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The Steel Construction Institute, Silwood Park, Ascot, Berkshire, SL5 7QN.
Telephone: +44 (0) 1344 636525
Fax: +44 (0) 1344 636570
Email: membership@steel-sci.com

For information on publications, telephone direct: +44 (0) 1344 636513
or Email: publications@steel-sci.com

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Composite highway bridge design

In accordance with Eurocodes and the UK National Annexes

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The Steel Construction Institute
Silwood Park
Ascot
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FOREWORD

This publication is the first in a set of SCI bridge design guides that reflect the rules in the Eurocodes. The design guidance covers multi-girder and ladder deck forms of construction and includes guidance in relation to integral bridges. It is a companion to a book of worked examples and will be complemented by further guides for bridges.

The guidance in this publication has been developed from earlier well-established guidance in a number of SCI bridge design guides. The previous guides referred to BS 5400 for the basis of design.

The publication was prepared by Mr D C Iles, of The Steel Construction Institute. It incorporates general best practice advice from experienced designers and constructors, members of the Steel Bridge Group. The author is grateful to all the members of the Steel Bridge Group for their reviews and contributions to the preparation of this publication.

The preparation of this guide was funded by Tata Steel* and their support is gratefully acknowledged.

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* This publication includes references to Corus, which is a former name of Tata Steel in Europe

Contents

SUMMARY

This publication provides guidance on the design of composite highway bridges which take the form of a reinforced concrete slab on top of steel girders. It describes two common forms of construction: one using multiple parallel girders and the other using twin main girders with regularly spaced cross girders - the so-called ladder deck form of construction. It gives general advice on initial design.

Guidance is given on detailed design in accordance with the Eurocodes. The application of the principles and the rules in the relevant Parts of the Eurocodes is explained, with comprehensive references to the clauses in those Standards. The detailed design of components and connections, in terms of both strength and best practice for construction and durability is discussed. Forms of integral abutment are described and their implications on the design of the superstructure are mentioned. Non-contradictory complementary information (to be used in conjunction with Eurocode rules) for determining the slenderness of the bare steel beams during construction is given in an Appendix.

1 INTRODUCTION

Composite construction, in the form of a reinforced concrete deck slab on top of a number of steel girders, is an efficient and widely-used form of construction for highway bridges. Composite construction is used over a wide range of span lengths and configurations. This publication provides a comprehensive introduction to the design of composite highway bridges, covering the two principal structural configurations that are used in the UK: multi-girder and ladder deck construction.

In the initial design stages for a composite bridge, many of the key decisions are made about the form, shape and size of the structural components. To make these decisions requires an understanding of how the different structural configurations that can be chosen for the particular site conditions behave under load, how they can be built, what the costs of the structural options are and what hazards must be considered during construction and during in-service maintenance. The two basic structural configurations are described in Section 2; initial design is discussed in Section 3.

Detailed design is in essence a verification process - an accurate modelling of structural behaviour that informs the designer about the internal forces and stresses under load, followed by reference to recognised design standards that provide rules to determine an adequate reliability (loosely referred to as 'margin of safety') against failure. Modelling and analysis are discussed in Section 5, detailed design is covered in Sections 6 to 9.

National structural design standards such as those published by BSI (notably BS 5400 for bridges) are being replaced by the Eurocodes, a comprehensive set of standards for all types of structures and for all the normal constructional materials produced by CEN for use throughout Europe. This publication therefore sets out design guidance in relation to the rules in the appropriate Eurocode documents. In addition to the Eurocode documents, and the National Annexes that implement them in the UK, there are other Standards, published guidance documents and client authority requirements that the designer needs to consult. These documents are summarized in Section 4.

The guidance provided in this publication is a development of information provided in a number of earlier SCI publications and it also draws on the experience and authority of the designers and constructors who form the Steel Bridge Group[1]. The guidance can thus offer 'best practice' advice appropriate to today's economic and health and safety considerations. This guide offers interpretation of the relevant Eurocodes and provides additional information, where necessary.

This publication is complemented by another SCI publication[1] that provides two worked examples, illustrating the application of Eurocode design rules for typical highway bridge configurations.

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[1] For further information about the Steel Bridge Group, see Appendix D.

References are made in the text to 'Guidance Notes', for example to GN 1.07. These Notes are part of the publication *Steel Bridge Group: Guidance notes on best practice in steel bridge construction*[2]. The Notes have recently been updated to refer to the latest Standards, including the Eurocodes.

2 STRUCTURAL CONFIGURATION

The majority of composite highway bridges in the UK are of 'deck type' beam and slab construction, where a reinforced concrete deck slab sits on top of I-section steel girders and acts compositely with them in bending. There are two common forms of deck type bridge - multi-girder bridges and ladder deck bridges. The features of each are discussed below; the choice between the two forms depends on economic considerations and site-specific factors such as form of intermediate supports and access for construction.

2.1 Multi-girder bridges

2.1.1 General

In multi-girder construction a number of similarly sized longitudinal plate girders are arranged at uniform spacing across the width of the bridge, as shown in the typical cross section in Figure 2.1. The deck slab spans transversely between the longitudinal girders and cantilevers transversely outside the outer girders. The girders are braced together at supports and at some intermediate positions. Composite action between the reinforced concrete deck slab and the longitudinal girders is achieved by means of shear connectors welded on the top flanges of the steel girders.

The arrangement shown in the Figure is common where permanent formwork is used and shows four girders of equal depth and with a slab surface that follows the camber of the road. A footway/verge is provided either side of a 2-lane single carriageway and parapets/restraint barriers are mounted on the edge beams. Alternative arrangements for the same carriageway configuration are discussed in Section 8.1.1.

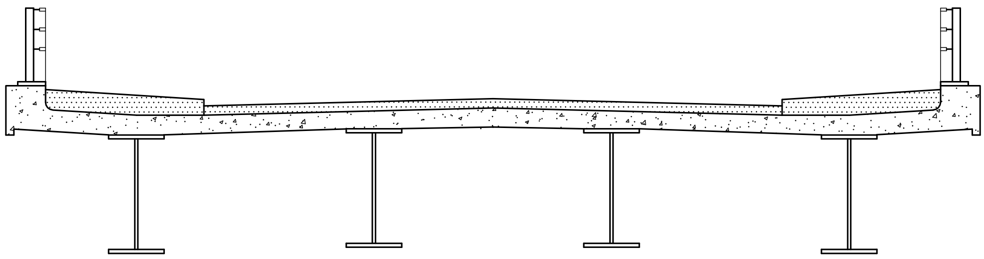


Figure 2.1 *Cross section of a multi-girder highway bridge*

Multi-girder construction is used for single spans and for continuous multiple spans.

2.1.2 Longitudinal girders

The steel girders are usually fabricated I-section plate girders; for smaller spans, it is possible to use rolled section beams (Universal Beams) but, for reasons discussed below, rolled sections are rarely used today.

Usually, girders are spaced between about 3.0 and 4.0 m apart, and thus, for an ordinary two-lane overbridge, four girders are provided. This suits the deck slab (see Section 2.3), which has to distribute the vertical loads from the wheels.

Plate girders

The use of plate girders gives scope to vary the girder sections to suit the loads carried at different positions along the bridge. The designer is free to choose the thickness of web and size of flange to suit the internal forces at different positions along the length of the span, though it must be remembered that too many changes may not lead to economy, because of the additional fabrication work. Splices are expensive, whether bolted or welded.

Most often, the girders have parallel flanges, that is, they have a constant depth. However, with plate girders, the designer can also choose to vary the depth of the girder along its length. For longer spans it is quite common to increase the girder depth over intermediate supports. For spans below about 50 m, the choice (constant or varying depth) is often governed by aesthetics. Above 50 m, varied depth may offer economy because of the weight savings possible in midspan regions. The variation in depth can be achieved either by straight haunching (tapered girders) or by curving the bottom flange. The shaped web, either for a variable depth girder or for a constant depth girder with a vertical camber, is easily achieved by profile cutting during fabrication.

Westgate bridge, Gloucester (Photo by courtesy of Corus)

Figure 2.2 *Typical multi-girder highway bridge*

Very occasionally, for reasons of appearance, the outermost girders are designed as a J-section girder; the bottom flange projects only on the inner side of the web. Requests for this detail arise from a dislike of the flange outstand, although there is little visible difference and the distinction is not noticed by most people. Use of such a section introduces torsional effects (because the shear centre is outside the line of the web) that require very careful consideration during design and construction, with significant penalty on costs. Such girders are outside the scope of this publication.

Also, on occasion, relatively small box girders are used in multi-girder construction. Box girders require special design consideration, because of their high torsional stiffness and high cost of fabrication. Advice on box girder design should be sought in other publications (general guidance can be obtained from SCI publication P140, although its detailed advice relates to BS 5400); advice on construction costs may be obtained from fabricators.

Rolled section girders

Universal beams up to 914 mm deep are available in the section range covered by BS 4; beams in the Corus Advance UKB section range are available up to 1016 mm deep. Such beams would provide sufficient bending resistance for single spans up to about 25 m and for continuous spans up to about 30 m, although the webs may be rather thin for the high shears associated with longer spans, unless the bridge is lightly loaded – a farm access bridge or a footbridge, for example.

Very little fabrication is necessary with universal beams, usually only the fitting of stiffeners over support bearings and the attachment of bracing. However, the beams often need to be curved in elevation to suit either the road profile or the pre-cambering for dead load; this can be carried out by specialist companies using heavy rolling equipment but it does add to cost. Even for smaller spans, universal beams can often be more economically replaced by similar size plate girder. Fabricators can advise on the relative economy.

2.1.3 Bracing

Support bracing

Girders need to be braced together at support positions, for stability and to effect the transfer of horizontal loads (wind and skidding forces) to the bearings that provide transverse restraint (usually one at each support position).

Restraints at supports are provided either by triangulated bracing systems or by horizontal beams, usually channel sections. The bracing systems at the end supports of non-integral bridges are usually also required to support the end of the deck slab. Integral bridges will require bracing at the end supports for the construction condition.

A typical bracing arrangement at an intermediate support is shown in Figure 2.3.

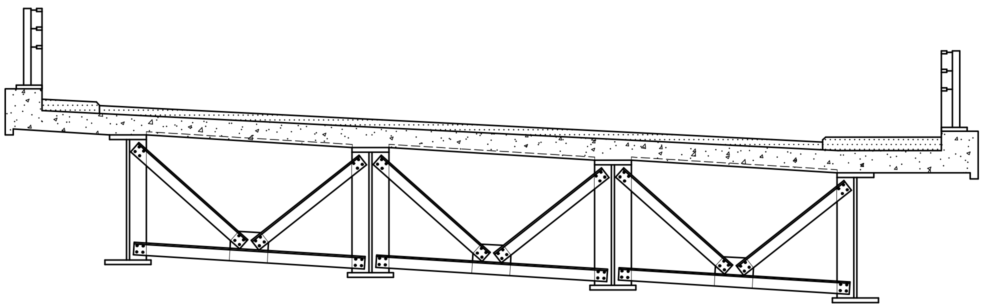


Figure 2.3 *Typical bracing arrangement at an intermediate support (shown for a super-elevated roadway)*

Intermediate bracing

In the completed bridge, intermediate bracing is usually needed at discrete positions in the spans of multi-span bridges, to stabilise the bottom flanges adjacent to intermediate supports (where they are in compression). During construction, bracing is needed to stabilise both the bottom flanges adjacent to intermediate supports and the top flanges in midspan regions. Where the girders are curved in plan, bracing will also be needed to provide 'radial' restraint to the bottom flanges (see Section 2.4.1).

In most cases, the most effective bracing system is a triangulated frame between adjacent girders. In the completed bridge this provides a very stiff restraint path from the plane of the deck slab through to the bottom flanges. In the construction condition, intermediate bracing between girders, without plan bracing, provides 'torsional restraint' – see discussion of the effectiveness of such bracing in Section 7.2.2. As an alternative, 'channel bracing' is often used with shallow main girders; the stiff channel has rigid connections to the main girders.

Intermediate bracing that is continuous across more than two main girders will participate in the global action and will distribute loading in any one lane to several main girders. However, such continuity does not provide much benefit to the design of the main girders (because the design case is usually with all lanes loaded) and introduces stress reversals in the bracing and its connections; the connection details are potentially prone to fatigue. To avoid this fatigue situation, designers use non-continuous bracing, where main girders are connected in pairs, with no bracing between one pair and the next, as shown in Figure 2.4.

Intermediate bracing may also be required if the headroom below the bridge is such that collision loading on the bridge soffit needs to be considered. Bracing at intervals provides restraint to the bottom flange and a load path to the bridge deck. In such cases, the bracing at supports has to be designed to transfer the collision loading down to the restrained bearings.

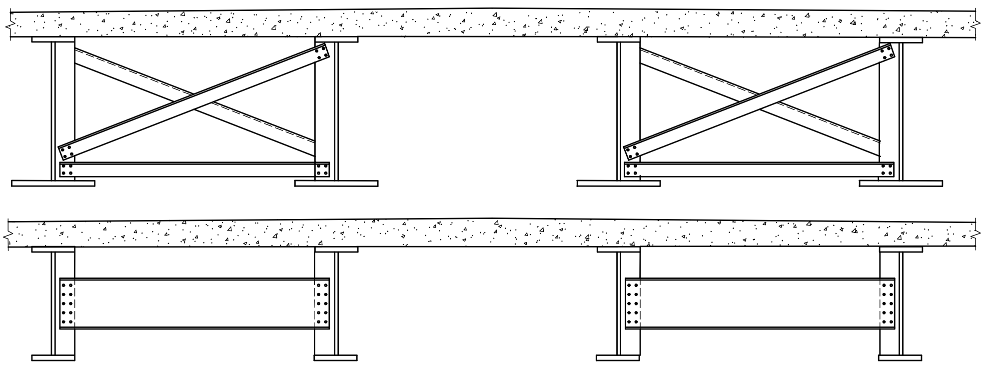


Figure 2.4 *Typical paired bracing arrangements*

Although continuity of transverse bracing is not needed (and not desirable, for the reason given above), tie/strut members are sometimes provided between the pairs of beams during construction in order either to share wind loads or to control the spacing between the pairs. Such members may need to be removed once the slab has been cast, because of their unwanted structural participation under traffic loading. Removal is a potentially hazardous activity that needs to be considered carefully when planning the construction method. Any construction bracing that is left in place should be assessed for fatigue.

Plan bracing

Plan bracing to the top flange is an alternative way to provide a stiff lateral restraint to the top flanges at the bare steel stage. Although such bracing is very effective in restraining the compression flange in midspan, its presence complicates construction. The two possible locations of plan bracing are above the top flange (connected to cleats on the top flange) and below the top flange: the former adds difficulty to the placing of reinforcement and conflicts with the use of permanent formwork; the latter would clash with temporary formwork and would need to be removed after casting (because it would attract unwanted forces when the slab is subject to local loading above it). Such bracing is rarely used now.

Plan bracing is occasionally provided to the bottom flanges of narrow bridges when the spans are long (over about 60 m) in order to improve the overall torsional stiffness of the bridge (at the completed stage) and thus reduce susceptibility to aerodynamic instability. Such improvement in torsional stiffness would also be beneficial for a bridge with significant curvature in plan. The presence of the bracing effectively creates a pseudo-box.

2.1.4 Crosshead girders

At intermediate supports, it is sometimes desirable to reduce the number of columns and bearings. Typically, instead of a bearing directly under each girder, one bearing is provided midway between each pair of girders, with a crosshead girder to transfer the reactions. Such an arrangement is particularly common with large skews (see Section 2.4.2). An example of a crosshead is shown in Figure 2.5. (This illustration also shows a continuity girder between the central girders; such a girder is advantageous for construction, to minimise twist during concreting, but is not normally needed for the permanent condition).

Overbridge on BNRR (Photo by courtesy of Mabey Bridge)

Figure 2.5 *Crosshead girder in a multi-girder bridge*

2.2 Ladder deck bridges

2.2.1 General

An increasingly common arrangement for highway bridges is to provide only two main girders, with the slab supported on cross-girders that span transversely between the two main girders - the slab then spans longitudinally between the cross girders. This arrangement is referred to as 'ladder deck' construction, because of the plan configuration of the steelwork, which resembles the stringers and rungs of a ladder.

A typical cross section of a ladder deck bridge is shown in Figure 2.6 and a photograph showing clearly the 'ladder' configuration is shown in Figure 2.7. (That example shows a bridge being constructed by launching, which is not typical; the triangulated plan bracing shown there was needed only for the launching stage.)

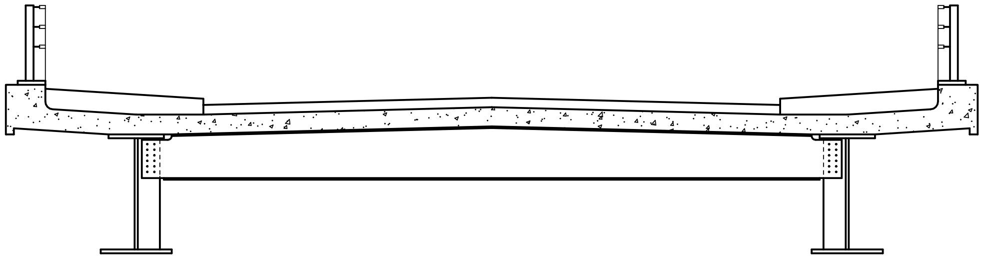


Figure 2.6 *Cross section of a typical ladder deck bridge*

M65 Whitebirk viaduct (Photo by courtesy of Mabey Bridge)

Figure 2.7 *Steelwork arrangement of a ladder deck bridge (launching nose, temporary bracing and falsework cantilevers are also shown in this example)*

The arrangement with two main girders is appropriate (and economic) for a bridge width up to that for a dual two-lane carriageway. Wider decks can be carried on a pair of ladder decks.

The main girders and cross girders are both provided with shear connectors, to develop composite action. Cross girders are usually connected to the main girders by bolting; intermediate transverse web stiffeners are provided at each cross girder connection.

Most ladder deck bridges are designed with uniform depth main girders but variable depth girders can be used. An example of a haunched girder ladder deck is shown in Figure 2.8.

Semmington Brook bridge (Photo by courtesy of Arup)

Figure 2.8 *Ladder deck bridge with haunched main girders*

Where the deck is wide (more than about 22 m), for example when a dual three-lane carriageway is carried, two adjacent ladder deck arrangements can be used. In such cases, the deck slab can be continuous across all four main girders or separate slabs may be provided, one on each pair of girders. Where the slab is continuous, it spans transversely between the innermost girders (which are thus limited to a spacing of about 3.5 m between them). Where separate slabs are provided, each deck cantilevers transversely and some form of joint may be required in the central reserve (see TD19[3] for requirements relating to gaps between decks).

2.2.2 Main girders

The main longitudinal girders are almost always fabricated plate girders; the heaviest rolled sections are unlikely to be sufficient, even for modest spans. Because there are only two webs, the web plate is thicker than it would be in a multiple girder arrangement; the web slenderness is lower and it is usually possible to develop the necessary shear resistance in the webs without use of web stiffening, other than that at the cross girders.

With longer spans, the size of the flanges, particularly the bottom flange, is likely to be quite large (in both width and thickness). Designers should check the availability of suitable plate material at an early stage, with particular attention to the toughness grade.

As ladder deck bridges have only two main girders, the question of structural redundancy might be raised in the choice of ladder deck configuration – if some accidental event were to damage one girder so severely that it could no longer carry even the dead loads, the bridge would collapse. There is no data on the likelihood of accidental events that could cause such damage, for either ladder deck or multi-girder bridges, and it is therefore not possible to make any quantitative assessment of reliability for either type. The girder sections of ladder deck bridges are generally larger than those of multi-girder decks and they are also restrained at close spacing by the cross girders; designers therefore consider this configuration to be sufficiently robust.

2.2.3 Cross girders

Cross girders are usually spaced at about 3.5 m centres, to suit a slab thickness of about 250 mm (see Section 2.3).

For a simple two or three lane bridge, where the main girders are 7 – 10 m apart, rolled sections (universal beams) may be sufficient for structural purposes, but plate girder sections are more likely to be used. Where there is a camber to the road surface (for example, with a two lane single carriageway, as shown in Figure 2.6) the top flange of a plate girder can follow the cross falls, allowing the use of a uniform thickness of both slab and surfacing. The bottom flange would normally be straight. If rolled section cross girders were used, either the sections would have to be cambered (which adds to fabrication cost), or the slab or surfacing must be tapered in thickness to provide the falls.

Where there is superelevation of the road surface, one main girder is arranged higher than the other and the cross girder depth is usually constant.

Cross girders are usually unstiffened and unbraced but long cross girders may require bracing for the construction condition (typically, channel bracing between pairs of girders at their mid-span).

Intermediate cross girders in sagging moment regions

Intermediate cross girders effectively act as simply supported beams in carrying the loading from the slab. The end moments, due to interaction with the main girders, are very small in relation to the strength of the cross girders, which can thus be designed as simply supported beams. However, the end moments may be large enough to influence the design of the cross girder to main girder connection.

In the composite condition, the cross girders in the sagging moment regions of the main girders are required to provide lateral restraint to the main girder bottom flanges only where the main girders are curved in plan or where lateral loads from vehicle impact on the soffit are to be resisted. The cross girders provide restraint through U-frame action (see further description below).

The cross girders also provide out-of-plane restraint to the slab where it is in compression; the stiffness of the cross girders and the slenderness of the slab both need to be considered. See further discussion in 6.2.1.

During construction, the cross girders provide torsional restraint to the main girders, both as restraint to lateral torsional buckling and, for curved main girders, in resisting the couple generated by the opposing 'radial' forces in the tension and compression flanges.

Intermediate cross girders in hogging moment regions

In the hogging moment regions of the main girders, adjacent to internal supports, the intermediate cross girders are required to provide lateral restraint to the bottom flanges of the main girders, which are in compression. This restraint is provided through the 'inverted U-frames' formed by the cross girders and web stiffeners to which they are attached. The connections between main and cross girders therefore need to transmit restraint moments and the frame needs to be stiff. If the cross girders are significantly shallower than the main girders, knee bracing or haunched cross girders may be needed, both to stiffen the frame and to reduce moments that need to be transmitted through the cross/main girder connections; see the descriptions of such arrangements for support cross girders, below.

Cross girders at internal supports (pier diaphragms)

At the internal supports of continuous spans, the cross girders are very often deeper than the intermediate cross girders, providing a stiffer and stronger ‘pier diaphragm’, with bolted connections that can transfer the larger restraint forces that occur at the supports (see Figure 2.9). The cross girder should not be quite as deep as the main girders, to avoid conflict with, and direct connection to, the bottom flange of the main girder.

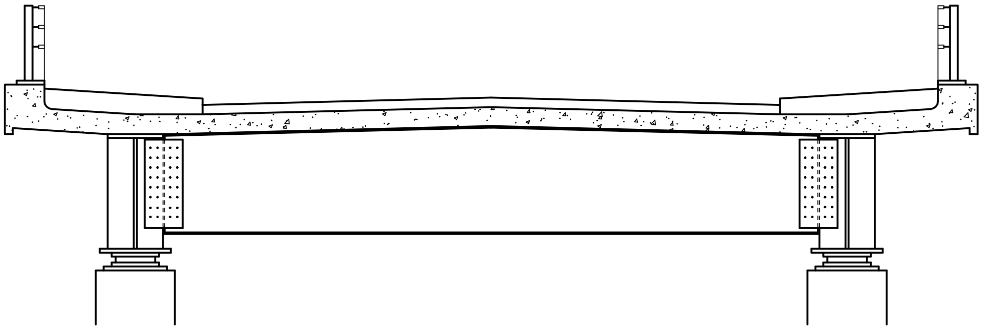


Figure 2.9 *Cross girder at an intermediate support of a ladder deck bridge*

As an alternative to using a deeper cross girder, knee bracing or a haunched cross girder can be provided, as shown in Figure 2.10 and Figure 2.11. This will stiffen the frame and reduce moments that need to be transmitted through the cross/main girder connections and may be advantageous if services or access ways are connected to the soffits of the cross girders along the length of the bridge. In practice, intermediate knee bracing is rarely provided – it is cheaper to use a deeper cross girder. Haunched cross girders are an even more expensive detail –a fabricator should be consulted before selecting this option.

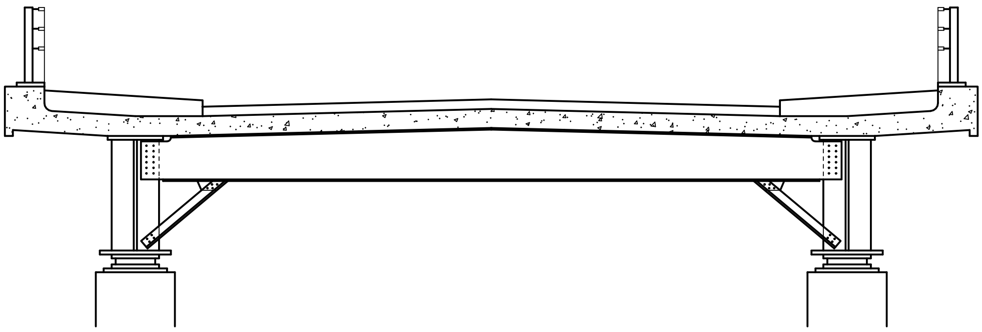


Figure 2.10 *Knee bracing arrangement*

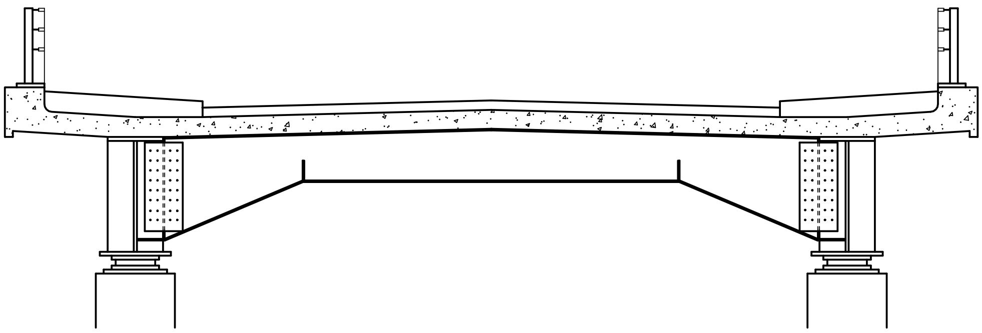


Figure 2.11 *Haunched cross girder at an intermediate support*

End supports

With non-integral construction, support diaphragms similar to those at intermediate supports are used. They provide an effective support to the end of the deck slab and to the expansion joint. Where the end supports are skew to the bridge axis, the diaphragms may act as trimmer girders - see page 16. For discussion of integral abutments, see Section 2.5.4.

Integral crossheads at internal supports

Supports are sometimes provided 'inboard' of the main girders, under the pier diaphragms, rather than directly under the main girders. The diaphragms are then more substantial and are often referred to as 'integral crossheads'. There may be good reasons for such an arrangement, particularly when it is difficult to provide support under one of the girders on a skew bridge, but it does add considerably to the fabrication and erection cost. If the main girders are haunched, such an arrangement, with no direct support under the most heavily loaded elements, is thought by many people to look rather unsettling.

An example of an integral crosshead is shown in Figure 2.12.

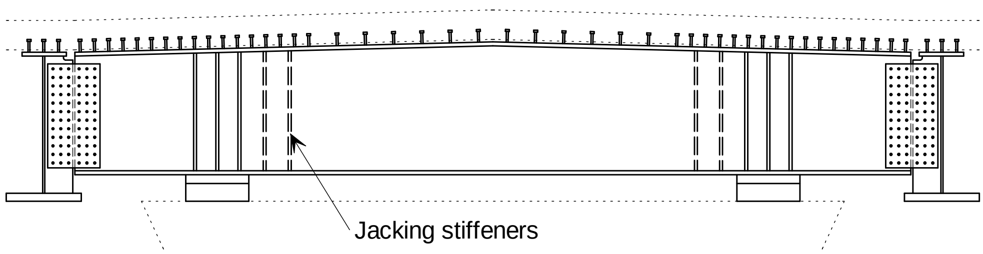


Figure 2.12 *Example of an integral crosshead*

(Note that if the main girders of the arrangement shown in Figure 2.12 were haunched, stiffeners would be required on both sides of the main girder webs and the web/flange connections would need to be designed for the tensile load due to vertical components of the forces in the inclined main girder flange.)

2.2.4 Cantilever girders

For normal lengths of deck cantilever outside the main girders (up to about 2 m), cantilever girders are not needed; the slab will cantilever transversely, as it does with multiple girder decks. (See further discussion in Section 2.3.)

Steel cantilever girders allow longer deck cantilevers to be provided but the main reason for considering them would be to avoid the need for cantilevered formwork. With cantilever girders, permanent formwork can be used across the full width of the deck.

The provision of cantilever girders leads to the requirement for moment continuity with the cross girders. This adds significantly to fabrication cost. Also, it is difficult to achieve good alignment at the tips of long cantilevers and this too adds to cost.

A cross section of a ladder deck with cantilever girders is shown in Figure 2.13.

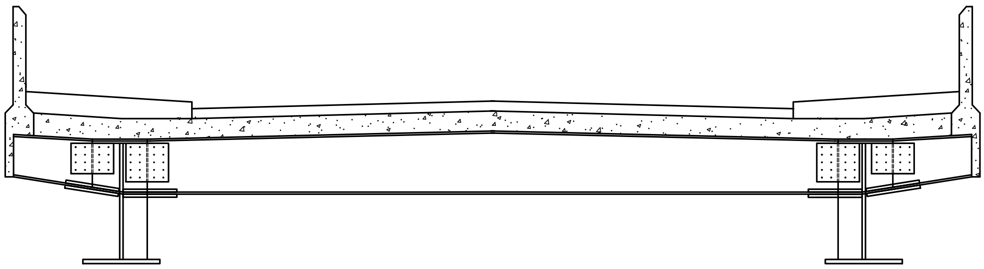


Figure 2.13 *Ladder deck with cantilever beams*

Figure 2.14 shows an example of cantilever brackets supporting permanent formwork and providing a visual feature, when the bridge is viewed from below.

Festival Bridge, Stoke (Photo by Courtesy of Cass Hayward)

Figure 2.14 *Haunched ladder deck bridge with cantilever beams (permanent formwork used across the full deck width)*

2.3 Deck slab

To sustain the combined load effects of local and global bending (particularly, for ladder decks, the global bending in hogging moment regions, which results in tensile forces in the slab) a deck slab thickness of about 240 - 260 mm is needed (the value depends partly on requirements for cover to reinforcement – see discussion in Section 6.3). The slab reinforcement is typically B20 bars at 150 mm centres top and bottom. A uniform thickness slab is normally used and this makes the deck suitable for construction using permanent formwork, either precast concrete planks or reinforced fibre panels, spanning longitudinally.

Composite action with the cross girders and with the main girders is achieved through the use of stud shear connectors.

This thickness of slab (240 – 260 mm) can be cantilevered up to about 2.0 m (overall length, from centreline of main girder to outside of edge beam), with footway or accidental traffic loading. When 'very high containment level' parapets or barriers are provided (see TD 19[3] for criteria), the slab thickness needs to be increased to resist the effects of collision loads (or alternatively, cantilever girders can be used, as described in Section 2.2.4, and the posts aligned with the cantilevers).

2.4 Dealing with curvature and skew

2.4.1 Curved decks

Where the bridge deck is curved horizontally (to suit the road alignment) the girders beneath the slab can either be straight or curved in plan. For large radii (over 300 m) a series of straight girders with angular change at discrete positions along the length can be used; typically these changes might be at approximately ¼ and ¾ span position, where splices are arranged at the points of contraflexure. The disadvantage of such an arrangement is that the length of the cantilever varies along the bridge. Appearance, from beneath the bridge, should be considered carefully when choosing this option.

Advances in computer modelling for fabrication have enabled fabricators to cut curved flanges from plate and thus provide ‘true’ curved beams. This overcomes the problem of varying length cantilevers and provides a better appearance from below the bridge. (In practice, the flange plates are still cut as a series of straights but these are so short, 1 m or less, that they appear truly curved.)

The change of direction of the bottom flange, either at discrete positions or ‘continuously’ requires a ‘radial’ force to balance the change in direction of the flange force. With multi-girder decks, transverse bracing is required at the change positions between a series of straights or at intervals along a curved girder (the interval needed depends on the curvature and the width of the flange). With ladder decks, the regular spacing of the cross girders and their attachment to the main girders is well able to providing this lateral restraint to the flange; the cross girders are arranged radial to the curve.

Highfield Lane Bridge (Photo by courtesy of Rotherham Metropolitan Borough Council)

Figure 2.15 *Curved multi-girder bridge, showing intermediate bracing*

2.4.2 Skewed bridges

Multi-girder bridges

For skewed multi-girder bridges, intermediate bracing is almost always arranged square to the main girders; there is no particular advantage in aligning such bracing on the skew for small skew angles and for large skew angles the interaction with bending of the main girders causes complications in design.

At intermediate and end supports, bracing is usually arranged on the line of the skew supports for small skew angles (less than about 25°); for large skew angles, bracing at intermediate supports is usually square to the main girders but bracing at the ends is along the line of the supports. Typical bracing arrangements are shown in Figure 2.16. Note that for small skews integral crossheads are usually continuous (although the continuity girders between the inner main girders are much lighter than those over the support). For larger skews there are no continuity girders between the inner girders, to avoid potential fatigue problems, although continuity bracing may be needed for construction, to control twist at the wet concrete stage. See further discussion of skew in GN 1.02[2].

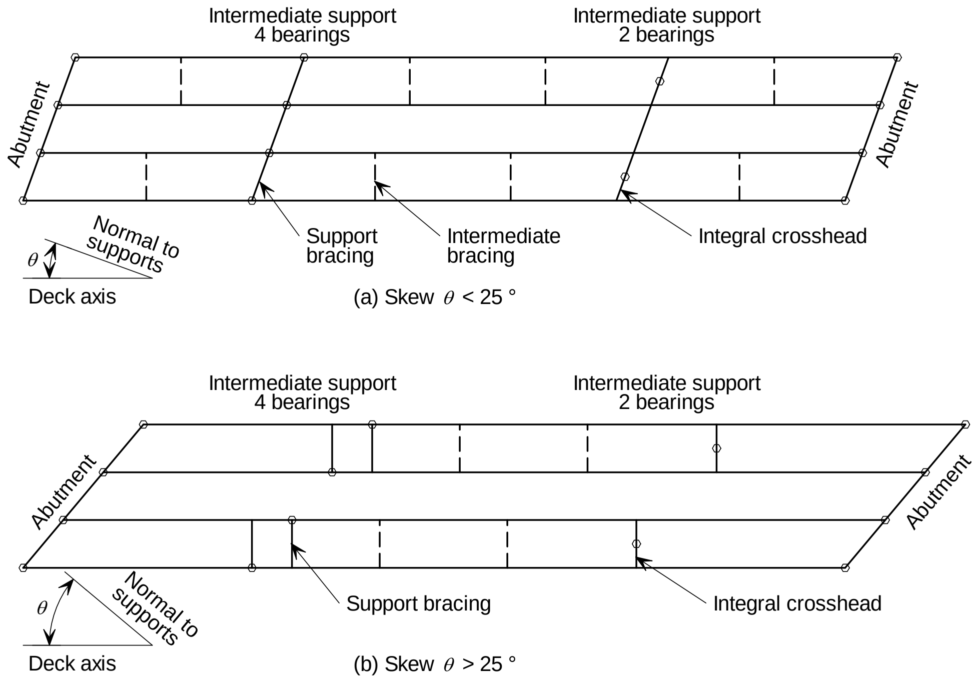
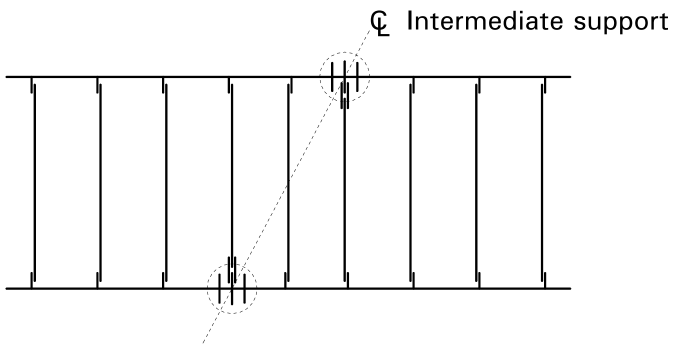


Figure 2.16 *Arrangements for skewed multi-girder decks*

Ladder deck bridges

Skewed intermediate supports

A particular merit of the ladder deck steelwork system is that skewed intermediate supports can be readily accommodated, as one end of a cross girder can be connected to the bearing stiffener over the support to one girder whilst the other end can be connected to an intermediate stiffener within the span. With such an arrangement the cross girders will not necessarily be at a regular spacing along the length of the deck, but will be spaced as dictated by the geometry of the skew (see Figure 2.17).



Note: only the girder webs and the web stiffeners are shown, for clarity

Figure 2.17 *Arrangement at skewed intermediate support*

Skewed end supports

At skew end supports, trimmed cross girders, connected into an end trimmer girder, may be required, as shown in Figure 2.18. This arrangement is usually preferred to a 'fanned' arrangement of cross girders. To simplify connection details, the connections at the obtuse corner for the end trimmer and the cross girder are separated, although the consequences on slab design in this area must be considered carefully and 3D modelling may be needed in order to predict the local behaviour with sufficient accuracy.

This arrangement is used even with integral abutments; the trimmer beam is then cast into the endscreen wall.

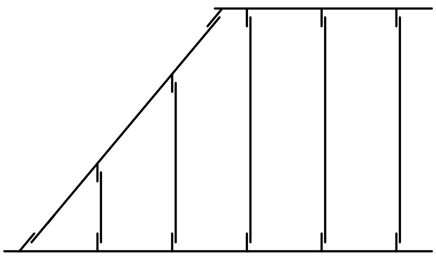


Figure 2.18 *Arrangement at skewed end support*

2.5 Substructures

Bridge substructures are usually of reinforced concrete construction. In non-integral bridges, the deck sits on bearings that are supported on the abutments and intermediate piers. Abutments may be spread footings (bankseats) or may be supported on piles; the abutments may also act as full-height retaining walls. Intermediate supports may take the form of individual columns (one under each bearing) or of a wall or 'leaf pier' that supports all the bearings at that intermediate position. Discussion of the forms of these supports is outside the scope of this publication but the articulation arrangements are discussed below.

In integral bridge construction, there is interaction between the sub- and superstructure; forms of integral abutment construction that are used are discussed in Section 2.5.4. Detailed design of integral abutments is outside the scope of this publication but some of the detailing issues are discussed in Section 9.

2.5.1 Bridge articulation

In non-integral bridges, the bridge deck is supported on bearings at each support and lateral restraint is provided at some of these bearings; the arrangement of the restraints, which must permit the thermal expansion and contraction of the deck, is known as articulation. A typical arrangement for a 2-span bridge supported on pot bearings is shown in Figure 2.19; alternative arrangements and a general discussion of articulation are given in GN 1.04.

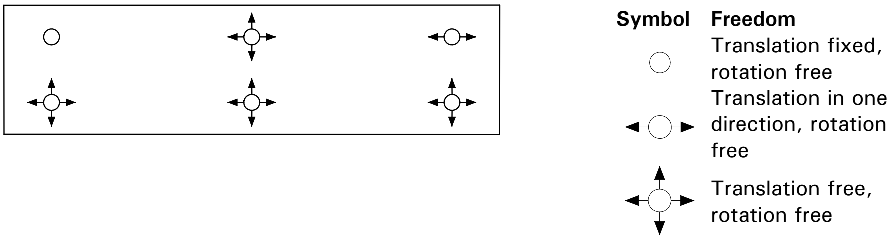


Figure 2.19 *Articulation of a 2-span bridge*

When the deck is curved in plan, the alignment of guided bearings must be considered carefully, since the deck tries to increase/decrease in radius as well as expand/contract in length. Examples of articulation for curved decks are included in GN 1.04.

For a fully integral bridge, there are no freedoms at the end supports but there is still a choice to be made about the freedom/restraint at intermediate supports; one guided bearing is usually provided at each support.

2.5.2 Intermediate supports

Multi-girder decks

Multi-girder decks are supported at intermediate positions on either leaf piers or individual columns. Individual columns under each girder can appear rather cluttered in some situations and an alternative arrangement is to put a column between each pair of girders and to use an integral crosshead between the girders

Ladder decks

Ladder decks are usually supported by columns directly under each main girder. This achieves an open appearance beneath the bridge. For river crossings, leaf piers may be preferred for hydrology reasons. Requirements for replacement of bearings may dictate the minimum size of columns, as it is preferable that jacks can be placed on the top of each column, so allowing the steelwork to be jacked off the columns.

Leaf piers, rather than individual columns, are sometimes used when the bridge bearings are located inboard of the main girders and so-called integral cross-heads are provided (see Figure 2.12).

Intermediate supports for bridges with integral abutments

There is no requirement for girders to be made structurally continuous with intermediate supports when a bridge is designed as an integral bridge. To do so adds complexity with little benefit and should be avoided. The reference to 'integral crossheads' above does not indicate integral construction between the sub- and superstructure.

2.5.3 End supports - non-integral abutments

For non-integral construction, bearings will be needed under the main girders and an expansion joint with inspection gallery will need to be provided. A typical arrangement (for a ladder deck bridge) is shown in Figure 2.20. In this figure the deck slab is shown with a downstand against the back face of the end cross girder facing the ballast wall, to reduce maintenance requirements. (In multi-girder decks a similar detail is used, with a trimmer girder below the end of the slab.)

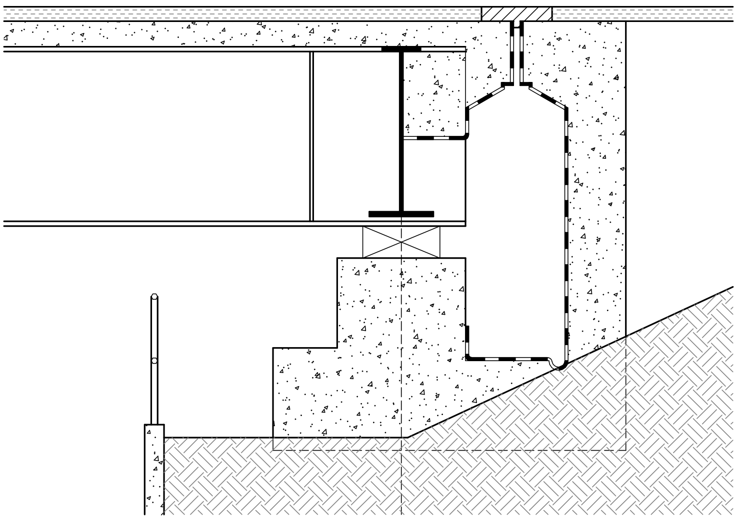


Figure 2.20 *Abutment gallery*

In ladder deck bridges where the main girders are widely spaced or the end of the deck is highly skewed, the vertical deflection of the end cross girder between two bearings might be greater than an expansion joint can accommodate (3 mm maximum for commonly used joints). If this is the case, one or more intermediate bearings should be provided under the cross girder. (But the economic case should be considered carefully; it may be cheaper to provide extra material in the girder or to encase it in concrete to increase stiffness.) If an intermediate bearing is provided, the bearing may need to be preloaded to avoid chattering[2] or to be restrained against uplift.

2.5.4 End supports - integral abutments

For bridges up to 80 m overall length, integral abutments can be used if the skew angle is not more than about 30°. There are three types of integral construction that are currently being used for composite bridges in the UK:

- Fully integral bridges - framed abutments
- Fully integral bridges - bank pad abutments
- Semi-integral bridges - with bearings

The forms of these abutments are discussed below; design and detailing issues are discussed in Section 9.

[2] The dynamic effects of traffic loading on the deck may at times cause upward load effects on the end cross girder and, if there is very little dead load on such a bearing, the end cross girder may deflect upward and lift off the bearing. Lift off and subsequent impact on closing is often referred to as 'chattering'. This behaviour is very onerous in terms of bearing life and must be avoided.

Framed abutments

Framed abutments are usually built with H-piles or reinforced concrete piles, with the piles inside sleeves (thus avoiding earth pressures on the piles as the bridge expands and contracts - see discussion in Section 9.1). A typical arrangement is shown in Figure 2.21, with a normal earth slope in front and in Figure 2.22 with a reinforced earth retaining wall. Typically, one or two piles are provided for each main girder in multi-girder bridges; for a ladder deck bridge of the same overall width, a similar total would be provided, though they might be concentrated around the positions of the main girders. Framed abutments are also built with reinforced concrete abutment walls on strip footings, although that form of construction is not discussed in this publication.

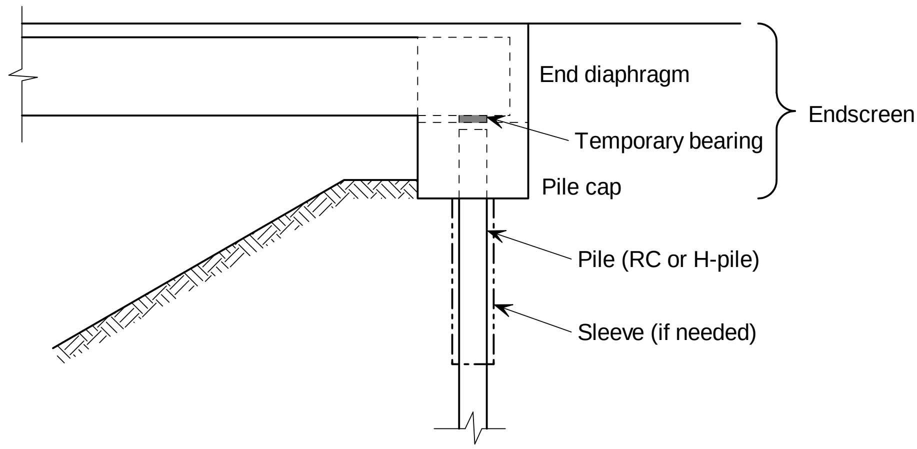


Figure 2.21 *Framed integral abutment - with normal earth slope*

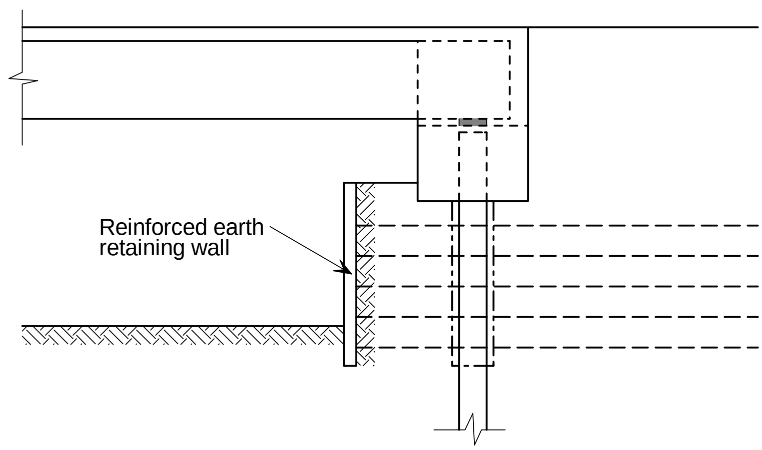


Figure 2.22 *Framed abutment – with reinforced earth retaining wall*

In principle, any type of bearing pile, including steel H-piles, can be driven into the ground and the endscreen wall cast around the tops of the piles. In practice, only a small number of bridges have been built with H-piles. Where construction has used H-piles, they have usually been encased in a pilecap just below the bottom of the main girders. Plates for temporary bearings are set into the pilecap and the endscreen wall is completed later, after the deck steelwork has been erected and the deck slab cast.

With fully integral construction, bracing for the construction condition may be arranged within the wall (and will be cast in) or just in front of it (but there must then be access for maintenance).

Integral bank pad abutment

In an integral bank pad abutment, an endscreen wall is cast around the ends of the girders and sits directly on the soil beneath. A typical arrangement is shown in Figure 2.23

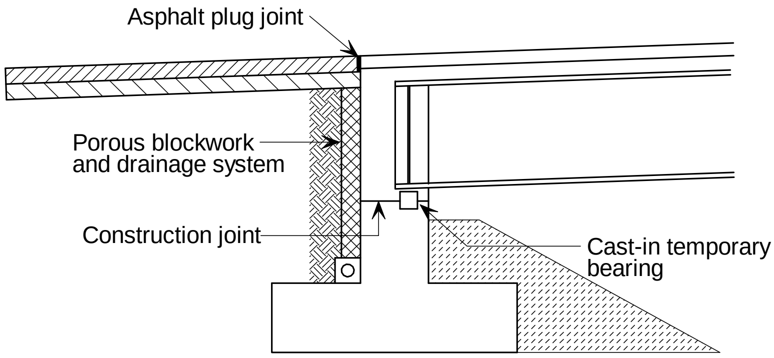


Figure 2.23 *Integral bank pad abutment*

Because the expansion and contraction of the deck causes the foundation to slide and rotate on the soil, the design bearing resistance of the soil has to be reduced; this type of abutment is better suited to situations where the soil is non-cohesive (or where cohesive material has been dug out and replaced with non-cohesive material).

Semi-integral abutment

In a semi-integral abutment there is an endscreen wall across the end of the deck but the girders are supported on bearings in front of the wall. A typical arrangement is shown in Figure 2.24. This form of abutment can be used either with side slopes in front of the abutment or behind a retaining wall. It is particularly suitable where there is a reinforced earth retaining wall. However, replacement of the bearings will require jacking and because of concerns about the forces involved and the movement at the interface with the soil, it is a less favoured solution (see further comment in Section 9.3).

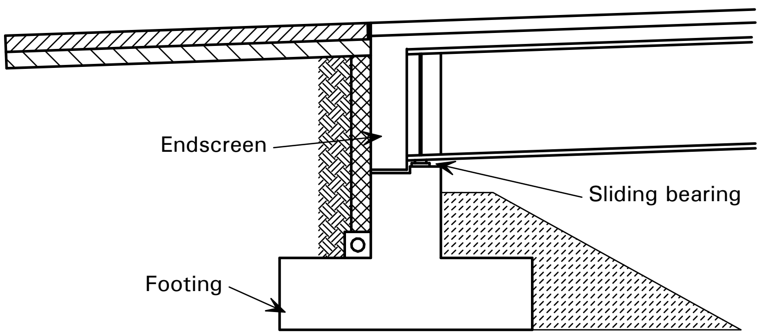


Figure 2.24 *Semi-integral abutment*

A semi-integral abutment is only suitable for up to about 15° skew because with larger skews the lateral component of earth pressure exerts large transverse forces on the bearings.

With semi-integral construction, the endscreen wall is usually connected to endplates across the ends of the girders. The endscreen wall will act as torsional restraint to the girders and as a trimmer beam. Some form of restraint to the main girders, either within the wall or in front of it, will be required for the construction condition.

With wide ladder decks, there is potentially a similar concern about excessive vertical deflection of the endscreen wall as noted above for the end cross girders in non-integral bridges but usually the wall is sufficiently stiff that deflections are small.

3 INITIAL DESIGN

3.1 General

In the initial design stage, the designer takes the outline requirements of the highway engineer (the highway layout, cross section and vertical profile) and derives a structural solution that suits the topography and restrictions of the site, whilst minimising both costs and risks. There may be little detailed calculation at this stage but there should be consultation with fabricators and contractors. Most bridge construction in the UK currently takes place under collaborative arrangements and thus access to fabricators and main contractors should be readily available to the designer. In the absence of a collaborative arrangement, designers should at least discuss the options with a fabricator at an early stage.

The following remarks relate principally to modestly sized highway bridge projects but many of the observations may be generally applied.

3.2 Design for construction

While minimising cost may be the most obvious consideration when embarking on the design of a highway bridge, the health and safety of all those concerned in the construction of the bridge and in its maintenance throughout its life is the responsibility of all those people making decisions about the procurement of the bridge. So, as well as aiming for a structurally efficient solution, the construction process and the hazards entailed must be fully appreciated from the outset.

3.2.1 Steelwork fabrication

Clean lines to the overall appearance and minimum use of complex details are most likely to lead to an economic and efficient bridge structure, though external constraints often compromise selection of the best structural solution.

The fabrication of the basic I-section is not expensive, especially with the use of modern semi-automatic girder welding machines (T and I machines). Overall fabrication cost is of the same order of cost as the material used. With the widespread use of computers in design and in control of fabrication shop machines, geometrical variations, such as curved soffits, varying superelevation, plan curvature and precambering, can be readily achieved with almost no cost penalty. Much of the total cost of fabrication is incurred in the addition of stiffeners, the fabrication of bracing members, butt welding, the attachment of ancillary items, and local detailing that leads to a significant manual input to the process. The designer can exercise freedom in the choice of overall arrangement but should try to minimise the number of small pieces that must be dealt with during the fabrication process.

Transportation by road imposes certain limitations on size and weight of fabricated assemblies. The most frequently noted limitation is a maximum length of 30 m, above which special notification and procedures apply. Nevertheless, UK fabricators are used to transporting longer loads – in exceptional cases girders well over 40 m long have been transported. See further comment in GN 7.06[2].

Expert advice should be obtained from fabricators to assist in the choice of details at an early stage in the design. Most fabricators welcome approaches from designers and respond helpfully to questions about their fabrication methods.

3.2.2 Erection scheme

A scheme for erection of all the major pieces of the bridge needs to be considered from an early stage. Access, temporary support arrangements, stability of the part-erected structure and the need to minimise work during road or railway closures can all have an effect on the form and detail of the structure.

Construction of a composite bridge superstructure usually proceeds by the sequential erection of the steelwork, usually working from one end to the other, followed by concreting of the deck slab and removal of falsework. However, situations vary considerably and constraints on access may well demand a sequence that differs considerably from the usual. In some cases the access constraints will determine which structural configuration can be safely and economically used.

In some circumstances, where access from below is difficult or impossible, launching from one or both ends may be appropriate. If so, this is likely to have a significant effect on girder arrangements and detailing - a uniform depth ladder deck arrangement is best suited to launching and a lower span/depth ratio may be needed. Advice should be sought from an experienced contractor.

Permanent connections on site are made using preloaded bolts, to achieve a slip resistant connection. It is much quicker to establish a secure connection using bolts than by welding. Welded joints are more expensive, more at risk of delay due to weather and more onerous on quality control on a small job but might be considered on larger jobs. One method or the other should be adopted throughout the bridge; it is normally uneconomic to use both methods.

The stability of girders during erection and under the weight of wet concrete will have a significant effect on the sizing of the top flange in midspan regions and, to a lesser extent, on the bottom flange adjacent to intermediate supports.

The main girders of multi-girder bridges are often lifted in braced pairs; the girders are then more stable than individual girders and installation of bracing members at ground level is less hazardous than at height.

Ladder decks are usually erected one girder at a time (main girders are usually of such proportions that they can be lifted singly, without the need for any temporary restraint systems, such as bowstring bracing to the top flange), although sometimes part-span lengths of girder are erected with their cross girders already in place. Occasionally, complete decks have been assembled close to the site and transported into position (usually because of restrictions on closure or possession times). The twin girder arrangement is also well suited to launching. During concreting, partial restraint of the main girders against lateral torsional buckling is provided by the cross girders; additional plan bracing is not normally provided.

If girders are erected by launching, some temporary plan bracing may be needed (see Figure 2.7, for example). Note that where the main girders are to be erected individually they will require torsional restraint at supports before the cross girders are connected. There needs to be sufficient space on the permanent supports to provide this restraint, or separate temporary works will be needed.

General guidance on the erection of bridge steelwork is given in BCSA publication 38/05[4].

3.2.3 Slab construction

The deck slab of a composite bridge is normally cast *in situ*, on either temporary or permanent formwork. Traditionally, timber formwork, fitted between the erected girders, was most commonly used. Recently, the use of permanent formwork has become common: it avoids the costly and potentially hazardous operation of stripping out temporary formwork after casting.

Precast plank permanent formwork ('Omnia' type) is now very commonly used with both multi-girder and ladder deck construction, though it is slightly better suited to ladder deck construction, where the top flanges of the cross girders are all in a common plane (in multi-girder construction, the flanges are normally horizontal transversely but the slab follows the camber of the road). Precast plank permanent formwork can be used for slab spans up to about 3.6 m (with relatively wide top flanges to the cross girders, 600 mm wide or greater, a girder spacing of up to 4.0 m is possible).

Reinforced fibre panel permanent formwork is also used: it can span up to about 4.0 m, which would allow a girder spacing of a little over 4.0 m. (But note that slab thickness may need to be slightly greater with wider girder spacing.)

For cantilevered deck slabs, proprietary 'clip-on' falsework systems have recently been developed, in place of individually designed and built falsework. These systems provide both the formwork for casting the cantilever slab, including edge beam, and a safe access (with handrail) beyond the end of the cantilever. There are relative simple devices by which to connect to the face of the outer girder without damaging the protective coating or contaminating the surface of weathering steel.

It is common not to pour the concrete over the full length of the bridge at one time but to place concrete over part lengths, in a number of stages. This choice is partly for practical reasons and partly, by concreting midspan regions first, to minimise hogging moments due to dead load. With integral abutments, the endscreen walls are usually poured last, so that no restraint moments are transferred into the abutment due to the weight of the concrete.

With multi-girder decks, the deck slab can be concreted either across the full width to the outer girders at each stage or in part-width stages. Cantilever falsework on the outer girder applies considerable torque to the outer girder, resulting in difficult-to-predict torsional deformations; the cantilevers are therefore often cast after the rest of the deck, particularly if they are long.

In ladder deck construction the restraint against twist of the main girder that is provided by the cross girder connections ensures that there is stiff restraint to the cantilever falsework during concreting and it is common to cast the slab full width.

Occasionally, full-thickness precast deck slab units have been used, minimising the amount of in-situ construction. However, there is still concern in some quarters about the performance of the in situ joint at SLS and this form may only be used with the agreement of the highway authority. Further research might alleviate these concerns. See discussion in SCI publication P316[5].

3.3 Design for in-service maintenance

Bridges are expected to have a long life (the 'design life' given in the National Annex to BS EN 1990 is 120 years) but they will certainly require maintenance within this period. The design, and in particular the detailing, should recognise the need for durability and facilitate whatever maintenance will be necessary. Particular issues to be addressed include:

- Access for repainting.
- Provision of drainage arrangements that require minimal maintenance and which do not cause durability problems if they fail.
- Facilities for bearing replacement.

Client authorities will normally establish a programme of regular inspection and maintenance. Access to critical areas should either be provided or be possible with the minimum of temporary works, although security must also be considered, to avoid unauthorised access.

The CDM Regulations require the assessment of hazards during maintenance work. The design must be such as to avoid or reduce, as far as practicable, risks during maintenance.

3.3.1 Corrosion protection

Traditionally, steel bridges have been protected against the effects of corrosion by the application of protective coatings. Coating systems have been developed that have a long life (over 30 years to first maintenance is now considered achievable), using products that comply with current health and safety requirements and environmental regulations.

To ensure complete and reliable application and to maximise the life of protective coatings, the arrangement and detailing of the steelwork should be such as to avoid any features that would limit access for proper application and maintenance or which would trap water and dirt in service. General guidance on detailing and the selection of a coating system is given in a publication by Corus[22]. With modern coating systems, there need be no allowance for corrosion of the structural steel material.

3.3.2 Weathering steel

Since 2001, an alternative to the application of protective coatings has become common: the use of weathering steel. Weathering steel is a special alloy of carbon steel that forms a stable and tightly adhering oxide layer (or 'patina') when subject to alternate wetting and drying. Unlike ordinary rust, the patina does not fall off the surface and it prevents further oxidation. Weathering steel does not require any maintenance, provided that it has been used in appropriate circumstances.

The use of weathering steel results in a slightly higher material unit cost and the additional cost of a 'corrosion allowance' to the steelwork (a small addition to the thickness required for design purposes) but saves the cost of applying a protective coating: the saving usually outweighs the extra costs. The benefits can be further justified in terms of whole life costs, which are particularly high when traffic delay costs during maintenance are included (the ‘cost’ of repainting is then very high). For guidance on the use of weathering steel refer to GN 1.07[2], a Corus publication[18] or ECCS publication 81[19]. Table NA.1 to BS EN 1993-2 gives values of sacrificial thickness for weathering steel, according to atmospheric corrosion classification.

3.3.3 Bearing replacement

The design must allow bearings, particularly sliding bearings and elastomeric bearings, to be replaced during the life of a bridge. It is relatively straightforward to stiffen the web of the main girders to permit jacking to replace bearings but there does need to be sufficient space (on top of columns, etc.) on which to sit the jacks[3]. The need for temporary support adjacent to an intermediate support, off which to jack the structure, should be avoided, because of the substantial costs and hazards that are introduced.

Jacking under a pier diaphragm (rather than under the main girder) should generally be avoided, unless integral crossheads have been chosen for other reasons, because it requires a stronger diaphragm and connection detail, at significant extra cost. In ladder deck construction, pier diaphragms can be designed for jacking loads, even if they are not integral crossheads, but it is difficult to provide jacking arrangements for a knee-braced diaphragm.

3.4 Choice of structural configuration

For a typical highway project, the choice of bridge type will be between a multi-girder and a ladder deck configuration.

The advantages of a ladder deck configuration are:

- A reduced tonnage of steel, relative to a multi-girder deck (although fabrication costs per tonne may be higher).
- Well-suited to efficient slab construction (uniformity in thickness, easily detailed to suit precast permanent formwork and full-width placing of concrete).
- The cross girders provide regular stiff restraint to cantilever construction, facilitating the use of either ‘clip-on’ cantilever falsework or precast cantilever units.
- Where horizontally curved girders are needed, the regular spacing of cross girders easily provides the restraint to the bottom flanges.
- Needs only two columns at each intermediate support, avoiding leaf piers and achieving a more open appearance.
- Reduced maintenance liabilities (less surface area of steelwork, fewer small bracing elements, fewer bearings).

The advantages of a multi-girder configuration are:

- Smaller piece sizes (of main girders), thus reducing crane requirements.

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[3] It is important that the spread of load from the jacks lies wholly within the reinforcement cage at the top of the column or leaf pier, otherwise spalling will occur.

- Braced pairs of girders require no additional temporary bracing to top flanges.
- Fewer bolted connections on site.
- Good load distribution (through transverse bending of the slab).
- Readily adaptable to any bridge width.
- Shallow construction depth can be achieved without resorting to excessively large flange plates.

Influence of substructure constraints

The form of the substructure at intermediate supports, whether for reasons of appearance or construction, often has a strong influence on the form of the superstructure. For example, a low clearance bridge over poor ground might use multiple main girders on a single broad pier, whereas a high level bridge of the same deck width and span over good ground might use a ladder deck with twin main girders on individual columns.

Influence of skew

Highly skewed bridges are sometimes unavoidable but it should be noted that the high skew leads to the need for a greater design effort, more difficult fabrication and more complex erection procedures. In particular, the analytical model, the detailing of abutment trimmer beams, pre-cambering and relative deflection between main beams must all be considered carefully.

Influence of requirements for drainage

Drainage of the roadway on the bridge can often be achieved solely by drainage channels on the bridge deck but drainage runs may also be required below the deck slab. Arrangements for such drainage runs may well influence the positioning of main girders in the cross section and possibly the detailing of cross girders or intermediate bracing.

3.5 Preliminary sizing - material selection

Steel

The main structural steel members in bridges are usually grade S355 (to EN 10025), which is more cost effective than grade S275. Higher strength steel grades (S420 and S460) are available and can be used in designs to EN 1993-2 but they are more expensive and have not yet been used in the UK to any significant extent in bridgework.

For guidance on the range of steel plate available, including the availability of toughness sub-grades and weathering steel, refer to advice from Corus[8]. Note that maximum plate length is typically 18 m (longer flange lengths are achieved by butt welding in the fabrication works) and there is a limit on the weight of an individual plate (i.e. on the combination of length, thickness and width) which may further limit the length for very thick plate.

For bracing members, rolled section angles, channels and I sections may be used, if the steelwork is to be given a protective coating. If weathering steel is to be used, there are no available rolled sections; fabricators can, however fabricate similar sections from plate; seek advice from a fabricator about what sections are economically feasible.

Selection of the particular sub-grade (for toughness) does not usually need to be considered in the initial design unless availability is likely to be a problem. For thicknesses of 60 mm and more, delivery periods are longer and certain combinations of grades and toughness are not available from Corus

Advice about the steel materials available and the choice of sub-grade is given in GN 3.01. Guidance on availability and delivery times for specific products and sizes can be obtained from Corus; steelwork contractors can also offer advice. If the construction period is limited, delivery times may affect the choice of components.

If weathering steel is used, a corrosion allowance has to be added to the thickness used for structural design (see Section 3.3.2).

Concrete

The concrete in the deck slab is usually class C40/50 to EN 206 (i.e. with a cylinder strength of 40 MPa). Class C30/37 would be strong enough in most cases but does require slightly greater cover. The choice between these two classes does not normally influence the initial design of the steelwork significantly.

3.6 Preliminary sizing - multi-girder bridges

3.6.1 Girder spacing

Multi-girder bridges

The total transverse moments in the slab of a multi-girder deck are not particularly sensitive to girder spacing in the range 2.5 to 3.8 m (the increase in local moment with span is more or less balanced by a reduction in the moment arising from the transfer of load from one girder to the next). It is advantageous to choose a spacing as high in the range as possible, consistent with other geometrical considerations and with the form of slab construction (see comment about the use of permanent formwork in Section 2.3).

In selecting a suitable girder spacing, attention must be paid to the cantilevers at the edges of the deck. The cantilever length from the outer girder centreline should normally be restricted to about 1.5 m to 2 m, including the edge beam. Whilst the cantilever could be increased to about 2.5 m if it carries a footway that is protected by a crash barrier (thus avoiding local accidental vehicle loading), use of a proprietary falsework system during construction also tends to lead to a practical limit of 2 m.

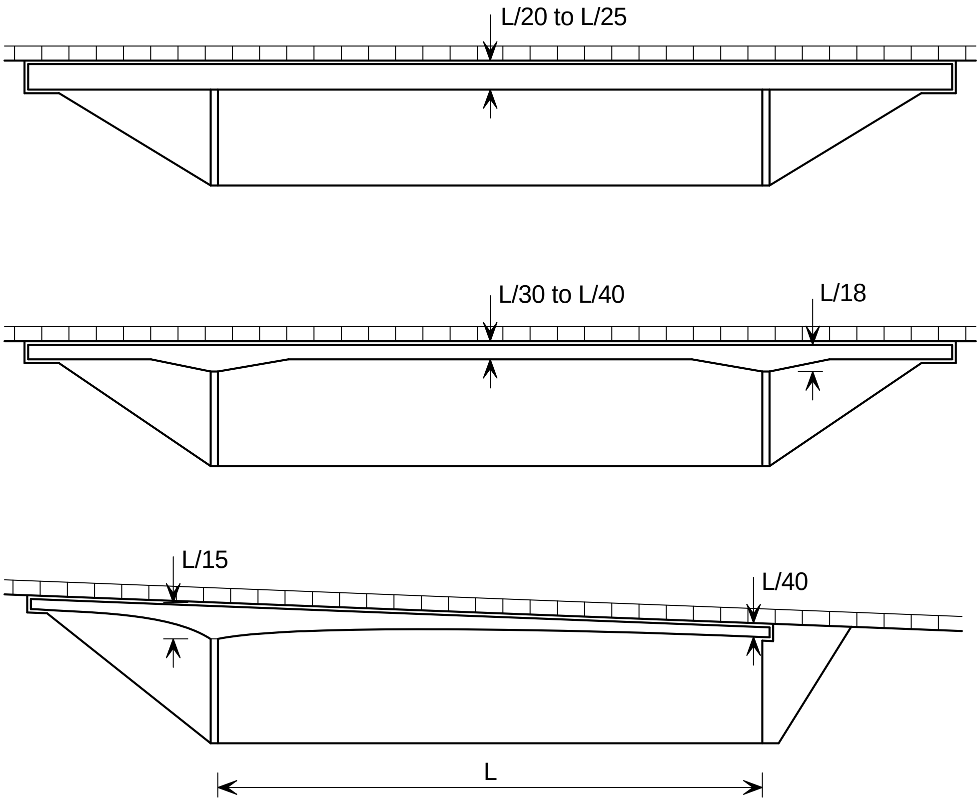
Where ‘very high containment level’ barriers are specified, the length of cantilevers may need to be restricted, and the slab increased in thickness locally.

It is preferable to use an even number of girders so that they may be paired during construction, but an odd number can be used, if due provision is made for erection of single girders.

3.6.2 Girder profile

For simple spans over about 25 m, a construction depth (top of slab to underside of beam) of between about 1/18 and 1/30 of the span can be achieved with fabricated beams, though the most economic solution will be toward the deeper end of this range. For shorter spans, the depth is likely to be proportionately greater, particularly for spans under 20 m.

For composite continuous spans with parallel flanges, the construction depth (again, from top of slab to underside of beam) is typically between 1/20 and 1/25 of the major span. The use of curved soffits or tapered haunches can reduce construction depth at midspan, at the expense of increased depth at the internal supports. A selection of typical arrangements is given in Figure 3.1.



NOTE: Depth measured to top of slab

Figure 3.1 *Typical span/depth proportions for continuous spans in a multi-girder bridge*

3.6.3 Flange and web sizes

Experience and a few rules of thumb can often be used for an initial selection of sizes. Because the weight of the steelwork contributes little to overall design moments, the selection can be quickly refined. One such set of simple rules is given in Appendix B.

Alternatively, initial plate sizes may be determined from a set of charts for preliminary design to the Eurocodes; these are available from Corus along with an associated spreadsheet design tool (website: www.corusconstruction.com).

Spans that must be fabricated with more than a single length to each main girder give the opportunity to vary the girder make-up in the different pieces required for each span. Maximum length of the pieces is influenced by transportation (loads over 30 m long require special arrangements) and by the length of plate that is available (see further comment below).

3.6.4 Slab thickness

For initial design, choose a slab thickness of 250 mm. This can be refined in detailed design. If the cantilever carries a high containment level barrier, use an initial thickness of 330 mm (for a 2.0 m cantilever) at the root of the cantilever, reducing to 250 mm at the first internal main girder

3.7 Preliminary sizing - ladder decks

3.7.1 Girder spacing

Arrange the main girder spacing such that cantilevers are about 1.5 m to 2 m long. Main girders should be between about 5.5 m and 18 m apart. For a wider deck, use either two separate ladder decks or two sets of ladder deck steelwork with a common slab. If very high containment barriers have to be provided at the edge of the deck, choose a shorter cantilever or thicken the slab.

Choose a cross girder spacing between 3.3 and 4.0 m. Spacings up to 4.0 m can be achieved with proprietary precast plank or reinforced fibre panel permanent formwork. Note that if the bridge is curved in plan, the cross girder spacing will be greater on the outside of the curve and formwork lengths have to vary across the width of the deck.

3.7.2 Girder profile

Main girders are usually of uniform depth, although haunched girders may suit some situations. For uniform depth girders, the overall depth (girder + slab) should normally be between about 1/15 and 1/25 of the major span (1/25 can appear quite slender). For wide decks, the depth should be toward the deeper end of the range.

Cross girders should have a depth of between about 1/12 and 1/18 of the span between main girders. Usually they will have a straight bottom flange, but the top flange will normally follow the transverse profile of the road.

3.7.3 Flange and web sizes

Choose initial flange sizes on the basis of previous experience or using simple line beam models (the proportion of load carried by each girder is easily determined by a 'statics' distribution transversely). As mentioned in relation to multi-girder deck bridges, charts for preliminary sizing are available from Corus.

For long spans, the flanges may be quite thick, up to 80 mm thick or possibly even slightly greater. The use of a higher strength grade (higher than S355) may be appropriate in some circumstances but higher strength grades are, at present, significantly more expensive and less readily available; delivery time is also likely to be longer. Grade S355 has been almost exclusively used for ladder deck bridges in the UK.

For cross girders, choose a plate girder section on the basis of the cross girders acting as simply supported beams.

3.7.4 Slab thickness

Choose a slab thickness of 250 mm for a cross girder spacing up to about 3.8 m, 260 mm for a 4.0 m spacing, with due regard to cover required for durability (and to the grade of the concrete, which is normally C40/50).

Where a very high containment level barrier is to be carried, there will be large moments and lateral forces (outward forces, causing tension in the slab) to be sustained, with consequences on slab thickness. To achieve a 2 m cantilever whilst carrying such a barrier, a slab thickness of up to about 330 mm may be needed. This thickness will need to be tapered back to the regular slab thickness inboard of the outer girder over a length of 1 to 2 m. Consideration must also be given to the means of transfer of the transverse moment into the ends of the cross girders.

4 DESIGN STANDARDS

4.1 The Eurocodes

4.1.1 Overview

List of Eurocodes

The Eurocodes are a set of structural design standards, developed by CEN over the last 30 years, to cover the design of all types of structures in steel, concrete, timber, masonry and aluminium. There are ten Eurocodes, each published in a number of separate Parts. The ten Eurocodes are:

- Eurocode: Basis of structural design
- Eurocode 1: Actions on structures
- Eurocode 2: Design of concrete structures
- Eurocode 3: Design of steel structures
- Eurocode 4: Design of composite steel and concrete structures
- Eurocode 5: Design of timber structures
- Eurocode 6: Design of masonry structures
- Eurocode 7: Geotechnical design
- Eurocode 8: Design of structures for earthquake resistance
- Eurocode 9: Design of aluminium structures

A full list of the Eurocode Parts is given in Appendix A.

The Eurocodes are designated by CEN as EN 1990 to EN 1999 respectively; when published by an individual national standards body, the designation is prefixed by the national standards body's usual designation – thus in the UK they are published as BS EN 1990 to BS EN 1999. [*But note that references to the Eurocode Parts in this publication generally omit the BS prefix.*]

The documents produced by CEN leave certain matters, including the setting of the reliability level, through the values of partial factors, for national choice. The Eurocode Parts are each accompanied by a National Annex (NA) which makes the necessary choices and sets the values of the partial factors. [*In this publication, references to the UK National Annexes are given as, for example, "the NA to BS EN 1993-1-8".*]

The Eurocodes became available for use in the UK as soon as they and their NAs were published by BSI and it has been agreed by all the members of CEN (of which BSI is the UK member) that corresponding national standards, such as BS 5400, will be withdrawn in 2010. The use of BS 5400 is not covered in this publication.

Format of the Eurocodes

The Eurocodes are intended to cover all forms of construction 'onshore' and the most common materials used in construction – concrete, steel, masonry, timber and aluminium. To achieve such a wide coverage, aspects which are common to more than one form or material have been set out in separate Parts; thus there should be no duplication of rules and a consistent level of reliability should be achieved wherever appropriate.

The design basis for all forms and materials is set out in EN 1990. This document has two Annexes, one for buildings and one for bridges.

Actions on structures are dealt with in EN 1991. It has seven Parts dealing with general actions (such as self weight, wind loading, etc.) and three Parts related to specific forms of construction (bridges, cranes, silos etc.).

Design rules for concrete, steel, composite construction, timber, masonry and aluminium are covered separately in EN 1992 to EN 1996 and EN 1999 respectively. For each material Part 1 comprises 'general' parts that cover rules common to different forms of construction and Part 2 is specific to bridges.

The CEN documents are only able to recommend design levels of reliability (achieved through the use of partial factors) to ensure adequate safety during construction and service, because safety is a matter for national determination. To enable the use of the Eurocodes in an individual country, the EN documents must be published by the national standards body, each accompanied by a National Annex that either accepts the CEN-recommended values of the partial factors or substitutes values that are deemed appropriate in that country.

The Eurocode Parts contain two distinct types of statement – 'Principles' and 'Application Rules'. The former must be followed, to achieve compliance; the latter are rules that will achieve compliance with the Principles but it is permissible to use alternative design rules, provided that they accord with the Principles (see EN 1990, 1.4(5)[4]). National Annexes may not vary the Principles nor delete the application rules, though in some cases a choice between application rules is allowed.

The Eurocodes are not comprehensive in detail. Designers familiar with UK Codes of Practice will find many 'detailed design rules' missing from the Eurocodes. These omissions can be covered by non-contradictory complementary information (NCCI). NCCI material can be produced by any organisation, it has no special status, it merely has to avoid conflict with the principles and the application rules in the Eurocode. (The responsibility for ensuring that any information used in design does not conflict with the Eurocodes rests with the designer.)

Eurocode Parts for bridges

The bridge design rules are a sub-set of each Eurocode, making reference to general rules but not repeating them; thus the designer will need to 'merge' the bridge rules and the general rules. (EN 1994-2 is an exception; the merging with EN 1994-1 has already been made in EN 1994-2.). For steel design, there are very many 'General' Parts to be consulted. Together with separate substantial Parts for design basis and loading, plus National Annexes and NCCIs, there are over 20 separate Parts to refer to for the design of a composite bridge.

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[4] A Note to that clause states: "If an alternative design rule is substituted for an application rule, the resulting design cannot be claimed to be wholly in accordance with EN 1990 although the design will remain in accordance with the Principles of EN 1990."

Generally, the application rules in the Eurocodes are more analytical and text-book-like than the 'simple design rules' (often empirical) that have traditionally been used in the UK. The rules are therefore 'more exact' in some ways but they may be less easy to apply in initial design stages.

Some of the terminology used in the Eurocodes will be new to UK designers – terms have been chosen carefully, for clarity and to facilitate unambiguous translation into other languages. The presentation of symbols has also been rigorously defined (although not always consistently applied) and some conventions are different.

Eurocode terminology

The chief differences in terminology are:

Actions	=	loads, imposed displacements, thermal strain
Effects	=	internal bending moments, axial forces etc.
Resistance	=	capacity of a structural element to resist bending moment, axial force, shear, etc.
Verification	=	check
Execution	=	construction – fabrication, erection

Eurocode symbols

The Eurocode symbols should follow the ISO convention of using italic fonts for symbols for variables and upright fonts for constants, text, abbreviations etc. In practice, the observance of this convention has been somewhat inconsistent.

Symbols are usually denoted by a single letter (Roman or Greek alphabet) and are differentiated by subscripts. Because the coverage of the Eurocodes is so wide, there has been a standardization of the main symbols and the use of compound subscripts to distinguish between symbols for related variables.

Some of the main variables used in EN 1993-1 are:

M	Bending moment	A	Area
N	Axial force	I	Second moment of area
V	Shear force	W	Section modulus
T	Torsional moment	$\overline{\lambda}$	Non-dimensional slenderness
F	Force (used for resistance of bolts)	χ	Reduction factor (for buckling)

Some of the main subscripts are

Ed	design effect	b	buckling
Rd	design resistance	cr	critical
Rk	characteristic resistance	op	out of plane
el	elastic	c	related to cross section
pl	plastic	y, z	related to y-y or z-z axis

Thus:

N_{Ed} is the design value of axial force (an effect)

$N_{c,Rd}$ is the design resistance of the cross section

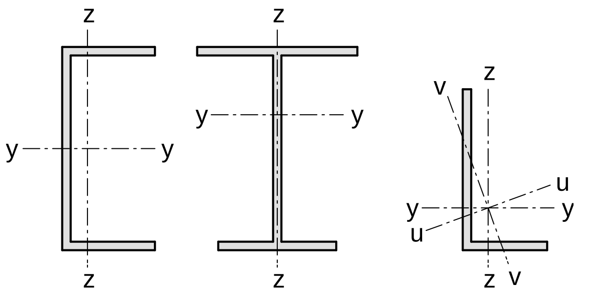
$N_{b,Rd}$ is the design buckling resistance (of a member)

M_{cr} is the elastic critical moment (due to lateral torsion buckling of a beam)

Generally, the rigorous use of these subscripts does contribute significantly to clarity in calculations: it should be clear from the subscripts whether the variable is an effect or a resistance, gross or effective, elastic or plastic, related to a plate of a stiffener, etc. although this does sometimes result in lengthy subscripts.

Geometrical axes

The sign convention for axes differs from traditional UK convention but is compatible with most software analysis packages.



x – along the member

Reference standards

The Eurocode Parts include lists of CEN reference standards (generally with an EN designation, sometimes with an EN ISO designation). Those standards have, in most cases, already been published in the UK by BSI with a BS EN designation. In this publication, only the simple EN designation is used (for example as a reference to EN 10025-2); the requirements are unchanged when published as a BS EN document.

4.1.2 Design basis

EN 1990 sets out the principles of limit state design, the basic variables involved and the procedures for verification by the partial factor method. It applies partial factors to actions (loads) and to resistances (strengths) to give 'design values' that will achieve an overall level of reliability for a given return period (the design life of the structure).

Ultimate Limit State

At the ultimate limit state it must be verified that the design value of the effect of actions (symbolically expressed by the subscript Ed) does not exceed the design value of the corresponding resistance (expressed by the subscript Rd).

The design values of the effects of the actions (i.e. the internal bending moments, axial forces etc.) are determined for the combinations of actions that can occur simultaneously. The basic expression for the effect of the fundamental combination of actions is:

$$E\left(\sum \gamma_{\mathrm{G},j} G_{\mathrm{k},j} + \gamma_{\mathrm{P}} P + \gamma_{\mathrm{Q},1} Q_{\mathrm{k},1} + \sum_{i>1} \gamma_{\mathrm{Q},i} \psi_{0,i} Q_{\mathrm{k},i}\right)$$

Where $G_{\mathrm{k},j}$ is the characteristic value of the j-th permanent action, P is the permanent action caused by controlled forces or deformation (such as prestressing), $Q_{\mathrm{k},1}$ is the characteristic value of the 'leading' variable action and $Q_{\mathrm{k},i}$ is the characteristic value of the i-th accompanying variable action. The $E(\ )$ 'denotes the effect of' and thus the expression represents the summation of all the effects due to factored values of actions. The partial factors γ take account of uncertainties in the values of the actions and in modelling them; the combination factors ψ allow for the lower level of accompanying action (lower than its characteristic value) that is likely to coexist when the leading action is at its characteristic value.

There are similar expressions for combinations of actions in accidental and seismic situations. In those combinations the partial factors are set at unity and different combination factors apply to these design situations.

Serviceability Limit State

At the serviceability limit state it must be verified that the design value of the effect of actions does not exceed some limiting criterion, such as the onset of permanent deformation.

There are three combinations of actions to consider at the serviceability limit state: characteristic, frequent and quasi-permanent. For bridges, the characteristic combination is used for checking that no inelastic response occurs; the frequent combination is used if deflection needs to be checked (this includes evaluation of dynamic response). The quasi-permanent combination relates to long-term effects; for bridges, provided that the appropriate modulus of elasticity is used for long-term actions, this combination only needs to be considered when determining crack widths in concrete.

For the characteristic loading combination, the characteristic values of actions are used but all the γ factors are taken as unity. Thus the expression for the effect of actions becomes:

$$E\left(\sum G_{\mathrm{k},j} + P + Q_{\mathrm{k},1} + \sum_{i>1} \psi_{0,i} Q_{\mathrm{k},i}\right)$$

For the frequent loading combination, the actions are again unfactored and the effects are reduced by application of ψ factors (less than unity).

$$E\left(\sum G_{\mathrm{k},j} + P + \psi_{1,1} Q_{\mathrm{k},1} + \sum_{i>1} \psi_{2,1} Q_{\mathrm{k},i}\right)$$

For the quasi-permanent loading combination, all variable actions are considered as ‘accompanying’ (but since, for bridges, the NA to BS EN 1990 sets $\psi_2 = 0$ for all except thermal actions, this combination is in essence ‘dead load only’).

$$E\left(\sum G_{\mathrm{k},j} + P + \sum_{i\geq 1} \psi_{2,1} Q_{\mathrm{k},i}\right)$$

4.1.3 Design rules for bridges

The particular rules relevant to composite highway bridges, and the Parts of the Eurocodes from which they come, are discussed separately in Sections 5 to 8.

4.2 Complementary Information

Published complementary information ranges from general information about behaviour and analysis in textbooks to information that has been specifically written to complement the Eurocode documents.

In the UK, BSI are issuing 'Published Documents' (numbered as PD xxxx) that are referred to in the National Annexes. These PD documents are intended to 'fill the gaps' between the Eurocode rules and the more comprehensive guidance for designers that was in the former British Standards. Although published by BSI, PD documents are not Standards and do not have normative status.

Many organizations are publishing 'NCCI' documents. Whilst these may be helpful in relation to design in accordance with the Eurocodes, they have no special status and can only be used in that context as long as they do not conflict with either the Eurocodes or the National Annex for the relevant Part.

4.3 Client requirements

In several places in the Eurocode documents there is reference to the 'National Authority' or similar phrase. In the UK there is no single body with statutory responsibility for the safety of highway bridge structures; reference has to be made to the relevant technical approval authority. The four Overseeing Organizations responsible for motorways and trunk routes in England Scotland, Wales and Northern Ireland publish the Design Manual for Roads and Bridges (DMRB) which historically has provided both normative Departmental Standards (BD documents) and informative Advice Notes (BA documents). The DMRB documents which relate to design of bridges are in the process of being rationalised to essential 'Clients' Core Requirements' that will effectively implement the Eurocodes and the associated European standards. Client organisations such as local authorities may wish to adopt the same approach in designs and to make use of these DMRB documents.

4.4 Execution Standards

During design, the designer makes certain choices about the strength and characteristics of the structural elements. The design rules necessarily make some presumptions about the quality of the materials and components, and about the quality of the workmanship. The Eurocodes therefore make reference to execution standards and product standards (generally referred to as specifications). As well as nominating the specifications that are required for construction of the particular structure, the designer has to inform the constructor which particular strength grades and quality classes are required; this information is usually communicated by means of notes on drawings and a project specification (an 'execution specification' in CEN terminology).

For the actual construction processes for a composite bridge, there are two main execution standards, one for steel and one for concrete.

Steel structures

The fabrication, assembly and erection of steel structures is covered by EN 1090-2[9]. This standard is a comprehensive document that covers all types and qualities of structure and there is no separate part for bridge structures; consequently, the size of the document is large. For bridge construction in the

UK, a 'model project specification'[10] has been produced by the Steel Bridge Group; it facilitates the identification of the rules pertinent to a bridge structure and guides the designer in selecting appropriate quality requirements for the particular structure, where choices are allowed.

EN 1090-2 has no National Annex; it replaces the British Standard BS 5400-6.

Concrete structures

The in-situ construction of concrete structures (including the erection of precast units) is covered by EN 13670. This Standard, which is due to be published in early 2010, leaves much of the detail to be given in the execution specification. Reference should be made to the concrete section of the Specification for Highway Works (Series 1700).

4.5 Product standards

Steel

The product standard for structural steel is EN 10025[11]. It is in six Parts, of which Part 2 (non-alloy steel) and Part 5 (weathering steel) are the most likely to be referenced for highway bridges.

Bolts

Generally, the product standards for bolts are given in EN 1993-1-8, clause 2.8 (and in its NA). The product standard for preloaded (friction grip) bolts is EN 14399[12].

Bearings

The product standard for bearings is EN 1337[13]. This Standard comprises 11 Parts, covering the various types of bearings.

Concrete

The 'product' standard for concrete is EN 206[14], covering concrete for both precast units and in-situ construction. Guidance on use of this Standard is given in BS 8500[15] and IAN 95/07[16]. Precast units for bridges will be covered by EN 15050; general rules for precast products are published in EN 13369.

4.6 CDM Regulations

Health and Safety considerations, such as those set out in the CDM Regulations[17], must lead to an assessment, during conception, design and later, of the risks at all stages of the construction, use, repair and final removal/ demolition of a structure. The aim should be to eliminate, or reduce to an acceptable level, the identifiable risks.

Designers are therefore required to anticipate how a structure will be built and to assess the risks involved in the construction, to consider how the structure will be maintained and, eventually, how it will be taken out of use. Arrangements must be made for all pertinent information to be passed on to others who must work on the structure, including those charged with future maintenance and the owners.

5 CALCULATION OF ACTION EFFECTS

5.1 Global analysis

Elastic analysis

EN 1993-2, 5.4.1 recommends the use of elastic global analysis, except possibly for accidental design situations. Although elastic analysis is recommended, the effects of settlement, differential temperature and shrinkage may be ignored at ULS if all the cross sections are Class 1[5] (this effectively means that those effects may be redistributed by plastic bending). The UK National Annex states that circumstances where plastic methods of global analysis are acceptable should be specified for the particular project.

Although EN 1994-2, 5.4.1.1 does not exclude the use of plastic global analysis at the ultimate limit state, it is easier, and consistent with EN 1993-2, to use only global elastic analysis for all design situations for composite highway bridges.

In EN 1994-2, the effects of shrinkage and differential temperature may be ignored at ULS if all the sections are either Class 1 or 2 sections, but only if no allowance needs to be made for lateral torsional buckling. This is more stringent than EN 1993-2.

For global analysis, EN 1994-2 refers to the use of the 'equivalent steel section' properties, notably the second moment of area, I_1. The determination of the 'equivalent' value is not explicitly defined in EN 1994-2, although there is reference to short- and long-term modular ratios in 5.4.2.2. It is not stated but it can be implied that EN 1994-2 requires separate global models (with short- and long-term properties) for short- and long-term effects. Note that 5.4.2.4(2) allows the effects of staged construction to be neglected at ULS where all cross sections are Class 1 or 2 and there is no allowance for lateral torsional buckling, but this would only apply to composite bridges (of such sections) of single span or of continuous spans where the bottom flanges are sufficiently restrained that they can develop the full cross sectional resistance. Even then, modelling of staged construction is still needed for verification at SLS.

Effects of shear lag and local buckling

For composite structures, EN 1994-2 generally requires shear lag to be taken into account in determining the distribution of bending moments at both ULS and SLS and in 5.4.1.2 it provides simple rules for determining effective width of flanges; these effective widths, which depend on 'equivalent span', are shown diagrammatically in Figure 5.1 of EN 1994-2.

Essentially, the rules give values of effective width at each midspan and support position; the midspan values may be assumed to apply over the central half of the span and the effective width may be assumed to vary linearly between the midspan value and the support value over the end quarter-lengths of the span. EN 1994-2, 5.4.1.2 allows the midspan values to be used over the full span when elastic analysis is used (which in practice is always the case). Use of this

[5] For an explanation of the classification system, see Section 6.1.1.

constant effective width will give slightly higher hogging moments over intermediate supports.

The effect of cracking of concrete is allowed for in most cases by using cracked section properties either side of internal supports. Transverse composite members (such as in ladder decks) are assumed to be uncracked.

For steel beams, allowance for shear lag in wide flanges is given by EN 1993-1-5. Rules for effective width and for stress distribution across the flange are also given in EN 1993-1-5, 3.2; EN 1994-2 refers to that clause for determination of the variation of stress distribution (once effective widths have been calculated in accordance with EN 1994-2).

Effects of cracking of concrete

EN 1994-2, 5.4.2.3 offers two options: one is that first an uncracked analysis may be carried out and the extent of cracking determined (when the concrete tensile stress exceeds a certain value), followed by another analysis using cracked section properties in these regions; the second allows a simpler one-stage method. The simpler method assumes that, in the global analysis for both ULS and SLS, the concrete is cracked adjacent to internal supports over 15% of the span; this method is allowed subject to the ratio of the lengths of spans being at least 60%. *Note that the 60% value is rather arbitrary and that EN 1993 and EN 1994 generally presume beams of uniform cross section. Where the girder section (and thus the beam stiffness) varies significantly between adjacent spans it would be prudent to consider the effect of the variation on the shape of the bending moment envelope before adopting the simplification.*

Cracked section properties should include the area of the reinforcement over the effective width of slab (determined taking account of shear lag) that acts with the steel girder. Note that although the modulus of elasticity of reinforcement given in EN 1992-2 is slightly lower than that for structural steel, EN 1994-2, 3.2 states that the structural steel modulus may be used for reinforcement in a composite section.

EN 1994-2 does refer to the need to take account in global analysis of the effects of tension stiffening of cracked sections at SLS, but that only needs to be considered directly where the deck is in overall tension, such as when it is the deck of a bowstring arch bridge. Such bridges are outside the scope of this publication (but even then detailed consideration can be avoided by carrying out two analyses, one fully cracked, one uncracked; the actual distribution of forces will be between these extremes).

Staged construction

It is usual for the deck slab of composite bridges to be concreted in stages, and for the steel girders to be unpropped between supports during this process. Part of the loading is thus carried on the steel beam sections alone, part by the composite sections; the bending stiffnesses and load distribution (and thus the bending moment diagrams) are different for each stage.

EN 1994-2, 5.4.2.4 addresses the sequence of construction and states the principle that appropriate analysis shall be made to cover the effects of staged construction.

For determination of SLS effects (and fatigue effects in the slab), separate analyses are required, one representing each successive stage that occurs. This series of analyses will follow the concreting sequence and will take account of the distribution of the weight of wet concrete, particularly that of the cantilevers. It will be a series of partially composite structures. Variable actions (traffic loads) are applied to a fully composite structure, usually with short-term composite properties.

Separate analyses are also required for ULS effects unless all the cross sections are Class 1 or 2 and there is no allowance for lateral torsional buckling. This exception occurs rarely (mainly with single spans and non-integral construction) and it is usual to allow for staged construction at both SLS and ULS.

Typically, there are about twice as many stages as spans, because concrete is placed successively in each of the midspan regions, followed by the remaining regions over each support. Where the cantilevers are concreted at a different stage from the main width of slab, this must be taken into account in the analyses.

Effects of creep

The effects of creep are taken into account by determining an appropriate modulus for long-term effects. The modular ratio is given by EN 1994-2, 5.4.2.2(2), which requires a creep coefficient according to EN 1992-1-1, 3.1.4. The situation with no creep is more onerous for stresses in concrete; the situation with full creep is more onerous for stresses in steel. Both situations must therefore be considered. See Hendy and Johnson[23] for guidance on creep, including the effects of waterproofing.

Effects of shrinkage

Shrinkage of the concrete slab gives rise to primary effects and, for continuous beams, secondary effects. The primary effects are internal self-equilibrating stresses that are derived by considering the axial force and bending moment that would be required to restrain the shrinkage and applying them to the composite cross section (in essence releasing the restraint). The release moment causes a curvature that would be constrained if the beam were continuous over several supports; the moments caused by the constraint are the secondary effects of shrinkage. The shrinkage strain is given by EN 1992-1, Annex B.2 and the modular ratio for shrinkage is given by EN 1994-2, 5.4.2.2(2). See Hendy and Johnson[23] for guidance on calculating these effects; the process is best described in a worked example, such as that in P357[1].

Combination of global and local effects

Global and local action effects should be combined, according to 1994-2, 5.4.4. The combination factor is defined in Annex E of EN 1993-2 and the NA to BS EN 1993-2 states that further information may be given in the project specification.

5.2 Modelling

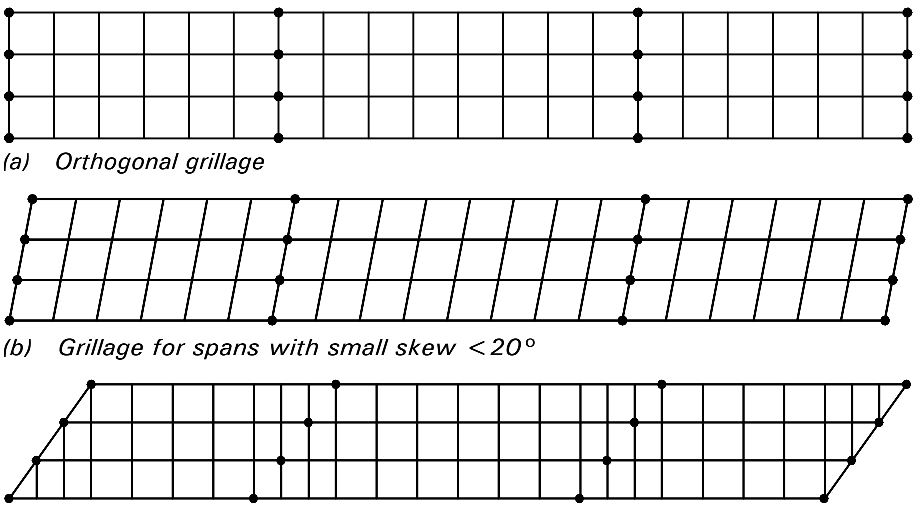
Nowadays, computer modelling of bridge decks is almost always employed, even for simple single spans where the supports are square to the deck and the moments and shear could be calculated manually (though, for multi-girder bridges, the distribution of action effects between the girders would be difficult to determine accurately by hand).

The increasing availability of comprehensive and powerful analytical software is likely to lead to wider use of the more sophisticated models. The need to determine critical buckling moments, on which to base the calculation of bending resistance, is also likely to lead toward software that can determine elastic buckling values.

5.2.1 2D Grillage models

Multi-girder decks

For multi-girder bridge decks, a simple 2D grillage will give adequate results for non-integral bridges (see Section 9.1.2 for guidance on modelling integral bridges). In such models, the structure is idealised as a number of longitudinal and transverse beam elements in a single plane, rigidly interconnected at nodes. The transverse beam elements may be orthogonal or skewed with respect to the longitudinal beams. Each beam element represents either a composite section (e.g. main girder with associated slab) or a width of slab (e.g. a transverse element may represent a width of slab equal to the spacing of the transverse elements). Where the supports are square to the main beams, an orthogonal grillage is used. Where the supports are at a small skew, the grillage may be skew (the lines of reinforcement will also probably be skew in these cases). Where the skew exceeds about 20 degrees, a skew grillage has difficulty in modelling the slab behaviour; an orthogonal grillage with skewed ends is used instead (but then a local model may also be needed in the obtuse corners because the grillage model cannot separate the torsional effects carried by the slab from those carried by the beam in warping, which is a particular concern in those regions). Examples of all three configurations are shown in Figure 5.1.



(a) Orthogonal grillage

(b) Grillage for spans with small skew <20°

(c) Grillage for spans with large skew (>20°)

Figure 5.1 *2D grillage models for a 3-span multi-girder bridge*

A line of elements should be provided along each main girder; intermediate lines (representing slab only) are used to refine the mesh so that the effects of wheel loads can be evaluated (rather than developing a separate model or referring to standard plate influence surfaces to determine local effects). An edge beam is usually provided, to facilitate modelling of the cantilevers (which frequently carry footway loading). Because the transverse beam elements do not represent discrete structural elements, the spacing can be chosen by the designer to facilitate the analysis. Generally, the spacing should not exceed about 1/8 of the span for modelling global effects. Uniform node spacing should be chosen in each direction where possible, though it may be helpful to locate nodes at splice or bracing positions (so that values of moments and shear forces at these positions are available in the output).

For multi-girder bridges, shear lag is unlikely to reduce the effective width of the slab below its actual width (see the reference to allowance for shear lag on page 39). Models for the bare steel condition (this may be a line-beam model), for the partially composite conditions, for the long-term condition and for the short-term condition are required.

Section properties for the composite main beams should use the full composite second moment of area; if there are intermediate longitudinal lines, the elements should be given only the properties of the slab itself. Section properties for transverse beam elements representing the slab alone should use a width equal to the element spacing.

Torsional stiffness of the slab should be divided equally between the transverse and longitudinal beams; use $bt^3/6$ in each direction, where b is the width of slab appropriate to the element concerned and t its thickness.

Intermediate bracing (between beam pairs) should be modelled, because it does affect the local transverse bending stiffness (although it does not significantly affect distribution of load between main beams). A shear-flexible member should be introduced (derived from a local plane frame model of the bracing); this will give rise to local forces that can be used to verify the adequacy of the web stiffener to flange connection. (The inclusion of bracing in the model gives a better distribution of effects than the alternative of applying deformations from an unbraced model to a local plane frame.)

Ladder decks

For ladder deck bridges, 2D grillage models can be used, although they are not fully able to model the local effects of the deflections of the cross girders and their interaction with the lateral bending of the main girder bottom flanges.

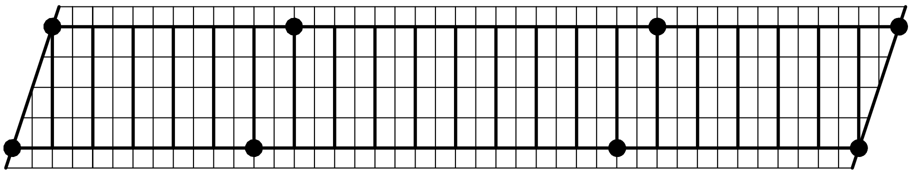
For cross girders, the appropriate width of slab acting with the cross girder is the spacing of the cross girders (i.e. one half of the distance to the next girder on either side) but not more than one quarter of the spacing between main girder webs plus the spacing to the outer studs on the cross girder (see EN 1994-2, 5.4.1.2).

Joints in the grillage model should be rigid connections. This applies not only to the joints between elements along a beam (which clearly must be rigid) but also to the joints between cross girders and main girders (for both the bare steel/wet concrete condition and the composite condition), assuming that either a lapped or spliced connection is used.

Torsional stiffness is provided mainly by the slab and the same assignment of half the stiffness in each direction as for the multi-girder grillage should be used. In a ladder deck model this use of only the ‘St Venant’ stiffness is a simplification; the model assumes that the main girder twists about the same axis as the end of the cross girder rotates. In reality, the slab and the lateral bending stiffness of the bottom flanges of the main girders provide additional restraint against the twisting of the main girder (in effect this is warping restraint) but this cannot be modelled in a simple 2D grillage

In many cases, the bridge will be straight and the cross girders square to the main girders. An orthogonal grillage model is well-suited to this arrangement. Where the bridge is skew, the cross girders will normally still be square to the main girders although the spacing may have been adjusted locally to the supports such that the bearings are below cross girder to main girder connections.

Figure 5.2 shows a typical 2D grillage model suitable for the global analysis of a 3-span ladder deck bridge. The slab mesh should be at about 3 m spacing transversely and half the cross girder spacing. Cantilever slabs and edge beams should also be modelled. (This Figure shows coincident cross girders and trimmer girders at the obtuse corners but this may not be practical, as mentioned in Section 2.2.3; careful attention should be given to modelling in these corners.)



Elements representing main girders, cross girders and trimmer girders
Elements representing deck slab

Figure 5.2 *2D grillage model for a skew 3-span ladder deck*

5.2.2 3D grillage models

For ladder deck bridges, a 3D grillage skeleton model, essentially the same plane model as a 2D model but with vertical elements at every cross girder connection, connected at the bottom to beam elements representing the bottom flanges, is better able to represent the interaction between cross girders and main girders. The vertical elements have negligible bending stiffness in the plane of the web but have the stiffness of the effective stiffener section out of that plane. Vertical bending stiffness of the main girders is assigned wholly to the upper members and the 'bottom flange' elements represent only the plan bending of those flanges. The differential loading on adjacent cross girders (with one carrying a bogie of the LM3 model for example) generates different deformations in the adjacent U-frames and thus plan bending of the bottom flange; see further discussion in Section 6.1.8.

Although this type of model can be limited to portions of the deck, to determine local effects that reflect the interaction between cross girder U-frames (and thus is used in conjunction with a coarser global 2D grillage), once the extra complexity is addressed it is probably better to model the whole bridge in this way. The one model then determines both global and local effects (although it is not possible to separate global and local effects). A portion of a 3D grillage model is illustrated in Figure 5.3.

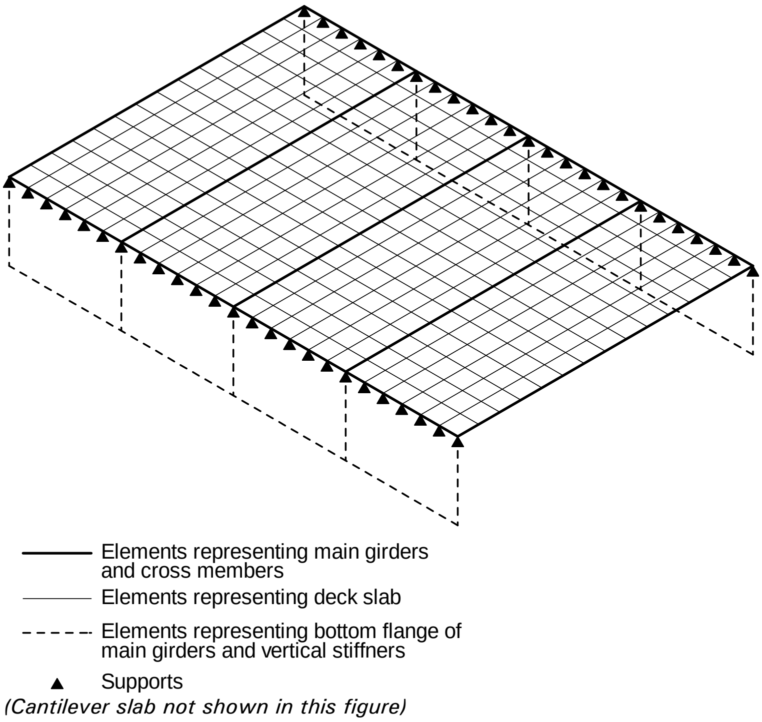
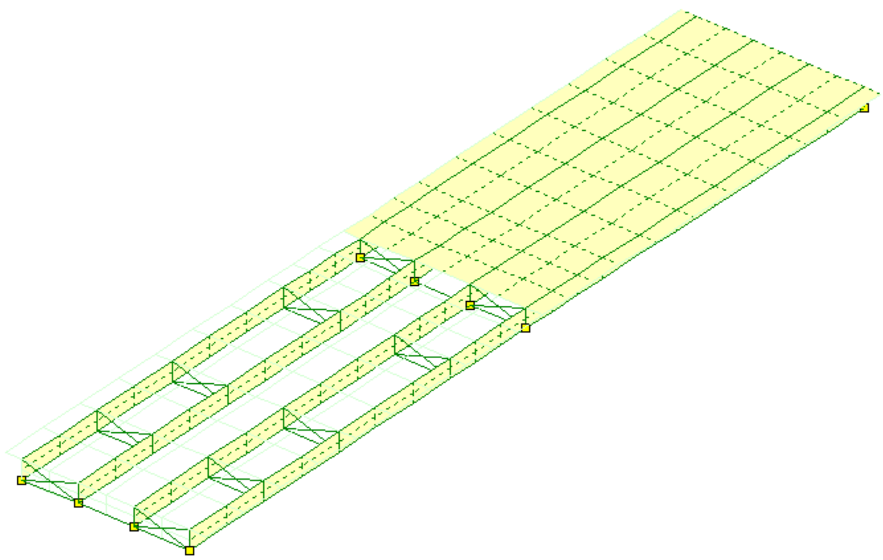


Figure 5.3 *Portion of 3D skeleton model*

5.2.3 FE models

A full linear 3D FE model can give a more realistic determination of the structural response, particularly for ladder decks. Models can be built using plate elements and beam elements. Plate elements are used for the deck slab and for the webs of the main girders (and for the webs of cross girders in ladder decks). Beam elements can be used for the flange plates (aligned with their bending stiffness in the plane of the flange and, for the top flange, having an offset from the slab elements), for web stiffeners (representing the effective Tee section) and for triangulated bracing. An example of a model using plate and beam elements is shown in Figure 5.4.



(Slab on near span not shown)

Figure 5.4 *3D model using beam and plate elements (Screen image from SAM model)*

Designers need to be aware of the capabilities of the different types of elements, how to assign appropriate properties and how to interpret the results of the analyses. For example: shell elements will automatically account for shear lag to some extent (dependent on the fineness of the mesh); a concrete slab that is cracked in longitudinal tension can be modelled with anisotropic properties; effective moments, shears and axial forces on composite beam sections can be determined (by the software) from the stresses determined in the global analysis.

Verification of buckling resistance sometimes requires an elastic buckling analysis of the structure to determine its critical loading. Software is available that can determine elastic buckling load using FE models, and these models can be used either to determine elastic critical moments of beams directly or with the general method of verification (see Section 6.1.8).

In some cases, second order (large displacement) analysis is also required. This requires more complex software; modelling and interpretation of the output requires previous experience in this type of analysis.

The use of a 3D FE model for multi-girder decks allows intermediate bracing to be modelled realistically, rather than the representative beam elements that need to be included in a 2D grillage in order to derive the local restraint forces referred to above.

5.3 Local slab analysis

Local analysis for dead load effects can readily be calculated manually; the effects are usually relatively small.

Local moments in the deck slab due to the effects of wheel loads can be calculated using recognised methods such as Pucher influence charts[25]. These moments are calculated for the slab on rigid vertical supports, making appropriate allowance for continuity of the slab from one slab bay to another with no torsional restraint from the steel girder webs.

Pucher Charts are a series of contour plots of influence surfaces, as illustrated in Figure 5.5. The simplification of support conditions to permit use of standard charts normally leads to a conservative assessment of worst moments.

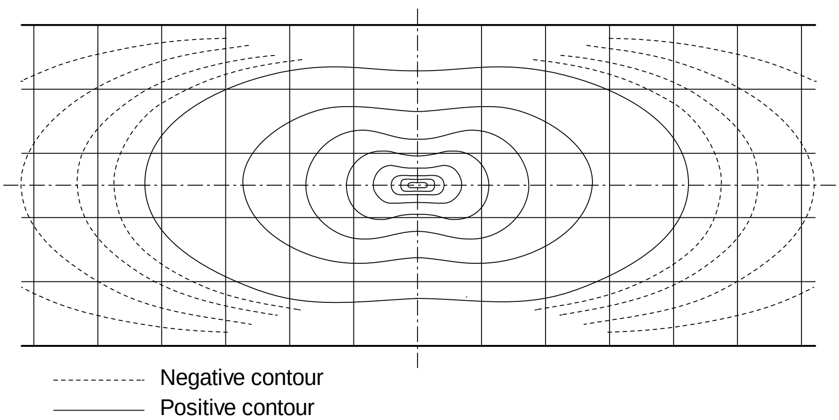


Figure 5.5 Typical influence chart for slab moments

If a fine mesh of shell elements is used in a 3D global model, local bending moments in the slab will be determined directly. A resolution (node spacing) of about 500 mm should be adequate in view of the loaded area under a wheel and dispersal through surfacing and slab.

Either the local analysis or a global model with shell elements can be used to determine orthogonal moments in the slab. However, where the bridge has a small skew (less than 20°), it is common to place the slab reinforcement parallel to the abutments. To design a slab with skewed transverse reinforcement, the moments and twists must be converted into effective design moments in the principal directions. EN 1992-2, 6.109 and Annex LL give guidance on this and Smith and Hendy[26] note that the equations of Wood[27], as modified by Armer[28] (which are available within some computer analysis programs) can, arguably, be used instead.

5.4 Actions and combinations of actions

Action effects are to be determined by the global analysis for the range of combinations of actions to which the bridge may be subjected.

EN 1991 has separate Parts for all the different types of actions. The Part most relevant to bridges is EN 1991-2 *Traffic loads*; the actions of temperature variation are given in EN 1991-1-5.

5.4.1 Self weight

The characteristic values of self weight of the structural parts of a bridge are based on nominal dimensions and nominal unit weights (referred to in EN 1991-1 as 'densities' and given in units of kN/m^3). The nominal density for normal concrete is given as 24 kN/m^3 (but add 1 kN/m^3 for reinforcement and a further 1 kN/m^3 for wet concrete); the density for steel is given as 77 to 78.5 kN/m^3 (use the lower value, unless the project specifies otherwise).

For non-structural parts, such as waterproofing and surfacing, nominal values of density are again used but an allowance must be made for deviation from nominal dimensions. Table NA.1 to BS EN 1991-1 recommends an allowance of +55% /−40% on nominal thickness of waterproofing and surfacing (a lesser positive deviation may be used where a "post execution coating is included in the nominal value" but it is not exactly clear what that provision means, so use the larger value). For services, a deviation of ±20% from nominal values is recommended. In either case, deviations should be either positive (the normal situation) or negative - it is not necessary to use different deviations for different parts of the structure.

5.4.2 Traffic loads

For highway bridges, the first step is to define the number of notional lanes on the bridge. For this purpose the standard lane width is taken as 3 m; the carriageway is assigned lanes of this width, in the position to give the most onerous loading, and any part of the carriageway not in these lanes is defined as "remaining area". For carriageways between 5.4 m and 6 m wide, two notional lanes are defined.

Vertical forces

For the traffic loading there are four load models:

Load Model 1 - normal traffic:

This comprises a uniform loading per unit area (independent of loaded length), referred to as a 'UDL' system, and a pair of heavy axles at one position in the lane, referred to as a Tandem system, 'TS'. The characteristic values of the UDL and TS are set by EN 1991-2 and may be adjusted by factors given in the National Annex. For the UK, the adjusted value of the UDL is 5.5 kN/m^2 in all the lanes and in the 'remaining area'. The adjusted value of the TS axle load is given as 300 kN in the first lane, 200 kN in the second lane and 100 kN in the third lane. There is no TS load in other lanes or in the remaining area.

Load Model 2 -Single axle for short span members

This single axle load, which covers dynamic effects, may govern the design of short span members, typically of up to 7 m. The value of the axle load, as given by the NA to BS EN 1991-2, is 400 kN.

Load Model 3 - Special vehicles

A number of special vehicles (sets of axle loads), to represent abnormal traffic, are defined in EN 1991-2 but the choice of special vehicles is left to the National Annex. Instead of those special vehicles, the NA to BS EN 1991-2, clause NA.3.1, specifies the loads and configurations of three 'SV' and four 'SOV' vehicles; the choice of what is to be used in design is to be agreed with the relevant authority.

Load Model 4 - Crowd loading ('for general verifications')

This loading is intended to be applied without any traffic loading and may be defined for the particular bridge (there is thus no provision in the NA). It does not represent loading for footbridges, cycle track bridges etc. for which other provisions are given.

Horizontal forces

Horizontal forces are defined to represent braking/acceleration and centrifugal force. Values are given in the NA for forces associated with LM1 and LM3.

Loads on footways

Footway loading is given by EN 1991-2, 5.3, and modified by the NA to BS EN 1991-2, but for bridges where the footway loading coexists with traffic loading (LM1) as a leading action (group gr1a), the NA specifies a reduced value of 0.6 times the characteristic value. Footways should also be designed for accidental loading (comprising two wheel loads, acting alone, without any other variable load) unless they are protected from the traffic by a safety barrier (see EN 1991-2, 4.7.3.1).

Groups of loads

Several groups of traffic loads are defined, to represent vertical and horizontal forces due to traffic that are likely to coexist. A group of loads is treated as a single variable and thus may be considered as the leading action, $Q_{k,1}$ or as an accompanying action (in which case the factor ψ_0 is applied to the whole group, apart from gr1a, where a separate factor is given for footway loading - see BS EN 1990, NA.2.3.6.2). For highway bridges, traffic loads are almost always the leading action. The groups of loads specified by the National Annex to BS EN 1990 that are most likely to be applied are as shown in Table 5.1.

Table 5.1 *Groups of loads (based on Table NA.3 in the NA to BS EN1991-2)*

Group	Load			
	LM1	LM3	Horizontal forces	Footway loads
Gr1a	characteristic	–	–	reduced
Gr2	frequent	–	characteristic	–
Gr5	frequent	characteristic	–	–

Notes: Frequent values are characteristic values multiplied by the ψ_1 factor
Reduced footway loading is given in the NA to BS EN 1991-2

5.4.3 Thermal actions

The range of bridge temperatures in the UK is given by the National Annex to BS EN 1991-1-5. Maximum and minimum bridge temperatures are given based on location, bridge type and return period (the isotherms in the NA are for a return period of 50 years). Temperature difference (vertical temperature gradient) due to radiation is also given.

In the NA to BS EN 1990, the partial factor applied to thermal actions on bridges at ULS (see Table 5.2 below) gives design values appropriate for a 120 year return period.

5.4.4 Wind actions

Wind actions are given by EN 1991-1-4. Pressures are derived from a fundamental value of the basic wind velocity (which is given in the UK National Annex), from which a mean wind speed and peak velocity pressure are determined, appropriate to the location and topology: forces are determined using drag coefficients given in the Standard.

EN 1991-1-4 covers aerodynamic effects; rules are given for evaluating the potential susceptibility and, where there is susceptibility, expressions are given for determining critical wind speeds. Further rules are given in the UK NA.

For short and medium span bridges, forces due to wind actions are unlikely to influence the design of the superstructure, except possibly in relation to lateral forces at restraint bearings and during construction, before the slab is cast. Non-divergent aerodynamic effects may need to be evaluated for longer spans but will probably not determine the governing design situation.

5.4.5 Accidental actions

EN 1991-2, 4.7 gives general requirements for accidental actions on bridges. In relation to the bridge superstructure, the two principal accidental situations are the collision of traffic with barriers (restraint systems) on the bridge and the collision of traffic under the bridge with the superstructure.

For forces due to collision with restraint systems for determining global effects, and for the coexisting vertical forces, see the National Annex to BS EN 1991-2. For the local forces on the deck from the restraint system, see TD 19[3].

For bridges with at least 5.7 m clearance above the roadway below, no forces due to impact on the superstructure from traffic below the bridge need be considered, according to the NA to BS EN 1991-7. For bridges with lesser clearance, the NA gives values of equivalent static forces.

5.4.6 Actions during execution

EN 1991-1-6, 3 sets out the design situations for actions during construction and EN 1991-1-6, 4.11 lists the types of construction loads that should be considered. Most notable for bridges are the loads during the casting of concrete, the loads due to the storage of movable items and the weight of machinery and equipment. Return periods for climatic actions and values for construction loads are given in the National Annex.

5.4.7 Fatigue loading

Five load models are defined in EN 1991-2, 4.6 and their use is given in the appropriate material Parts, EN 1992 to EN 1999. For highway bridges up to 80 m span, 'Fatigue Load Model 3', the single vehicle model, is used in all but exceptional cases and this is used to derive the design value of the stress range corresponding to 2×10^6 cycles according to EN 1993-2, 9.4.1 (1) to (5) and EN 1993-1-9, 6. For this determination an elastic global analysis is needed; allowance should be made for shear lag and, where the cross section is Class 4, stresses should be determined on the basis of the effective cross section. No partial factor is applied to the load at this stage but note that an amplification factor is applied near to expansion joints (EN 1991-2, 4.6.1(6)).

For this determination, the same notional lanes as for 'static' loading can be used[6]. The load model is traversed the length of the bridge in each lane in turn[7] and the value of the stress range for each traverse is determined; the 'reference stress range' referred to in EN 1993-2, 9.4.1(3) is the largest of these ranges. For further discussion of fatigue assessment, see Section 6.6.

Combinations of actions for fatigue assessment of reinforcement

Fatigue assessment of reinforcement is only needed where the reinforcement is in tension. However, the global stress in the slab depends on the permanent and variable actions as well as on the fatigue vehicle. EN 1992-2, 6.8.3 sets out the 'basic combination' and 'cyclic combination' that are to be considered together. These combinations will lead to values of $M_{\mathrm{Ed,\ max,f}}$ and $M_{\mathrm{Ed,\ min,f}}$ that will be needed to determine the stress range. For assessing the damage equivalent stresses in reinforcement, EN 1992-2, NN.2.1(101) introduces a factor that increases the weight of the fatigue vehicle by a factor of 1.75 (for verification over supports) or 1.4 (for verification in other areas).

Combinations of global and local effects

Where stresses in steel or concrete are due partly to global effects and partly to local effects, they should be evaluated separately, if possible, for separate evaluation in fatigue assessment. If they cannot be separated, they should be considered as local effects (because this will be more onerous in fatigue assessment) unless it is believed that the component due to local effects is very small.

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[6] EN 1991-2, 4.2.4(3) refers to lanes for "traffic to be expected in normal conditions" which could be taken to refer to actual lane widths and positions according to the marked white lines, but 4.6.1(4) seems to refer to the 3 m wide notional lanes. For convenience, the same lanes can be used for both static and fatigue loading; this should normally be conservative.

[7] Although EN 1991-2, 4.6.4(3) refers to the possibility of a second fatigue vehicle in a lane, the NA does not invoke that option - only a single vehicle needs to be considered in one lane.

5.4.8 Geotechnical actions in integral bridges

Geotechnical actions on bridge superstructures are the pressures of the backfill at the abutments. These pressures arise initially from the backfilling behind the abutments but are modified (increased) over time due to the 'strain ratchetting' effect due to the cyclic expansion/contraction of the deck with temperature variation. Soil pressures due to the movements at the ends of the deck may be calculated using guidance in PD6694-1[36], but see further discussion under Section 5.5.

5.4.9 Partial factors applied to actions

The partial factors that are to be applied to the actions and the factors to be applied to accompanying actions are given in the National Annex to BS EN 1990, Table NA.A2.4(B) and Table NA.A2.1. A selection of the more common values is given in Table 5.2 and Table 5.3. Reference should be made to the National Annex to confirm the rules governing their application.

Table 5.2 *Partial factors on actions (taken from National Annex to BS EN 1990, Table NA.A2.4(B))*

Partial factor on:		ULS	SLS
Permanent actions			
Steel weight and superimposed loads	γ_G	1.20	1.00
Concrete weight	γ_G	1.35	1.00
Variable actions			
Road traffic actions	γ_Q	1.35	1.00
Wind actions	γ_Q	1.70	1.00
Thermal actions	γ_Q	1.55	1.00

Table 5.3 *Factors applied to accompanying actions (taken from National Annex to BS EN 1990, Table NA.A2.1)*

Action	ψ_0	ψ_1	ψ_2
Traffic loads (TS and UDL)	0.75	0.75	0
Wind forces (in service)	0.50	0.20	0
Thermal actions	0.60	0.60	0.50

For the accidental design situation, the values of the accidental actions are considered as characteristic values and the factors ψ_1 and ψ_2 are applied to the variable actions (see the NA to BS EN 1990). For fatigue assessment, partial factors to be applied to the fatigue loads are given by the NA to BS EN 1993-2.

5.5 Effects of geotechnical actions

The determination of the effects of geotechnical actions, and their combination with the effects of other actions, is a little more complex because the effects depend not only on the partial factors applied to the actions but also on partial factors applied to soil parameters.

EN 1990, A2.3.1(5) offers three "Approaches" to the determination of design values of actions where geotechnical actions are involved and the UK NA adopts Approach 1. That approach is to apply separately design values due to

'Set B' and 'Set C' partial factors. The Set B factors are those used for normal structural verification (and are the factors referred to in Section 5.4.9 above); the Set C factors are all lower values than the Set B factors.

EN 1997-1, 2.4.7.3.4 also sets out the three alternative design approaches, and the UK NA similarly confirms that Approach 1 should be used. However, for partial factors on actions, EN 1997-1 confusingly refers to Set A1 and Set A2 factors; these are the same Sets as Sets B and C of the NA to BS EN 1990 and are given the same values for the partial factors.

The difference between the applications of the two Sets of factors is that Set B/A1 is used in conjunction the partial factor on soil parameters set at $\gamma_M = 1.0$ whereas Set C/A2 is used in conjunction with values of γ_M greater than unity; this principle is set out in 2.4.7.3.4.2 of EN 1997-1 and the factors are given in Annex A (as modified by the NA).

The soil pressures are determined from the design values of the actions of self weight and traffic loading and are thus the effects of the combined actions. No further partial factors are applied when these pressures are used to determine, for example, axial force due to restraint of thermal expansion. Advice on the determination of soil pressures behind integral abutments is given in PD 6694-1[36].

In practice, the level of axial force in the deck structure in integral bridges with abutments of the types described in Section 2.5.4 will usually be modest and the use of Set B/A1 will govern the design of the superstructure (with traffic loads as the leading action). The design of the abutments themselves will probably be governed by the use of Set C/A2; the most onerous combination of actions might nevertheless be with traffic loading as leading action, since the value of the K^* pressure coefficient might not be significantly different for the alternative treatments of thermal action as leading or accompanying action.

Settlement

The effects of differential settlement of supports should be considered. Settlement is treated as a permanent action and partial factors to be applied to settlement values are given in the NA to BS EN 1990. However, EN 1990 refers to 'best-estimate predicted values' and there are no clear rules in EN 1997 to determine appropriate values of differential settlement. Advice should be sought from geotechnical engineers and agreed values recorded in the project files. Generally, EN 1990 recommends that two individual foundations, selected to give the most unfavourable effect, should be considered to have a differential settlement, relative to the other foundations.

6 DETAILED DESIGN: IN-SERVICE STAGE

EN 1994-2 provides detailed rules for composite construction. Where necessary, it refers to EN 1992 for the design of concrete elements and to EN 1993 for the design of steel elements but EN 1994-2 is nevertheless the 'leading' Eurocode for the design of composite bridges.

The designer is required to verify the resistance (strength) of the composite girders and other structural elements at the Ultimate Limit State, considering the bending resistance of the cross sections, the shear resistance of the web panels and the shear connection between the steel and the concrete elements. The resistance of the bottom flanges to buckling, where they are in compression, also needs to be verified. In integral bridges, the effect of compressive forces on the resistances has to be taken into account. The resistance of the deck slab to combined local and global loading must be verified. Adequacy at the fatigue limit state has to be verified and toughness of the steel against brittle fracture must be ensured by selecting a suitable material sub-grade.

Adequacy at the Serviceability Limit State also needs to be verified; principally the checks are to ensure that inelastic behaviour does not occur under SLS actions, that crack widths in concrete are not excessive and that deformations are within acceptable limits.

6.1 Main girders

6.1.1 Cross sectional classification of beams

There are four classes of steel cross section defined in EN 1993-1-1, 5.5.2; the class determines the cross-sectional resistance that can be developed and the type of global analysis that may be considered.

- Class 1 can form a plastic hinge, with rotation capacity
- Class 2 can develop plastic resistance, but has insufficient rotation capacity to act as a hinge
- Class 3 can develop the elastic resistance of the full cross section
- Class 4 can only develop an elastic resistance that is less than that of the full cross section; this is usually expressed as the resistance of an 'effective' cross section.

The different behaviour of beams of the four classes of cross section are illustrated in Figure 6.1.

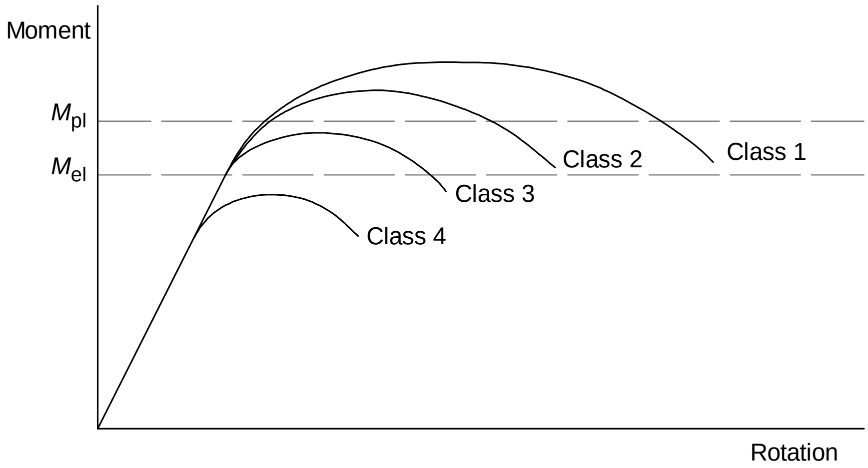

Figure 6.1 *Behaviour of classes of steel cross section*

For Classes 1, 2 and 3, the limiting dimensions for an element of cross section in compression are given in Table 5.1 of EN 1993-1-1. For elements not meeting the Class 3 limit, a Class 4 effective area, allowing for plate buckling effects, is determined in accordance with EN 1993-1-5, 4. The class of the cross section is that of the lowest class of its elements.

EN 1994-2, 5.5.1 uses the same classification, dependent on the classes of the steel elements of the composite cross section; additional requirements relating to the ductility of reinforcement in tension are given in that clause.

Plate girders in bridges are often Class 3 or Class 4. Composite sections in sagging moment regions are often Class 2; composite sections in hogging moment regions are sometimes Class 2 but more usually Class 3 or 4. Compression flanges should always be proportioned to be at least Class 3.

6.1.2 Bending resistance of cross sections of beams

The design resistance of the cross section is expressed as $M_{pl,Rd}$ or $M_{el,Rd}$, according to whether the plastic or elastic resistance is referred to. Plastic resistance may only be used for Class 1 and 2 cross sections; elastic resistance must be used for Class 3 and Class 4 sections (EN 1994-2, 5.5.1). The bending resistance is that for the effective cross section (allowing for shear lag and for local buckling of elements in Class 4 sections) based on the design values of the material strengths f_{yd}, f_{cd} and f_{sd}. These design values are derived from the characteristic values by dividing by the appropriate partial factor γ_M. Care needs to be exercised in the use of design values for steel because although γ_{M0} is used for cross sectional resistance γ_{M1} is used where member buckling resistance is to be calculated (the values of these factors, according to the National Annex to BS EN 1993-2 are 1.0 and 1.1 respectively).

The plastic resistance of composite cross sections $M_{pl,Rd}$ is calculated in accordance with EN 1994-2, 6.2.1.2.

The elastic resistance of a composite cross section is attained when the stress in either the structural steel, the concrete or the reinforcement reaches the limiting value of f_{yd}, f_{cd} or f_{sd} – see EN 1994-2, 6.2.1.5. That clause also gives rules for determining stress distribution in Class 4 sections.

The determination of the effective areas of Class 4 webs is made by means of notional ‘holes’ within the width of the element concerned. EN 1993-1-5, 4.3

and 4.4 provide rules for determining these effective widths. See further comment in Section 7.2.1.

6.1.3 Design resistance of beams constructed in stages

In a composite beam that behaves essentially in an elastic manner, the total stresses and strains in the fibres of the beam where the deck slab is added in stages are determined by summation of the stress distributions for the bending moments at each stage of construction (see page 40 for discussion of staged construction). Some bending is carried on the bare steel beam (its own weight and that of wet concrete); some is carried on a beam with long-term section properties (concrete added elsewhere on the structure and superimposed loads such as the weight of surfacing and parapets); and some is carried on a beam with short-term section properties (traffic loads, etc.). The summation process is shown diagrammatically in Figure 6.2. The position of zero stress will not necessarily correspond with any particular centroid level.

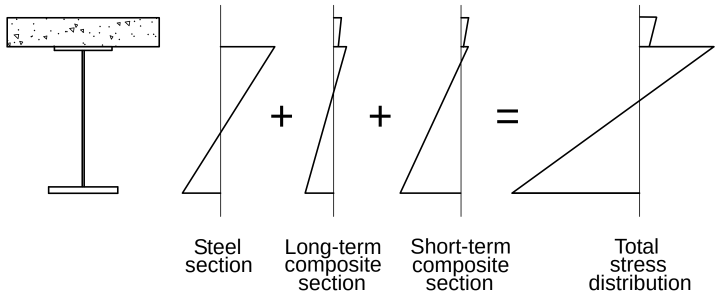

Figure 6.2 *Summation of stresses for staged construction*

Composite beams that are Class 3 or 4 are designed to their elastic bending resistance, which means that the total stress (from the summation) is compared with the limiting stresses for the steel, concrete and reinforcement.

However, for verification of resistance to buckling, the bending resistance of the cross section M_{Rd} is required. For beams designed as Class 3 or 4 $M_{Rd} = M_{el,Rd}$ (note that γ_{M1} applies for this value, rather than γ_{M0}) but to determine an appropriate value of $M_{el,Rd}$, the effect of the construction sequence must be considered, noting that, for a beam subject to hogging bending, the beam section is subject to a total bending moment given by:

$$M_{Ed} = M_{a,Ed} + M_{c,Ed}$$

Where $M_{a,Ed}$ is the design bending moment applied to the steel section alone and $M_{c,Ed}$ is the design bending moment applied to the (cracked) composite section.

The elastic design bending resistance can then be determined using the following expression:

$$M_{el.Rd} = M_{a,Ed} + kM_{c,Ed}$$

Where k is an amplifying factor that just causes the stress limit (determined using γ_{M1} for steel strength) to be reached in either the structural steel section or the reinforcement (whichever occurs first).

As noted above, composite beams that are Class 1 or 2 can be designed to their plastic bending resistance. Although the effects depend on the construction sequence (as explained on page 40), the verification of bending resistance compares the total effects against the plastic resistance of the cross section. Thus, for verification of bending resistance, $M_{Rd} = M_{pl,Rd}$ (again, using γ_{M1} on steel strength for hogging regions).

6.1.4 Shear resistance of beam webs

The shear resistance of the composite beam section is taken as that of the steel section.

The shear resistance of webs that are not prone to shear buckling is given by EN 1993-1-1, 6.2.6. However, the webs of most bridge girders are slender (see EN 1993-1-5, 5.1(2) for limits) and then the shear resistance of the steel section is the shear buckling resistance $V_{b,Rd}$, as given by EN 1993-1-5, 5.2.

Shear resistance of slender webs is enhanced significantly by the provision of intermediate transverse web stiffeners. These stiffeners divide the web into individual rectangular panels that have a much higher shear resistance than an unstiffened web. In multi-girder bridges, transverse stiffeners are typically provided at a spacing of 1 to 1.5 times the web depth in regions of high shear. In ladder deck bridges web stiffeners are provided at the connection of each cross girder and it is unlikely that additional stiffeners will be needed (the panel aspect ratio will usually be less than 2.5 and webs tend to be less slender than in multi-girder decks).

The shear resistances given in EN 1993-1-5, 5.2 are based on the 'rotated stress field model', rather than the tension field model in BS 5400-3, although the results are similar. There is an implied 3-stage development of shear resistance:

- An initial development of shear resistance, up to the point where the web (or web panel, for beams with transverse stiffeners) buckles in an elastic shear buckling mode.
- Further development of shear resistance by means of a diagonal band of tensile stress (sometimes referred to as tension field action); these bands are effectively anchored by the in-plane restraint of the web in the adjacent panel. (For an end panel without any restraint beyond the bearing stiffener, termed a non-rigid end post, this enhancement does not develop fully.)
- A final enhancement of the resistance by the restraint of the tensile band provided by the top and bottom flanges.

The contribution from the web (including the enhancement by the development of a tensile band) is expressed in EN 1993-1-5, 5.2 as:

$$V_{bw,Rd} = \chi_w \frac{f_{yw} h_w t}{\sqrt{3}\,\gamma_{M1}}$$

In this expression, the contribution factor χ_w depends on the web slenderness parameter $\bar{\lambda}_w$ which in turn depends on both the depth/thickness ratio h_w/t and the aspect ratio of panels between transverse stiffeners (see clause 5.3(3) and Annex A.3). The relationship between χ_w and $\bar{\lambda}_w$ is shown in Figure 6.3, which is a partial reproduction of Figure 5.2 of EN 1993-1-5.

The EN 1993-1-5 rules for shear resistance include a factor η (recommended value 1.2) that allows for strain hardening but the NA to BS EN 1993-1-5 sets η = 1.0. This enhancement is therefore omitted in Figure 6.3.

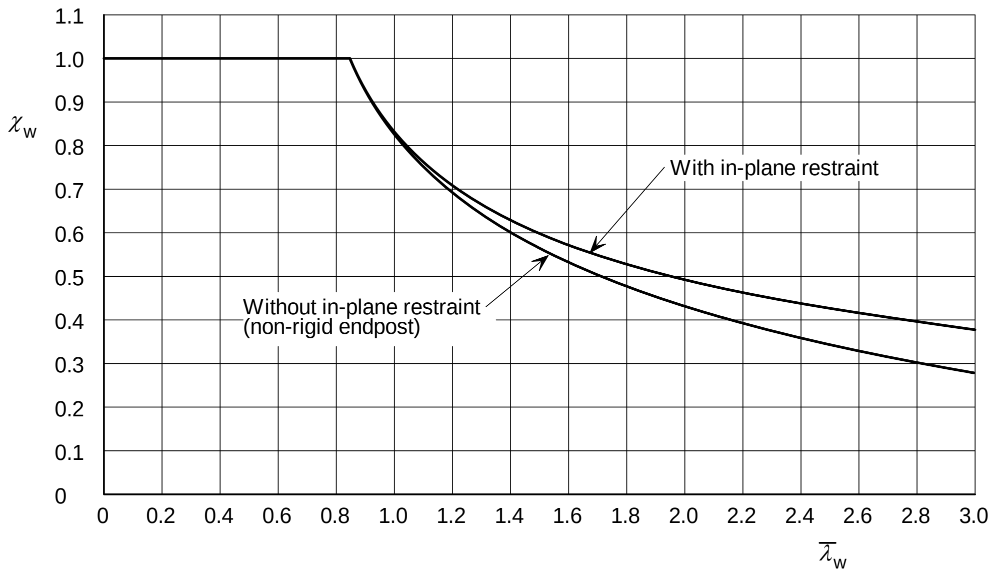

Figure 6.3 *Shear buckling factor χ_w (based on EN 1993-1-5, Figure 5.2)*

The contribution from the flanges, expressed as $V_{bf,Rd}$, depends on the plastic resistance of the flange plate, bending out of its plane. For steel beams, the smaller flange is used but for composite beams, the bottom flange should be used, even if it is larger (see EN 1994-2, 6.2.2.5).

The mechanism whereby the flanges contribute is shown in Figure 6.4.

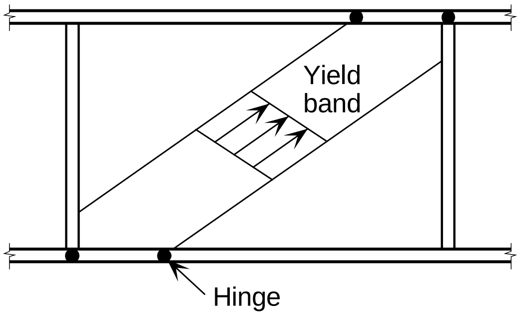

Figure 6.4 *Support of tensile band by bending of flanges*

Where there are longitudinal stiffeners on the web, the procedure for determining shear resistance remains the same, the only difference being that the evaluation of $\bar{\lambda}_w$ takes account of the presence of the stiffener – see EN 1993-1-5, 5.3(5).

EN 1993-1-5 does not explicitly evaluate the resistance of non-rectangular web panels (such as occur in tapered and haunched beams). For such beams, the web buckling resistance may conservatively be based on the slenderness of a rectangular panel of the same width and of the maximum depth of the actual panel, and on the depth of the web at the shallower end[8]. In such cases, the flanges will not be parallel and some of the shear will be carried by the vertical component of the force in the inclined flange; this reduces the shear in the web panel.

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[8] This is slightly more conservative than the note to EN 1993-1-5, 2.3(1) but can also be used when the angle between the flanges is greater than 10°.

6.1.5 Bending / shear interaction

Where the shear in the web is high, the bending resistance is reduced, because of the interaction of bending and shear stresses. The limiting envelope for interaction of bending and shear resistances is given by EN 1993-1-5, 7.1 and is shown graphically in Figure 6.5. The envelope is curved and the features to note are as follows:

The shear resistance of the web alone (i.e. without the contribution from the flange $V_{\mathrm{bf,Rd}}$) can be sustained at the same time as a bending resistance $M_{\mathrm{f,Rd}}$ of a section comprising flanges only (the web is ignored).

The full shear resistance V_{Ed} (= $V_{\mathrm{bw,Rd}}$ + $V_{\mathrm{bf,Rd}}$) can only be sustained in the absence of bending. Between zero bending and the development of the resistance $M_{\mathrm{f,Rd}}$, the shear resistance gradually reduces to the value of $V_{\mathrm{bw,Rd}}$.

The full plastic bending resistance of the cross section can be sustained at the same time as a shear resistance equal to half the resistance of the web alone (i.e. $0.5 \times V_{\mathrm{bw,Rd}}$). However, for Class 3 and Class 4 sections, the bending resistance is further limited to the elastic resistance, but that does not affect the shape of the envelope, it only introduces a cut-off.

Between shear resistances of 0.5 $V_{\mathrm{bw,Rd}}$ and $V_{\mathrm{bw,Rd}}$ the bending resistance varies according to a quadratic relationship.

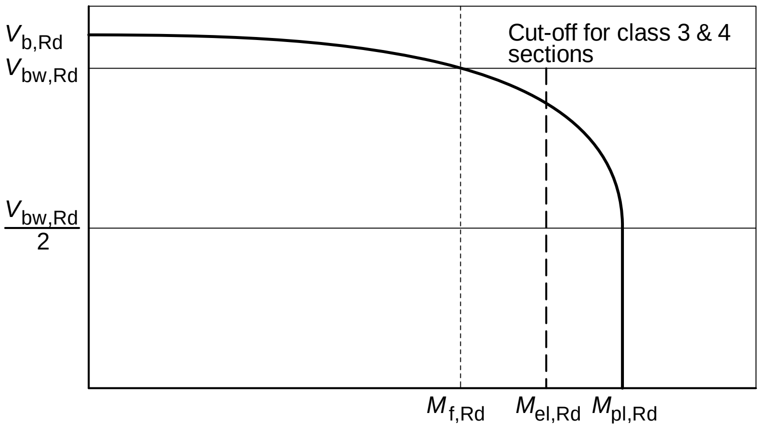

Figure 6.5 *Interaction of bending and shear resistances - limit envelope*

Where there is axial compression, the values of bending resistance are reduced (see EN 1993-1-5, 7.1(4)) but otherwise the interaction relationship is not affected.

EN 1993-1-5, 7.1 recognises that, close to a bearing stiffener at a support, the shear buckling effects are small and, within $h_w/2$ of the support it allows the interaction to be evaluated on the basis of the plastic shear resistance, rather than the shear buckling resistance. However, it is simpler to ignore this enhancement unless the check then fails by a small margin; if that is the case, carry out two interaction checks, one at the support, based on $V_{\mathrm{pl,Rd}}$ and one at a distance of $h_w/2$, using the effects at that section and $V_{\mathrm{b,Rd}}$.

6.1.6 Buckling resistance of composite beams

In service, for beams with the deck slab on top of the main girders, the only regions of the main girders that are potentially susceptible to buckling are the bottom flanges where they are in compression. These regions occur adjacent to

In these regions, the form of buckling is not true lateral torsional buckling, which involves only lateral and rotational displacements of the cross section, but distortional buckling, because the transverse bending resistance of the slab provides rotational restraint, which means that the bottom flange can only displace laterally by distortion of the cross section. Nevertheless, EN 1994-2 refers to EN 1993-1-1 and EN 1993-2 for rules to determine the lateral torsional buckling resistance of beams, based on a value of non-dimensional slenderness $\bar{\lambda}_{LT}$ (which EN 1994-2 calls relative slenderness).

The buckling resistance moment of a composite section can be expressed as:

$$M_{b,Rd} = \chi_{LT} M_{Rd}$$

where M_{Rd} is the design value of either the elastic or plastic resistance of the cross section in hogging bending (which is calculated using design strengths determined by dividing by γ_{M1} as noted in Section 6.1.2), as appropriate. This expression is only given in EN 1994-2 for the particular configuration covered by clause 6.4.2 but it mirrors that of EN 1993-1-1, 6.3.2.1(3) and is applicable generally for composite beams.

The factor χ_{LT} is a reduction factor that depends on the parameter $\bar{\lambda}_{LT}$, which in turn depends on the elastic critical moment $M_{cr.}$ - see further comment on determining non-dimensional slenderness in Section 6.1.7.

The value of χ_{LT} is given by buckling curves defined in EN 1993-1-1. Two sets of lateral torsional buckling curves are given. One set, which applies to most welded plate girders (and which should be used for all composite sections) is given by 6.3.2.2 and is essentially identical to the set of buckling curves for flexural, torsional and torsional-flexural buckling of compression members. Five separate curves, for different values of imperfection factor, are illustrated in Figure 6.4 of EN 1993-1-1; four of these curves are also used for lateral torsional buckling and are reproduced here as Figure 6.6. For welded plate girders in bridges, curves c or d will be used (curve d in most cases). The 'plateau' (where χ_{LT} = 1.0) extends as far as $\bar{\lambda}_{LT}$ = 0.2.

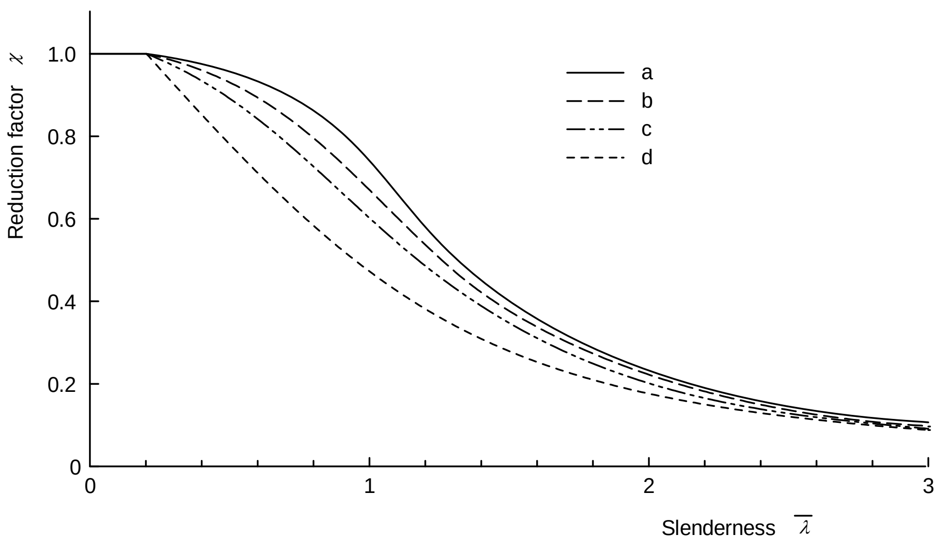

Figure 6.6 *Buckling curves*
(Figure 6.4 of BS EN 1993-1-1, omitting curve a_0)

A second set of lateral torsional buckling curves is given in EN 1993-1-1, 6.3.2.3; the curves are similar in form but with a longer plateau, extending to $\bar{\lambda}_{LT} = 0.4$. Those curves apply to rolled sections and equivalent welded sections (which is understood to mean doubly symmetric welded beams of the same dimensions as rolled sections). As it is rarely economic to use rolled sections, or even their equivalent welded sections, the second set of buckling curves will not normally be used in bridge design. (Also, those curves would not apply to a composite beam using a rolled section.)

6.1.7 Non-dimensional slenderness

The non-dimensional slenderness for composite beams can be expressed as:

$$\bar{\lambda}_{LT} = \sqrt{\frac{M_{Rk}}{M_{cr}}}$$

Where M_{Rk} is the characteristic value of the (elastic or plastic) bending resistance of the cross section and M_{cr} is the elastic critical moment. This expression for $\bar{\lambda}_{LT}$ is only given in EN 1994-2, 6.4.2 but it mirrors that for steel beams (see EN 1993-1-1, 6.3.2.2) and is generally applicable for composite beams.

Neither EN 1993 nor EN 1994 give solutions for M_{cr} and its value must be determined either by elastic buckling analysis or by reference to other sources.

For hogging regions of composite bridges it is difficult to find suitable theoretical models that will give realistic (not overly conservative) values of M_{cr}. However, EN 1994-2, 6.4.2 refers to EN 1993-2, 6.3.4, which does provide two general methods to determine $\bar{\lambda}_{LT}$, one called 'general method' (not to be confused with the 'general case' of EN 1993-1-1, 6.3.2.2) and one called 'simplified method'. These are discussed separately below. Both methods can take account of combined bending and axial force.

Additionally, EN 1994-2, 6.4.2 introduces a method that can be used for 'continuous inverted U-frames' i.e. hogging regions where the only lateral restraint to the compression flange is by the out-of-plane bending of the webs. This method is discussed in Section 6.1.10.

6.1.8 General method of verification, EN 1993-2, 6.3.4.1

For the general method, EN 1993-2-2, 6.3.4.1 refers to EN 1993-2, 6.3.4. The clause states that the general method may be used for structural elements or for whole frames and sub-frames and is valid for uniform and non-uniform configurations and sections. It is generally only suitable where elastic buckling analysis (out of plane) and non-linear analysis (in-plane) is available.

The first step is to calculate an amplifier ($\alpha_{ult,k}$) on the full design loading at which the characteristic resistance of the most critical cross section is reached. The model should only consider in-plane effects (i.e. no out-of-plane buckling effects) but should model all the effects of in-plane geometrical deformation. This means that the effect of staged construction and, if there is axial compression (e.g. in an integral bridge), in-plane second order effects should be considered. If plastic resistance is to be relied upon at any cross section, the model should also take account of material non-linear behaviour.

The next step is to calculate an amplifier ($\alpha_{cr,op}$) on full design loading at which the elastic critical resistance for out-of-plane buckling would be reached. The model should be elastic and should not take account of in-plane buckling.

The non-dimensional slenderness is then given by:

$$\bar{\lambda}_{op} = \sqrt{\frac{\alpha_{ult,k}}{\alpha_{cr,op}}}$$

This term is equivalent to the usual slenderness $\bar{\lambda}_{LT}$ and thus the reduction factor χ_{op} is given by the same buckling curves[9].

The final step is the verification

$$\frac{\chi_{op}\alpha_{ult,k}}{\gamma_{M1}} \geq 1.0$$

This method is of greatest use where an elastic buckling analysis can be carried out using a computer model. The use of this method and a buckling analysis does allow the designer to deal with non-uniform configurations much more easily than manual calculation.

6.1.9 Simplified method of verification, EN 1993-2, 6.3.4.2

A simplified method for calculation of $\bar{\lambda}_{LT}$, suitable for manual calculation, is given in 6.3.4.2 (2) to (7). In this method, the bottom flange and part of the web are treated as a column subject to compression and the slenderness $\bar{\lambda}_{LT}$ is derived from an evaluation of N_{crit} for the column. This is likely to be the most commonly used manual method for deriving slenderness for the in-service condition.

The model relates to a beam in hogging bending and uses a Tee section comprising the bottom flange and one-third of the compression zone of the web. (For a beam in combined bending and compression, see further discussion bleow). For a span length L between rigid restraints at the supports, EN 1993-2, 6.3.4.2(6) gives the critical axial load as:

$$N_{crit} = mN_E$$

N_E is the elastic critical load of a pin-ended column of length L and m is a modifying parameter that can take account both of flexible intermediate restraint and of varying compressive force.

The use of the column model method for beam spans with effective intermediate restraints to the compression flange and for beam spans with flexible intermediate restraints is described below.

Slenderness of beams with effective intermediate restraints

In multi-girder bridges, bracing between pairs of beams provides a very stiff (effectively rigid) lateral restraint to the compression flange. Bracing is

[9] Because in some cases different curves apply for lateral and lateral torsional buckling, 6.3.4(4) refers to both possibilities and explains how to deal with this situation. The simplest course is to use the lower curve.

commonly provided at one or two positions either side of an intermediate support to restrain the compression flange (typically at a spacing of 10 and 20 times the flange width) – see Section 8.2.1. With some relatively narrow ladder decks, the restraint from the cross girders may also be effectively rigid (particularly if knee bracing is provided).

For this situation, the length L may be taken as the distance between restraints. The only additional restraint between bracing positions is the relatively very flexible continuous U-frame restraint referred to in EN 1994-2, 6.4.2; its contribution is very modest and may conservatively be ignored (i.e. take $c = 0$ in 6.3.4.2(6)). The value of m may then be taken at its minimum value (1.0) and $N_{crit} = N_E = \pi^2 EI/L^2$, where I is the second moment of area of the Tee section. However, where the moment in the beam reduces over the length between restraints (there is often a significant reduction over the length from a support to the first bracing position), a larger value of m can be determined using the Note to 6.3.4.2(7). (For cases where the moment changes sign over this length, the Note allows the conservative assumption of zero moment at the far end.)

The test for whether the intermediate restraints are 'rigid' is given in EN 1993-2, 6.3.4.2(6). If the bracing is not stiff enough to be considered rigid, the slenderness should be evaluated as for beams with flexible restraints (although if the restraints are not regularly spaced it may be difficult to derive an appropriate resistance by manual calculation and a computer buckling analysis may be needed).

Slenderness of beams with flexible intermediate restraints

The restraint from the cross girders of ladder decks is usually 'flexible' and this flexibility must be taken into account in determining the slenderness. (Note that ladder decks are usually analysed in a 3D computer model and the software suitable for this is often able to determine elastic critical buckling loads. If this is available, the general method described in Section 6.1.8 may be used.)

For manual calculation of the slenderness of beams with flexible restraints to the bottom flange, the method of EN 1993-2, 6.3.4.2 may be used, considering the full span length between the rigid restraints at the supports, provided that the flexible restraints are regularly spaced.

The flexibility of intermediate restraints should be determined by considering the displacement due to a unit lateral force at compression flange level. Where the restraint is by U-frame action, such as with cross girders in a ladder deck, a simple plane frame model subject to equal and opposite unit forces at the two bottom flanges can be used. The flexibility is taken into account in the m parameter by means of its stiffness C_d and an intermediate parameter γ - see 6.3.4.2(6).

The variation of bending moment over the length between rigid restraints at supports is again taken into account by means of the Note to 6.3.4.2(7). For whole spans, the bending moment usually varies from hogging at the support adjacent to which verification is needed to sagging moment in midspan and either to zero at the far end of the span (if it is an end support) or back to a hogging moment. The expressions for m in 6.3.4.2(7) are not valid for reversal of bending moment but this can be overcome by conservatively assuming that the far end moment $M_2 = 0$ and that the variation of shear is such that there is no sagging moment over the span: a value of $V_2 = 0$ may be assumed. This will imply the notional bending moment diagram shown in Figure 6.7. Over most of the span, the notional bending moment is greater (in hogging) than the actual moment diagram; the fact that it is less at the far end has little effect on the near end and this simplification is considered to be conservative (see Hendy and Johnson[23]).

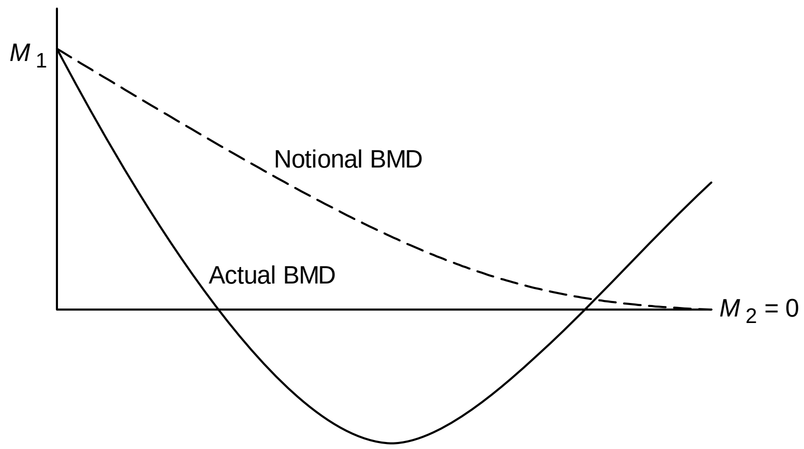

Figure 6.7 *Bending moment diagrams for determining 'm' parameter*

Thus the procedure is:

- Calculate N_E for the Tee section at the more highly stressed end of the length L between rigid restraints
- Calculate the restraint flexibility C_d for each intermediate restraint and thus the parameter γ (the spacing of the restraints should be essentially uniform)
- Calculate m for a notional bending moment diagram where $M_2 = 0$ and $V_2 = 0$ (and taking account of the value of γ)
- Calculate $\bar{\lambda}_{LT}$ using equation (6.10) in EN 1993-2, 6.3.4.2.

The above procedure can equally be used for the central portion of a multi-girder bridge with restraints only adjacent to the supports when that portion (between the outermost restraints at each end of the span) is subject to hogging moments at its ends. The restraint flexibility for that portion is then only that due to the out-of-plane bending of the web.

For ladder decks, it may also be feasible to consider the slenderness over a length of two or three cross girder spacings – the U-frame stiffness may be sufficient to be considered effective for buckling over this length.

Verification of resistance to LTB

At the end of the Note to 6.3.4.2(7), it is stated that the verification of resistance may be carried out at a distance $0.25L/\sqrt{m}$ from the support with the largest moment. The reduction in the value of M_{Ed} over this length is often very significant and it is usually well worth taking advantage of this permissive Note.

Combined bending and axial force

The simplified method in EN 1993-2, 6.3.4.2 is intended for verification of resistance to lateral torsional buckling and no comparable method is offered in either EN 1993-2 or EN 1994-2 for compression. For the hogging regions of composite bridges, 'local' lateral buckling (i.e. of the bottom flanges, not of the whole cross section) under axial compression is likely to occur before buckling in a vertical plane and therefore resistance to lateral buckling under the combined effects of moment and compression should be considered. In the absence of specific directions, it is suggested that the Tee section in the simplified model should be based on the depth of web in compression under combined bending and axial force, not that under bending alone (this will also take account of the effect of construction in stages). The slenderness derived from that model can be used to determine separate buckling resistance and axial resistance values and the verification carried out assuming a linear interaction relationship. (Note that, for this purpose, concrete that is cracked under combined effects should be taken as cracked in determining the axial resistance).

6.1.10 Continuous U-frame model, EN 1994-2, 6.4.2

For beams of uniform cross section of Class 1, 2 or 3, and with restraint to the bottom flange only by means of 'continuous inverted U-frames' comprising the slab and unstiffened[10] web plates, EN 1994-2, 6.4.2 gives the simple direct relationship for buckling resistance moment:

$$M_{\mathrm{b,Rd}} = \chi_{\mathrm{LT}} M_{\mathrm{Rd}}$$

where M_{Rd} is the design value of either the elastic or plastic resistance of the cross section in hogging bending (which is calculated using design strengths determined by dividing by γ_{M1} as noted in Section 6.1.2), as appropriate. The mode of buckling is illustrated in Figure 6.8.

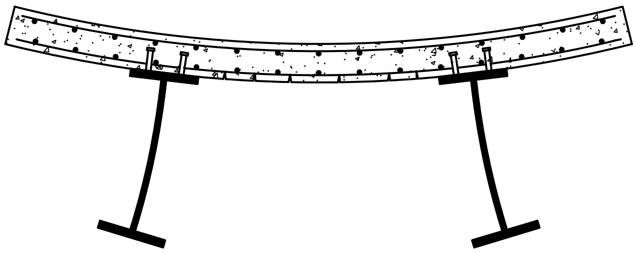

Figure 6.8 *Distortional buckling of bottom flanges*

As explained above, the reduction factor χ_{LT} depends on the slenderness $\bar{\lambda}_{\mathrm{LT}}$ but 6.4.2 only describes the model that may be used to determine M_{cr}, it does not give a solution for M_{cr}. The model that is derived is a beam that is unrestrained laterally but flexibly restrained continuously against torsion. No method of solution is suggested; either a computer elastic buckling analysis must be performed or a suitable 'textbook' model found. For this configuration, the manual method described above under *slenderness of beams with flexible intermediate restraints* but using the value of stiffness per unit length, rather than the stiffness at discrete spacing, may be easier to apply.

This model also does not consider combined bending and axial compression; the method of assessing interaction described above could be used.

[10] If the web does have transverse web stiffeners, it is best to ignore them in the U-frame model, since their greater stiffness (than the out-of plane bending of the web plate) would attract transverse moments between girder and slab that are difficult to design for.

6.1.11 Other effects in main girders

Restraint effects in integral bridges

The use of integral construction gives rise to axial compression in the main girders and, depending on the abutment detail, restraint moments at the ends. See further comment about deriving these effects in Section 9. The greatest restraint effects arise when temperature change is treated as the leading action and traffic loading as an accompanying action; clearly, the effects of thermal actions are less when they are accompanying actions and traffic loads are the leading variable action. Depending on the bridge configuration, either design situation might give the most onerous total effects at a particular position. Combined bending and axial load are not explicitly dealt with in EN 1994-2 but Hendy and Murphy[24] conclude that the simple linear interaction criterion:

$$\frac{N_{\text{Ed}}}{N_{\text{b,Rd}}} + \frac{M_{\text{Ed}}}{M_{\text{b,Rd}}} \leq 1$$

may be used, provided that care is taken to consider the level of the compressive force relative to the centroid and that $M_{\text{b,Rd}}$ is reduced for shear where it is sufficiently large.

Beams curved in plan

Where a flange is curved in plan, there is effectively a radial force on the flange as well as the direct stress due to bending. The radial force per unit length (along the flange) is given by dividing the flange force by the radius of curvature. The radial force will give rise to plan bending and needs to be restrained at intervals to avoid excessive bending stresses and displacements; intermediate transverse bracing and the inverted U-frames in ladder decks are effective means to provide this restraint. With such restraint, the plan bending of the flange and the forces on the restraints is easily determined by consideration of a continuous beam model. With a multi-girder deck and a small radius, the maximum spacing of the bracing is likely to be governed by this consideration.

Flanges curved in elevation

Where the main girders are haunched and the bottom flange is curved in elevation, there is a similar vertical radial force (flange force divided by radius) that is resisted by the web. The radial force is effectively applied across the width of the flange and thus gives rise to transverse bending of the flange (the flange must be symmetric about the web or there is a torsional effect) as well as vertical in-plane stresses in the web. Guidance on the effect on the flange design is given in Hendy and Murphy; the verification of the interaction of effects in the web is covered by EN 1993-1-5, 7.2, treating the transverse force as continuous, although the level of stress is not likely to be significant in most cases.

Plan bending due to interaction with cross girders

In ladder deck bridges, the non-uniform loading on adjacent cross girders (for example, one girder may be carrying both the TS pair of axles in one lane and LM3 axles in another lane) results in different deformations of the inverted U-frames and this forces the bottom flange into plan bending. This plan bending needs to be taken into account, and it should be amplified where the flange is in compression, to allow for second order effects.

6.2 Cross girders

The processes of section classification and calculation of bending and shear resistances are generally similar to those for the main girders. The main differences are noted below.

6.2.1 Effects in cross girders

The majority of the bending in cross girders is due to the local loading from the heavy axles of the TS and LM3 vehicles. However, the cross girders also provide restraint to the deck slab where it is in compression (the midspan regions): they need to be strong and stiff enough to provide this restraint. To provide adequate stiffness, the cross girder section properties (which may conservatively be based on a cracked section) should comply with 9.2.1(5) of EN 1993-1-5; for this purpose, an initial out-of straightness must be considered (of the form shown in Figure 9.2 of EN 1993-1-5). The value of w_0' is given by the sum of the specified construction tolerance (which may be taken as 20 mm in most cases) and the displacement due to transverse loading. The design force on the cross girder due to the restraint of the deck slab depends on this displacement, on the force in the slab and on second order effects: the value at the midspan of the cross girder is given by $q = 2\left(w_0' + \delta\right)N_{\mathrm{Ed}}/ab$ (see Hendy and Murphy[24] for further guidance).

6.2.2 Bending resistance

Cross girders in ladder decks receive very little end restraint from the main girders and are thus subject mainly to sagging moments. In service, there is therefore no requirement to verify lateral torsional buckling resistance, only bending and shear resistances. There is also no requirement for intermediate bracing.

The composite cross section is likely to be Class 1 or 2 and thus plastic moment resistance could be utilised. In practice, most designers choose to design only to elastic bending resistance; this avoids the need for SLS checks, the more complex evaluation of longitudinal shear resistance and the consequences of reduced stiffness in restraining the slab at ULS.

6.2.3 Shear resistance

The cross girders are likely to be without intermediate transverse web stiffeners, other than perhaps at midspan (when there is bracing between cross girders for the construction condition, see Section 7.3.2). The webs may well be more slender than the limit of $72\varepsilon/\eta$ given in EN 1993-1-1, 6.2.6 and thus the shear resistance has to be calculated in accordance with EN 1993-1-5.

For shear and bending interaction, it is conservative to consider the moment at midspan with the shear at the support but this is much simpler than considering interaction at several cross sections.

6.3 Deck slab

The deck slab acts globally as part of the composite section and locally as a plate element in bending. For verification of resistance at ULS, the combination of global and local effects in the slab should be considered. EN 1994-2, 5.4.4 says that effects should be combined, using a combination factor given by EN 1993-2, E.2. However, Annex E relates to effects in stringers of orthotropic steel decks. Guidance in PD 6696-2[40] and in Hendy and Johnson[23] suggests that the flexural effects in deck slabs do not need to be combined at ULS (the combination of shear effects should be considered); this is based on previous design practice and the acceptance that peak flexural effects do not coexist in deck slabs. For verification of the slab at SLS, see Section 6.8.

6.3.1 Bending resistance of slabs

The bending resistance of a slab is given by EN 1992-1-1, 6.1 and the assumed stress-strain distribution according to EN 1992-1-1, 3.1.7 (usually the rectangular stress distribution given in 3.1.7(3)) is used. Clearly, it is advantageous to place the layer of reinforcement that resists the largest local moment as the outer layer; the inner layer then resists the lesser orthogonal moment. Positioning of the reinforcement in the slab depends on the specified cover to the outer layer.

The minimum reinforcement cover required c_{min} is given by EN 1992-2, 4.4.1.2, according to the exposure class defined in 4.2; see also the National Annex. The value is defined by a basic requirement according to exposure class, modified by the service life, strength class, slab geometry and quality control. An allowance for deviation is then added (see 4.4.1.3 and the NA to BS EN 1992-2), to give nominal cover c_{nom} , which is the value to be stated on drawings and used in determining position of reinforcement for calculation of bending resistance. For highway bridges using C40/50 concrete or better, the rules usually lead to a value of c_{nom} = 35 mm for the top reinforcement (under the waterproofing) and 40 mm for the bottom reinforcement (provided that the soffit is more than 6 m above any roadway below).

Slabs in multi-girder decks

In a multi-girder deck, the greatest bending moments due to the traffic loading are transverse; both sagging and hogging moments are generated (sagging is usually greater). Global bending due to the differential deflection of the main girders also contributes to transverse bending in the slab.

Consequently, the transverse reinforcement is usually placed as the outer layer in multi-girder bridge slabs.

Slabs in ladder decks

In a ladder deck bridge, the greatest bending moments in the slab due to traffic loading occur in the longitudinal direction: both hogging and sagging moments are generated. With long cross girders the deflection under the most heavily loaded girder does make a significant contribution to the total sagging moment.

Transverse moments (due to the local effect of wheel loads) are smaller and mainly sagging in nature - the exception is over the length of the cantilever and immediately inboard of the main girders.

Consequently, the longitudinal reinforcement is usually placed as the outer layer in ladder deck bridge slabs. However, this reduces the lever arm for the transverse moment, which is a disadvantage for the transverse reinforcement in the cantilever, but the penalty is normally accepted.

6.3.2 Axial resistance of deck slab

The axial resistance of the deck slab is accounted for in the determination of the bending resistance of the composite section.

Where the slab is in axial compression, a question then arises about its slenderness, over the lengths between cross girders. Generally, where the spacing of the main girders does not exceed about 30 times the thickness of the slab, the slab may be considered as fully effective in compression (it acts as a plate supported on four sides). For a wider spacing of the main girders, the slab tends to act as a wide slender strut in compression and its slenderness reduces its axial resistance, because second order effects are introduced. However, the slab is not usually fully utilised in compression and the reduction is acceptable.

The second order effect in a slender element is determined according to EN 1992-1-1, 5.8.7 and 5.8.8 as a moment that is to be considered in conjunction with the compression force when verifying the resistance of the cross section of the slab. As noted on page 66, the local flexural effects in the slab do not need to be combined with the global effects, so the only moments to be considered for second order effects are those due to the vertical stress gradient through the slab and the nominal second order moment given by 5.8.8.2(3) of EN 1992-1-1.

6.3.3 Edge beams to deck slab cantilevers

The reinforced concrete edge beams provide stiffening to the edge of the cantilever slab and also support the parapets or barriers at the edge of the deck.

Edge beams are frequently constructed using precast units and as such are discontinuous. Edge beams that are cast *in situ* may be continuous or made discontinuous by regular jointing; the choice between discontinuous and discontinuous edge beams may be influenced by considerations of limiting crack width.

Loading on the parapet is a significant factor in the design of the edge beam to cantilever slab connection and may also be significant in the design of the cantilever slab.

6.4 Connections

Connections between steelwork components are made by welding or bolting. Generally, welding is used for connections made in the fabrication works and bolting is used for connections made on site, although for larger projects site welding may be employed.

Rules for bolted and welded connections are given separately in EN 1993-1-8.

6.4.1 Bolting

Most bolted connections in bridges transfer forces between parts by means of shear: tensile connections are rarely used between primary components.

EN 1993-1-8, 3.4.1 defines three categories of bolted shear connections:

Category A – Bearing type

Category B – Slip resistant at SLS

Category C – Slip resistant at ULS

In category A connections, non-preloaded bolts are used in clearance holes and parts have to slip slightly, relative to one another, to bring the bolts in bearing/shear. EN-1993-2 effectively rules out the use of this category, mainly because of load reversal and fatigue reasons.

Category B and C connections use preloaded bolts (traditionally referred to as High Strength Friction Grip (HSFG) bolts) that compress the 'faying' surfaces (the mating surfaces) with a specified level of preload. In category B connections, load is carried in friction between the faying surfaces at SLS but under the greater ULS loads, the bolts slip into bearing and shear (the bearing shear resistance is greater than the frictional resistance in most cases). In category C connections, no slip is allowed, even at ULS, and all the load has to be transferred in friction; such connections are used where the consequences of slip would be detrimental, for example when the connection is providing restraint against buckling.

Each bolted connection has many bolts. A key consideration in design is therefore the sharing of load between the bolts in the group. In some cases a quasi-elastic linear distribution of forces between bolts is used (i.e. load per bolt is proportional to its distance from a centre of rotation) and in some cases a plastic distribution can be used (i.e. the full resistance of each bolt is used).

For guidance on the design of bolted splice connections, see Section 8.4.

6.4.2 Welding

Welded connections are made using either fillet welds or butt welds. Design of welded connections is covered EN 1993-1-8, 4.

Full penetration butt welds are taken to be as strong as the weaker of the two parts that are joined by the weld.

In most cases, fillet welds are subject predominantly to shear along their length and in such cases the strength is given by the simplified method in EN 1993-1-8, 4.5.3.3 as:

$$f_{\mathrm{vw,d}} = \frac{f_{\mathrm{u}}/\sqrt{3}}{\beta\gamma_{\mathrm{M2}}}$$

Where f_{u} is the ultimate tensile strength of the weaker part joined and β is a factor that enhances the strength for steel grades S355 and below (because the weld metal is slightly stronger than the parent material).

Where fillet welds are subject to forces normal to their length as well as to shear along their length, the directional method of 4.5.3.2 is used.

Because weld details are a potential source of local defect, the choice of weld detail has a significant effect on fatigue performance - see Section 6.6.3.

6.5 Shear connection

Shear connectors are required on the top flanges of the girders, to provide the shear transfer that is required for composite action between the steel girder and the concrete slab. The most commonly used form of connector is the headed stud, though bar-and-hoop connectors are sometimes used. At ULS, the design process consists of deriving the value of the longitudinal shear and the verification of the resistance of the connectors and of the resistance of the slab adjacent to the connectors. For verification under fatigue loading and at SLS, see further comment in Sections 6.6.3 and 6.8.1 respectively.

6.5.1 Longitudinal shear

The longitudinal shear is the means by which load is transferred from the girder into the slab. Where a uniform composite section is designed elastically, the longitudinal shear is calculated from the vertical shear force using the simple relationship:

$$v_{\mathrm{L,Ed}} = V_{\mathrm{Ed}} \frac{A\bar{z}}{I}$$

where V_{Ed} is the vertical shear force on the cross section, I is its second moment of area, A is the area of the part of the cross section to which the shear force is transmitted (in this case the effective area of the slab) and $\bar{z}$ is the distance of the centroid of that area from the neutral axis of the whole cross section.

Where the composite section is designed plastically, a slightly more complex evaluation is needed; EN 1994-2, 6.6.2.2 provides rules for such evaluation. More complex evaluation is also needed if there is a concentrated introduction of shear force, for example at a change of cross section or where temperature and shrinkage effects are introduced at the end of a beam - see EN 1994, 6.6.2.4. If the beam cross section varies continuously (e.g. for a tapered section) the longitudinal shear can be evaluated by consideration of the difference in the force in the slab at different locations.

In hogging moment regions, where the slab is in tension, longitudinal shear may be calculated using uncracked section properties; this gives a safe value without the need for more complex calculation, even when the plastic resistance of the cracked section is relied upon. Short term uncracked properties may be used for this purpose.

In multi-girder decks, and for the cross girders of ladder decks, the effective area either side of the girder is usually the same and thus half the shear flow goes to each side. In ladder decks, particularly wide ladder decks, the effective width on one side of a main girder is likely to be greater than on the other; this should be taken into account when verifying the resistance on shear surfaces through the slab (surfaces a-a in Figure 6.9).

6.5.2 Positioning of shear connectors

The longitudinal shear varies along the length of the beam, being highest near the supports, and it is customary to vary the number and spacing of connectors to provide just sufficient shear resistance, for economy.

For the main girders, studs are set in groups of typically 2, 3 or 4 across the width of the flange; the spacing, and sometimes the number across the width, varies in a series of 'steps' along the beam, with wider spacing and/or fewer studs per row in regions of low shear. For cross girders in ladder decks, fewer studs are needed and the flange is often too narrow for more than two studs across the width.

The spacing of studs (for main girders and cross girders) needs to be coordinated with the spacing of transverse reinforcement, to avoid potential clashes in positions.

See Sections 8.6 and 8.7 for guidance about positioning and height of studs.

6.5.3 Resistance of shear connectors

The design shear resistance of a stud shear connector is given in EN 1994-2, 6.6.3.1. There the rules give the value as the lower of that for the steel shank of the stud itself and that for the concrete into which it is embedded. Studs are usually 19 mm diameter and for this size, in normal-density concrete of grade C40/50 or better, the resistance of the stud governs.

Over lengths where the stud spacing is constant, EN 1994-2 allows the ULS value of shear per unit length at any particular location to exceed the resistance per unit length by up to 10%, provided that the total shear over the length where the spacing is constant does not exceed the total design shear resistance over that length. However, if this potential enhancement were utilised, it is likely that SLS limitations would govern; this depends on the partial factors applied at ULS - see further comment in Section 6.8.1.

The welded connection of the stud must also be checked for fatigue, under cyclic loading - see Section 6.6.3. Fatigue may well govern the spacing of connectors in midspan regions.

No other types of shear connector are explicitly referred to in EN 1994-2 but block and hoop shear connectors are sometimes used (particularly in some of the connections at the end of integral bridges). Guidance on the design of this type of connector is given in PD 6696-2[40]. Note that the load/slip characteristics of stud and block/hoop connectors are different; design values of resistance cannot simply be added together if they are used in combination on a flange.

6.5.4 Longitudinal shear resistance of the deck slab

The slab must also be checked to verify its ability to transfer the longitudinal shear transmitted from the girder by the shear connectors. Verification of the adequacy of the slab on the potential failure surfaces (see Figure 6.9) uses the compression strut/reinforcement tie model that is set out in EN 1992-1, 6.2.4. The strut/tie model allows the designer to select the angle of the compression struts (within a specified range) but it is easiest to make the simple assumption that $\theta = 45°$.

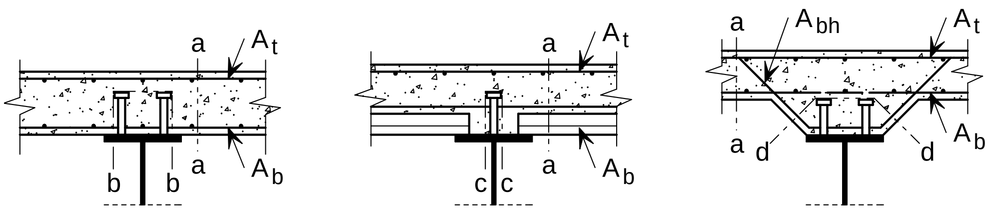

Figure 6.9 *Shear surfaces to be considered for verification of longitudinal shear resistance of slabs (taken from EN 1994-2, Figure 6.15)*

EN 1992-1-1, 6.2.4 requires that interaction of the shear on surfaces a-a with transverse bending should be taken into account; this situation applies both to the main girders in multi-girder decks and to the cross girders in ladder decks. Hendy and Johnson[23] advise that tension in reinforcement from bending should be accounted for using the method in EN 1992-2 (i.e. not full addition with shear). Direct tension should be fully combined with shear.

In the hogging moment regions of ladder deck bridges, the slab is in overall tension and the longitudinal reinforcement is called upon to provide the tensile resistance to global bending, local resistance to longitudinal bending and the transfer of longitudinal shear from the cross girder. Rules for interaction are not given in EN 1994-2 but it would be pragmatic to provide an area of reinforcement that is at least the sum of the areas needed to resist each of the effects separately.

See Section 8.7 for guidance on detailing the deck slab in the region of the shear connectors.

6.6 Fatigue considerations

Resistance to fatigue is covered generally in both EN 1993-2 and EN 1994-2. Detailed rules for structural steel are given in EN 1993-1-9; for reinforcing steel in EN 1992-1-1; and for stud connectors in 6.8.7.2 of EN 1994-2. It is unlikely that an assessment is needed for concrete (see the simple check in EN 1992-1-1, 6.8.7(2)).

The rules for determining resistance to fatigue are quite separate from those for determining the fatigue loading; the latter are given in EN 1991-2 (see Section 5.4.7 above).

6.6.1 Fatigue assessment - structural steel

EN 1993-1-9 presents methods for both "damage-tolerant" and "safe-life" methods of fatigue assessment but the NA to BS EN 1993-1-9 specifies the use of the safe life method, unless otherwise agreed by the Maintaining Authority. (The damage-tolerant method uses a lower partial factor on fatigue strength and requires a regular in-service inspection and maintenance regime to ensure adequate reliability.)

In the safe life method, the verification is based on either damage equivalent stress ranges (at 2×10^6 cycles) or on a damage accumulation method (more commonly known as Miner's summation); for highway bridges, the former method is used in all but exceptional cases.

There is some duplication in the verification procedures in EN 1993-1-9 and EN 1993-2; this leads to some minor inconsistency in terminology but the intent of both is the same. In EN 1993-2, 9.5.1 the verification for direct stress is expressed as:

$$\gamma_{\mathrm{Ff}} \Delta\sigma_{\mathrm{E2}} \leq \frac{\Delta\sigma_{\mathrm{c}}}{\gamma_{\mathrm{Mf}}}$$

That is, the design value of the stress range corresponding to 2 million cycles should not exceed the design value of fatigue strength for the particular detail at 2 million cycles.

There is a similar expression for shear stress (see further comment on page 75).

The NA to BS EN 1993-1-9, 5(2) allows the effect of out of plane deflections of slender plate panels (web breathing) to be neglected in most bridge girders.

6.6.2 Design value of stress range (structural steel)

The design value of stress range may be determined as follows, with reference to the relevant clauses of EN 1993-2:

1. Determine the stress range due to the passage of the fatigue load model 3 vehicle[11] (see Section 5.4.7). This is referred to in 9.4.1(3) as $\Delta\sigma_p$.

2. Where there is a stress magnification due to local geometry that is not taken account of in the classification detail, determine an appropriate stress concentration factor[12] k_f (>1) and apply it as an amplification factor to $\Delta\sigma_p$. This amplification is not mentioned in 9.4.1 but the factor is applied in EN 1993-1-9, 6.3.

3. Determine a 'damage equivalence factor' λ in accordance with 9.5.2. This factor is the product of four sub-factors as follows:

   (a) λ_1 is a damage effect factor that depends on the length of the critical influence line[13]. Its values is given by rules in 9.5.2(2) (these have not been varied by the NA to BS EN 1993-2).

   (b) λ_2 is a factor that depends on intensity of heavy vehicles in the "slow lane" (it seems to be presumed that this is the lane that determines $\Delta\sigma_p$). Determination of this factor requires the parameter N_{Obs} and the average weight of lorries Q_{m1}. According to EN 1993-2, Q_{m1} depends on the spectrum of commercial vehicles (comprising sets of n_i vehicles of gross vehicle weight Q_i) for the particular highway and lane. However, the NA to BS EN 1993-2 simply sets the value of Q_{m1} as 260 kN. The value of N_{Obs} is given by the NA to EN 1991-2, in relation to 4.6.1(3).

   (c) λ_3 is a factor that depends on the design life of the bridge. For the 120 year design life given by the NA to BS EN 1990, the value given by 9.5.2(5) is 1.037.

   (d) λ_4 is a factor that allows for the effect of loading in lanes adjacent to that for which $\Delta\sigma_p$ was calculated. To determine its value, traffic spectrum parameters are required and also influence coefficients for each lane. NA.2.41 to BS EN 1993-2 declares that the average weight is the same for all lanes (it does not mention numbers of vehicles per lane but the numbers may be taken as given in the NA to BS EN 1991-2). The expression for determining λ_4 is given in 9.5.2(6). Conservatively, for 2 lanes of equal traffic intensity and where the influence coefficient for each lane is the same[14], the value is 1.15.

   (e) The value of $\lambda = \lambda_1\ \lambda_2\ \lambda_3\ \lambda_4$ but not more than the limit given by 9.5.2(7) (which is not varied by the NA to BS EN 1993-2).

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[11] It is not normally necessary to consider global and local effects separately because the steel elements of these types of bridge are subject predominantly to one or the other.

[12] Stress concentration factors are given in text books. Factors for 'large' circular and elongated holes are given in PD 6696-1-9; they have been taken from BS 5400-10. The factors do not need to be applied to bolt holes but should be applied at cope holes.

[13] Influence line lengths for local effects are usually much shorter than for global effects. If the stress range is due to a combination of local and global effects, the λ_2 parameters should be evaluated separately and the damage due to combined effects evaluated according to EN 1993-2, 9.5.4

[14] But if the coefficient for the second lane is only 50% of that for the first lane, the value is only 1.01. It is therefore well worth taking account of the influence coefficients.

4. Determine the design value of the stress range as:
$\gamma_{Ff}\Delta\sigma_{E2} = \gamma_{Ff}\ \lambda\phi_2\Delta\sigma_p$
where γ_{Ff} is given by the NA to BS EN 1993-2 as 1.0 and ϕ_2 is given by 9.4.1(4).

In the above procedure, determination of the parameters λ_2 λ_3 and λ_4 depends on the slope of the fatigue curve, expressed as the parameter m, with $m = 5$ for structural steel under direct stress.

The same procedure is used for damage equivalent shear stress $\Delta\tau_{E2}$ in structural steel and stud connectors. For shear stress in stud connectors, the value of m to be used in determining λ_{v2} λ_{v3} and λ_{v4} is 8 and $\lambda_1 = 1.55$ (for road bridges); there is no ϕ parameter - see EN 1994-2, 6.8.6.2(4).

6.6.3 Fatigue strength (structural steel)

The fatigue endurance of a particular detail is commonly expressed in terms of a level of applied stress that can, with a certain level of reliability, be expected to be resisted for a (large) number of repeated applications. In EN 1993-1-9 a range of structural details is classified according to a design value of stress level that will achieve the reliability target set by EN 1990 for 2×10^6 cycles.

Fatigue cracks grow from very tiny imperfections when there is a fluctuation of stress across the imperfection that tends to open it. However, this does not mean that only an overall tensile stress in the part is relevant to crack growth, because any stress range will tend to open and close the initiating imperfection; the reference stress level for classification is therefore the stress range, i.e. the difference between maximum and minimum stress level due to the application of the cyclic load.

Fatigue detail classification

The fatigue detail classifications (called detail categories in EN 1993-1-9) relate to the size of the potential imperfections at welds, holes or other discontinuities, and their relationship to the direction of the stress variation. The greater the potential imperfection, the lower the stress range that can be tolerated for a given fatigue endurance.

EN 1993-1-9 presents ten separate tables of detail categories; for composite highway bridges, Tables 8.1 to 8.5 are relevant. The detail categories range from 160 (as-rolled plates and sections without any fabrication work on them) to 36 (cruciform joints transmitting load through fillet or partial penetration welds). The number of each category gives the reference stress level $\Delta\sigma_c$ or $\Delta\tau_c$ in N/mm^2.

The reliability of the classification of a particular detail depends on a presumption about the quality of workmanship; in some cases specific mention is made of non-destructive testing (NDT). There are no detailed references to EN 1090 but that Standard does provide four ‘execution classes’. The ‘default’ class in EN 1090-2 is Class EXC2, but Class EXC3 will normally be needed for highway bridges and this is the class specified in the *Model Project Specification*[10]. The NA to BS EN 1993-1-9 effectively restricts the reference value of fatigue strength of some details in Table 8.1 to 8.5 unless there are “special testing and inspection requirements”. It may be assumed that Class EXC3 is adequate to ensure that the reference stress level is that for the detail category in the Tables or, where listed in Table NA.1, the minimum strength level in that Table. The requirements that would be needed to confirm a quality adequate to use reference stress levels higher than those in Table NA.1 are not normally practical or achievable.

Generally, lower category details are introduced by making attachments to the steelwork component. Fatigue assessment therefore needs to be carried out chiefly in regions of significant variations of stress and at the locations of the attachments. Typical locations requiring detailed assessment are web stiffener details at intermediate supports and splice connections (welded or bolted).

The attachment of web stiffeners or other elements not carrying load in the stressed direction usually produces a category 80 detail. Reinforcing plates and bearing plates welded to the underside of the flange usually introduce a category 40 or 45 detail. Shear stud connectors introduce a category 80 detail in the plate to which they are attached, but see further comment below about the shear stress category for load transferred to the stud. Preloaded bolted splices introduce category 112 details for double covers or category 90 for single covers. Transverse butt welds create category 80, 90 or 112 details and a size effect reduction factor applies for thicknesses over 25 mm (see EN 1993-1-9, 7.2.2).

For weathering steel, Table 8.1 reduces the detail category for plain steel (details 1 to 5 in the Table) but since the NA limits these categories, as noted above, the reduction has no effect in practice.

Fatigue detail classification for shear transfer

Most of the details in Tables 8.1 to 8.5 relate to direct stress $\Delta\sigma$. Two detail groups relate to shear stress: details 6/7 in Table 8.1 (not normally a concern for bridges) and details 8/9 in Table 8.5. For these the shear stress $\Delta\tau$ is used but otherwise the procedure for determining the design strength is the same as for direct stress. Shear stress range is also mentioned in details 1 to 3 of Table 8.5, for shear transmitted through a Tee or cruciform joint.

For shear stud connectors, EN 1994-2, 6.8.3 gives a reference stress $\Delta\tau_c$ of 90 N/mm^2, for a shear stress determined on the basis of the cross sectional area of the shank of the stud.

Design value of fatigue strength

The design value of fatigue strength is derived by dividing the reference stress $\Delta\sigma_c$ or $\Delta\tau_c$ by the partial factor γ_{Mf}. The value given by the NA to BS EN 1993-1-9 is $\gamma_{Mf} = 1.1$.

Shear connection - interaction of shear and direct stress

In addition to the separate verifications of the category 80 detail for the flange to which a stud is connected and the fatigue resistance of the stud in shear, EN 1994-2, 6.8.7.2 requires an interaction check for flanges in tension to which studs are attached.

6.6.4 Fatigue assessment (reinforcing steel)

Where the reinforcing steel is in tension, the fatigue endurance must be assessed. Generally, a procedure similar to that for structural steel is used but the calculation of stress range is more complicated and the fatigue strength to which damage equivalent stress is evaluated is at 10^6 cycles instead of at 2×10^6 cycles.

For fatigue assessment of reinforcement, the stress range may be due to a combination of global and local effects (i.e. global tension due to hogging bending and local bending due to the wheels of the fatigue vehicle). EN 1994-2, 6.8.6.1(3) provides a way of combining the two effects (the damage equivalence factor is different for each, because the length of the influence line is different) but the total stress could conservatively be taken as a local effect.

The verification of fatigue endurance is expressed in EN 1992-1, 6.8.5 as:

$$\gamma_{\mathrm{F,fat}} \Delta\sigma_{\mathrm{S,equ}}(N^*) \le \frac{\Delta\sigma_{\mathrm{Rsk}}(N^*)}{\gamma_{\mathrm{s,fat}}}$$

That is, the design value of the stress range should not exceed the design value of fatigue strength at N^* ($=10^6$) cycles.

6.6.5 Design value of stress range (reinforcing steel)

Rules for the determination of stress range in the reinforcement are given in EN 1994-2, 6.8.5.4; it requires knowledge of stresses as the fatigue vehicle traverses the bridge. The behaviour of the cracked concrete is non-linear, because of cracking and because of tension stiffening; the maximum stress is therefore determined on the basis of the stress due to the maximum moment on the cracked composite section $M_{\mathrm{Ed,\ max,f}}$ plus an additional stress due to tension stiffening; the minimum stress is more complex to calculate but 6.8.5.4(2) gives a simple expression, for the case where the minimum moment $M_{\mathrm{Ed,\ min,f}}$ also causes tensile stress. If the minimum moment causes compression, uncracked properties may be used to determine the minimum stress. Implicit in this procedure is the need to know the total moments on the composite section, not just those due to the fatigue vehicle - see Section 5.4.7. Note also that the weight of the fatigue vehicle is increased by a factor of 1.75 or 1.4, according to EN 1992-2, NN.2.1(101), as also mentioned in Section 5.4.750.

As for structural steel, the damage equivalent stress is determined from the stress range. EN 1994-2, 6.8.6.1(2) gives the following expression for the equivalent stress range:

$$\Delta\sigma_{\mathrm{E}} = \lambda\phi \left| \sigma_{\mathrm{max,f}} - \sigma_{\mathrm{min,f}} \right|$$

However, EN 1992-1 uses the term $\Delta\sigma_{\mathrm{S,equ}}$ for this stress range and refers to EN 1992-2 for its evaluation. Annex NN.2 of EN 1992-2 gives a similar procedure to that in EN 1993-2 (described in Section 6.6.2 above) but the rules in Annex NN.2 are slightly different:

- $\lambda_{\mathrm{s,1}}$ is given by Figures NN.1 and NN.2
- The expressions for $\lambda_{\mathrm{s,2}}$ $\lambda_{\mathrm{s,3}}$ and $\lambda_{\mathrm{s,4}}$ are different and the value of m to be used in determining them is 9 (rather than 5).
- An amplifier for surface roughness ϕ_{fat} is also applied (ϕ_{fat} is given in EN 1991-2 Annex B).

The design value of stress range is thus $\gamma_{\mathrm{F,fat}}\Delta\sigma_{\mathrm{S,equ}}$ and the NA to BS EN 1992-1-1 gives the value of the partial factor $\gamma_{\mathrm{F,fat}} = 1.0$.

6.6.6 Fatigue strength (reinforcing steel)

EN 1994-2, 6.8.3(2) refers to EN 1992-1-1 for the fatigue strength of reinforcing steel. EN 1992-1-1, 6.8.4 gives the value of the resisting stress range $\Delta\sigma_{Rsk}$ at $N^* = 10^6$ cycles as 162.5 MPa for straight bars. However, for bent bars (including cranked bars) a reduction factor ζ ($=0.35 + 0.026\ D/\phi$) must be applied; this is very severe for bars bent around a minimum mandrel diameter (4ϕ for bars up to 16 mm, 7ϕ for larger bars).

The design value of the fatigue strength is thus $\Delta\sigma_{Rsk}$ / $\gamma_{S,fat}$ and the NA to BS EN 1992-1-1 gives the value of the partial factor $\gamma_{S,fat} = 1.15$.

6.7 Selection of steel sub-grade

All parts of structural steelwork are required to have adequate notch toughness, to guard against the possibility of brittle fracture. Rules for ensuring adequate toughness and through-thickness ductility are given in EN 1993-1-10.

Brittle fracture can initiate from a stress concentration when transient loading is applied at low temperature, if the material is not sufficiently ‘tough’. The toughness of steel material is expressed as a Charpy impact value (determined from tests carried out on a sample of material). The requirements for toughness depend on the thickness of the material, its minimum temperature in service, the stress level and the rate of loading.

The verification of adequacy of toughness is expressed in EN 1993-1-10, 2.3 as a “maximum permitted element thickness”. (Verification by a fracture mechanics method may also be used but this is a specialist technique and is not suitable for ordinary highway bridge design.) Thickness values are tabulated in relation to a reference temperature and to the specified material toughness; the tables can be used either to determine a maximum permitted thickness for a given combination of reference temperature and material toughness or may be used to determine the required material toughness for a given element thickness and temperature.

Reference temperature

The reference temperature is expressed in EN 1993-1-10, 2.2(5) as:

$$T_{Ed} = T_{md} + \Delta T_r + \Delta T_\sigma + \Delta T_R + \Delta T_{\dot{\varepsilon}} + \Delta T_{\varepsilon cf}$$

The first two parameters express the lowest temperature that the steel will actually experience. This depends first on the minimum shade air temperature at the bridge site, T_{md}. The second parameter makes allowance for radiation loss, depending on the type of construction (because radiation losses differ, depending on whether the bridge is of steel, concrete or composite construction) and on the deck surfacing (because of differing radiation and thermal inertia characteristics). Additionally, radiation losses give rise to temperature difference (the non-linear vertical temperature gradient given in EN 1991-1-5, 6.1.4.2); this effect is considered to occur simultaneously with uniform temperature change (see 6.1.5.1 and the NA to BS EN 1991-1-5) and thus, strictly, the adjustment ΔT_r varies through the depth of the steel section. However, Hendy and Murphy[24] note that since the vertical difference profile includes a component of uniform temperature change, the actual addition for temperature difference is small and may reasonably be neglected. Thus, for any part of the steelwork, the value (T_{md} + ΔT_r) in the above expression is the minimum effective bridge temperature, as given by EN 1991-1-5, 6.1.3. Values for minimum shade temperature for UK bridges are given in the NA to BS EN 1991-1-5 (note that the values of minimum and maximum ambient temperature in the isographs need to be adjusted for a 120 year return period). Typical values of $T_{md} + \Delta T_r$ (= $T_{e.min}$) for composite bridges in the UK are −15°C in England and −18°C in Scotland.

The steel stress adjustment ΔT_σ takes account of the level of stress in the element, relative to its yield strength. To determine the stress in the element, the combination of actions given in EN 1993-1-10, 2.2(4) should be used, which has temperature change as the leading action. Consequently, as the clause notes, the stress in the element for this design situation will not normally exceed 75% of the yield strength (and in most cases is significantly less). The adjustment ΔT_σ is not made explicitly but is made within the columns of Table 2.1. However, the NA to BS EN 1993-1-10 does not make use of this adjustment but uses an adjustment $\Delta T_{R\sigma}$ (see the fourth component of ΔT_R as described below).

The adjustment ΔT_R is given by the NA as the sum of a number of separate adjustments as follows:

(a) ΔT_{RD} is an adjustment for certain details that have better or worse susceptibility to brittle fracture.

(b) ΔT_{Rg} is an adjustment for sites of gross stress concentration, where the element is more susceptible to brittle fracture.

(c) ΔT_{RT} is an adjustment for situations where the Charpy test temperature is higher than the minimum temperature of the steel element (i.e. higher than T_{md} + ΔT_r). For bridges, there is no adjustment for up to 20°C difference but a greater difference is not acceptable - the specified test temperature must never be more than 20°C higher.

(d) $\Delta T_{R\sigma}$ is an adjustment for stress less than 75% of the tensile yield stress. This adjustment should be made instead of the use of the columns for 50% and 25% of yield in Table 2.1.

(e) ΔT_{Rs} is an additional adjustment for steel strength grades greater than S355 (i.e. additional to the use of the appropriate row for strength in Table 2.1).

Where load is applied suddenly, an adjustment ΔT_ε is made; its value is given by EN 1993-1-10, 2.3.1(2). However the combination of actions appropriate to impact loads at low temperatures is not clear, since the evaluation is already considered to be an accidental situation (see EN 1993-1-10. 2.2(4) Note 1) and separate accidental actions would not normally coexist; nevertheless, it could conservatively be assumed that the two actions (impact load and low temperature) occur simultaneously for elements such as crash barrier bases.

The adjustment $\Delta T_{\varepsilon cf}$ is for cold formed material but this is not normally relevant for composite bridges, because that material is not used.

The adjusted reference temperature T_{Ed} is typically between 30°C above the steel temperature T_{md} + ΔT_r (e.g. for longitudinal welds between web and flange in compression zones) and 30°C below (e.g. for welded lapped connections in tensile zones).

Material toughness sub-grades

Structural steel to EN 10025 is manufactured to specified sub-grades, to give a minimum specified Charpy impact value at a prescribed temperature. The range of sub-grades for low alloy and fine grain steels is as follows:

Sub-grade	Charpy Impact Value	Test Temperature
JO	27J	0°C
J2	27J	−20°C
K2, M, N	40J	−20°C
ML, NL	27J	−50°C

NOTE: Sub-grade JR is not suitable for bridges, because the test temperature is 20°C

Bridge steelwork of strength grade S355 is typically sub-grade J2 - this is appropriate for the usual values of (adjusted) reference temperature. For thick flanges sub-grade K2/M/N may be needed; sub-grade ML/NL may sometimes be needed for very thick plates with 'poor' (very susceptible) details.

If the actual thickness of the part exceeds the limiting thickness for the material grade, either a tougher grade must be selected or the detail must be revised.

It is not necessary to use the same sub-grade throughout the whole structure - thick tension flanges could be K2 grade while the remainder is J2, for example. The drawings should always identify clearly where different sub-grades are to be used, especially where the required sub-grade depends on location as well as on thickness.

6.7.2 Through thickness ductility

Specifying though-thickness testing of steel plate is rarely necessary. In high risk situations (i.e. where there is a risk that welding will cause lamellar tearing) Z35 quality should be specified; otherwise, no requirement should be made. The following are considered to be high risk situations:

- Tee joints where the governing thickness exceeds 35 mm
- Cruciform joints where the governing thickness exceeds 25 mm

The governing thickness is the thickness of the 'incoming' material (the element(s) attached to the face of the other element) for butt welded joints or the throat size of the largest fillet weld, for a fillet welded joint.

Although through-thickness quality is not significantly more expensive, availability may be a problem, particularly in small quantities.

The NA to BS EN 1993-1-10 chooses 'quality class 2' according to 3.1 and this allows the selection by the designer of Z quality for specific locations. The procedure in 3.2 does not need to be applied. The fabricator will also take measures to minimize the risk of lamellar tearing, by careful sourcing of material and selection of procedures.

Even when the risk of lamellar tearing is not high, certain areas of plate need to be checked for internal discontinuities (laminations) - examples are web plates to which bearing stiffeners are attached and cruciform or Tee joints subject to tensile loads. Such areas should be identified on the drawings.

For further guidance on through-thickness properties, see GN 3.02.

6.8 Serviceability Limit State

At the serviceability limit state, verifications of stress levels, deflections and crack widths in concrete are required. Action effects are calculated using elastic global analysis and allowing for the effects of shear lag, shrinkage and creep.

6.8.1 Stresses

Stress levels at SLS are verified for the characteristic combination of actions – i.e. unfactored values of characteristic permanent and variable actions (see Section 4.1.2). Essentially, the requirement is to ensure that there is no inelastic behaviour.

The stress in the structural steel is limited to yield stress divided by $\gamma_{M,ser}$ - see EN 1993-2, 7.3(1); the NA to BS EN 1993-2 sets $\gamma_{M,ser} = 1.0$.

The stress in the concrete is limited by EN 1994-2, 7.2.2(2) to $k_1 f_{ck}$ (the NA to BS EN 1992-1-1gives $k_1 = 0.6$) for durability reasons. (But note that if the stress exceeds $0.45 f_{ck}$ higher creep values need to be used, and thus the long-term modulus will be reduced, to allow for non-linear creep - see EN 1992-1-1, 3.1.4(2)).

The shear force per connector is limited by EN 1994-2, 6.8.1(3) to $k_s P_{Rd}$ (the UK NA gives $k_s = 0.75$) for fatigue reasons. That clause is in the fatigue section of EN 1994-2 and fatigue is actually an ultimate limit state but the clause places a limit on the value of the total shear force transmitted under characteristic (i.e. SLS) loading, rather than on the range of force due to cyclic loading. However, since the 'average' partial factor on loading at ULS is likely to be close to 1.35 (the NA to BS EN 1990 sets the γ factors on actions at 1.35 for concrete weight and traffic loading and 1.2 for surfacing) the SLS and ULS requirements are comparable. However, this means that the 10% 'overstress' at ULS allowed offers little benefit, since the SLS requirements (without the 10% allowance) will then govern.

6.8.2 Deflections

The geometry of the bridge should be such that deflection under characteristic loading does not cause the infringement of a clearance gauge; this is usually achieved by selecting a suitable soffit profile (deflections are normally relatively small). Performance criteria related to deflection for road bridges are given in EN 1993-2, 7.8; reference is made to dynamic performance and to avoidance of adverse effects on drainage of the road surface. Dynamic performance of ordinary highway bridges is not usually a problem. The modest change of road profile is also rarely a problem for drainage (the exception might be if the bridge deck were level).

6.8.3 Cracking of concrete

Generally, the amount of reinforcement in the deck slab is determined by strength requirements at ULS, in resisting both global loading (giving rise to tension in the slab in hogging moment regions) and local loading (giving rise to bending of the slab under the actions of wheel loads etc.). For these purposes, only the required area of reinforcement needs to be determined; the actual bar size and spacing are not of direct concern, although an initial selection is usually made at this stage.

Once the level of reinforcement has been determined by the ULS verification, consideration must be given to control of cracking of concrete at SLS, for durability reasons. There are two main causes of cracking in the slab, imposed deformation (due to shrinkage and differential temperature) and tensile stress due to direct loading.

Control of cracking is covered in EN 1994-2, 7.4.

Minimum reinforcement

Wherever the slab is in tension due to indirect loading (e.g. shrinkage), a minimum area of reinforcement is required, according to EN 1994-2, 7.4.2. This area is usually much less than that either to control cracking due to direct loading or to provide resistance at ULS. Note that the minimum required area is less when smaller bars are used, because the ratio of bond area to cross sectional area is greater and this permits the reinforcement to be at a higher stress level across the crack - see Table 7.1 of EN 1994-2.

Control of cracking due to direct loading

Tensile stresses in the slab are caused by global loading (tension in the slab in hogging moment regions) and by local loading (bending of the slab under local load). However, the limitations placed on crack widths for durability reasons relate only to the quasi-permanent load combination and for this combination the bending stresses in the slab are small (the slab is spanning no more than about 4 m and carrying only its self weight and that of the surfacing in this combination).

The rules for limitation of crack width given in EN 1994-2, 7.4 relate the design crack width to the limiting calculated crack width w_{max} given by EN 1992-2, 7.3.1(105); the value of w_{max} given there, for the quasi-permanent load combination, is w_{max} = 0.3 mm and this value is adopted by the NA to BS EN 1992-2.

Global tensile stresses in the hogging moment region (including the effects of shrinkage and differential temperature) are enhanced by an addition for 'tension stiffening'. This addition arises because the uncracked concrete (between the cracks) provides a stiffer cross section and attracts more load per unit width of slab than does the fully cracked cross section. To maintain equilibrium and strain compatibility, the stress and the strain in the reinforcement across the cracks increases, with corresponding decreases in the reinforcement within the concrete. There is no overall change in the moment distribution in the hogging region, only the local increase in reinforcement stress across the cracks. The increase depends on the tensile strength of the concrete and the relative section properties of the cracked and uncracked sections; it is thus a simple addition to the stress due to direct loading.

Once the total tensile stress is known, a maximum bar spacing can be found from EN 1994-2, Table 7.2, for the limiting crack width w_{max}. Additional to the limit on spacing, there is a limit on maximum bar size, depending on the total tensile stress; this uses the same relationship between bar size and stress level as for the determination of minimum reinforcement area (see above), again dependent on the ratio of bond area to cross sectional area. However, the stress in the reinforcement under the quasi-permanent loading is unlikely to be sufficiently high to impose any practical upper limit on bar size.

7 DETAILED DESIGN: CONSTRUCTION STAGE

7.1 General

Designers are required to consider how the structure is to be built and to design the structure to sustain the appropriate actions (loading) for at least one viable and safe erection scheme. Effects such as the 'locked in' stresses that need to be taken into account in the design for the in-service stage and allowances for permanent deformations (deflection under dead load) are determined from the analysis of the construction stage.

The adequacy of the steel girders during construction, without the benefit of composite action with the deck slab, is a significant consideration during the design of any composite highway bridge. The bare steel sections lack both the contribution to strength from the presence of the slab and the restraint from the slab against buckling and lateral loads.

7.1.1 Construction sequence

The construction sequence that most commonly needs to be evaluated for a composite bridge is completion of the substructures, up to bearing level, erection of the structural steelwork (piece by piece), provision of formwork and casting the deck slab, and finally completion of the surfacing and fixtures such as barriers and drainage. Each construction stage needs to be analysed; a series of models of the partly completed structure is required for each stage. Where construction methods such as launching and transportation of the part-completed structure are used, the local effects at temporary support positions need to be evaluated.

Where it is not practicable to cast the full length of deck at once, the series of analytical models must represent the development of the composite structure as the portions of slab are cast.

7.1.2 Girder erection

Girder lengths are usually chosen to suit transportation, although the weight of individual pieces may limit the sizes where crane access is restricted. Strength verification at this stage is unlikely to require detailed evaluation but stability and buckling resistance do require careful consideration, particularly before bracing or cross girders are fully installed.

7.1.3 Bracing

Bracing of the steelwork in the bare steel and partly complete stages is a key to the effective performance of the main girders. Several bracing schemes may need to be evaluated.

7.1.4 Slab construction

Although deck slabs have traditionally been cast on temporary timber falsework, the use of permanent formwork, notably precast concrete planks that form part of the final slab and reinforced fibre panels, are now very common. Timber falsework is often supported off the bottom flanges of the girders; precast permanent formwork sits on the top flanges and thus needs to be considered as a destabilising load.

Whichever type of formwork, the weight at the wet concrete stage imposes quite high stresses in the top flanges of the girders: their strength and stability at this stage require a detailed evaluation of the progressive changes in structural behaviour as load is added.

The weight of the concrete cantilevers needs particular attention, because of the moment (about the longitudinal axis) that is imposed on the outer girder. See further comment in Section 7.2.3.

7.2 Design of main girders

7.2.1 Cross sectional resistances

As mentioned in Section 6.1.1, steel cross sections are classified in relation to the local buckling performance of the elements of the cross section; the bending resistance of a steel cross section is given by EN 1993-2, 6.2.1 for Class 1, 2 and 3 cross sections.

Class 4 cross sections are most likely to be encountered at the bare steel stage in midspan regions, because the web depth/thickness ratio is likely to be high and at that stage more than half of the web will be in compression, assuming that the top flange is smaller than the bottom. At an intermediate support, the composite cross section for a deep girder may also be Class 4. For very deep slender webs longitudinal stiffeners are sometimes provided to improve the effectiveness of the section, although such stiffeners add considerably to fabrication costs.

For Class 4 cross sections, the effective cross section is given by EN 1993-1-5, 4.3. The rules allow the determination of an 'effective width' (less than the gross area) of a panel of a slender web; the difference between the gross area of the web and the effective area is treated as a 'hole' for the determination of section properties (see Figure 7.1). Both the size and position of this hole are given by 4.3, according to the slenderness and the variation of longitudinal stress across the panel width. The rules for effective areas also cover longitudinally stiffened sections.

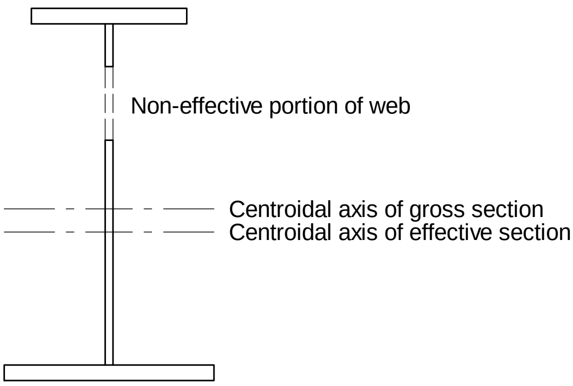


Figure 7.1 *Class 4 effective cross section (sagging bending)*

The shear resistance of cross sections is unaffected by the classification. Shear forces during construction are much less than in service and the resistance requirements will not normally need to be verified explicitly for the construction stage. However, if launching is used, midspan regions (of the final configuration) will be heavily loaded during the launching procedure - see further comment in Section 7.2.4.

7.2.2 Buckling resistances

Non-dimensional slenderness

The buckling resistances of the main girders at the wet concrete stage depend very much on the bracing to the girders, which determines the lateral torsional buckling slenderness of the girders. With staged construction (concreting part lengths of the bridge at a time) the situation is further complicated by the continuous restraint that is provided to the top flanges of parts of the spans.

In EN 1993-2 the rule in 6.3.2.2 for determining the reduction factor does not give an explicit expression for non-dimensional LTB slenderness, it only defines it in terms of elastic critical moment. One means to derive the slenderness is to determine M_{cr} by a computer buckling analysis; such determination is almost essential if non-uniform situations are to be verified (e.g. irregular spacing of restraints, or partly cast deck slab). The results of such analysis can then feed into either 6.3.2.2 or the general method of 6.3.4.1. But if the situation is essentially regular, manual determination of slenderness is possible.

For manual calculation of the slenderness required in 6.3.2.2, the following general expression can be used:

$$\bar{\lambda}_{\mathrm{LT}} = \frac{1}{\sqrt{C_1}} UVD \frac{\lambda_z}{\lambda_1} \sqrt{\beta_w}$$

where:

C_1 is a parameter dependent on the shape of the bending moment diagram

U is a parameter dependent on the section geometry

V is a parameter related to the slenderness and section geometry

D is a parameter to allow for the destabilising effect of the applied loading

λ_z is the slenderness over the half wavelength of buckling ($=L_w/i_z$)

β_w is a parameter related to $M_{b,Rd}$

Guidance on the calculation of the values of these parameters is given in Appendix C.

For manual determination of slenderness, there are two alternative configurations to be considered; with effective (i.e. essentially rigid) intermediate restraints and with flexible intermediate restraints. The buckling modes for the two are quite different and thus some of the parameters in the above expression are calculated differently.

Slenderness for beams with effective intermediate restraints

At the bare steel stage, effective intermediate restraints are only likely to exist if there is plan bracing to the top flange and triangulated bracing between pairs of beams in hogging moment regions (or alternatively, the beam is braced laterally to a ‘rigid’ restraint such as an adjacent portion of deck). For constructional reasons, plan bracing to the top flange is a non-preferred form of restraint but it is sometimes used.

In such cases, LTB occurs in the lengths between restraint positions and the mode is the classic LTB mode.

Slenderness for beams with flexible intermediate restraints

Where intermediate restraints are flexible, the mode of buckling is with one or two half wavelengths over the span, with the restraint positions in each half wavelength being displaced by the buckling. This mode occurs in multi-girder and ladder deck bridges during construction when there is no plan bracing; the only bracing is in planes (triangulated bracing or stiff cross girders) between beam pairs. These planes offer torsional restraint to the main beams, by virtue of the vertical stiffness of the main beams themselves. The mode of buckling with torsional restraints is shown diagrammatically in Figure 7.2, for a single half wave in a simply supported span. The mode is illustrated with stiff cross girders but could equally be for beams with triangulated bracing. With some configurations, the second mode of buckling, with two half waves in the span, might occur at a lower load.

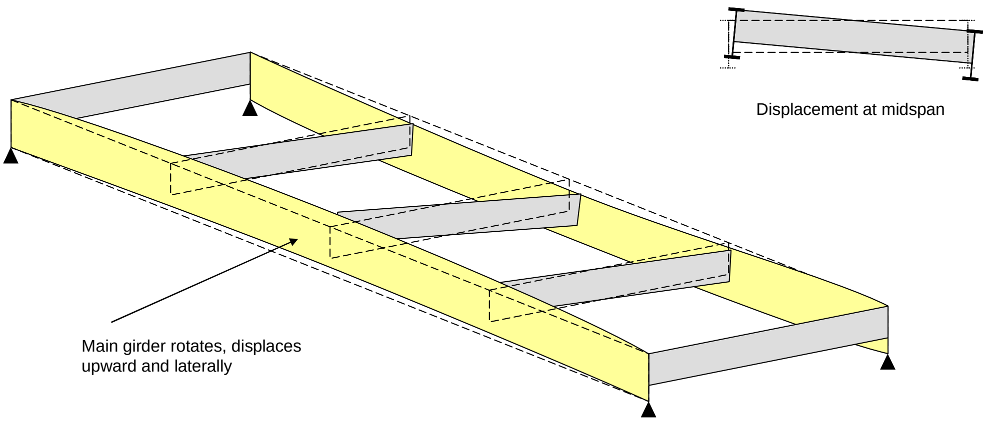


Figure 7.2 *Buckling mode with torsional restraints*

Buckling curves for rolled section beams

As explained in Section 6.1.6, a different set of buckling curves may be used for rolled I sections, and for welded sections that are equivalent to rolled sections (taken to mean bisymmetric sections of the same flange and web sizes). During construction, the non-dimensional slenderness of bare steel beams may well be of the order of 1.0 and in such cases the reduction factor according to the curves in EN 1993-1-1, 6.3.2.3 is much higher than that according to the curves in 6.3.2.2.

7.2.3 Loading from cantilevers

Two methods of cantilever construction are currently favoured - the use of proprietary systems that clip onto the girder face (and which can be released as a unit once the concrete has cured) and the use of precast units (installed after the central deck slab has been cast).

The moment due to weight of cantilevered falsework and wet concrete is transferred to the main girder as a couple of horizontal forces at top and bottom flange levels; these forces cause horizontal bending of the flanges between restraint positions. This is in effect warping torsion, rather than St Venant torsion. In ladder deck bridges this warping is modest, because the cross girders provide restraint at close regular intervals; in multi girder decks the restraint positions are further apart and the effects are greater. Deflection at the restraint positions (due to bending of cross girders or the vertical displacement of the main girders due to the eccentric moment) adds twisting effects. Warping stresses, distortional displacements and twists all need to be determined. Although the warping stresses (transverse bending stresses) in the top flanges are locked in once the concrete hardens, it is not necessary to include these effects for the in-service condition because at ULS they will redistribute and at SLS any relaxation would be unlikely to lead to any noticeable permanent deformation.

The alternative method of constructing cantilevers is to add full thickness precast units once the central portion of the deck slab has been completed. Support for these units can be from overhead temporary frames on the deck and although the weight causes twist (because of differential deflection of the main girders), there are no warping effects in the main girders.

7.2.4 Patch loading on webs

For girders that are erected by launching, reactions under the girder as it is progressively launched impose local 'patch loading' on the unstiffened portions of the main girder webs. The web will need to be checked for the effects of combined stresses and for buckling.

The design resistance to local buckling is given by EN 1993-1-5, 6.2 and the interaction between transverse force, bending moment and axial force should be verified according to EN 1993-1-5, 7.2; this is in effect an 'equivalent stress' check.

For longer loaded lengths, where local buckling due to the transverse force is not significant, the combined transverse and longitudinal buckling effects need to be checked. This can be verified using the 'reduced stress method' of EN 1993-1-5, 10; it is not then necessary to use 7.2.

7.3 Design of cross girders

7.3.1 Cross sectional resistances

The cross sectional resistance of the bare steel section is readily determined, although, like the main girders, the cross girders may well be Class 4 in sagging bending.

7.3.2 Buckling resistances

At the wet concrete stage, there is no lateral restraint to the top flange and its width needs to be adequate to ensure stability. With long cross girders there may be an advantage in bracing together adjacent pairs of cross girders, using a channel section with stiff (moment resisting) connections to the cross girders. The cross girders must then be verified as beams with a central torsional restraint (see references to torsional restraint in Section 7.2.2 and Appendix C.4).

7.4 Design of cantilever edge beams

Cantilever edge beams are usually the last part of the deck slab to be concreted, in order to achieve a good alignment along this very visible feature. Their contribution to structural behaviour of the cantilevers cannot therefore be relied upon until a late stage during construction. In many cases the edge beams are discontinuous and therefore only provide local stiffening, with no contribution to global bending.

7.5 Allowances for permanent deformations

The deflections under unfactored dead and superimposed loads should be calculated to enable the girders to be pre-cambered. This information should be produced by the designer and a breakdown of the effects of the various actions included on the drawings. Where staged construction has been presumed, the sequence should be stated on the drawings.

A residual hogging profile is often specified, for aesthetic reasons, even when not needed to meet a clearance requirement at SLS.

For the calculation of deflections of composite sections, it is necessary to assume an age at first loading, so that the appropriate parameters for concrete can be determined. The steelwork should normally be pre-cambered to offset the predicted deflection at the end of construction.

8 DETAILED DESIGN: COMPONENTS AND CONNECTIONS

8.1 Geometric configuration

The designer should clearly and unambiguously define the geometric configuration of all the main structural elements of the bridge. The following comments relate to certain specific issues of good practice for multi-girder and ladder deck bridges.

8.1.1 Multi-girder bridges

Road camber and crossfall

Usually the top flanges of the main girders are square to the vertical webs; the relationship with a deck slab that follows the road camber or a transverse crossfall then needs to be considered carefully. There are four main options:

1. Keep the slab soffit level and the thickness uniform; the crossfall is achieved by varying the thickness of the surfacing.
2. Keep the slab soffit level and vary the slab thickness, so that the top surface follows the required crossfall
3. Slope the slab soffit between the edges of the girder flanges; the top surface follows the required crossfall and the slab thickness varies across the width between girders
4. Provide small haunches above the girder flanges and use a uniform thickness slab, following the crossfall.

The first and second options are only appropriate for 2- or 3-lane bridges with no superelevation, where the weight penalty is modest. Variation of surfacing thickness is preferred to variation of slab thickness, for economy in construction. The first three options all suit the use of permanent formwork; option 3 is perhaps the most common and is the arrangement shown in Figure 2.1. Options 1 and 2 are illustrated in Figure 8.1.

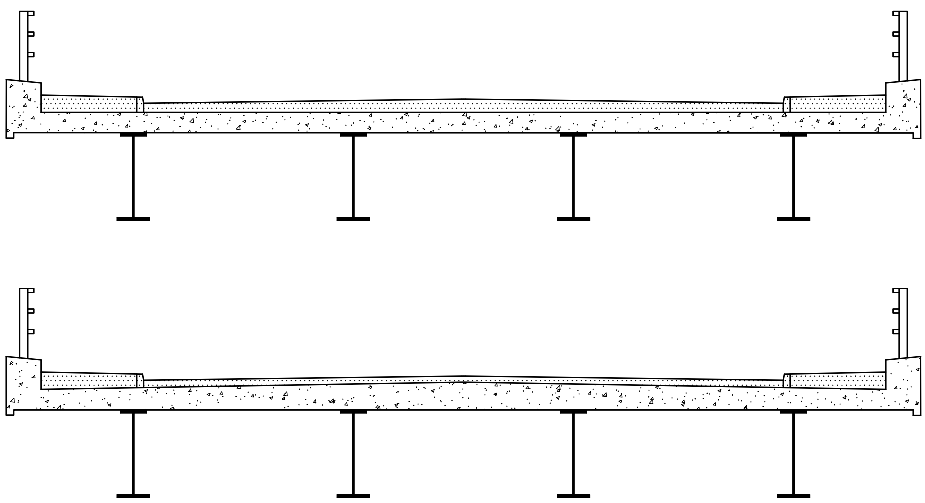


Figure 8.1 *Two options for dealing with road camber in single carriageway multi-girder bridges*

Option 4 is suited to the use of timber temporary formwork and, until the increased use of permanent formwork, was the most common arrangement. A typical arrangement is shown in Figure 8.2. The haunches can be formed relatively easily and the resulting slab is of uniform thickness. The use of haunches also makes it easier to accommodate any unintended differences in relative level between adjacent girders that are found after steelwork erection.

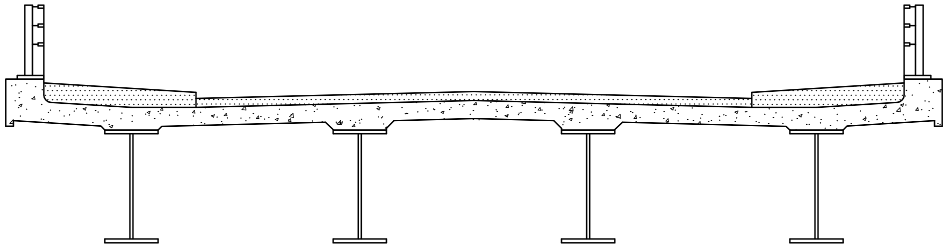


Figure 8.2 *Use of haunches with a multi-girder slab constructed on temporary formwork*

For all four options, designers usually choose the same girder depth for all the girders. For options 3 and 4, the girders are at slightly different levels.

Where the deck is wider, or where there is superelevation (uniform gradient across the full width of the carriageway), the arrangements are then similar to those illustrated in Figure 8.3. See further comment on the effect of a varying slab thickness in Section 8.7.

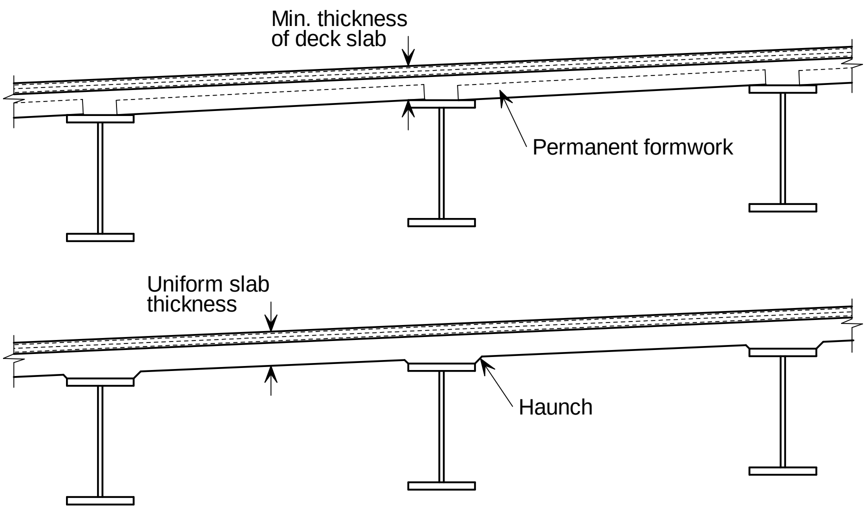


Figure 8.3 *Two options for dealing with deck slab with superelevation in multi-girder bridges*

Bracing planes

Planes of bracing are usually square to the top flange, rather than vertical. As noted in Section 2.4.2, the planes are usually square to the main girders in plan.

8.1.2 Ladder decks

Plan layout

The position or spacing of cross girders should be defined in relation to their centrelines (i.e. the mid-thickness of the web). This is particularly important where a lapped connection is used, to avoid confusion if the position were defined to one face of the web or to the centreline of the stiffener to which the cross girder is attached. It also helps to ensure that the fabricator and the supplier of the formwork are working to the same dimensions

On curved bridges, the configuration of the main girders can be arranged to follow the curvature of the roadway, to a uniform radius, to a spiral or to a mixture of straights and curves. Cross girders should be arranged radially to the defining curve (normally the centreline of the road).

Road camber, crossfall and longitudinal gradient

Transversely, the top flange of the cross girder will usually follow the camber of the roadway or the superelevation of the roadway. The alignment of the flange in relation to the flanges of the main girders needs to be considered: if the main girder flanges are horizontal (across the bridge), variations in slab thickness and the consequences of any variation in width of main girder flange need to be taken into account. For the usual crossfall (2.5%), or a modest crossfall to provide superelevation, the top flanges of the cross girders can be aligned as shown in Figure 8.4; the small step at the edge of the main girder flange does not introduce construction difficulties, although care will be needed in sealing between permanent formwork and the main girders.

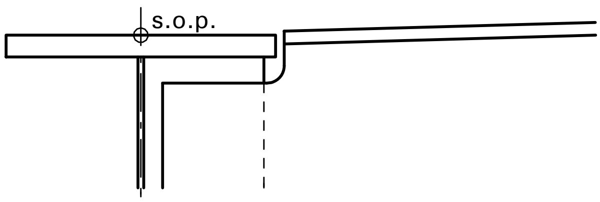


Figure 8.4 *Alignment of flanges*

Longitudinally, the top flanges of the cross girders should be aligned with the longitudinal profile of the main girders, which follow the longitudinal road profile; this will maintain uniform slab thickness along the bridge. To avoid complexity in fabrication and the need for a different cross sectional geometry of every cross girder, each cross girder should be detailed with parallel flanges square to its web and the cross girder should then be connected with the web square to the main girder top flange. This means that the cross girders and the main girder web stiffeners they connect to will, in general, not be truly vertical and their inclination will vary along the bridge. This does not cause difficulty for the fabricator.

There is no explicit requirement for bearing stiffeners at intermediate or end supports to be truly vertical, although designers usually prefer to detail them to be vertical under dead load (it is visually better for the bearing stiffeners on the outer faces to be vertical). Where there are integral crossheads, it is structurally better to make their webs vertical, to minimise the twisting effects from the bearing reaction. If a pier cross girder, diaphragm or integral crosshead is detailed with the web vertical, its top flange should still follow the longitudinal profile of the main girders and the flange will then be slightly tilted relative to the web, as shown in Figure 8.5.

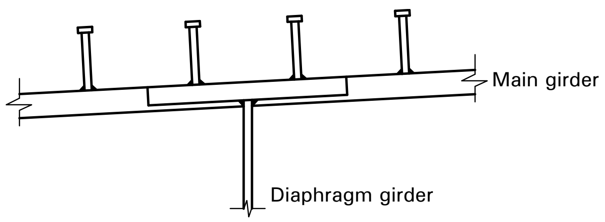


Figure 8.5 *Alignment of diaphragm girder flange when its web is vertical*

8.1.3 Allowances for permanent deformation

The steelwork is normally fabricated with allowances for permanent deformation due to the weight of the complete structure (see Section 7.5) and for cutting and welding during fabrication. See GN 4.03. The fabricator can deal with these allowances in determining the shape of all the elements that are to be cut from steel plate, provided that the designer advises the allowances for the intended construction sequence (see Section 7.5). However, there are still some questions that arise with a composite structure, such as at what stage are the webs at supports to be truly vertical: under the weight of steelwork alone or after completion? Although designers might wish to select the latter option, it is well known that it is very difficult to predict rotations of main girders (about their longitudinal axes) at supports, particularly for skew bridges. It is therefore commonly arranged that the webs are fabricated to be vertical under the weight of bare steelwork and the girders are designed for the out-of plumb that would result under the weight of concrete and superimposed load (assuming that rotations occur as predicted).

8.2 Bracing systems

8.2.1 Multi-girder bridges

As noted in Section 6.1.6, bracing is commonly provided at one or two positions either side of an intermediate support to restrain the compression flange. Bracing is also usually provided at one side of a site splice (to control the geometry) and at a few positions in midspan to provide torsional restraint during construction. If the bridge has to be designed for collision loading (see Section 5.4.5) bracing will be needed at regular and fairly close intervals, to carry the forces due to impact.

The bracing system needs to be designed to provide a sufficiently stiff and strong restraint to the main girders.

EN 1993-2, 6.3.4.2(6) states that a restraint to a compression flange may be taken as rigid if its stiffness is at least $4N_E/L$, where N_E is the critical load of the flange over a length L. Where beams are paired together, the stiffness needs to be doubled, to stabilise the two flanges. Stiffness can be evaluated using a simple plane frame model, for either of the usual configurations shown in Figure 2.4. Apply a unit force transversely at each bottom flange and ensure that the deflection does not exceed the following limit:

$$\delta \leq \frac{L}{4N_{\mathrm{E}}} = \frac{L^3}{4\pi^2 EI_{\mathrm{c}}}$$

The bracing system should be designed to resist lateral forces of 1% of the force in the compression flanges that are restrained by the bracing (see EN 1993-2, 6.3.4.2(5)). If the main girders are curved in plan (or a series of straights to a curve) the restraints must also resist the radial components of force. If the bracing is also to restrain the bottom flange in the event of a collision impact, the forces in the bracing due to the accidental action are likely to be large.

8.2.2 Ladder deck bridges

The inverted U-frames of ladder decks are a form of bracing but may not be sufficiently stiff to meet the criterion for rigid support given above. In such cases, their flexibility is taken into account in determining the slenderness of the main girders.

For verification of strength, the U-frame members and their connections need to carry the forces and moments associated with the differential bending of the frames under load (see Section 6.1.11) together with forces to restrain compression flanges laterally (see EN 1993-2, 6.3.4.2(5). For main girders curved in plan, the radial components of force need to be carried by the U-frames. If collision loading has to be resisted, knee bracing may be required.

8.2.3 Restraints at supports

Bracing or pier diaphragms at supports provide torsional restraint to the main girders and a load path for transferring lateral forces to restraint bearings.

The same requirement for stiffness can be applied as for ‘rigid’ intermediate restraints in multi-girder bridges (see Section 8.2.1).

The forces to be considered in the design of the restraint system include:

- Lateral forces due to traffic loading (taken as applied at the level of the road surface and resisted at the level of the restraint bearing).
- Lateral forces due to wind. Forces are again resisted at the level of the bearing.
- Forces due to non-verticality of main girder webs at the support.
- Forces due to imperfection in alignment of compression flanges of the main girders.
- Forces due to distortion introduced at skew supports.

Values for the last three of these may be determined as follows.

Forces due to non-verticality of webs

In general, the webs of girders will not be truly vertical at supports and the bearing reaction is therefore eccentric relative to the vertical forces in the girders. For a steel beam, the vertical force in a girder may be considered to be at mid-height of the webs; in a composite girder it may be considered conservatively to be at the top of the steel girder.

The acceptable non-verticality of webs due to fabrication and erection tolerances should be limited by the project specification (EN 1090-2 does not give tolerances for girders with bearing stiffeners); a typical limit would be 1/300. For design, a value of 1.5 times the specified tolerance should be used.

Additional out-of-verticality is introduced in bridges with skew supports, depending on the form of bracing and the rotations of the main girders (in the plane of the web) under load. Out-of-verticality displacements under traffic loading are given by member twists determined by global analysis; these effects are already factored and do not require the 1.5 multiplier mentioned above. For further guidance see GN 7.03. That Note comments that while it may be specified that the effects due to construction loads (notably the weight of the wet concrete) shall be offset by dimensioning the steelwork to ensure that the specified verticality is achieved on completion, designers often specify that the verticality requirement applies to the bare steelwork. They then allow for an element of out-of-vertical displacement due to the weight of the non-steel permanent loads.

Forces due to misalignment of main girder flanges

Main girder flanges will not be truly straight; the tolerance on straightness depends on EN 1090-2 and on the execution specification. For a composite bridge with intermediate restraints, the effect at an intermediate support may be considered as the design force in the flange multiplied by the change of angle from its direction either side; if an out of straightness of 1 in 500 is specified, the angle would be 1/125 and a multiplier of 1.5 should be applied, Strictly, a magnification for second order effects should be applied, since the restraint system is effectively a flexible lateral restraint to a compression strut, but the flexibility of triangulated bracing or pier diaphragms is usually sufficiently small in comparison to that needed to restrain the strut (consider the limiting value for a 'rigid' restraint in Section 8.2.1) that the magnification is insignificant.

Forces due to distortion of cross section

The out-of-verticality of main girder webs introduced at intermediate supports also induces lateral forces – the lateral displacement of the bottom flange (the top flanges do not displace because they are connected to the slab) results in plan bending of the flange over the length to the first lateral restraint. These forces are likely to be small in most cases but can be determined from a simple line beam model of the bottom flange.

8.3 Web stiffeners

8.3.1 Intermediate web stiffeners

Intermediate transverse web stiffeners are provided to enhance the shear resistance of slender webs and for the attachment of transverse bracing or cross girders. These stiffeners take the form of a simple flat plate welded to one face of the web. The outstand and a portion of the web plate on either side form an effective Tee section that has its centroid just outside the face of the web.

To be effective in enhancing the shear resistance of the web, the stiffener does not need to be connected to either flange, although it is usual to connect it to at least the top flange. Intermediate stiffeners are frequently not connected to the bottom flange. Where the stiffener is not connected, the clearance from its end to the flange should not exceed about three times the web thickness.

Where the stiffener acts as a connection for transverse bracing or cross girders, it should be connected to both flanges. Failure to provide such attachment may lead to fatigue cracking in the web at the point of curtailment of the stiffener.

Where bracing is attached to a web stiffener, the stiffener may need to be shaped to provide sufficient lap to connect the bracing members. See further advice in GN 2.05.

The fillet welding of the end of a stiffener to a flange does not introduce a lower class of fatigue detail on the flange than is likely to be present already. However, the attachment should be detailed such that the toe of the weld is at least 10 mm from the edge of the flange (a weld that terminates at the edge is more susceptible to fatigue, though such a detail does not have a separate classification in EN 1993-1-9). Web stiffeners should be proportioned such that they are narrower than the flange outstand; wide stiffeners (for the attachment of bracing) may need to be notched at the end to ensure that the welds are not too close to the edge of the flange.

Intermediate stiffeners should normally all be attached to the same face of each girder web. On the outermost girders, the stiffeners should be on the hidden face, rather than the exposed face, for better appearance. This location also suits the attachment of bracing between girder pairs.

Where a sloping flange changes direction in elevation (at the end of a tapered haunch), a transverse stiffener is required to carry the transverse component of force. Such stiffeners should be provided on both faces of the web but may only need to extend over part of the height of the web.

Design rules for transverse web stiffeners are given in EN 1993-1-5, 9. Generally, the requirements are:

- To provide an effectively rigid restraint out of the plane of the web, so as to develop the shear resistance of the web panels.
- To sustain such axial forces and moments as arise from the functions of restraint to the shear panels and interaction with bracing systems or cross girders, plus forces and moments due to local vertical loading on the deck or from change of inclination of the bottom flange.

Restraint of the web against buckling due to direct (compressive) stresses does not need to be considered, provided that the web class is 3 or better or, if Class 4, that the effective breadth of web in compression does not take account of the presence of any transverse stiffeners.

The effective cross section of the stiffener is taken as the stiffener itself plus a width of plate of $15\varepsilon t$ on either side (if available) - see EN 1993-1-5, 9.1(2).

The requirements for stiffness are given in EN 1993-1-5, 9.3.3. Additionally, to ensure that the stiffener section is not susceptible to torsional buckling, the criterion of 9.2.1(8) must be met. For a flat stiffener this criterion effectively limits the outstand ratio $h_s/t_s \leq 13\varepsilon$. (for S355, the limit is about 10.5).

An axial force due to the restraint of the shear panel needs to be considered when the design shear force exceeds a certain proportion of the elastic critical shear resistance. The force is given by a Note to 9.3.3(3) but the Note is modified by the NA. This axial force arises in the plane of the web, so it imposes both axial force and moment on the stiffener section.

Forces due to interaction with bracing systems or cross girders are given by either a comprehensive global analysis or a local plane frame model of the bracing system. Note that where the forces are due to differential U-frame bending in ladder deck bridges, the lateral bending imposed on the main girder flanges needs to be amplified for second order effects. The amplification is given in EN 1993-2, 5.2.2(5); for this situation the amplification factor may be taken as $1/(1 - N_{Ed}/N_{cr})$, where N_{Ed} and N_{cr} relate to the flange plus part of the web.

A wheel load from one of the wheels of the TS or LM3 vehicles directly above the stiffener should also be considered; the load disperses through the surfacing and deck slab and a proportion of it acts at the top of the stiffener. The presence of the stiff plane of bracing and the attachment of the web stiffener to the top flange also attracts moment from the slab, and this must be included in the verification.

The stiffener section is verified for strength according to the EN 1993-2, 6.3.3, which gives a simplified interaction criterion. For ordinary intermediate stiffeners the buckling length is the distance between flanges but where the stiffener is connected to channel bracing or a cross girder a shorter length would be appropriate.

Where there is axial load (other than that due to the restraint of the shear panel), the connection to the flange should be checked, especially where the stiffener is notched to clear the flange edge or has a cope hole to clear a weld or the radiused fillet of a rolled section.

8.3.2 Bearing stiffeners at supports

At supports, heavier transverse web stiffeners are provided to facilitate the transfer of the shear forces from the web to bearings below the girders. Flat plates are normally used and stiffeners are provided on both faces; the stiffeners are usually symmetrical about the web, which ensures that the centroid of the effective stiffener section is concentric with the web. In smaller bridges, a single flat on either face may be sufficient but it is common to use pairs of flats, as illustrated in Figure 8.6. As for intermediate stiffeners, a width of $15\varepsilon t$ on either side of a flat may be assumed to act as part of the effective section; where there are two flats on each face a single effective section encompassing both may be used if the spacing between them does not exceed $30\varepsilon t$. When pairs of flats are used, ensure that there is sufficient access for welding and inspection (see GN 2.04).

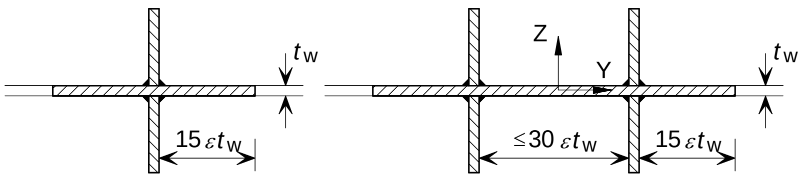


Figure 8.6 *Typical bearing stiffener arrangements and effective sections*

Bearing stiffeners should be welded to both flanges. At the bottom, a smaller weld may be used if the ends of the stiffeners are fitted to the flange to achieve full contact bearing (see further comment below).

The stiffness requirements of EN 1993-1-5, 9.3.3 are easily satisfied for bearing stiffeners and the principal requirement is the verification of the adequacy under combined axial force and biaxial bending.

The axial load on a bearing stiffener varies from the value of the reaction at the bottom to nearly zero at the top (only the load from local wheel loading need be considered). The moments (about both axes) depend on the eccentricity of the bearing reaction. No guidance is given in EN 1993 on what values of eccentricity should be considered. For restrained bearings, a nominal eccentricity should always be assumed; a value of 10 mm is suggested. For movement bearings, a similar allowance should be made where the sliding surface is the lower part of the bearing but where the sliding surface is the upper part, movement due to temperature variation needs to be determined (thermal action is then 'accompanying' the leading action and the value of ψ_0 = 0.6 should be used - see 5.4.9), in addition to a nominal allowance of 10 mm.

For bearing stiffeners subject to biaxial bending, the simplified interaction criterion of EN 1993-2, 6.3.3 needs to be modified to include bending in the plane of the web. Expression (6.9) then becomes:

$$\frac{N_{\mathrm{Ed}}}{\dfrac{\chi_{\mathrm{y}} N_{\mathrm{Rk}}}{\gamma_{\mathrm{M1}}}} + C_{\mathrm{mi,o}} \frac{M_{\mathrm{y,Ed}}}{\dfrac{M_{\mathrm{y,Rk}}}{\gamma_{\mathrm{M1}}}} + \frac{M_{\mathrm{z,Ed}}}{\dfrac{M_{\mathrm{z,Rk}}}{\gamma_{\mathrm{M1}}}} \leq 0.9$$

(The term for the shift of neutral axis in a Class 4 section has been omitted.)

Elastic values of bending resistance should be used. Note that this interaction criterion is conservative for cruciform sections because there is no common 'extreme fibre' for bending about the two axes.

The factor for moment gradient $C_{\mathrm{mi,o}}$ given by EN 1993-1-1, A.2 will be close to 0.7 for the near-triangular bending moment diagram. It would be reasonable to take $M_{\mathrm{z,Ed}}$ as the maximum value within the middle third of the stiffener.

Additionally, the cross sectional resistance should be checked at bottom flange level, deducting any notches or cope holes and including only the area within a 45° dispersal zone from the contact area of the bearing. If the bottom of the stiffener is fitted to achieve full contact bearing, then the reaction may be assumed to be transferred in bearing over this area at ULS. However, the reaction due to fatigue loads should not be assumed to be transferred in bearing; all the fatigue design force should be assumed to pass through the welds.

So-called 'jacking stiffeners' are often provided close to bearings to enable bearings to be replaced by jacking the structure off the permanent bearings. Such stiffeners should generally be designed as bearing stiffeners, although the design loading may be less, if it is acceptable to restrict traffic during the replacement procedure.

8.4 Joints, connections and splices

EN 1993-1-8 distinguishes between a joint (a "zone where two or more members are interconnected") and a connection (a "location at which two or more elements meet"). It does not define a splice in words but the implication (in Figure 1.2 therein) is that a beam splice is a type of joint. The use of the word joint does not always seem to be consistent and the clauses should always be read carefully to ascertain the scope and intent of the individual rule.

Connections are made using either bolts, rivets (very rarely) or pins, referred to generically as fasteners, or by welding.

8.4.1 Bolting

As noted in Section 6.4.1, EN 1993-1-8 defines three categories of bolted connection transferring shear (A, B and C). EN 1993-2 effectively rules out Category A for bridges: a recommendation to use preloaded bolts in Catgory B or C connections is given in EN 1993-2, 2.1.3.3(4) , although fitted bolts and rivets are alternatives (as is welding).

In most connections, it is acceptable to allow the bolts to slip at the higher ULS loads; the bolts then act in bearing and shear (this is usually significantly greater than the ULS slip resistance). This is a Category B connection. In some cases, it is desirable to prevent slip at ULS as well as at SLS; this is a Category C connection

The bolts that can be used in Category B and C connections are grade 8.8 and grade 10.9 bolts in accordance with a range of reference standards listed in EN 1993-1-8. In the UK, 'system HR' preloaded bolts, grade 8.8, are normally used; this should be stated, along with other requirements for bolts, in the project specification. System HRC grabe 10.9 preloaded bolts (to EN 14399-10) may also be used; these are more commonly known by the proprietary name TCB.

The design resistance of bolted connections is based on the resistances for individual bolts. For slip-resistant connections, the resistance is given by EN 1993-1-8, 3.9. For bolts in bearing and shear, the resistances are given by EN 1993-1-8, 3.6.

Bolts should be positioned in accordance with the limits in EN 1993-1-8, 3.5. Note that where a minimum spacing or edge distance is used the bearing resistance of a fastener is reduced. Where weathering steel is used, it is common practice to determine minimum spacing and edge distance values on the basis of 1 inch size bolts but to design the connection for M24 bolts because bolts are likely to be supplied from the US.

Preloaded bolts are usually used in normal clearance holes but the designer may wish to allow the use of oversize holes in some locations, for constructional reasons. If oversize holes are to be accepted, this needs to be recognised in design, since the design resistances are reduced and the minimum hole spacing is increased (because it is based on hole diameter).

8.4.2 Welding

Welded connections are made using either fillet welds or butt welds. The design resistances of fillet and butt welds are given by EN 1993-1-8, 4.5, 4.6 and 4.7. Reference is made to the quality level that is required to be consistent with the design rules. If weathering steel is used, it must be remembered that the 'corrosion allowance' applies to welds as well as to plates and sections.

Because weld details are a potential source of local defect, the choice of weld detail has a significant effect on fatigue performance. EN 1993-1-9 classifies weld details according to their effect on fatigue endurance (see Section 6.6.3). The inspection and testing of welds is an important aspect of quality control in fabrication; see GN 6.01, GN 6.02 and GN 6.03[2] for further advice.

8.4.3 Splices in main girders

For all but short single spans, each main girder is fabricated in a number of pieces and joined together on site, either prior to or during erection. The lengths of the pieces are usually chosen to suit economical fabrication and transport restrictions, and splice positions are usually arranged to be away from positions of maximum moment. The splice may then be designed to transmit the most onerous design force and moment at that position, which is likely to be significantly less than the full design resistance of the girder. Splices may be bolted or welded.

Bolted splices

At a bolted splice, cover plates are normally provided on both faces of each flange and web. The number of bolts required may be determined either on the basis of slip resistance at ULS (Category C) or, more economically, on the basis of no slip at SLS and bearing/shear resistance at ULS (Category B). When a connection is assumed to act in bearing/shear at ULS, the requirement for no slip at SLS usually governs (the chief exception being for thin material when large bolts are used).

In a bolted splice, a key design task is to determine the distribution of forces between the individual bolts. EN 1993-2 prescribes that "if a moment is applied to a joint, the distribution of internal forces should be linearly proportional to the distance from the centre of rotation". Unfortunately, this requirement appears rather unclear when applied to a whole member cross section in a beam splice. However, if the traditional approach of calculating stresses elastically in the beam cross section and considering the flanges and web elements separately is used, the requirements would appear to be met.

For each flange, the number of bolts in the connection can be determined on the basis of the flange force, calculated on the basis of an elastic stress distribution in the girder section. For a Category B connection at ULS, the resistance of the bolt group is the sum of the resistances of all the fasteners, which in most cases is the sum of the shear resistances. However, if the cover plates are thin, the resistance may be governed by bearing (rather than shear on the bolt). Note that end bolts and edge bolts may have a slightly lower bearing resistance than inner bolts, depending on edge spacing.

For the web, the force on the web plate connection, calculated from the elastic stress distribution, is a combination of moment and axial force (the centroidal axis of the section is not usually at mid-depth in the web). Additionally, shear is transferred across the connection; this imposes both a shear force and an additional moment (shear force times eccentricity) on the bolt group. The force in each bolt is the vector sum of a number of forces:

- the vertical force due to sharing the shear force equally between all the bolts
- the horizontal force due to axial force on the web (again shared equally)
- the force due to moment on the web (each bolt force is directly proportional to the distance from the centre of the bolt group and acts tangentially to that radius).

The force on the outermost bolt determines the design of the group. For Category B connections at ULS the bearing resistance may govern, rather than shear resistance, because webs and their cover plates are usually thin.

In the flange and web cover plates, stresses should be checked where the plate is in tension or where the holes are oversize or slotted, allowing for holes for fasteners in determining net sections (see EN 1993-1-1, 6.2.3, 6.2.4 and 6,2,5).

On the upper top flange cover plate it may be necessary to provide shear studs, to comply with maximum longitudinal spacing limitations for shear studs. Only a single row of studs should be provided, if possible, to avoid complications in tightening the bolts.

For further advice on the design of bolted connections, see Guidance Note 2.06.

A typical bolted splice is shown in Figure 8.7.

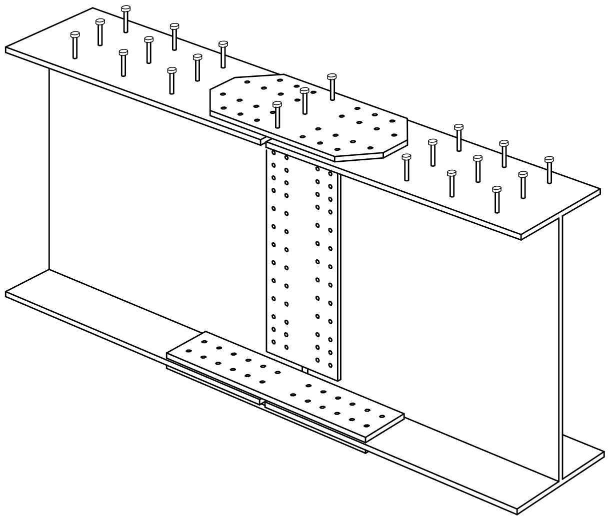


Figure 8.7 *Typical bolted splice*

Welded splices

Welded splices in girders usually involve full penetration butt welds in webs and flanges. Although double-sided partial penetration butt welds in thick flanges would be permitted according to EN 1993-2, the fatigue classification of such welds is very poor. Full penetration welds should always be used; this makes the splice capable of the same bending resistance as the girder section. The weld in the web is often staggered relative to the bottom flange, to assist in locating the web during erection (the web can sit on the projecting length of flange). A semi-circular cope hole is usually provided in the web above the flange weld, to facilitate the butt welding of the flange. Where the splice is not staggered, the cope hole is usually filled after the splice has been welded, to avoid the stress concentration (around the open hole) at the end of the butt weld in the web. Details are shown in Figure 8.8. Where the cope hole does not have the termination of a butt weld it may be left open but the fatigue verification should include the application of a stress concentration factor when checking the stress at the exposed edge of the web.

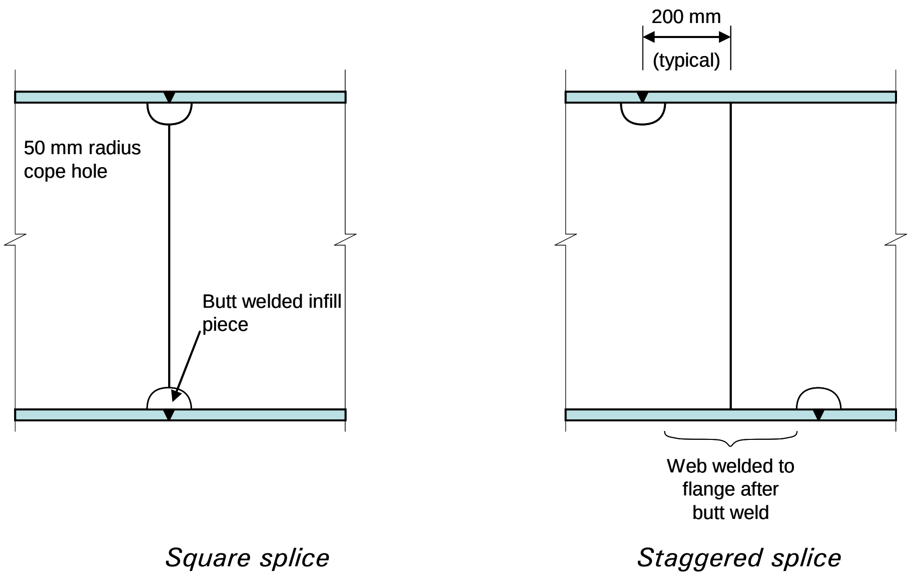


Figure 8.8 *Arrangement at welded splice*

Long joints

Where a bolted or welded joint made with cover plates or by lapping is long, the transfer of force is not uniform along the length of the joint and the resistance of the fasteners or welds should be reduced - see EN 1993-1-9, 3.8 or 4.11. For bolted joints, the reduction applies where the length is more than 15d; for welded joints the reduction applies where the length is more than 150a (d is the bolt diameter, a is the weld throat).

8.5 Cross girder end connections

8.5.1 Intermediate cross girders

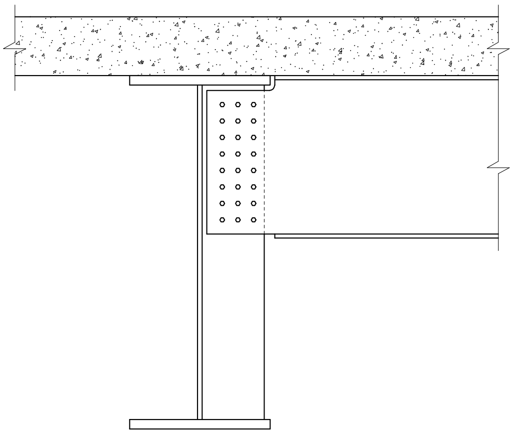


Figure 8.9 *Lapped connection of intermediate cross girder*

Intermediate cross girders (i.e. away from supports) are connected to web stiffeners by simple lapping of the web plate onto the stiffener, as shown in Figure 8.9. Both flanges are stopped short of the end of the web; this is easily achieved with a fabricated girder. Double cover plate splice connections are inappropriate because the same number of bolts is required as with a single lapped connection, and additional cover plates have to be fabricated, making the detail more expensive. However, if rolled sections are chosen for the cross girders, a double cover detail may be preferable (allowing the section to be cut with a plane end) because of the cost of the work that would otherwise be needed in cutting back of flanges of a rolled section.

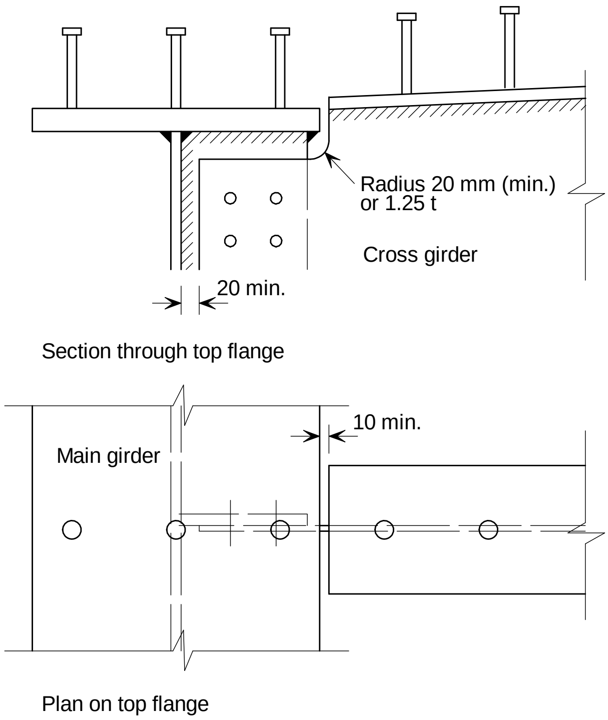


Figure 8.10 *Details of lapped connection*

In most cases, this simple lap detail, with no connection of the flanges, will be sufficient to transmit the moments in the U-frame (see *Design basis*, below). If the moments are greater than can be transmitted this way (perhaps because a very shallow cross girder is chosen) a connection of the bottom flange similar to that sometimes used for pier diaphragms (see Figure 8.14) may be effective.

With a lap detail, the cross girders are longer (over the length of the web) than the clear gap between the main girder flanges. Consequently, during erection they are lifted at a skew (in plan) so that they can be lowered past the top flanges and then rotated and brought into lapping contact with the main girder stiffeners. See Figure 8.11. Ideally, plan rotation as shown should be possible with both the adjacent cross girders in position. It may be necessary to keep the end of the cross girder web sufficiently clear of the face of the main girder web (and this may require a wider web stiffener) so that the cross girder can be erected in this manner even when the cross girders on both sides are already in place. The laps at the two ends of the cross girder should be to the same face of the web, to avoid the risks of confusion and error in setting out and installation.

Special attention should be given where there are jacking stiffeners between intermediate cross girders, or small bays between cross girders. The cross girders could be trapped between stiffeners, preventing the rotation illustrated above. The detailing may need to be adjusted in these areas, depending on the layout and the construction sequence.

Lapped connections are less accommodating of deviations of the cross girder length (and of the layout of the bolt holes) than a spliced connection. With modern fabrication techniques this should not be a problem, although some designers have allowed in the design for oversized holes (the slip resistance is reduced) in case reaming should prove necessary.

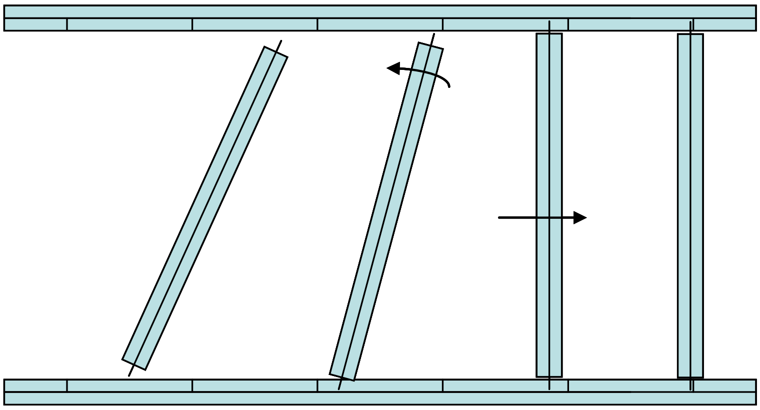


Figure 8.11 *Schematic arrangement for erecting intermediate cross girders*

Design basis

During construction, there will be very little end moment on the cross girder due to gravity loads (i.e. there is very little end fixity from the main girder) and the single lap connection is assumed to transmit the vertical shear and a sagging moment equal to the shear multiplied by the distance from the web to the centroid of the bolt group. There should be no slip at ULS; the connection should be designed as Category C.

Once the slab has been cast, the connection will act compositely with the deck slab. The connection will need to transfer the vertical shear (acting on the line of the web) and moments about the main girder axes due to U-frame action as a result of differential loading on adjacent cross girders and the restraint provided to the compression flanges of the main girders in the hogging moment regions.

Hogging moments from the deck slab cantilever reduce the sagging moment to be transmitted at the centroid of the bolt group. Such moments 'disperse' from moment carried by the slab alone to moment carried by the composite section over the length of the first few stud connectors and can be assumed to act on the composite section at the position of the bolt group. However, unless the governing design case for the bolt group is with net hogging moment, the cantilever slab moments can be neglected.

The bolted connections provide restraint to the main girder against LTB in regions adjacent to the intermediate support and should therefore be designed for no slip at ULS (Category C). In midspan regions, slip could be tolerated at ULS but it is better to use the same connection detail for all intermediate cross girders.

The forces on the bolts in the composite condition are determined by considering a Tee section comprising a width of slab plus the web of the cross girder. There are no effective width rules for this design situation but it is suggested that a width of slab equal to the width of the main girder flange will suffice for the connection design. The horizontal force and moment on the web should be determined from the stress distribution in this Tee section and the bolt group then designed for the combination of shear, horizontal force and moment. The force on the bolt in the bottom row will govern.

Strictly, the forces in the bolts due to bending moments should be derived by considering the three stages of bare steel, long-term composite Tee and short-term composite Tee, and adding the results from all three stages. Forces due to the moment in the bare steel condition would be pro rata to the distance of the bolt from the centre of the bolt group. However, it would be adequate to verify that the connection does not slip under the total moment; the short term composite section may be used for such a verification.

8.5.2 Pier diaphragms

Lap type connections cannot readily be made at intermediate supports as, once the main girders are in place at the required spacing, the pier diaphragm girder will foul on the jacking stiffeners as it is swung into place. Hence, cross girders framing into the bearing stiffeners will normally be connected using double cover plates to the web. See Figure 8.12 and Figure 8.13.

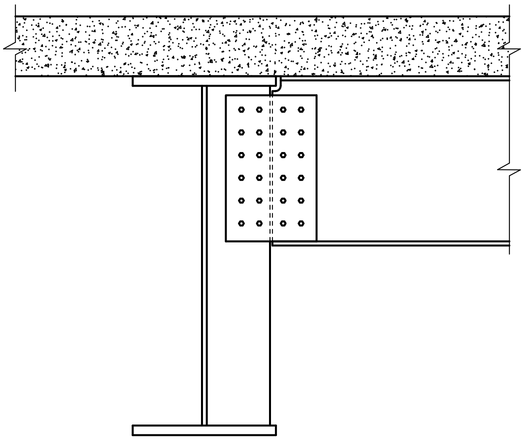
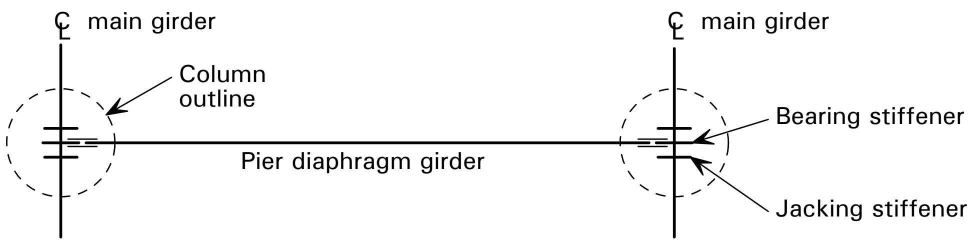


Figure 8.12 *Spliced connection of pier diaphragm*



(Flanges omitted for clarity)

Figure 8.13 *Arrangement of double lap splice connection of pier diaphragm*

To transfer larger moments at supports, a cover plate connection to a 'stub flange' attached to the bearing stiffener may be needed (see Figure 8.14). A double cover plate connection should be provided. Connection of the top flange should not be necessary. The cross girder should be less deep than the main girder, to avoid any need to connect to the main girder bottom flange.

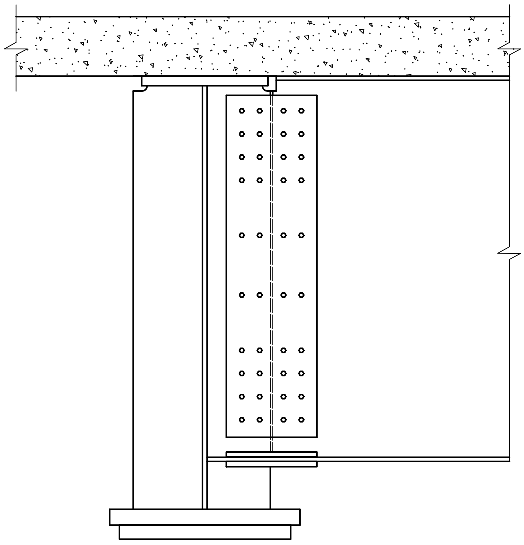


Figure 8.14 *Elevation on splice connection of pier diaphragm with stub flange*

Design basis

A similar design basis to that for the intermediate cross girders is adopted, though it could be argued that the crosshead connection may be designed to slip into bearing and shear at ULS as the flexibility that this creates has little effect on the LTB of the main girder (a more flexible end support would need to be assumed in evaluating the buckling resistance but the effect is small).

In this splice configuration the bolt group to be considered is the one on the crosshead side of the splice; it carries greater moment due to its eccentricity from the main girder.

Clearly, the use of a stub flange, with bolts in the cover plates at maximum distance from the slab, is more effective in transmitting moment: the stub flange then has to be designed to transfer the forces into the stiffener

8.5.3 Integral cross heads

Where the bridge is supported on bearings under an integral crosshead, the load on the connection is clearly much greater and more bolts will be required. A typical connection arrangement is shown in Figure 2.12 and some of the details are shown at larger scale in Figure 8.15.

If longitudinal restraint is provided at an integral crosshead, a suitable load path (in terms of both strength and stiffness) must be provided for horizontal restraint forces at a longitudinally restrained bearing. This should normally be provided by designing for plan bending of the bottom flange of the crosshead.

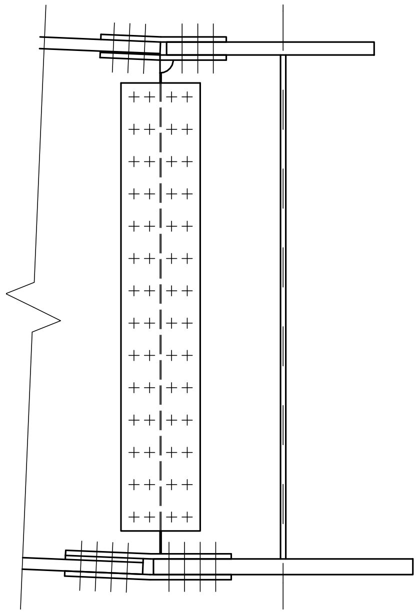


Figure 8.15 *Connection of integral crosshead to main girder*

8.5.4 Crosshead girders in multi-girder bridges

Where crosshead girders are used (such as shown in Figure 2.5) bolted connections will usually be needed. Alternatively, the crosshead and lengths of the two main girders which it supports can be fabricated as a single H section (in plan). This can reduce the amount of site work making connections. The overall dimensions of the H section are limited by transport restrictions.

8.6 Shear connection

Shear connectors are usually 19 mm diameter, as this size is readily available and can easily be welded using a special semi-automatic welding tool. Stud dimensions and minimum spacing limits are given in EN 1994-2, 6.6.5.7. Because studs are required to prevent separation, the undersides of the heads of the studs need to be a minimum distance above the bottom layer of reinforcement. Requirements for haunched and unhaunched configurations are shown in Figure 8.16 (reproduced from EN 1994-2, 6.6.5.4 and 6.6.5.1).

The use of 125 mm or 150 mm long stud connectors will ensure that the heads are well above all bottom transverse reinforcement in most cases.

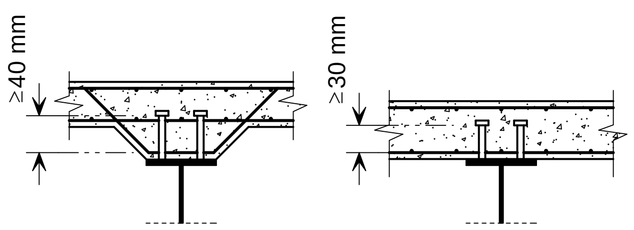


Figure 8.16 *Detailing for resistance to separation*

Stud spacing should not exceed 800 mm longitudinally (which leads to the need for studs on cover plates on the top flange, as shown in Figure 8.7).

Studs should not be closer than 25 mm to the edge of a flange (50 mm if the slab is haunched); larger edge distances are needed when precast permanent formwork is used, to ensure secure seating of units.

8.7 Deck slab

The detailing issues related to deck slabs concern chiefly the location of reinforcement.

Multi-girder decks

In multi-girder decks, the transverse reinforcement is normally placed as the outer layers and the longitudinal reinforcement is placed as the inner layers.

Where precast permanent formwork is used between the main girders, only the in-situ lower transverse rebars are effective in transferring shear to the main girders; the adequacy of these bars needs to be verified. The location of the upper longitudinal rebars in the inner layer suits fixing of reinforcement, as they can sit on the top of the protruding hoops of the precast units (although see note below about varying slab thickness). Note that the fixing of the top rebars on top of the hoops constitutes "reinforcement affected by construction process" and thus is not eligible for the reduction by 1 class in exposure class in Table 4.3 of EN 1992-1; the value of c_{nom} will thus be 5 mm greater than for a fully in-situ slab.

Where permanent formwork is not horizontal, the thickness of the slab varies, as noted in Figure 8.3. The arrangement of the transverse reinforcement at a main girder is illustrated in Figure 8.17. It is not usual to crank the transverse bars in either the top or bottom mat, although they will bend a little. In the top, the bars may lift off slightly from the plank reinforcement on the 'higher' side and the cover may be slightly greater. The bottom bars may also be slightly higher at the same location.

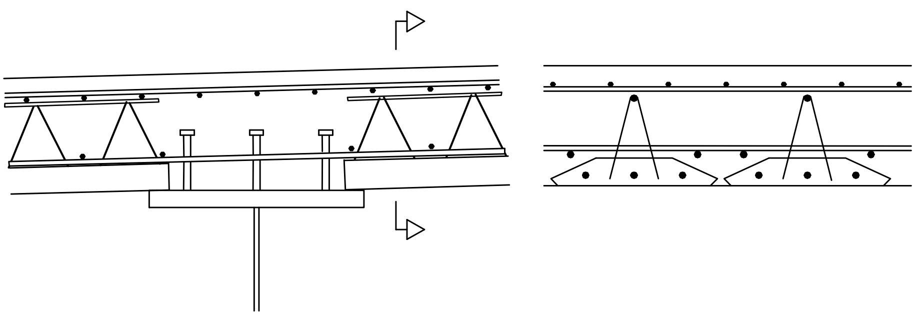


Figure 8.17 *Local cross section at main girder in multi-girder deck with precast permanent formwork*

Permanent formwork needs to be constrained by the positions of the studs at each end so that it cannot displace and fall through, between the girders, during construction. The nominal bearing length for precast planks is typically 55 mm and studs should be no more than 25 mm inside the nominal position of the plank ends; this will ensure that the planks cannot be displaced along their length and then fall through. Similar provisions should be made for reinforced fibre panels.

Any protective treatment to the steelwork should be continued inward from the top edges of the flanges; where permanent formwork is used it should extend beyond the end of the formwork. A sealant will also be required where permanent formwork is sloped, relative to the flange and in all cases where the girder is of weathering steel.

Ladder decks

The transverse reinforcement is usually the inner layers in the slab and the higher position of the bottom rebars (higher than the outer layer) needs to be recognised when considering their position relative to the underside of the shear stud heads.

Where precast permanent formwork is used, the transverse rebars in the main body of the slab are even higher. To achieve the necessary clearance below the head of the studs bars need to be cranked, or additional U-bars provided. See Figure 8.18 and Figure 8.19. Alternatively, taller studs can be used.

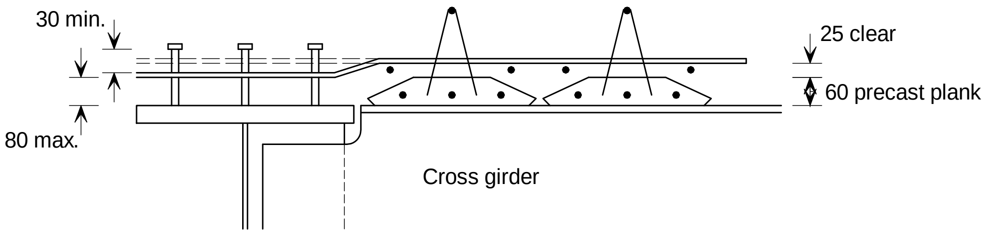


Figure 8.18 *Cranking of transverse reinforcement*

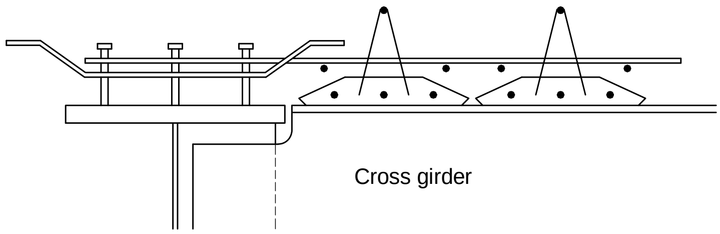


Figure 8.19 *Alternative use of U-bars*

Consideration should also be given to ease of fixing the top mat of the slab reinforcement. Usually the transverse bars would be detailed in the top layer as this gives maximum lever arm for the tension reinforcement at the root of the deck cantilever. However for steel fixing it is easier to place the transverse bars as the lower layer, directly onto the precast plank lattice (layer T2 in Figure 8.20), and then place the longitudinal bars in the top layer (i.e. layer T1 in Figure 8.20).

As with multi-girder decks, the positions of the studs should be such that the precast units cannot fall through during construction.

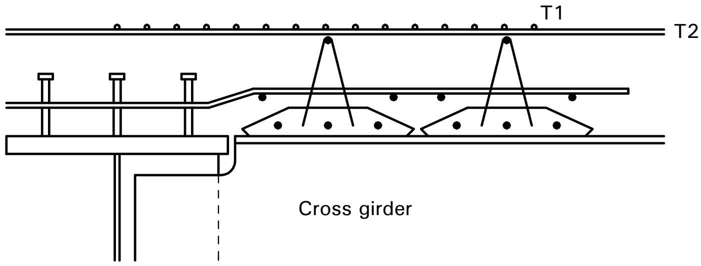


Figure 8.20 *Positioning of top reinforcement in slab*

As for multi-girder decks, protective treatment should be returned along the top surface of the top flanges and sealants used where appropriate.

8.8 Bearing specification

Bearing design and construction are covered by EN 1337. EN 1337 is not part of the Eurocodes but does include design rules for steel elements. Because it has been developed separately and before the final version of EN 1993-1-1 was published, there is some inconsistency between the design rules.

Bearing design is usually the responsibility of the bearing manufacturer; the bridge designer should provide a specification for each bearing, listing the range of reaction forces and movements (translational and rotational). Annex A of EN 1993-2 gives guidance on the preparation of technical specifications for bearings. This includes a template for a bearing schedule; this schedule calls for reactions and displacements due to characteristic values of the separate actions. EN 1337-1 provides a different template for a bearing schedule, this schedule calls for design values due to combined actions. The EN 1337 schedule is more useful to the bearing designer, since he does not need to know the factors to apply in each combination of action. Further advice in completing the bearing schedule is given in PD 6703[15].

The most commonly used type of bearing for highway bridges is the pot bearing; a disk of elastomer confined in a short cylinder, onto which the reaction is transferred by a 'piston'. This type of bearing accommodates moderate rotations in any direction but is relatively stiff vertically. If a sliding surface is provided within the bearing, translational movement can be accommodated; freedom can be provided in any direction, or guides may be provided to confine movement to one direction. Without a sliding surface, full translational restraint is provided.

Displacements due to permanent actions in heavily skewed decks may include large rotations (about each main girder axis) at both intermediate and end supports, and thus large transverse displacements at the bottom flange level. This is particularly true for ladder deck bridges. These effects are a function of the plan geometry of the deck and are related to the magnitude of the dead load precamber required; they cannot be avoided. Due allowance for these rotations should be made in the design of the bearings (include the rotations in the bearing schedule) and the detailing of tapered plates to which the bearings are attached.

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[15] It is expected that a more comprehensive schedule will be included in a revised issue of EN 1337-1, due to be published in 2010.

The increased flexibility of the cross-section in decks with shallow cross-girders can lead to significant splaying of the main girders under both permanent and variable actions, causing relatively large transverse translations in the bearings. The use of deep cross-girders at intermediate supports and concrete diaphragm beams at end supports will greatly reduce these movements.

As well as specifying the bearing, the designer should consider the requirements for attaching the bearing, both to the steelwork and to the substructure. The attachment, usually with a taper plate between the flange soffit and the bearing, should be designed for both the vertical and horizontal forces involved (see GN 2.08). The alignment of the bearings and the identification of the direction of the principal movement should be made clear on the drawings (see GN 2.09).

8.9 Expansion joints

Expansion joints are also items that are usually designed and supplied by specialist manufacturers. The bridge designer is required to provide a specification, giving the displacements and actions on the joint. Guidance on preparation of a technical specification is given in Annex B of EN 1993-2, although no template is given; the general format of the template for bearings may be adapted but, as for bearings, it is usually more helpful to give the manufacturer design values than to give effects due to each characteristic action.

9 DETAILED DESIGN: INTEGRAL ABUTMENTS

9.1 Fully integral framed abutments

In this form of construction, the endscreen walls are supported on columns or piles that are fully moment-connected to the wall, thus forming a portal frame in elevation. See Figure 2.21.

A key consideration is likely to be the provision of sufficient flexibility in the pile/column to accommodate the thermal movement range. With RC columns, an upper limit to the column size is typically about 750 mm diameter or square.

9.1.1 The use of sleeves around piles

The simplest arrangement for the columns beneath the end screen wall is for them to be in contact with the soil, either as a result of driving piles through existing ground, or as a result of backfilling around them after construction. However, the tops of the columns will be displaced laterally by the expansion and contraction of the bridge; this creates a tendency for 'post holes', with a small gap, while at the same time increasing soil resistance (at maximum displacement) because of the repeated straining of the soil.

To avoid this situation, it is common practice in the UK to provide a stack of manhole rings or a length of plastic pipe around each column, creating a clear annular space and leaving the soil undisturbed by any displacement of the columns. These rings or pipes are commonly referred to as 'sleeves' or 'isolation tubes'. Examples of this form of construction are shown in Figure 9.1 and Figure 9.2.

(Photo by courtesy of Mott MacDonald)

Figure 9.1 *RC columns in manhole rings for framed abutment*

(Photo by courtesy of Mott MacDonald)

Figure 9.2 *Use of plastic pipe sleeves around H-piles*

The question of confidence about the durability of the columns within the annulus then arises and one answer has been to provide an access duct into the top of the open space to permit future inspection (using a boroscope or similar means of viewing remotely).

Where sleeves are provided, they should not be filled; otherwise there is no benefit in providing them. Also, sleeves must be disconnected from the endscreen wall at the top; otherwise they will be dragged sideways by any movement. Careful detailing is needed to ensure that the top of the annulus is sealed against soil ingress (see Section 9.1.5).

Avoidance of the use of sleeves would eliminate that element of the construction cost but if they are eliminated, the effects of the pressures on the piles need to be considered.

9.1.2 Modelling

Framed abutments need to be modelled with full moment continuity at the connection between the deck and the supporting structure. The endscreen walls are subject to soil pressure from the retained fill and the pressures are related to displacements. If the columns are in contact with the soil, they too will be subject to earth pressures related to displacements, although the forces involved should be relatively small.

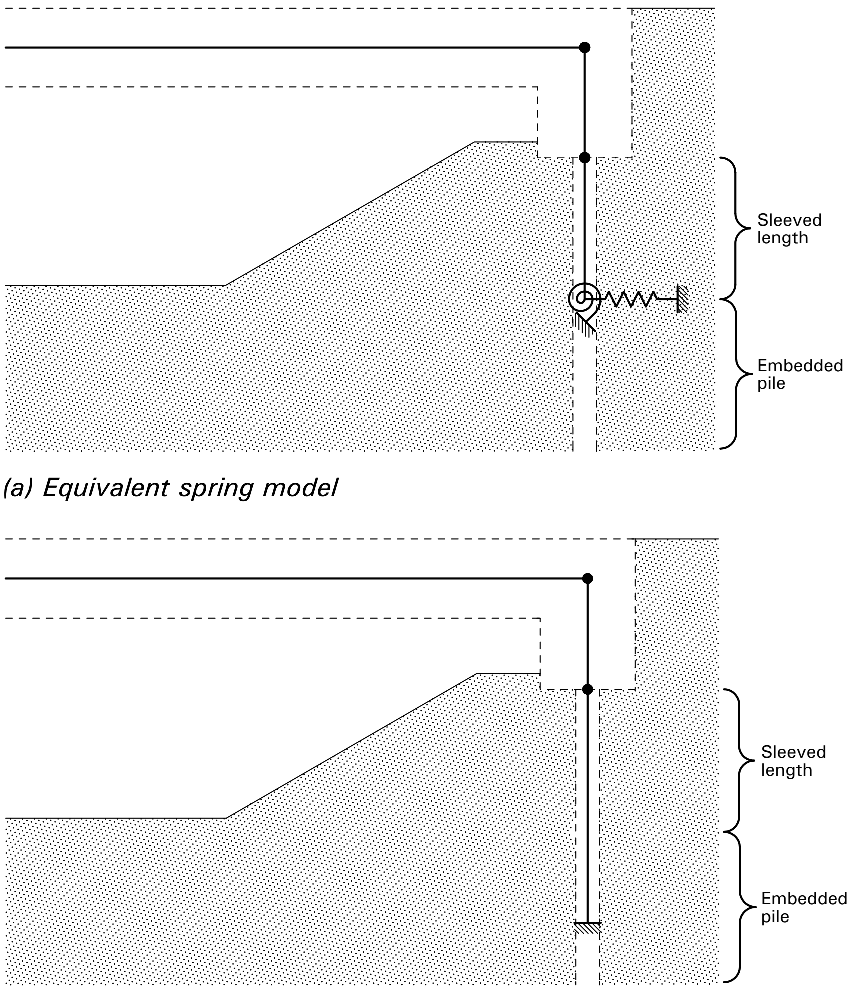
The placing of columns inside manhole rings removes any soil pressures on them and this simplifies the modelling of the whole bridge. Whilst it is then possible to derive lateral and rotation spring restraints for use with a conventional grillage model, the interaction between rotations and displacements would need to be resolved through an iterative process and some designers consider it more expeditious to use a 3D model.

In a 3D model, the columns can either be modelled realistically, using equivalent springs for the soil behaviour, or an 'equivalent cantilever' can be derived. The two options are shown diagrammatically in Figure 9.3. In the derivation of an equivalent cantilever, four properties (area, second moment of area, shear area and length) need to be derived from the actual lengths and soil stiffnesses; these can be derived by consideration of the flexibility matrix.

Fabric stiffening

Recent research has discovered a previously unrecognised soil behaviour, sometimes referred to as fabric stiffening. Under repeated cycles of loading there is a tendency for preferential load paths to develop along lines of soil particles that are in direct contact. This provides an additional stiffness within the soil, but only in the direction of the cyclic loading. This stiffening occurs without significant compaction of the soil, unlike the more familiar soil stiffening that is a consequence of the compaction process. In an integral bridge, the fabric stiffening may develop over several decades.

A design approach that reflects this behaviour is given in PD 6694-1[36]. The document provides a cyclic envelope of soil pressure coefficients that can be applied over the full range of thermal movement. The maximum coefficient depends on the characteristic value of the thermal movement range, reflecting a history of cycling over this range.



(a) Equivalent spring model

(b) Equivalent cantilever model

Figure 9.3 *Modelling support restraints in 3D model*

9.1.3 Column design

The requirements for the columns are for minimum bending stiffness whilst supporting the vertical reactions from the deck. The columns therefore need to be of the smallest feasible cross section; the column is quite short and is rotationally restrained at its ends, so buckling slenderness is unlikely to be a problem but the slenderness and buckling resistance of lengths in sleeves do need to be checked.

Concrete columns or piles

Where the columns are within manhole rings, heavily reinforced square and circular concrete sections have been used. The size tends to be dictated by the minimum cross section that can be arranged with the maximum practical level of reinforcement .

Steel piles

Greatest flexibility in bending, for a given axial resistance, is offered by the use of H-section piles or plunge columns with orientation such that the web is orthogonal to the direction of bridge deck movement. The question of durability of the steel then arises. If the steel is in contact with the ground, a corrosion allowance in accordance with BD 42/00 should be made – a greater allowance must be made where there is groundwater. If the steel is in a space inside manholes rings or pipes, similar allowances should be adequate.

Plunge columns

Steel H piles can be used as ‘plunge columns’ inserted into cast in-situ bored concrete piles. This has the following advantages over the use of driven H piles:

- The pile length is known in advance
- The pile head level can be set accurately
- There is no damage to the pile during driving
- Shear connectors can be welded to the pile heads before delivery to site.

9.1.4 Endscreen wall and the connection to columns and deck girders

The endscreen wall serves the functions of transferring vertical load, horizontal load and moment between the deck and the columns. It is therefore subject to bending in two directions and to torsion. To perform these functions, substantial reinforcement is usually required.

To transfer moment from the deck girders to the endscreen wall, shear connection may be required on both flanges of each beam. On the bottom flange, because of space limitations, bar connectors may be needed, rather than the usual stud connectors. Bar connectors are much more expensive than stud connectors and should only be used where strictly necessary. Additional shear transfer can be provided by threading some of the reinforcement through the webs of the beams. Holes for bars are usually made generously oversize (50 or 75 mm diameter) to accommodate tolerances in bar positions and may be slotted to allow for the effects of skew. A typical detail of the end of a main girder is shown in Figure 9.4. When putting connectors on the beam web, they should be kept clear of the bolted connections for the bracing, to ensure that there is access for the wrench.

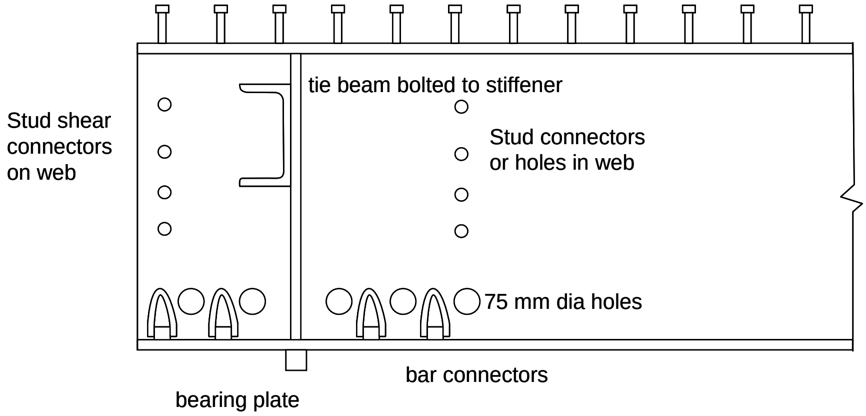


Figure 9.4 *Typical detail at the end of a main girder*

Transfer of load between an endscreen wall and an RC pile may be achieved using a normal pilecap detail. Load transfer to an H pile may require shear connectors to be welded to the top length of the H pile. If the pile is driven (rather than being a plunge column) these shear connectors will need to be welded after driving. Note that shear studs cannot be more than 19 mm diameter, because there is difficulty welding 22 mm studs onto a vertical surface. An alternative detail, where plates with shop-welded studs are site welded to the pile, is shown in Figure 9.5.

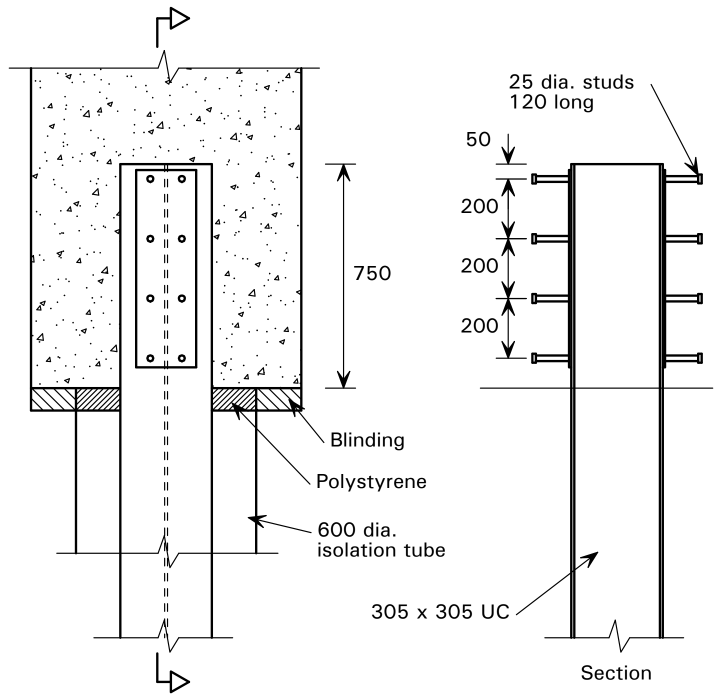


Figure 9.5 *Shear connection at the top of an H pile*

9.1.5 Interface of manhole rings with endscreen wall

The detailing of the interface requires consideration of two aspects – provision for movement and provision for inspection.

A particular issue that needs to be addressed in the detailing and construction is to ensure that the pilecap/endscreen wall is free to move without dragging the top of the stack of manhole rings or pipe with it. This needs to be considered in conjunction with sealing the top of the annulus against the ingress of debris, soil and the wet concrete when the pilecap is cast. A typical detail is shown in Figure 9.6.

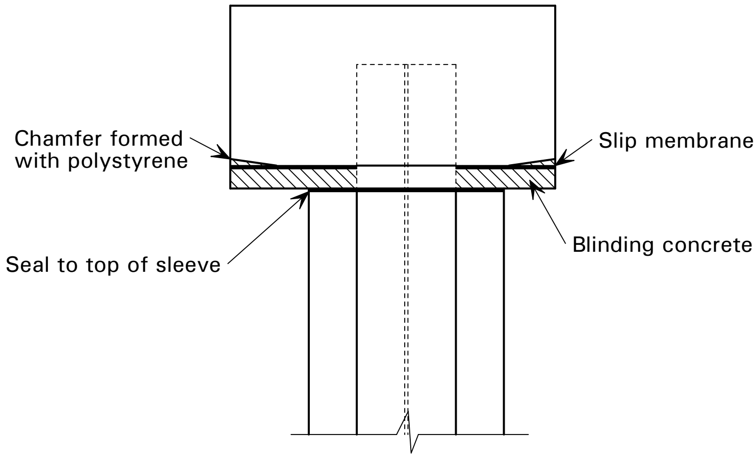


Figure 9.6 *Interface at the top of a manhole sleeve*

As mentioned in Section 9.1.1, inspection access is often provided to the annular space between manhole rings and the columns that they surround. This can be achieved using relatively small diameter UPVC piping, from the top of the annulus to a convenient location at the front of the endscreen wall. The exact detail will depend on the facility for access to the wall (i.e. where a safe position for inspection can be provided), the detailing of the cap to the sleeve and the provisions for relative movement at the interface.

9.1.6 Construction issues

As mentioned earlier, the end screen wall is usually built in two principal lifts, a lower section around the tops of the columns (or as a pilecap) and an upper section around the deck girders. To aid lining and levelling of the main girders, a bearing plate is set into the lower part of the endscreen wall; a very simple line rocker can be provided on the underside of the girders to sit on this bearing plate. One method of achieving precise levels on the bearing plates is to cast them into a plinth as a second stage. Typical details are shown in Figure 9.7.

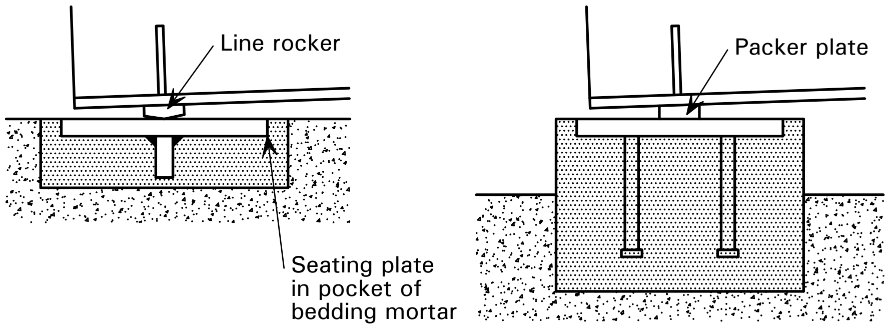


Figure 9.7 *Temporary bearing details*

It is relatively easy to provide positional restraint to the main girders in the temporary condition, longitudinally and transversely, by clamping or wedging against vertical reinforcement projecting from the lower part of the wall.

Torsional restraint to the main girders can be provided by channels or triangulated bracing between pairs of girders; the reinforcement in the upper part of the endscreen wall must be detailed accordingly, recognising the sequence of construction in this region. Alternatively, triangulated bracing in front of the endscreen wall can be used for temporary torsional restraint and it may be left in place.

Examples of a reinforced endscreen wall where torsional restraint is provided are shown in Figure 9.8.

Where bar connectors are used, there is no need to thread reinforcement through the hoops.

(Photos by courtesy of Atkins)

Figure 9.8 *Reinforcement in endscreen wall*

9.2 Fully integral bankseat abutments

In this form of construction, the endscreen wall is cast around the longitudinal girders and sits directly on the soil beneath. See Figure 2.23. The form is equally appropriate to multi-girder and ladder deck construction.

Because this form of construction introduces longitudinal forces onto the soil, it is normally used only where there are side slopes in front of the abutment; it is not suitable with reinforced earth retaining wall abutments unless the overall bridge length is short (and thus the movement is small).

9.2.1 Design issues

Bearing on the soil

According to PD 6694-1, the bearing resistance under these bankseat footings is only 50% that for the same soil when there is no rocking or sliding. Consequently, the bearing area has to be quite large. The reduced bearing resistance can be quite limiting, even on good soils such as chalk. One way to improve the situation without increasing the size of the wall is to introduce a layer of granular fill, thus permitting greater dispersal before bearing on the soil itself.

Where ladder deck construction is employed, the bearing pressure might vary along the length of the endscreen wall. The vertical and horizontal bending of the wall needs to be considered, although the wall is usually able to sustain that bending easily.

Wall/main girder connection

Since there are no significant restraining moments at the abutments, the principal additional forces on the deck girders are the axial forces due to restraint of expansion. Generally, for these types of bridge, the forces should be fairly modest and rarely govern the design of the deck.

The ends of the main girders are detailed similarly to those of frame abutment bridges, except that, since the moments transferred are less, the requirements for shear connection are lower.

Drainage behind the wall

Drainage behind the wall is typically provided by 225 mm thick porous (or hollow) blockwork, above a perforated UPVC pipe. This is the detail indicated in Figure 2.23.

9.2.2 Construction issues

As for the frame abutment, a construction joint is usually formed just below the level of the soffit of the main girders. A temporary bearing, as shown in Figure 9.7, is then provided.

9.3 Semi-integral abutments

In this form of construction, the foundation that supports the vertical loads from the main girders is separate from the endscreen wall. The main girders sit upon movement bearings (such as sliding pot bearings) on that foundation. There is a provision for relative movement between the foundation and the endscreen wall. See Figure 2.24.

An example of a support arrangement is shown in Figure 9.9. This shows a sliding bearing on a plinth, the face of the endscreen wall and the movement joint at the base of the endscreen wall.

Overbridge on BNRR (Photo by SCI)

Figure 9.9 *Example of a semi-integral bridge support arrangement*

Although this form of abutment has been used on many bridges, it does have the disadvantage that the bearings will probably need to be replaced during the life of the bridge and this will require jacking to release the bearings. The required movements for replacement are small, provided that the bearing attachment has been appropriately detailed, and the forces required should be well within practical capability but concerns have been expressed about the forces involved and the movement at the interface with the soil, hence this form of abutment is a less favoured solution. This type of abutment may not be acceptable to some client authorities.

9.3.2 Design issues

Bearing on the soil

Since there is no imposed displacement on the footing, normal soil bearing pressures can be used.

Endscreen wall and connection to main girder

The endscreen wall is subject principally to bending in plan due to the soil pressures on the back face; with a ladder deck configuration this may require substantial reinforcement in what otherwise would only need to be a fairly thin wall. Shear connection between a main girder and the endscreen wall can be provided by means of an endplate to the girder, with shear stud connectors, as shown in Figure 9.10.

(Photo by SCI)

Figure 9.10 *End plate of main girder, for connection to endscreen wall*

Movement joint

Provision for movement between the endscreen wall and the footing can either be a sealant between two horizontal surfaces or between the face of the wall and a small step in front of it, as indicated in Figure 2.24. The latter is probably easier to maintain but the former is easier to construct.

Drainage

A similar arrangement for drainage to that for the bank pad abutment is typically used and that is shown in Figure 2.24. However, the blockwork will have to tolerate the relative movement between wall and footing, which takes place at the bottom of the endscreen wall.

9.3.3 Construction issues

Since this type of bridge is usually supported on sliding bearings at both ends, a temporary form of longitudinal restraint will be needed, unless the bridge is restrained longitudinally at an intermediate support. Torsional and lateral restraint will also be required, as previously mentioned for the other types of integral bridge.

9.3.4 Bearing replacement

It is normal practice to require highway bridges to be designed for the replacement of bearings. For this type of highway bridge, jacking the girders to free the bearings will also mean lifting the endscreen walls, with the potential consequence that the asphaltic plug joint will be damaged and need to be replaced.

The vertical movement needed to free bearings can be small (of the order of 5-10 mm), provided that the fixing details permit removal of fixing bolts without any vertical movement. The issues to consider are thus only the jacking loads and how the soil and road formation respond to the vertical movement.

If the amount of movement needed is greater than can be tolerated by the plug joint, the joint would have to be remade after an operation to replace bearings.

9.4 Dealing with skew

Departmental Standard BD 57/01[7] requires integral construction to be considered "for bridges with lengths not exceeding 60 m and skews not exceeding 30°". In practice, the limit on skew has mostly been observed and semi-integral bridges have generally been built with smaller skew angles.

The displacement of a bridge with a skew abutment, where the rear face of the wall is not square to the bridge axis, introduces two effects:

(f) There is a lateral component of force on the bridge deck, due to the earth pressure behind the wall. This tends to cause plan rotation of the deck.

(g) Because an abutment wall is fully supported vertically, it tends to rotate about an axis along its length, not about an axis square to the main girders.

9.4.1 Plan rotation

The tendency to cause plan rotation of the deck is shown in Figure 9.11.

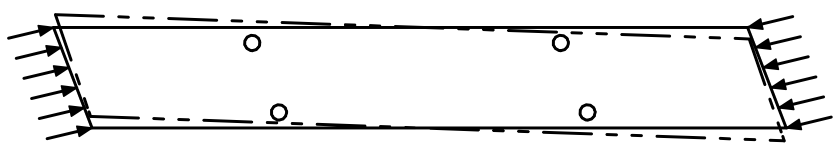


Figure 9.11 *Soil pressures tending to cause plan rotation of the bridge deck*

With a framed endscreen abutment, the lateral flexibility of the columns (or piles) accommodates some of the tendency toward transverse displacement. Skew angles up to about 30° do not lead to excessive displacements of the piles. With a bank pad abutment there is similarly some transverse flexibility through sliding and skews up to about 30° can be accommodated.

With semi-integral bridges, guided bearings are needed at the abutments. The guides are normally aligned along the bridge axis and thus provide restraint to lateral movement. To avoid creating excessive lateral forces on the foundation (or to avoid slip under the foundation), it is better to limit the skew angle to about 15-20°.

9.4.2 Torsion

The rotation of a skew end support about a line skew to the bridge axis conflicts with the rotation of the main girders in the planes of the webs. This leads to twist of the girders and torsional moments in the wall.

9.5 Continuity with deck slab

Where the bridge has integral abutments it is usual for designers to specify that the top part of the endscreen wall, which creates the moment continuity between deck and supporting structure, is concreted after the deck slab. The camber calculations must reflect that any subsequent superimposed load will be applied on the integral structure, rather than on one with simple end supports.

9.6 Corrosion protection

For all types of integral and semi-integral bridge described above, all the bridge girders, irrespective of whether they are painted or of weathering steel, should be provided with drip deflectors so that water that has run along the flanges cannot run down the face of the abutment wall, causing unsightly staining. This is particularly true for the fully integral bridges, because the wall is likely to be clearly visible. For semi-integral bridges, the run-off can fall onto the bearing plinth and, with suitable drainage channels, can be led to a drainage system.

For painted girders, the coating should extend 25 mm into the concrete for any embedded steelwork.

For weathering steel, BD 7/01 Clause 4.13 requires that a sealant be provided around the profile of weathering steel girders where it is cast into the concrete end wall of an integral bridge.

9.7 Pavement at the ends of the bridge

9.7.1 Joints between abutments and pavements

Asphaltic plug joints are recommended by the Highways Agency for the joints between the abutments and pavements and these are almost universally used in these locations. The characteristic value of thermal strain given by the Eurocodes for a composite bridge is typically ± 0.0006 and thus the frequent value is ± 0.00036. This strain means a change of ± 36 mm for a 100 m long bridge deck, or ± 18 mm at each end (assuming equal longitudinal stiffness at the two abutments). The maximum total acceptable movement range for plug joints, as given by BD 33/94[29], is 40 mm, which thus creates an upper limit to the length of bridge deck for which plug joints can be used of just over 100m. In practice, bridges up to about 100 m overall length have been built and the joints are believed to be performing satisfactorily.

9.7.2 Approach slabs

Previous guidance from the Highways Agency does not address the use of approach slabs with integral abutments and the majority of integral bridges are being built without them. However, some client authorities do have a preference for the use of approach slabs and they have been provided on some integral bridges.

9.7.3 Wing walls

Wing walls are usually connected to the endscreen wall and are aligned as a continuation of the bridge deck parapet line (i.e. in the direction of bridge deck movement) and kept as small as practicable.

10 REFERENCES

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    EN 14399-4:2005 Part 4. System HV. Hexagon bolt and nut assemblies
    EN 14399-5:2005 Part 5. Plain washers
    EN 14399-6:2005 Part 6. Plain chamfered washers
    EN 14399-7:2007 Part 7. System HR. Countersunk head bolt and nut assemblies
    EN 14399-8:2007 Part 8. System HV. Hexagon fit bolt and nut assemblies
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APPENDIX A List of Eurocode Parts

The following is a list of all the Eurocode Parts that might be needed for the design of composite highway bridges. Some Parts would not be needed in a normal design situation but have been included for completeness.

All the Parts are designated by CEN with an EN prefix; the documents are published by BSI, unchanged apart from the addition of cover pages, a short National Foreword, and with a BS EN prefix. Each Part is accompanied by a UK National Annex; these National Annexes have been published.

Eurocode Parts

EN 1990:2002, Eurocode: Basis of structural design (incl. amendment A1:2005)

EN 1991, Eurocode 1: Actions on structures

- EN 1991–1-1:2002, General actions. Densities, self-weight, imposed loads for buildings
- EN 1991–1-3:2003, General actions. Snow loads
- EN 1991–1-4:2005, General actions. Wind actions
- EN 1991–1-5:2003, General actions. Thermal actions
- EN 1991–1-6:2005, General actions. Actions during execution
- EN 1991-1-7:2006, General actions. Accidental actions
- EN 1991-2:2003, Traffic loads on bridges

EN 1992, Eurocode 2: Design of concrete structures.

- EN 1992-1-1:2004, General rules and rules for buildings
- EN 1992-2:2005, Concrete bridges. Design and detailing rules

EN 1993, Eurocode 3: Design of steel structures

- EN 1993-1-1:2005, General rules and rules for buildings
- EN 1993-1-5:2006, Plated structural elements
- EN 1993-1-8:2005, Design of joints
- EN 1993-1-9:2005, Fatigue
- EN 1993-1-10:2005, Material toughness and through-thickness properties
- EN 1993-1-11:2006, Design of structures with tension components
- EN 1993-1-12:2007, Additional rules for the extension of EN 1993 up to steel grades S700
- EN 1993-2:2006, Steel bridges

EN 1994, Eurocode 4: Design of composite steel and concrete structures.

- EN 1994-2:2005, General rules and rules for bridges

EN 1997, Eurocode 7: Geotechnical design.

- EN 1997-1:2004, General rules
- EN 1997-2:2007, Ground investigation and testing

APPENDIX B Initial sizing of main girders

The following very basic guidance is offered to give a rough first estimate of girder size for a multi-girder bridge using plate girders. It is intended only for use where the basic configuration has already been selected and does not give any indication of the bracing needed to stabilize the girders during construction or in the final condition. It allows the process of analysis, verification and refinement to commence; it is not intended for determining quantities for cost estimation.

Reference to loads should be taken to be the design value, i.e. that after the appropriate partial factors on actions have been applied.

At an intermediate support, the maximum shear will occur when the shortest LM3 vehicle is positioned in the lane directly over a girder, with the vehicle on the longer span side. The adjacent lane should have the TS axles in a similar position. The shear can then be estimated as the sum of components from the LM3 vehicle, the TS axles and the LM1 UDL over the span (on the longer side). For the first two components, use factors representing the proportion of load carried at the end support of a simply supported span, both longitudinally and transversely (e.g. for a LM3 vehicle 60% for its position along the span and 90% for a lane that is almost directly above the girder, giving a shear of 54% of the weight of the LM3 vehicle carried by the girder). For the third, use 70% of the load in the span times the proportion for lane position.

The maximum moment at an intermediate support occurs with a different distribution of load but the worst values can be assumed to coexist. Position the LM3 vehicle straddling the support and determine the moment at the end of the longer span (with half the LM3 vehicle on it) assuming it to be a fixed ended span; multiply by a lane position factor, as above. For the TS load, position it at ¼ way into the longer span and determine the fixed end moment; multiply by the lane factor. For the LM1 UDL, determine the fixed end moment in the longer span and multiply by the lane position factor.

Size the web adjacent to the support so that it can carry 150% of the governing shear (the reserve is valuable in carrying dead load and contributing to bending resistance). If the bottom flange is inclined, it will carry some of the shear and the web can be reduced in thickness accordingly.

Determine a force in the bottom flange adjacent to the support by dividing the total moment by the distance between the flange and the slab. Size the bottom flange so that the stress is 80% of the yield strength (the 20% allows for dead load and a small reduction for slenderness). Choose a top flange that has an area of 60% of the bottom flange.

In midspan, determine moments due to the same three components, with the LM3 vehicle and the TS positioned at midspan; consider the span as simply supported. Consider that 40% of the total moment is carried by a single girder (this makes some allowance for continuity and assumes transverse sharing between girders).

In midspan, provide a web that has an area of 60% of that at supports and size the bottom flange to carry a force of the total moment divided by the depth between the flange and the middle of the slab, at a stress level of 95% of the yield strength. Choose a top flange area that is 80% of that of the bottom flange (this will be needed for stability during construction).

Compression flanges should always be proportioned so that they are at least Class 3 (for S355 steel, limit the outstand/thickness ratio to 11.2). Tension flanges should be limited to an outstand/thickness ratio of 20, for robustness (but if they could go into compression during construction, comply with the Class 3 limit).

APPENDIX C Non-dimensional slenderness $\bar{\lambda}_{LT}$

C.1 General expression

A general expression that may be used to determine non-dimensional slenderness of beams $\bar{\lambda}_{LT}$, for use in EN 1993-2, 6.3.3.2 in determining the reduction factor χ_{LT} is:

$$\bar{\lambda}_{LT} = \frac{1}{\sqrt{C_1}} UVD \frac{\lambda_z}{\lambda_1} \sqrt{\beta_w}$$

where:

C_1 is a parameter dependent on the shape of the bending moment diagram over the half wavelength of buckling, such that the value of M_{cr} for with the actual bending moment diagram is equal to C_1 times that for the same beam and wavelength with a uniform bending moment.

U is a parameter dependent on the section geometry. Conservatively, U may be taken as 1.0 or, for doubly symmetric hot rolled sections, $U = 0.9$.

V is a parameter related to the slenderness and section geometry.

D is a parameter to allow for the destabilising effect of the applied loading. $D = 1.2$ if the load is applied to the top flange and both the flange and the load are free to move laterally. Otherwise $D = 1.0$.

$\lambda_z = \frac{kL_w}{i_z}$, in which:

L_w is the half wavelength of buckling

i_z is the radius of gyration of the beam about the minor axis

k is an effective length parameter

$\lambda_1 = \pi\sqrt{\frac{E}{f_y}}$ in which f_y is the yield strength appropriate to the thickness of the steel and E is the modulus of elasticity.

$\beta_w = \frac{W_y}{W_{pl,y}}$ in which:

W_y is the modulus used to calculate $M_{b,Rd}$

For Class 1 and 2 sections $W_y = W_{pl,y}$

For Class 3 sections $W_y = W_{el,y}$

For Class 4 sections $W_y = W_{eff,y}$

$W_{pl,y}$ is the plastic modulus of the cross section

C.2 Parameter C_1

There are a number of sources that give values of C_1.

Values for some standard bending moment diagrams, and a link to software for more general evaluation, are given in an NCCI document that is available on the internet. See document SN003 on the Access Steel website[16].

The value of $1/\sqrt{C_1}$ is equivalent to the parameter η in clause 9.7.2 of BS 5400-3:2000. In that Standard, the value of η is given by Figure 10.

Conservatively, C_1 may be taken as 1.0.

C.3 Parameters for beam segments between effective restraints

C.3.1 Parameter C_1

With several effective restraints in a span, the effect of moment variation over each segment of beam between restraint positions will have only a modest effect in most cases and the assumption that $C_1 = 1.0$ is not overly conservative.

C.3.2 Parameter V

For buckling over a half wavelength L_w, the parameter V is given by:

$$V = \left[\left\{ 4a(1-a) + 0.05\lambda_F^{\,2} + \psi_a^{\,2} \right\}^{0.5} + \psi_a \right]^{-0.5}$$, in which

$$\psi_a = 2a - 1 \quad \text{when} \quad I_{z,c} < I_{z,t} \quad \text{and}$$
$$\psi_a = 0.8(2a-1) \quad \text{when} \quad I_{z,c} \geq I_{z,t} \ .$$

$$a = \frac{I_{z,c}}{I_{z,c} + I_{z,t}}$$

$$\lambda_F = \frac{L_w}{i_z} \cdot \frac{t_f}{h}$$

h is the depth of the cross section;

t_f is the mean thickness of the two flanges of an I or channel section

$I_{z,c}$, $I_{z,t}$ are the second moments of area of the compression and tension flanges, respectively, about their z-z axes.

C.3.3 Parameter λ_z

The parameter λ_z depends on the effective length parameter k. For unrestrained buckling over the half wavelength, $k = 1$.

[16] Visit www.access-steel.com and search on: Elastic critical moment for lateral torsional buckling

C.4 Parameters for beams with intermediate torsional restraints

C.4.1 Half wavelength of buckling

For a beam that is only restrained torsionally between support positions, the buckling mode will be with one or more half wavelengths over the span. Usually the mode with a single half wave will have a lower buckling load but in some cases the mode with two half waves (or, exceptionally, three) would have a lower buckling load. The lowest mode should always be determined.

Thus $L_w = L$, $L/2$, $L/3$, depending on the mode, where L is the span of the beam. (In most cases, $L_w = L$.)

C.4.2 Parameter C_1

Since the value of $1/\sqrt{C_1}$ is usually significantly less than unity for a bending moment diagram that varies from hogging to sagging and back to hogging, it is usually worth making an estimate of its value in this situation.

The value of C_1 is that for variation of bending moment over the half wavelength; if two or three half waves in the span are considered, the variation is that over the half or third of the span length.

C.4.3 Parameter V

The parameter V for this form of buckling is calculated in the same way as given in Section C.3.2.

C.4.4 Parameter λ_z

For buckling with flexible restraints over the half wavelength of buckling, the effective length parameter k must be determined, rather than taken as unity. For this determination, several other parameters need to be evaluated, as given below.

C.4.5 Effective length parameter k

When a beam span between supports without lateral restraints is provided with a central torsional restraint or a number of equally spaced torsional restraints of the same stiffness in a span, the effective length parameter k may be derived from Figure C.1, using the restraint parameter $V_{eq}^4 L_w^3/[EI_{z,c}\theta_R d_f^2(1-a)]$.

For beams with multiple torsional restraints in the half wavelength, k should not be less than $(1.7 - 0.7V_{eq})L_r/L_w$.

where:

- d_f is the vertical distance between the centroid of the compression and tension flanges respectively at the position of the torsional restraint
- m is the number of restraints in the half wavelength of buckling (= 1 for a single central restraint within the buckling length)
- $I_{z,c}$ is the second moment of area of the compression flange about the minor axis
- L_r is the spacing of the torsional restraints ($= L_w/(1+m)$)
- L_w is the half wavelength of buckling (see C.4.1).

V_{eq} is a parameter that takes account of warping properties of the section and is calculated as follows:

a) for sections symmetrical about both their major and minor axis, $V_{eq} = V$ where the value of V is determined in accordance with C.3.2.

b) for sections symmetrical about their minor axis only,

$$V_{eq} = \left[\frac{2\,a\omega}{\left[\sqrt{4+\tau\omega}+\psi_a\sqrt{\omega}\right]^2} \right]^{0.25}$$

where:

$$\tau = 4a(1-a)+\psi_a{}^2$$

a and ψ_a are as defined in C.3.2

$$\omega = \frac{\pi^2 d_f^2 EI_z}{GI_T L_w^2}$$

I_z is the second moment of area of the beam about its minor axis

I_T is the St Venant torsional constant

θ_R is the greatest value of the rotation of a restraint about the longitudinal axis of the beam, due to a torque equal to a unit torque multiplied by $1/m$, applied to each restraint. When restraint is provided by uniform diaphragms interconnecting beams, the value of θ_R should be taken as $\theta_{R1} + \theta_{R2}$ where:

θ_{R1} is the rotation due to the flexibility of the diaphragm calculated as the greatest rotation about the longitudinal axis of a beam at a connection between the diaphragm and the beam under unit moments in the plane of the diaphragm multiplied by $1/m$, applied to each connection, in the same sense on each beam. (see Figure C.2).

θ_{R2} is the greatest value of rotation of a beam at the middle of a half wavelength of buckling due to the vertical deflections of the beams. A unit torque multiplied by $1/m$ should be applied to each beam at each diaphragm connection, the diaphragms being assumed to be rigid for this calculation, in directions of opposite sense in consecutive waves and the same sense on each beam.

When such restraint is provided by diaphragms interconnecting two or more beams, the rotations should be calculated assuming the above torque to be applied in the same rotational direction by equal and opposite horizontal forces on the diaphragms. In such cases the restraining torques must be resisted by equal and opposite vertical forces on the connected beams, equal to the value of the torque divided by the beam spacing, and account should be taken of the deflections of the beams due to all the restraining torques, θ_R being taken as that at the restraint where there is the greatest total rotation.

NOTE 1: The effective length relationships in Figure C.1 do not apply when considering buckling with a half-wavelength equal to L_r. In such a case, the slenderness should be determined assuming that there is fully effective intermediate lateral restraint.

NOTE 2: The resistance of a beam with $kL_w < L_r$ and $L_w > L_r$ may be less than for buckling between restraints with $L_w = L_r$

NOTE 3: For beams with a single central restraint, a single half wavelength over the span should be assumed, unless the restraint parameter derived from that value is such that it is to the right of the vertical arrow on the appropriate curve for the value of V_{eq}, in which case either:

a) the span should be considered to buckle in two half waves, between effective restraints

or

b) the span should be considered to buckle in a single half wave and k should be derived from Figure C.1

whichever gives the lower moment of resistance.

NOTE 4 Where, for multiple restraints, the value of $V_{eq}{}^4 L_w{}^3/[EI_{z,c}\theta_R d_f{}^2(1-a)]$ exceeds the maximum value shown in Figure C.1, the procedure given in C.4.6 may be used to derive the appropriate value of k.

NOTE 5 When only pairs of beams are interconnected by diaphragms, θ_{R2} may be taken as $(m+1)L_w^3 / 24mEI_y B^2$ where I_y is the second moment of area of each beam about its major axis, B is the spacing of the beams and m is the number of restraints within the half wavelength.

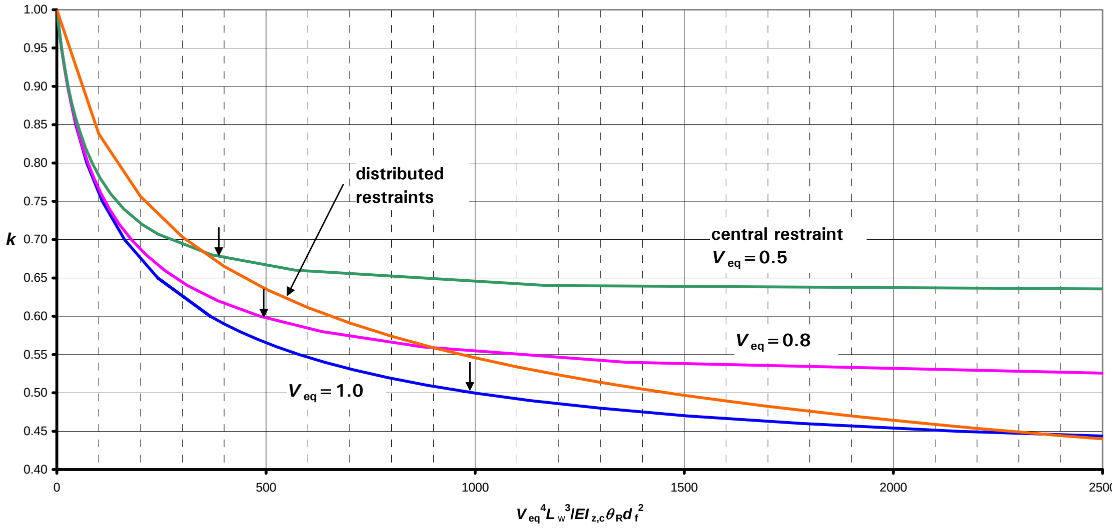

Figure C.1 *Effective length factor for beams with discrete torsional restraints*

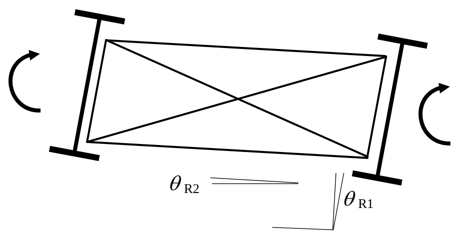

Figure C.2 *Rotations of paired beams subject to unit torques*

C.4.6 Equations for deriving *k*

Relationship for multiple torsional restraints

The value of k is given by:

$$k = \left[1 + \frac{V_{eq}^4 L_w^3}{\pi^4 \, EI_{z,c} d_f{}^2 \theta_R (1-a)}\right]^{(-0.25)}$$

Where the parameters are as defined in C.4.5.

Relationship for single central torsional restraint

The relationships k, V_{eq} and $V_{eq}^4 L_w^3 \left[EL_{z,c} \theta_R d_f^2 (1-a)\right]$ can be derived from the following equation:

$$2\pi^2\left(1-V_{eq}{}^4\right)\left[\frac{\alpha\left(2\mu(nL)^3 Cosh\dfrac{nL}{2} - (mL)^3 Cos\dfrac{mL}{2}\right)}{\left(Sin\dfrac{mL}{2} + 2\mu Sinh\dfrac{nL}{2}\right)}\right]$$

$$= \frac{V_{eq}{}^4 L_w{}^3}{\theta_R EI_{z,c} d_f{}^2 (1-a)}$$

where:

$$mL = \sqrt{-\frac{1}{2\alpha} + \sqrt{\frac{1}{4\alpha^2} + \frac{\pi^2 c^2 \left(1+\pi^2\alpha\right)}{\alpha}}} \quad \text{and}$$

$$nL = \sqrt{\frac{1}{2\alpha} + \sqrt{\frac{1}{4\alpha^2} + \frac{\pi^2 c^2 \left(1+\pi^2\alpha\right)}{\alpha}}} \text{, in which}$$

$$\alpha = \frac{V_{eq}{}^4}{\pi^2\left(1-V_{eq}{}^4\right)} \quad \text{for } V_{eq} \le 0.999 \text{ or otherwise } = 25$$

$$c = \left(\frac{1}{k}\right)^2$$

$$\mu = -\frac{mLCos\left(\dfrac{mL}{2}\right)}{2nLCosh\left(\dfrac{nL}{2}\right)}$$

and the other parameters are as defined in C.4.3.

These expressions may be used directly to derive the restraint stiffness parameter k required to achieve a given effective length for buckling with a half wavelength equal to L_w.

The value of the effective length at which the critical buckling moment for a beam with a central restraint and a half-wavelength equal to the span L equals that for buckling in the second mode with a half-wavelength equal to $L/2$ is given by:

$$k = \left[\frac{1 + \pi^2 \alpha}{4\left(1 + 4\pi^2 \alpha\right)} \right]^{0.25}$$

These values are shown by vertical arrows on Figure C.1 for three values of V_{eq}.

APPENDIX D Steel Bridge Group

The Steel Bridge Group is a technical forum that has been established to consider matters of high-priority interest to the steel bridge construction industry and to suggest strategies for improving the use of steel in bridgework. At the time of preparation of this publication, the Group included the following members:

Mr C R Hendy (Chairman)	Atkins
Mr E Atherton	Cass Hayward LLP
Mr S Battacharya	Mouchel
Mr I K Bucknall	Network Rail
Mr S Chakrabarti	Consultant (formerly of Highways Agency)
Mr C P E Cocksedge	AECOM
Mr D Dickson	Mabey Bridge Ltd
Mr C Dolling	Corus
Mr J E Evans	Flint & Neill Ltd
Mr R Hornby	Arup
Mr I E Hunter	Consultant (formerly of Cleveland Bridge Ltd)
Mr D C Iles	The Steel Construction Institute
Mr S J Matthews	WSP Consulting Engineers
Mr B R Mawson	Capita Symonds
Dr D Moore	The British Constructional Steelwork Association Ltd
Mr C J Murphy	Flint & Neill Ltd
Mr J D Place	Mott MacDonald
Mr R G Thomas	Rowecord Engineering Ltd
Mr G Waley	Edmund Nuttall Ltd

The Group has been responsible for the production of the comprehensive set of *Guidance Notes on Best Practice in Steel Bridge Construction* that is referred to in this publication.