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Design Guide for Composite Box Girder Bridges

(Second edition)

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FOREWORD

Foreword to Second edition

This second edition of the publication has been updated to reflect changes consequent on the revision in 2000 of BS 5400-3. Technically, the changes affecting this publication are modest in extent, unlike the changes that were needed to the other SCI bridge design guides (because the rules for I-section beams were much more significantly affected by the revision). Opportunity has been taken to correct a small number of minor errors in the text and to update some of the references. This publication is complementary to the two revised design guides for composite bridges.

Foreword to first edition

This guide is the fourth in a series of complementary design guides for composite highway bridges. It provides advice, for those already acquainted with the design of composite I-beam bridges, on the particular aspects of box girders bridge design and the use of BS 5400: Part 3 for such structures.

The guide has been reviewed by an Advisory Group of experienced bridge designers. Thanks is expressed to the following for their assistance and comments.

Mr C W Brown	The Steel Construction Institute
Mr C V Castledine	Butterley Engineering Ltd
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Mr J D Place	Mott MacDonald
Mr W Ramsay	British Steel, Sections Plates & Commercial Steels

Worked examples included in the book are based on designs by Mott MacDonald and N J Prescott Consulting Engineers Ltd. We are grateful to the London Dockland Development Corporation and to Cheshire County Council for permission to use the designs for this purpose. Thanks are also expressed to Mr M R Milnes and Ms P Ribbeck for their help in the preparation of the worked examples.

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Contents

SUMMARY

Design Guide for Composite Box Girder Bridges

Composite box girder construction offers an attractive and economic form of construction for medium span highway bridges. The torsional properties of the closed section are often advantageous in reducing and simplifying the support arrangements and are particularly useful when curvature in plan is required.

This publication provides guidance on the design of composite box girder bridges, generally in accordance with BS 5400. The guide describes features of initial and detailed design and explains how the Standard is applied to the design of these structures. Flow diagrams are provided as further guidance to the use of the Standard. Two worked examples are included, based on the designs for actual structures. These give extracts of the design relating to the features particular to the box girder form, together with a commentary on the calculations.

This second issue of the publication has been updated to reflect changes consequent on the revision in 2000 of BS 5400-3.

Guide de dimensionnement des ponts mixtes en caisson

Résumé

Les ponts mixtes en caisson constituent une solution intéressante et économique pour les ponts routiers de moyennes portées.

Les excellentes propriétés torsionnelles de la section fermée permettent de réduire et de simplifier les appuis, en particulier dans le cas de ponts courbes.

Cette publication se présente comme un guide de dimensionnement des ponts mixtes en caisson et est, en grande partie, basée sur la norme BS 5400. Elle décrit les différentes étapes du dimensionnement préliminaire et du dimensionnement détaillé et explique comment appliquer la norme à ce type de structure. Des organigrammes permettent d'expliciter clairement les procédures de dimensionnement. Deux exemples de dimensionnement sont développés. Ils donnent des extraits du dimensionnement concernant les points particuliers relatifs à la forme en caisson ainsi que des commentaires sur les calculs.

La deuxième impression de la publication a été mise à jour afin de prendre en compte les changements suite à la révision de l'année 2000 de la norme BS 5400-3.

Leitfaden zur Berechnung von Kastenträgerbrüken als Verbundquerschnitt

Zusammenfassung

Kastenträger als Verbundquerschnitt sind attraktiv und wirtschaftlich für Straßenbrüken mittlerer Spannweite. Die Torsions-Eigenschaften des geschlossenen Querschnitts sind oft vorteilhaft bezüglich der Auflager-Anordnung und besonders nützlich bei im Grundriss gekrümmten Brücken.

Diese Veröffentlichung vermittelt eine Anleitung bei der Berechnung von Kastenträgerbrücken als Verbundquerschnitt, im allgmeinen in Übereinstimmung mit

BS 5400. Der Leitfaden beschreibt Grundzüge der Vor- und Ausführungsberechnung und erläutert die Anwendung der Vorschrift. Flußdiagramme werden als weitere Hilfsmittel bereitgestellt. Zwei Berechnungsbeispiele, auf aktuellen Projekten basierend, sind enthalten. Sie enthalten Auszüge der Berechnung, speziell hinsichtlich der Kastenträgerform und einen Kommentar zur Berechnung.

Diese zweite Ausgabe der Publikation wurde auf den neuesten Stand gebracht um Änderungen von BS 5400-3 aus dem Jahr 2000 aufzuzeigen.

Guía de proyecto para puentes mixtos con sección en cajón

Resumen

La contrucción de vigas mixtas es atractiva para puentes de carretera con luces medias. Las propiedades a torsión de las secciones cerradas son ventajosas al reducir y simplificar los apoyos especialmente en los casos con curvatura en planta.

La guía es una ayuda para los proyectistas de puentes mixtos y se ajusta generalmente a la BS 5400. Se describen características de anteproyecto y proyecto completo y se explica como se pueden aplicar las Normas a esas estructuras. Se incluyen diagramas de flujo que ayudan a aplicar la Norma así como dos ejemplos desarrollados que se basan en estructuras reales. Dan detalles relativos a la forma característica del puente en cajón así como comentarios sobre los cálculos.

Esta segunda edición ha sido actualizada para que incluya los cambios consecuentes a la revisión de la BS 5400-3 llevada a cabo en el año 2000.

Guida alla progettazione di ponti composti con travata a sezione scatolare

Sommario

Le construzioni composte a sezione scatolare costituiscono una soluzione efficiente e vantaggiosa nel campo dei ponti autostradali di media luce. Le caratteristiche torsionali delle sezioni chiuse consentono nella maggior parte dei casi di ridurre e semplificare i dettagli relativi agli appoggi e risultano particolarmente utili nel caso di impalcato curvo.

Questa pubblicazione fornisce, per il progetto di ponti composti con travata a sezione scatolare, indicazioni, nella maggior parte dei casi in accordo con la normativa BS 5400. Questa guida riporta i principali passaggi sia del predimensionamento sia della progettazione e mostra l'applicabilita' delle norme a queste strutture. Le sequenze della progettazione sono anche riassunte in diagrammi di flusso per semplificare l'uso della normativa.

Sono proposti due esempi applicativi (la cui progettazione e' basata su dati realistici) in modo da fornire sia dettagli progettuali relativi alle caratteristiche peculiari della forma scatolare sia commenti sulle calcolazioni.

Questa seconda versione della pubblicazione è stata aggiornata a causa delle modifiche apportate nell'anno 2000 alla normativa BS 5400 parte 3

Dimensioneringsregler för samverkansbroar med lådbalkar

Sammanfattning

Samverkande konstruktioner med lådbalkar erbjuder en attraktiv och ekonomisk lösning för motorväggsbroar med medelstora spännvidder. Den stora vridsyvheten som en sluten tvärsektion har är ofta fördelaktig med hänsyn till möjligheter att reducera och förenkla upplagsanordningar vilket är speciellt värdefullt när det är fråga om kurva i plan.

Denna publikation innehåller dimensioneringsregler för samverkansbriar med lådbalkar enligt BS 5400. Dimensioneringsregler behandlar både förprojektering och detaljprojektering samt förklarar tillämpning av BS för respektive konstruktioner. Det presenteras också flödesschema som fortsatt ledning vid användning av standardföreskrifter. Publikationen innehåller två övningsexempel baserade på dimensionering av aktuella knonstruktioner. De illustrerar huvudprinciper för dimensionering m h t frågeställningar som är specifika för lådbalkar tillsammans med kommentarer till beräkningar.

1 INTRODUCTION

For medium span highway bridges, composite box girders offer an attractive form of construction. Design and construction techniques already popular and common for the I-beam form of composite bridges can be utilised to produce box girder structures of clean appearance whilst maintaining relative simplicity and speedy construction procedures. The scope of application of such designs could cover the span range from about 45 m to 100 m.

This guide provides an explanatory text which covers the design principles relevant to composite box girders and the use of codified rules for design. It includes a series of flow diagrams which illustrate the sequence of procedures involved in implementing the code rules, followed by selected worked examples of key aspects, based on designs for real structures.

This publication is complementary to other SCI design guides[1] on the design of composite bridges using I-section girders. It has been produced generally to the same format, for ease of use, and may be used independently of the other guides, although for more detailed treatment of slab design, reference should be made to the guide for simply supported bridges.

The guide assumes that the reader is familiar with the general principles of limit state design and has some knowledge of structural steelwork for bridges. Some of the detailed design aspects are more complex than for I-beam bridges, but an advanced knowledge of analysis techniques is not required.

Further guidance on various aspects of steel bridge design and construction are given in *Guidance notes on best practice in steel bridge construction*[2]. Where specific reference is made to one of those Notes in this publication, it is given in the form 'GN 1.02'.

2 DESIGN BASIS

2.1 Forms of composite construction

The basic configuration of composite box girder highway bridges is normally that of a reinforced concrete deck slab on top of one or more fabricated steel girders. In this publication, attention is concentrated upon medium-span highway bridges - those with the longest span in the range 45 m to 100 m.

Twin boxes will normally be used for carrying minor roads (two lanes and two footways). Multiple boxes, perhaps four in number, may be needed for wider roads, such as dual carriageways. Wide roads can alternatively be carried on twin box sections with cross-girders, so that the slab spans longitudinally, rather than transversely between the lines of the box webs, though this form is not common for spans less than 100 m. Single box sections might be feasible for narrow roads, if used in conjunction with haunching of the slab over the web lines. Wide single boxes with crossbeams and cantilevers are more appropriate to longer spans and are outside the scope of this book.

Two different classes of composite box girders may be considered - those where complete closed steel boxes are fabricated, and those where an open 'U' section is fabricated. For either class, the box section may be either rectangular or trapezoidal (narrower at the bottom flange level than at the top).

In elevation, box girders may have a constant depth or may be haunched. Because their situation is often visually prominent, the use of a curved soffit is frequently encouraged for better appearance.

In plan, box girders can be curved, to suit the layout of the highway which they carry. The very good torsional properties of box sections make them particularly suited when truly curved girders are required.

2.2 Design standards

2.2.1 National Standards

The design and construction of composite bridges is covered by British Standard *BS 5400: Steel, concrete and composite bridges*[3]. The Standard comprises Codes of Practice for design and Specifications for design loadings, construction materials and workmanship. A Limit State design basis is used in the Codes.

Part 5 of BS 5400 covers the design of composite bridges; it deals with general principles and the details of the interaction between steel and concrete elements. Design of steel elements is covered in Part 3 and of concrete elements in Part 4. The loading to be applied is specified in Part 2.

When using Parts 3, 4 and 5 in conjunction, it should be noted that the treatment of the partial factors λ_{f3} is different. It is suggested that the method of Part 3 be used consistently throughout, to avoid confusion. This means that λ_{f3} should always be applied as a divisor on strength, rather than as a multiplier on loads.

The design rules in Part 3 for box girders were developed from earlier, and in many respects more detailed, rules which were published in 1973[4]. The introduction of BS 5400, and the relationship to those earlier rules, is described in the proceedings of a conference held in 1980[5].

2.2.2 Departmental Standards

Within the United Kingdom, responsibility for highway bridges is held by the government's four Overseeing Departments for highways - the Highways Agency (in England), the Scottish Executive Development Department, the Welsh Assembly Government and the Department fro Regional Development Northern Ireland. The requirements of these Overseeing Departments are given in the *Design Manual for Roads and Bridges*, which is introduced by document DMRB 1.0.1[6]. This design manual system comprises a set of Departmental Standards, which specify the requirements and implement the BSI Standards, and Departmental Advice Notes, which provide guidance. A list of relevant Standards and Advice Notes is given in Appendix B.

Departmental Standard BD 37/01 contains an amended version of Part 2; in particular the intensity of HA traffic loading has been increased. Part 5 has been amended by BD 16/82, and a composite version of Part 5 with these amendments is available. A small number of amendments are made to Part 4 by BD 24/92. Document BD 13/90, which implemented BS 5400-3:1982 and gave a number of technical amendments to it has not yet (December 2003) been updated to the 2000 issue of the Standard. It is not expected that there will be any technical changes, when it is issued, that would affect box girder design.

Within this book, reference will be made to the modified versions of Parts 2 and 5, rather than the original BSI documents. To emphasise this, the modified versions will be referred to as Part 2* and Part 5*.

3 BEHAVIOUR OF BOX GIRDER BRIDGES

3.1 General

Clearly, the feature which differentiates the behaviour of box girder bridge structures from I-beam structures is the much greater torsional stiffness of the closed section. The prime effect this has on global bending behaviour is to share the vertical shear more equally between the web planes.

Consequent upon this equal sharing and, in the case of the bottom flange at least, on the lesser number of flange plates, bending stresses are also more evenly shared.

As a result, box girders behave more efficiently - there is less need to design for peak load effects which occur on only one plate girder at a time.

On the other hand, the choice of box girders can lead to use of wide thin plate panels for web and flange, and these may be less efficient than more stocky sections. In particular, if more webs are introduced (than would be used with plate girders) the thinner web panels will need greater stiffening. Despite this they still might have a lower value of limiting shear stress and be less effective in bending. Wide compression flanges may also be less than fully effective, because of buckling considerations (plate girder flanges are normally fully effective). Care should be exercised in choosing a configuration that minimises any reduction in effective section on account of panel slenderness.

As well as the relatively straightforward behaviour in pure torsion, the use of box girders gives rise to other effects which must be considered - notably distortion and warping. For many bridges these effects can be minimised by appropriate internal stiffening and proportioning of the cross-section, but the effects do need to be considered.

Sections 3.2 to 3.7 describe briefly the different behaviour effects in box girders. For a more comprehensive explanation, see CIRIA Guide 3[7].

3.2 Bending, torsion and distortion

The general case of an eccentric load applied to a box girder is in effect a combination of three components - bending, torsion and distortion.

The first two of these components are externally applied forces, and they must be resisted in turn by the supports or bearings. As a first step, the bending and torsion components can easily be separated as shown in Figure 3.1.

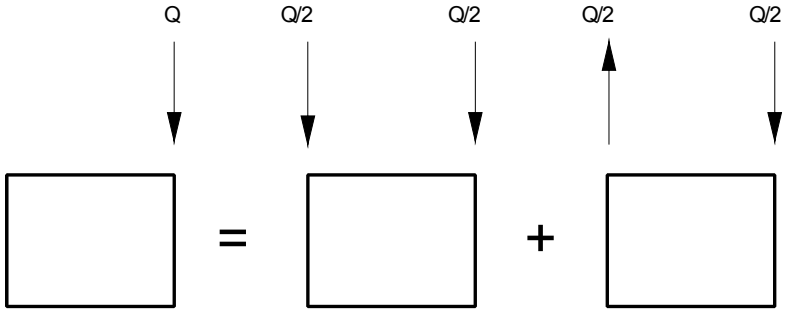

Figure 3.1 *Bending and torsion components of an eccentric load*

The torsion component is shown in Figure 3.1 simply as a force couple. However, torsion is in fact resisted in a box section by a shear flow around the whole perimeter. The couple should therefore be separated into two parts, pure torsion and distortion, as shown in Figure 3.2. The distortion component comprises an internal set of forces, statically in equilibrium, whose effects depend on the behaviour of the structure between the point of application and the nearest positions where the box section is restrained against distortion.

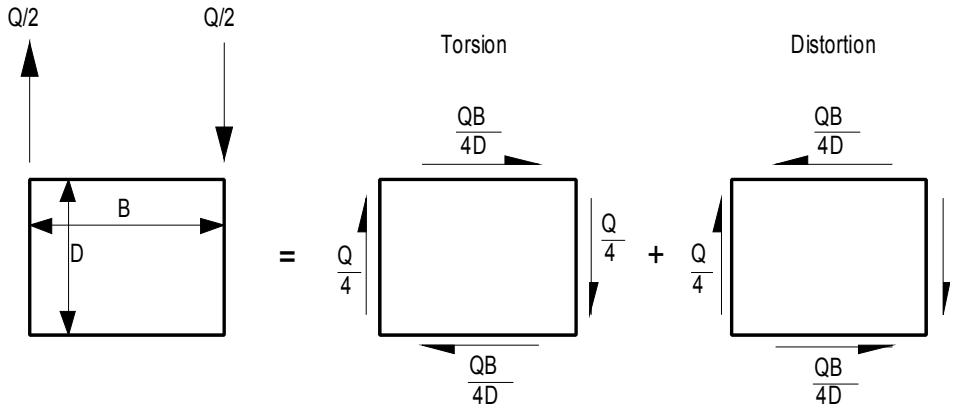

Figure 3.2 *Pure torsion and distortion components*

At supports, bearings will be provided. Where a pair of bearings is provided, they are usually either directly under each web or just inside the line of the webs. To resist forces reacting on the bearings as a result of the bending and torsion components, bearing support stiffeners will be required on the web. In addition, a diaphragm (or at least a stiff ring frame) will be required to resist the distortional effects consequent in transmitting the torsion from the box to a pair of bearing supports.

In some cases only a single bearing is provided (see further comment in Section 3.7); a stiffened diaphragm will be needed to resist the reaction and to distribute the force to the webs.

Between points of support, intermediate transverse web stiffeners may be provided to develop sufficient shear resistance in a thin web. Intermediate diaphragms or cross frames may be provided to limit the distortional effects of eccentrically applied loads; they are particularly effective where concentrated eccentric effects are introduced, such as from a cantilever on the side of the box. Intermediate cross-frames may also be provided to facilitate construction (see Section 4.8)

3.3 Torsion and torsional warping

The theoretical behaviour of a thin-walled box section subject to pure torsion is well known and is treated in many standard texts. For a single cell box, the torque is resisted by a shear flow which acts around the walls of the box. This shear flow (force/unit length) is constant around the box and is given by $q = T/2A$, where T is the torque and A is the area enclosed by the box. (In the torque is $QB/2$ and the shear flow is $Q/4D$.) The shear flow produces shear stresses and strains in the walls and gives rise to a twist per unit length, θ, which is given by the general expression:

$$\theta = \frac{T}{4A^2G}\oint\frac{ds}{t} \quad \text{or,} \quad \theta = \frac{T}{GJ}$$

where J is the torsion constant.

However, it is less well appreciated that this pure torsion of a thin walled section will also produce a warping of the cross-section, unless there is sufficient symmetry in the section. To illustrate how warping can occur, consider what would happen to the four panels of a rectangular box section subject to torsion.

Assume that the box width and depth are B and D respectively, and that the flange and web thicknesses are t_f and t_w. Under a torque T, the shear flow is given by $q = T/2BD$.

Consider first the flanges. The shear stress in the flanges is given by $\tau_f = q/t_f = T/2BDt_f$. Viewing the box from above, each flange is sheared into a parallelogram, with a shear angle $\phi = \tau_f/G$; if the end sections were to remain plane, the relative horizontal displacement between top and bottom corners would be ϕL at each end (see Figure 3.3), and thus there would be a twist between the two ends of $2\phi L/D = 2\tau_f L/DG = TL/BD^2Gt_f$.

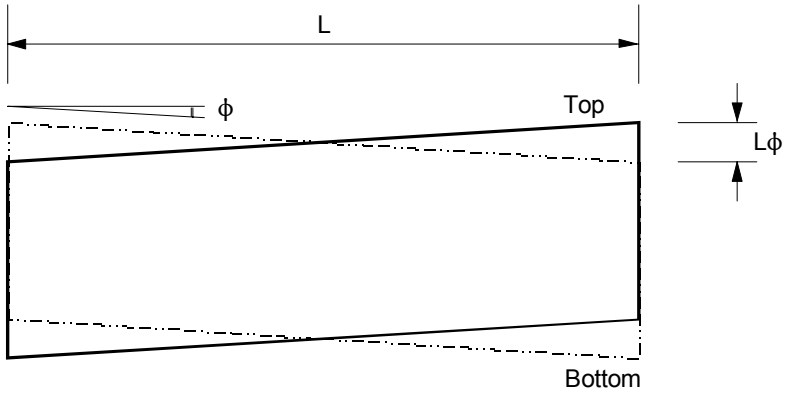

Figure 3.3 *Shear displacement of top and bottom flanges (ends kept plane)*

By a similar argument, viewing the box from the side and considering the shear displacements of the webs, if the end sections were to remain plane the twist of the section would be TL/B^2DGt_w. As the twist must be the same irrespective of whether we consider the flanges or the webs, it is clear that the end sections can only remain plane if $TL/BD^2Gt_f = TL/B^2DGt_w$, i.e. $Dt_f = Bt_w$. If this condition is not met, the end sections cannot remain plane; instead, there will be a slight

counter-rotation in their planes of the two flanges and of the two webs, and a consequent warping of the section. Typical warping for this example is shown in Figure 3.4.

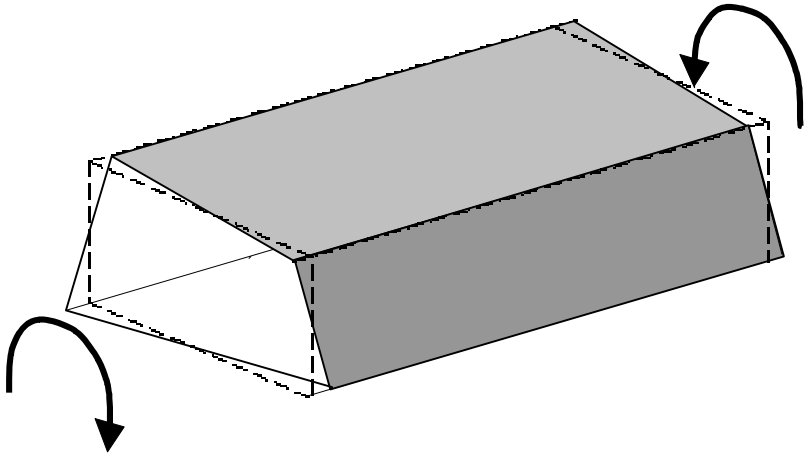

Figure 3.4 *Warping of a rectangular box subject to pure torsion*

Of course, for a simple uniform box section subject to pure torsion this warping is unrestrained and does not give rise to any secondary stresses. But if, for example, a box is supported and torsionally restrained at both ends and then subjected to applied torque in the middle, warping is fully restrained in the middle by virtue of symmetry and torsional warping stresses are generated. Similar restraint occurs in continuous box sections which are torsionally restrained at intermediate supports.

This restraint of warping gives rise to longitudinal warping stresses and associated shear stresses in the same manner as bending effects in each wall of the box. The shear stresses effectively modify slightly the uniformity of the shear stress calculated by pure torsion theory, usually reducing the stress near corners and increasing it in mid-panel. Because maximum combined effects usually occur at the corners, it is conservative to ignore the warping shear stresses and use the simple uniform distribution. The longitudinal effects are, on the other hand greatest at the corners. They need to be taken into account when considering the occurrence of yield stresses in service and the stress range under fatigue loading. But since the longitudinal stresses do not actually participate in the carrying of the torsion, the occurrence of yield at the corners and the consequent relief of some or all of these warping stresses would not reduce the torsional resistance. In simple terms, a little plastic redistribution can be accepted at the ultimate limit state (ULS) and therefore there is no need to include torsional warping stresses in the ULS checks.

3.4 Distortion

When torsion is applied directly around the perimeter of a box section, by forces exactly equal to the shear flow in each of the sides of the box, there is no tendency for the cross section to change its shape.

If torsion is not applied in this manner, a diaphragm or stiff frame might be provided at the position where the force couple is applied to ensure that the section remains square and that torque is in fact fed into the box walls as a

shear flow around the perimeter. The diaphragm or frame is then subject to a set of distortional forces as shown in Figure 3.2.

Provision of such diaphragms or frames is practical, and indeed necessary, at supports and at positions where heavy point loads are introduced. But such restraint can only be provided at discrete positions. When the load is distributed along the beam, or when point loads can occur anywhere along the beam such as concentrated axle loads from vehicles, the distortional effects must be carried by other means.

To illustrate how distortion occurs and is carried between effective restraints, consider a simply supported box with diaphragms only at the supports and which is subject to a point load over one web at midspan. If there is no transverse moment continuity at the corners (a pinned connection between web and flange) the cross section will distort as shown in Figure 3.5. Each side of the box bends in its own plane and since the four sides remain connected along their common edges, the cross section of the box has to change shape in the manner shown.

The in-plane bending of each side gives rise to longitudinal stresses and strains which, because they are in the opposite sense in the opposing faces of the box, produce a warping of the cross section (in the example shown the end diaphragms warp out of their planes, whilst the central plane can be seen to be restrained against warping by symmetry). The longitudinal stresses are therefore known as *distortional warping stresses*. The associated shear stresses are known simply as *distortional shear stresses*.

If a flexible intermediate cross-frame (a ring stiffener without any triangulated bracing in its plane) is introduced to this example at the point of application of the load, it tends to resist the distortion of the cross section by 'sway bending' of the form shown in Figure 3.6. Obviously, the stiffer the frame the less the distortion of the cross section. (Cross bracing or a plated diaphragm would be even more effective.)

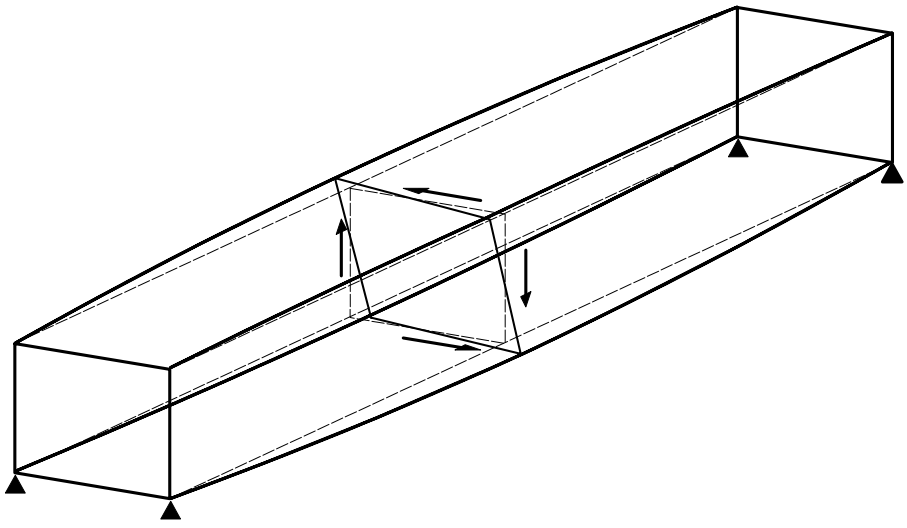

Figure 3.5 *Distortion of unstiffened box (pinned corners)*

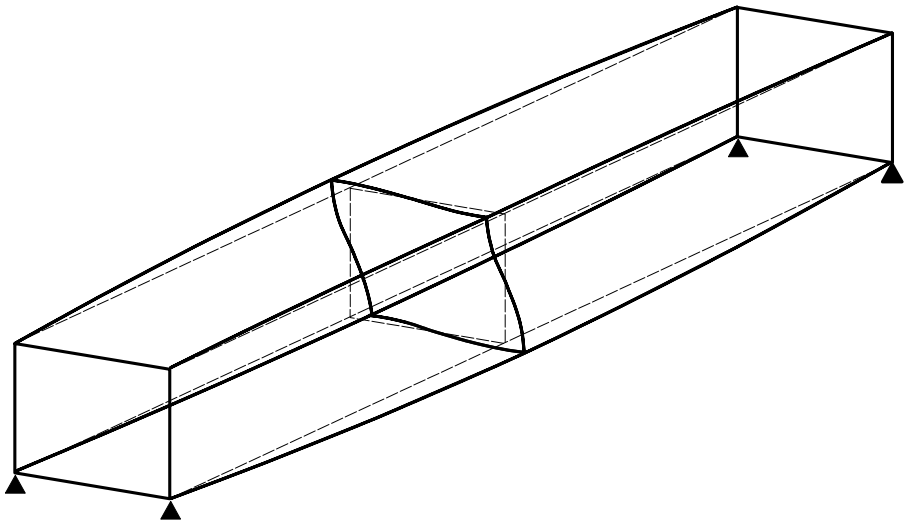

Figure 3.6 *Distortion of box with stiff corners or cross-frames*

On the other hand, if there is no intermediate cross-frame, but there is transverse moment continuity at the corners, the box walls are subject to the same sway deflection pattern as seen in Figure 3.6, but the bending now takes place in the walls of the box.

The bending of cross-frames and the walls of a box, as a result of the distortional forces, produces transverse distortional bending stresses in the box section.

In general the distortional behaviour depends on interaction between the two sorts of behaviour, the warping and the transverse distortional bending. The behaviour has been demonstrated to be analogous to that of a beam on an elastic foundation (BEF), with the beam stiffness representing the warping resistance and the elastic foundation representing the transverse distortional bending resistance. A comprehensive description of the analogy is given in a paper by Wright[8]. The BEF model is used as the basis for the rules in Annex B of BS 5400-3. (An alternative method, based on a pair of effective beams at the spacing of the box webs is described by Richmond[9].)

When a point load is applied eccentrically to a box section, the distortional effects are greatest local to the point of application. The way that they reduce away from the point of application can be appreciated by considering the BEF analogy for a load in the middle of a box girder which is simply supported and which has diaphragms only at the supports. A diagrammatic representation of the response is shown in Figure 3.7. Warping stresses are represented by the bending of the beam and distortional bending stresses by the displacement of the foundation. The rate at which the effects decrease depends on the relative magnitudes of distortional bending and warping resistances and on the length of the beam.

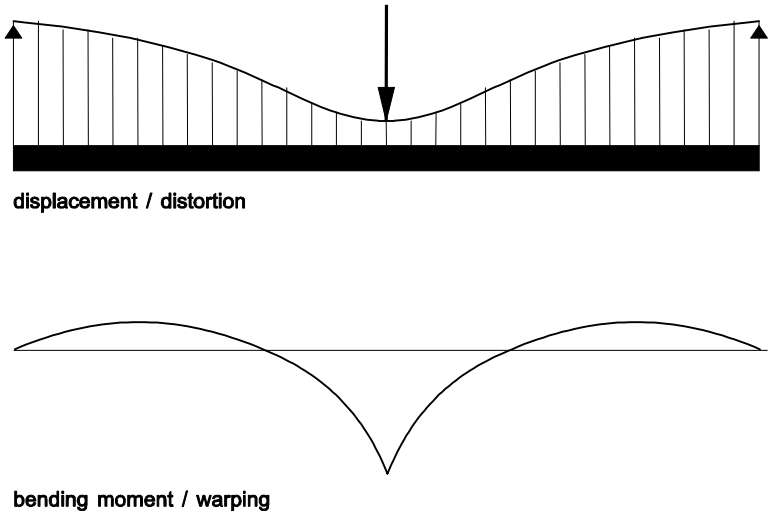

Figure 3.7 *Beam on elastic foundation analogy*

The introduction of intermediate diaphragms in the box girder can be represented in the BEF analogy by the addition of discrete vertical restraints, or springs. If these restraints were rigid they would effectively reduce the length of the BEF model to the spacing of the restraints (ignoring continuity effects, which are relatively small), with consequent reductions in warping and distortional bending. However, in general the intermediate restraints should be considered as flexible springs and the BEF model analyzed accordingly. A modified response with flexible intermediate restraints is shown in Figure 3.8. Flexible restraints are quite effective in reducing distortional effects (particularly distortional bending), even when they themselves displace significantly. (A numerical example illustrating the benefit of flexible diaphragms is given in Hambly[10], pp 140-141.)

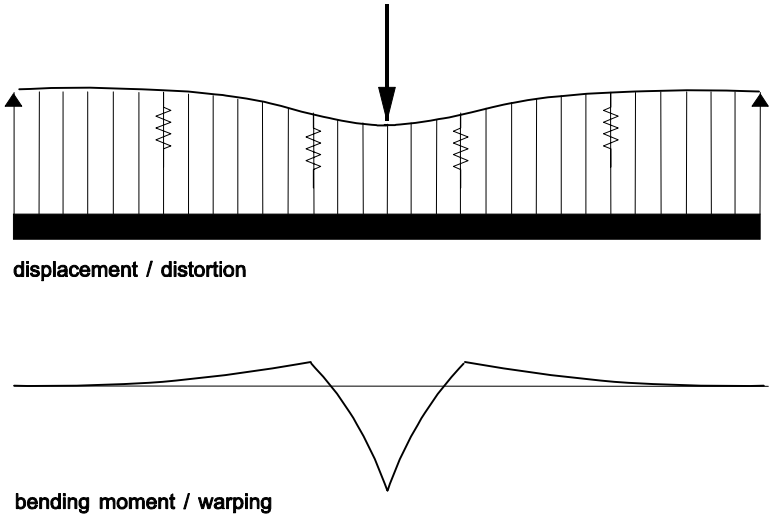

Figure 3.8 *BEF model with intermediate springs*

It must be emphasised that distortional effects are primary effects - they are an essential part of the means of carrying loads applied other than at stiff diaphragms - and they should not be ignored, even at ULS.

3.5 Stiffened compression flanges

The use of box girders allows, and indeed encourages, the use of wide flanges. But slender plate panels subjected to compression buckle at a load which is less than the 'squash load'. Consequently, wide flanges may need to be stiffened to carry sufficient load.

A long plate panel in compression, bounded on two sides and compressed along its length, tries to buckle in a square wave pattern, with a single half wave between the edges and alternating half waves of the same length along the panel (see standard texts, such as Timoshenko[11] for further description). The elastic buckling load depends on the reciprocal of the square of the wave length. To increase the load resistance, longitudinal stiffeners can be introduced which restrict the width of the individual panels. Because stiffeners share in carrying the load they become, effectively, struts in compression; they in turn need to be restrained at intervals against buckling out of the plane of the panel. This restraint is provided by transverse stiffeners, cross-beams or diaphragms. (In very wide and long flanges, with longitudinal and transverse stiffeners, the buckling of the whole stiffened panel needs to be considered, but that type of panel requires calculation of orthotropic properties and is beyond the scope of this publication.)

Actual buckling loads for plate panels depend on slenderness, yield strength and initial imperfection. Explicit expressions can be found for taking account of these variables, but more simple rules have been derived which simply express the strength of a plate panel in terms of an effective width which, if loaded to yield stress, would carry the same load as the failure load of the plate panel.

In a flange with longitudinal stiffeners, half of the effective width of each plate panel is considered to be attached to the stiffeners along the two boundaries to form effective struts between out-of-plane restraints. The strength of the flange is then the sum of the strengths of the effective struts.

3.6 Shear lag

In composite I-beam and slab construction only the deck slab is susceptible to shear lag. In box girders, wide steel flanges are also susceptible, particularly at the supports. Whilst shear lag can usually (but not always) be neglected at the Ultimate Limit State, it does need to be considered for fatigue behaviour, which must be analysed elastically, in the same way as a Serviceability Limit State. Exact calculation of shear lag for real situations can be very complex, but simple tabular relationships for standard cases are quite adequate for normal purposes.

3.7 Support of box girders

Clearly, traffic loads on any bridge will not normally be symmetrically disposed about the longitudinal centreline of the bridge; the support arrangements must be able to carry the twisting moments from any feasible disposition of the traffic loading. Plate girder bridges are torsionally flexible and weak; consequently at least two bearings must be provided at each support. (Commonly four bearings, one under each girder, are provided.)

Box girders however, are torsionally stiff and strong - it is usually adequate to provide only a single bearing under each box at intermediate supports and to carry the torsional forces to the end supports, where twin bearings are provided, one under each web.

When there is significant plan curvature, single bearings can sometimes be used at all supports, since the curvature of the line of supports generates torsional restraint.

4 INITIAL DESIGN

4.1 General

The initial design stage is considered here to cover the selection of structural arrangement and member sizes, after the highway layout has been determined by the highway engineer. The initial design is then followed by a detailed design stage (Section 5 of this guide), which covers checking in accordance with the Code and which leads to confirmed structural arrangements and details.

It is presumed in this guide that spans are in excess of about 45 m and that for such bridges the position of supports is largely determined, at least for the major span, by physical constraints. However, the bridge may well be a viaduct of successive spans over land and the designer may have the freedom to vary span lengths.

Naturally, the selection of a span length will require consideration of the costs of both sub- and super-structure, and a balance will have to be struck for overall economy. Such a balance is influenced strongly by the foundation conditions and their consequent cost. In considering the cost of the superstructure, the designer should make full use of the advantages gained by using composite box girder construction:

- economic span lengths are likely to be longer than with concrete construction
- span-long girder sections can be erected by mobile crane
- torsional performance may reduce bearing requirements (particularly with curved girders)
- torsionally stiff sections are stable (after erection) without intermediate bracing
- improved resistance to aerodynamic excitation.

The designer should also consider the benefits in appearance which box girders can offer:

- smooth lines, on the side faces and below
- clean surfaces, with no external visible web stiffeners
- use of sections curved in plan, where appropriate
- sloping webs.

Subsequent maintenance of the bridge should also be considered. The total external area to be painted is much less than for a comparable I-beam bridge, but there are no outstands on which to position temporary accesses. However, runway beams can often be provided inconspicuously for use by maintenance cradles. The clean surfaces of boxes mean that there are fewer corrosion traps and that, once access is achieved, painting is easier and quicker.

Alternatively, Weather Resistant Steel (WRS) should be considered in non-marine environments. Allowance for corrosion loss of WRS should be made in

accordance with BD 7/01. Guidance on the use of weather resistant steel is given in GN 1.07.

Access for internal inspection and maintenance should not be overlooked, even at the early stages of design. Access routes, manhole sizes and means of ventilation can influence the choice of configuration; lack of early consideration can be difficult to remedy at later stages. To minimise requirements for future access into boxes, weathering steel is now being used for box girders, even when the external surfaces are painted; this avoids the need for internal repainting.

4.2 Loadings

Highway bridges are usually designed to carry a combination of uniformly distributed loading (type HA) and an abnormal heavy vehicle (type HB). These loads, together with other secondary loads, are specified in Part 2* of the Code, except that the magnitude of the abnormal vehicle is chosen to suit the particular requirements for the road (usually 30, 37.5 or 45 units of loading). It should be noted that BD 37/01 has modified the applicable loading, particularly the intensity of HA loading.

In addition, the Highways Agency requires the consideration of Abnormal Indivisible Loads (AIL) on routes designated as Heavy or High Load Routes, where these loads have a more severe effect than HB loading on the particular superstructure.

4.3 Choice of a box girder form

Although for straight bridges box girders may prove more expensive (than I-beam girders and slab construction) in terms of simple capital cost of the superstructure, the advantages of the box girder form, such as better appearance and reduced maintenance, may well merit the evaluation of a box girder as an alternative for any bridge in the span range of 45 m to 100 m. For bridges with a significant plan curvature, box girders should always be considered.

Generally a box girder alternative will require approximately the same weight of steel as an I-beam bridge, possibly slightly less if the design is optimised to make best use of the advantages of box girders. Deck slab thickness will normally be similar for both forms of construction.

With box girders, the use of torsionally stiff beams can often enable the number of bearings or support positions to be reduced and this can lead to a more slender sub-structure.

Curvature is more easily achieved with box girders, although curvature of girders in plan is not common in the UK. (Such plan curvature of the road as is needed can usually be accommodated in I-beam construction by making continuous girders from a series of straight sections.) If true plan curvature is wanted, either for appearance or because the radius is unusually tight, box girders can effect curvature much more readily, and accommodate the torsional effects more easily. I-beams would require significant transverse bracing in these situations.

Box girders require more complex analysis and design than simple I-beams. It is therefore even more essential that the designer appreciates the consequences of his choice of structural configuration on plate thickness, plate stiffening, bracing arrangements, fatigue design and construction details. A good choice of initial design will minimise the detailed design work and lead to details which can be economically produced by the fabricator.

4.4 Cross section arrangements

The basic variables in choosing a cross section with box girders are:

- the shape of the box - trapezoidal or rectangular
- closed or open steel section
- with or without cross girders

Cross-girders are usually only found in larger span bridges, either when providing a very wide deck on twin boxes or when carrying a carriageway on a single box of large cross section. This form is beyond the scope of the present publication.

To illustrate the basic variables, typical examples of sections which have been used for actual structures are shown in Figures 4.1 to 4.3. The three cross sections illustrated demonstrate some of the different ways in which the torsionally stiff box section have been used to support the deck slab.

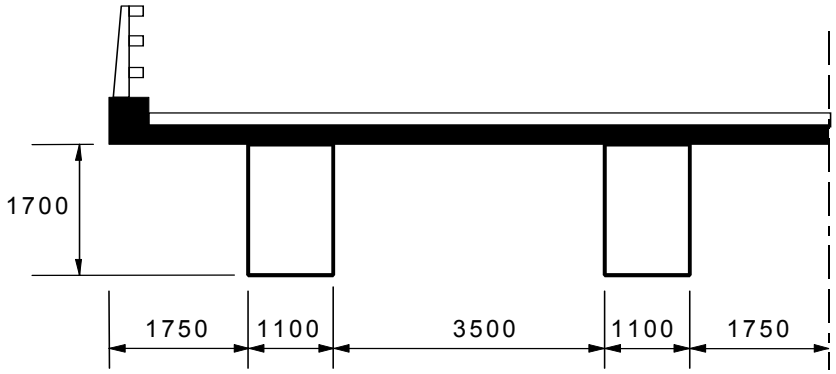

Figure 4.1 *Section with closed rectangular steel boxes*

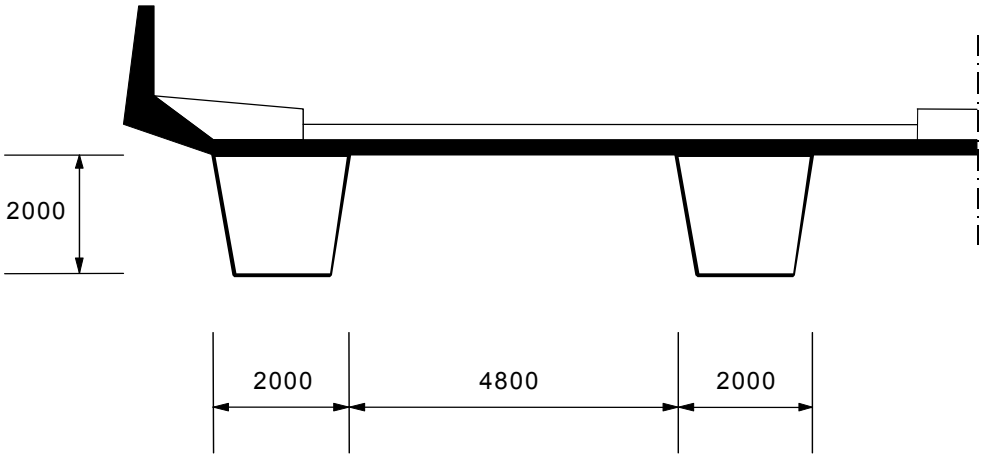

Figure 4.2 *Section with closed steel boxes*

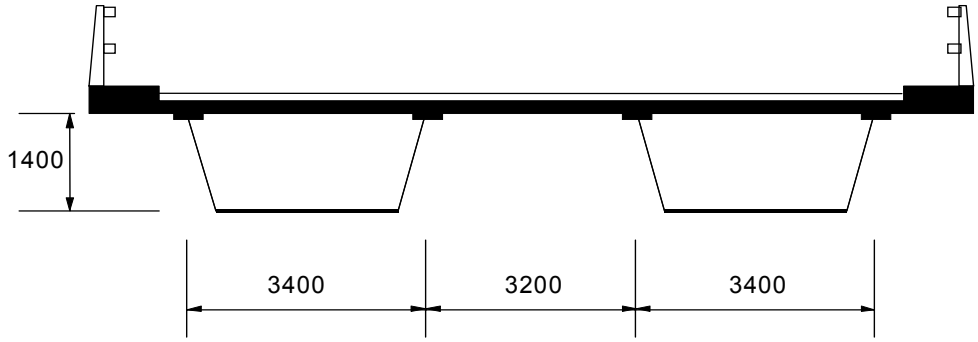

Figure 4.3 *Section with open steel boxes (46 m span)*

In Figure 4.1 the box provides support which is effectively a line support (albeit a broad line) and the slab span is similar to what might be used with ordinary plate girder and slab composite construction.

In Figure 4.2 a wider box is used, in conjunction with a wider spacing between boxes. A thicker slab (300 mm) is used which, in conjunction with the torsional restraint provided by the slightly wider boxes and stiffened steel top flange, allows the spacing between boxes to be increased.

In Figure 4.3 the open steel box is widened to create approximately equal spans for the slab.

With trapezoidal sections, the inclined webs reduce the width of the bottom flange and, for a given area, increase its thickness. The flange is therefore more likely to be fully effective. In the initial design stage, it should be considered that a wide trapezoidal box girder can often be used rather than a pair of plate girder I-beams.

For longer spans, narrow rectangular box girders can be substituted in place of heavy plate girders and the spacing between girders increased. Rectangular sections are suited to wide decks on multiple boxes, at wide spacing. Haunched

boxes are more easily arranged with rectangular sections (with trapezoidal sections, the bottom flange is narrower at supports than at midspan).

Whilst bracing arrangements between boxes do not need to be considered in detail at this stage, it might be noted that with the arrangement in Figure 4.3, deep crossbeams were provided between boxes at the third points of the main span to ensure that the tops of the four web lines remain essentially in the same plane. In the other two examples, no bracing was provided; any differential vertical displacement has to be accommodated by flexing of the slab.

4.5 Section depth

Typically, the construction depth of a parallel-flanged box girder might be between 1/20 and 1/25 of the major span. Shallower sections can be used, with possible benefit to appearance, at the expense of greater weight.

Variable depth sections are relatively straightforward with rectangular sections and can give an attractive slender appearance, particularly over a river. The use of a curved soffit leads to the requirement for internal transverse flange stiffeners to resist the radial component of force, though this is not onerous with large radii. Curvature is usually applied only to the major span and to the spans either side of it.

With trapezoidal sections, a variation in depth will result in either a change (along the bridge) in the width of one of the flanges, or the web inclination will change (the web plate will be warped). The appearance of the latter is likely to be somewhat disquieting, unless unnoticeably minor, and the former is to be preferred. Indeed, when well executed the former arrangement can produce a particularly good appearance (see Reference [12], for example).

4.6 Initial selection of flange and web sizes

Flange and web sizes depend of course on the configuration of the cross section and the moments to be carried. A first estimate of sizes can be based on very simple approximations and these can be quickly refined to a better initial selection suitable for use in the detailed design. It is suggested that the first coarse estimate is used to determine properties for a simple grillage model and that model is used to give a better indication of the distribution of bending moments so that a better initial design can be made. Several iteration cycles are likely to be needed at this stage.

Some guidance on making the first estimate is given in Appendix A.

The girders will be made up in several sections, in lengths suitable for transportation. This gives scope for variation of make-up between the different sections. At the initial design stage, splice positions should be considered and advantage taken to change plate thicknesses where appropriate.

The main girders should normally be structural steel to grade S355 of BS EN 10025[13], since it is more cost-effective than lower grades.

4.7 Availability of steel plate and sections

Plates are generally available in a range of sizes, typically up to a maximum length of 18.3 m. Corus can supply details of the full range which they produce[14]. All the standard grades to BS EN 10025 are normally available, though it is most likely that grade S355, quality J2G3 or J2G4 will be required. (For thicknesses over 55 mm, quality K2G3 or K2G4 may be needed.) Up-to-date information about availability should be confirmed during the initial design stage.

Rolled sections (beams, angles, etc and hollow sections) for stiffeners and bracing may only be required in fairly small quantities and can be purchased by the fabricator from a stockist, or direct from the producer. For economy, it is best to standardise on as few section sizes as possible. Most commonly-used sections are readily available. Again, Corus can supply details of the ranges which they produce[15,16].

4.8 Economic and practical considerations

It is important that the initial design (the configuration in section and elevation) takes proper account of the particular features of box girders, their construction, performance and maintenance. A box girder is not just a pair of plate girders with a common bottom flange. If proper account is not taken during the initial stage, the design will be less efficient and is likely to give rise to problems later which will be difficult to overcome.

The designer should understand how the box is constructed. Automatic T and I welding machines are not yet able to cope with box girders, so the girders must be assembled by traditional methods. (This inevitably means that they will be more expensive to make than I-girders.) The flanges and webs will be fitted with stiffeners before they are assembled. Cross-frames or diaphragms will be needed at this stage to ensure that the cross section is held in shape during welding (the designer should therefore normally provide them at regular spacing, even if not strictly essential for control of distortion). Closed trapezoidal boxes are usually assembled in inverted position and the bottom flange added last of all. Internal welding after closure is usually necessary; support diaphragms at least must be welded all round. Access and ventilation are more easily arranged in the shop than on site but even so the amount of internal welding should be minimised where possible.

It is difficult to ensure perfect alignment of every web and flange transverse stiffener at the corners and a connection detail, such as lapping, which will accommodate small differences should be chosen.

Joints between flanges and webs are easier and cheaper to make as fillet welds, rather than as butt welds. (Butt welds are used in box girders for railway loading, where fatigue is more onerous; they are not necessary for highway bridges.)

The box will have to be transported after assembly. There are limits on length (27.4 m long) and width (4.3 m wide) for unrestricted travel on public roads, but larger sizes can be carried by special permission. Advice should be sought from the appropriate highway authority for travel in the relevant localities. Fabricators are familiar with the procedures and with the transport of large

loads; advice can also be sought from them. Further advice is given in GN 7.06

Open boxes will require some plan bracing on the open side, to provide torsional stiffness during construction. With both open and closed boxes a cross-frame should be positioned close to the end of one girder at each splice but not close to the end of the other girder (or it may be difficult to match the two ends).

Fitting and welding of stiffeners is expensive and it is often cheaper to use thicker plate with less stiffening. Butt welds allow a change of plate thickness where stresses are lower but making the weld may be more expensive than using the thicker plate throughout an individual length of girder. Stiffened diaphragms can be very expensive to fabricate: it is not worth trying to minimise the weight of a support diaphragm, since it is a tiny fraction of the whole structure and, being over the bearings, does not contribute to dead load moments. When detailing a stiffened diaphragm be sure to allow sufficient space for the welder to make the welds. Thick unstiffened diaphragms can even be considered for smaller boxes.

Bolted splices are quicker to make on site, but sealing details at the ends of cover plates must be considered. If welding is used for the web and flange splices, bolting can still be effective internally for splicing longitudinal stiffeners. Such stiffeners should always be spliced with cover plates, because true alignment is very difficult to achieve.

Articulation arrangements (the configuration of fixed, guided and free bearings) should be established at an early stage, so that bearing positions, bearing stiffener requirements and the need for bracing between boxes at supports can be determined. The diaphragm details, stiffeners and manhole sizes/positions may affect the box section size.

The 1994 Construction (Design and Management) Regulations require a formal record by the designer of the consideration and provision of access for such issues as working in enclosed spaces (on site and in the shop) and making site joints. Provision of access through holes in the web or bottom flange at intermediate positions may be necessary, rather than entry from the end of the bridge, through the box. See further advice about the CDM regulations in GN 9.01.

Drainage internally should be considered - avoid ‘closed’ corners where moisture and dirt can collect.

Composite box girders in this span range are often considered in comparison with prestressed concrete box girders. In such a comparison, the advantages of the steel girder in speed and ease of construction on site should be fully recognized. Externally, the surface of the steel girder is durable, using modern protection systems or weather resistant steel. Internally, the environment is closed and should require no more than routine inspection.

Advice on fabrication details and construction schemes should be sought from an experienced fabricator during the design stage, though it must be recognised that individual fabricators do have particular preferences, arising from their experience and workshop facilities.

5 DETAILED DESIGN

The detailed design stage confirms or refines the outline design produced in the initial design stage. It is essentially a checking process, applying a complete range of loading conditions to a mathematical model to generate calculated forces and stresses at critical locations in the structure. These forces and stresses are then checked to see that they comply with the 'good practice' expressed in the Code. The detail of the checking process is sufficiently thorough to enable working drawings to be prepared, in conjunction with a specification for workmanship and materials, and for the bridge to be constructed.

5.1 Global analysis

A global analysis is required to establish the maximum forces and moments at the critical parts of the bridge, under the variety of possible loading conditions. Local analysis of the deck slab is usually treated separately from the global analysis; this is described further in Section 5.8.

For proper and efficient evaluation of bending and torsion effects it is necessary to use computer analysis. Programs are available over a wide range of sophistication and capability; the selection will usually depend on the designer's in-house computing facilities. However, for global analysis of what is fundamentally a simple structure, quite simple programs will usually suffice.

For the type of box girders considered in this guide, there will normally be sufficient intermediate cross-frames or diaphragms to restrain distortional effects and to ensure that simple global analysis will be adequate. In the event of needing to investigate box girders which are provided with very little distortional restraint, more detailed analysis may be needed, perhaps even involving the use of finite element programs.

5.1.1 Computer models

The basis of most commonly used computer models for I-beam and slab bridges is the grillage analogy, as described by West[17] and Hambly[10]. In this analogy the structure is idealised as a number of longitudinal and transverse beam elements in a single plane, rigidly interconnected at nodes. Transverse beams may be orthogonal or skewed with respect to the longitudinal beams.

The analogy is also applicable, with appropriate modification, to box girder bridges, provided that distortional effects are not significant (this is discussed further in Section 5.1.2).

The global structural action of a composite bridge deck can be seen as the essentially separate actions of a reinforced concrete slab which bends transversely and a series of longitudinal beams which deflect vertically and twist. The slab bends as a result of being supported along several lines which deflect by different amounts and in a manner which varies along the span. The global analysis therefore needs to model accurately the way in which these support lines deflect, so that the interaction between longitudinal and transverse bending is properly established.

The slab is effectively supported along each web line. The vertical deflection of each web line depends on a combination of the vertical and torsional deflections of the box girder of which it is a part. The best way to model these effects is to create a torsionally stiff beam element along the centreline of each box (i.e. the shear centre) and to connect it to the slab at the web positions. To do this, short 'dummy' transverse beams are needed; they do not physically represent any particular part of the structure and the forces in them do not need to be analyzed, but they must be given sufficient stiffness that their bending is insignificantly small. This form of model for a twin-box bridge with cantilevers is illustrated in Figure 5.1 (note that, for clarity, the dummy beams and longitudinal beams are shown slightly below the slab, whilst they would actually be treated in the analysis as co-planar).

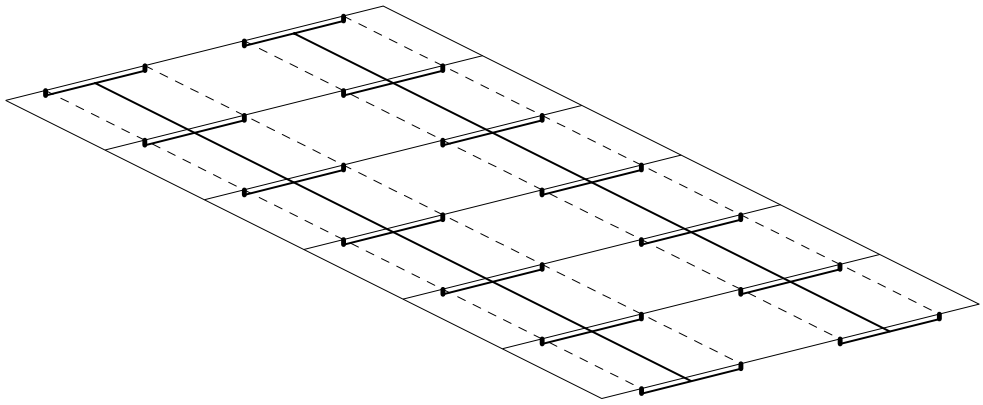

Figure 5.1 *Grillage model for twin-box bridge with cantilevers*

The main longitudinal beam elements represent the composite section (main girder with associated slab). The bending stiffness should be calculated in the usual manner and properties for cracked sections used adjacent to intermediate supports. The torsional stiffness should be calculated assuming uncracked concrete, although for open top boxes consideration should be given to the effect of cracking in hogging moment regions (see Clause 5*/7.6).

The longitudinal elements representing the slab (shown dotted) are not strictly necessary, as they are much more flexible than the main girders, though they may be helpful in the application of distributed loads. They are shown here to illustrate the division of the slab.

The longitudinal edge elements may be added to represent the edge beam. They do not have a major effect on overall performance but are often helpful in the application of load on the cantilevers.

Each transverse element simply represents a width of slab (equal to the spacing of the transverse elements). The stiffness of reinforced slab should be of a section which is uncracked. The same stiffness may be used over the width of the box, even if the steel section is closed and the concrete is cast on the top flange. Transverse elements over cross-beam and diaphragms should represent the stiffness of the effective composite transverse member.

The slab elements are supported only on the dummy elements; they are not connected directly to the longitudinal beams. There is no moment continuity between slab elements and the dummy beams.

The increasing availability of sophisticated analytical software may lead to wider use of the more complex models, though at present the use of simple grillages is usually accepted as perfectly adequate and usually yields results which are easily interpreted.

5.1.2 The effects of distortion

In Section 3.4 it was explained how vertical loads applied eccentrically to the shear centre of the box can lead to distortion of the cross section. Figure 3.6 showed the change in shape as a result of distortional effects. Whilst distortion is not the same as twist, the effect of distortional displacement of the box is to increase the apparent twist of the slab supported on the box, because one web deflects upward and the other downward. If there were no restraint to distortion in the span length of the section, the effect on the global behaviour would be similar to a reduction in torsional stiffness, except that the amount of reduction depends on the distribution of the loading (whether uniformly distributed or point loads) not just on section dimensions.

If there were no intermediate distortional restraints in box girders of this span range, reduced apparent stiffness would lead to significant distortional deflections and would have a marked effect on the interaction between girders in the global analysis. However, a few intermediate restraints, even ones not deemed to be fully effective by Part 3 (see discussion in Section 5.3.5), lead to substantial reductions in the distortional deflections. As a simple guide, restraints at a spacing of about three times the depth of the girder will usually limit the reduction in effective stiffness to a level which can be neglected in global analysis.

Distortion of open sections during construction also needs to be considered carefully. The open section is torsionally very weak and the deflection under the weight of the wet concrete should be checked to ensure that the correct geometry is achieved on completion. In staged construction, the forming of the slab at discrete locations introduces torsional warping restraint; the deflection of the open section between such restraints should take account of that restraint (though the calculation of stresses at ULS need not include torsional warping stresses, as explained in Section 5.3.5).

5.1.3 Model mesh size

Clearly, the spacing and width of the main longitudinal beams control the transverse node spacing. Note that no intermediate nodes are required in the slab between the webs of adjacent boxes, otherwise local bending effects will be partly included in the global effects and there will be double counting.

The spacing of transverse beams (representing the slab) should not exceed about 1/8 of the span. Uniform node spacing should be chosen where possible. It would be convenient for considering distortional effects to arrange node spacing to coincide roughly with the spacing of intermediate cross-frames.

For skew spans, the transverse beams should be parallel to the transverse reinforcement - usually parallel to the abutments for small skew angles (less than 20°).

Section properties for longitudinal beams must be calculated for the bare steel girders (for the construction condition) and for composite girders with a fully effective deck slab. Many designers consider it adequate to use only short-term

concrete properties for the analysis, rather than deal with two sets of composite properties. This results in a very slightly higher design moment in cracked sections over supports and correspondingly lower midspan moments. Long term and short term load effects should nevertheless be determined separately, since they will be applied separately to long and short term section properties in the stress analysis of sections.

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 equally divided 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.

Section properties for transverse beam elements representing transverse bracing or cross girders should be determined on the basis of both the bending stiffness and the shear stiffness of the members acting with the deck slab.

5.1.4 Analysis of dead load for staged construction

It is usual for the deck slab to be concreted in stages and for the steel girders to be unpropped between supports during this process. Part of the load is thus carried by the steel beam sections alone, part by the composite sections. A number of separate analyses are therefore required, each representing a different stage. Typically there are about twice as many stages as spans, since concrete is usually placed alternately in midspan regions and over supports. Where the cantilevers are concreted at a different stage from the main width of slab, this must be taken into account in the analyses.

When an open box is concreted in stages, there will inevitably be stages when parts of the beams are closed sections and parts are open sections. In such staged construction concrete may well be placed over the supports before the span regions, to develop an effective top flange at an early stage. As mentioned above, distortional effects should be considered carefully when concreting open sections in stages.

5.2 Load effects and combinations

The loadings to be applied to the bridge are all specified in Part 2*, except for the standard fatigue vehicle, which is specified in Part 10. Table 1 of Part 2* specifies the appropriate partial factors to be applied to each of these loads, according to the combinations in which loadings occur.

Because many different load factors and combinations are involved in the assessment of design loads at several principal sections, it is usual for each load to be analyzed separately and without load factors. Combination of appropriate factored loadcases is then either performed manually (usually by presentation in tabular form) or, if the program allows, as a separate presentation of combined factored forces. Since so many separate loadcases and factors are used to build up total figures, the designer is advised to include routine checks (such as totalling reactions) and to use tabular presentation of results to avoid errors. The graphical displays and printouts now available through analysis and spreadsheet software can also be recommended for checking results.

The object of the analysis is to arrive at design load effects for the various elements of the structure. The most severe selection of loadings and combinations needs to be determined for each critical element. The main design load effects which are to be calculated include the following:

- Maximum moment with co-existent shear and torsion in the most heavily loaded main girder: at midspan, over intermediate supports, and at splice positions.
- Maximum shear with co-existent torque and moment in the most heavily loaded main girder: at supports, and at splices.
- Maximum torque with co-existent shear and moment in the most heavily loaded main girder: at supports, and at splices.
- Maximum distortional torques in main beams
- Maximum forces in transverse bracing at supports (and in intermediate bracing if it is participating).
- Maximum and minimum reactions at bearings.
- Transverse slab moments (to be combined with local slab moments for design of slab reinforcement).
- Range of forces and moments due to fatigue loading (for shear connectors and any other welded details which need to be checked).

In addition, displacements and rotations at bearings will need to be calculated.

The total deflections under dead and superimposed loads should be calculated, using long-term concrete properties, so that the designer can indicate on his drawings the precamber for dead load deflections.

Selection of the girder most heavily loaded in bending and shear can usually be made by inspection, as can the selection of the more heavily loaded of intermediate supports. Influence lines can be used to identify appropriate loaded lengths for the maximum effects (see Clause 2*/6.2.1 and Figure 11 of Part 2*). If cross sections vary within spans, or spans are unequal, more cases will need to be analyzed to determine load effects at the points of change or at critical points in each span.

Selection of the loading to give maximum torsional effects usually requires more detailed consideration. Worst torsion may well occur when bending and shear effects are modest. Worst torsion may sometimes occur on the girder which is not directly loaded.

Distortional effects depend on the increment of torque applied to a section between effective restraints. Where there are intermediate frames, the choice of grillage nodes at roughly the same spacing as the frames will help to determine the appropriate effects.

It is usually found that the specified Combination 1 (see Clause 2*/4.4.1) governs most or all of the structure. Some parts, notably top flanges, are governed by construction conditions, Combination 2 or 3. For spans over about 50 m, Combination 2, including wind load, may determine design of transverse bracing and bearing restraint.

In the preparation of a bridge bearing schedule (Clause 9.1/A.1) it should be made clear that the tabulated forces are load effects, i.e. they include both λ_{fL} and λ_{f3}.

The effects of differential temperature and shrinkage modified by creep are calculated in two parts. The first is an internal stress distribution, assuming that the beam is free to adopt any curvature that this produces (primary effects). The second is a set of moments and shears necessary to achieve continuity over a number of fixed supports. These moments and shears give rise to further longitudinal and shear stresses (secondary effects). Part 3 deals separately with these primary and secondary stresses when considering bending/shear interaction for beams without longitudinal stiffeners (Clause 3/9.9.7, modified by Amendment 2); it omits primary effects at ULS, since these may be relieved by redistribution locally. No omission of primary effects is mentioned for longitudinally stiffened beams, but where they are in the opposite sense to secondary effects it would be prudent to neglect them.

Partial factors λ_{fL} are given for temperature effects in Part 2*. For shrinkage, values of λ_{fL} are given in Clause 5*/4.1.2.

5.3 Design of beams

5.3.1 General

The main longitudinal beams must be designed to provide adequate strength in bending and shear to resist the combined effects of global bending, local effects (such as direct wheel loading or compression over bearings) and structural participation with any bracing system. Torsional effects, calculated in the global analysis, are taken into account as additional shear stresses. Distortional effects must also be included.

In Part 3 there are three principal mechanisms for the determination of bending strength: as a compact section, as an unstiffened non-compact section and as a longitudinally stiffened non-compact section.

Compact sections might occasionally be found in single span box girders, where the major part of the steel in the composite section is in tension. Design of such beams could generally follow the sequence given below for non-compact unstiffened sections, except that the ULS bending resistance could be based on the plastic resistance. Design as a compact section is not explicitly covered in this guide.

The strength of beams of non-compact section, both stiffened and unstiffened, is generally evaluated in the code by reference to a limiting compressive stress which depends on the lateral buckling of slender flanges. With completed box girders, the section is almost always stable in terms of lateral or lateral-torsional buckling; different considerations therefore apply. However, the flange breadth to thickness ratios are often sufficiently high that they are less than fully effective in compression; this must be allowed for. Limiting compressive stresses do need to be determined for the top flanges of open top boxes in midspan during construction; in such cases it is easier to consider the flanges as compression struts between effective cross-frames and, for trapezoidal boxes, noting that the flange buckles normal to the plane of the web, not in its own plane.

All sections must satisfy the requirements for strength at the Ultimate Limit State (ULS) (Clause 3/9.2.1). At the ULS, the effects of shear lag and restraint of torsional warping may be neglected. The code also states that the effects of restraint of distortional warping may be neglected, but, as mentioned in Section 3.4 this appears to be an unconservative simplification. It is likely that this clause will be amended; it is therefore prudent to include the distortional warping effects. Having said that, it should be noted that for box girders suitable for the present span range, provided there are sufficient intermediate diaphragms, bracing or cross-frames, the magnitude of the distortional stresses and deflections should be small.

The strength of non-compact sections needs to be checked at the Serviceability Limit State (SLS), according to Clause 3/9.2.3, only when:

(a) Shear lag is significant (see Clauses 3/9.2.3 and 5*/5.2.3). A high level of shear lag is unlikely in the present form of construction, where the boxes are relatively narrow in relation to the span.

or

(b) Tension flange stresses have been redistributed at ULS in accordance with Clause 3/9.5.5. This redistribution is not usually employed in this form of construction.

It must be noted that Part 10 (Clauses 6.1.4 and 6.1.5) requires that, in designing for fatigue, all stresses, including warping stresses, be calculated elastically and allowing for the effects of shear lag. Shear lag in boxes should be calculated in accordance with Clause 5*/7.3.

The following Sections deal separately with the evaluation of stiffened and unstiffened sections.

5.3.2 Beams without longitudinal stiffeners

Unstiffened box sections might be used for:

- a simply supported span
- a trapezoidal open steel section where the bottom flange is relatively narrow
- narrow rectangular boxes.

Clause 3/9.9 provides rules for calculating the bending and shear resistances of beams without longitudinal stiffeners in the section (and with parallel flanges). Resistances are expressed as moments and shear forces, rather than as stresses. An interaction limiting envelope is defined for combined effects of bending and shear.

No reference is made within these sub-clauses to the effects of torsion and the consequence that the shear in the two webs will not be equal, nor to distortional warping stresses. It would appear reasonable to overcome this omission by considering the two halves of the box separately. Then, for the interaction between moment and shear, the shear on the more heavily loaded web (including the effects of torsion) is compared with the shear resistance of the half box, and the moment on the half box (including an effective moment due to warping stresses) is compared with the moment resistance of the half box.

(Alternatively, if warping stresses at the section are negligible, it may be more convenient to express the moment effect and resistance as those of the whole box: the relationship is clearly the same.)

The bending and shear resistances are calculated as follows:

Bending Resistance

The bending resistance M_D of a beam at ULS is determined from the limiting moment of resistance M_R. The value of M_R depends on the resistance of the cross section, which is calculated from the effective section properties (no allowance for shear lag but reduced for holes, Clause 3/9.4.2.2; for slender webs, Clause 3/9.4.2; and for wide compression flanges, Clause 3/9.4.2.4).

The ratio M_R/M_{ult} depends on the lateral torsional buckling slenderness of the beam and generally for slender I-beams is a value less than unity. A composite box section is normally stable against LTB and so there is no reduction. The slenderness of a bare steel box bending about its major axis can be calculated according to Clause 3/9.7.3.1, but unless it is an extremely slender long box, there will again be no reduction.

The cross section resistance M_{ult} is either the elastic or plastic moment capacity of the cross section, depending on whether the section is compact or non-compact. If compact resistance is used, an additional check against yield is required at SLS.

The top flange of an open box is restrained during construction by cross-frames at discrete positions and, unless the box is very narrow (which is unlikely for an open top box), these form fully effective intermediate restraints. However, the determination of slenderness using Clause 3/9.7.2, which involves the u parameter, is not appropriate for this configuration. It is better to treat the top flange as a compression member in a truss and to use Clauses 3/12.4.1 and 3/10.6 to check compression stress in the top flange and to check tensile stress in the bottom flange against (factored) yield stress. If the open box has inclined webs, buckling will be normal to the web, not in the plane of the flange.

The flange on an open trapezoidal box will also be bent in plan, since the weight of wet concrete will be supported by an inclined force in the plane of the web, and the consequent stresses should be taken into account.

Shear Resistance

The design shear resistance of the beam V_D is given by Clause 3/9.9.2.2. An unstiffened slender web is unable to develop full shear yield resistance because its resistance is limited by buckling. However, when the web is provided with vertical stiffeners the buckling resistance is increased and some of the shear load is carried by tension field action.

The magnitude of the tension field component is enhanced where the flanges are stiff and plastic hinges develop in them (see the m_{fw} parameter). This enhancement is applicable to a box section, but since m_{fw} depends on the lesser outstand from the web in the smaller flange it normally has a very small value and there is little benefit.

The reduced shear resistance V_R without contribution to tension field action from the flanges (i.e. $m_{fw} = 0$), is given by Clause 3/9.9.3.1, for use in the bending-shear interaction relationship.

Intermediate web stiffeners are an effective means of increasing the limiting shear stress when the web depth/thickness is in excess of about 75. Typically, intermediate web stiffeners are provided at a spacing of between 1.0*d* and 2.0*d*.

Bending-Shear Interaction

Under combined bending and shear, a bending resistance M_f, equal to that provided by the flanges alone, ignoring any contribution from the web , can be carried at the same time as the shear resistance V_R if contribution from the flanges is ignored. Further, it has also been shown that the full bending resistance M_D can be developed if the shear is not more than half of V_R; similarly, full shear resistance V_D can be developed if the bending is not more than half of M_f. These limits to the interaction are expressed in Clause 3/9.9.3.1.

Interaction is checked for the worst moment and worst shear anywhere within the panel length (between transverse web stiffeners), rather than both effects at a single section.

Multi-Stage Construction

The stresses through a section which is constructed in stages have to be determined by summation of several stress distributions. The resulting stress variation is therefore non-linear and discontinuous, and could not be derived from any single application of loads to one section.

The requirement for adequate resistance in bending in those cases is expressed simply in terms of limiting stresses at extreme fibres (Clause 3/9.9.5). In the consideration of combined bending and shear, an equivalent bending moment must be derived for use in the interaction formulae. This is obtained by multiplying the total stress at an extreme fibre in the section by the modulus (for that fibre) which is appropriate to the stage of construction being checked (see Clause 3/9.9.5.3).

Stresses during construction may well govern the design of some elements, such as the top flange. The designer should consider at least one feasible construction sequence and ensure that the strengths are adequate at critical locations at every stage of construction.

5.3.3 Beams with longitudinal stiffeners

Beams with longitudinal stiffeners in the section (on the web, flange or both) and beams with non-parallel flanges must be subject to a more detailed check, on an element by element basis (Clauses 3/9.10 and 3/9.11).

As for unstiffened beams, resistances at all sections must satisfy the requirements of the ULS. Requirements for SLS need only be satisfied in the deck slab (according to Part 5) and in the special circumstances mentioned in Section 5.3.1.

Calculation of all stresses is based on the effective sections determined in accordance with Clause 3/9.4.2. Distortional warping stresses should be included, where appropriate.

For compression flanges, the effective area of the flange is calculated in accordance with Clause 3/9.4.2.4. If the stiffener spacing (*b/t*) is greater than

24 the effective area is less than the gross area. For spacings over 30 the reduction quickly becomes very significant.

In a stiffened beam, the web can be taken to contribute fully to bending resistance (see Clause 3/9.4.2.5.2), but it may not be economic to provide longitudinal web stiffeners to ensure that the resistance in compressive regions meets the consequent combined load effects. In that case the web may be taken to contribute only partially, by shedding up to 60% of its load into the flanges. This is achieved by using only a proportion of the web thickness in calculating the effective section.

Once the stresses have been calculated throughout the section, checks are required on the flanges and the webs. For each, there are checks on the plate and on the stiffeners.

For the flanges, the checks are set out separately for unstiffened and stiffened flanges.

Unstiffened flanges

The requirements for bending resistance are expressed in terms of limiting stresses in the extreme fibres of the flanges. According to Clause 3/9.10.1, the stresses calculated on the effective section must not exceed the limiting values, in compression and in tension. Again it must be pointed out that a box section is normally stable and the resistance in compression does not need to be reduced on account of LTB slenderness, though during construction the stresses in the top flange of an open steel box may be limited on account of lateral buckling (see Section 5.3.2).

Stiffened flanges

Although it is not explicitly stated, Clause 3/9.10.2 presumes that a stiffened flange is part of a beam which is stable against lateral torsional buckling. The clause therefore takes account of the stability of the effective stiffener sections, but makes no reduction for lateral buckling of the flange as a whole.

The flange plate is considered to carry longitudinal and shear stresses. The effects of longitudinal stress at mid-flange (not the extreme fibre) are combined with an average value of shear stress across the width of the flange to give a 'von Mises effective stress', and this must be less than yield stress (Clause 3/9.10.2.1).

The effective stiffener section, a portion of flange plus a stiffener, is checked for stresses at its centroid (Clause 3/9.10.2.3), considering it as a compression strut spanning between transverse stiffeners or cross-frames. The limiting stress is reduced below yield on account of slenderness and the vertical stress gradient. Typically, the requirements on stiffener strength effectively limit the maximum stress at the extreme fibres to about 10 - 20% below yield stress, but because the stress varies along the flange, the check only has to be made at 0.4 of the distance from the higher stressed end, and this reduction is usually acceptable there.

Longitudinal stiffeners can be used without transverse stiffeners, but then an effective length must be calculated in accordance with Clause 3/9.10.4.

Flats or angles are normally used to stiffen flanges; their proportions must comply with Clause 3/9.3.4. Note that if the b/t of the flange exceeds 30, then

angles cannot be used because their height/thickness ratio exceeds the limits in Clause 3/9.3.4.1.4.

Webs in longitudinally stiffened beams are required to be checked panel by panel for yield and buckling. The code does not invoke the post buckling tension field action which is allowed in unstiffened beams. This Code provision has been shown experimentally to be conservative.

Resistance of web panels

Each web panel (which is bounded by vertical stiffeners and by longitudinal stiffeners or the flanges) has to be sufficiently strong to carry the direct and shear stresses on it, as determined from the section analysis and allowing for any redistribution. (If, for example $0.7t_w$ has been assumed in calculating the effective section for stress analysis, use 70% of the stresses based on that section.) The checks for yield allow a degree of plasticity under combined bending and shear stresses by considering only 77% of the bending stress component in determining the equivalent stress (Clause 3/9.11.3). The checks for buckling consider direct stresses in both directions in combination with shear stresses (Clause 9.11.4).

The web stiffeners (vertical and longitudinal) which divide the web into panels must in turn be adequate to resist buckling under the forces they carry. Vertical stiffeners are checked over the full height of the web; longitudinal stiffeners are checked over their length between vertical stiffeners.

5.3.4 Intermediate web stiffeners

Vertical web stiffeners

Vertical web stiffeners improve the shear resistance of a slender web. They also act as intermediate restraints for any longitudinal stiffeners. Some stiffeners form part of cross frames which restrain the box against distortion.

Intermediate web stiffeners should be designed at ULS to resist a number of axial forces, in accordance with Clause 3/9.13:

- axial force due to tension field action
- direct loading (from a wheel at the stiffener position)
- bending about a longitudinal axis due to eccentricity of axial forces (in the plane of the web) relative to the centroid of the stiffener section
- destabilising influence of the web (buckling check only)
- forces and moments due to action with a cross-frame system
- forces due to change of slope of the bottom flange.

Stiffeners must be checked for strength against yield (Clauses 3/9.13.5.1 and 3/9.13.5.2) and against buckling out of the plane of the web (Clause 3/9.13.5.3). The stiffeners are often simply flats, though Tees are sometimes used for greater stiffness and strength

Intermediate stiffeners should be connected to the top flange, to avoid fatigue problems. Their presence increases the distortional stiffness of the section (because the webs are stiffer against out-of-plane bending) and this can give rise to local bending at the top of the web unless a proper connection is made.

Intermediate stiffeners are frequently not connected to the bottom flange. The code specifies a maximum clearance of five times the web thickness, but it is suggested that a clearance of not more than three times should be aimed for.

When intermediate stiffeners are notched to allow longitudinal stiffeners to pass, the two should be welded together over at least one third of the perimeter of the cut-out. The reduced section of the vertical stiffener should be adequate to carry the load in the stiffener at that point.

Longitudinal web stiffeners

Longitudinal stiffeners are normally continuous through intermediate vertical stiffeners, which are notched to allow them to pass through. Stiffeners which are continuous contribute to the bending resistance (i.e. to the effective section in bending). The design stress for the stiffener should be taken at a distance of 0.4 of its length (between vertical stiffeners) from the more highly stressed end, in accordance with Clause 3/9.5.2. To be consistent with a linear strain distribution, the stress should be calculated on the reduced effective section in bending if load has been shed from web panels in accordance with Clause 3/9.5.4, but no load should then be shed from the stiffener.

5.3.5 Distortional effects

In box girder bridges, torsion is applied to the top flanges of box sections along their length, not just at positions where there are diaphragms. The distortional effects are therefore an essential part of the load-carrying system for the box; both warping and transverse distortional bending effects need to be taken into account. Plastic redistribution does not relieve the necessity of resisting the distortional effects and they should not be neglected at ULS (although Clause 3/9.2.1.3 allows distortional effects to be neglected if the torque (i.e. the effect of off-centre loading) is applied only at cross frames).

Calculation of distortional effects is a complex task, requiring the evaluation of many factors. Fortunately, Part 3 provides simplified rules which cover common configurations of box girders. The Code rules (Part 3, Annex B) are based on the BEF analogy and give expressions for the calculation of distortional warping and distortional bending effects between restraints which are fully effective (i.e. as good as rigid). Rules are given for determining when such restraints are sufficiently stiff and sufficiently strong to be considered as fully effective.

The restriction to fully effective restraints appears to have been introduced to avoid the greater complexities of dealing with flexible restraints. However, this limitation can be over-conservative, because it demands that intermediate restraints are almost rigid, whereas even flexible supports can give substantial relief to distortional stresses (as discussed in Section 3.4 and illustrated in Figure 3.7).

The rules in Annex B require the calculation of the parameter K, which is a measure of the distortional stiffness of the box section (i.e. it depends on section geometry and transverse bending stiffness of the webs and flanges). Warping stiffness is taken to be proportional to the vertical bending inertia I_x (a simplification which has been validated by parametric studies). The interaction between the two is expressed in terms of a parameter β, which reflects the rate at which distortional effects disperse away from the point of application and which typically has a value between 0.1 m^{-1} and 0.5 m^{-1}. The

non-dimensional product βL_D, where L_D is the distance between effective restraints, is a convenient measure of the influence of the restraints on the warping and distortional stresses.

It has been shown by the BEF analogy that where a distortional torque (i.e. a couple of vertical forces applied at the top corners of the box) is applied midway between effective diaphragms or cross-frames, warping stresses do not increase with L_D once the value of βL_D exceeds 1.0; distortional stresses do not increase once the value exceeds 2.0. Corresponding values for udl torque are 1.6 and 2.65. Closer spacing of the diaphragms will reduce the warping and distortional stresses. Expressions for calculating the distortional warping stresses due to point and distributed loads between effective diaphragms are given in Clauses B.3.3 and B.3.4.

If a cross-frame does not comply with the limits, i.e. it is not 'effective', the code makes no provision for taking its restraining effect into account. This is conservative, particularly for distortional bending. If a designer does need to take such a frame into account, reference will have to be made to more detailed texts (e.g. references [7], [8] and [10]) or a more complex finite element analysis undertaken. Alternatively, the stiffness of flexible frames can be 'smeared' along the box (i.e. the box is treated as without cross-frames but with the distortional stiffness increased by the stiffness of one frame divided by the frame spacing), though this is not mentioned in the Code. (See further comment on effective cross-frames in Section 5.4.4.)

In calculating distortional stresses according to Annex B of Part 3, it must be understood that the distortional forces to be applied in calculating stresses are only those forces between a pair of effective diaphragms; the reference in the code clauses to applied torque is only to the torque applied at top flange level between the two diaphragms, not the total torque carried by the box at any given section.

5.4 Diaphragms and cross-frames

5.4.1 Diaphragms at supports

At supports, forces are transferred from the box girder, through bearings, to the substructure below. Principally, these forces are vertical, though lateral restraint also has to be provided at certain selected positions. Where there is only a single bearing under the box and it offers little resistance to transverse rotation (e.g. elastomeric pot bearings), there will be no torsional restraint; the loads transferred from the two webs will be equal (presuming that the bearing is on the centreline). When there are two bearings, under or close to each of the webs, torsional restraint is provided to the box; the load from each web will be different, and there will be a transfer of torsional shear from the flanges. Whenever there is lateral restraint there will be an associated torque, because the restraint will not be at the level of the shear centre of the box.

The principal function of a support diaphragm is to provide an adequate load path to transfer shear forces from the webs to the bearings below the box. In doing so it also maintains the cross section of the box against distortional forces.

Plated diaphragms are normally provided at supports, since they provide these functions most easily, although, strictly, an adequately braced cross-frame could

also do so. In Part 3, only plated diaphragms are considered at supports; the rules for cross-frames relate only to their use at intermediate positions.

Clearly, full diaphragms close the box section, yet access into the box is necessary for completion of fabrication and for future inspection and maintenance. Openings are usually provided to permit access along the box, but the effect of these openings on the performance of the diaphragm has to be carefully considered; the size and position of any opening needs to be limited. This can be a particular problem with small boxes, because the minimum hole size may be a large proportion of the diaphragm size.

The design of diaphragms is covered by Clause 9.17 of Part 3. The first part of that clause relates to general limits on configuration and is accompanied by illustrations of the notation used and position limits for openings (Figures 31 to 33 of Part 3). These Figures reveal that the rules were developed primarily for larger box girders, suitable for very long spans. As a result, use of the rules for more modest box girders can sometimes face difficulties because of inappropriate arbitrary limits or presumptions; in such cases the alternative is to resort to a more detailed evaluation of fundamental behaviour, though this is likely to be time-consuming.

Diaphragms are usually provided with vertical stiffeners above the bearings because of the large forces involved, though with small boxes a thick unstiffened diaphragm may on occasion be appropriate. Rules for both types of diaphragm are provided.

The rules for diaphragms essentially treat each diaphragm as a portion of a beam, with the diaphragm plate acting as its web and an effective width of each of the box flanges acting as its top and bottom flange. Load is applied at the junction between diaphragm and box web (with a T-shaped effective section performing as a web stiffener) and is resisted at the bearings. Under the applied load the effective diaphragm section carries the loads in bending and shear.

The resistance of the diaphragm is determined differently for unstiffened and stiffened arrangements:

Unstiffened diaphragms

The design of unstiffened diaphragms is covered by Clause 9.17.5. The diaphragm is treated as a plate panel subject to combined axial stresses (in two directions), bending stress and shear stress.

Buckling is checked by determining a combined coefficient which takes account of the panel geometry and bearing disposition. (Note that both single and twin bearing situations are covered, though Figure 33 might mislead the designer into presuming that only single bearing situations are covered.) The buckling resistance, in terms of an effective vertical load, is then calculated from this coefficient (Clause 3/9.17.5.5).

A vertical reference stress and the combined horizontal and shear reference stresses are each checked against yield, or a reduced value taking account of the proportion of buckling resistance which has been utilised (Clause 3/9.17.5.4).

Stiffened diaphragms

The design of stiffened diaphragms is covered by Clause 9.17.6. The vertical bearing loads are carried by the effective stiffener sections; the horizontal and shear stresses are carried by the effective diaphragm section.

Plate panels and stiffener sections are checked separately.

Stresses in the plate panels are calculated in accordance with Clause 3/9.17.6.2, which takes account of the influence of boundary shears on horizontal stresses. The equivalent stress in any plate panel must be checked against yield (Clauses 3/9.17.6.4). Plate panels must also be checked against buckling, except that for relatively simple rectangular box configurations, no check is needed (Clause 3/9.17.6.5.1).

The effective sections of all stiffeners are checked for axial stress (Clause 3/9.17.6.6) and for buckling out of the plane of the diaphragm (Clause 3/9.17.6.7). The bearing stress at the bottom of bearing stiffeners must comply with the limit in Clause 3/9.14.4.2.

At the junction between box web and diaphragm, the effective T-shaped section is determined in accordance with Clause 3/9.17.4.5. This section is checked in a similar manner to an ordinary web stiffener at a support (Clause 3/9.17.7.2). No benefit of any restraint from the diaphragm (in its plane) is assumed, unless the diaphragm happens to have full width horizontal stiffeners (which is not likely for relatively small boxes).

5.4.2 Bearings under diaphragms

The setting of bearings relative to the box girder can have a significant effect on the stresses in the diaphragm and needs to be considered carefully.

Any error in setting the bearing transversely to the box (i.e. parallel to the plane of the diaphragm) will have only minor effect on stresses and can usually be neglected. Transverse movements, on unguided bearings, are also likely to be minor and usually may be neglected.

Any errors in setting the bearing longitudinally (i.e. normal to the plane of the diaphragm) and any eccentricities of the reaction due to movement are much more significant. If the diaphragm is unstiffened, there is very little resistance to bending out of the plane of the diaphragm; indeed Part 3 requires that bearings be symmetrically positioned below such a diaphragm. Small moments from errors in setting will have to be resisted by local bending of the flange or, preferably, by internal longitudinal stiffeners.

Even with stiffened diaphragms it is usually only practical to provide for modest longitudinal eccentricity of the reaction relative to the centreline of the diaphragm. The consequence is that where relative movement has to be accommodated between super- and sub-structures, the fixed part of the bearing should be attached to the box, with the sliding surface on the sub-structure. Clearly this is not ideal for the bearing, since debris can collect on the sliding surface, but it is normally quite feasible to design protective screens, possibly using flexible membranes (e.g. neoprene), to exclude such material. Alternatively the box can be fixed longitudinally at each support and the support designed to deflect under the imposed displacements.

When there are two bearings under a single box, any error in relative level between the two will cause twisting of the box and, as the box is torsionally stiff, may give rise to significant forces. In composite construction, the dead load torque on a pair of bearings will depend on the concreting sequence of the slab. The designer should make allowance for both of these aspects and take account of the tolerances specified in Part 6. Any limitations to the construction procedures, special procedures required to equalise bearing loads or extra requirements in respect of fabrication tolerances should be set out clearly in the contract specification.

5.4.3 Access holes in diaphragms

Access is usually required through diaphragms, during construction and during service. The absolute minimum requirement is for a hole 457 mm diameter, but it is generally recognised that larger holes are normally necessary. However, the cross section of a typical box is not unduly large; holes need to be carefully positioned and framed to permit the diaphragm to function structurally. A manhole 600 mm high is usually considered the minimum in a vertical plate and a hole 600 mm x 600 mm should be provided if possible. Tight corner radii should be avoided; if they are necessary, stress concentration effects must be carefully considered.

Access holes should be positioned, as far as possible, for ease of use. If the hole is positioned part way up a deep diaphragm, step-irons or a ladder must be provided. A grab-rail above the manhole will assist passage through it.

Access into the box (for maintenance) should be either through the end diaphragm or through a hole in the bottom flange or web (choose an inner web). Access through the deck should be avoided, as it is very difficult to achieve permanently watertight seals; drainage into the box from the roadway will inevitably result.

5.4.4 Intermediate cross-frames

Intermediate restraint to the cross section may be needed to control distortion. Such restraint can be provided by ring cross-frames, by braced cross-frames, or by a plated diaphragm.

Rules for the design of ring and braced cross-frames are given in Clauses 3/9.16. The design load effects to be considered are those due to the restraint of distortion, plus the usual forces specified for the design of intermediate web stiffeners (Clause 3/9.13), plus any transverse loads acting directly on the top flange.

Part 3 does not cover the use of plated diaphragms for intermediate restraint (see the note to Clause 3/9.16.1). If the designer wishes to use a plated diaphragm, with a central access hole, it can be designed as a ring cross-frame.

As explained in Section 5.3.5, intermediate diaphragms or cross-frames reduce the values of distortional warping and bending stresses. Cross-frames may also be needed to control distortional deflections, even though the stresses might be acceptable without them. Some form of cross-frame is likely to be provided at relatively close spacing to brace an open steel box during construction.

To be considered effective against distortion, cross-frames must comply with the stiffness and strength requirements of Clause B.3.2. In Clause B.3.2 the

stiffness is expressed in terms of a minimum value for the dimensionless parameter *S*. This parameter is in effect a ratio between the distortional stiffness of the box and the distortional stiffness of the cross-frame. Unfortunately, the closer the spacing of the cross-frames, the stiffer they must be, to be considered as effective according to the rules in Annex B. Frames introduced for reasons other than to limit the distortional stresses may not meet the minimum value of an 'effective' diaphragm. In that case the designer could consider the effectiveness of every second (or even every third) frame in evaluating distortional effects: the stiffness required to suit a panel length L_D which is then double or treble the frame spacing will be proportionately less. It may be that the distortional stresses calculated neglecting all intermediate frames are acceptable, in which case none of the frames need meet the stiffness requirements.

The strength of all cross-frames should be adequate to carry the loads specified by Clause B.3.4.3.2, even if the frames do not meet the stiffness requirements. Although they are flexible, they will still experience a substantial proportion of the load that a stiff frame would carry when the load is at the position of the frame. Designing flexible frames for the full load effects on a stiff frame is slightly conservative, but not unduly so.

5.5 Bracing between main beams

Bracing between main beams may be needed for load distribution purposes, either at supports or at selected intermediate positions. Design load effects should then be determined from the global analysis. Such bracing normally takes the form of a cross-beam acting compositely with the slab. Detailed design of the cross-beam should follow the normal rules in Part 3; the beams will normally be of non-compact cross section.

Any cross-beams will be joined to the main box girder beams on site. The box girders will be provided with an intermediate diaphragm or cross-frame at that position; a stub should be provided on the outer face, to which the cross-beam can be spliced.

When each box is supported on a single bearing at intermediate supports, temporary cross-bracing may be needed for torsional restraint during construction.

5.6 Shear connection

Shear connectors are required on the top flange for composite action, to provide the necessary shear transfer between the steel girder and the concrete slab. The shear flow varies along the length of the beam, being highest near the supports. For economy, it is customary to vary the number and spacing of connectors to provide just sufficient shear resistance. The most commonly used form of connector is the headed stud.

Shear connectors must be designed to provide static strength and for fatigue loading. With non-compact sections, the required resistance at SLS generally governs the design for static strength. Shear flows should be calculated at supports, at midspan and at least one position in between, i.e. quarter points. The ULS need only be considered when there is uplift or redistribution of

tension flange stresses (Clause 5*/6.3.4). Fatigue may well govern the spacing of connectors in midspan regions.

The nominal static strength of shear connectors is given in Part 5*, Table 7, the design static strength in Clauses 5*/5.3.2.5 and the design procedures in Clauses 5*/5.3.3.5 and 5*/6.3.4. The design of the connectors must provide a resistance per unit length of at least the maximum design load shear flow over 10% of the length of the span each side of a support. In other parts of the span a series of groups of connectors at constant spacing may be used to provide a 'stepped' resistance, subject to the provision of sufficient total resistance over each length. The maximum calculated shear flow within the length of any such group must not be more than 10% in excess of its design resistance per unit length. Additional requirements for spacing of shear connectors on the top of closed box girders are given in Clause 5*/7.5.1.

Transverse reinforcement is required in the slab to provide shear resistance at ULS in a similar manner to the requirements for shear stud spacing (Clause 5*/6.3.3). This requires an amount of bottom layer reinforcement which is usually adequately provided by continuity from mid span of the slab.

5.7 Fatigue considerations

5.7.1 General

The fatigue endurance of the steelwork in a bridge is assessed using Part 10 of the Code. Fatigue failure of steel arises from the propagation of cracks in regions which are subject to fluctuating stress. The fatigue life depends on the size of the initial defect or stress concentration and on the range of the stress variation.

The Code provides three methods for the assessment of fatigue life which involve different determinations of the effective range of stress variation. In order of increasing complexity they are:

- Without damage calculation - limiting stress range (Clause 8.2)
- Damage calculation - single vehicle method (Clause 8.3)
- Damage calculation - vehicle spectrum method (Clause 8.4)

The first method is most commonly used for smaller bridges and is much the quickest, though somewhat conservative. It can be applied to box girder configurations, though the inherent conservatism means that designers would usually opt for the second method. The third method would normally only be employed on large and complex structures where economy requires greater precision of assessment.

The method given in Clause 10/8.2 requires the determination of the maximum and minimum stress as a 'standard fatigue vehicle' (Clause 10/7.2.2.1) crosses the bridge. The range (maximum to minimum) is then compared with a limiting stress range appropriate to the classification detail and the spectrum for the road category (Clause 10/8.2.2 and Figure 8). If the designer finds that his range exceeds the limiting range, he may choose to specify a better class of detail, modify the design to reduce the stress range or to re-assess by the Clause 8.3 method.

The method given in Clause 10/8.3 requires the determination of a number of different ranges, of differing magnitude, as the standard fatigue vehicle transverses the length of the bridge in each of the traffic lanes. Fatigue life is then assessed by summing the damage caused by repeated application of all these ranges. It therefore involves more calculation, but is less conservative.

The method given in Clause 10/8.4 is a still more detailed method which requires calculations of stress ranges for a spectrum of different vehicles.

5.7.2 Detail classification

Part 10 allows for the size of defect or stress concentration by means of a comprehensive classification of welded and non-welded details - see Table 17 of Part 10. The designer simply identifies the appropriate classification for the detail he is considering. The code provides figures which set out the relationship between stress range and fatigue life for each class of detail. Ten detail classes are referred to by letter designations: A to F, F2, G, S and W. The highest classes (A and B) are not normally found in bridgework.

Welded details are more susceptible to fatigue than non-welded details, because they contain defects, particularly on the surface, which are of sufficient size to promote crack propagation under varying stress. Welded details which run transverse to the direction of principal stress are most susceptible, because the defects are across the line of stress.

The attachments of web stiffeners, or other elements not carrying load in the stressed direction, introduce a Class F fatigue detail to a flange. Cope holes introduce a class F detail. Reinforcing plates, welded-on bearing plates and the attachment of cross-beam flanges introduce a Class G detail. Shear connectors produce a Class F detail in the stressed direction of the flange plate. The class for butt welds between plates depends on the weld procedure and any subsequent grinding; Classes C, D or E can be achieved, but normal inspection is unable to confirm defect sizes less than that appropriate to Class F, so the higher classes should not be presumed.

Grip bolted splices introduce Class D details around the bolt-holes. Cross bracing introduces a variety of details, including Class W at the end connections. (Class W is for load transferred through a weld.) Stud shear connectors are considered as a special detail, class S, in relation to the transfer of shear from flange to slab.

5.7.3 Regions subject to stress variation

In a bridge, the regions which require consideration of fatigue life are those where variation in stress is relatively large and where welded details occur. In terms of fatigue, 'relatively large' means a stress variation of about 20 N/mm^2, when the 320 kN fatigue vehicle travels across the bridge.

Regions over or adjacent to intermediate supports, where stiffeners or diaphragms are welded across the flanges and webs, are perhaps the most obvious for consideration. In determining the stress range at any detail it must be remembered that for fatigue it is the actual stresses at the detail which are important. In box girders the steel flange can be quite wide; shear lag can significantly affect the magnitude of the peak stress range.

Midspan regions also suffer a fairly wide stress range, though shear lag effects are usually small at these positions.

In determining stress ranges for both regions, warping stresses due to restraint of torsional warping, and distortional warping stresses should be included; they are part of the actual stress variation which is experienced by the plates of the box.

Cross frames which restrain the box against distortion also suffer significant stress variation. The effects of loads in different lanes can be particularly onerous, since passage of the vehicle along one web line produces distortion in the opposite sense to that when it is over the other web. Ring frames provide restraint by virtue of their bending stiffness and strength; moments are highest at the corners, where web and flange stiffeners meet, and these regions must be checked. Cross bracing provides restraint by the axial stiffness and strength of the bracing members; the connections at their ends, between brace and transverse stiffener must be checked (the details often depend on fillet welds and involve the Class W detail). When plated diaphragms are used to provide restraint, they must be connected along all four edges.

Fatigue checks of the shear connectors (which always use the simplified procedure of Clause 10/8.2) considers the stress range in the weld detail between connector and flange (Clause 10/6.4.2). This may be more onerous than design for static strength in midspan regions. Flanges with shear connectors are treated in the same way as with other transverse connections.

In some circumstances the torsional restraint offered to the slab by the box girder can result in local uplift at the corner of the box. This results in tension in the first line of shear connectors and in local bending of the flange. In such circumstances a double fillet weld detail (with the flange oversailing the web) may prove better than a partial penetration weld which would be subject to bending across its narrow throat (see Figure 5.2).

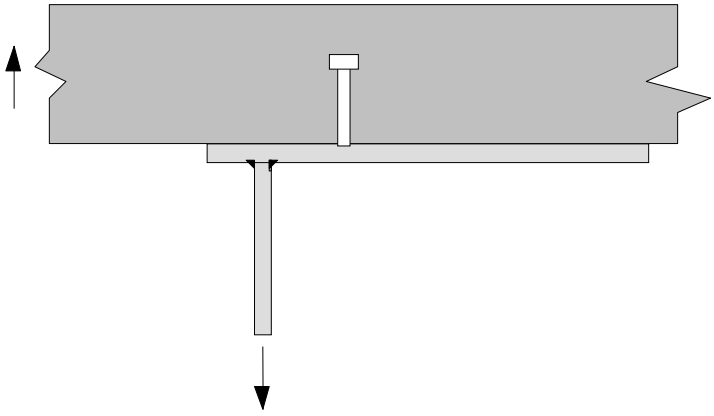

Figure 5.2 *Local uplift at a box corner resisted by paired fillet welds*

5.8 Deck slab

5.8.1 Local analysis

Local moments in the deck slab due to the effects of HA or HB wheel loads can be calculated using recognised methods such as the use of Pucher Influence Charts[18]. 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. These moments must then be added to the transverse live load moments from the grillage analysis and to dead load moments calculated (usually by hand) for a transverse strip of deck. To avoid double counting of local effects they must be excluded from the global analysis. This is achieved by applying all the loads to the nodes at the web positions in the global analysis.

With box girders it is usual, even when intermediate supports are at a skew to the line of the bridge, to place the slab reinforcement orthogonal to the beams. Local moments can be directly added to global moments.

Part 5 allows the top flange of a closed box to be considered to act compositely with the slab (Clause 5*/7.7), though designers often neglect this because of the increased requirements on shear connector design.

5.8.2 Slab design

Bending of the main beams results in compressive stresses in midspan regions of the deck slab and these are normally well within the compressive resistance of the concrete. Tension reinforcement is provided over intermediate supports to carry global effects. The design of the deck slab transversely is determined mainly by the local effects of the wheel loads in conjunction with the distribution moments arising from bending of the main girders.

The deck slab needs to be checked at both limit states. At ULS the bending resistance must be adequate. At SLS the stresses and crack widths must be within the prescribed limits. Usually the moments at only two positions, midway between main beams and over the main beams, are used to determine a uniform pattern of reinforcement.

Twisting moments in the slab about longitudinal and transverse axes are normally small and thus the principal moments correspond sensibly with the two orthogonal moments. If there are twisting moments, effective design moments can be determined by methods such as Wood[19] and Armer[20]. The method is generally recognised and is available within some computer analysis programs. Once these moments have been calculated they can be used to determine the required resistances at SLS or ULS.

Design at ULS

Design moments at ULS will determine the amount of reinforcement required in the slab. The bending resistance is given by Clause 4/5.4.2. Shear stresses under the concentrated loads should also be checked (Clause 4/5.4.4), though the amount of reinforcement provided for bending resistance is usually sufficient for shear as well.

For slabs spanning transversely, transverse reinforcement is usually placed as the outer layer (outside the longitudinal reinforcement) because the transverse moments are greater than the longitudinal moments. Longitudinal shear

between the girder and the slab requires the provision of sufficient transverse reinforcement in the bottom face (within the height of the connector) and this may govern near the ends of the span. Transverse reinforcement is usually uniform across the width of the slab, for simplicity and economy. If precast planks are used as permanent formwork the plank reinforcement does not usually need to achieve continuity over the main beams: the in situ bottom reinforcement should be sufficient for shear transfer.

Design at SLS

The limiting stress in the reinforcement at SLS is $0.7f_y$ (Clauses 4/4.3.1 5*/5.2.2). The spacing of the reinforcement (and thus the size of the bars) is determined by the requirements for crack control: the minimum longitudinal reinforcement required in each face to satisfy requirements for control of cracking is given by Clause 4/5.8.9; the maximum spacing of transverse reinforcement is given by Clause 4/5.8.8.2. The loading appropriate to crack width calculations is given by Clause 4/4.2.2; note that this has been modified by BD 24/92.

Additional requirements for control of early thermal cracking, particularly for the situation where one strip of slab is cast alongside an existing slab, are given in Departmental Standard BD 28/87.

New fatigue requirements for reinforcing bars were introduced by BD 24/92; they are far more stringent than those originally in Part 4.

It should be noted that the minimum cover to reinforcement is increased (above that given in BS 5400-4) by BD 57/01.

5.9 Construction

5.9.1 Splices

Each box girder will be fabricated in a number of lengths and joined together on site, either prior to or during erection. Splice positions are normally arranged to be away from positions of maximum moment, though consistent with the use of the longest feasible individual lengths, for economy.

Splices can be either bolted or welded. Bolted splices offer quicker completion of site work and avoid the more rigorous requirements of quality control necessary for site welding. On the other hand, welded splices offer a clean finished surface with better appearance and easier maintenance of the protective treatment.

When the splice is away from the maximum moment in the section, it should be designed simply to transmit the greatest design load effects at that position, rather than the full strength of the section (Clause 3/14.3.1).

Bolted splices are usually connected using High Strength Friction Grip (HSFG) bolts. Black bolts are not permitted in any structural connection (Clause 3/14.5.3.1). The use of higher grade bolts (e.g. Grade 8.8) to BS 3692 in clearance holes is considered in the same category as black bolts and is thus prohibited. Cover plates are normally provided on both faces of each flange and web (Clause 14.4.1.1). The number of bolts required may be determined in accordance with Clause 3/14.5.4, either at ULS on the basis of friction resistance, or, more economically, on the basis of no slip at SLS and bearing/shear

at ULS. Note that this second method cannot be used when holes are slotted or oversize; design must then be on the friction resistance at ULS (see Clause 3/14.5.4.1.1). Bolts should be spaced in accordance with Clause 3/14.5.1. Stresses should be checked in the cover plates and in the girder on the weaker side of the splice, allowing for holes in determining net sections - see Clause 3/14.4.

It may be necessary to provide shear connectors on the upper cover plate to the top flange, to comply with maximum spacing limitations. The number of connectors should be kept to a minimum, since their presence complicates the tightening of the bolts and may distort the cover plates.

Splice welds in flanges and webs are usually specified as full penetration butt welds. Partial penetration welds are prohibited where the stresses may be tensile (Clause 3/14.6.2.2). Longitudinal stiffeners are normally spliced by cover plates which are lapped and fillet welded. Covers should be provided on both faces of the stiffener and at least one of the covers must be welded to the plate which is being stiffened (Clause 3/14.4.1.2).

5.9.2 Open steel boxes

Open steel boxes save weight in the top flange and are usually easier to fabricate than closed boxes, but during construction require lateral bracing to the top flange (usually cross frames will be adequate); additional bracing and the sequence of concreting must be considered carefully. Some plan bracing, usually just below the top flange, is also likely to be necessary to stiffen the girder against torsional effects during erection. Cross-ties are necessary with trapezoidal sections to stop the 'U' opening out. Both of these must be positioned so as not to interfere with formwork for the slab. Over the box, permanent formwork is normally preferred, to avoid the subsequent difficult task of stripping out.

5.9.3 Closed steel boxes

Closed steel boxes avoid temporary bracing and offer a ready-made form for the slab. However, the top flange must be designed to carry the weight of wet concrete, and the deflection under that load should not be overlooked. Shear connectors are required over the full width of the flange (Clause 5*/7.5.1).

5.9.4 Notch toughness of steel

Any part of the steelwork which is subject to tensile stress during erection or service is required to have adequate notch toughness (Clause 3/6.5). The degree of toughness required is expressed as a Charpy impact value and depends on the thickness of the material and its minimum temperature in service. Steels to BS EN 10025 have specified Charpy impact values at a given temperature. This temperature depends on the grade 'quality' (typical quality designations for toughness properties are J2 or K2).

Minimum bridge temperatures are specified in Part 2* (Clause 2*/5.4.3) for composite bridges and are typically -15°C in England and -18°C in Scotland. Appropriate grade qualities of steel must be selected to give the required toughness (impact value) for the flange thickness chosen, at this temperature (see Clause 3/6.5.4 and Table 3). It is not necessary to use the same quality throughout the whole structure - thick tension flanges could be quality K2 for example, while the remainder is quality J2. The designer should be aware of the possibilities for confusion if he uses more than one grade.

6 FLOW DIAGRAMS

The requirements of BS 5400 for the design of composite box girder bridges are presented in the form of a series of flow diagrams. The diagrams relating to deck slab, shear connection and bolted splices are essentially the same as those included in SCI publication P289[1]

There are ten separate diagrams:

6.1 An overall diagram, leading to a series of checks

6.2 Checks for beams without longitudinal stiffeners

6.3 Checks for beams with longitudinal stiffeners

6.4 Checks for cross-frames and distortional effects

6.5 Checks for transverse web stiffeners

6.6 Checks for unstiffened diaphragms

6.7 Checks for stiffened diaphragms

6.8 Checks for deck slab

6.9 Checks for longitudinal shear connection

6.10 Checks for bolted splices

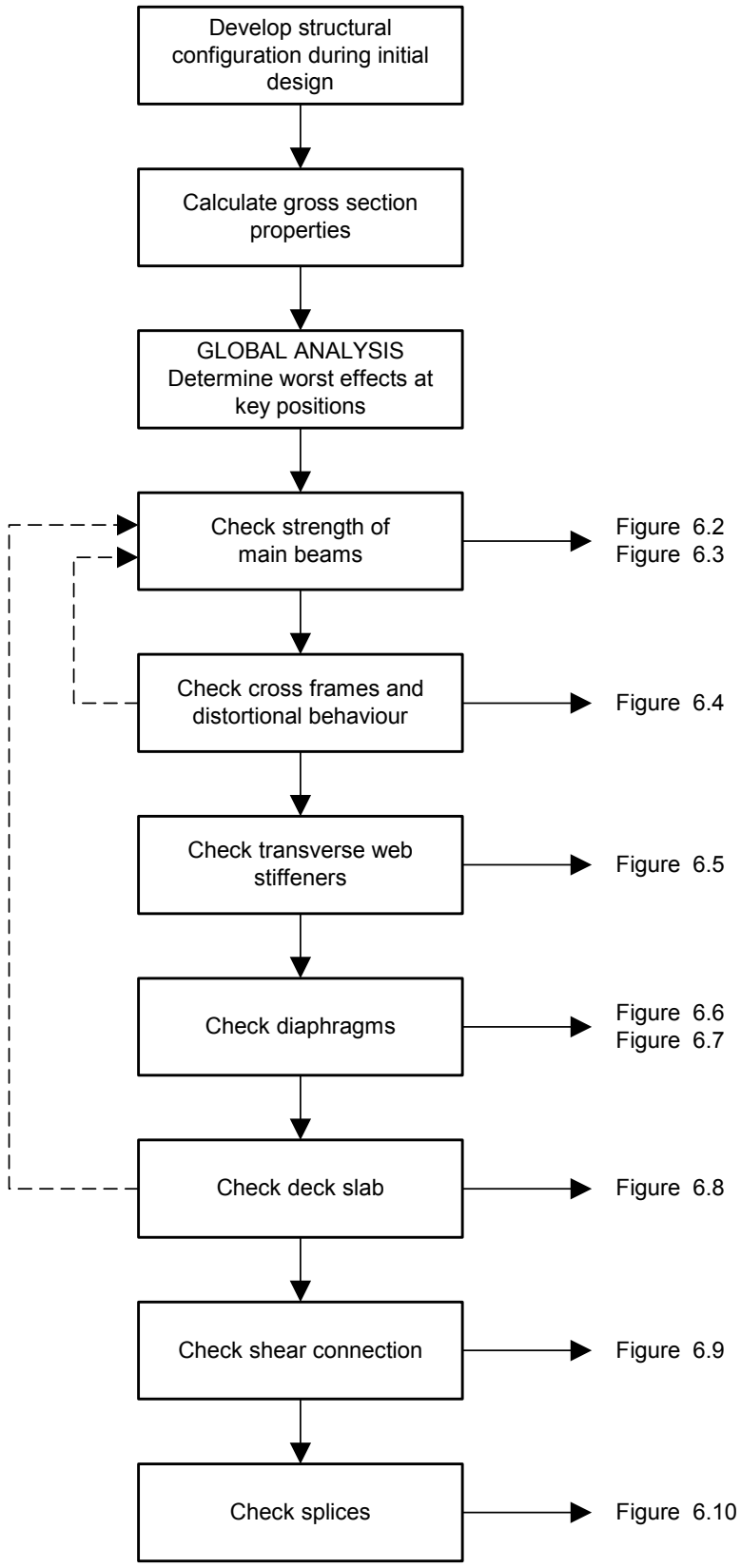

Figure 6.1 *Flow diagram for design of box girder bridges - overall diagram*

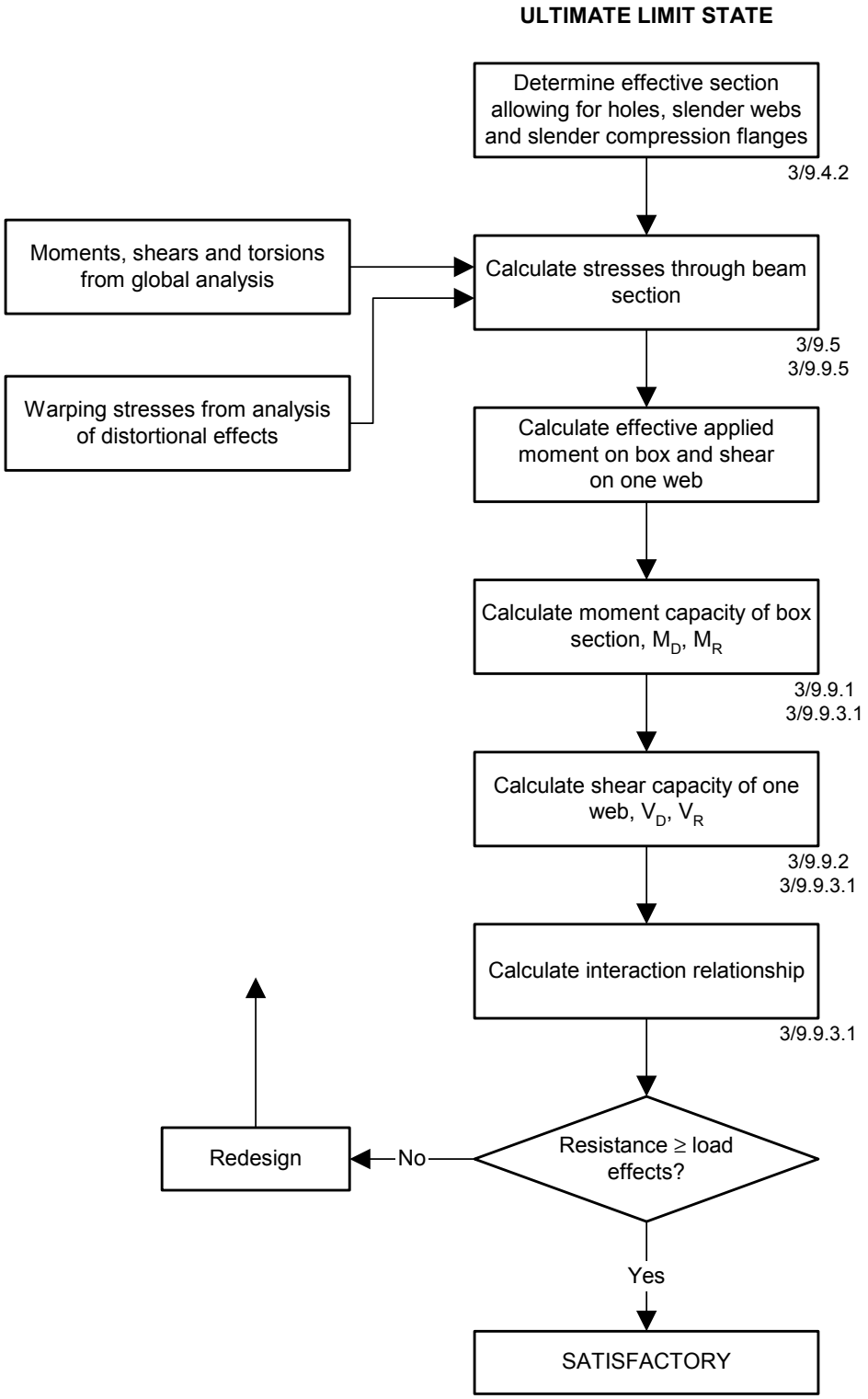

Figure 6.2 *Box section beams without longitudinal stiffeners*

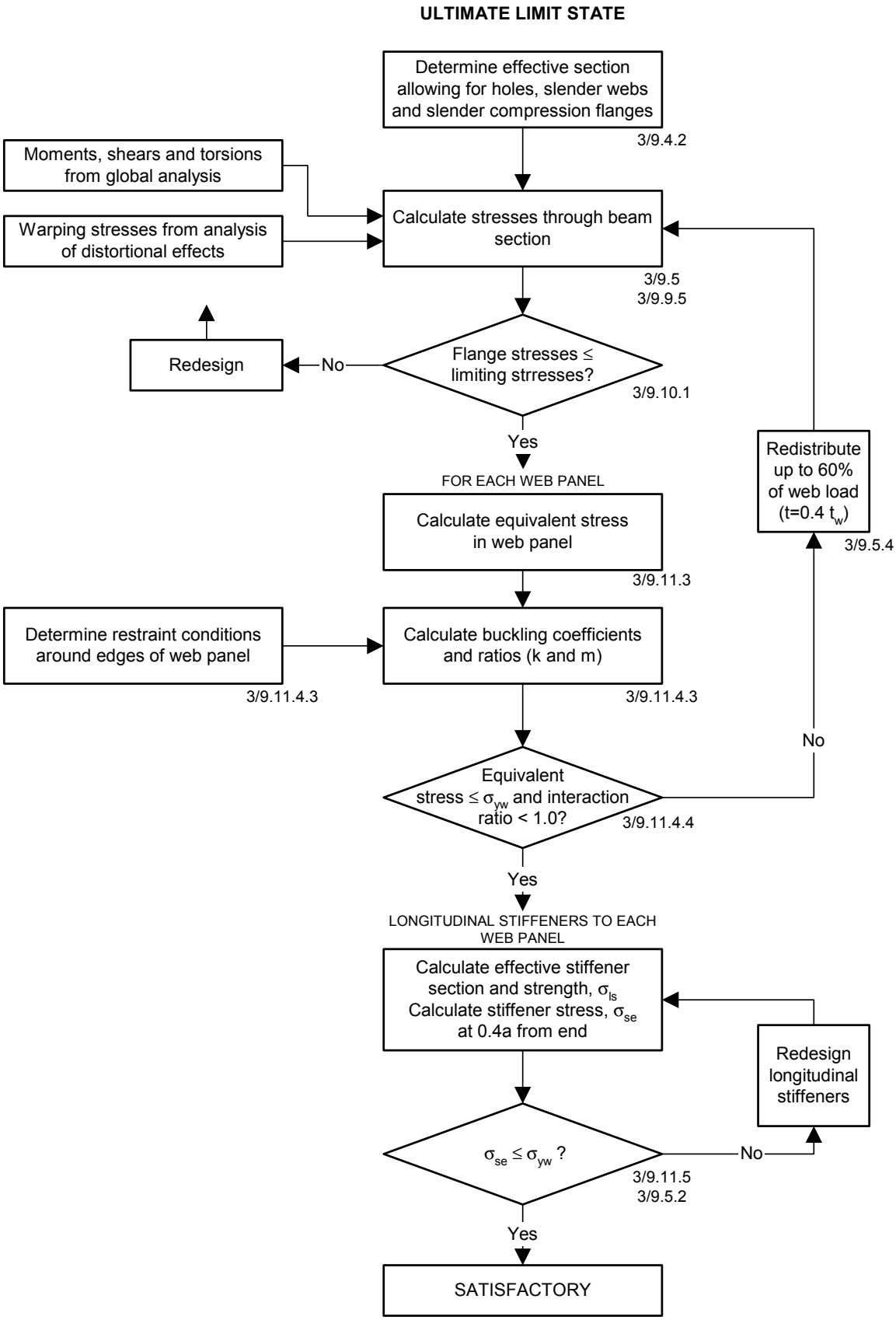

Figure 6.3 *Box section beams with longitudinal stiffeners*

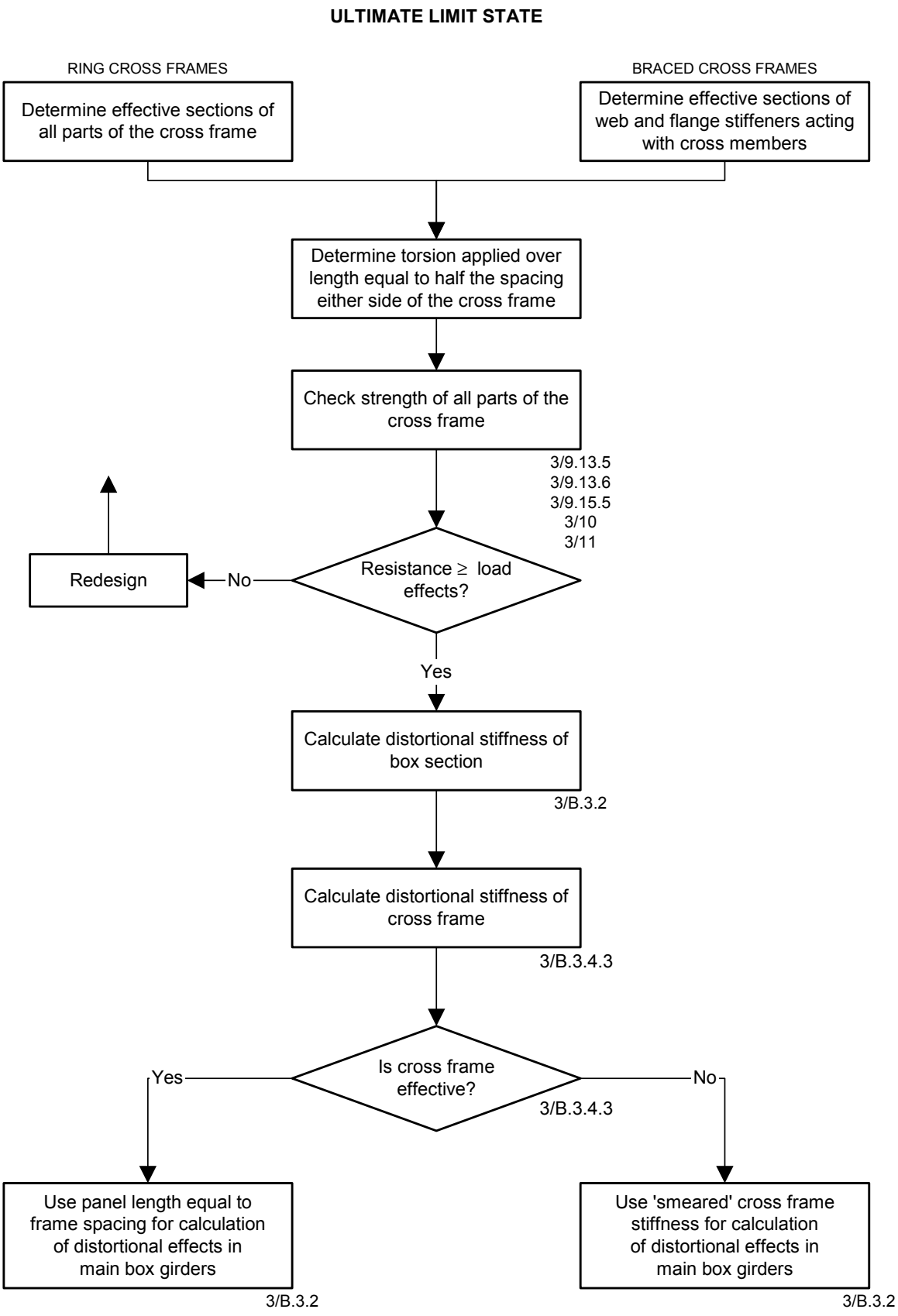

Figure 6.4 *Cross-frames and distortional effects*

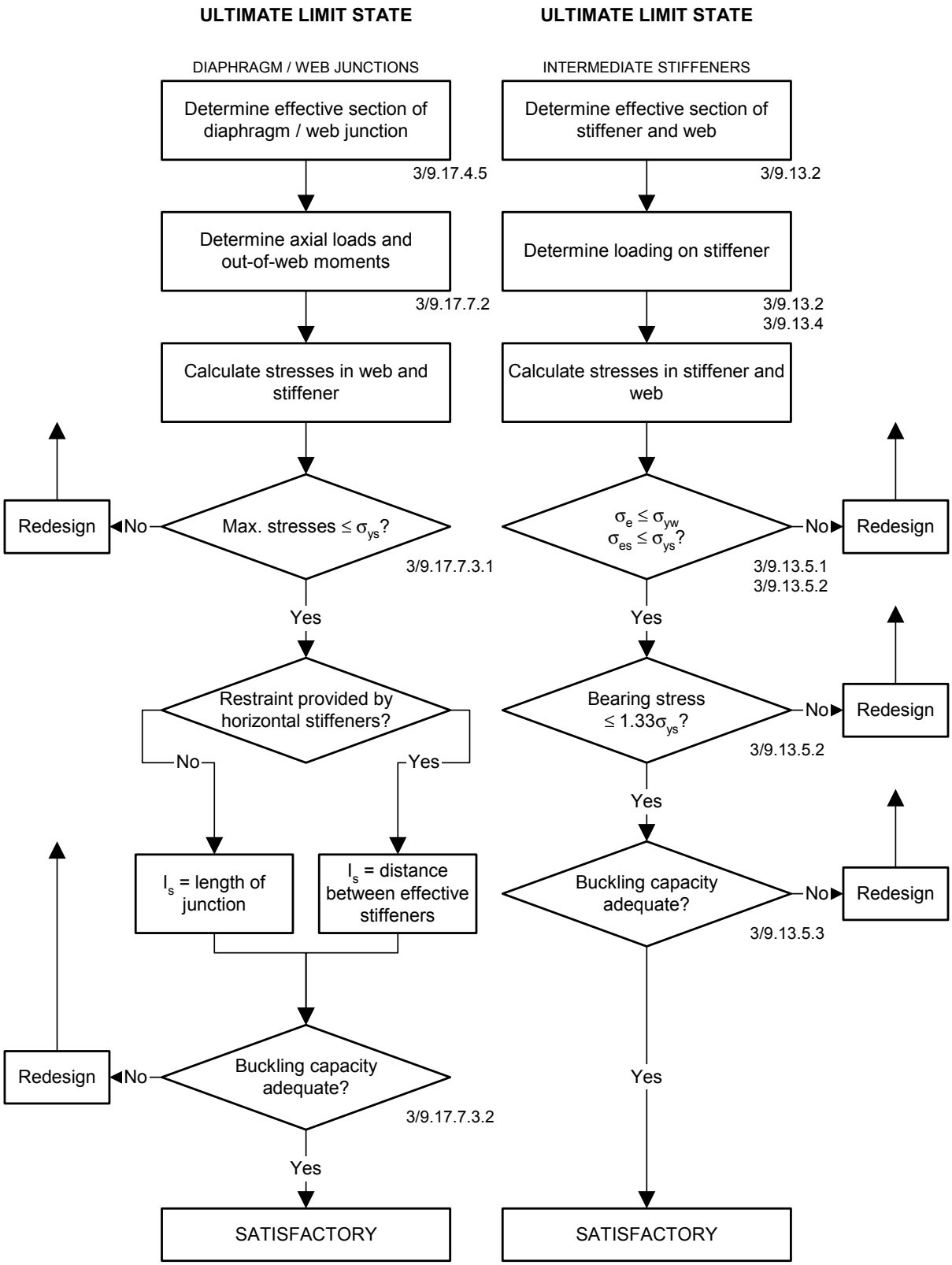

Figure 6.5 *Transverse web stiffeners*

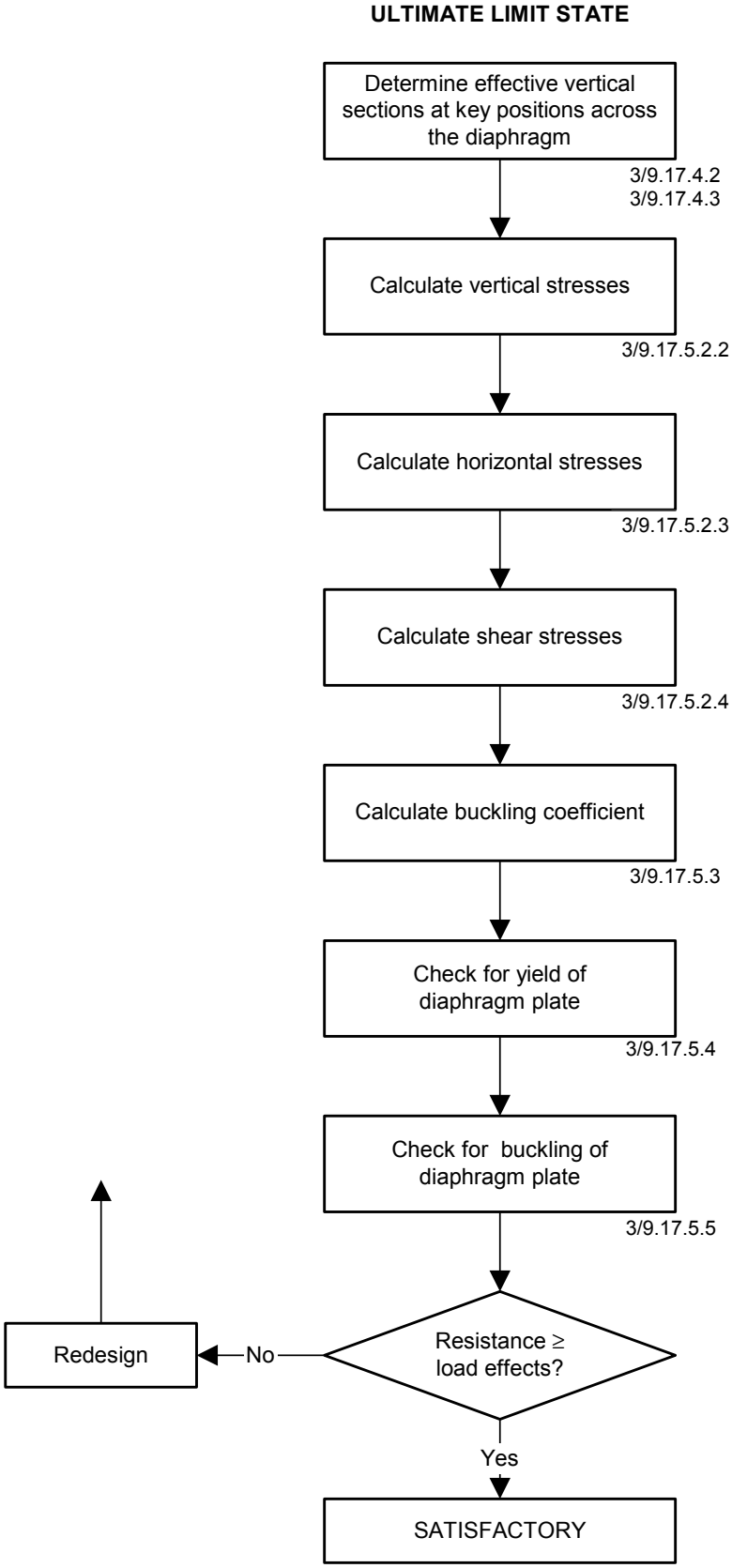

Figure 6.6 *Unstiffened support diaphragms*

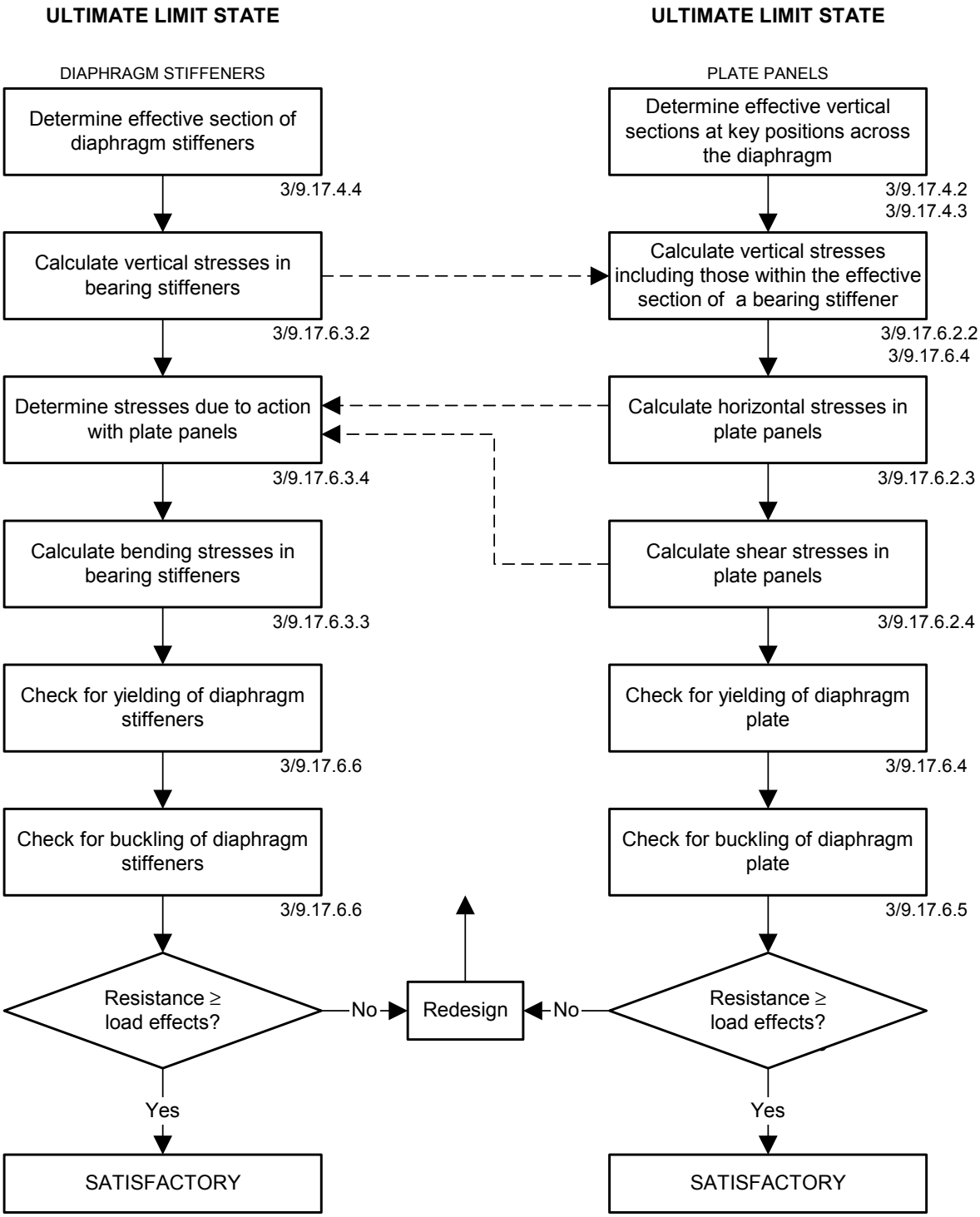

Figure 6.7 *Stiffened support diaphragms*

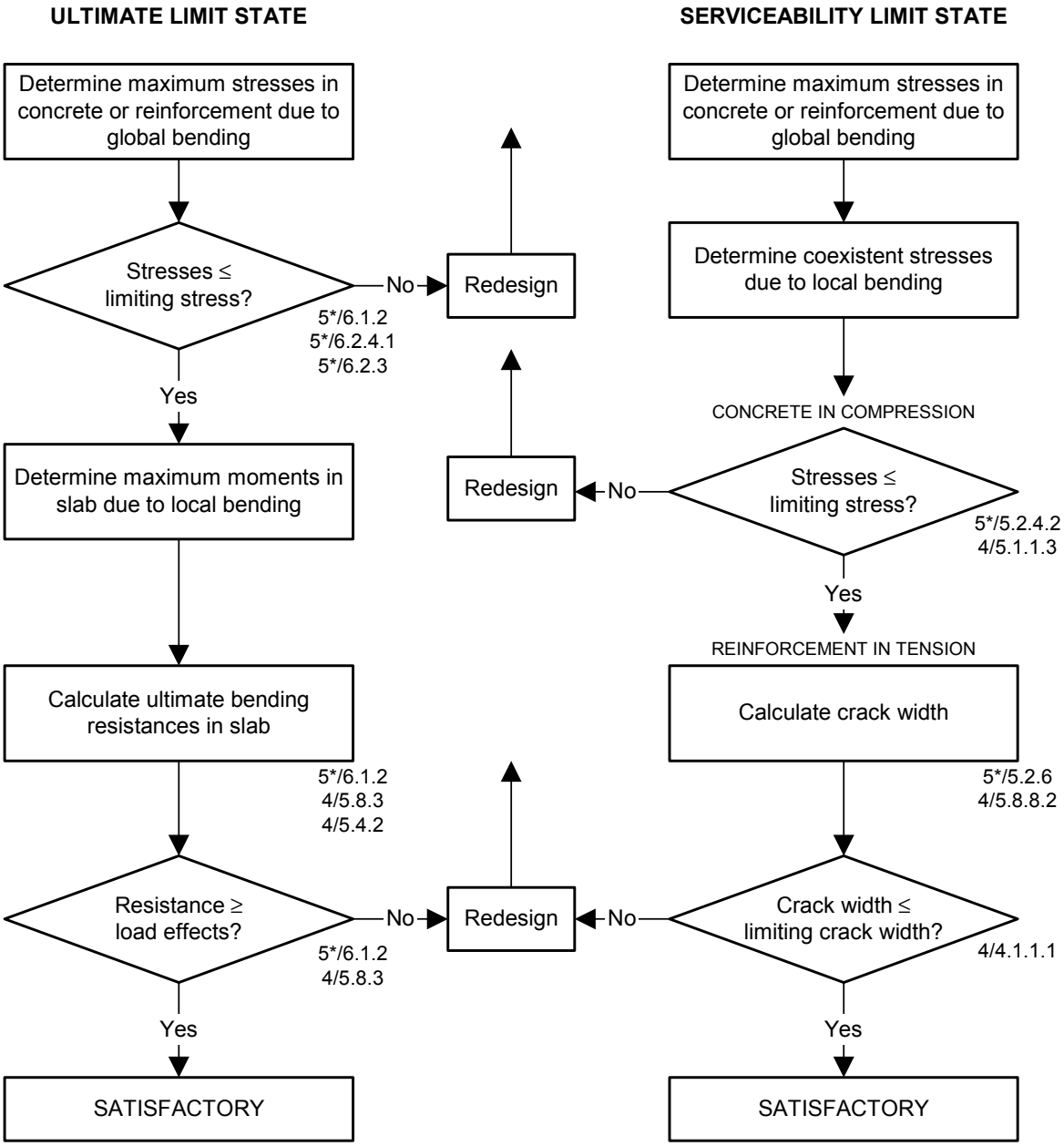

Figure 6.8 *Deck slab*

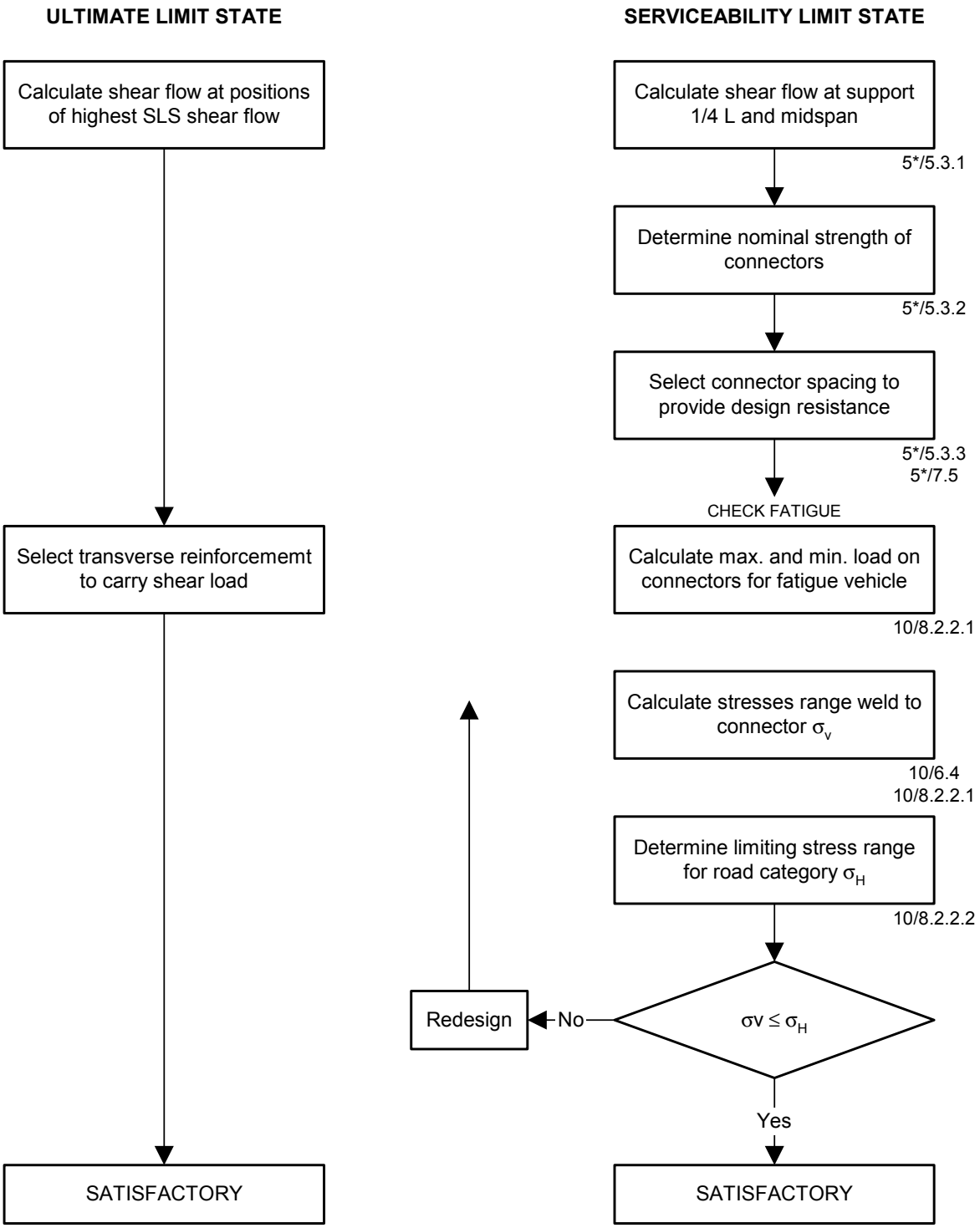

Figure 6.9 *Shear connection*

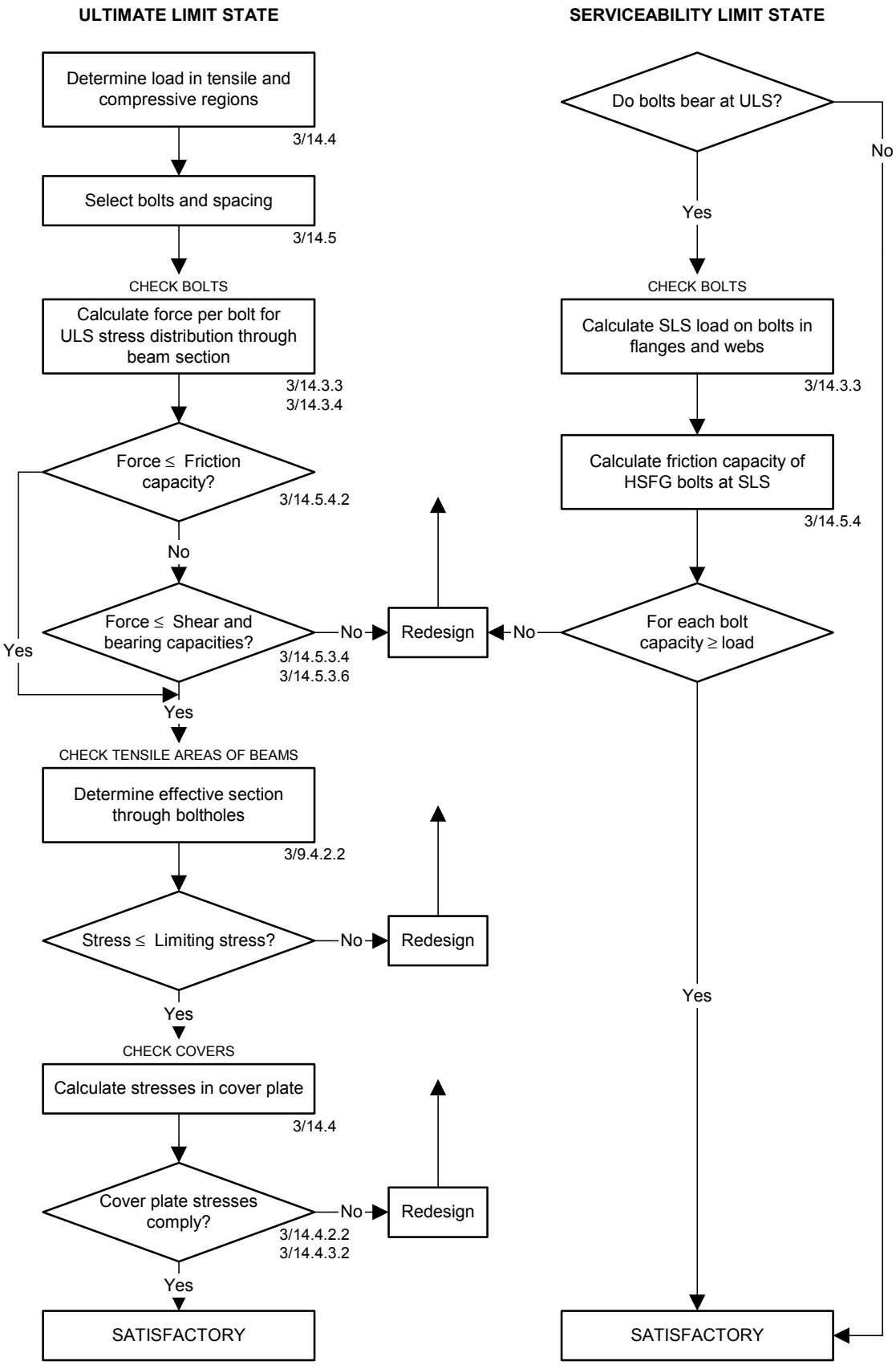

Figure 6.10 *Bolted splices*

7 REFERENCES

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Design guide for composite highway bridges (P289), SCI, 2001
Design guide for composite highway bridges: Worked examples (P290), SCI, 2001

2 EVANS, J.E. and ILES, D.C. (Editors)
Steel Bridge Group: Guidance Notes on Best Practice in Steel Bridge Construction (P185) (3rd issue)
The Steel Construction Institute, 2002

3 BRITISH STANDARDS INSTITUTION
BS 5400: Steel concrete and composite bridges

Part 1: 1988	General statement
Part 2: 1978	Specification for loads
Part 3: 2000	Code of practice for design of steel bridges
Part 4: 1990	Code of practice for design of concrete bridges
Part 5: 1979	Code of practice for design of composite bridges
Part 6: 1999	Specification for materials and workmanship, steel
Part 7: 1978	Specification for materials and workmanship, concrete, reinforcement and prestressing tendons
Part 8: 1978	Recommendations for materials and workmanship, concrete, reinforcement and prestressing tendons
Part 9: 1983	Bridge bearings
Part 10: 1980	Code of practice for fatigue

4 Inquiry into the basis of design and method of erection of steel box girder bridges
Report of the Committee, Appendix 1, Interim Design and Workmanship rules
HMSO, 1973 (now, The Stationery Office)

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Discussion on reference 19 above
Concrete, Vol 2, No. 8, Aug 1968

APPENDIX A Guidance on initial selection of flange and web sizes

Dead load effects will form a fairly small proportion of the total design load effects. The loading can be easily estimated for the concrete and surfacing based on the deck width and thickness - typically 250 mm for the concrete and 120 mm for surfacing/waterproofing. The dead load of the steel can be taken approximately as between 150 kg/m^2 and 500 kg/m^2 for spans between 45 m and 80 m (this forms such a small proportion of the total load that accuracy is not necessary at this stage).

Live load depends on the number of lanes of traffic which are carried. Footways should also be loaded. The effects due to the abnormal vehicle (HB) will usually govern in midspan regions, when 45 units of load are to be applied. For regions over intermediate supports, HA loading can be assumed to govern at this stage, in this span range.

Where the section is approximately constant along the length of the beam and the spans are roughly equal, bending moments due to dead load can be based generally on moments in a fixed-ended beam. Total live load bending moment at midspan and support regions can similarly be calculated as simple proportions of moments in fixed ended beams, making some judgement on the effects of relaxation due to continuity. For example: use 90% of fixed-ended support moments where the spans either side of the support are in ratio of less than 2:1; use 150% of central moment when adjacent spans are over half the central span

When the beams are haunched the distribution of bending moments is changed significantly; more moment is attracted to intermediate support regions, less is carried at midspan. A simple line-beam analysis should be used to indicate how the moments will be distributed.

Box girders are better at sharing the load than I-beams. Divide equally the total load carried over two boxes.

After the above simple approximations, increase live load moments by about 20% to cater for the approximation of the estimation.

The total dead and live load moments can then be used to make a first estimate of flange sizes; ignore the webs in calculating the flange size needed.

It is quite likely that several cycles of estimation and revision will be needed, even when the initial choice of section depth is maintained.

APPENDIX B Departmental Standards and Advice Notes

In the UK, Departmental Standards and Advice notes are published and distributed by the Stationery Office on behalf of the Government's four Overseeing Departments for highways. They are published as part of the *Design Manual for Roads and Bridges (DMRB)*.

The DMRB is introduced by document DMRB 1.0.1, which includes a comprehensive index to all the Standards and Notes.

The Departmental Standards and Advice Notes relating to the use of the various Parts of BS 5400 which were effective at November 2003 are:

Part 1: BD 15/92, December 1992

Part 2: BD 37/01, August 2001
(Includes amended version of Part 2)*

Part 3: BD 13/90, February 1991
BA 19/85, January 1985

Part 4: BD 24/92, October 1992

Part 5: BD 16/82, November 1982
Amendment No 1, December 1987
(Also, combined document, Part 5 and BD 16/82)*

Part 9: BD 20/92, October 1992

Part 10: BD 9/81, December 1981
BA 9/81, December 1981
Amendment No 1, November 1983

*The combined documents are issued with the permission of the British Standards Institution.

BD 13/90 is due to be updated to implement BS 5400-3:2000.

The following may also be applicable to construction of deck slabs:

BD 28/87 Early thermal cracking of concrete, November 1986
Amendment No 1, August 1989

BA 24/87 Early thermal cracking of concrete, January 1987
Amendment No 1, August 1989

BA 36/90 The use of permanent formwork, February 1991

The following Standards are referred to in the text of this publication:

BD 7/01 Weathering steel for highway bridges, November 2001

BD 57/01 Design for Durability, August 2001.

APPENDIX C GUIDANCE NOTES

The following is a list of all the Guidance Notes in the SCI publication P185 *Steel Bridge Group: Guidance notes on best practice in steel bridge construction*.

GN	Title
Section 1	Design – general
1.01	Glossary
1.02	Skew bridges
1.03	Bracing systems
1.04	Bridge articulation
1.06	Permanent formwork
1.07	Use of weather resistant steel
1.08	Box girder bridges
1.09	Comparison of bolted and welded splices
1.10	Half through bridges
Section 2	Design – detailing
2.01	Main girder make-up
2.02	Main girder connections
2.03	Bracing and cross-beam connections
2.04	Bearing stiffeners
2.05	Intermediate transverse web stiffeners
2.06	Connections made with HSFG bolts
2.07	Welds – how to specify
2.08	Attachment of bearings
Section 3	Materials and products
3.01	Structural steels
3.02	Through thickness properties
3.03	Bridge bearings
3.04	Welding processes and consumables
3.05	Surface defects on steel materials
3.06	Internal defects in steel materials
3.07	Specifying steel material
Section 4	Contract documentation
4.01	Drawings
4.02	Weld procedure trials
4.03	Allowing for permanent deformations
4.04	Alternative construction sequences
Section 5	Fabrication
5.01	Weld preparation
5.02	Post-weld dressing
5.03	Fabrication tolerances
5.04	Plate bending
5.05	Marking of steelwork
5.06	Flame cutting of structural steel
5.07	Straightening and flattening
5.08	Hole sizes and positions for HSFG bolts
5.09	The prefabrication meeting

APPENDIX D Worked examples

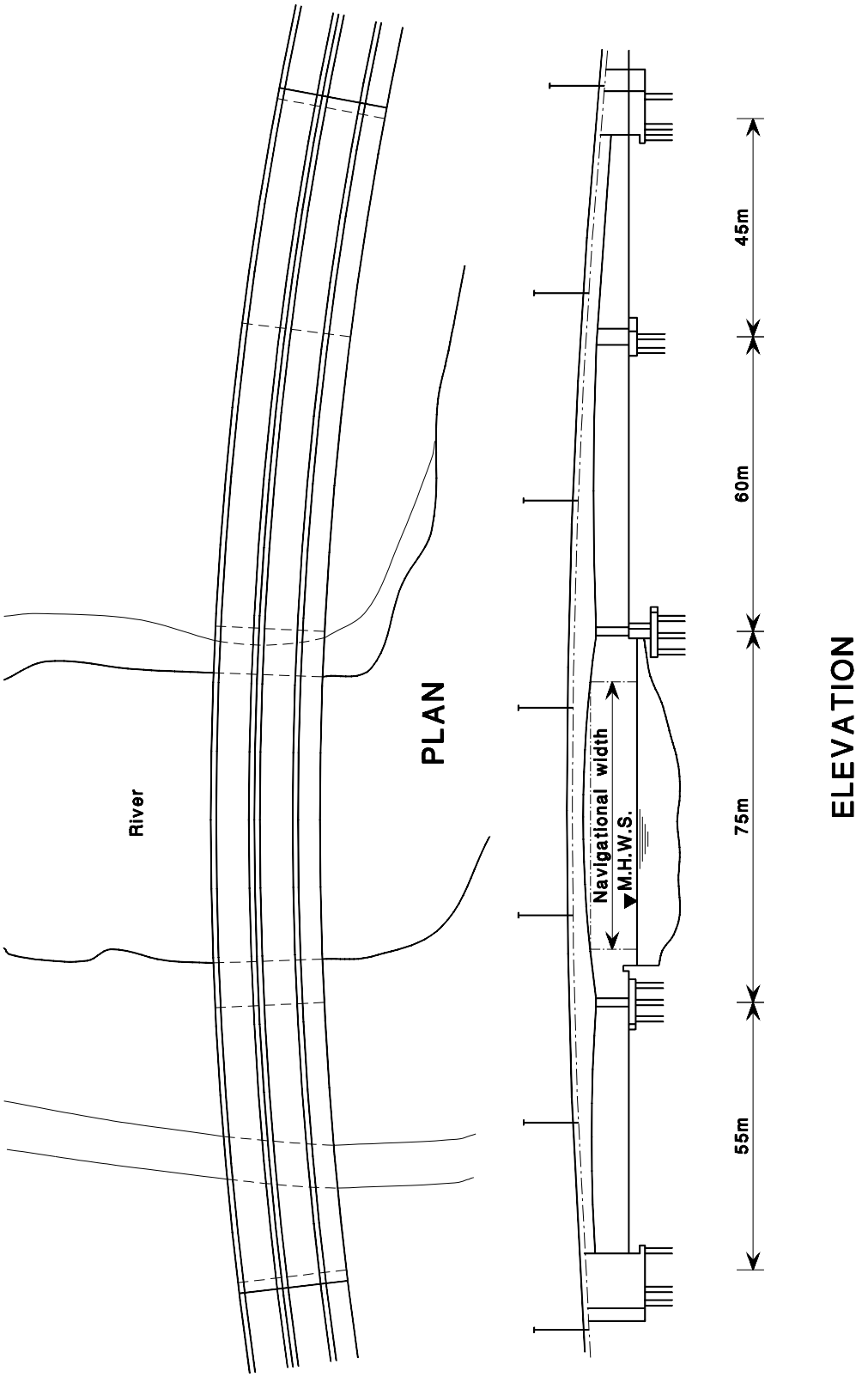
Selected example calculations are included in this appendix to illustrate aspects of the design process for composite box girder bridges. Two examples are provided, one for a bridge using closed rectangular steel boxes with a reinforced concrete slab on top, and the other using open trapezoidal steel boxes which are closed by a concrete slab across the top.

The calculations illustrate the initial design and aspects of the detailed design which are particular to box girders. It is assumed that the reader will already be familiar with detailed calculations for composite I-beam bridges, and examples of such matters as the calculation of section properties, wind and differential temperature loads, design of intermediate web stiffeners, shear connection, and slab design; these topics are therefore not included. If examples of such matters are needed, reference can be made to the other SCI bridge design guides.

The two examples are:

Example 1 - Closed rectangular boxes, 75 m span

Commentary to calculation sheet



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Fax: (01344) 622944

CALCULATION SHEET

Job No. BCR 433		Sheet 1 of 26	Rev.
Job Title	Box Girder Design Guide - Example 1		
Subject	Design Data		
Client SCI	Made by DCI	Date	Mar 1994
	Checked by: KB	Date	Mar 1994

DESIGN DATA

General

Spans: 55 m, 75 m, 60 m, 45 m (Total 235 m)

Carriageway: Dual 7.3 m carriageway with 1 m verges cycle/footway on one side

Surfacing: 100 mm including waterproofing

Location: South-East England

Design Life: 120 years

Loading

Unit Weights:

Steel	77 kN/m^3
Concrete	25 kN/m^3
Surfacing	22 kN/m^3
Parapets	0.5 kN/m

Live Loads: HA 4 notional lanes (to BD 37/01)
HB 45 Units
Cycletrack (Clause 6.5 of BD 37/01)

Temperature: Minimum effective bridge temperature −14°C
Maximum effective bridge temperature +40°C

Wind: Mean hourly windspeed 26 m/sec

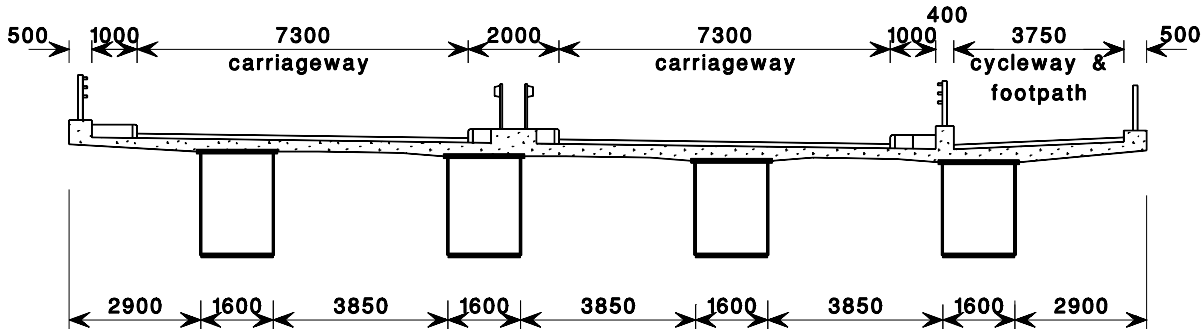
Design Parameters

Steel	σ_y	=	355 N/mm^2 (up to 16 mm thick)
	E	=	205 kN/mm^2
Concrete	f_{cu}	=	40 N/mm^2
	E_{cs}	=	31 kN/mm^2
	E_{ci}	=	15.5 kN/mm^2
Reinforcement	f_{ry}	=	460 N/mm^2
	E_y	=	200 kN/mm^2

Commentary to calculation sheet

Maximum spacing is that which is considered to be a reasonable limit for shear considerations in the slab.

Rectangular box sections without any bracing between them were chosen to suit the installation and maintenance of various service pipes and cables. Non-structural platforms were fitted between the boxes, for maintenance staff, as well as pipes and cable trays.



A haunched configuration, with a curved soffit is generally regarded as pleasing in appearance in river crossings. The cantilever provides some degree of shelter for the outer face and creates a shaded area which contrasts with the lighter edge beam.

INITIAL DESIGN

Overall width of deck = 23.75 m
Assume max spacing between boxes = 4.0 m (for 300 mm slab)

Choose 4 rectangular closed boxes, 1.6 m wide
Choose cantilevers 2.9 m each side

Hence spacing between boxes = 3.85 m

Reduce the slab thickness to 230 mm between the boxes (i.e. haunched transversely)

Girder Depth

Use a haunched configuration over piers 1 and 2, constant depth over pier 3.

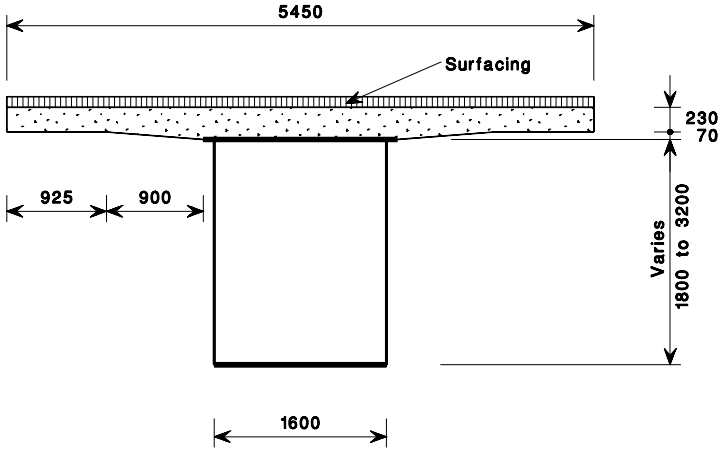
Mid main span: use span/depth ratio approx 35 : 1
Say, D = 2.1 m (i.e. 1800 mm box, 300 mm slab)

Haunch over piers 1 and 2: use span/depth ratio approx 20 : 1
i.e. D = 75.0/20 = 3.75
But limit box depth to twice its width
i.e. D = 3.5 m (3200 mm box, 300 mm slab)

Side spans (including pier 3): use D = 2.1 m, as mid main span

Cross Section

The chosen cross section for analysis and detailed design is thus:



Commentary to calculation sheet

A haunched configuration attracts more moment to the support regions and reduces midspan moments. A simple line beam model is used; an approximate variation of sectional inertia will give a better distribution of moments than uniform properties. Here the designer used a model with nodes at 5 m centres and an inertia which was proportional to the square of the depth of the section (absolute values are not important for longitudinal distribution, only relative values). The distribution of live load between boxes was simply to share the load on one carriageway between two boxes. For torsionally stiff sections and long spans this is better than simple 'static' distribution, which would have put about 1½ loaded lanes on one box.

In the event, the initial estimates gave plate thicknesses fairly close to the final design values.

Initial Design Loads

The loading on the beam section on the previous page is given by:

Steel Weight say 12 kN/m

Concrete Weight $(5450 \times 300 + 1800 \times 70 + 900 \times 70) \times 10^{-6} \times 25 = 36$ kN/m
say 40 kN/m for outer box

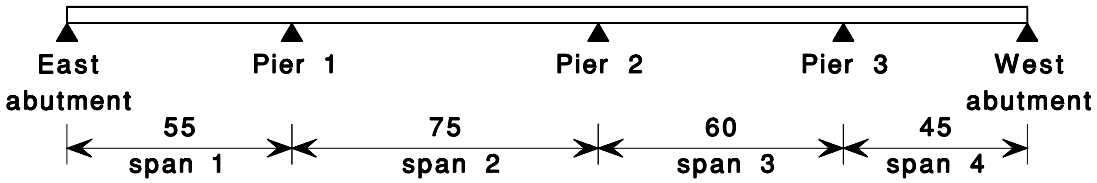
Surfacing Weight $(5.450 \times 0.100) \times 22 = 12$ kN/m

Parapets, barriers, etc. - say 10 kN/m

Live Loading
HB will be critical, but for initial design use 120% × HA loading
There are two notional lanes per carriageway
Maximum load effects will occur in an outer box
Assume that the outer box carries 1 lane of loading

Calculation of Load Effects

For distributions of moment and shears, consider a simple line beam model



From a computer analysis of this model, under UDL's of 1 kN/m, the moments are:

	Span 1	Pier 1	Span 2	Pier 2	Span 3	Pier 3	Span 4
All Spans	152	- 545	185	- 489	98	- 231	147
Span 1	270	- 230		89		- 20	
Span 2		- 384	309	- 397		87	
Span 3	32	87		- 229	239	- 176	
Spans 1+2	132	- 614	249	- 308		- 68	
Spans 2+3		- 297	250	- 626	125	- 89	

Commentary to calculation sheet

Maximum moments for udl based on results from line beam analysis.

Worst moment over main piers, with all spans loaded, is –545 kNm at pier 1, for 1kN/m.

Worst moment with spans 1 + 2 loaded (live load) is –614 kNm at pier 1.

The contribution from webs is ignored in this initial design.
The bottom flange is unstiffened and $\gamma_M = 1.05$ for design strength at extreme fibres (Clause 3/9.10.1)
Yield strength is for material over 40 mm thick

Section at Pier

The design moment at ULS are:

Dead load:

Steel Weight	$1.05 \times 12 \times -545$	= - 6 870 kNm
Concrete Weight	$1.15 \times 40 \times -545$	= -25 070 kNm
Surfacing	$1.75 \times 12 \times -545$	= -11 450 kNm
Furniture	$1.2 \times 10 \times -545$	= - 6 540 kNm

Live Load:

Loaded length	=	55 + 75	= 130 m (spans 1 & 2)	
HA UDL	=	22.1 kN/m (BD 37/01)		
Load	=	$22.1 \times 120\%$	= 290 kN/m	
M_{UDL}		$1.5 \times 26.6 \times -614$		= -24 500 kNm
KEL	=	120 kN × 120%	= 144 kN	
M_{KEL}		1.5×-0.1 WL		
	=	$1.5 \times -0.1 \times 144 \times 75$		= -1 620 kNm
			Total Moment	= -76 050 kNm

(M on bare steel = 31 940 M on composite section = 44 110)

Bottom Flange

Force in bottom flange $= \frac{76050}{3200} \times 10^3 =$ 23 770 kN

ULS strength $= \frac{335}{1.1 \times 1.05} =$ 290 N/mm^2

Required thickness $= \frac{23770 \times 10^3}{1600 \times 290} =$ 51.2 mm Say 60 mm

Commentary to calculation sheet

A conservative assumption of the share of moment carried by the reinforcement.

The values for shear due to unit udl are taken from the same analysis as that which gave the bending moments on Sheet 3.

An approximation to the shear capacity is obtained by determining the limiting shear stress with the web treated as though it were part of an unstiffened beam (Clause 3/9.9.2 and Figure 11) Yield strength is for material between 16 mm and 40 mm thick

Top Flange

ULS Strength $= \dfrac{335}{1.1 \times 1.05} =$ 290 N/mm^2

Assume reinforcement takes 10% of moment on composite section

Hence steel top flange has to resist 31 940 + 0.9 × 44 120 = 71 650 kNm

Required thickness $= \dfrac{71650 \times 10^6}{3200 \times 290 \times 1800} =$ 42.9 mm - say 45 mm

Webs

Based on values from line beam model with 1 kN/m, the design shears at ULS are:

Steel Weight	1.05 × 12 × 38.0	=	479
Concrete Weight	1.15 × 40 × 38.0	=	1748
Surfacing	1.75 × 12 × 38.0	=	798
Furniture	1.2 × 10 × 38.0	=	456
HA UDL	1.5 × 26.6 × 37.7	=	1504
HA KEL	1.5 × 144	=	216
	Total shear	=	5201 kN

i.e. 2600 kN per web

Assume $d_{we}/t = 150$, $\phi = 1$

Then $\tau_l \simeq 0.55 \times \tau_y = \dfrac{0.55 \times 345}{1.05 \times 1.1 \times \sqrt{3}} =$ 95 N/mm^2

To allow for interaction, utilise only half of τ_l - say 50 N/mm^2

Required thickness $= \dfrac{2600 \times 10^3}{(3200 - 45 - 60) \times 50} =$ 16.8 mm - say 20 mm

Commentary to calculation sheet

See sheet 3 for moments due to unit udl.

Again the contribution to moment capacity from the webs is neglected.

Yield strength is for material between 16 mm and 40 mm thick

The large area of the concrete slab will carry most of the compression force due to the bending moments.

Section at Midspan

Using output from line beam model, as before, design moments at ULS are:
Dead load:

Steel Weight	$1.05 \times 12 \times 185$	=	2 330 kNm
Concrete Weight	$1.15 \times 40 \times 185$	=	8 510 kNm
Surfacing	$1.75 \times 12 \times 185$	=	3 890 kNm
Furniture	$1.2 \times 10 \times 185$	=	2 220 kNm

Live Load:

Loaded length = 75 m (span 2)
HA UDL = 23.4 kN/m
Load = $23.4 \times 120\% = 28.1$ kN/m
M_{UDL} $1.5 \times 28.1 \times 309$ = 13 020 kNm
M_{KEL} 1.5×0.167 WL $= 1.5 \times 0.167 \times 144 \times 75 =$ 2 710 kNm

Moment on bare steel	10 840
Moment on long-term section	6 110
Moment on short-term section	15 730
Total Moment	= 32 680 kNm

Bottom Flange

Strength $= \dfrac{345}{1.1 \times 1.05} = 299$ N/mm^2

Required thickness $= \dfrac{32\,680 \times 10^6}{1800 \times 299 \times 1600} = 38.0$ mm - say 40 mm

Top Flange

Moments carried by the steel flange

load on bare steel	10 840		
load on long-term section: say	$20\% \times 6110$	=	1 220
load on short-term section: say	$10\% \times 15730$		1 570
	Total	=	13 630 kNm

Required thickness $= \dfrac{13\,630 \times 10^6}{1800 \times 299 \times 1800} = 14.1$ mm - say 20 mm to allow for the reduced effective area of the bare flange in midspan.

Webs

Use 15 mm for rigidity during transport/erection.

Commentary to calculation sheet

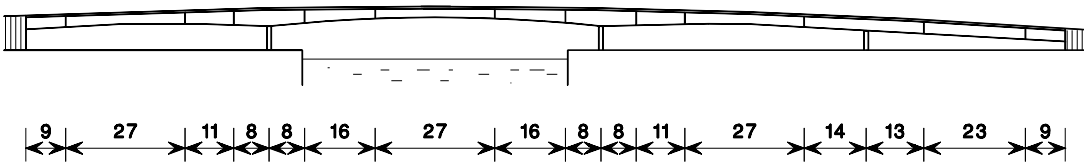
Erection considerations led to the positioning of splices to suit erection of girder sections up to 70 m long. These were assembled on the ground from shorter sections which were fully welded together before erection.

The dry land below spans 3 and 4 enabled them to be concreted whilst propped. This avoided the need for heavy flanges or haunching at Pier 3.

MAKE-UP

A similar procedure is adopted to choose preliminary material section size throughout all the spans.

Splice positions are chosen with regard to available plate length and to keep transport lengths within the limit of 27 m for unescorted transport by road.



Top	15	15	25	45	25	20	15	25	15	15	20-45-20	15	15
Web	15	15	15	20	15	15	15	20	15	15	15-20-15	15	15
Bot	25	40	50	60	50	40	50	60	50	50	35-60-35	40	25

Material

All material to be grade S355

Notch ductility to suit the minimum effective bridge temperature of –14°C

Grade designations to BS 10 025: 1993 will be:

Material up to 55 mm thick: S355 J2 (G3 or G4)
Material over 55 mm thick: S355 K2 (G3 or G4)

Commentary to calculation sheet

All global analysis was carried out using a grillage model. The effects of dead and live loads, erection sequence and concreting sequence were all evaluated.

The model was essentially comprised of two layers, referred to as the upper and lower layers.

The lower layer modelled the behaviour of the box sections. A single line of members along each box centreline were assigned the stiffness properties of the box (bare steel or composite, according to the stage considered) in bending, torsion and shear.

From each line, short very stiff 'dummy' members extended laterally to the two web lines.

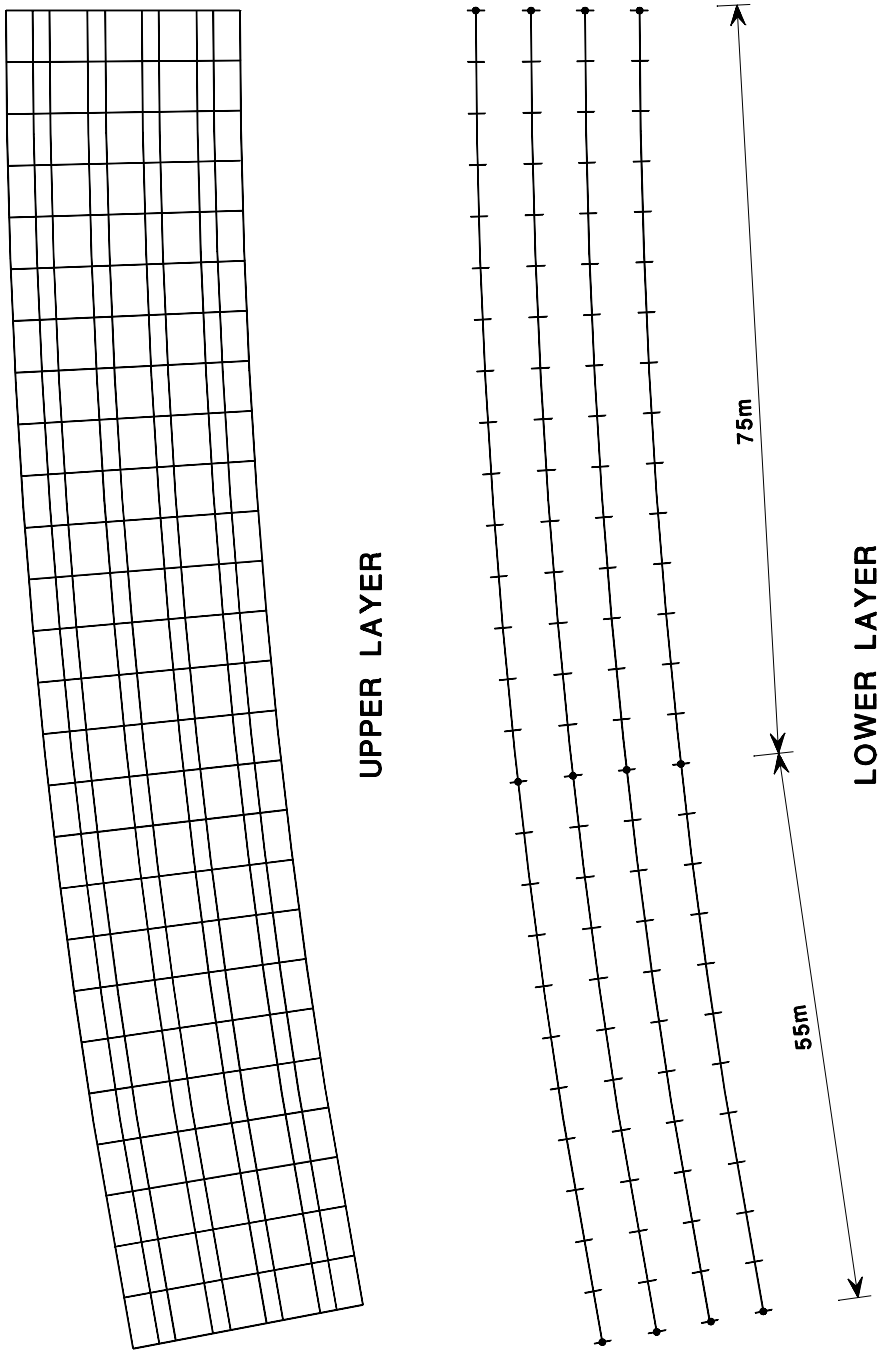
The upper layer modelled only the slab properties of bending and torsional stiffness. The edge beams were modelled with the deck slab, to facilitate application of loads.

The two layers were connected at common nodes on the web lines. These connections were only pin connections, no moment was transferred.

A representative portion of the grillage model is shown opposite.

Note: There is no bracing between any of the boxes, except for shallow cross beams at the piers.

DETAILED DESIGN - GLOBAL ANALYSIS



Commentary to calculation sheet

The section make-up is slightly different to that on sheet 7, as a result of intermediate calculations not included here.

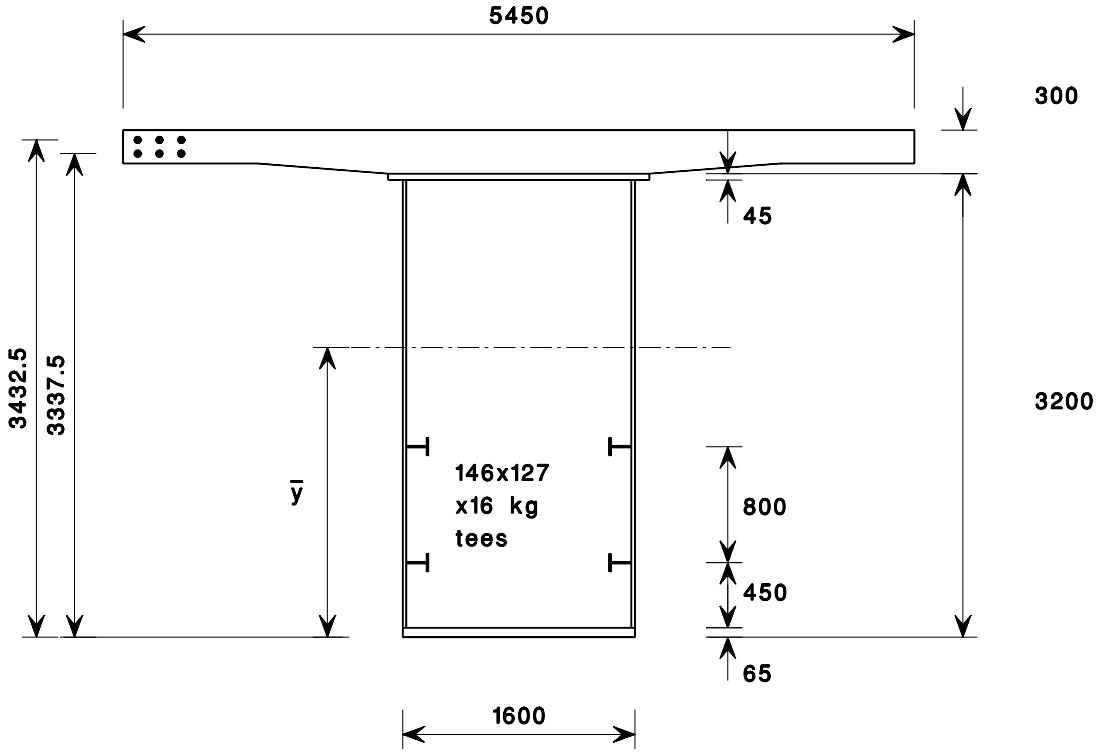
The section is of variable depth and with longitudinal stiffeners. The section properties for stress analysis may therefore use the full thickness of the web. There was no redistribution of stresses in the lower web panel, as will be seen later.

If there were a need to redistribute stresses, a reduced thickness of web (in the panel concerned) would be used in calculating the section properties, subject to the limitation of Clause 3/9.5.4.

The bottom flange is fully effective in compression ($b/t = 1560/65 = 24 \therefore K_c = 1.0$).

DESIGN OF BEAMS AT PIERS

Section Properties



The top and bottom reinforcement is T25 @ 150 crs (positioned inside the transverse bars)

The calculated section properties are:

	Bare Steel	Cracked Composite	
A	316 600	351 950	mm^2
$\bar{y}$	1475	1667	mm
I_{xx}	556 × 10^9	672 × 10^9	mm^4
$Z_{top\ flange}$	322 × 10^6	438 × 10^6	mm^3
$Z_{bottom\ flange}$	376 × 10^6	403 × 10^6	mm^3
Section Class	Non-compact	Non-compact	
I_{yy}	124.9 × 10^9		mm^4
Z_{pe}	384 × 10^6		mm^4

Commentary to calculation sheet

Slenderness of uniform rectangular or trapezoidal box sections, Clause 3/9.7.3.1. This is strictly only applicable when the section is uniform along the length of the beam, but as noted opposite it can be used to give an upper bound to the effective slenderness of a non-uniform section.

Limiting compressive stress is given by Clause 3/9.10, which refers to Clause 3/9.8 and Figure 11. The 'plateau' in Figure 11 extends as far as an effective slenderness of 30 (for $\ell_e/\ell_w = 1$)

Note that even though the section is torsionally stiff, it may need positive restraint at piers 1 and 2 during construction, since the boxes are each on a single bearing. A cross-beam is provided at these positions for that purpose. Cross-bracing was provided and removed after concreting.

LTB Slenderness

Since the box is relatively slender and may not be braced to other boxes during construction, check the LTB slenderness of the bare steel section.

For a box section $\lambda_{LT} =$ $2.25\ \eta\ \xi \sqrt{\dfrac{Z_{pe}\ \ell_e}{r_y \sqrt{AJ}}}$

Using section properties from sheet 9:

$r_y = \sqrt{\dfrac{I_{yy}}{A}} = \sqrt{\dfrac{124.9 \times 10^9}{316\,600}}$

$= 628$ mm

Torsional Inertia

$J = \dfrac{4A_o^2}{\sum B/t} = \dfrac{4 \times (1580 \times 3145)^2}{\left(\dfrac{1580}{45} + \dfrac{1580}{65} + 2 \times \dfrac{3145}{20}\right)} = 264 \times 10^9\ \text{mm}^4$

$\xi = \left[\dfrac{(I_x - I_y)(I_x - 0.385J)}{I_{x^2}}\right]^{0.25} = \left[\dfrac{(556 - 125)(556 - 0.385 \times 264)}{556^2}\right]^{0.25}$

$= 0.892$

Take $\eta = 1$

$\ell_e = 75$ m

Take as an upper bound to the slenderness of the whole span (which is of variable section) the slenderness calculated on the basis of the deepest section.

Then

$\lambda_{LT} = 2.25 \times 1.0 \times 0.892 \times \left[\dfrac{384 \times 10^6 \times 75\,000}{628 \times \sqrt{316\,600 \times 264 \times 10^9}}\right]^{0.5}$

$= 25$

The parameter for Figure 11 $= 25 \times \sqrt{(335/355)\ (376/384)} =$ 24

Which is less than 30

Hence the section can be fully stressed during concreting without the need for any intermediate or plan bracing.

(Also, line beam analysis of the concreting indicates that relative deflections between boxes will be small, so transverse bracing is not needed for load distribution.)

Commentary to calculation sheet

Similar tables of load effects can be compiled for Pier 2 and for the midspan region. Only the Pier 1 section is examined in the worked example; only the table of effects for Pier 1 is presented.

SUMMARY OF LOAD EFFECTS AT PIER 1

LOAD CASE			ULS γ_{fl}	MT	ULS MT	SHEAR	ULS SHEAR	TORSION	ULS TORSION
Steel Weight			1.05	-7,629	-8 011	530	557	0	0
Concrete			1.15	-18 996	-21 811	1 297	1 496	83	95
Bare Steel Total					-29 822		2 049		95
Concrete Long Term			1.15	-5 197	-5 977	443	510	154	177
Surfacing			1.75	-10 811	-18 920	499	873	214	374
Super Imposed			1.2	-3 093	-3 712	417	500	1 087	1 304
Shrinkage (Secondary M)			1.2	-6 615	-7 938	88	106		
Settlement			1.2	-536	-643	10	12		
Total (Long Term)					-37 190		2001		1 855
COMBINATION 1	MAX MT	HA	1.5	-12 236	-18 354	863	1 295	97	146
		HA+HB	1.3	-16 327	-21 225	1 129	1 468	1 427	1 855
		Footway	1.5	-193	-290	0	0	2	3
		Total			-21 515		1 468		1 858
	MAX SHEAR	HA	1.5	-10 990	-16 485	1 078	1 617	649	974
		HA+HB	1.3	-12 560	-16 328	1 399	1 819	2 744	3 567
		Footway	1.5	-193	-290	0	0	2	3
		Total			-16 618		1 819		3 570
	MAXTORSION	HA	1.5	-5 595	-8 393	598	897	1 008	1 512
		HA+HB	1.3	-10 868	-14 128	1 076	1 399	3 086	4 012
		Footway	1.5	-193	-296	0	0	2	3
		Total			-14 418		1 399		4 015
COMB'ATION 3		HA	1.25	-12 236	-15 295	1 078	1 348	97	121
		HA+HB	1.1	-16 327	-17 960	1 399	1 539	1 427	1 570
		Footway	1.25	-193	-241	0	0	2	3
		Diff Temp	1.0	-2 851	-2 851	49	49		
		Total			-21 052		1 588		1 573

Commentary to calculation sheet

Torsional warping is neglected at ULS, in accordance with Clause 3/9.2.1.3. Distortional warping is not neglected, for reasons discussed in the main text (Section 5.3.5).

Annex B to Part 3: Distortion and warping stresses in box girders.

Clause 3/B.3.2, corner stresses.

Similar considerations should be made for the distortional stiffness at midspan, where the box depth is less and the plate thicknesses are different.

The inverse of the β value opposite (13.1 m) represents the rate at which distortional effects reduce in a section comprising composite top flange, two webs and a bottom flange. This length is quite long; to be able to confine a 'panel length' to the much shorter length between cross-frames formed at each web stiffener position (every 1667 mm) would require very stiff cross-frames. For the initial evaluation shown opposite, cross-frames at 5 m centres are considered, to see what the distortional effects would be and how stiff the frames would need to be.

DISTORTIONAL EFFECTS

Distortion of the Box

Consider first the basic section, comprising webs and flanges. Calculate the short-term composite properties.

$$D_{YB} = \frac{Et_{B^3}}{12} = \frac{205\,000 \times 65^3}{12} = 4.69 \times 10^9 \text{ Nmm}$$

Similarly $D_{YC} = 0.1367 \times 10^9$ (20 mm web)
and $D_{YT} = 209 \times 10^9$ Nmm (45 mm flange and 300 mm slab)

Consider the stiffness of a section close to the pier, where depth = 2900 mm

$$\frac{D_{YB}}{D_{YT}} = \frac{4.69 \times 10^9}{209 \times 10^9} = 0.0224$$

$$\frac{D_{YT}}{D_{YC}} \frac{d}{B} = \frac{209 \times 10^9 \times (2900 - 110)}{0.1367 \times 10^9 \times 1560} = 2730$$

From Figure B.2(a) $R_D = 0.0029$

$$\text{Hence } K = \frac{24 D_{YT}}{B_T^3} R_D = \frac{24 \times 209 \times 10^9 \times 0.0029}{1580^3} = 3.69 \text{ N/mm}^2$$

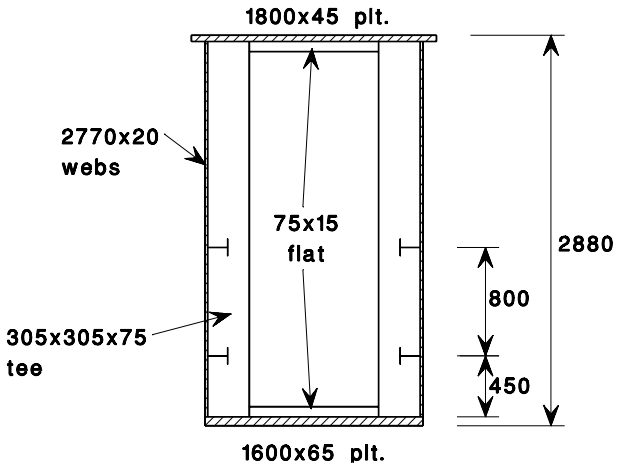
$$\text{and thus } \beta = \left(\frac{K}{EI_x}\right)^{0.25} = \left(\frac{3.69}{205\,000 \times 552 \times 10^9}\right)^{0.25} = 7.56 \times 10^{-5} \text{ mm}^{-1}$$

If cross-frames are provided at 5 m centres

$$\beta_{LD} = 0.0756 \times 5 = 0.38$$

Commentary to calculation sheet

In accordance with Clause 3/9.16.4.1, the effective widths of flange acting with these transverse stiffeners is given by Clause 3/9.15.2.1 and in this case are ¼ of the clear width. The effective width of web is given by Clause 3/9.13.2 and here is $32t_w$.



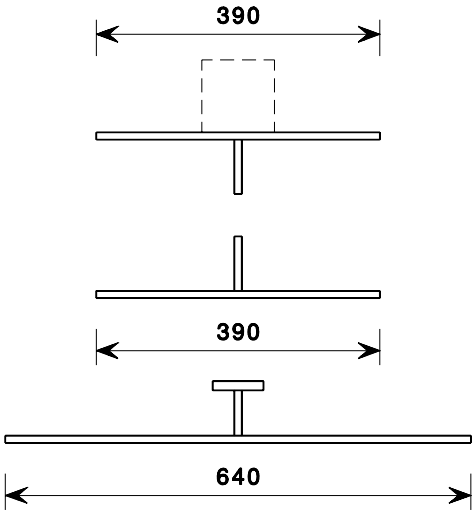
The stiffness of a effective ring-frame is given in Clause 3/B.3.4.3, as $\frac{K_R}{KL_D}$ *, where* K_R *is a stiffness per frame calculated in the same manner as for the box section. Values of less than unity are quite possible, they merely show that the stiffness of an individual frame is less than that of the box section summed over the length of one panel.*

The letter R is added to the suffixes of the variables D_{YT} *, etc., to show that they relate to ring frame stiffeners, not to the stiffness per unit length of box.*

Effective ring frames

Each ring frame is formed by Tee stiffeners welded on the webs and flats welded on the flanges.

From section property calculations the bending stiffnesses are:



Top Flange

Inertia = 422×10^6 mm^4

Bottom Flange

Inertia = 14.7×10^6 mm^4

Webs

Inertia = 413×10^6 mm^4

To calculate the effective stiffness of the ring frame, determine stiffnesses of flanges in web in a similar manner to that of the box section.

D_{RYT} = 86.5×10^{12} Nmm^2 (NB not per unit width)
D_{RYB} = 3.02×10^{12} Nmm^2
D_{RYC} = 84.6×10^{12} Nmm^2

Hence $\frac{D_{RYB}}{D_{RYT}} = 0.035$ and $\frac{D_{RYT}}{D_{RYC}} \frac{d}{B} = 1.83$

From Figure B.2(a) $R_D = 1.10$

Hence $K_R = \frac{24 D_{RYT} R_D}{B_T{}^3} = \frac{24 \times 86.5 \times 10^{12} \times 1.10}{1580^3} = 579 \times 10^3$ N/mm

And thus $S = \frac{K_R}{K L_D} = \frac{579 \times 10^3}{3.69 \times 5000} = 31$

The ring frames are therefore not stiff enough to be fully effective over a panel length of 5000 mm. See the limiting value of S in Table B.1 of Part 3, with a value of $\beta L_D = 0.38$.

Commentary to calculation sheet

The effective stiffness of the web stiffeners is 'smeared' along the box by dividing their stiffness by their spacing. The effective distortional stiffness of the section is thus increased significantly. If flange stiffeners were also provided, the stiffness would increase further, slightly.

The ring frames now add very little to the resistance to distortion; the distortional warping and bending stresses depend only on the stiffness of the box section (including the effect of the smeared stiffeners).

Note that the web stiffeners must be connected to the top flange, so that they can transfer the distortional transverse bending moments (Clause 3/9.16.2.3).

Consider the situation if the webs are stiffened at 1667 mm centres by Tee sections. The box section stiffness is increased; determine what effect this has on the effectiveness of the frames.

The D_{YC} parameter is increased to $\frac{84.6 \times 10^{12}}{1667} = 50.8 \times 10^{9}$ Nmm

and thus $\frac{D_{YT}}{D_{YC}} \frac{d}{B} = 7.36$

R_D = 0.41 from Figure B.2(a)
K = 520 Nmm
βL_D = 1.30 (β = 0.260 m^{-1})
S = 0.22

Since the cross-frames are too weak (relatively), consider the maximum values of distortional warping and bending stresses.

When $\beta_{LD} > 1.0$, warping stresses due to concentrated loads are limited to the value given by Clause 3/B.3.2(b).

The warping stress due to distributed torque is limited to the value given by Clause 3/B.3.2(a) when $\beta_{LD} = 1.6$, that is:

$$\sigma_{DW} = \frac{0.6 T_{UD} \bar{y}}{\beta^2 B_T I_x}$$

Similarly, the distortional bending stresses due to concentrated loads are limited to the value given by Clause 3/B.3.4.2(a) when $\beta_{LD} = 2.0$; the stresses due to distributed load are limited to the values given by B.4.2(b) when $\beta_{LD} = 2.65$, that is:

$$\sigma_{DB} = \frac{T_{UDL} F_D}{B_T Z}$$

For the present section:

$V_D = 0.129$ (Figure B.3(a)) and $Z = 2.01 \times 10^6/1667 = 1206$ mm^2

At the top flange $F_D = \frac{B}{2}(0.5 - V_D) = \frac{1580}{2} \times 0.371 = 293$ mm

Commentary to calculation sheet

Stresses are calculated for the cracked section. Stresses in the reinforcement and crack widths can be calculated from the load effects. Only the stresses in the steel section are presented in this example.

The connection of the Tee web stiffeners to the bottom flange creates a class G fatigue details. Separate calculations for fatigue considerations showed a worst stress range of 13 N/mm² in the bottom flange in span 1, compared with a limit of 16 N/mm² derived from Figure 8 of Part 10 for a dual 2-lane all purpose road and for a span of 55 m.

Note that if the cross-frames had been effective, the distortional warping stress would have been, from Clause 3/B.3.2(a):

$$\sigma_{DW} = \frac{T_{UD}\,\bar{y}\,L_D^2}{4.5 B_T I_x} = \frac{920 \times 10^6/5000 \times 1510 \times 5000^2}{4.5 \times 1580 \times 552 \times 10^9} = 1.8\ N/mm^2$$

Bending resistance is not limited by LTB, so $M_R = M_{ult}$ and thus the limiting stress given by Clause 3/9.10.1.1 may be based simply on the yield stress. Note that $\gamma_M = 1.05$ for this clause.

The most severe bending stresses occur with HB loading, combination 1. Then, the total stresses are:

Top flange $\frac{29822}{322} + \frac{37190 + 21515}{438} = 227 \text{ N/mm}^2$

Bottom flange $\frac{29822}{376} + \frac{(37190 + 21515)}{403} = 225 \text{ N/mm}^2$

Consider distortional warping stresses and interaction with shear stresses.

From the grillage analysis, the maximum torque increment over a 5 m length is 920 kNm. Consider this as a udl torque.

For a udl torque of this value, the warping stress would not be greater than that given by:

$\sigma_{DW} = \frac{0.6 T_{UD} \bar{y}}{\beta^2 B_T I_x} = \frac{0.6 \times 920 \times 10^6 / 5000 \times 1510}{(0.260 \times 10^{-3})^2 \times 1580 \times 552 \times 10^9} = 2.8 \text{ N/mm}^2$, at the bottom flange and a similar value at the top flange.

The total longitudinal stresses at the extreme fibres of the steel box are:

Top flange $227 + 3 = 230 \text{ N/mm}^2 < 335/1.1 \times 1.05 = 290 \text{ N/mm}^2$ OK

Bottom flange $225 + 3 = 228 \text{ N/mm}^2 < 290 \text{ N/mm}^2$ OK

The distortional bending stress at the top flange is given by:

$\sigma_{DB} = \frac{T_{UDL} F_D}{B_T Z} = \frac{920 \times 10^6 / 5000 \times 293}{1580 \times 1206} = 28 \text{ N/mm}^2$

Commentary to calculation sheet

Bending shear stress $\tau = \dfrac{V}{2 d_{\mathrm{w}} t_{\mathrm{w}}}$ *where* d_w *is measured vertically*

Torsional shear stress $\tau = \dfrac{T}{2 A t_{\mathrm{w}}}$

The inclined bottom flange will carry part of the bending shear in most of the web panel but this has been neglected. The bottom flange changes direction close to the diaphragm (see sectional elevation facing Commentary to Sheet 18) and the web adjacent to the diaphragm has to carry the full shear without contribution from the flange.

Yielding of web panels, Clause 3/9.11.3.

WEB PANELS AT PIER 1

The coexistent load effects, for the maximum bending shear loading condition are:

	Shear (kN)	Torsion (kNm)	Moment kNm	Bending Stress	
				Top	Bottom
Dead Load	4050	1950	67 012	-178	172
Live Load (HB + F'way)	1468	3570	16 618	-38	41
Warping				-3	3
	5518	5520	83 630	-219	216

The shear stresses in the web are:

Due to bending: $\dfrac{5518 \times 10^3}{2 \times 3090 \times 20} = 45 \text{ N/mm}^2$

Due to torsion: $\dfrac{5520 \times 10^6}{2 \times (1580 \times 3145) \times 20} = 28 \text{ N/mm}^2$

Beam is longitudinally stiffened.
Consider the individual web panels

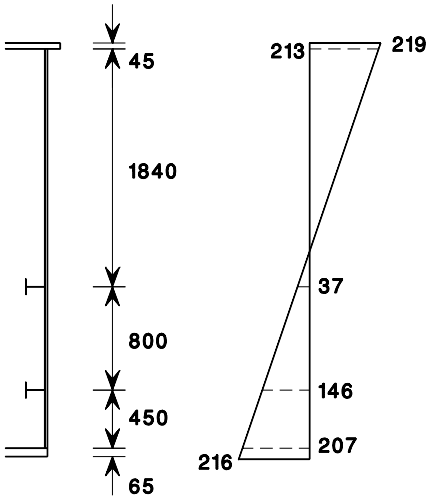
The first vertical stiffener is 1667 mm from the diaphragm

Lower web panel

$a = 1667, \quad b = 450, \quad t = 20$

$\sigma_1 = \frac{1}{2}(207 + 146) = 177 \text{ N/mm}^2$

$\sigma_b = \frac{1}{2}(207 - 146) = 31 \text{ N/mm}^2$



Check for yielding

$\sigma_{1e} = 177 + 0.77 \times 31 = 201 \text{ N/mm}^2$

$\tau = 45 + 28 = 73 \text{ N/mm}^2$

$(\sigma_{1e}^2 + 3\,\tau^2)^{1/2} = (201^2 + 3 \times 73^2)^{1/2} = 237 < \dfrac{345}{1.05 \times 1.1} = 299 \text{ N/mm}^2$

Commentary to calculation sheet

Buckling of web panels, Clause 3/9.11.4.

Since $\lambda < 24$, use the formula in 3/9.11.4.3.2 for K_1.
Similarly, use the formula in 3/9.11.4.3.3 for K_q.

The upper web panel is deemed to be satisfactory by inspection.

Check for buckling

$\lambda = \frac{450}{20} = 22.5 < 24 \therefore$ panel may be treated as restrained

$\phi = \frac{1667}{450} = 3.7$

Hence $K_1 = (20/450)^2 \times 204500/355 = 1.14$

$K_q = (20/450)^2 \times 435000(1+(450/1667)^2/355 = 2.60$

$K_b = 1.26$ (Figure 23(c))

$m_c = \frac{177 \times 1.05 \times 1.1}{345 \times 1.14 \times 1.0} = 0.52$ ($\rho = 0$ for restrained panel)

$m_b = \left(\frac{31 \times 1.05 \times 1.1}{345 \times 1.26 \times 1.0} \right)^2 = 0.01$

$m_q = \left(\frac{73 \times 1.05 \times 1.1}{345 \times 2.60} \right)^2 = 0.01$

$m_c + m_b + 3\, m_q = 0.52 + 0.01 + 3 \times 0.01 = 0.56 < 1$ OK

Middle panel

Buckling check

$a = 1667 \quad b = 800$

$\lambda = \frac{800}{20} = 40 \qquad \phi = \frac{1667}{800} = 2.1$

$\sigma_1 = ½ (146 + 37) = 92$ N/mm^2

$\sigma_b = ½ (146 - 37) = 55$ N/mm^2

$K_1 = 0.77$ (Figure 23(a))

$K_q = 0.98$ (Figure 23(b))

$K_b = 1.22$ (Figure 23(c))

$m_c = \frac{92 \times 1.05 \times 1.1}{345 \times 0.77} = 0.40 \qquad m_b = \left(\frac{55 \times 1.05 \times 1.1}{345 \times 1.22} \right)^2 = 0.02$

$m_q = \left(\frac{73 \times 1.05 \times 1.1}{345 \times 0.98} \right)^2 = 0.06$

$m_c + m_b + 3\, m_q = 0.40 + 0.02 + 3 \times 0.06 = 0.60 < 1$ OK

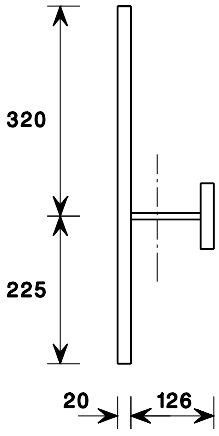
Commentary to calculation sheet

Longitudinal web stiffeners, Clause 3/9.11.5.

The longitudinal Tee stiffeners are continuous through the intermediate transverse stiffeners on the web.

Note that $\gamma_M = 1.2$ *for this clause*

Longitudinal Web Stiffeners



Stiffener is a 146 × 127 × 16 kg Tee

$16\ t_w = 16 \times 20 = 320$ mm

$b/2$ = 225 mm below, 400 mm above

Effective stiffener section properties

$A_{se} = 128\ cm^2$
$I_{xx} = 2270\ cm^4$
$r_{se} = 4.21$ cm

$$\lambda = \frac{a}{r_{se}}\sqrt{\frac{\sigma_{ys}}{355}} = \frac{1667}{42.1} = 40$$

$$k_s = 0.158$$

$$\frac{\sigma_{\ell s}}{\sigma_{ys}} = 0.775 \text{ (Figure 24)}$$

$$\sigma_{\ell s} = 0.775 \times 345 = 267\ N/mm^2$$

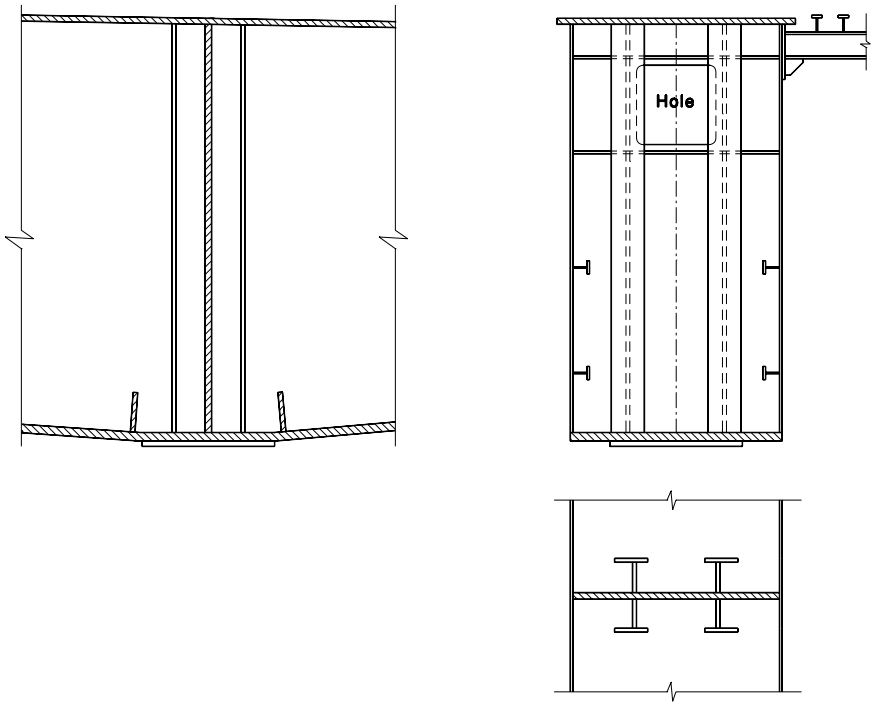
$$\sigma_{se} = \sigma_1 + \left(2.5\tau + \frac{a^2}{b^2}\sigma_2\right)\frac{bt_w k_s}{A_{se}}$$

$$\sigma_{se} = 149 + (2.5 \times 73 + 0)\ \frac{(450 + 800)/2 \times 20 \times 0.158}{12800}$$

$$= 177\ N/mm^2$$

$$< \frac{275}{1.2 \times 1.1} = 208\ N/mm^2$$

Commentary to calculation sheet



In practice, step irons were fixed up both sides of the diaphragm, for access, and grab rails at the top. These are not shown here.

Design of stiffened diaphragm, Clause 3/9.17.6. Stresses are calculated at the corners of the plate panels in accordance with 3/9.17.6.2.1. It is generally presumed by the code that there will be two bearing stiffeners over each bearing in a stiffened diaphragm; the reaction is taken to be on the lines of the two stiffeners, for the purpose of calculating shear (or at j/4 from the inner edge for calculating moments between twin bearings).

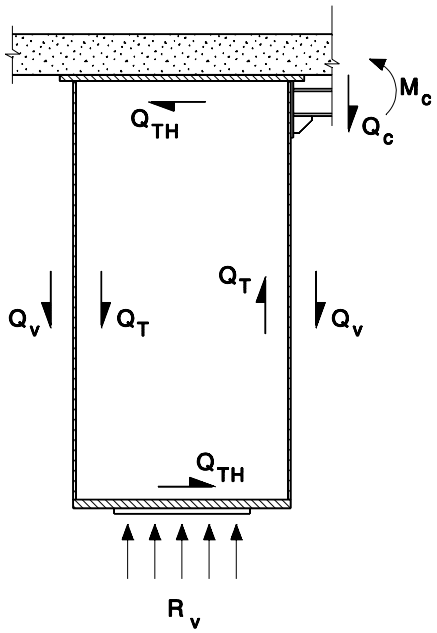
The moment on the diaphragm specified by Clause 3/9.17.6.2.3(a) is intended to be used in deriving stresses at the corners of the plate panels. It is not the same as a simple 'static' moment: note in particular that K_d is "to allow for the effects of boundary shears and should be taken as 2.0 ……". The moment from the cross-beam is implicitly taken into account by the inclusion of Q_T in the expression for M.

The expression for shear flow between bearings was introduced to allow for error in planarity/alignment of bearings and is more appropriate to the situation where there is a stiff bearing below the diaphragm - the load could be shared unevenly between the two stiffeners. There is a good case for taking a much lesser value where an elastomeric pot bearing is used since there will be no moment transmitted through the bearing itself; the only transverse moment on the diaphragm would be that due to any eccentricity.

The horizontal shear, Q_h, is that due to skidding forces - strictly that is a Combination 4 load but it has been included here

DIAPHRAGM AT PIER 1

Maximum reaction at ULS = 13 136 kN
Maximum eccentricity longitudinally = 50 mm



Forces on diaphragm from grillage analysis
River Side Shear = 6620 kN
Land Side Shear = 6194 kN
Cross Beam Shear = 322 kN

Single bearing offers zero torsional restraint.
Some torque is transferred to the composite cross-beam:
from global analysis T = 490 kNm
(which means that M= 490–322×0.8 = 232 kNm at the connection to the web).

$$Q_T = \frac{490}{2 \times 1.58} = 155 \text{ kN}$$

Q_V = ½ (6620 + 6194) = 6407 kN

Stresses are to be calculated at the edge of the outer and inner panels along the connection line. Moment on diaphragm is:

$$M = (K_d\, Q_V + 2Q_T)x_W + K_d\, Q_c\, x_c + \Sigma\,(P_i\, x_i) - R_v x_b + \left(\frac{Q_{fv}\,\ell_f}{2}\right) \text{ (Cl 3/9.17.6.2.3(a))}$$

No wheel loads over diaphragm $\therefore\ \Sigma P_i\, x_i$ term = 0; x_b = 0 for single bearing
No change in flange slope $\therefore\ Q_{fv}$ term = 0
For K_d = 2
$M = (2 \times 6407 - 2 \times 155)\, x_w + 2 \times 322 \times x_c = 13148\, x_w$ ($x_c = x_w$)

Shear flow between box web and bearing stiffener

$$q = (Q_v + Q_T + Q_{fv} + Q_c + \Sigma P_i)/D_e + Q_h/B_e \text{ (Clause 3/9.17.6.2.4(a))}$$

$$= \left(\frac{6407 - 155 + 322}{3090} + \frac{375}{1560}\right) \times 10^3 = 2370 \text{ N/mm at inner web } (Q_{fv} \text{ \& } P_i = 0)$$

Shear flow between bearing stiffeners

$$q = \left(\frac{Q_v}{4} + \frac{T}{S_s} - Q_T\right)/D_e + Q_h/j \text{ (Clause 3/9.17.6.2.4(d))}$$

$$j = 1000 + 3 \times 50 = 1150 \text{ mm}$$

$$q = \left(\frac{6407}{4} + \frac{490}{0.68} - 155\right) \times \frac{10^3}{3090} + \frac{375 \times 10^3}{1150} = 1027 \text{ N/mm}$$

Commentary to calculation sheet

Stresses in diaphragm plates, Clause 3/9.17.6.2.

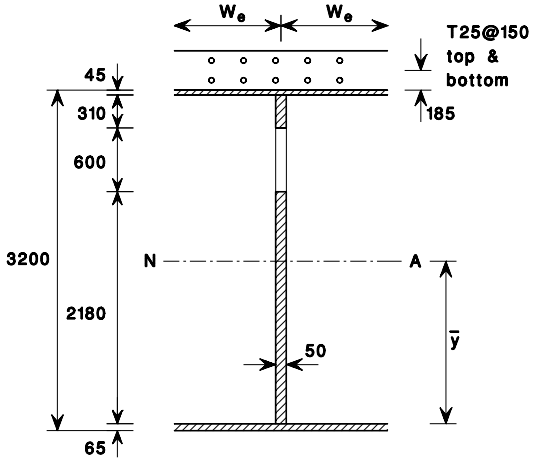
Effective section, Clause 3/9.17.4.2.

The transverse reinforcement is T25 at 150mm centres. This was shown in separate calculations to be adequate for transverse moments in the deck slab (hogging over the box webs).

In accordance with Clause 3/9.17.6.2.2, vertical stress in the panel, σ_{d1}, is neglected, since the bottom flange is parallel to the top at the diaphragm position and stresses due to local wheel loads are small, but σ_{d1} due to action as part of the bearing stiffener will have to be taken into account later.

Stresses in outer panels

$x_w = x_c = 790 - 340 = 450$ mm
$W_e = ¼ \times 450 = 112$ mm
$A_r = \frac{2 \times 112}{150} \times 491 \times 2 = 1466$ mm^2



Properties of effective section are:

Area = 180 700mm^2 $\bar{y} = 1585$ mm
$I = 188.7 \times 10^9$ mm^4
Section moduli: $Z_t = 120.2 \times 10^6$ mm^3
$Z_b = 124.2 \times 10^6$ mm^3

Moment $13148 \times 0.45 = 5\,920$ kNm

Horizontal stress $\sigma_{d2} = \frac{5920 \times 10^6}{120.2 \times 10^6} = 49$ N/mm^2 (tension) at top of diaphragm

$\sigma_{d2} = \frac{5920 \times 10^6}{124.2 \times 10^6} = 48$ N/mm^2 (compression) at bottom

Shear stress $\tau = \frac{2370}{50} = 47$ N/mm^2

Stresses in inner panels

The effective widths are the same, but deduct the manhole.
Properties of effective section are:

Area = 150 700 mm^2 $\bar{y} = 1394$ $I_{NA} = 154.7 \times 10^9$ mm^4
Section moduli: $Z_t = 87.8 \times 10^6$ mm^3 $Z_b = 116.4 \times 10^6$ mm^3

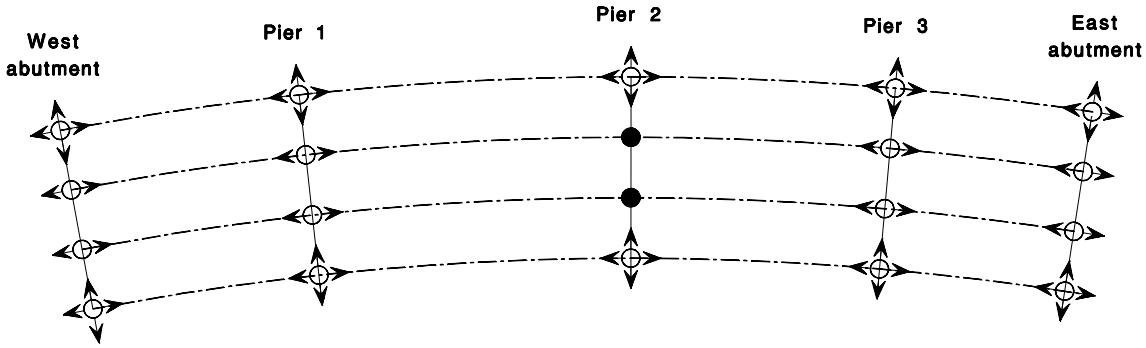
Horizontal stress $\sigma_{d2} = \frac{5920 \times 10^6}{87.8 \times 10^6} = 67$ N/mm^2 (tension) at top

$\sigma_{d2} = \frac{5920 \times 10^6}{116.4 \times 10^6} = 51$ N/mm^2 (compression) at bottom

Shear stress $\tau = \frac{1027}{50} = 21$ N/mm^2

Commentary to calculation sheet

The articulation arrangements for the bridge are as shown below.



The maximum eccentricity at Pier 1 is calculated on the basis of thermal expansion (from the fixed point at Pier 2), rotation of the beams in their planes (load on the span will result in a displacements at bottom flange level) and allowances for setting etc. A 'rounded' value of 50 mm was derived on this basis.

An alternative arrangement with bearings fixed longitudinally at both Pier 1 and Pier 2 was considered in the actual design. In that case the piers were considered to flex slightly as a result of the relative displacements resulting from applied and thermal loads.

Stress in bearing stiffeners, Clauses 3/9.17.6.6 and 3/9.14.4.2.

Note that in calculating the bearing stress, account is taken of the eccentricity of the reaction relative to the centroid of the bearing area.

Effective stiffener section, Clause 3/9.17.4.4.

Note that, strictly, the opening in the diaphragm does not comply with 3/9.17.2.8(a), since it is closer than 12t to the connection line of the bearing stiffeners. Only the actual width is included in the effective section at the opening. Arguably, the variation of load in the stiffener section (to zero at the top) should be modified (as required by Clause 3/9.17.6.3.2 when there are openings between the stiffener and the web).

BEARING STIFFENERS

P_s = 13 136 kN
e_x = 20 mm
e_y = 50 mm

Bearing stress at bottom of diaphragm

Pot bearing diameter = 650 mm

Dispersal through plates and bottom flange = 2 × (65 + 85) tan 60° = 520 mm

Bearing area = (650 + 520) × 50 + 4 (220 × 30 + 250 × 30)
= 114.9×10^3 mm^2

Modulus for longitudinal bending = 9.6×10^6 mm^2

$$\text{Stress} = \frac{13136 \times 10^3}{114.9 \times 10^3} + \frac{13136 \times 10^3 \times 50}{9.6 \times 10^6} = 114 + 68 = 182 \text{ N/mm}^2$$

$$\text{Limiting stress} = \frac{1.33\,\sigma_{ys}}{\gamma_m \gamma_{f3}} = \frac{1.33 \times 345}{1.05 \times 1.1} = 397 \text{ N/mm}^2 > 182 \text{ N/mm}^2 \quad \text{OK}$$

Stress on effective stiffener section

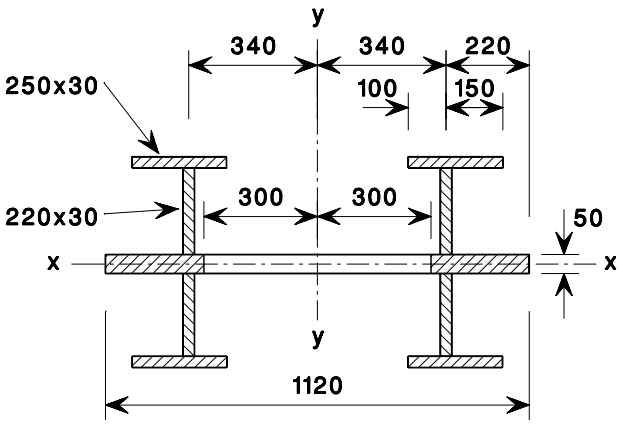
Section for calculation of vertical stresses

Full Section

A_{se} = 112 400 mm^2
I_{xx} = 2.63×10^9
I_{yy} = 13.06×10^9

Section at Opening

A_{se} = 82 400 mm^2
I_{xx} = 2.62×10^9
I_{yy} = 12.16×10^9



Commentary to calculation sheet

Vertical stresses in bearing stiffeners, Clause 3/9.17.6.3.2.

Bending stresses in bearing stiffeners, Clause 3/9.17.6.3.3.

The loads and stresses vary linearly to zero at the top of the diaphragm. To check stresses in the upper panels, where there is a hole, the stress gradient is the slightly higher value calculated on the basis of the properties at opening.

Yielding of diaphragm stiffeners, Clause 3/9.17.6.6.

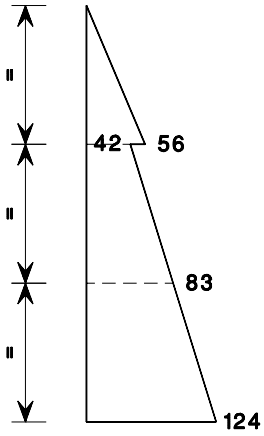
Note that σ_{1sT} is taken on the connection line, as shown in Figure 32 of Part 3, which is referred to from Clause 3/9.17.6.3.2.

Equivalent stress for buckling check, Clause 3/9.17.6.3.4.

Using section properties for the diaphragm in the middle panels, the stress σ_{2d} varies from +51 N/mm^2 at the bottom to –67 N/mm^2 at the top, i.e. from +12 N/mm^2 at the lower ⅓ point to -28 N/mm^2 at the upper ⅓ point (compression positive).

Bearing Stiffener Stresses

$T_s = 13\ 136 \times 0.020 = 263$ kNm
$M_s = 13\ 136 \times 0.050 = 657$ kNm



Vertical Stress
At the bottom of the diaphragm:

$$\sigma_{1s} = \frac{P_s}{A_{se}} = \frac{13\,136 \times 10^3}{112\,000} = 117 \text{ N/mm}^2$$

$$\sigma_{1sT} = \frac{T_s x}{I_{yse}} = \frac{263 \times 10^6 \times 340}{13.06 \times 10^9} = 7 \text{ N/mm}^2$$

Along the connection line:
$\sigma_{d1} = \sigma_{1s} + \sigma_{1sT} = 117 + 7 = 124$ N/mm^2

(Using section properties at opening, $\sigma_1 = 160$ N/mm^2, $\sigma_{1sT} = 8$ N/mm^2)

Out-of-plane bending stress σ_{bs}

$$\sigma_{bs} = \frac{657 \times 10^6 \times (25 + 220 + 30)}{2.63 \times 10^9} = 69 \text{ N/mm}^2$$

Stiffener Yielding

$$\sigma_{1s} + \sigma_{1sT} + \sigma_{bs} = 117 + 7 + 69 = 193 \text{ N/mm}^2 \le \frac{\sigma_{ys}}{\gamma_m \gamma_{f3}} = \frac{345}{1.1 \times 1.05}$$

$= 299$ N/mm^2 OK

Stiffener Buckling

$$\sigma_{se} = \sigma_a + \frac{1}{A_{se}} \left(\frac{\sigma_q \times \ell_s^2 \times t_d \times k_s}{a} \right)$$ at the third point ($A_s = 0$)

$\sigma_a = \sigma_{1s} + \sigma_{1sT}$
$\sigma_a = \frac{2}{3} \times 124 = 83$ N/mm

$\sigma_q = \sigma_{2s}$ = Average horizontal stress at centreline bearing stiffener within middle third.

$\sigma_q = ½\ (12 - 28) = -8$ N/mm^2

Commentary to calculation sheet

Buckling of diaphragm stiffeners, Clause 3/9.17.6.7. Note that $\gamma_M = 1.2$ for this clause.

Yielding of diaphragm plate, Clause 3/9.17.6.4.

Checks are shown opposite for four panel corners. Generally all panel corners should be checked; here the remainder are deemed satisfactory by inspection. Clause 3/9.17.6.4 actually calls for adequacy "at all points in every panel", whereas Clause 3/9.17.6.2.1 calls for stresses to be calculated at the corners of each plate panel. Checks at the corners are sufficient.

Stiffener $r_{se} = \sqrt{\dfrac{2.63 \times 10^9}{112400}} = 153$ mm

a = Half of panel widths on either side = 1120/2 = 560 mm

$$\lambda = \frac{\ell_s}{r_{se}} \sqrt{\frac{\sigma_{ys}}{355}} = \frac{3090}{153} \times \sqrt{\frac{345}{355}} = 20$$

From Figure 24 $K_s = 0.044$

$$\therefore \sigma_{se} = 83 + \frac{1}{112400}\left(\frac{-8 \times 3090^2 \times 50 \times 0.045}{560}\right)$$

$$= 83 - 2 = 81 \text{ N/mm}^2$$

Limiting stress From Figure 24 for $\lambda = 20$

$\sigma_{\ell s} = 0.955 \times 345 = 339$ N/mm^2

$$\therefore \frac{\sigma_{se}}{\sigma_{\ell s}} + \frac{\sigma_{bs}}{\sigma_{ys}} = \frac{81}{339} + \frac{69}{355} = 0.44 < \frac{1}{1.1 \times 1.2} = 0.76 \quad \text{OK}$$

$\therefore$ Diaphragm stiffeners safe in Buckling

Yielding of diaphragm plate

Outer panels:

Top of diaphragm

$\sigma_{d1} = 0$, $\sigma_{d2} = 49$, $\tau = 47$

$$(49^2 + 3 \times 47^2)^{1/2} = 95 \text{ N/mm}^2 < \frac{\sigma_y}{\gamma_m \gamma_{f3}} = \frac{335}{1.1 \times 1.05} \quad 290 \text{ N/mm}^2 \quad \text{OK}$$

Bottom of diaphragm

$\sigma_{d1} = 124$, $\sigma_{d2} = 48$, $\tau_{d1} = 47$

$(124^2 + 48^2 - 48 \times 124 + 3 \times 47^2)^{1/2} = 135$ N/mm$^2 \leq$ 290 N/mm^2 OK

Inner panels:

Top of diaphragm

$\sigma_{d1} = 0$, $\sigma_{d2} = 67$, $\tau = 21$

$(67^2 + 3 \times 21^2)^{1/2} = 76$ N/mm$^2 <$ 290 OK

Bottom of diaphragm

$\sigma_{d1} = 124$, $\sigma_{d2} = 51$, $\tau_{d1} = 21$

$(124^2 + 51^2 - 124 \times 51 + 3 \times 21^2)^{1/2} = 114$ N/mm$^2 \leq$ 290 N/mm^2 OK

Commentary to calculation sheet

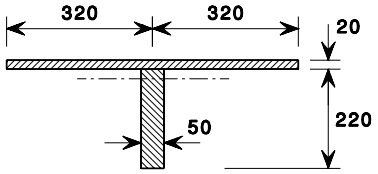
The effective section given in Clause 3/9.17.4.5 is not appropriate; the bearing stiffener and half of the plate have already been taken in the effective section of bearing stiffener.

The web panel dimensions are given on Sheet 16.

Axial force due to tension field action, Clause 3/9.13.3.2. Here τ_0 *is shown to be in excess of* τ_y*, so there will be no tension field action.*

WEB / DIAPHRAGM JUNCTION AT PIER

Effective section:



Web: $16 \times t_w = 320$ each side

Diaphragm: half of plate between web and stiffener $= \frac{780 - 340}{2} = 220$

Section properties are:

Area $= 23\,800$ mm^2

$I_{xx} = 439 \times 10^6$ mm^4 $\quad Z_{xx} = 1.37 \times 10^6$ mm^3

$I_{yy} = 129 \times 10^6$ mm^4 $\quad$ Web $Z_y = 1.98 \times 10^6$ Diaphragm $Z_y = 0.738 \times 10^6$

$$r_{se} = \sqrt{\frac{129 \times 10^6}{23800}} = 74\text{mm} \quad l_s = 3090 \quad \lambda = 42$$

From Figure 24 $\quad k_s = 0.17 \quad \sigma_{ls} = 0.77 \times 335 = 258$ N/mm^2

Consider max shear case

Stresses as sheet 16. Loads are:

1. From cross beam:

$P = 322$ kN (From grillage analysis)

2. From tension field action:

In bottom panel $\quad a = 1667 \quad b = 450 \quad t_w = 20 \quad \sigma_1 = 177$

$$2.9E\left(\frac{t_w}{b}\right)^2 = 2.9 \times 205 \times 10^3 \times \left(\frac{20}{450}\right)^2 = 1174 > \sigma_1$$

$$\tau_o = 3.6 \times 205 \times 10^3 \left[1 + \left(\frac{450}{1667}\right)^2\right]\left(\frac{20}{450}\right)^2 \sqrt{1 - \frac{177}{1174}}$$

$$= 738 \times 10^3 \times 1.073 \times 0.001975 \times 0.922 = 1440 \text{ N/mm}^2$$

$\tau_o > \tau \therefore$ No tension field action

In middle panel $\quad a = 1667 b = 800 \quad \sigma_1 = 92$

$$2.9E\left(\frac{t_w}{b}\right)^2 = 372 > \sigma_1$$

$$\tau_o = 3.6 \times 205 \times 10^3 \left[1 + \left(\frac{800}{1667}\right)^2\right] \times \left(\frac{20}{800}\right)^2 \sqrt{1 - \frac{92}{372}}$$

$$= 738 \times 10^3 \times 1.230 \times 6.25 \times 10^{-4} \times 0.868 = 492 \text{ N/mm}^2$$

$\tau_o > \tau \therefore$ No tension field action

Commentary to calculation sheet

Axial force representing the destabilising effect of the web, Clause 3/9.14.3.2

In upper panel $a = 1667$ $b = 1840$ $\sigma_1 = 91$ tension

$$2.9E\left(\frac{t_w}{b}\right)^2 = 70 > \sigma_1$$

$$\tau_o = 3.6 \times 205 \times 10^3 \left[1+\left(\frac{1840}{1667}\right)^2\right]\times\left(\frac{20}{1840}\right)^2\sqrt{1-\frac{-91}{70}}$$

$$= 738 \times 10^3 \times 2.22 \times 1.18 \times 10^{-4} \times 1.52 = 294 \text{ N/mm}^2$$

$\tau_o > \tau$ No tension field action

3. Web destablilising force

$\sigma_1 = \frac{1}{2}(207 - 213) = -3$ $\sigma_b = \frac{1}{2}(207 + 213) = 210$

$\Sigma A_c = 2 \times 20 = 40 \text{ cm}^2 = 4000 \text{ mm}^2$

$$\sigma_R = \left(1 + \frac{2\times 2000}{3090\times 20}\right)\left(-3+\frac{210}{6}\right) = 34 \text{ N/mm}^2$$

$$F_{wi} = \frac{3090^2}{1667} \times 20 \times 0.17 \times 34 \times 10^{-3} = 662 \text{ kN}$$

Inertia for two effective stiffener sections, each made up of 146 × 127 × 16 kg Tee and 640 ×20 mm plate = 4680 cm^4

$$\therefore \eta_s = \frac{1}{1+\dfrac{0.5 \times 4680\times 10^4 \times 3090^3}{129\times 10^6 \times 1667^3}} = 0.464$$

(contribution from stiffness of web plate neglected in the above expression)

Take $F_{wi} = 0.464 \times 662 = 307$ kN

Eccentricity of $F_{wi} = 65 - \frac{20}{2} = 55$ mm

Total load in web/diaphragm junction

$P = 307 + 322 = 629$ kN

$$\text{Axial stress} = \frac{629\times 10^3}{23800} = 26 \text{ N/mm}^2$$

Bending moment

$M_T = 307 \times 0.055 = 16.9$ kNm

$$\text{Bending stress in web} = \frac{16.9}{1.98} = 9 \text{ N/mm}$$

$$\text{Bending stress in diaphragm} = \frac{-16.9}{0.738} = -23 \text{ N/mm}$$

Maximum stress = 26 + 9 = 35 N/mm^2 < 290 N/mm^2 OK

Commentary to calculation sheet

Limiting stress $\sigma_{\ell s}$ from Sheet 24.

Check buckling strength

Assume cross beam load reduces linearly to zero at bottom. Within middle third of junction:

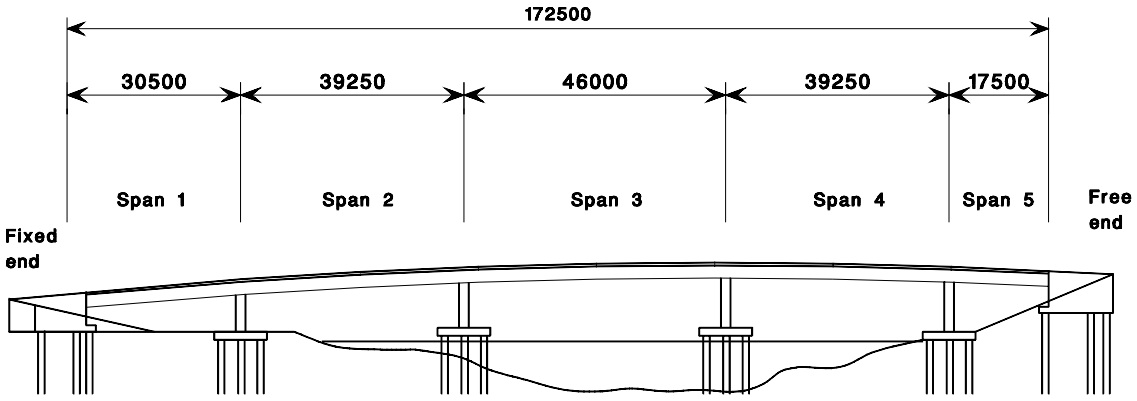
$$P = 307 + \frac{2}{3} \times 322 = 522 \text{ kN}$$

$$\therefore \frac{P}{A_{se}\ \sigma_{\ell s}} + \frac{M}{Z_{se}\ \sigma_{ys}} = \frac{522 \times 10^3}{23800 \times 258} + \frac{16.9 \times 10^6}{1.98 \times 10^6 \times 335} = 0.110$$

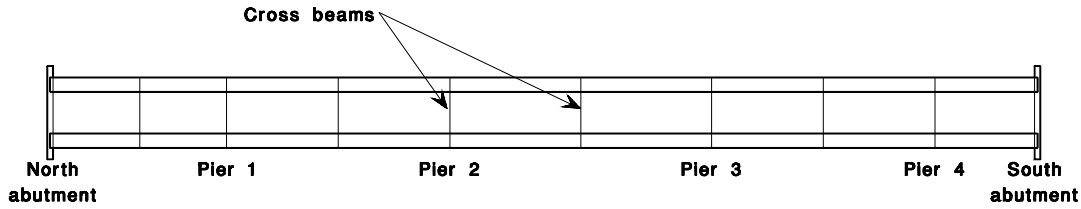
$$\frac{1}{\gamma_m\ \gamma_{f3}} = \frac{1}{1.2 \times 1.1} = 0.757 > 0.110 \quad \text{OK}$$

Example 2 - Open trapezoidal boxes, 46 m span

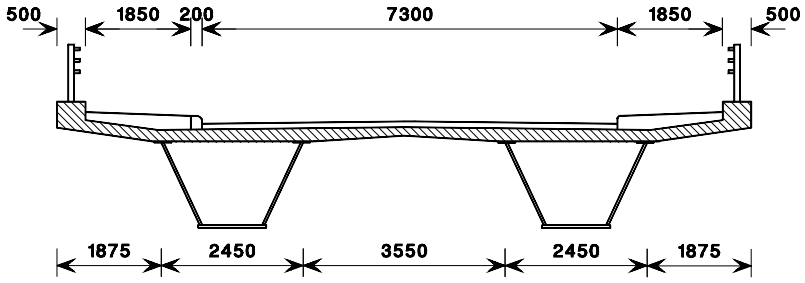
Commentary to calculation sheet



ELEVATION ON BRIDGE



PLAN ON STEELWORK



SECTION

In BD 7/01, the minimum allowance for exposed faces in 'severe' environments is 1.5 mm per face. The allowance of 2 mm shown opposite was the value specified in BD 7/81, which was the current standard when this example was written. The calculations for the worked example have not been reworked with the reduced allowance - this is clearly conservative.

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CALCULATION SHEET

Job No. BCR 433		Sheet 1 of 18	Rev. A
Job Title	Box Girder Design Guide - Example 2		
Subject	Design Data		
Client SCI	Made by DCI	Date	Mar 1994
	Checked by: KB	Date	Mar 1994

DESIGN DATA

General

Spans: 172.5 m, over 5 spans. Largest span 46 m

Carriageway: Single 7.3 m carriageway with 1.85 m verge/footpath each side

Surfacing: 125 mm including waterproofing

Location: North-West England

Design Life: 120 years

Loading

Unit Weights:

Steel	77 kN/m^3
Concrete	26 kN/m^3
Surfacing	24 kN/m^3
Parapets	0.3 kN/m
Permanent formwork	0.4 kN/m^2

Live Loads: HA 2 notional lanes (to BD 37/01)
HB 30 Units
Footpath (Clause 6.5 of BD 37/01)

Wind: Mean hourly windspeed 30 m/sec

Design Parameters

Steel (S355 J2 G2W) (Weathering Steel) σ_y = 355 N/mm^2 (up to 16 mm thick)

E = 205 kN/mm^2

Corrosion allowance: 2 mm per face external (BD 7/01 states 1.5 mm minimum - see facing commentary)
0.5 mm per face internal

Concrete (Grade 40)

f_{cu} = 40 N/mm^2
E_{cs} = 31 kN/mm^2
E_{ci} = 15.5 kN/mm^2

Reinforcement

f_{ry} = 460 N/mm^2
E_y = 200 kN/mm^2

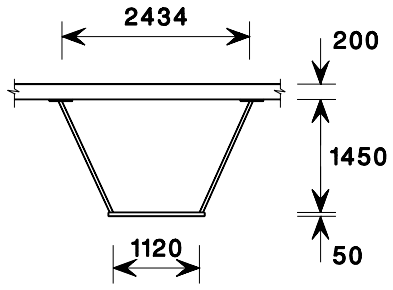
Commentary to calculation sheet

The basic cross-section was derived as shown, using some simple rules of thumb for proportions.

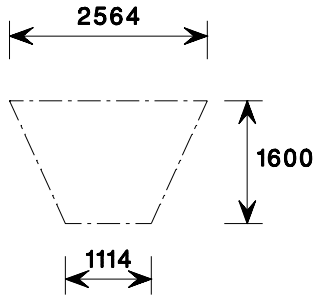
The angle of the web is such that the web plates can simply be cut square and a single sided weld with partial penetration can be used.

The complete initial design stage for this bridge was quite lengthy and is not suitable for presentation in a short precis. A staged construction sequence was devised which minimised the size of the steel section by developing composite action over the piers at an early stage. The sequence involved placing concrete over and between the boxes at successive positions, starting with the region over the first intermediate support, then the first span, then over the second support, etc. After this the cantilevers were cast.

The early placing of concrete over pier regions stiffened the cross beam action (by providing a substantial top flange) and ensured that the load of wet concrete in the spans was largely carried by hogging moments in the composite section, relieving the compressive stress slightly (from what it otherwise would have been) in the top flanges in midspan and significantly reducing the tensile stress due to dead loads in the top flanges over the piers.



Basic section geometry



Dimensions on centrelines of box walls

INITIAL DESIGN

Overall width of deck = 12.2 m (including edge beams)

Choose 2 open-topped trapezoidal boxes
Choose box spacing about half the deck width - say 6.0 m
Choose max span depth ratio between 25 and 30 - say 27

Then $D = \frac{46.0 \times 10^3}{27} = 1704$ mm

Try girder depth = 1500, slab thickness = 220

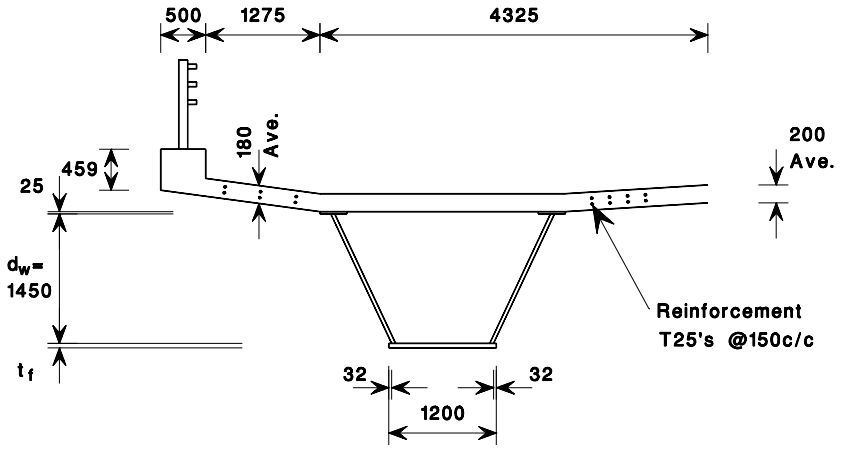
For bottom compression flange to be fully effective b/t $\leq$ 24

For t = 50 mm, width $\leq$ 1200 mm, say 1100 mm

Slope webs at approximately 25°
Therefore, top width is 1100 + 1500 Tan 25° × 2 = 2499 mm

Chosen Section

After development during the initial design stage, the section chosen for detailed design was:



The girder make-up consisted principally of two sections, one at the piers and one in the midspan regions

	Pier Section	Span Section
Top flanges	2/300 × 25	2/300 × 25
Web	15 thick	10 thick
Bottom flange	1200 × 50	1200 × 30

(All plate sizes are nominal, before deduction of corrosion allowance)

Commentary to calculation sheet

GRP permanent formwork was provided for the slab over the box and for the portion between the two boxes.

A longitudinal construction joint was provided just outside the line of the outer flange. Dead load effects were therefore carried on two different composite sections, as well as on the steel section.

DETAILED DESIGN

Design Loads

The nominal loads used for detailed design are as follows.

Steel weight

Pier section	9.4 kN/m
Span section	6.4 kN/m
Concrete weight	35 kN/m
Permanent formwork	3 kN/m
Surfacing	16.8 kN/m
Parapet/Services	1.7 kN/m

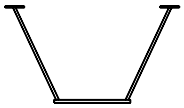
Live Loads

30 units HB (300 kN/axle)
HA udl
46 m loaded length = 25.8 kN/m
85.5 m loaded length = 23.1 kN/m
HA KEL 120 kN
Footpath 5 kN/m^2 (basic loading)

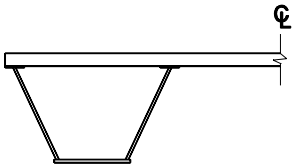
Design Sections

Three basic cross sections are considered during design

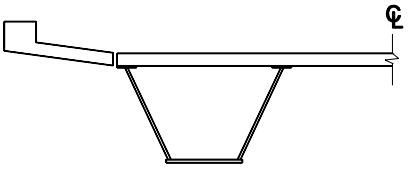
Steel only (G)



Steel + internal slab (S)



Steel + whole slab (F)



Commentary to calculation sheet

Properties are tabulated for the three basic cross-sections shown on the previous calculation sheet. Values are given for long-term and short-term loading on the complete section.

Values were used in the global analysis and in the stress analysis of sections.

Properties for the composite cross beams at midspan and at the piers were similarly calculated, based on an effective width of slab of one quarter the distance between the two inner webs (see Clause 3/9.15.2.1).

SECTION PROPERTIES

Gross section properties (after deduction of the corrosion allowance) are tabulated for sections over the pier and in midspan:

Pier Section (cracked) - Girder A

	A	y	$I \times 10^3$	$Z_T \times 10^3$	$Z_B \times 10^3$
	cm^2	cm	cm^4	cm^3	cm^3
(G)	1107.4	104.05	3 729	35.8	76.9
(S)	1390.5	80.01	6 874	85.9	94.8
(F)	1506.6	71.93	8 053	112.0	100.0

Span Section (uncracked) - Girder B

	A	y	$I \times 10^3$	$Z_T \times 10^3$	$Z_B \times 10^3$
	cm^2	cm	cm^4	cm^3	cm^3
(G)	709.2	94.60	2 717	28.7	48.6
(S)	1364.5	42.39	6 764	159.6	62.6
(F_{LT})	1712.0	28.74	8 069	280.8	66.3
(F_{ST})	2715.3	11.55	9 585	831	69.0

Torsional properties of the closed section are calculated on the basis of an uncracked slab.

Section A (Pier)

B_T = 2564 mm
B_B = 1114 mm
D = 1600 mm
t_w = 12.5 mm (corroded)
t_{bf} = 47.5 mm (corroded)
t_{tf} = 200/6.6 = 30.3 mm (short term)

$$J = \frac{4A^2}{\oint \frac{ds}{t}} = \frac{4 \times (1600 \times (2564 + 1114)/2)^2}{\left(\frac{2564}{30.3} + 2 \times \frac{1600 \sec 24.37^\circ}{12.5} + \frac{1114}{47.5} \right)} = 8.90 \times 10^{10} \text{ mm}^4$$

Similarly for Section B, $J = 5.83 \times 10^{10}$ mm^4

Commentary to calculation sheet

A grillage analysis of the bridge was carried out by computer for the in-service condition and for the successive construction stages

A model generally similar to that used in Worked Example No. 1 was used, with longitudinal members along the centreline of each box and 'dummy' members to each web line.

During sequential placing of concrete over the boxes, the open sections were torsionally flexible. Separate calculations were made of deflections during those stages, taking account of torsion and torsional warping of open 'U' sections to ensure that geometric tolerances were met.

Values of coexistent M, V and T are not available. The designer noted the maximum values of each and designed the section for the combined action of the three maxima. This is conservative. It should usually be possible to tabulate co-existent values, though the above approach does offer some simplification in the detailed design.

GLOBAL ANALYSIS

The key load effects determined from a grillage analysis are:

Summary of Load Effects Over Pier 3

Load Case	ULS Moment	ULS Shear	ULS Torsion
Weight on Steel (G) Concrete on Box (S)	2807 6193		
Concrete (long term) Surfacing Superimposed Settlement	} } 5676 } 426	} } 1856 } 20	} } } 75 }
Total (F_{LT})	6102	1876	
COMBINATION 1 HA Live (Max M) HA + HB (Max M) Footway (Max M)	 7566 - 1310	 - - -	 - - -
Total (Max M)	8876		
HA Live (Max V) HA + HB (Max V) Footway (Max V)	- - -	 - 1630	- - -
Total (Max V)		- 1630	
HB (Max T) Footway (Max T)	3877 0	- 364 0	- 810 0
Total (Max T)	3877	- 364	- 810

Summary of Load Effects Span 3

Load Case	ULS Moment	ULS Shear	ULS Torsion
Weight on Steel (G) Concrete on Box (S)	- 3284 - 884		
Concrete (long term) Surfacing Superimposed Settlement	 - 2772 0		
	- 2772		
COMBINATION 1 HA Live (Max M) HA + HB (Max M) Footway (Max M)	 - - 7754 - 1830	 30 0	 263 0
Total (Max M)	- 9584	34	263
HB (Max T) Footway (Max T)	- 4868 -	- 150 -	516 -
Total (Max T)	- 4868	- 150	516

Commentary to calculation sheet

Effective section for bending stress analysis, Clause 3/9.4.2.

Effective compression flange, Clause 3/9.4.2.4

Effective web, Clause 3/9.4.2.5.

Shear lag in the top flange does not need to be taken into account at ULS (Clause 3/9.2.1.3)

DESIGN OF BEAMS AT PIERS

Effective section for bending stress analysis

Compression flange b = 1126 mm (clear width between webs)

$$\lambda = \frac{b}{t_f}\sqrt{\frac{\sigma_{yf}}{355}} = \sqrt{\frac{335}{355}} \times \frac{1126}{47.5} = 23.0$$

K_c = 1.0 (Figure 5, curve 2) The flange is fully effective.

Web - Bare steel Section

$$y_c = (1525 - 50 - 1040) \times \text{Sec } 24.37° = 478 \text{ mm}$$

$$\frac{y_c}{t_w}\sqrt{\frac{\sigma_{yw}}{355}} = \frac{478}{12.5} = 38.2 \therefore \text{fully effective}$$

Web - with central slab (cracked)

$$y_c = (1525 - 50 - 800) \times \text{Sec } 24.37° = 741 \text{ mm}$$

$$\frac{y_c}{t_w}\sqrt{\frac{\sigma_{yw}}{355}} = \frac{741}{12.5} = 59$$

$\therefore t_{we} = t_w$

Web - with full slab (cracked)

$$y_c = (1525 - 50 - 719) \times \text{Sec } 24.37° = 830 \text{ mm}$$

$$\frac{y_c}{t_w}\sqrt{\frac{\sigma_{yw}}{355}} = \frac{830}{12.5} = 66.4$$

$\therefore t_{we} = t_w$

The section is fully effective, so the properties on Sheet 4 may be used for stress analysis.

Commentary to calculation sheet

Construction in stages, summation of stresses, Clause 3/9.9.5. The limiting stress is the yield stress, σ_y

The beam is being checked as a non-compact section and the LTB slenderness is zero, so the check on summation of stresses will govern.

Distortional warping stresses at the pier are taken as zero.

Vertical and torsional shears are vertical components.

The shear is calculated in the more heavily loaded web.

$$Q_{\mathrm{v}} = \frac{T}{(B_{\mathrm{T}} + B_{\mathrm{B}})} \quad = \text{ is used, rather than } \frac{T}{2B}.$$

($B_T + B_B$*) is slightly greater than 2B (B is at the mid height of the web plate). The former is more appropriate for the composite box.*

Shear resistance of one web, Clause 3/9.9.2.2.

Since m_{fw} *is the smaller of the values for the two flanges and depends on the outstand of the nearer edge of each flange, the actual value of* m_{fw} *is very small.*

Note that the shear capacity is based on d_w *(vertical dimension) not* d_{we} *(in-plane dimension), since the shear loads are expressed as vertical components.*

Stress in Flanges

Bottom flange

$$\sigma = \frac{2807\times10^6}{76.9\times10^6}+\frac{6193\times10^6}{94.8\times10^6}+\frac{6102\times10^6}{100.0\times10^6}+\frac{8876\times10^6}{100.0\times10^6}$$

$$= 252 \text{ N/mm}^2 < 290 \text{ N/mm}^2 \left(= \frac{335}{1.05\times1.1} \right) \quad \text{OK}$$

Top flange

$$\sigma = \frac{2807\times10^6}{35.8\times10^6}+\frac{6193\times10^6}{85.9\times10^6}+\frac{6102\times10^6}{112.0\times10^6}+\frac{8876\times10^6}{112.0\times10^6}$$

$$= 284 \text{ N/mm}^2 < 290 \text{ N/mm}^2 \left(= \frac{335}{1.05\times1.1} \right) \quad \text{OK}$$

Shear in webs

Max vertical shear $= 1876 + 1630 = 3506$ kN

shear per web $= \frac{3506}{2} = 1753$ kN

Max torsion $= 810$ kNm

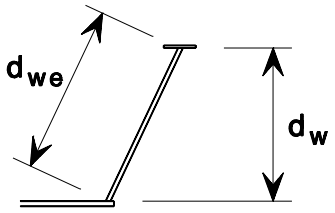
Torsional shear in webs $= \frac{810}{2\times(2.564+1.114)/2} = 220$ kN

Total shear $= 1753 + 220 = 1973$ kN

Shear capacity of webs

$d_w = 1450$

$d_{we} = 1592$



Spacing of transverse stiffeners, $a = 1750$

$$\phi = \frac{1750}{1592} = 1.10 \quad \lambda = \frac{d_{we}}{t_w}\sqrt{\frac{\sigma_{yw}}{355}} = \frac{1592}{12.5} = 127.4$$

Assume $m_{fw} = 0$

Then from Figure 12 $\frac{\tau_\ell}{\tau_y} = 0.62$, $\tau_\ell = 0.62 \times \frac{355}{\sqrt{3}\times1.05\times1.1} = 110$ N/mm^2

Hence $V_D = V_R = 110 \times 1450 \times 12.5 \times 10^{-3} = 1994$ kN

Commentary to calculation sheet

Effective moments for non-compact section constructed in stages, Clause 3/9.9.5.3.

Combined bending and shear, Clause 3/9.9.3.

Top flanges: *2/300 × 25, less corrosion allowance*
Reinforcement: *between and over boxes, A_r = 283 cm^2; in cantilevers, A_r = 116 cm^2*

Height of centroid of the combined areas = 12.4 cm above the top flange

Bottom flange: 1200 × 50, less corrosion allowance.

Interaction during the construction stages (Sections G and S) should also be checked.

The designer accepted the slight excess over unity, since M, V and T are not coexistent.
The checker used co-existent values and demonstrated compliance.

Interaction between bending and shear

Effective moment = $252 \times 100 \times 10^6$ Nmm = 25 200 kNm

Moment capacity $M_D = 290 \times 100 \times 10^6$ Nmm = 29000 kNm

Since $V > V_R/2$, calculate M_R

Top flange $A_s = 14.1 \times 10^3$ mm^2 $A_r = 39.9 \times 10^3$ mm^2

$$\text{Force} = 14.1 \times 307 + 39.9 \times \left(\frac{460}{1.15 \times 1.1} \right) = 18840 \text{ kN}$$

Bottom flange $A_s = 56.8 \times 10^3$ mm^2

Force = 56.8×269 = 15 280 kN

Distance between top and bottom flanges = 1525 – 25 + 124 = 1624 mm

$$M_R = 15\,280 \times 1624 \times 10^{-3} = 24\,810 \text{ kNm}$$

$$\frac{M}{M_D} + \left(1 - \frac{M_R}{M_D}\right)\left(2\frac{V}{V_R} - 1\right) = \frac{25200}{29000} + \left(1 - \frac{24810}{26900}\right)\left(2 \times \frac{1973}{1994} - 1\right) = 1.010$$

Accepted, since max M, max V and max T are not actually coexistent

Commentary to calculation sheet

Effective web, Clause 3/9.4.2.5.

The value y_c is calculated for the cross section appropriate to the stage considered, not to the level of zero stress (see definition in Clause 3/9.4.2.5.1).

Effective web, Clause 3/9.4.2.5.

Summation of stresses, Clause 3/9.9.5. The limiting stress is the yield stress σ_y. As for the pier section, the check on summation of stresses governs.

DESIGN OF BEAMS AT MIDSPAN

Effective sections

For the service condition, depth to neutral axis = 115 mm (Sheet 4)

y_c = (115 – 25) × Sec 24.37° = 99 mm

$\frac{y_c}{t_w}$ = 13 ∴ web is fully effective

For the construction condition, without the concrete slab, consider the uncorroded section properties

Gross section

I = 3050 × 10^3 cm^4 $\bar{y}$ = 93.7 cm

Effective section

y_c = (937 – 25) × Sec 24.37° = 1001 mm

$\frac{y_c}{t_w}\sqrt{\frac{\sigma_{yw}}{355}} = \frac{1001}{10} = 100.1$

$\therefore t_{we} = \left(1.425 - 0.00625\frac{y_c}{t_w}\right) t_w = 0.800\, t_w =$ 8.0 mm

With this reduced thickness of web, the properties of the bare section are:

I = 2913 × 10^3 cm^4 z_t = 30.5 × 10^3 cm^3 z_b = 52.7 × 10^3 cm^3

Stresses in service

Bottom flange

$$\sigma = \frac{3284\times10^6}{48.6\times10^6} + \frac{884\times10^6}{62.6\times10^6} + \frac{2772\times10^6}{66.3\times10^6} + \frac{9584\times10^6}{69.0\times10^6}$$

= 262 N/mm^2 < 307 N/mm^2 OK

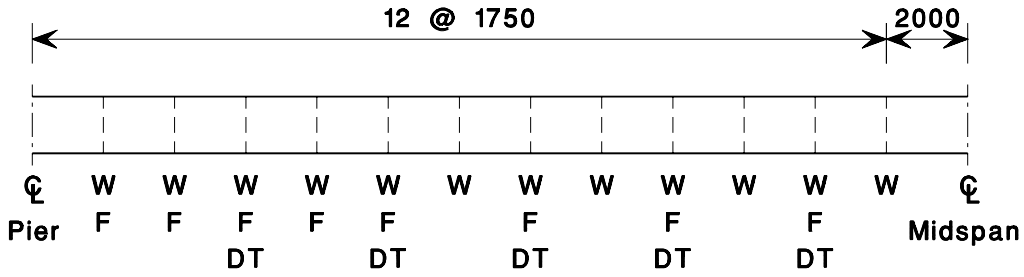
Top flange

$$\sigma = \frac{3284\times10^6}{28.7\times10^6} + \frac{884\times10^6}{159.6\times10^6} + \frac{2772\times10^6}{280.8\times10^6} + \frac{9584\times10^6}{831\times10^6}$$

= 141 N/mm^2 < 307 N/mm^2 OK

Commentary to calculation sheet

The arrangement of transverse stiffeners and cross-frames is shown below



W = web transverse stiffeners
F = flange transverse stiffeners
DT = diagonal and lateral tie

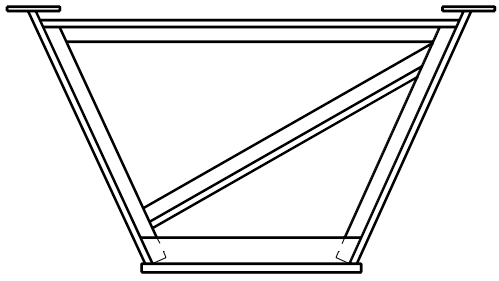
In addition each individual girder section was braced at the end in a horizontal plane just below the top flanges, to achieve torsional stiffness during erection by restraint of torsional warping.

Lateral-torsional buckling limitations (Clause 3/9.7) are not fully appropriate to the configuration when placing concrete in midspan, after the pier sections had already been concreted. The designer has adopted the simple view that torsional buckling of each 'half-box' is prevented at each cross-frame and that the only buckling mode which can occur is that similar to the buckling of the compression chord of a truss. The buckling resistance is therefore calculated according to Clause 3/12.4 and 3/10.6. The usual factor of 0.85 has been applied to the distance between cross-frames.

The flange can only buckle normal to the plane of the web.

The design force for the lateral restraint is taken from Clause 3/9.12.2.

The diagonal also needs to be checked. It carries a component from both flanges. By inspection, the same angle section as the cross-tie will suffice.



Further checks would be made on additional bracing added to suit detailed erection arrangements.

Stresses during construction

Load on bare steel section = 3284×10^6 kNm

Top flange: $\sigma = \dfrac{3284 \times 10^6}{30.5 \times 10^3} = 108$ N/mm^2 (uncorroded section)

Top flanges are 300 × 25, braced at 3.75 m centres at midspan
Bending load is shared equally, because of stiff cross-beam at midspan

Consider the top flange as a strut, $\ell_e = 3750 \times 0.85 = 3188$ mm

$r_y = \dfrac{300 \cos 24.37°}{\sqrt{12}} = 79$ mm ; $\dfrac{\ell_e}{r_y} = \dfrac{3188}{79} = 40$

As a strut, $\dfrac{\sigma_c}{\sigma_y} = 0.83$ (Figure 37, curve c)

$\dfrac{\sigma_c}{\gamma_m \gamma_{f3}} = \dfrac{0.83 \times 355}{1.2 \times 1.1} = 223$ N/mm^2 > 108 N/mm^2 OK

Plan bending of flange at midspan between cross-ties

Load of concrete + permanent formwork + live load during concreting = 36 kN/m
Load in plane of flange = 36/2 × tan 24.37° = 8.15 kN/m

Moment at midspan = $\dfrac{wL^2}{24} = \dfrac{8.15 \times 3.75^2}{24} = 4.78$ kNm

Bending stress = $\dfrac{4.78 \times 10^6}{25 \times 300^2/6} = 13$ N/mm^2

Check adequacy of cross-tie

Take load = 1¼% of load in two flanges to stabilise flange - in compression or tension and as a tie against plan bending (tension only)

2½% of load in one flange: $0.025 \times 108 \times (300 \times 25) \times 10^{-3} = 20.25$ kN
As a tie against bending: $(3.50 + 3.75)/2 \times 8.15 = 29.5$ kN

As a strut: $\ell_e \simeq 2450$ mm $\dfrac{\ell_e}{r_y} = \dfrac{2450}{16.3} = 150$; $\dfrac{\sigma_c}{\sigma_y} = 0.2$ (Figure 37, curve c)

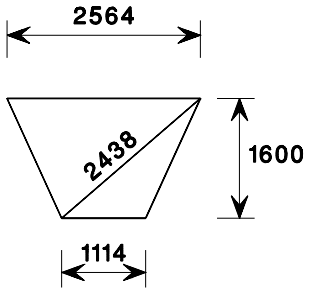
Capacity = $\dfrac{0.2 \times 355 \times 1550}{1.2 \times 1.1} = 83.4$ kN OK

Commentary to calculation sheet

Restraint of distortional warping, Clause 3/B.3

The geometric properties are those for the pier section, using 'corroded' thicknesses (i.e. t_w = 12.5 mm, t_b = 47.5 mm). Effects on Section B should also be checked.
Notional geometry for the checks opposite:

Cross-ties and diagonals are both 125 × 75 × 8 angles.



Although the frames are not 'effective' the benefit to global analysis in terms of torsional stiffness can be demonstrated as follows:

Consider the stiffness of the girder between pier and midspan (where there are stiff cross-beams and effective diaphragms) under a single torque applied midway along that length.

The twist of a simple beam (restrained against twist at the supports) under a unit torque (1 mNm) at midspan gives rise to a vertical displacement W at the top corners $\frac{\theta B_T}{2}$*, where* θ *is given by:*

$$\theta = \frac{TL}{4GJ} = \frac{1 \times 23}{4 \times 78 \times 10^3 \times 9 \times 10^{-2}} \quad (J = 90 \times 10^{10}\ mm^4,\ Sheet\ 4)$$

$$= 8.1 \times 10^{-4}\ rad$$

$$W = \frac{2564 \times 8.1 \times 10^{-4}}{2} = 1.04\ mm$$

The distortional displacement due to a torque applied in this manner is given by Wright (Ref. 5) as:

$$W = \frac{P}{8EI_b\beta^3}\ w$$

Where $P = T/B_T$, $I_b \simeq I_x/4$ *and* $w = 1$ *when* $\beta L_D > 2.0$

Using $I_x = 9.6 \times 10^{10}\ mm^4$ *and ignoring the cross-frames completely*

$$W = \frac{1/2.564}{8 \times 205 \times 10^3 \times (96 \times 10^{-3}/4) \times 0.128^3} = 4.72 \times 10^{-3}\ m = 4.72\ mm$$

Which is more than the torsional flexibility. The global behaviour will be affected. However, if the cross-frames stiffness calculated opposite (S = 79) is 'smeared', K is magnified by 80 and β *becomes* $0.128 \times 80^{0.25} = 0.382\ m^{-1}$*. Then W becomes 0.176 mm.The distortional flexibility can then be ignored in the global analysis.*

DISTORTIONAL EFFECTS

Cross-frames and ties are provided generally at 3500 centres
Consider whether the frames are effective (Girder section A, corroded)

$$D_{YT} = \frac{Et^3}{12} = \frac{31{,}000 \times 200^3}{12} = 20.7 \times 10^9 \text{ Nmm}$$

Similarly $D_{YB} = 1.83 \times 10^9$ Nmm $D_{YC} = 3.34 \times 10^7$ Nmm

$$\frac{D_{YT}}{D_{YC}}\frac{d}{B_T} = \frac{20.7 \times 10^9}{3.34 \times 10^7} \times \frac{1757}{2564} = 425$$

$$\frac{D_{YB}}{D_{YT}} = \frac{1.83 \times 10^9}{20.7 \times 10^9} = 0.089$$

$$\phi_T = \frac{d}{B_T} = \frac{1757}{2564} = 0.685$$

$R_D = 0.15$ (from formula for Figure 58)

$$K = \frac{24 D_{YT} R_D}{B_T{}^3} = \frac{24 \times 20.7 \times 10^9 \times 0.15}{2564^3} = 4.40 \text{ Nmm}^{-1}$$

$$\beta L_D = \left(\frac{K L_D{}^4}{E I_x}\right)^{0.25} = \left(\frac{4.40 \times 3500^4}{205{,}000 \times 80.5 \times 10^9}\right)^{0.25}$$

$= 0.447$ $(\beta = 0.128 \text{ m}^{-1})$

Stiffness of cross-braced frame (Clause 3/B.3.4.3)

$$S = \frac{1}{2} \times \frac{E A_b \delta_b^2 K}{L_b L_D} \quad \text{(only one diagonal)}$$

$$\delta_b = \frac{2}{K} \frac{1 + B_T/B_B}{\sqrt{1 + \left((B_T + B_B)/2D\right)^2}} = \frac{2 \times (1 + 2564/1114)}{4.40 \times \sqrt{1 + (3678/3200)^2}} = 0.985$$

$$S = \frac{205{,}000 \times 1550 \times 0.985^2 \times 4.40}{2 \times 2438 \times 3500} = 79$$

The frames are not effective (see Table 17, Part 3)

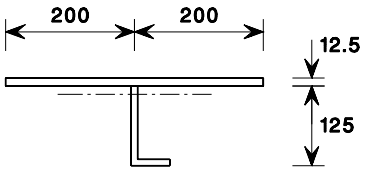
Commentary to calculation sheet

The single web stiffener at the centre of the panel length is smeared over half the panel length.

It can be very difficult to estimate interpolated values for R_D between Figures B.2(a) (b) (c) and (d), particularly for a trapezoidal box when the webs do not slope at 30° and ϕ exceeds 0.5.

Instead, use the equations given in Annex G.17 to Part 3.

Consider the effect of web stiffener at 1750 crs



Properties of effective section
$I = 10.6 \times 10^6$ mm^4
$A_{se} = 6340$ mm^2 $\bar{y} = 25.1$ mm $Z_s = 95000$ mm^3

The intertia per unit length

$$= \frac{10.6 \times 10^6}{1750} = 6057 \text{ mm}^4/\text{mm}$$

$D_{YC} = 205000 \times 6057 = 1240 \times 10^6$ Nmm (was 33.4×10^6)

So now $\frac{D_{YT}}{D_{YC}} \frac{d}{B_T} = \frac{20.6}{1.24} \times \frac{1757}{2564} = 11.4$

Hence $R_D = 4.25$ (from formula for Figure 58)
$K = 125$ Nmm^{-1}
$\beta L_D = 1.03$ ($\beta = 0.295$ m^{-1})
$S = 3$

The frames are still not effective over a panel length of 3.5 m (Table 17, Part 3)

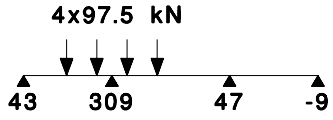
Distortional warping and bending stresses

Consider the effects due to single torque, applied to section without effective cross-frames

Worst effect is that due to one axle of the HB vehicle (300 kN $\times 1.3$).

Assume that the distortional torque is given by a simple line-beam analysis

$$T = \left(\frac{309 - 43}{2} \right) \times 2.564 = 341 \text{ kNm}$$



In a box without effective intermediate diaphragms, σ_{DW} due to a concentrated torque is not more than that given by Clause B.3.2(b) when $\beta L_D > 1.0$

$$\text{Max } \sigma_{DW} = \frac{T \bar{y}}{\beta I_x B_T}$$

$$= \frac{341 \times 10^6 \times 115}{0.295 \times 10^{-3} \times 9585 \times 10^7 \times 2564} = 0.5 \text{ N/mm}^2 \text{ top}$$

$$= \frac{341 \times 10^6 \times 1390}{0.295 \times 10^{-3} \times 9585 \times 10^7 \times 2564} = 7 \text{ N/mm}^2 \text{ bottom}$$

Commentary to calculation sheet

These 'worst' values of σ_{DW} and σ_{DB} are those which would arise in a box where the cross-frames offer no benefit to distortional restraint (though in the calculation opposite the web stiffeners are taken to give bending stiffness to the web) and where there is no significant deflection of the loaded box relative to the unloaded box. The latter assumption is approximately true close to the very stiff cross beams, though elsewhere the distortional load is generally offset by shear transferred through the slab from the less deflected box.

It can be seen that the values of σ_{DW} and σ_{DB} are modest. The web stiffener should be welded to the flanges so that it can transfer the σ_{DB} effects.

The effects of a second axle, 1.8 m away, would be to add the effects calculated for one axle multiplied by $(\cos \beta_x - \sin \beta_x)\, e^{-\beta x}$ for distortional warping and by $(\cos \beta_x + \sin \beta_x)\, e^{-\beta x}$ for distortional bending (see Clauses 3/B.3.2(d) and 3/B.4.2(d). For $\beta = 0.295\ m^{-1}$ these factors equate to 0.209 and 0.805.

Generally, if distortional stresses are to be taken into account the axle load should be applied to the grillage model along a line of transverse nodes and the resulting increment of torque in the main beam determined. This will give a much better value for the distortional torque than the simple estimate on the previous sheet.

Actual effective length is less, to centres of connecting angles.

Similarly, σ_{DB} is not more than that given by B.4.2(b) when $\beta L_D > 2.0$

$$\sigma_{DB} = \frac{T\,F_D}{2\,B_T\,Z}\,\beta$$

$$F_D = \frac{2564}{2}\left(\frac{1114}{2564+1114} - V_D\right) \text{ at top}$$

$$V_D = 0.224 \quad \therefore F_D = 101 \text{ mm}$$

$$F_D = \frac{B_B\,V_D}{2} \text{ at bottom} = 125 \text{ mm}$$

$$Z = 95000/1750 = 54.2 \text{ mm}^3/\text{mm}$$

$$\sigma_{DB} = \frac{341\times10^6\times125\times0.295\times10^{-3}}{2\times2564\times54.2} = 45 \text{ N/mm}^2 \text{ at top of stiffener}$$

Effect of axle load over cross-frames

Treat cross-frame as effective, to calculate the load in the diagonal under one axle of the HB vehicle.

Frame should be able to carry $2 \times F_B$ (Clause B.3.4.2, but only a single diagonal)

$$F_B = T\times\sqrt{1+\left(\frac{B_t+B_B}{2D}\right)^2} \;/\; 2B_T\times\left(\frac{1+B_T}{B_B}\right)$$

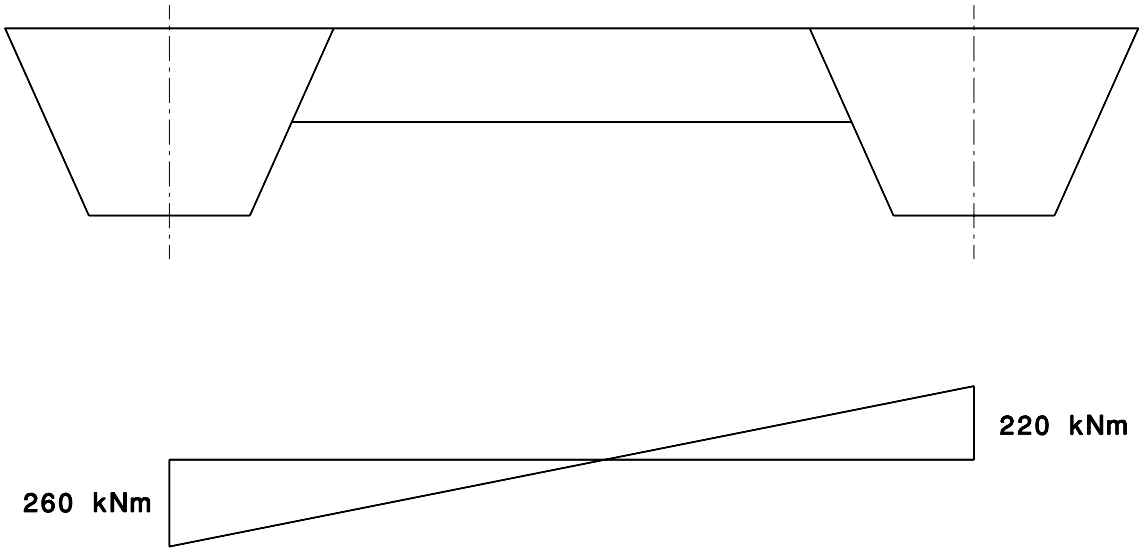
$$= 341\times10^6\times\sqrt{1+\left(\frac{2564+1114}{2\times1600}\right)^2} \;/\; 2\times2564\times\left(1+\frac{2564}{1114}\right)$$

$$= 30700 \text{ N (30.7 kN)}$$

Tie should carry $2 \times 30.7 = 61.4$ kN

ℓ_e = 2438 mm Capacity ≃ 83 kN (similar to sheet 10) OK

Commentary to calculation sheet



Arrangement of cross-beam and diaphragm at pier.

The details of the diaphragm are shown facing sheet 15.

Design of stiffened diaphragms, Clause 3/9.17.6.
See also the commentary to Worked Example No. 1 on diaphragm design.

$B = 1114 + 2 \times 750 \text{ Tan } 24.37^{o} = 1794 \text{ mm}$

Moment for calculating horizontal stresses, Clause 3/9.17.6.2.3(a)

Shear flow, Clause 3/9.17.6.2.4

Since an elastomeric pot bearing is used, which has little rotational stiffness, T_s is used rather than T, where T_s is the moment due to eccentricity of the bearing.

The bearing stiffeners are inclined and the stress at the upper plate corners should strictly be calculated for a vertical section through them.

PIER DIAPHRAGM

Max ULS reaction at pier = 6050 kN
Max torque transmitted to cross-beam = 260 kNm
(with co-existent shear 80 kN)

Q_v = ½ × 6050 = 3025 kN

Q_T = $\dfrac{260}{2 \times 1.794}$ = 73 kN

Moment on diaphragm

M = $(K_d\, Q_v + 2\, Q_T)\, x_w + K_d\, Q_c\, x_c$ (other terms zero)

= $(2.0 \times 3025 + 2 \times 73)\, x_w - 2.0 \times 80\, x_c$

= $6196\, x_w - 160\, x_c$ (on cross-beam side)

Shear flow between box web and bearing stiffener

q = $(Q_v + Q_T + Q_c)/D_e + Q_h / B_e$ (other terms zero)

= $(3025 + 73 - 80)/1.45 + 375/1.12$ = 2420 N/mm

Shear flow between bearing stiffeners

Take q = $\left(\dfrac{Q_v}{4} + \dfrac{T_s}{S_s} - Q_T\right)\dfrac{1}{D_e} + \dfrac{Q_h}{j_e}$ where $T_s = 0.020 \times 6050$

and j = $500 + 3 \times 47.5$

= 642 mm

= $\left(\dfrac{3025}{4} + \dfrac{121}{0.424} - 73\right)\dfrac{1}{1.45} + \dfrac{375}{0.642}$ = 1252 N/mm

Consider moments and stresses on the effective section vertically through the centreline of the bearing stiffeners at its connection to the bottom flange (i.e. 212 from centreline)

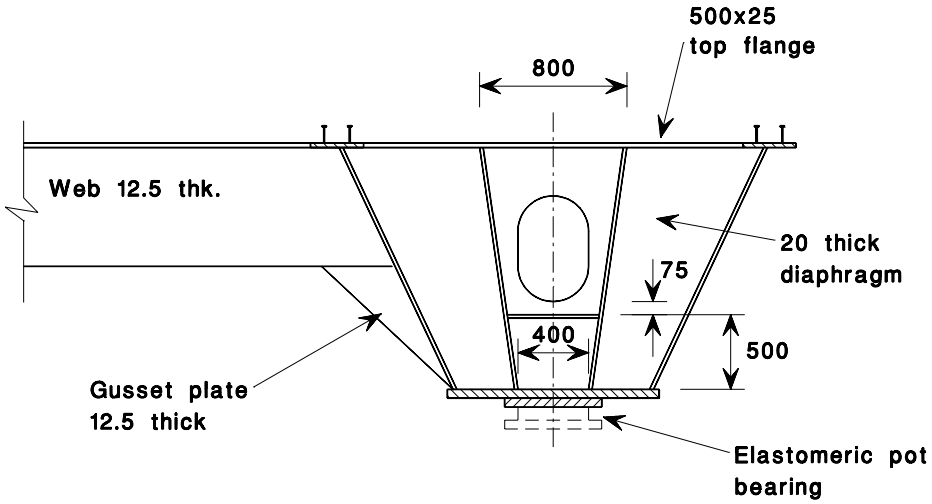
x_w = 1794/2 - 212 = 685 mm x_c = 2466/2 - 212 = 1021 mm
M = 6196 × 0.685 - 160 × 1.021 = 4081 kNm

At top $w_e = (2564/2 - 212)/4 = 268$ mm
At bottom $w_e = (1120/2 - 212)/4 = 87$ mm (which is $\gg$ 12t $\therefore$ fully effective)

Commentary to calculation sheet

The reinforcement in a width of slab equal to ¼ the distance from the corner to the section is taken on either side of the diaphragm (Clause 3/9.17.4.2.3)

Horizontal stress, σ_{2q}, due to component of shear from inclined webs, Clause 3/9.17.6.2.3(b)



Diaphragm and cross-beam at pier.

Properties of effective diaphragm section (corroded thicknesses)

Full section

$A = 501.9\ \text{cm}^2$ $I = 1609 \times 10^3\ \text{cm}^4$
$Z_t = 27\ 800\ \text{cm}^3$ $Z_b = 20\ 100\ \text{cm}^3$

Section after deducting for access hole (includes 8t at top & bottom of upper panel)

$A = 363.6\ \text{cm}^2$ $I = 1540 \times 10^3\ \text{cm}^4$
$Z_t = 25\ 500\ \text{cm}^3$ $Z_b = 19\ 900\ \text{cm}^3$

Stress in outer panel

At top $\sigma_{2b} = \dfrac{4081 \times 10^6}{27.8 \times 10^6} = 147\ \text{N/mm}^2$ (tension)

At bottom $\sigma_{2b} = \dfrac{4081 \times 10^6}{20.1 \times 10^6} = 203\ \text{N/mm}^2$ (compression)

$\sigma_{2q} = \dfrac{3025 \times 10^3 \tan 24.37^\circ}{50.19 \times 10^3} = 27\ \text{N/mm}^2$ (compression)

$\tau = \dfrac{2420}{19} = 127\ \text{N/mm}^2$

Stress in inner panel

At top $\sigma_{2b} = \dfrac{4081 \times 10^6}{25.5 \times 10^6} = 160\ \text{N/mm}^2$ (tension)

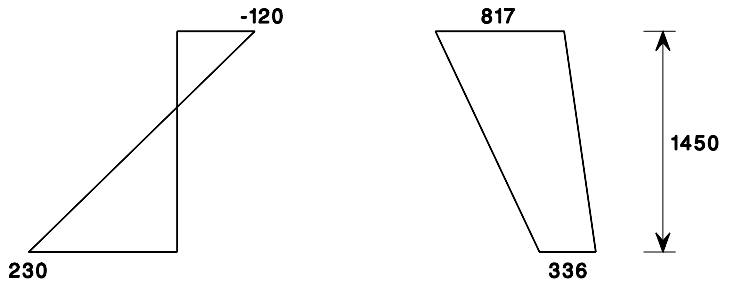
At bottom $\sigma_{2b} = \dfrac{4081 \times 10^6}{19.9 \times 10^6} = 205\ \text{N/mm}^2$ (compression)

$\sigma_{2q} = \dfrac{3025 \times 10^3 \tan 24.37^\circ}{36.36 \times 10^3} = 38\ \text{N/mm}^2$ (compression)

$\tau = \dfrac{1252}{19} = 66\ \text{N/mm}^2$

Commentary to calculation sheet

Yielding of diaphragm plate, Clause 3/9.17.6.4



Variation of σ_{d2} Panel dimensions

The vertical stress σ_{d1} is 175 N/mm^2 over 174 mm width adjacent to the stiffener at the bottom of the panel. Average stress = 175 × 174 / 336 = 91 N/mm^2.

The check on yielding just fails (1% overstress). It would be sufficient to add 1 or 2 mm to the thickness of the diaphragm plate to make the check satisfactory; all other checks on the diaphragm are satisfactory and the recalculation with a thicker plate has not been made in this revision of the example.

For checks according to Clause 3/9.11.4:

$\sigma_1 = 55 \qquad \sigma_b = 175 \quad \sigma_2 = 91 \qquad$ *and* $\qquad \tau = 55$ N/mm^2 .

For the derivation of K_2, the dimension a is taken at the bottom of the panel.
For the derivation of K_q, the dimension a is taken at the mid-height of the panel.

Yielding of diaphragm plate

Outer panels

Top of diaphragm $\sigma_{d1} = 0\ \sigma_{d2} = -147 \quad \sigma_{2q} = 27 \quad \tau = 127$

$((-147 + 27)^2 + 3 \times 127^2)^{½} = 251\ \text{N/mm}^2 < 307\ \text{N/mm}^2$ OK

Bottom of diaphragm $\sigma_{d1} = \sigma_{1s} + \sigma_{1sT} = 175$ (see Sheet 18) $\sigma_{d2} = 203$
$\sigma_{2q} = 27 \quad \tau = 127$

$(175^2 + (203+27)^2 - 175\times(203+27) + 3\times127^2)^{½} = 303\ \text{N/mm}^2 < 299\ \text{N/mm}^2$ Fails. See facing commentary

Inner panels

Satisfactory by inspection

Buckling check on outer panel

$b/t = 1450/19 = 76 \quad \therefore K_1 = 0.40$ (Figure 23(a))
$a/t = 336/19 = 18 \quad \therefore K_2 = 1.78$ (Clause 3/9.11.4.3.5)
$\phi = 577/1450 = 0.40 \quad \therefore K_q = 1.53$ (Clause 3/9.11.4.3.3)
$K_b = 1.09$ (Figure 23(c))

$$m_1 = \left(\frac{55\times1.1\times1.05}{345\times0.40}\right)^2 = 0.46^2 = 0.212$$

$$m_2 = \left(\frac{91\times1.1\times1.05}{345\times1.78}\right)^2 = 0.18^2 = 0.029$$

$$m_c = \sqrt{0.212 + 0.029} = 0.049$$

$$m_b = \left(\frac{175\times1.1\times1.05}{345\times1.09}\right)^2 = 0.31$$

$$m_q = \left(\frac{66\times1.1\times1.05}{345\times1.53}\right)^2 = 0.02$$

$m_c + m_b + 3m_q = 0.49 + 0.31 + 3 \times 0.02 = 0.86$ OK

Commentary to calculation sheet

The bearing was arranged with the sliding surface on the pier, rather than on the box, to avoid large eccentric reactions due to expansion. The eccentricities used are 'nominal' values to account for setting errors and any slight unevenness in the pressures in the elastomer. Canvas screens were fixed to the box to prevent debris from collecting on the sliding surface.

Bearing stress at the bottom of the stiffeners, Clause 3/9.14.4.2

Effective stiffener section, Clause 3/9.17.4.4

BEARING STIFFENERS

P_s = 6050 kN
e_x = 20 mm
e_y = 20 mm

Bearing stress at bottom of diaphragm
Pot bearing diameter = 415 mm
Dispersal through plates and bottom flange = 2 × (40+45+50) tan 60° = 468 mm

Bearing area = (415 + 468) × 19 + 4 × (224 × 24) = 38280 mm^2

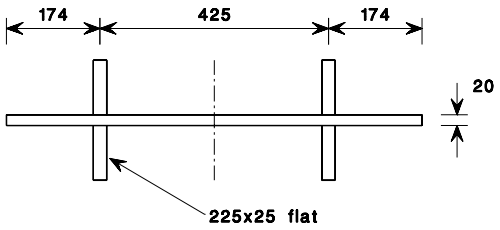
Modulus for longitudinal bending = 1548 × 10^3 mm^3

$$\text{Max bearing stress} = \frac{6050 \times 10^3}{38280} + \frac{6050 \times 10^3 \times 20}{1548 \times 10^3} = 158 + 78 = 236 \text{ N/mm}^2$$

$$\text{Limiting stress} = \frac{1.33\,\sigma_y}{\gamma_m\ \gamma_{f3}} = \frac{1.33 \times 345}{1.05 \times 1.1} = 397 \text{ N/mm}^2 > 236 \text{ N/mm}^2 \text{ OK}$$

Effective Stiffener Section

Section for calculation of vertical stresses



Effective width on web side
= ½ (560 – 212) = 174 mm

Full section:

Effective width in middle = 212 mm ($\leq$ 12t $\sqrt{355/345}$ = 12.17 × 19 = 231 mm)

A_{se} = 362 cm^2 (corroded section)
I_{xx} = 40 800 cm^4
I_{yy} = 170 000 cm^4

Section at opening:

Effective width in middle = 75 mm (min clear distance) $\leq$ 8t = 8 × 19 = 152 mm

A_{se} = 310 cm^2
I_{xx} = 40 800 cm^4

Commentary to calculation sheet

Stiffener yielding, Clause 3/9.17.6.6

Buckling of stiffeners, Clause 3/9.17.6.7. Note that $\gamma_M = 1.20$

The dimension, a, is the sum of half the panel widths either side of the stiffener, within the middle third of the stiffener.

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Job Title	Box Girder Design Guide - Example 2		
Subject	Diaphragms bearing stiffeners		
Client SCI	Made by DCI	Date	Mar 1994
CALCULATION SHEET	Checked by: KB	Date	Mar 1994

Bearing stiffener stresses

$$\sigma_{1s} = \frac{P_s}{A_{se}} = \frac{6050 \times 10^3}{36200} = 167 \text{ N/mm}^2 \text{ (at the bottom)}$$

$$\sigma_{1sT} = \frac{T_s x}{I_{yse}} = \frac{6050 \times 10^3 \times 20 \times 212}{3220 \times 10^6} = 8 \text{ N/mm}^2$$

(Using section properties at opening, $\sigma_{1s} = 195$ N/mm^2 at bottom)

Out-of-plane bending

$$\sigma_{bs} = \frac{6050 \times 10^3 \times 20}{408 \times 10^6/234} = 69 \text{ N/mm}^2$$

Stiffener yielding

$$\sigma_{1s} + \sigma_{1sT} + \sigma_{bs} = 167 + 8 + 69 = 244 \text{ N/mm}^2 < 299 \text{ N/mm}^2 \left(= \frac{345}{1.05 \times 1.1} \right) \text{ OK}$$

Stiffener buckling

$$\sigma_{se} = \sigma_a + \frac{1}{A_{se}} \left(\frac{\sigma_q \ell_s{}^2 t_d k_s}{a} \right) \text{ at the third point}$$

$$\sigma_a = \sigma_{1s} + \sigma_{1sT} = \frac{2}{3} (167 + 8) = 117 \text{ N/mm}^2$$

σ_q = σ_{2s} = average horizontal stress within middle third

= ½ (133 - 7) = 63 N/mm^2

$$r_{se} = \sqrt{408 \times 10^6/31000} = 115 \text{ mm}$$

λ = 1450/115 = 13 $\therefore k_s = 0.01$, $\sigma_{ls}/\sigma_{ys} = 1.0$ (Figure 24)

Take a = 530 mm

$$\sigma_{se} = 117 + \frac{1}{31000} \left(\frac{63 \times 1450^2 \times 19 \times 0.01}{530} \right) = 117 + 2 = 119 \text{ N/mm}^2$$

$$\frac{\sigma_{se}}{\sigma_{\ell s}} + \frac{\sigma_{bs}}{\sigma_{ys}} = \frac{119}{345} + \frac{69}{345} = 0.54 < \frac{1}{1.1 \times 1.2} = 0.76 \quad \text{OK}$$

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