transverse reinforcement

Strength reduction factor for concrete cracked in shear ν = 0.51

Rearrange equation 6.22 to find minimum angle of compression strut, θ_f, that avoids concrete crushing:

$$\theta_f = \frac{1}{2}\sin^{-1}\left(\frac{2v_{Ed}}{\nu f_{cd}}\right) \qquad \text{but } 26.5^\circ \leq \theta_f \leq 45^\circ$$

$$= 0.5\sin^{-1}(2 \times 1.81/0.51 \times 23) \text{ or } 26.5^\circ = 26.5^\circ$$

Equ. 6.21

Transverse reinforcement per unit length A_{sf}/s_f = $v_{Ed}h_f/f_{yd}\cot\theta_f$
= 1.81 x 160/[(500/1.15) cot 26.5]
= 332 mm^2/m

6.2.4(5)

As the slab will also be subject to tansverse bending (see section 11) the area of transverse steel should equal the greater of 332 mm^2/m or half this value plus the area required to resist transverse bending.

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Project details

CBDG Design Example
ULS shear check at interface

Chapter Ref. 10
Clause Ref. 10.3, 10.3.3
Sheet No. A10.10

BS EN 1992-1-1

INTERFACE SHEAR AT END OF DIAPHRAGM AT PIER B

Design shear force, V_{Ed}: 693 kN (Average over 3 m length)

Shear reinforcement: B12 links

6.2.5(1)

Design value of the shear stress at the interface

Interface width b_i = 285 mm

Effective depth of section d = 1080 mm

Inner lever arm of composite section z = $0.9d$
= 972 mm

Proportion of longitudinal tensile force carried in top slab, β = 1.00

Equ. 6.24

Design value of interface shear stress v_{Edi} = $\beta V_{Ed}/(z\, b_i)$
= $1.0 \times 693 \times 10^3/(972 \times 285)$
= 2.50 MPa

6.2.5(1)

Design shear resistance at the interface

Area of the interface A_i = 285000 mm^2/m

Angle of reinforcement to interface α = 90 degree

Stress resulting from forces normal to interface σ_n = 0.0 MPa

6.2.5(2)

Factors dependent on interface roughness $\begin{cases} c = 0.25 \\ \mu = 0.6 \end{cases}$

BS EN 1992-2, 6.2.5(105)

However, BS EN 1992-2 requires c to be taken as zero for fatigue and dynamic verification

$f_{cd} = \alpha_{cc} f_{ck}/\gamma_C$ = 23 MPa

$f_{ctd} = \alpha_{ct} f_{ctk,0.05}/\gamma_C$ = 1.5 MPa

Strength reduction factor $\nu = 0.6\,(1 - f_{ck}/250)$ = 0.52

$f_{yd} = f_{yk}/\gamma_S$ = 435 MPa

Equ. 6.25

Maximum value of interface stress $v_{Rdi,max} = 0.5\,\nu f_{cd}$ = 6.0 MPa

$v_{Rdi,max} > v_{Edi}$ OK

Equ. 6.25

Design shear resistance of interface $v_{Rdi} = c f_{ctd} + \mu\,\sigma_n + \rho\, f_{yd}\,(\mu \sin\alpha + \cos\alpha)$

$\geq v_{Edi}$

Rearrange to give minimum reinforcement ratio

$$\rho = \frac{v_{Edi} - c f_{ctd} - \mu\sigma_n}{f_{yd}\,(\mu \sin\alpha + \cos\alpha)}$$

= 0.010

Maximum spacing of links s = $(2 \times 6^2 \times \pi) \times 1000/\rho A_i$ = 83 mm

So space links at 75 mm centres near supports. The amount of reinforcement can be reduced in steps away from the support, as the shear force reduces. The amount of reinforcement required in each region can be calculated using the method outlined above, and results in the following possible arrangement:

Table 10.3

Shear force range (kN)	Link diameter	Link spacing (mm)
380-693	B12	75
190-380	B12	150
Less than 190	B12	300

CONCRETE BRIDGE DEVELOPMENT GROUP

Project details

CBDG Design Example
Fatigue

Chapter Ref. 10
Clause Ref. 10.4
Sheet No. A10.11

	FATIGUE IN LONGITUDINAL REINFORCEMENT OVER PIER
	Stress range check
	Moment range due to Frequent Live Load = $\psi_1 \times (205 + 96 + 453)$ = 0.75 x 754
BS EN 1992-1-1	(values from sheet A9.1) = 566 kNm
6.8.6	
	Resulting reinforcement stress range
	(from a cracked section analysis using SAM) = 128 MPa
	Limit in BS EN 1992-1-1 is 70 MPa. This is increased to 85 MPa in PD 6687-2:2008. Therefore approximately 50% more reinforcement is required over the pier if a full fatigue check is to be avoided.
BS EN 1992-1-1	Damage equivalent stress range check
6.8.5 &	Moment range caused by fatigue load model 3 = 429 kNm
BS EN 1992-2	Resulting reinforcement stress range
Annex NN	(from a cracked section analysis) $\Delta\sigma_{s,Ec}$ = 96 MPa
Fig. NN.1	Factor allowing for element type
	(for a 40 m long influence line) $\lambda_{s,1}$ = 1.02
	Number of lorries per year according to BS EN 1991-2, Table 4.5 N_{obs} = 1.0 million
Table 6.3N	k_2 = 9
Table NN.1	Q = 1.00
Equ. NN.103	Factor allowing for traffic volume $\lambda_{s,2} = Q\,(N_{obs}/2.0)^{1/k_2}$ = 0.93
	Design life of bridge N_{Years} = 120
Equ. NN 104	Factor allowing for design life $\lambda_{s,3} = (N_{years}/100)^{1/k_2}$ = 1.02
Equ. NN 105	Factor applied when element is loaded by more than one lane $\lambda_{s,4} = (\Sigma N_{obs,i}/N_{obs,1})^{1/k_2}$
	$= (2/1)^{1/9}$ = 1.08
BS EN 1991-2	Damage equivalent impact factor
Annex B	(controlled by surface roughness) $\varphi_{fat} = 1.4$
	Damage equivalent correction factor $\lambda_s = \varphi_{fat}\,\lambda_{s,1}\,\lambda_{s,2}\,\lambda_{s,3}\,\lambda_{s,4}$
	$= 1.46$
2.4.2.3	Partial factor on fatigue load $\gamma_{F,fat} = 1.00$
	Damage equivalent stress range $\Delta\sigma_{s,equ} = \Delta\sigma_{s,Ec}\,\lambda_s\,\gamma_{F,fat}$ = 96 x 1.46 x 1.00
	= 140 MPa
Table 6.3N	Resisting stress range at N* cycles $\Delta\sigma_{Rsk}(N^*)$ = 162.5 MPa
2.4.2.4	Partial factor for reinforcing steel under fatigue load $\gamma_{S,fat}$ = 1.15
	Permissible stress range = $\Delta\sigma_{Rsk}(N^*)/\gamma_{S,fat}$ = 141 MPa
	The permissible stress range is greater than the damage equivalent stress range. Therefore, adequate reinforcement has been provided over the pier for fatigue resistance.

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Project details

CBDG Design Example
Fatigue

Chapter Ref. 10
Clause Ref. 10.4
Sheet No. A10.12

BS EN 1991-2

FATIGUE IN TENDONS AT MID SPAN

Stress range caused by fatigue load model 3 $\Delta\sigma_{s,\,Ec} = 17$ MPa

The stress range is small as the section remains uncracked under the applied loads. The resulting damage equivalent stress range is 25 MPa, far less than the permissible stress range of 185 MPa.

FATIGUE OF BEAM LINKS DUE TO GLOBAL SHEAR

Shear force due to fatigue load model 3 $V_{Fat} = 238$ kN

Equ. 6.65

Angle of compressive struts in fatigue calculation $= \tan^{-1}(\sqrt{\tan\theta})$

where θ is the angle of concrete compression strut assumed in ULS design (see sheet A10.7)

$\therefore\ \theta_{fat} = \tan^{-1}(\sqrt{\tan 22°}) = 32°$

Rearranging equation 6.8:

$\Delta\sigma_{s,\,Ec} = (V_{Fat})s/(A_{sw}z\cot\theta) = 51$ MPa

$\lambda_{s,1} = 0.7$

$\lambda_{s,2} = 0.93$

$\lambda_{s,3} = 1.02$

$\lambda_{s,4} = 1.08$

$\varphi_{fat} = 1.4$

Damage equivalent correction factor $\lambda_s = 1.00$

Damage equivalent stress range $\Delta\sigma_{s,equ} = \Delta\sigma_{s,\,Ec}\,\lambda_s\,\gamma_{F,fat}$

$= 51 \times 1.00 \times 1.00$

$= 51$ MPa

Table 6.3N

Stress range at N^* cycles $\Delta\sigma_{Rsk}(N^*) = 163$ MPa

Link diameter $\phi = 12$ mm

Table 8.1N

Minimum mandrel diameter $D = 4\phi$ mm

Table 6.3N, Note 1

Reduced value due to bars being bent $\zeta\Delta\sigma_{Rsk}(N^*) = 0.35 + 0.026\,D/\phi$

$= 74$ MPa

Permissible stress range $= \zeta\Delta\sigma_{Rsk}(N^*)/\gamma_{S,fat} = 74/1.15$

$= 64$ MPa

Permissible stress range is greater than the damage equivalent stress range. Therefore, fatigue reistance of shear links is adequate.

CONCRETE BRIDGE DEVELOPMENT GROUP

Project details

CBDG Design Example
ULS bending check for slab

Chapter Ref. 11
Clause Ref. 11.1
Sheet No. A11.1

ULTIMATE LIMIT STATE VERIFICATION

BS EN 1992-1-1
6.1

SAGGING MOMENT

Design flexural moment $M_{Ed} = 65$ kNm/m

Assuming reinforcement consistes of B20 bars, effective depth, $d = 160 - 50 + 20/2 = 120$ mm

Steel force at design ultimate yield $F_s = A_s f_{yk}/\gamma_S = 500\ A_s/1.15$

Depth of concrete compression block $\lambda x = F_s/(b\eta f_{cd}) = F_s/(b\eta\alpha_{CC}f_{ck}/\gamma_C)$

Lever arm $z = d - (\lambda x/2)$

Design moment of resistance $M_{Rd} = F_s z = F_s d - F_s^2/(2b\eta f_{cd}) \geq M_{Ed}$

$120F_s - F_s^2/[2 \times 1000 \times (1.0 \times 0.85 \times 35/1.5)] \geq 65 \times 10^6$

Which gives a required steel force of: $F_s = 623$ kN

Required area of steel $A_s = 623000 \times 1.15/500$

$= 1433\ \text{mm}^2$

Reinforcement will be at alternate 100 mm and 200 mm centres. Therefore 20 mm bars are needed to provide the required area of reinforcement, giving a total area of 2094 mm^2/m.

HOGGING MOMENT

Design flexural moment $M_{Ed} = 34$ kNm/m

Initially , ignore concrete within the ribs, unless the resulting reinforcement requirement is excessive. Assume reinforcement consists of B16 bars, giving an effective depth, $d = 160 - 45 -16/2 = 107$ mm.

Design moment of resistance $M_{Rd} = F_s z = F_s d - F_s^2/(2b\eta f_{cd}) \geq M_{Ed}$

$107F_s - F_s^2/(2 \times 1000 \times 1.0 \times 0.85 \times 35/1.5) \geq 34 \times 10^6$

Which gives a required steel force of: $F_s = 346$ kN

Required area of steel $A_s = 346000 \times 1.15/500$

$= 796\ \text{mm}^2$

B16 bars at alternate 100 mm and 200 mm centres (to match lower layer) gives a total area of 1340 mm^2/m.

MINIMUM REINFORCEMENT REQUIREMENTS according to BS EN 1992-1-1 9.2.1.1:

Bottom reinforcement $d = 120$ mm

Equ. 9.1N $A_{s,min} = 0.26\ (f_{ctm}/f_{yk})\ b_t d \geq 0.0013 b_t d$

$= 200\ \text{mm}^2/\text{m}$

Top reinforcement $d = 107$ mm

$A_{s,min} = 178\ \text{mm}^2/\text{m}$

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Project details

CBDG Design Example
Punching shear and SLS checks

Chapter Ref. 11
Clause Ref. 11.2
Sheet No. A11.2

BS EN 1992-1-1
6.4

PUNCHING SHEAR - LM2 wheel is clearly the worst case.

Punching load $V_{Ed} = \gamma_{sup} \times 200 = 270$ kN

The loaded area should be calculated allowing for dispersal through the surfacing but not through the slab, giving an area of 0.65 m x 0.90 m. Critical perimeter is taken to be 2*d* from the loaded area.

Equ. 6.32

“Average” effective depth $d_{eff} = (104 + 120)/2 = 112$ mm
Basic control perimeter $u_1 = 2(0.65 + 0.90) + 2\,\pi\,(2 \times 0.112)$
$= 4.51$ m

6.4.4

Punching shear resistance

Allowance for size effect $k = 2.0$
Longitudinal reinforcement ratio $\rho_{ly} = 754/(1000 \times 112) = 0.0072$
Transverse reinforcement ratio $\rho_{lz} \quad 2094/(1000 \times 120) = 0.0175$
Effective reinforcement ratio $\rho_l = \sqrt{(\rho_{ly}\,\rho_{lz})} \leq 0.02$
$= 0.011$

Average normal concrete stress $\sigma_{cp} = 0$ MPa
$C_{Rd,c} = 0.12$

Equ. 6.3N

Minimum shear strength $v_{min} = 0.59$ MPa
$k_1 = 0.1$

Equ. 6.47

Punching shear resistance $v_{Rd,c} = C_{Rd,c}k(100\rho_l f_{ck})^{1/3} + k_1\sigma_{cp} \geq (v_{min} + k_1\sigma_{cp})$
$= 0.12 \times 2(100 \times 0.011 \times 35)^{1/3} \geq 0.59$
$= 0.81$ MPa

Punching strength $V_{Rd,c} = 4.51 \times 1000 \times 112 \times 0.81/1000$
$= 409$ kN

$V_{Rd,c} \geq V_{Ed}$ OK

SERVICEABILITY LIMIT STATE VERIFICATION

STRESS LIMIT - check under characteristic load combination

Table A11.1

	Moment	Normal stress (MPa)	
		Concrete	Reinforcement
Limiting stress		21	-400
Sagging moment	50 kNm/m	20.1	-220
Hogging moment	26 kNm/m	12.6	-152

CRACK WIDTH - check under quasi-permanent load combination

Stress in reinforcement due to sagging moment = 18 MPa
Stress in reinforcement due to hogging moment = 45 MPa

Comparison with Tables 7.2N and 7.3N indicates that crack width will not be a concern.

CONCRETE BRIDGE DEVELOPMENT GROUP

Project details

CBDG Design Example
Pier Design
ULS section checks

Chapter Ref. 12
Clause Ref. 12.2
Sheet No. A12.1

GEOMETRIC IMPERFECTIONS

BS EN 1992-1-1 — Depth of section $h = 600$ mm

5.2(1) & 6.1(4) — Assumed eccentricity of loading $e_0 = h/30 \geq 20 = 20$ mm

BS EN 1992-2 5.2(105) — Represent geometric imperfection by an inclination, θ_l

Basic value of inclination $\theta_0 = 0.005$ rad

Height of member $l = 6100$ mm

Reduction factor for height $\alpha_h = 0.81$

Equ. 5.101 — Inclination $\theta_l = \theta_0\alpha_h = 0.004$ rad

Additional moments due to imperfections $M_{add} = (e_0 + \theta_l l)\ N_{Ed} = 0.044N_{Ed}$

FLEXURAL RESISTANCE

Maximum flexural moment $M_{Ed} = 651$ kNm/1.5 m width

Corresponding design value of normal force $N_{Ed} = 1501$ kN/1.5 m width

Design value of flexural moment inc. geometric imperfections

$M_{Ed} = 651 + 0.044 \times 1501$

$= 717$ kNm/1.5 m width

Assume second order effects are negligible, confirm once reinforcement is decided.

Thickness of pier wall $h = 600$ mm

Height of pier wall $l = 6100$ mm

Assume reinforcement consists of 20 mm bars contained within 16 mm links:

Effective depth of section $d = 600 - 65 - 16 - 20/2$

$= 509$ mm

Steel force at design ultimate yield $F_s = A_s f_{yk}/\gamma_S = 500A_s/1.15$

Depth of concrete compression block $\lambda x = (F_s + N_{Ed})/(b\eta f_{cd})$

$= (F_s + N_{Ed})/(b\eta\alpha_{CC}f_{ck}/\gamma_C)$

taking moments around centroid of section:

Design moment of resistance $M_{Rd} = F_s(d - h/2) + \lambda x b\eta f_{cd}(h/2 - \lambda x/2) \geq M_{Ed}$

$F_s(509 - 300) + (F_s + 1501000)\ [300 - (F_s + 1501000)/(2 \times 1500 \times 1.0 \times 0.85 \times 35/1.5)] \geq 717 \times 10^6$

Which gives a required steel force of: $F_s = 682$ kN

(Depth to neutral axis, $x = 1.25 \times (F_s + N_{Ed})/(b\eta f_{cd}) = 92$ mm. This is much less than the effective depth, therefore reinforcement will have yielded).

Required area of steel $A_s = 682000 \times 1.15/500$

$= 1569$ mm^2/1.5 m per face

B20 bars at 200 mm centres provide 2356 mm^2/1.5 m per face

CONCRETE BRIDGE DEVELOPMENT GROUP	Project details CBDG Design Example Pier Design ULS section checks	Chapter Ref. 12 Clause Ref. 12.2 Sheet No. A12.2

SECOND ORDER EFFECTS - confirm that second order effects are negligible:

BS EN 1992-1-1 — Height of pier $l = 6100$ mm

5.8.3.2(1) — Effective length $l_0 = 0.7 \times l$ (Figure 5.7c)

$= 4270$ mm

Radius of gyration $i = (I_x/A)^{0.5}$

$= [600^3/(12 \times 600)]^{0.5}$

$= 173$ mm

Slenderness ratio $\lambda = l_0/i = 24.7$

BS EN 1992-1-1 — Total area of longitudinal reinforcement (B20 at 200 mm centres)

$A_s = 3142$ mm^2/m

Area of concrete $A_c = 600000$ mm^2/m

5.8.3.1(1) — Mechanical reinforcement ratio $\omega = A_s f_{yd}/(A_c F_{cd})$

$= 3142 \times 435/(600000 \times 19.8)$

$= 0.12$

Ratio of end moments $r_m = -0.50$ for a column pinned at top, fixed at base, as shown in BS EN 1992-1-1, Figure 5.7c

5.8.4(2) — Effective creep ratio $\varphi_{eff} = \varphi(\infty,t_0)M_{OEqp}/M_{OEd}$

where:

M_{OEqp} is the 1st order bending moment under quasi-permanent (SLS) load = 0 kNm

M_{OEd} is the 1st order bending moment under the design (ULS) load = 651 kNm

And $\varphi(\infty,t_0)$ is the final creep coefficient, calculated in the manner shown on sheet 7.1

Therefore: $\varphi_{ef} = \varphi(\infty,t_0) \times 0.0/651 = 0.00$

Maximum design axial force $N_{Ed} = 1971$ kN/m

$A = 1/(1 + 0.2\varphi_{ef}) = 1.00$

$B = (1 + 2\,\omega)^{0.5} = 1.11$

$C = 1.7 - r_m = 2.20$

Relative normal force $n = N_{Ed}/(A_c f_{cd}) = 0.17$

5.8.3.1(1) Equ. 5.13N — Limiting value to slenderness ratio $\lambda_{lim} = 20ABC/\sqrt{n}$

$= 119.8$

$\lambda < \lambda_{lim}$ SECOND ORDER EFFECTS CAN BE IGNORED

SHEAR RESISTANCE - Shear forces due to accidental loading are far more severe, so consider them.

cbdg CONCRETE BRIDGE DEVELOPMENT GROUP	Project details CBDG Design Example Pier Design Early-age crack control	Chapter Ref.	12
		Clause Ref.	12.2
		Sheet No.	A12.3

BS EN 1992-1-1	EARLY-AGE THERMAL CRACK CONTROL - Edge restraint at base of pier			
	Using the Simplified Method given in section 3.6 of the CIRIA guide "Early-age thermal crack control in concrete":			
3.6.1	Coefficient allowing for restraint and creep	K	=	0.50
Table 3.3	Maximum temperature drop during casting (for C35/45 concrete and assuming plywood formwork)	T_1	=	43°C
	Annual temperature change for summer casting	T_2	=	20°C
	Coefficient of thermal expansion	α_c	=	$12\ \mu\varepsilon/°C$
	Drying shrinkage (UK external exposure)	ε_{cd}	=	$150\ \mu\varepsilon$
Equ. 3.17	Early-age restrained strain	ε_r	=	$K\,[\alpha_c(T_1 + T_2) + \varepsilon_{cd}]$
			=	$0.50\ [12 \times (43+0) + 0]$
			=	$258\ \mu\varepsilon$
Equ. 3.17	Long-term restrained strain	ε_r	=	$K\,[\alpha_c(T_1 + T_2) + \varepsilon_{cd}]$
			=	$0.50\ [12 \times (43 + 20) + 150]$
			=	$453\ \mu\varepsilon$
	These values are significantly greater than the tensile strain capacities under sustained loading, which are assumed to be 70 με for the early age condition and 100 με for the long-term condition. Therefore cracking will occur, and sufficient reinforcement must be provided to limit the crack widths.			
CIRIA, 3.3.1	Minimum area of reinforcement required to control crack widths:			
	Coefficient allowing for stress distribution	k_c	=	1.0 for pure tension
	Coefficient allowing for non-uniform self-equilibriating stress distribution	k	=	0.8 for 600 mm thick section
Table 3.2	Early age concrete tensile strength	$f_{ctm}(t=3)$	=	1.92 MPa
3.3.3	Yield stress of reinforcement	f_{ky}	=	500 MPa
	Area of concrete in the tensile zone (per face)	A_{ct}	=	$300000\ mm^2/m$ length
Equ. 3.12	Minimum area of reinforcement	$A_{s,min}$	=	$k_c k A_{ct}\ (f_{ctm}(t)/f_{ky})$
			=	$1.0 \times 0.8 \times 300000 \times 1.92/500$
			=	$910\ mm^2/m$ per face
	This could be achieved using 16 mm bars at 200 mm centres at the base of the pier, where the edge restraint is highest.			
CIRIA, 3.4	Calculate early-age crack spacing:			
	Concrete cover	c	=	60 mm
	Coefficient taking accound of bond properties	k_1	=	1.14
	Note: The CIRIA guide recommends the conservative approach of reducing the normal value of $k_1 = 0.8$ for high bond bars by a factor of 0.7. This may be neglected in the future if experience shows it to be unnecessary.			

CONCRETE BRIDGE DEVELOPMENT GROUP

Project details

CBDG Design Example
Pier Design
Early-age crack control

Chapter Ref. 12
Clause Ref. 12.2
Sheet No. A12.4

Bar diameter ϕ = 16 mm
Bar spacing = 200 mm

Effective depth in tension around reinforcement $h_{c,eff}$ = $h/2 \leq 2.5\ (c + \phi/2)$
= $600/2 \leq 2.5\ (60 + 20/2)$
= 170 mm

Effective area of concrete in tension around reinforcement $A_{c,eff}$ = $bh_{c,ef}$
= 170000 mm^2

Reinforcement ratio $\rho_{p,eff}$ = $A_s/A_{c,eff}$
= 0.00591359 mm^2

CIRIA, 3.13

Maximum crack spacing: $S_{r,max}$ = $3.4c + 0.425\ k_1\phi/\rho_{p,eff}$
= 1515 mm

The crack pattern is assumed to form at early age, as has been calculated above. However, the largest restrained strains will occur long term, and hence the long term crack widths will govern.

CIRIA, 3.5.2

Long term crack width: w_k = $s_{r,max}\varepsilon_r$
= $204 \times 453 \times 10^{-6}$
= 0.6862405 mm

The minimum reinforcement (16 mm diameter bars at 200 mm centres) is not sufficient to limit the crack widths to 0.3 mm when calculated using the simplified method. Using this simplified method, 25 mm diameter bars at 100 mm centres are required to meet the crack width requirements.

MINIMUM REINFORCEMENT REQUIREMENTS

Minimum and maximum area of vertical reinforcement in the pier wall according to BS EN 1992-1-1 Cl. 9.6.2

Min. reinforcement $A_{s,v\ min}$ = $0.002A_c/2$ = 600 mm^2/m per face
Max. reinforcement $A_{s,v\ max}$ = $0.04A_c/2$ = 12000 mm^2/m per face

Minimum horizontal reinforcement in the pier wall according to BS EN 1992-1-1 Cl. 9.6.3

Actual area of vertical reinforcement $A_{s,v}$ = 2455 mm^2/m per face
Minimum area of horizontal reinforcement $A_{s,h\ min}$ = $0.25\ A_{s,v} \geq 0.001A_c$
= 614 mm^2/m per face

Use B16 bars at 250 mm centres, giving 804 mm^2/m

Minimum area of transverse reinforcement in the pier wall according to BS EN 1992-1-1 Cl. 9.6.4

Total area of vertical reinforcement $A_{s,v}$ = 4910 mm^2/m in total
Vertical reinforcement ratio ρ_v = $0.008\ A_c$
ρ_v < $0.02\ A_c$

Therefore transverse reinforcement not required, unless needed for shear resistance.

CONCRETE BRIDGE DEVELOPMENT GROUP

Project details

CBDG Design Example
Pier Design
Section checks under accidental loading

Chapter Ref. 12
Clause Ref. 12.2
Sheet No. A12.5

PEAK MOMENTS DUE TO ACCIDENTAL LOADING

Table A12.1

		Pier below edge beams	Central region of pier
Width of pier section being considered	m	1.125	1.500
Peak moment due to accidental load	kNm	775	512
Location of peak moment		Base of pier	Base of pier
Value of accompanying moment at top of pier	kNm	-179	-496
Value of accompanying moment at bottom of pier[1]	kNm	90	248
Total design value of moment, M_{Ed}	**kNm**	**865**	**760**
Quasi-permanent axial load, N_{Ed}[2]	**kN**	**818**	**611**

1. Calculated using standard beam formulae : $M_{bottom} = -M_{top}/2$. Moments include additional component due to geometric imperfections.
2. Quasi-permanent axial forces are used rather than the higher Frequent axial loads. This is because the critical situation could be when there is no traffic on the bridge providing a beneficial compressive force, or some intermediate stage of loading. It is therefore conservative to take the most severe moment due to traffic, and the least beneficial axial compression.

BS EN 1992-1-1
2.4.2.4

The edge section of pier is critical

FLEXURAL RESISTANCE OF PIER

Partial factor for concrete in accidental situation $\gamma_C = 1.20$
Partial factor for reinforcement in accidental situation $\gamma_S = 1.00$

Effective depth of section $d = 509$ mm
Area of tension reinforcement
(B20s at 200 mm centres) $A_s = 1767\ \text{mm}^2$
Steel force at design ultimate yield $F_s = 500A_s/1.00 = 884$ kN
Depth of concrete compression block $\lambda x = (F_s + N_{Ed})/(b\eta\alpha_{cc}f_{ck}/\gamma_C)$
$= 61$ mm

Design moment of resistance $M_{Rd} = F_s(d - h/2) + \lambda x b\eta f_{cd}(h/2 - \lambda x/2)$
$= 884000 \times 209 + (61 \times 1125 \times 0.85 \times 35/1.2) \times (300 - 61/2)$
$= 643$ kNm

$M_{Rd} < M_{Ed}$ Therefore section inadequate. Decrease bar spacing to 125 mm:

$F_s = 500 \times 2827/1.00 = 1414$ kN
$\lambda x = 80$ mm
$M_{Rd} = 876$ kNm $> M_{Ed}$

CONCRETE BRIDGE DEVELOPMENT GROUP

Project details

CBDG Design Example
Pier Design
Section checks under accidental loading

Chapter Ref. 12
Clause Ref. 12.2
Sheet No. A12.6

PEAK SHEAR FORCE DUE TO ACCIDENTAL LOADING PERPENDICULAR TO CARRIAGEWAY

Table A12.2

		Pier below edge beams	Central region of pier
Width of pier section being considered	m	1.125	1.500
Peak shear due to accidental load	kN	648	496
Value of accompanying shear force[1]	kN	35	113
Total design value of shear, V_{Ed}	**kN**	**683**	**609**
Quasi-permanent axial load, N_{Ed}	**kN**	**818**	**611**

1. Calculated using standard beam formula: $V = 3M_{top}/2L$

As BS EN 1992 allows enhancement of shear capacity due to axial loading, it is not apparent which section is critical.

BS EN 1992-2 6.2.2

Shear resistance of edge of pier assuming no shear reinforcement

Width of cross-section in tensile area b_w = 1125 mm

Effective depth of reinforcement d = 509 mm

Area of tensile reinforcement A_{sl} = 2827 mm^2

$C_{Rd,c} = 0.18/\gamma_C = 0.15$

Allowance for size effect k = 1.62

Reinforcement ratio ρ_l = 0.005

k_1 = 0.15

Equ. 6.3N — Minimum shear strength v_{min} = 0.43 MPa

Mean compressive stress σ_{cp} = 1.3 MPa

Equ. 6.2 — Shear resistance $V_{Rd,c}$ = 472 kN

$V_{Rd,c} < V_{Ed}$. Therefore shear reinforcement is required

BS EN 1992-2 6.2.3

Shear resistance with calculated shear reinforcement

Allowance for state of stress in compression chord α_{cw} = 1.05

Inner lever arm $z = 0.9\,d = 458$

Equ. 6.6N — Reduction factor for concrete cracked in shear v_1 = 0.52

Equ. 6.9 — Angle of compressive struts θ = 22°

Use 12 mm stirrups at 375 mm centres transversely:

Equ. 6.8 — Maximum spacing of links $s = 6^2\pi\,(1125/375) \times 458 \times 500 \cot(22)/683000$

$= 282$ mm

Place stirrups at 250 mm centres vertically. Similar calculations show that shear reinforcement is also required for the central section of the pier.

Shear capacity parallel to the carriageway must also be checked. The shear capacity without reinforcement is adequate for the accidental loads.

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CONCRETE BRIDGE DEVELOPMENT GROUP

Project details

CBDG Design Example
Pile foundation design

Chapter Ref. 13
Clause Ref. 13
Sheet No. A13.1

Characteristic actions for the pier and abutment loading, obtained from the grillage analysis of the deck:

Table A13.1

Action	Pier		
	R (kN)	M (kNm)	Q (kN)
Permanent, Unfavourable	5388	370	182
Variable, Unfavourable	2146	1143	323

Action	Abutment		
	R (kN)	M (kNm)	Q (kN)
Permanent, Unfavourable	1745	-678	0
Variable, Unfavourable	1156	-510	0

where:
R is the vertical action.
Q is the horizontal action parallel to the long axis of the bridge.
M is the overturning moment applied to the foundation.

BS EN 1997-1
2.4.7.3.4.1

BS EN 1997-1 presents three different Design Approaches in section 2.4.7.3.4. These design approaches define the partial factors that are to be applied to the actions or effects of actions (denoted by A), soil parameters (M) and resistances (R). The National Annex specifies that only Design Approach 1 may be used.

Design Approach 1 requires that 2 combinations of partial factors be considered. For axially loaded piles and anchors, they are:

- Combination 1: *A1 "+" M1 "+" R1*
- Combination 2: *A2 "+" (M1 or M2) "+" R4*

where "+" implies "to be combined with" and A1, A2 are different sets of partial factors to be applied to actions, etc.

For all cases other than Axially loaded piles and anchors, Combination 2 becomes:

- Combination 2: *A2 "+" M2 "+" R1*

Table A.NA.3 of the National Annex to BS EN 1997-1 directs that the partial factors applied to actions (γ_Q, γ_G) for the Structural (STR) and Geotechnical (GEO) limit states be taken from the NA to the BS EN 1990 bridges annex. These are given in Tables NA.A2.4(B) for set A1 and NA. A2.4(C) for set A2:

Table A.NA.3

Table A13.2

Action	Design Approach 1 (DA1)	
	Set A1	Set A2
Permanent, Unfavourable (γ_Q)	1.35	1
Variable, Unfavourable (γ_G)	1.35 Combination (DA1-1)	1.15 Combination (DA1-2)

Note: The partial factor for superimposed dead loads and settlement has conservatively been taken as 1.35 rather than 1.2 for simplicity.

CONCRETE BRIDGE DEVELOPMENT GROUP

Project details

CBDG Design Example
Pile foundation design

Chapter Ref. 13
Clause Ref. 13
Sheet No. A13.2

The design actions imposed by the bridge structure are therefore:

Table A13.3

Action	Design approach	Pier		
		R (kN)	M (kNm)	Q (kN)
Permanent, Unfavourable	DA1-1	7274	500	246
	DA1-2	5388	370	182
Variable, Unfavourable	DA1-1	2897	1543	436
	DA1-2	2468	1314	371

Action	Design approach	Abutment		
		R (kN)	M (kNm)	Q (kN)
Permanent, Unfavourable	DA1-1	2356	915	0
	DA1-2	1745	678	0
Variable, Unfavourable	DA1-1	1561	689	0
	DA1-2	1329	587	0

Design action per support	Pier		
	R (kN)	M (kNm)	Q (kN)
Combination (DA1-1)	10171	2043	682
Combination (DA1-2)	7856	1684	553
	Abutment		
Combination (DA1-1)	3917	1604	0
Combination (DA1-2)	3074	1265	0

Based on these actions, allow for 2 piles per support below pier and 1 pile per support for abutment.

CHARACTERISTIC GROUND CONDITIONS

The ground conditions have been interpreted from three boreholes each drilled to 35 m depth. The ground conditions are summarised as follows:

Made Ground	Assorted granular made ground comprising brick, ash, tile, and pipe garments.
River Terrace Gravels	Well graded sand and gravel of flint, chalk and limestone.
Stiff Clay	Firm becoming very stiff clay of medium plasticity with occasional sand partings.
Ground Water	Ground water level was struck at depths of between 2.25 and 2.75 m.

Based upon the ground investigation the characteristic ground profile is as follows:

Table A13.4

	Depth to top of layer (m)
Made Ground	Ground level
River Terrace Gravels	2.25
Stiff Clay	4.5
Ground Water	2.25

CONCRETE BRIDGE DEVELOPMENT GROUP

Project details	Chapter Ref.	13
CBDG Design Example Pile foundation design	Clause Ref.	13
	Sheet No.	A13.3

CHARACTERISTIC GROUND PROPERTIES

The ground properties have been derived from in-situ soil tests and laboratory test measurements. These results are plotted against depth on Figures A13.1 to A13.4.

DERIVATION OF SOIL STRENGTH

- Made Ground - Ignore for pile capacity as too variable
- River Terrace Gravels - Derive internal angle of friction (ϕ') from SPT *N* using Peck, Thorburn & Houson
- Stiff Clay - Derive undrained shear strength (5 m) from SPT *N* values using Stroud (1988) and direct laboratory measurements

RIVER TERRACE DEPOSITS

Summary of SPT *N* data:

Table A13.5

Borehole number	N_{min}	N_{ave}	N_K	$\phi'd$
BH1	14	18	16	33°
BH2	15	17	16	32°
BH3	18	20	19	34°
Overall	14	18	17	32°

STIFF CLAY

For design purposes soil strength will be expressed in terms of depth below top of the stiff clays.

Table A13.6

Borehole number	Mean S_u (kPa)	Characteristic S_u (kPa)
BH1	$44 + 8.2\,Z$	$41 + 8.1\,Z$
BH2	$36 + 9.2\,Z$	$34 + 8.2\,Z$
BH3	$48 + 8.6\,Z$	$48 + 8.0\,Z$
Overall	$39 + 9.0\,Z$	$34 + 8.4\,Z$

BS EN 1992-2
Table A.NA.4

Material partial factors to determine pile resistance are taken to be set *M1* for both DA1-1 and DA1-2 combinations. These partial factors are all set to unity for soil strength parameters.

Figure A13.1
Undrained shear strength.

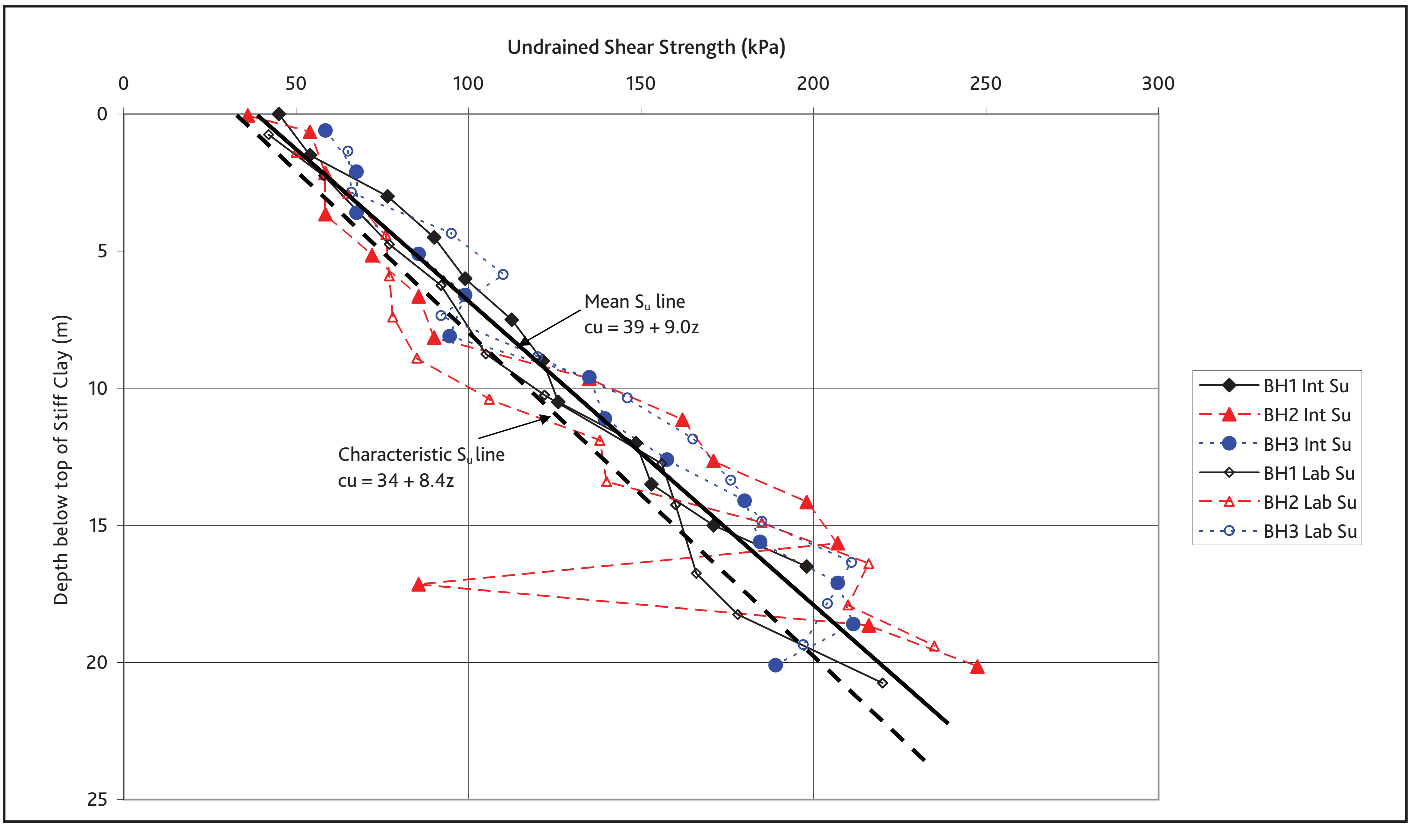


Figure A13.2
BH1 data.

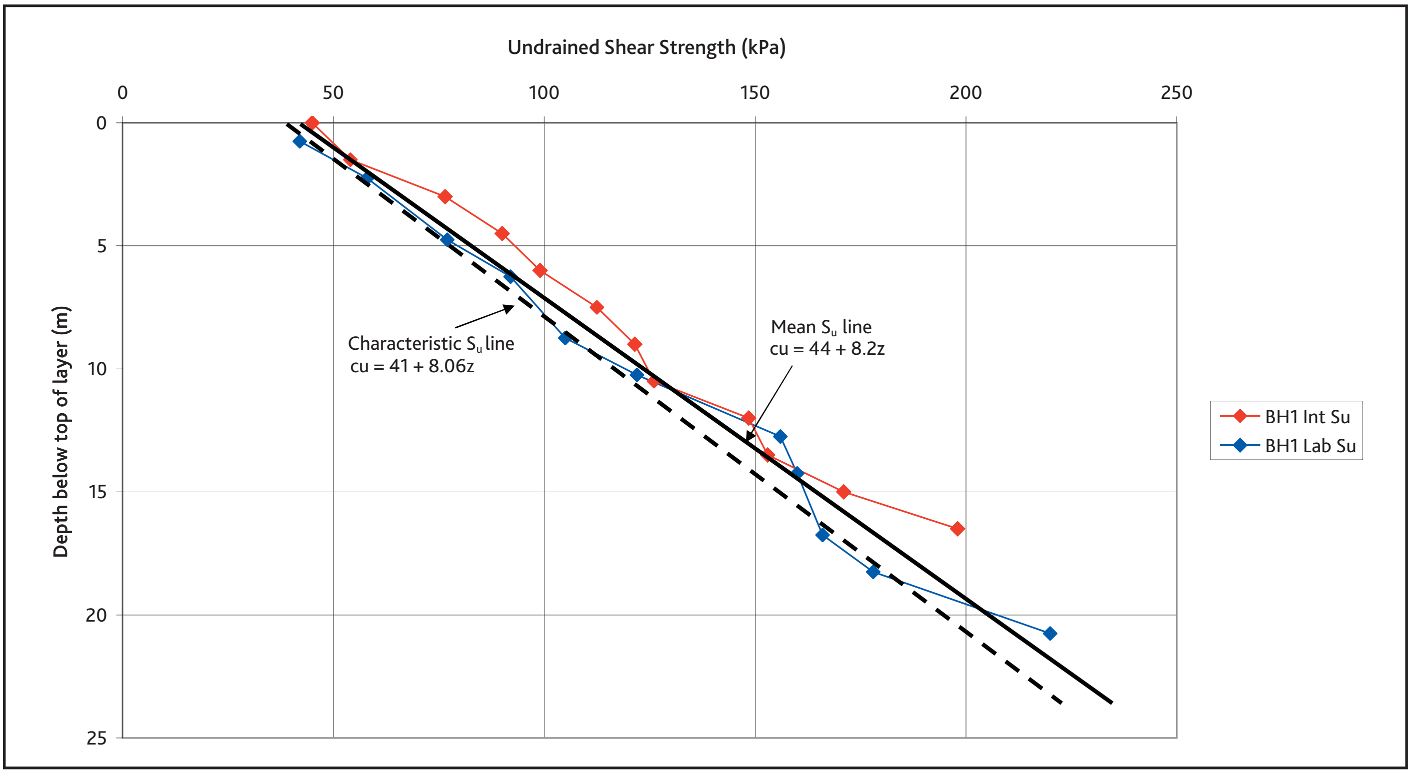


Figure A13.3
BH2 data.

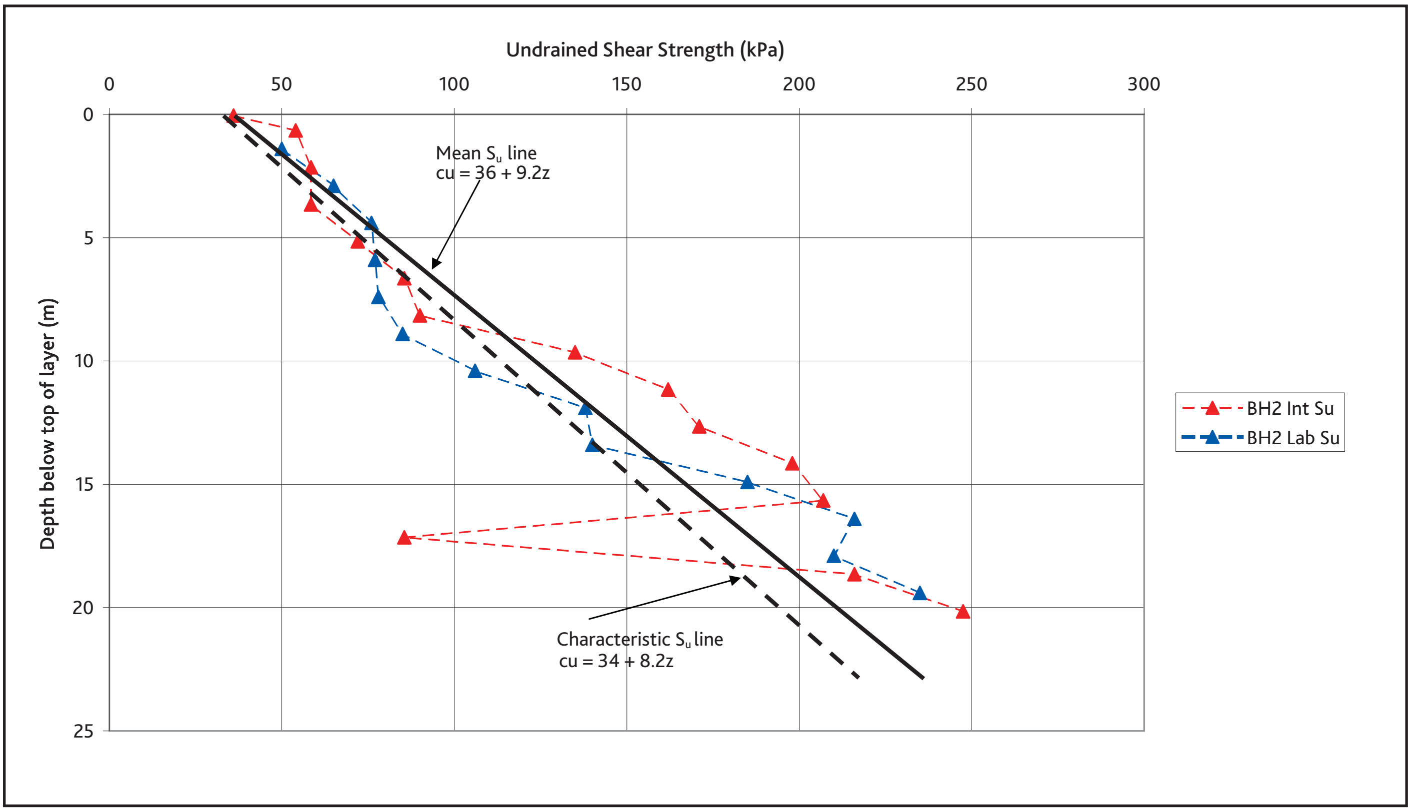
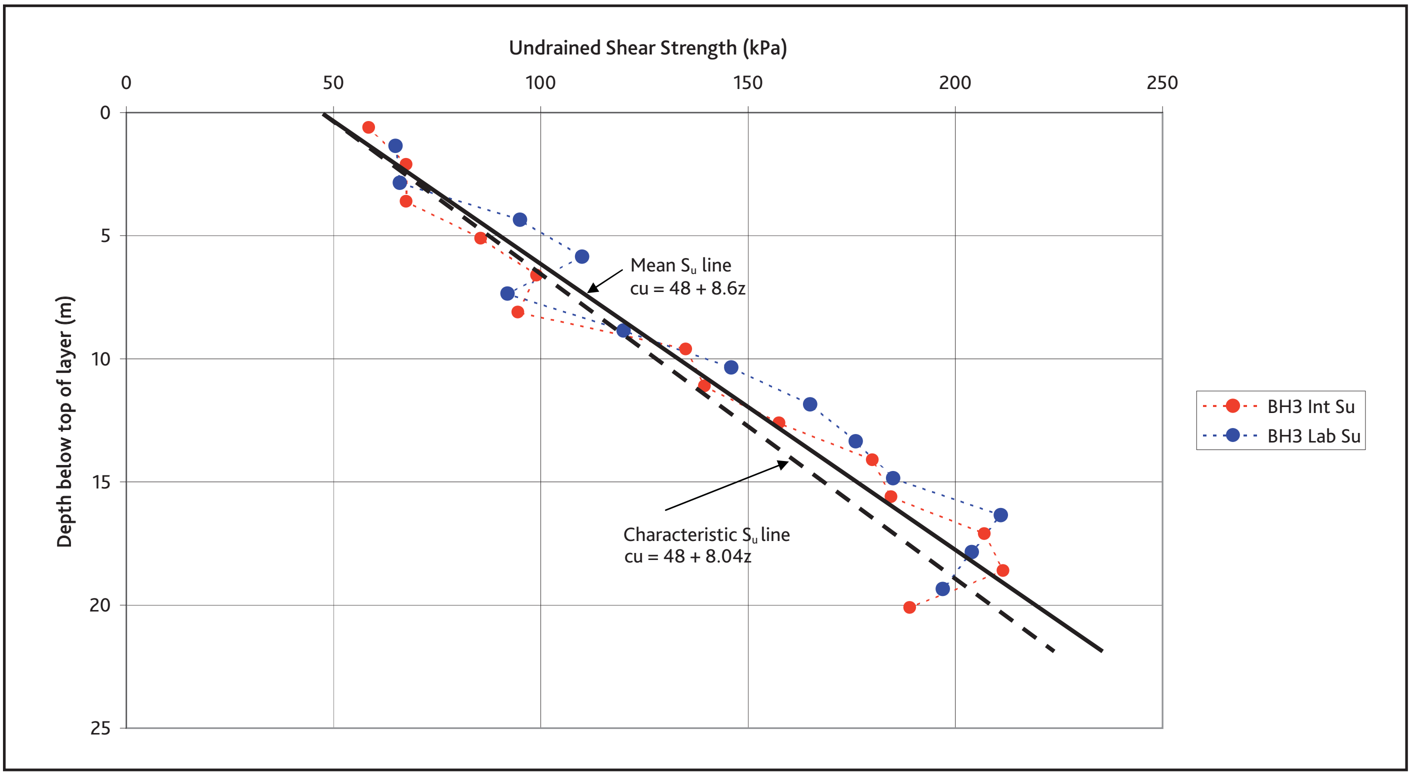


Figure A13.4
BH3 data.



CONCRETE BRIDGE DEVELOPMENT GROUP

Project details

CBDG Design Example
Pile foundation design

Chapter Ref. 13
Clause Ref. 13
Sheet No. A13.4

BS EN 1997-1

CALCULATION OF CHARACTERISTIC PILE RESISTANCE

7.6.2.3

Design from Ground Testing (Model Pile Approach)

The characteristic axial resistance of the piles, $R_{c,k}$, will be calculated from the design material parameters modified by the correction factors (ξ) which are dependent upon the number of ground tests used:

$$R_{c;k} = (R_{b;k} + R_{s;k}) = \frac{R_{b;cal} + R_{s;cal}}{\xi} = \frac{R_{c;cal}}{\xi} = \mathrm{MIN}\left\{ \frac{(R_{c;cal})_{mean}}{\xi_3} ; \frac{(R_{c;cal})_{min}}{\xi_4} \right\}$$

where:
$(R_{c;cal})_{mean} = [R_{c;cal}\,(\mathrm{BH1}) + R_{c;cal}\,(\mathrm{BH2}) + R_{c;cal}\,(\mathrm{BH3})]/3$
$(R_{c;cal})_{min} = \mathrm{MIN}\,\{R_{c;cal}\,(\mathrm{BH1}); R_{c;cal}\,(\mathrm{BH2}); R_{c;cal}\,(\mathrm{BH3})\}$

R_b is the pile base resistance, R_s the shaft resistance and R_c is the total compresive resistance of the pile. The suffix “cal” indicates the calculated value of the resistance and “k” the characteristic value.

Table A.NA.10

For 3 sets of ground tests ($n = 3$):
$\xi_3 = 1.42$
$\xi_4 = 1.33$

7.6.2.3(8)

Design from Ground Testing (Alternative Design Approach)

Alternatively, characteristic resistance can be calculated from the characteristic model properties with model factors (γ_{Rd}) instead of correlation factors. The model factor specified in the UK National Annex is 1.4 unless a maintained load test to the calculated unfactored resistance is carried out in which case a reduced model factor of 1.2 can be used. In this example no maintained load tests are assumed, therefore $\gamma_{Rd} = 1.4$.

CALCULATION METHOD

For granular soils:

$$R_{s;k} = A\sigma_v \tan \delta K_s$$

where:
A_s = area of the pile shaft (m^2)
σ_v = mean vertical effective stress (kPa)
δ = shaft soil interaction angle = 2/3 x ϕ'K
K_s = earth pressure coefficient, 0.7

For cohesive soils:

$$R_{s;k} = A_s \alpha S_{UK}$$

where:
A_s = area of the pile shaft (m^2)
α = pile shaft interaction coefficient, 0.6
S_{UK} = mean characteristic undrained shear strength (kPa)

For base resistance in cohesive soils:

$$R_{b;k} = A_b N_c\, S_{UK;base}$$

where:
A_b = area of pile base (m^2)
N_c = bearing capacity factor 9
$S_{UK;base}$ = characteristic base undrained shear strength (kPa)

CONCRETE BRIDGE DEVELOPMENT GROUP

Project details

CBDG Design Example
Pile foundation design

Chapter Ref.	13
Clause Ref.	13
Sheet No.	A13.5

CALCULATION OF PILE DESIGN RESISTANCE

Once the characteristic pile resistance has been calculated the design resistance can be determined from:

$$R_{c;d} = \frac{R_{b;k}}{\gamma_b} + \frac{R_{s;k}}{\gamma_S}$$

Design resistance partial factors:

Table A.NA.7

Table A13.7

	DA1-1	DA1-2
	Set R1	Set R4
Base resistance (R_B)	1.0	2.0
Shaft resistance (R_S)	1.0	1.6
Total/Combined (R_K)	1.0	2.0
Note: Partial factors selected for bored piles without any explicit verification of SLS of the piles.		

See Figures A13.5 and A13.6 for the increase in pile resistance with depth for both the "Model Pile" and alternative design approach for 900 mm diameter bored piles.

Calculated pile lengths below top of stiff clay:

Table A13.8

	Pier	Abutment
DA1-1; Model pile approach	24.0 m	20.25 m
DA1-1; Alternative design approach	24.75 m	20.75 m
DA1-2; Model pile approach	31.5 m	27.5 m
DA1-2; Alternative design approach	32.25 m	27.75 m

Therefore using the alternative design approach pier piles should be a minimum of 36.75 m and abutment piles should be a minimum of 32.25 m.

Figure A13.5
Design approach 1 - combination 1 pile resistances.

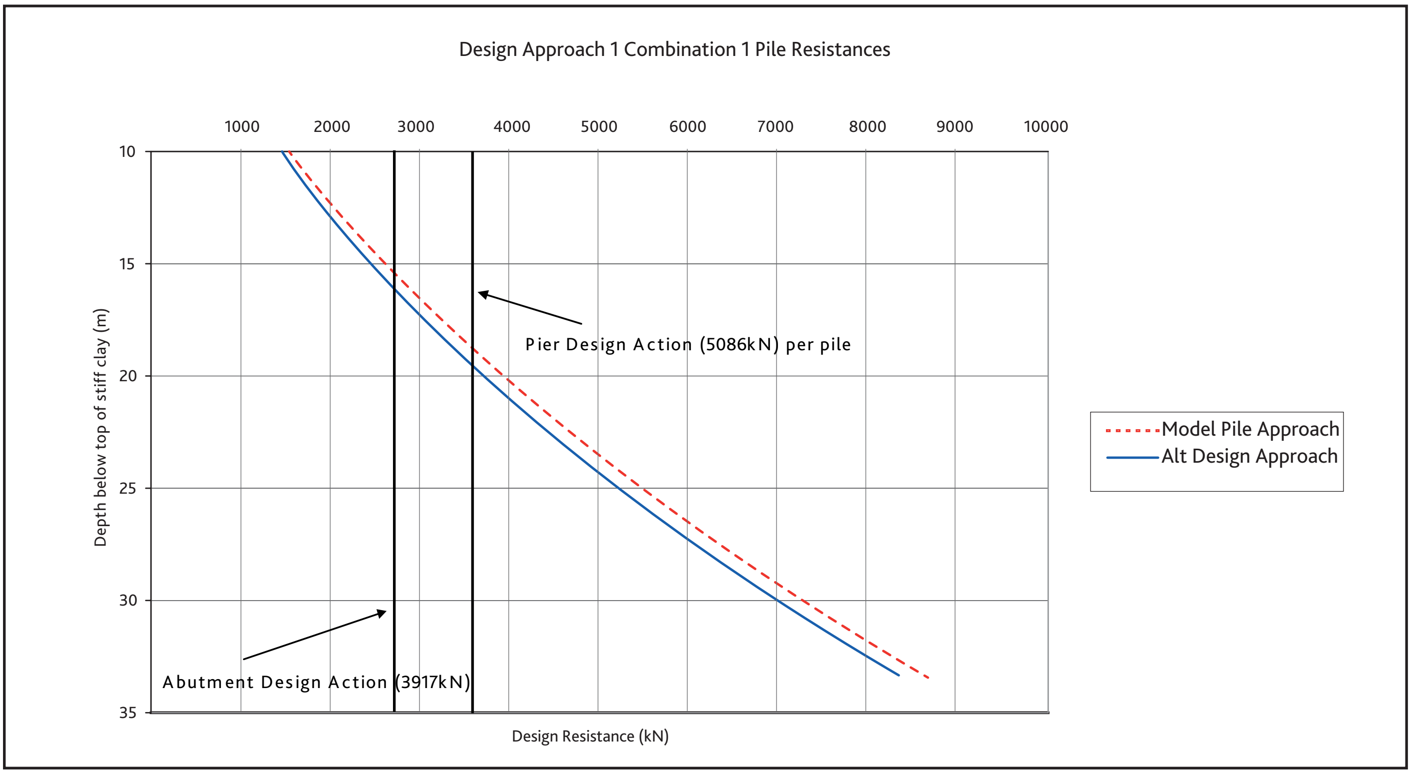
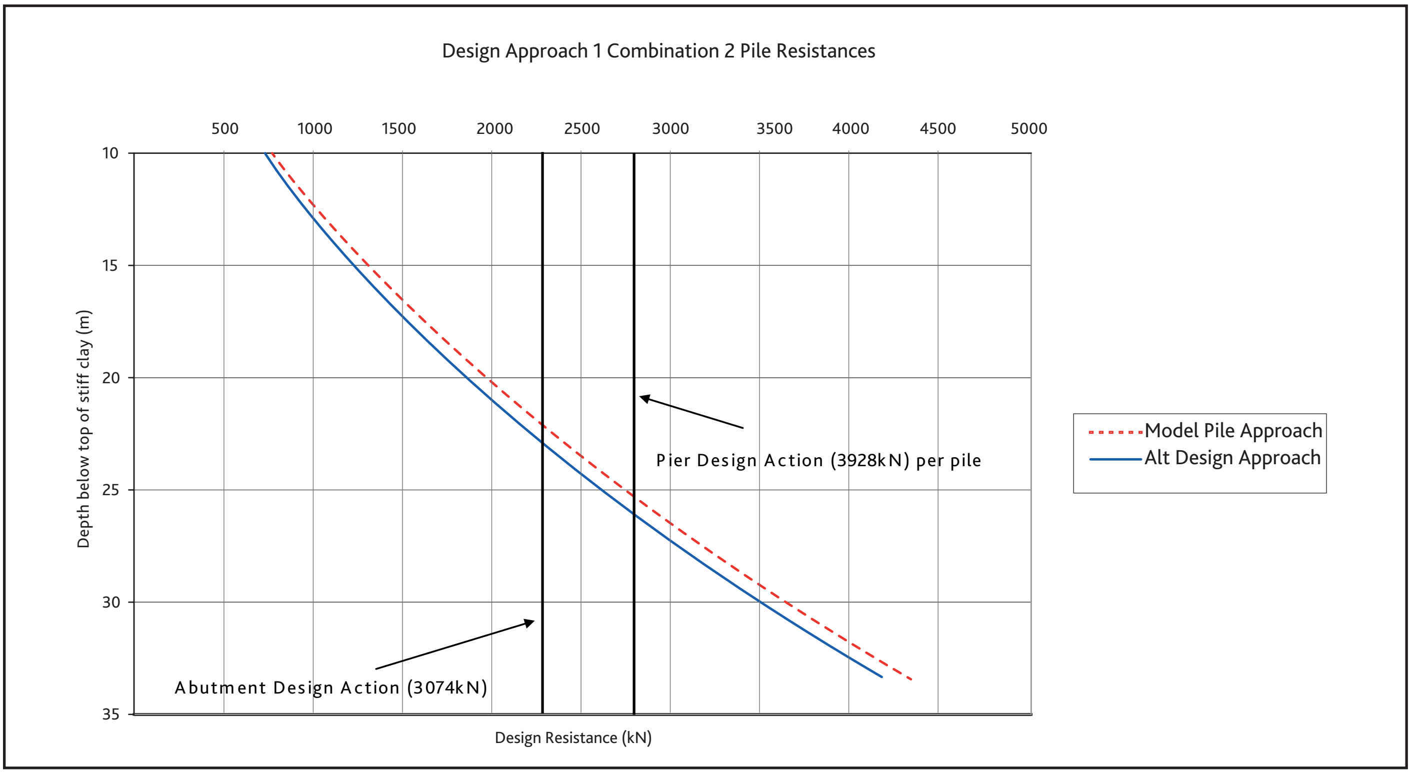


Figure A13.6
Design approach 1 - combination 2 pile resistances.



CONCRETE BRIDGE DEVELOPMENT GROUP

Project details		Chapter Ref. 13
CBDG Design Example Pile foundation design		Clause Ref. 13
		Sheet No. A13.6

BS EN 1997-1
7.7.1

RESISTANCE TO TRANSVERSE LOADS

The piles are to be of substantial length ($D/L = 0.035$). Therefore these piles shall be treated as long members. The key failure mechanism identified is:

7.7.1(3)

> Bending failure of the pile, accompanied by local yielding and displacement of the soil near the top of the pile (STR and GEO limit state)

Attention is drawn to PD 6694-1, recommendations for the design of structures subject to traffic loading to BS EN 1997-1:2004

BS EN 1997-1
2.4.7.3.4.2

Material partial factors

Partial factor sets M1 and M2 for DA1-1 and DA1-2 respectively.

Table A.NA.4

Table A13.9

Soil parameter	M1	M2
Angle of shearing resistance	1.0*	1.25
Undrained sheer strength	1.0	1.4

* to be applied to $\tan\phi'_K$

Therefore the following material properties will be used for this analysis:

Table A13.10

Soil parameter		M1 MG	M1 Gravels	M1 Clay	M2 MG	M2 Gravels	M2 Clay
Angle of shearing resistance	ϕ_K	30	32	27	30	32	27
	ϕ_d	30	32	27	24.8	26.5	22.2
Undrained shear strength	S_{Ud}	N/A	N/A	34+8.4z	N/A	N/A	34+8.4z
	S_{Uk}	N/A	N/A	34+8.4z	N/A	N/A	24.3+6z

BS EN 1997-1
2.4.6.3.4.2

ACTION PARTIAL FACTORS

Partial factor sets A1 and A2 to be used for DA1-1 and DA1-2 respectively.

Table NA.A2.4
(Set C)

Table A13.11

	Design approach 1	
	Combination 1 (A1)	Combination 2 (A2)
Permanent, Unfavourable (γ_G)	1.35	1.0
Variable, Unfavourable (γ_Q)	1.35	1.15

Therefore the following design actions will be applied in the pile analysis.

Table A13.12

		Pier (2 No. Piles)		Abutment (1 No. Pile)	
		DA1-1	DA1-2	DA1-1	DA1-2
Permanent, Unfavourable	Q	123	91	0	0
	M	250	185	915	678
Variable, Unfavourable	Q	218	186	0	0
	M	772	657	689	587

CONCRETE BRIDGE DEVELOPMENT GROUP

Project details

CBDG Design Example
Pile foundation design

Chapter Ref. 13
Clause Ref. 13
Sheet No. A13.7

Pile analysis results

Table A13.13

Pile ref.	Load combination	Max. BM (kNm)	Max. S.F (kN/m)
Abutment	DA1-1	1604	275
Abutment	DA1-2	1265	219
Pier	DA1-1	1577	341
Pier	DA1-2	1291	277
Abutment	SLS verification	1188	203
		Max pile deflection = 16 mm	
Pier	SLS verification	1152	253
		Max pile deflection = 26 mm	

CONCRETE BRIDGE DEVELOPMENT GROUP

The Concrete Bridge Development Group aims to promote excellence in the design, construction and management of concrete bridges.

With a membership that includes all sectors involved in the concrete bridge industry –bridge owners and managers, contractors, designers and suppliers– the Group acts as a forum for debate and the exchange of new ideas. A major programme of bridge assessment, strengthening and widening is already underway to accommodate European standards and the increasing pressures on the UK road network. The Group provides an excellent vehicle for the industry to co-ordinate an effective approach and to enhance the use of concrete.

Through an active programme of events and seminars, task groups, newsletters, study visits and publications, the Concrete Bridge Development Group aims to:

- Address the challenge of the national bridge programme
- Provide a focus for all those involved in concrete bridge design, construction and management
- Promote an integrated approach and encourage development of innovative ideas and concepts
- Promote best practice in design and construction through education, training and information dissemination
- Make representations on national and international codes and standards
- Identify future research and development needs
- Maximise opportunities to develop the wider and better use of concrete.

Membership of the Concrete Bridge Development Group is open to those who have an interest in promoting and enhancing the concrete bridge industry. Five main types of membership are available:

- Group membership for industry organisations and associations
- Corporate membership for contractors, consultants, suppliers and specialist service companies
- Associate membership for academic organisations
- Bridge owners for all organisations that commission, own, maintain and manage concrete bridges
- Individual consultants

By being representative of the whole industry, the Concrete Bridge Development Group acts as a catalyst for the best in concrete bridge design, construction, maintenance and management.

PUBLICATIONS FROM THE CONCRETE BRIDGE DEVELOPMENT GROUP

Integral bridges *Technical Guide 1*
A report of a study visit in August 1997 by a CBDG delegation to North America, sponsored by DTI (1997)

Guide to testing and monitoring of durability of concrete structures *Technical Guide 2*
A practical guide for bridge owners and designers (2002)

The use of fibre composites in concrete bridges *Technical Guide 3*
A state-of-the-art review of the use of fibre composites (2000)

The aesthetics of concrete bridges *Technical Guide 4*
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Integral Concrete Bridges to Eurocode 2: A Task Group report

This document describes the design of a two-span integral concrete bridge, comprising precast pretensioned concrete beams with a reinforced concrete in-situ deck, to the relevant European Standards - applied in accordance with their UK National Annexes.

The guidance given in this publication is with respect to pretensioned concrete but the difference in the approach between BS 5400 and BS EN 1992-2, which generally treats prestressed and reinforced concrete in the same way, means that some of the guidance given in this document is equally valid in the design of a reinforced concrete or post-tensioned concrete bridge structure.

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