Steel-concrete composite bridge design guide

September 2013

Raed El Sarraf, HERA, Auckland, New Zealand
David Iles, SCI, Ascot, United Kingdom
Amin Momtahan, AECOM, Auckland, New Zealand
David Easey, AECOM, Auckland, New Zealand
Stephen Hicks, HERA, Auckland, New Zealand

NZ Transport Agency research report 525

ISBN 978-0-478- 40769-3 (electronic)
ISSN 1173-3764 (electronic)

NZ Transport Agency
Private Bag 6995, Wellington 6141, New Zealand
Telephone 64 4 894 5400; facsimile 64 4 894 6100
research@nzta.govt.nz
www.nzta.govt.nz

The New Zealand Heavy Engineering Research Association (HERA) was contracted by NZTA in 2009 to carry out this research

El Sarraf, R, D Iles, A Momtahan, D Easey and S Hicks (2013) Steel-concrete composite bridge design guide. *NZ Transport Agency research report 525*. 252pp.

HERA, HERA House, 17-19 Gladding Place, Manukau 2241, Auckland, New Zealand, telephone 64 9 262 4756, facsimile 64 9 262 2856, structural@hera.org.nz, www.hera.org.nz

Keywords: bridges, composite, concrete, construction, cost-effective, design, durability, efficiency, fabrication, fatigue, guide, ladder deck, life cycle, multi-girder, New Zealand, NZ Transport Agency, shear studs, steel, Steel Bridge Development Group, sustainability, vibration, welding.

An important note for the reader

Acknowledgements

This project was undertaken by the New Zealand Heavy Engineering Research Association (HERA), which is based in Manukau, Auckland City, New Zealand. The researchers gratefully acknowledge the following individuals and organisations for their support in the development of this document:

Dr Charles Clifton of the University of Auckland

Shane Culham of Culham Engineering

Jacques De Reuck of Metal Spray Suppliers (NZ) Ltd

Bob Delacey of George Grant Engineering

Alistair Fussell of Steel Construction New Zealand (SCNZ)

Phil Gaby of Holmes Consulting Group

David Gifford of New Zealand Steel

Kenyon Graham of SKM

Matthew Holland of Tenix Robt Stone

Donald Kirkcaldie of Opus

Peter Lipscombe of URS

Jamie Macredie of Steltech

Willie Mandeno of Opus

Bruce Melsop of Eastbridge

Amin Momtahan of AECOM

David Moore of Grayson Engineering

Will Pank of BECA

Rob Presland of Holmes Consulting Group

Anand Reddy of Holmes Consulting Group

Craig Ross of Napier Sandblasting

Dr Wolfgang Scholz of HERA

Mike Sullivan of D&H

Tony Vos of Altex

Peter Wiles of Opus

This report is a key deliverable of a three-year project that was part-funded by the New Zealand Transport Agency, the Steel Bridge Development Group and New Zealand Steel Industry representatives.

Abbreviations and acronyms

BSI	British Standards Institution
CAE	computer-aided engineering tools
CEN	European Committee for Standardization
FCM	fracture critical members
IZS	inorganic zinc silicate
NZTA	New Zealand Transport Agency
RRU	Road Research Unit, National Roads Board (predecessor of Transit NZ)
SH	state highway
SLS	serviceability limit state
SNZ	Standards New Zealand
ULS	ultimate limit state

Contents

Executive summary

Steel-concrete composite bridges provide an efficient and cost-effective form of bridge construction. By utilising the tensile strength of steel in the main girder and the compressive strength of concrete in the slab, the bending resistance of the combined materials is greatly increased and larger spans are made possible.

Two types of composite bridge are considered in this document. The typical multi-girder steel-concrete composite bridge, which consists of a number of steel girders with bracing in between and a slab on top, and a ladder deck bridge, which consists of two main girders with a number of secondary cross girders in between that support and act with a deck slab. Both provide a cost-effective solution and the choice between the two types depends on economic considerations and site-specific factors such as the form of intermediate supports and construction access.

This research report provides guidance on the general considerations for the preliminary and detailed design process, in addition to guidance on the verification of structural adequacy in accordance with the NZ Transport Agency *Bridge manual* and the relevant design and material standards.

The guide describes the determination of design forces, identifies key features relating to the design of the different structural components and gives structural detailing advice. It also provides additional guidance on cost-effective design philosophy and durability design.

The aim of the document is to provide guidance to both the novice and experienced bridge designer on the design of cost-effective steel-concrete composite bridges.

Abstract

This report provides guidance on the design of steel-concrete composite bridges, which consist of steel girders and reinforced concrete slabs on top. Two common forms are considered: multi-girder and ladder deck bridges. Guidance is given on the general considerations for the preliminary and detailed design process, in addition to guidance on the verification of structural adequacy in accordance with the NZ Transport Agency *Bridge manual* and relevant design and material standards. Additional guidance on cost effective design philosophy and durability design is also provided.

The aim of the report is to provide guidance for both the novice and experienced bridge designer on the design of cost-effective steel-concrete composite bridges.

1 Introduction

Steel-concrete composite bridges provide an efficient and cost-effective form of bridge construction. By utilising the tensile strength of steel in the main girder and the compressive strength of concrete in the slab, the bending resistance of the combined materials is greatly increased and larger spans are made possible.

This report provides guidance for both the novice and experienced bridge designer on the design of steel-concrete composite bridges, whether the bridge is simply supported or continuous and for multi-girder and ladder deck forms of construction. Guidance on the cost-effective design philosophy and durability design is also provided.

The guide assumes the reader is familiar with the general principles of limit state design and has some knowledge of structural steelwork. It provides advice on the general considerations for the preliminary and detailed design process, in addition to guidance on the verification of structural adequacy in accordance with the *Bridge manual*[1] and relevant design and material standards.

The guide describes the determination of design forces, identifies key features relating to design of the different structural components and gives structural detailing advice. It also provides additional guidance on cost-effective design philosophy and durability design.

[1] Unless otherwise indicated, the guide refers throughout to the 3rd edition of the NZTA publication *Bridge manual,* published in 2013.

2 Typical composite bridge configurations

The main aim of a bridge designer is to provide a cost-effective solution in accordance with the client's requirements. Steel-concrete composite bridges utilise the tensile strength of steel in the main girder and the compressive strength of concrete in the slab to provide a cost-effective solution over a wide range of spans.

The steel and concrete elements of a composite bridge are connected via shear connectors that are welded to the top flange of the steel girder and are embedded in the concrete deck. The composite action is achieved through the longitudinal shear force transferred by the shear connectors, increasing the bending resistance significantly compared to that achieved by the non-composite beam.

This guide considers two types of composite bridge. The typical multi-girder steel-concrete composite bridge, which consists of a number of steel girders with bracing in between and a slab on top, and a ladder deck bridge, which consists of two main girders with a number of secondary cross-girders in between that support and act with a deck slab. Both provide a cost-effective solution and the choice between the two types depends on economic considerations and site-specific factors such as form of intermediate supports and construction access.

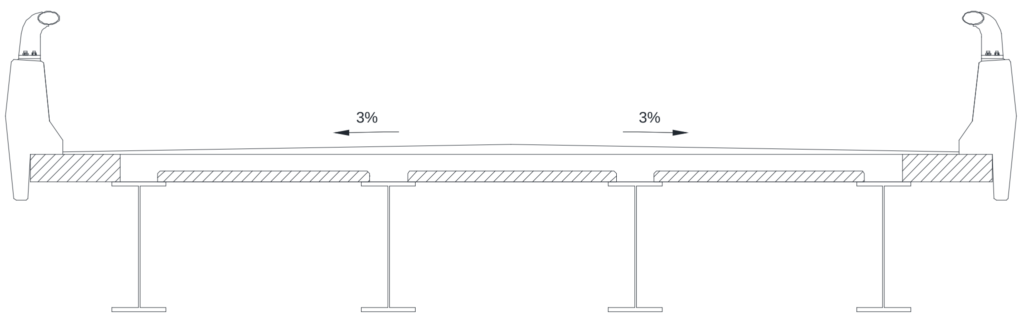
Other types of composite construction are summarised in chapter 11, although they will not be covered in detail in this document.

2.1 Multi-girder bridges

2.1.1 General

In multi-girder bridge construction a number of similarly sized longitudinal girders are arranged at uniform spacing across the width of the bridge, as shown in the typical cross section in figure 2.1 for a two-lane road with a rigid traffic barrier and no footway. The deck slab spans transversely between the longitudinal girders and cantilevers transversely outside the outer girders. The girders are braced together at supports and at some intermediate positions. Composite action between the reinforced concrete deck slab and the longitudinal girders is achieved by means of shear connectors welded on the top flanges of the steel girders. Multi-girder construction may be used for both single spans and continuous multiple spans.

Figure 2.1 Typical multi-girder cross section of a composite bridge with rigid traffic barriers and no footway



The arrangement shown in figure 2.1 is typical when either the slab is cast on temporary formwork or precast concrete decking permanent formwork is used; it shows four girders of equal depth and with a slab surface that follows the camber of the road. A footway is sometimes provided either side of the road, while traffic and/or pedestrian barriers are mounted on the deck slab. The configuration can be adapted for alternative carriageway arrangements, such as given in figure A1 of the *Bridge manual*. Different arrangements for dealing with camber and super-elevation are discussed in section 9.1.1.

2.1.2 Longitudinal girders

The steel girders are usually fabricated I-section plate girders; for smaller spans, it is possible to use hot rolled section beams (Universal Beams) but, for the reasons discussed below, rolled sections are rarely used in current construction. Alternatively, welded beams (see below) may be used in some circumstances. Usually, girders are spaced between about 3m and 4m apart and, therefore, for an ordinary two-lane over bridge, four girders are typically provided. This arrangement supports the deck slab (see section 2.3), which distributes the vertical loads from the wheels in transverse bending.

2.1.2.1 Hot rolled sections – Universal Beams

Universal Beams are hot rolled sections that are readily available in New Zealand up to 612mm deep as tabulated in *Design capacity tables for structural steel* (ASI 1999). Other types of hot rolled sections, some of which are up to 1016mm deep, are available from overseas; however, in New Zealand, welded beams are commonly used for such girder sizes (see comment on welded beams below). Bridges that use Universal Beams are usually lightly loaded farm access bridges or pedestrian bridges that may span up to 12m simply supported or up to 16m for continuous spans. Minimal fabrication is required for Universal Beams, with only the welding of stiffeners and drilling of bolt holes being be required.

2.1.2.2 Plate girders

Plate girders are fabricated from steel plate in accordance with the designer's requirements. They provide the designer with the flexibility of specifying different flange sizes and web thicknesses at different positions along the span, to optimise the girder, depending on the span and applied loading on the bridge. However, the designer must be aware of the availability of plate sizes and welding requirements, as inappropriate selection can result in a costly fabrication; proper optimisation must be considered through the design process.

The depth of the fabricated plate girder can also be varied along the span, as required. It is common to increase the girder depth over the intermediate support where increased negative moment capacity is required. For spans below 50m, the selection of constant or varied depth is often dependent on aesthetics. For spans greater than 50m, a varied depth may provide a certain degree of cost savings in the mid-span regions. The variation in depth can be achieved either by straight haunching (tapered girders) or by curving the bottom flange upwards.

Fabrication lengths of 26m or more require the use of splice connections, due to transportation and fabrication limitation (see section 4.2.1 for further details). The number of connections is dependent upon the required length of the girder. Girder splices are expensive, whether bolted or welded. Except for long span bridges, the most economical solution is usually to specify splices at or near the point of contraflexure.

2.1.2.3 Welded beams

Welded beams are a form of fabricated plate girder with standard dimensions (ASI 1999), that are available from 700mm to 1200mm deep. They can span up to 30m simply supported and up to 35m for continuous spans. At those spans, beams may be required to be curved in elevation to suit the road profile and pre-cambered for dead load; this may be carried out by bridge fabricators using heavy rolling equipment for the shallower beams but it does add to cost. In many cases, welded beams may be more economically replaced by plate girders of similar depth but in which the flange sizes and web thickness have been optimised. Fabricators can advise on the relative economy.

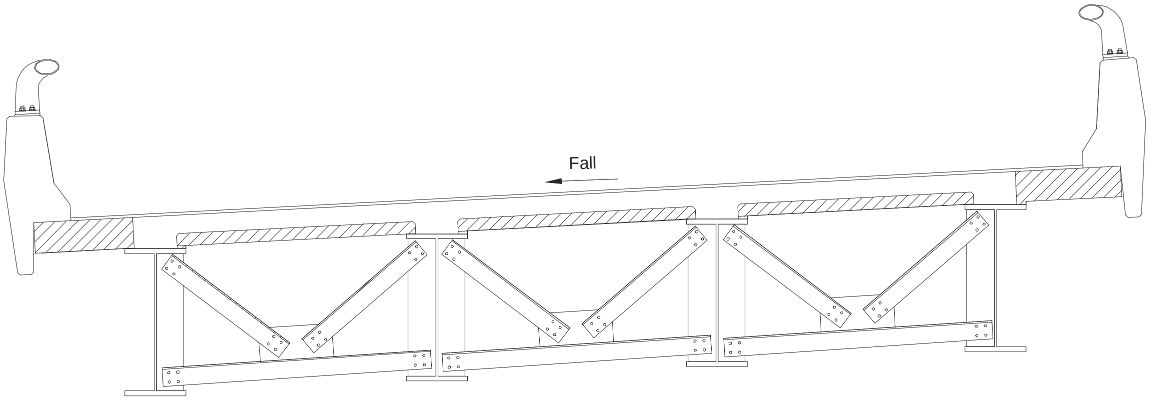
2.1.3 Bracing

2.1.3.1 Support bracing

The steel girders need to be braced together at support positions, for stability and to assist in the transfer of horizontal loads (wind and seismic forces) to the bearings and/or shear keys, that provide transverse restrains at each support position. The support bracing is usually provided by triangulated bracing systems, such as a K-brace or an X-brace, using angle sections, or by horizontal beams (usually channel sections). The bracing systems at the end supports of non-integral bridges may be required to support the end of the deck slab. Integral bridges will require bracing at the end supports for the construction condition.

A typical bracing arrangement (using K-bracing) at an intermediate support is shown in figure 2.2.

Figure 2.2 Typical bracing arrangement at an intermediate support, known as a K-brace (shown for a super-elevated roadway)



2.1.3.2 Intermediate bracing

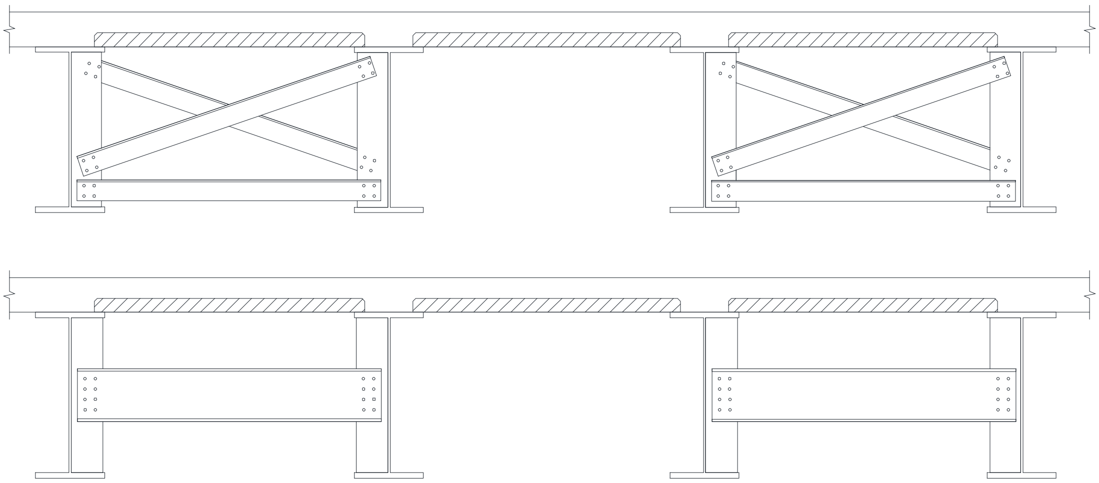
For completed bridges, intermediate bracing is usually required at discrete positions in the spans of multi-span bridges, to stabilise the bottom flanges adjacent to intermediate supports (where they are in compression). During construction, bracing is needed to stabilise both the bottom flanges adjacent to intermediate supports and the top flanges in mid-span regions. Where the girders are curved in plan, bracing will also be needed to provide 'radial' restraint to the bottom flanges (see section 2.5.1).

In most cases, the most effective bracing system is a triangulated frame between adjacent girders. In the completed bridge; this provides a very stiff restraint path from the plane of the deck slab through to the bottom flanges. In the construction stage, intermediate bracing between girders, without plan bracing, provides 'torsional restraint' (see discussion of the effectiveness of such bracing in section 7.3.6). As an alternative, 'channel bracing' is often used with shallow main girders; the stiff channel is rigidly connected to the main girders.

Intermediate bracing that is continuous across more than two main girders will participate in the global action and will distribute loading in any one lane to several main girders. However, such continuity does not provide much benefit to the design of the main girders (especially when the design case considers all lanes loaded) and introduces stress reversals in the bracing and its connections; the connection details are potentially prone to fatigue. To avoid this fatigue situation, designers may use non-continuous bracing, where main girders are connected in pairs, with no bracing between one pair and the next, as shown in figure 2.3.

Intermediate bracing may also be required if the headroom below the bridge is such that collision loading on the bridge soffit needs to be considered. Bracing at intervals provides restraint to the bottom flange and a load path to the bridge deck. In such cases, the bracing at supports has to be designed to transfer the collision loading down to the bearings or shear keys. Guidance on collision loads is given in section 3.4.18 of the *Bridge manual*.

Figure 2.3 Typical paired bracing arrangements, X-brace (top) and the channel brace (bottom)



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

2.1.3.3 Plan bracing

Plan bracing to the top flange is an alternative way to provide a stiff lateral restraint to the top flanges at the bare steel stage during construction. Although such bracing is very effective in restraining the compression flange at mid-span, its presence complicates construction. The two possible locations of plan bracing are above the top flange (connected to cleats on the top flange) and below the top flange. The top of top flange bracing, however, adds difficulty to the placing of reinforcement and conflicts with the use of permanent formwork such as precast decking. Bracing just below the top flange may clash with temporary formwork and would need to be removed after casting (because it may attract unwanted forces when the slab is subject to local loading above it); such bracing is not recommended and is rarely used now internationally.

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

2.1.4 Crosshead girders

It is sometimes desirable to reduce the number of columns and bearings at the intermediate supports (see section 3.5 for further details). Typically, instead of a bearing directly under each girder, one bearing is provided midway between each pair of girders, with a crosshead girder to transfer the reactions. Such an arrangement is particularly suitable with large skews (see section 2.5.2). An example of a crosshead is shown in figure 2.4, which also shows a continuity girder between the central girders; such a girder is advantageous for construction, to minimise twist during concreting, but is not normally needed for the permanent condition.

Figure 2.4 Crosshead girder in a multi-girder bridge

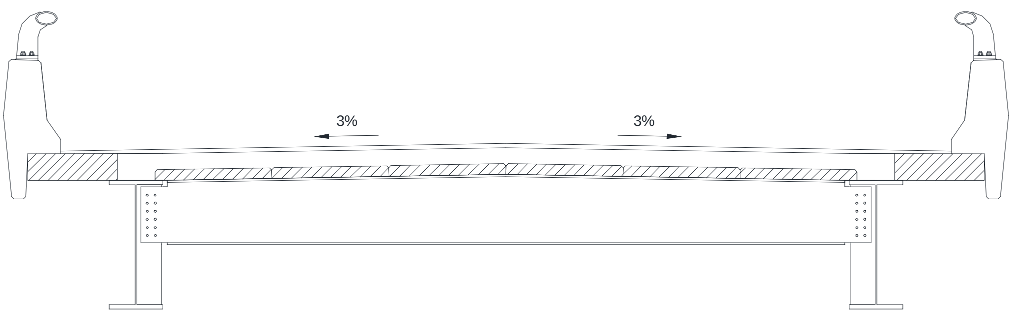
2.2 Ladder deck bridges

2.2.1 General

Ladder deck bridges are a common arrangement in the UK and are also becoming popular in New Zealand. This type of bridge provides only two main girders, with the slab supported on cross-girders that span transversely between the two main girders; the slab spans longitudinally between the cross girders. This arrangement is referred to as 'ladder deck' construction, because of the plan configuration of the steelwork, which resembles the stringers and rungs of a ladder.

A typical cross section of a ladder deck bridge is shown in figures 2.5 and 2.6 (which illustrate the 'ladder' configuration). It should be noted that the ladder deck bridge in figure 2.6, was built with a fully integral abutment so that the main girders and deck slab were eventually cast into an abutment beam/diaphragm at the ends of the bridge, see section 2.6.4 and Iles (2010) for details. Furthermore, the plan bracing shown is permanent but was only required during construction. It is more cost effective to leave the bracing than to remove it after completion of construction. However, future maintenance and its cost must be considered as part of the overall maintenance regime of the structure. For comment on the use of cantilever girders see section 2.2.4.

Figure 2.5 Cross section of a typical ladder deck bridge with rigid traffic barriers and no footway



The arrangement with two main girders is appropriate (and economical) for a bridge width up to that for a dual two-lane carriageway. The economy comes principally from eliminating the cross head girder and directly supporting each stringer on its own set of columns, as shown in figure 2.6.

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

Most ladder deck bridges are designed with uniform depth main girders but variable depth girders can be used. An example of a haunched girder ladder deck is shown in figure 2.7.

Figure 2.6 Steelwork arrangement of a curved ladder deck bridge (image courtesy of Holmes Consulting Group)

Figure 2.7 Ladder deck bridge with haunched main girders (image courtesy of Holmes Consulting Group)

Where the deck is wide (greater than about 22m), for example when a dual three-lane carriageway is carried, two adjacent ladder deck arrangements can be used. In such cases, the deck slab can be continuous across all four main girders or separate slabs may be provided, one on each pair of girders. Where the slab is continuous, it spans transversely between the innermost girders (which are thus limited to a maximum spacing of 3.5m between them). Where separate slabs are provided, each deck cantilevers transversely and some form of joint may be required in the central reserve; guidance can be found for requirements relating to gaps between decks in *NZS 3101* (SNZ 2006) and *TD19/06* (Highways Agency 2006).

Where the deck slab is continuous, depending on the arrangements of the lanes and barriers, high moments may be generated in the deck. This may require the use of an intermediate cross girder to make the row of cross girders continuous across the total width of the bridge.

2.2.2 Main girders

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

With longer spans, the size of the flanges, particularly the bottom flange, is likely to be quite large (in both width and thickness). It is prudent for designers to check the availability of suitable plate material at an early stage, with particular attention to the toughness grade and available dimensions, as discussed in section 4.5.1.

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

2.2.3 Cross girders

Cross girders are typically spaced at between 3.5m and 4m centres, to suit a slab thickness of about 250mm (see section 2.3). For a simple two or three-lane bridge, where the main girders are 7m –10m apart, hot rolled sections (Universal Beams) may be sufficient for structural purposes, but plate girder sections are more likely to be used. Where there is a camber to the road surface (for example, with a two-lane single carriageway, as shown in figure 2.6) the top flange of a plate girder can follow the cross fall, allowing the use of a uniform thickness of both slab and surfacing. The bottom flange would normally be straight. If rolled section cross girders were used, either the sections would have to be cambered (which adds to fabrication cost), or the slab or surfacing would have to be tapered in thickness to provide the falls (which complicates slabs construction).

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

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

2.2.3.1 Intermediate cross girders in sagging moment regions

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

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

The cross girders also provide out-of-plane restraint to the slab where it is in compression; the strength of the cross girders and the slenderness of the slab both need to be considered (see further discussion in sections 7.6 and 7.10.2).

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

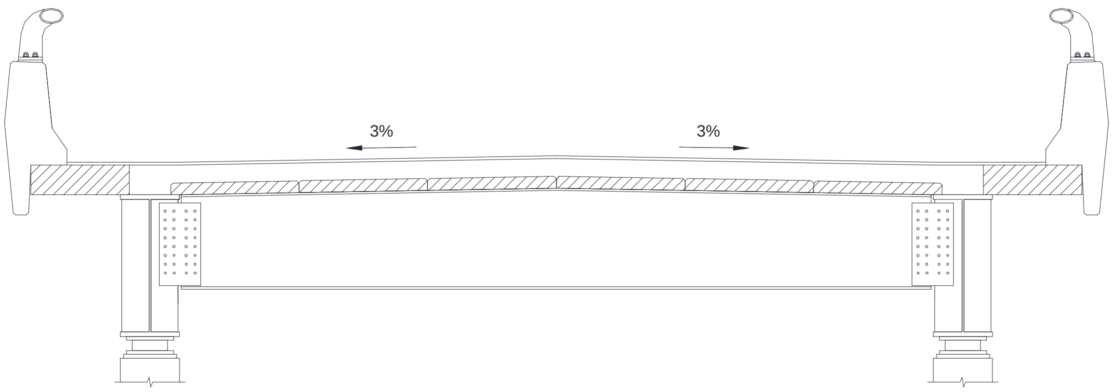
2.2.3.2 Intermediate cross girders in hogging moment regions

In the hogging moment regions of the main girders, adjacent to internal supports, the intermediate cross girders are required to provide lateral restraint to the bottom flanges of the main girders, which are in compression. This restraint is provided through the 'inverted U-frames' formed by the cross girders and web stiffeners to which they are attached. The connections between main and cross girders therefore need to transmit restraint moments and the frame needs to be stiff. If the cross girders are less than half the depth of the main girder, knee bracing or haunched cross girders may be needed, both to stiffen the frame and to reduce moments that need to be transmitted through the cross/main girder connections (see section 2.2.3.3. for descriptions of such arrangements for support cross girders).

2.2.3.3 Cross girders at internal supports (pier diaphragms)

At the internal supports of continuous spans, the cross girders are very often deeper than the intermediate cross girders, providing a stiffer and stronger 'pier diaphragm', with bolted connections that can transfer the larger restraint forces that occur at the supports (see figure 2.8). The cross girder should not be as deep as the main girders, to avoid conflict with, and direct connection to, the bottom flange of the main girder.

Figure 2.8 Cross girder at an intermediate support of a ladder deck bridge



As an alternative to using a deeper cross girder, knee bracing or a haunched cross girder can be provided, as shown in figures 2.9 and 2.10. This will stiffen the frame and reduce moments that need to be transmitted through the cross/main girder connections and may be advantageous if services or access ways are connected to the soffits of the cross girders along the length of the bridge. In practice, intermediate knee bracing is rarely specified; it is more economical to use a deeper cross girder. Haunched cross girders are an even more expensive detail and a fabricator should be consulted before selecting this option.

Figure 2.9 Knee bracing arrangement

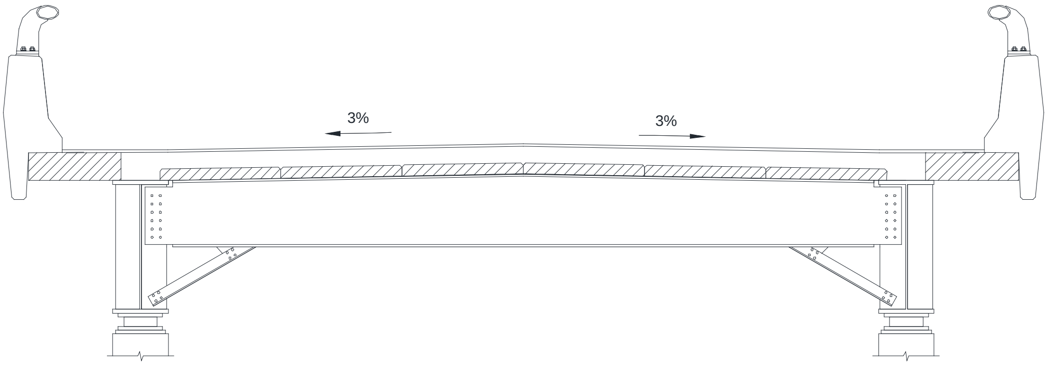
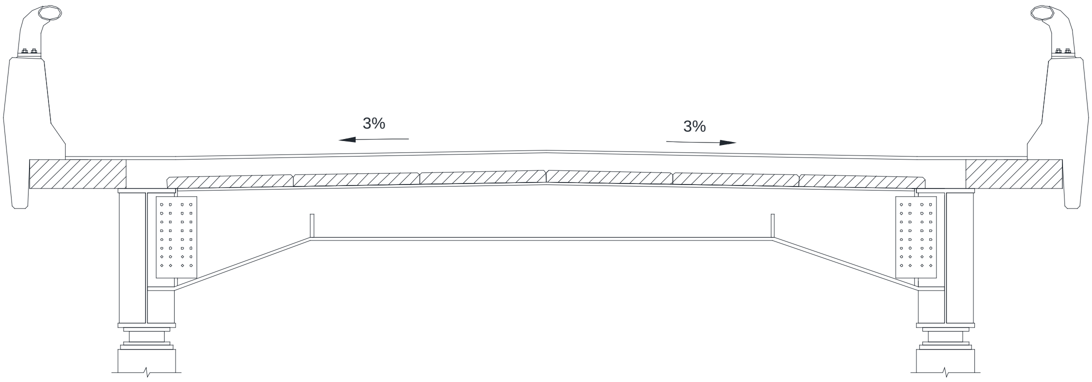


Figure 2.10 Haunched cross girder at an intermediate support



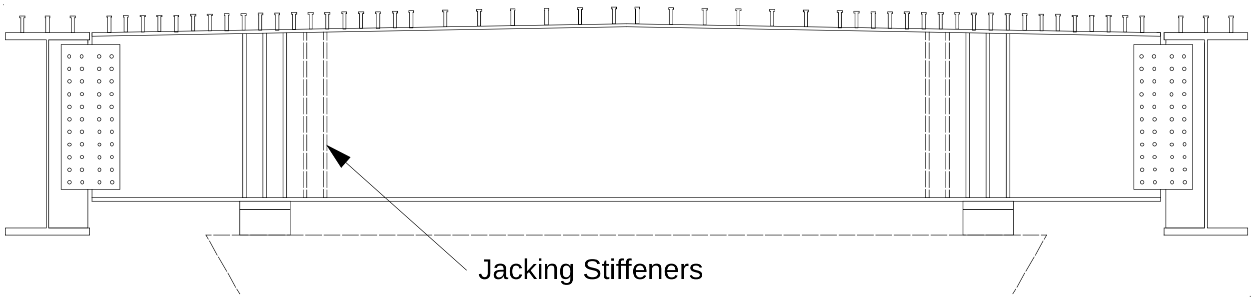
2.2.3.4 End supports

With non-integral construction, support diaphragms similar to those at intermediate supports are used. They provide an effective support to the end of the deck slab and to the expansion joint. Where the end supports are skew to the bridge axis, the diaphragms may act as trimmer girders (see section 2.5.2). For discussion of integral abutments, see section 2.6.4.

2.2.3.5 Integral crossheads at internal supports

Supports are sometimes provided ‘inboard’ of the main girders, under the pier diaphragms, rather than directly under the main girders as shown in figure 2.11. The diaphragms are then more substantial and are often referred to as ‘integral crossheads’. There may be good reasons for such an arrangement, particularly when it is difficult to provide support under one of the girders on a skew bridge, but it does add considerably to the fabrication and erection cost. If the main girders are haunched, such an arrangement, with no direct support under the most heavily loaded elements, may cause visual concern to some observers.

Figure 2.11 Example of an integral crosshead



Note that if the main girders of the arrangement shown in figure 2.11 were haunched, stiffeners would be required on both sides of the main girder webs and the web/flange connections would need to be designed for the tensile load due to vertical components of the forces in the inclined main girder flange.

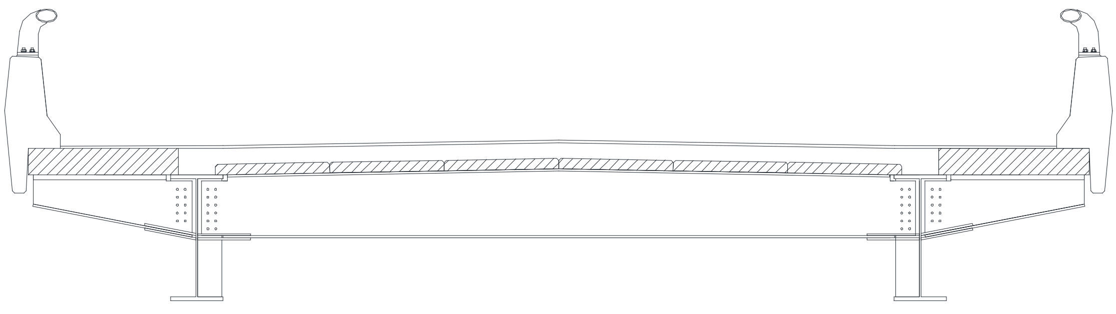
2.2.4 Cantilever girders

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

Steel cantilever girders allow longer deck cantilevers to be provided, but the main reason for considering them would be to avoid the need for cantilevered formwork or to support ‘rigid’ traffic barriers if needed. With cantilever girders, permanent formwork, in the form of precast concrete decking, can be used across the full width of the deck. In particular, cantilever girders are required when the concrete slabs are placed longitudinally to the main girders.

The provision of cantilever girders leads to the requirement for moment continuity with the cross girders. This adds significantly to fabrication cost. Also, it is difficult to achieve good alignment at the tips of long cantilevers and this too adds to cost. A cross section of a ladder deck with cantilever girders is shown in figure 2.12.

Figure 2.12 Ladder deck with cantilever girders



Alternatively, if longitudinally (ie perpendicular to the main girders) placed concrete slabs are used; the cantilever girders can be designed without the need for moment continuity. This can be done by using full depth precast deck edge slabs with reinforcement extending into the main deck slab, which becomes part of the main slab once the in situ concrete topping is set. This will allow the edge slab to behave as a traditional cantilevered element off the main girder. This is shown in figures 2.6 and 2.13, where the cantilever girders are used during the construction stage only but left on the bridge after completion as a visual feature.

Figure 2.13 Ladder deck bridge with cantilever beams (image courtesy of SKM)

2.3 Deck slab

To resist the combined load effects of local and global bending (particularly, for ladder decks, the global bending in hogging moment regions results in tensile forces within the slab) a typical deck slab thickness of 250mm is needed (the value depends partly on requirements for cover to reinforcement – see discussion in section 7.10). The slab reinforcement is typically ϕ20 bars at 150mm centres top and bottom. This thickness of slab (up to 250mm) can be cantilevered up to about 2m and carry applied loads (overall length, from centreline of main girder to outside of slab edge). When the deck slab undergoes accidental traffic loading through the barrier, the design of the deck must ensure that the failure is confined to the barrier only with no damage to the fixings to the deck, the deck slab and supporting structure. Alternatively, cantilever girders can be used, as described in section 2.2.4. Guidance on design of barriers is given in appendix B of the *Bridge manual*.

Casting all the concrete in situ, on temporary formwork, is the traditional method of slab construction. However, the costs of providing and removing temporary formwork, together with health and safety concerns for those activities, has resulted in a preference for the use of permanent formwork either in the form of precast decking or profiled steel decking. Full thickness precast units are also used.

In recent projects, partial depth precast decking has shown to be the most cost-effective solution for composite bridges in comparison with the in situ decking and full depth precast slabs (see section 3.3 and appendix A for details). The partial depth units act initially as permanent formwork, and then the remaining deck thickness is poured in situ. Projecting reinforcement on the upper face assists in ensuring that the precast and in situ components act fully as a slab. When the precast decking is positioned transversally across the bridge, it is either placed between the girders or pockets are designed to allow for the shear connectors depending on the type of precast decking used. If the decking is placed longitudinally with the main girders, pockets are not required as the units will be placed on the edge of the girder flanges, clear of shear connectors.

Use of profiled steel decking as permanent formwork has been shown to provide further cost benefits compared with other types of decking systems (as shown in section 3.4.1). However, durability is the prime concern with this form of decking, both in terms of fatigue and corrosion. In such cases, the bridge designer should follow the appropriate NZ Transport Agency (NZTA) requirements for the design of this type of formwork.

2.4 Shear connection

There are a number of ways of providing a shear connection between the steel girders and the deck slab, but by far the most common is the headed stud connector. This is a headed dowel that is welded to the top flange using a special semi-automatic welding tool that supplies an electrical pulse sufficient to fuse the end of the dowel to the flange. In New Zealand, the technology is available to be able to weld studs on-site, regardless of weather conditions. However, most studs are welded in the fabrication shop during the fabrication of the beams, which is the most cost-effective option and avoids the risk of damage to protective coatings.

There are other forms of shear connectors, such as welding steel bars with hoops, perforated plates, welded T-shaped plates and short lengths of channel. Although these connectors provide a higher longitudinal shear resistance per unit, as demonstrated by Vianna et al (2008) for perforated plates and T-shaped plates, they each have to be welded manually and consequently are more expensive.

2.5 Dealing with curvature and skew

2.5.1 Curved decks

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

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

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

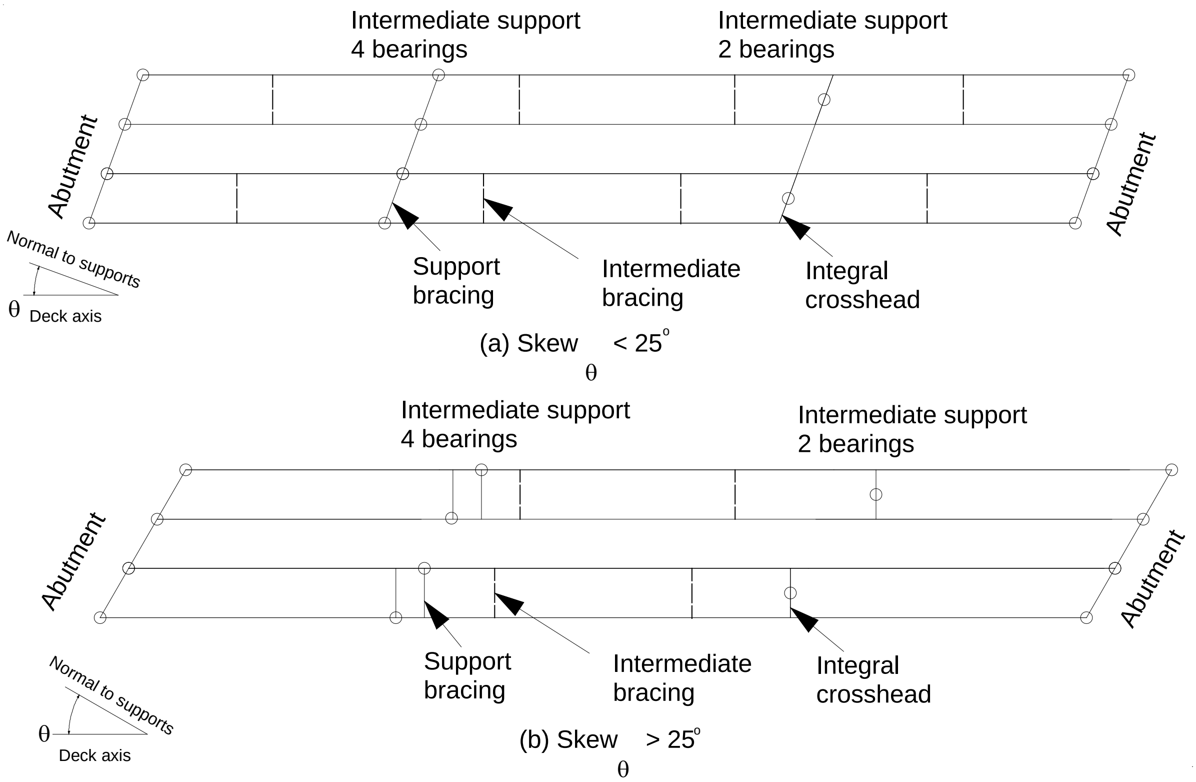
2.5.2 Skewed bridges

2.5.2.1 Multi-girder bridges

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

At intermediate and end supports, bracing is usually arranged on the line of the skew supports for small skew angles (less than about 25°); for large skew angles, bracing at intermediate supports is usually square to the main girders but bracing at the ends is along the line of the supports. Typical bracing arrangements are shown in figure 2.14. Note that for small skews integral crossheads are usually continuous (although the continuity girders between the inner main girders are much lighter than those over the support). For larger skews there are no continuity girders between the inner girders, to avoid potential fatigue problems, although continuity bracing may be needed for construction, to control twist at the wet concrete stage. Further guidance on skew is found in section 4 of the *Bridge manual*, with additional complementary guidance given in guidance note no.1.02 of Hendy and Iles (2009).

Figure 2.14 Arrangements for skewed multi-girder decks

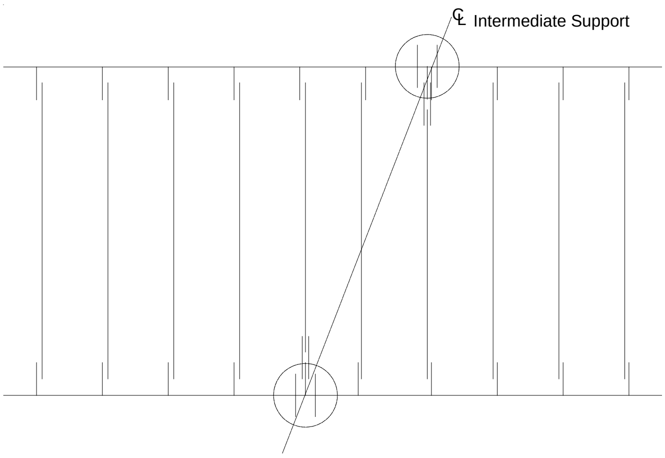


2.5.2.2 Ladder deck bridges

Skewed intermediate supports

A particular merit of the ladder deck steelwork system is that skewed intermediate supports can be readily accommodated, as one end of a cross girder can be connected to the bearing stiffener over the support to one girder whilst the other end can be connected to an intermediate stiffener within the span. With such an arrangement the cross girders will not necessarily be at a regular spacing along the length of the deck, but will be spaced as dictated by the geometry of the skew (see figure 2.15).

Figure 2.15 Arrangement at skewed intermediate support



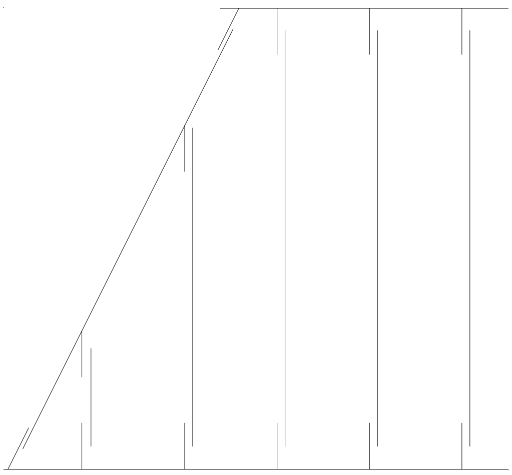
Note: Only the girder webs and the web stiffeners are shown, for clarity.

Skewed end supports

At skew end supports, trimmed cross girders, connected into an end trimmer girder, may be required, as shown in figure 2.16. This arrangement is usually preferred to a 'fanned' arrangement of cross girders. To simplify connection details, the connections at the obtuse corner for the end trimmer and the cross girder are separated, although the consequences on slab design in this area must be considered carefully and 3D modelling may be needed in order to predict the local behaviour with sufficient accuracy.

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

Figure 2.16 Arrangement at skewed end support



2.6 Substructures

Substructures are often reinforced concrete construction and consist of the structural components of the bridges that are below the superstructure – the piers, abutments, retaining walls, pile caps and piles. The superstructure is allowed to expand and contract by the use of expansion joints at the roadway level and is 'resting' on structural bearings on top of the abutments and intermediate supports. However, the use of concrete filled steel tubular columns can be cost-effective, with design guidance available in *NZS 3404* (SNZ 1997b; 2009).

The bridge substructure is an important component of the overall cost of the composite bridge and it is where most of the cost savings may be achieved due to the light weight of steel-concrete composite bridges (demonstrated in section 3.4). These are typically 40% lighter overall when compared to concrete bridges; thereby lighter substructures can be used. The design of substructures is outside the scope of this publication; however, it is recommended that the designer is familiar with the substructure design requirements that are given in sections 4 and 6 of the *Bridge manual*, as well as the effect of the substructure on the total cost of the structure.

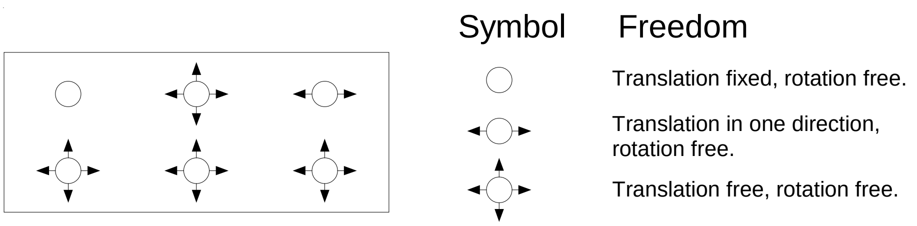
Throughout the years, experience has shown that the use of expansion joints and bearings results in relatively expensive maintenance problems that require regular maintenance and replacement. This is due to normal wear and tear and the ingress of dirt and contaminated road run-off that results in extensive deterioration of the structure as a whole. This experience has led to the development of integral bridges, where there is no joint at roadway level and the support structure is forced to displace with the movements of the deck. Different forms of integral abutment construction that are used are summarised in section 2.6.4. Detailed design of integral abutments is outside the scope of this publication, but some of the detailing issues are covered in Iles (2010) and reproduced in section 2.6.4.

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

2.6.1 Bridge articulation

In non-integral bridges, the bridge deck is supported on bearings at each support and lateral restraint is provided at either some of these bearings or by shear keys located either side of the bearings. This arrangement of the restraints, which permits the thermal expansion and contraction of the deck, is known as articulation. A typical arrangement for a two-span bridge supported on pot bearings is shown in figure 2.17; alternative arrangements and a general discussion of articulation are given in guidance note no. 1.04 in Hendy and Iles (2009).

Figure 2.17 Articulation of a two-span bridge, where fixity is provided by bearings



When the deck is curved in plan, the alignment of guided bearings must be considered carefully, since the deck tries to increase/decrease in radius as well as expand/contract in length. Examples of articulation for curved decks are also included in guidance note no. 1.04 in Hendy and Iles (2009).

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

2.6.2 Intermediate supports

2.6.2.1 Multi-girder decks

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

2.6.2.2 Ladder decks

Ladder decks are usually supported by columns directly under each main girder; this achieves an open appearance beneath the bridge as well as economy. For river crossings, leaf piers or full length cutwater (ie a long bridge pier with wedge-shaped ends) with columns above may be preferred for hydrology reasons. Requirements for replacement of bearings may dictate the minimum size of columns, as it is preferable that jacks can be placed on the top of each column, so allowing the steelwork to be jacked off the columns to replace bearings.

Leaf piers, rather than individual columns, are sometimes used when the bridge bearings are located inboard of the main girders and so-called integral cross-heads are provided (see figure 2.11).

2.6.2.3 Intermediate supports for bridges with integral abutments

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

2.6.3 End supports - non-integral abutments

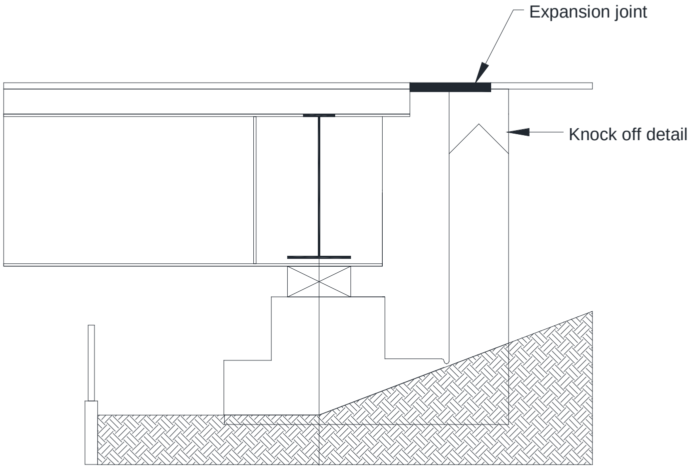
For non-integral construction, bearings will be needed under the main girders and an expansion joint will need to be provided in the deck above. In the UK, an inspection gallery is also provided beneath the expansion joint; however, this is not mandatory in New Zealand, with the cost of installing such a gallery adding to the construction cost, especially for small and medium-sized bridges. Even though it is a good idea in principle, the designer must consider the cost implication of such an addition to the overall bridge cost.

A typical arrangement for a non-integral abutment for a ladder deck bridge, with knock off detail that is required in seismic design, is shown in figure 2.18. It should be noted that the following abutment figures exclude settlement slabs for clarity, which are required by the *Bridge manual*.

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

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

Figure 2.18 Non-integral abutment with knock off detail



2.6.4 End supports - integral abutments

For bridges up to 55m overall length, integral abutments can be used if the skew angle is not more than about 30°, as stated in section 4.7.1(c) of the *Bridge manual*. Longer integral bridges are acceptable providing suitable analysis methods are used. There are three types of integral construction that can be used for composite bridges in New Zealand:

- fully integral bridges – framed abutments
- fully integral bridges – bank pad abutments
- semi-integral bridges–- with bearings

The forms of these abutments are discussed below; design and detailing issues are discussed in detail in Iles (2010).

2.6.4.1 Framed abutments

Framed abutments are usually built with H-piles or reinforced concrete piles, with the piles inside sleeves, thus avoiding earth pressures on the piles as the bridge expands and contracts (Iles 2010). A typical arrangement is shown in figure 2.19, with a normal earth slope in front and in figure 2.20 with a reinforced earth retaining wall. Depending on soil conditions, one or two piles are provided for each main girder in multi-girder bridges; for a ladder deck bridge of the same overall width, a similar total would be provided, though they might be concentrated around the positions of the main girders. Framed abutments are also built with reinforced concrete abutment walls on strip footings, although that form of construction is not discussed in this guide.

Figure 2.19 Framed integral abutment - with normal earth slope

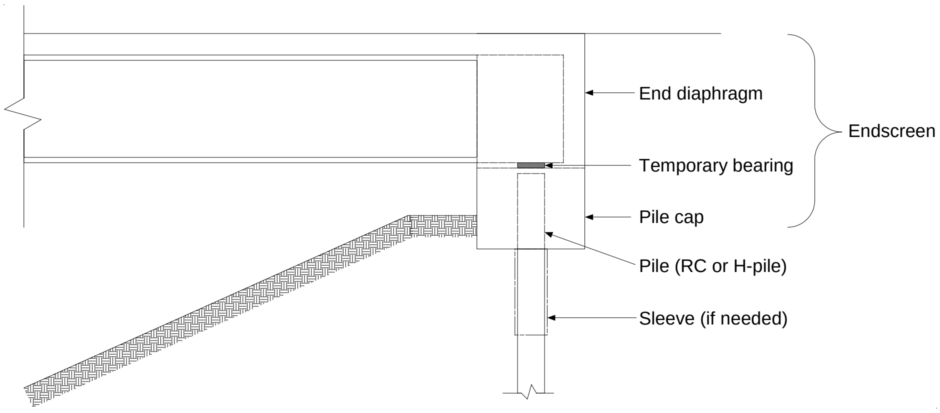
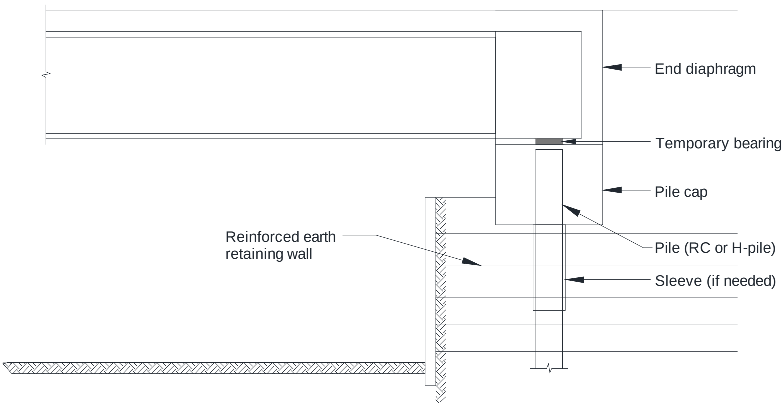


Figure 2.20 Framed abutment – with reinforced earth retaining wall



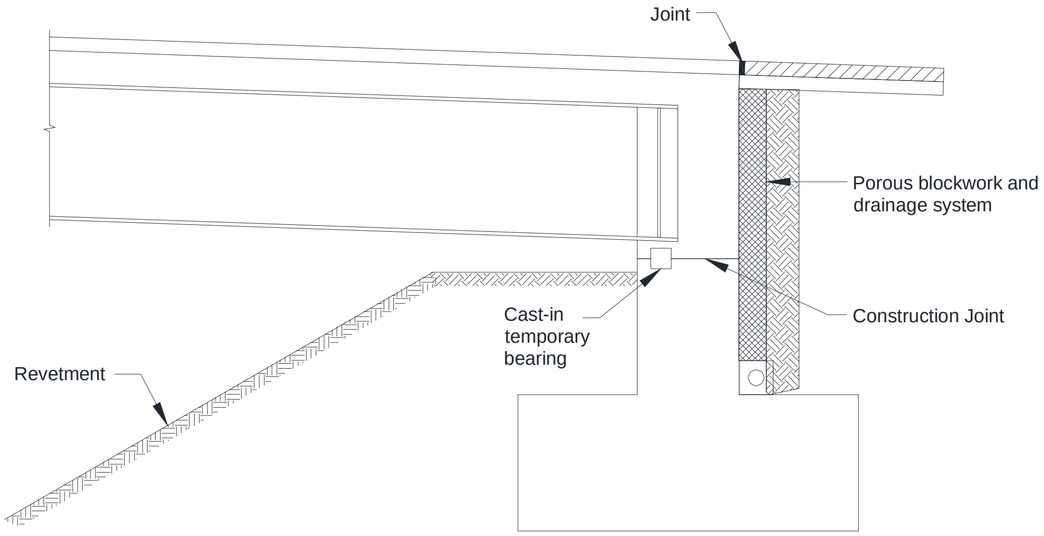
In principle, any type of bearing pile, including steel H-piles, can be driven into the ground and the end-screen wall cast around the tops of the piles. In practice, only a small number of bridges have been built with H-piles. Where construction has used H-piles, they have usually been encased in a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

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

2.6.4.2 Integral bank pad abutment

In an integral bank pad abutment, an end-screen wall is cast around the ends of the girders and this wall sits directly on the soil beneath. A typical arrangement is shown in figure 2.21. Guidance on the type of deck joint to use is given in section 4.7.4 of the *Bridge manual*. Finally, even though the porous block work and drainage system is not currently used in New Zealand, designers should consider the benefits of reducing water pressure build-up and reduction of potential durability issues in future bridge design.

Figure 2.21 Integral bank pad abutment

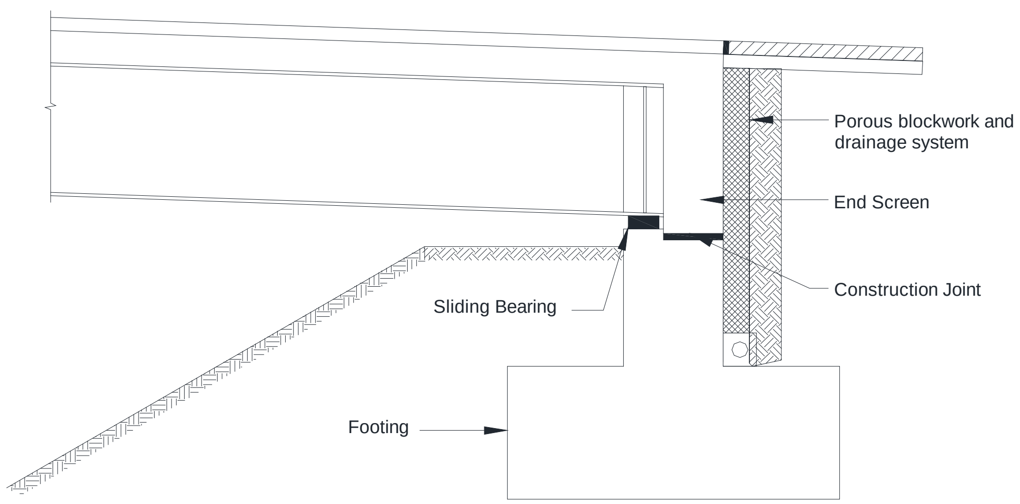


Because the expansion and contraction of the deck causes the foundation to slide and rotate on the soil, the design bearing resistance of the soil has to be reduced. This type of abutment is better suited to situations where the soil is non-cohesive (or where cohesive material has been dug out and replaced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

2.6.4.3 Semi-integral abutment

In a semi-integral abutment an end screen wall is generally provided across the end of the deck, with the girders supported on bearings in front of the wall. A typical arrangement is shown in figure 2.22. This form of abutment can be used either with revetment slopes in front of the abutment or behind a retaining wall. It is particularly suitable where there is a reinforced earth retaining wall. However, replacement of the bearings will require jacking and because of concerns about the forces involved and the movement at the interface with the soil, it is a less favoured solution (Iles 2010).

Figure 2.22 Semi-integral abutment



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

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

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

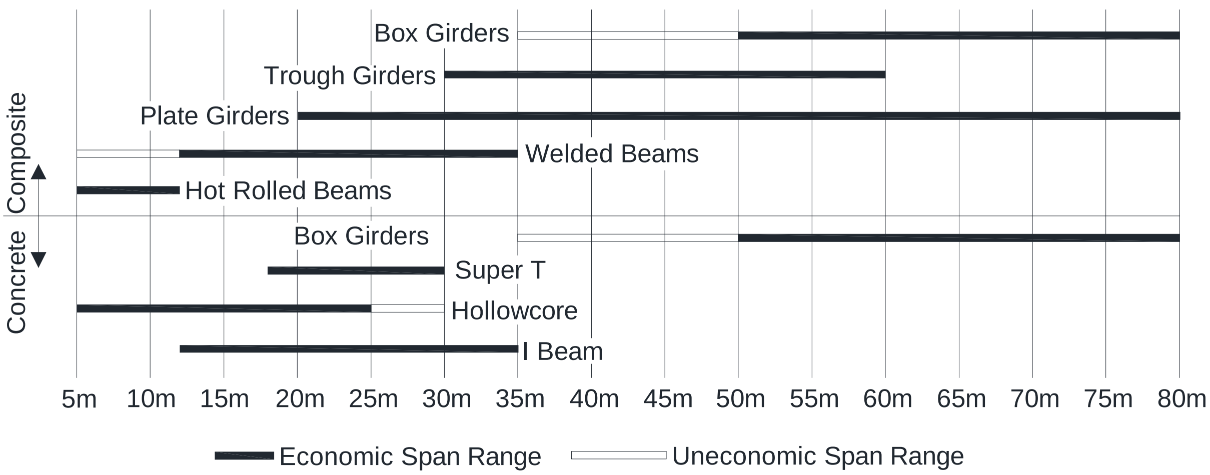
3 Benefits of steel-concrete composite construction

3.1 Why specify a composite bridge?

Most short span bridges in New Zealand (and in most countries around the world) with spans up to 30m have been built in concrete, typically comprising precast concrete beams and reinforced concrete decks. However, a well-designed steel-concrete composite bridge comprising steel girders and reinforced concrete decks will also provide an economical and sustainable solution. Figure 3.1 demonstrates the potential spans to superstructure type for both steel-concrete composite and concrete bridges, even though longer spans may be possible for some of the options shown.

Although steel bridges are commonly used in medium spans (between 30m to 80m) and especially for long spans (greater than 80m), the guidance and design philosophy outlined in this section provides the designer with the tools to design a cost-effective steel-concrete composite bridge, whether it is a short, medium or long span structure.

Figure 3.1 Span to superstructure type for road bridges



3.2 Sustainability benefits

The combination of steel and concrete in a single composite structural element enhances the individual advantages of both materials. By utilising the high tensile strength of steel together with the compressive strength of concrete, the resulting elements have one and a half times or even double the strength and stiffness in comparison with a non-composite element. This is the main advantage of composite bridge construction, which is recognised worldwide and the reason why it is widespread in the UK, Europe and Northern America.

In regards to the sustainability aspects of both materials, there are a number of articles and papers written on this topic, such as *Sustainable steel construction* (Corus 2006) and *Concrete^3 economic, social and environmental* (CCANZ 2007). This section provides an overview of the sustainability benefits of steel, concrete and composite elements.

3.2.1 Supply

Both steel and concrete are readily available in New Zealand and from overseas.

3.2.2 Recycling and reuse

An important sustainability attribute of steel is its ability to be repeatedly re-used or recycled without any degradation in the quality and mechanised properties of the material. Other materials are often recycled only once before down cycling, which means they eventually find their way to landfill.

Steel never loses its value and has a sustainable economic life cycle that is unrivalled by most other construction materials. All used steel has a value, whether it is being re-used or recycled. This recycling property was already being utilised before sustainability became an issue. There was never any need to legislate for it to be recycled, as steel has an intrinsic value as a scrap material and is always in demand for the production of new steel.

In regards to concrete, due to the limited local supply of aggregate in some areas around New Zealand (such as Auckland), it is now a viable option to recycle concrete aggregate in purpose-built facilities. These plants are able to accept, process and sell recycled concrete aggregate. Recycled concrete aggregate is separated from its recyclable reinforcing steel and is processed into specific aggregate sizes.

3.2.3 Waste minimisation

Little waste material is generated during the manufacture of steel components, and most of this is recovered and recycled. On construction sites, which can often generate large volumes of waste of other material, off-site fabrication ensures that no steel is wasted, as only what is needed comes to site. Almost all of the material waste generated in the fabrication shop is recovered for re-use or recycling.

With enhanced understanding of the impact of water and waste disposal, there has been considerable progress in reducing wastewater discharge across the concrete industry's production facilities. It is now common practice to use chemical wash waste systems and aggregate reclaimers to minimise wash waste and water from the cleaning of truck-mixer bowls and plant.

3.2.4 Off-site manufacture

Off-site manufacture has always been a key feature of steel construction as it is for concrete with precast construction such as bridge decking, which allows a composite structure to score highly on many sustainability criteria.

More accurate components can be achieved with composite elements manufactured and fabricated off-site. Waste is minimised and high-quality, defect-free products are possible. In the modern fabrication workshops and pre-casting yards, where state of the art numerically controlled machinery is fully integrated with computer aided design (CAD) and other software, composite elements can be easily standardised, tested and certified. Corrosion protection coatings can be applied to steel elements at the fabrication stage, reducing the overall site construction programme. Anti-graffiti coatings can also be applied to concrete elements if needed.

Local communities benefit from off-site manufacture, as there is much less traffic to sites resulting in less local traffic congestion. Another benefit for local people is that pre-fabricated construction is dry, dust-free and relatively quiet.

Off-site manufactured elements lead to more predictable construction programmes. Site managers benefit from just-in-time delivery, being able to hold these elements at depots or at the contractors' workshop until needed, saving space and reducing the possibility of damage from on-site storage. Once delivered to site, pre-engineered elements are speedily and safely erected. Prefabricated elements are also inherently safer, requiring fewer and generally well trained people to install them. The site activities are predictable and well practised.

3.3 Economic benefits

Appendix A outlines the findings of a cost comparison study for three and four-girder options for a two-lane bridge; this is an abridged version of El Sarraf (2008). The results for bridges with the spans of 12m, 15m, 21m, 27m and 30m, have been reconfigured as a graphical representation for ease of view in figure 3.2. The Y-axis represents, in the following order, the bridge span, number of girders, number of braces and the deck type (which is given in table A.2 in appendix A6). The X-axis represents the cost of the bridge minus the decking and then separately the decking cost. The sum of these figures provides the total cost of the bridge.

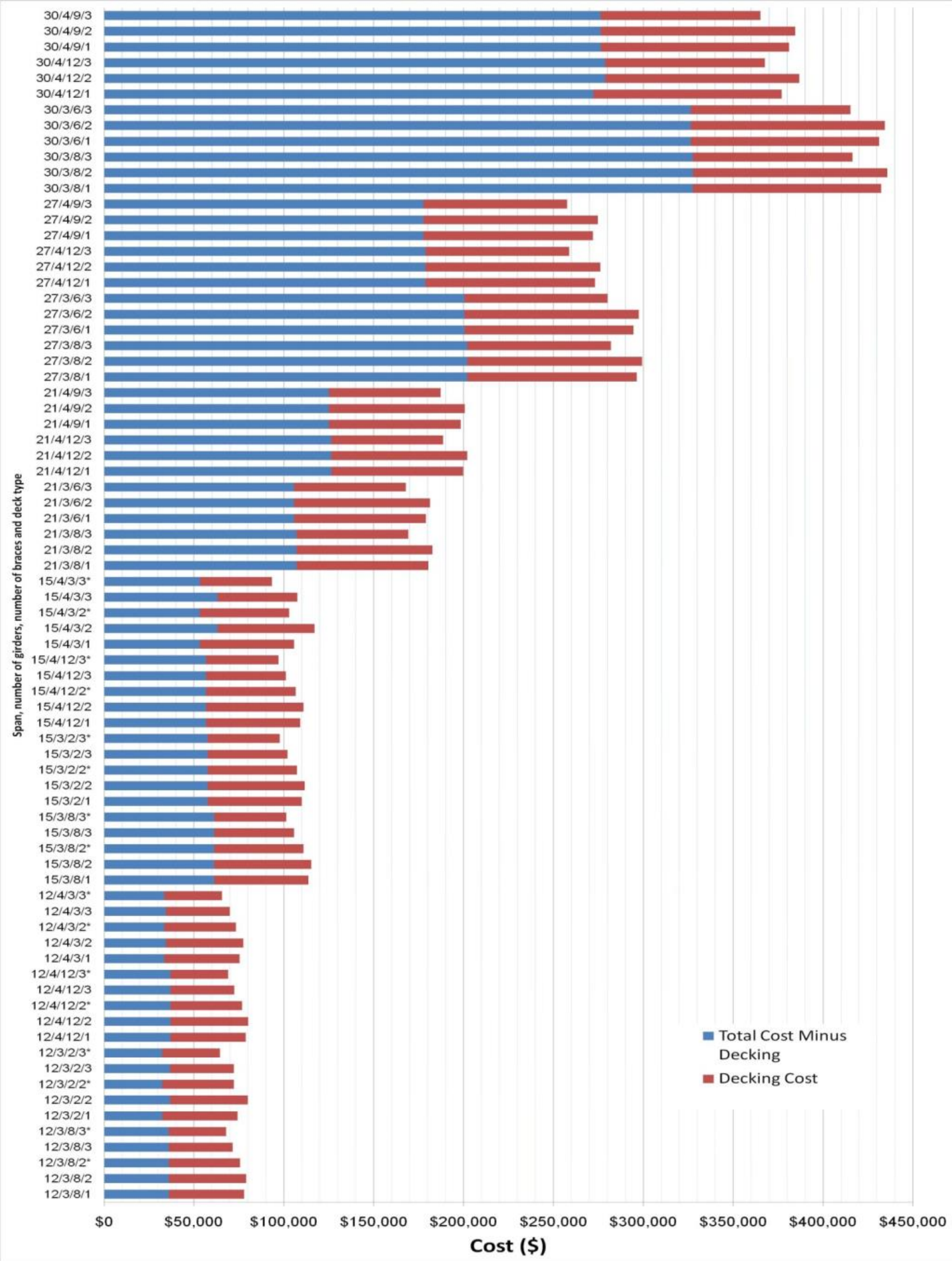
In table A.4 and figure 3.2, the percentage difference between the different decking options indicates that the partial depth precast decking is the cheapest option, followed by the in situ decking and then followed closely by the full depth precast decking. This is consistent with feedback from contractors. The other issue that determines the use of the type of decking is the reinforcement bending requirements (see section 4.2.3).

Figure 3.2 also shows that the chosen decking makes up a large percentage of the total cost of the bridge. Concrete decking cost can account for up to 55% of the total cost of shorter span bridges but can be as low as 21% of the total cost of longer span bridges. As the span increases, the ratio of decking cost to total cost decreases, making the correct choice of decking especially critical for shorter span bridges. An interesting result is that the four-girder option provides a more cost-effective solution than the three-girder option; this shows that the main issue designers must be aware of is that the total steelwork tonnage cost is the governing factor and not the number of girders used (this tonnage includes the weight of the bracing units).

The effect of the number of bracing units on the main girder size is also significant. Braces are required during the erection of the superstructure to provide lateral restraint to the top flange until the decking is placed and acts compositely with the main girders; the less bracing used, the bigger the main girder size.

Taking all these factors into account, the results suggest that the optimum decking option for a composite steel/concrete bridge is the partial depth precast decking with the lowest steelwork tonnage regardless of the number of girders.

Figure 3.2 Graphical representation of the two-lane, three and four-girder bridge costing data



3.4 New Zealand case studies

The following bridge case studies provide a cost comparison between a conventional concrete bridge and steel-concrete composite ladder deck system (see section 2.2).

3.4.1 Weiti Stream Bridge

In late 2006, a three-span private bridge was designed originally with a conventional double hollow core deck on pier cross-head beams. The 17m span is within the optimum range for hollow core units. However, by changing the superstructure to a steel/concrete composite ladder deck system, taking advantage of continuity over the three spans and eliminating the expensive cross-head girders, a more cost-effective solution was obtained. The ladder deck system consisted of two main girders at 9m centres, with 18 intermediate cross girders at 3m centres. Initially, the chosen decking system for the composite bridge was a partial depth precast concrete decking; however, due to manufacturing issues which affected the project delivery date, a steel decking option was considered as permanent formwork. The chosen decking system was a Tata International ComFlor 80 (Tata International 2005) steel decking with a total of 330mm in situ concrete topping to minimise formwork requirements. The decking was only used as permanent formwork and was not considered to act compositely with the concrete. Table 3.1 shows the cost comparison between the hollow core decks, partial depth precast decking and the use of the steel decking option.

From table 3.1 it is clear that the composite steel bridge options were more cost effective than the conventional hollow core concrete decking, once the total cost of the project was considered. The superstructure costs of the three options are comparable, with most of the cost savings attributed to the substructure, especially with the removal of the pier cross head beam and the replacement of the abutments with a concrete segmental retaining wall. An interesting fact is that the steel decking option provided greater cost savings than the partial depth precast concrete decking option, with a savings of NZ$37,492 between the two composite steel bridges options. The steel-concrete composite bridge provided a total of NZ$76,295 savings in comparison to the concrete double hollow core bridge.

Further savings were achieved by removing the abutment and replacing it with a concrete segmental retaining wall with a fill behind the wall. The fill had driven timber piles to support the crane bearing pads during construction.

This bridge was for a private client and had a specified design life of 50 years, whereas the *Bridge manual* requires 100 years. Appearance was not a concern; therefore unprotected steel was used with a design allowance for loss of steel from corrosion in accordance with El Sarraf and Clifton (2011). However, the cost of a coatings system to give a period of 35 years prior to its first maintenance would have been NZ$28,000, giving a net cost saving for a coated bridge of NZ$48,295 for the steel decking composite steel bridge option.

It should be noted that there are limitations on the use of steel decking from a durability perspective. However, as this option was used as a permanent formwork only, this was not considered to be an issue. Certain corrosion protection measures can be undertaken to increase the design life of the steel decking as required. The fact that this was a privately funded project allowed the innovative use of steel decking, as a permanent formwork. For a publicly funded project, the guidance given in the NZTA contract documents on the use of steel decking should be followed for each project.

Table 3.1 Cost comparison between the concrete and a composite steel bridge options

Item	Concrete bridge	Steel bridge	
	DHC deck NZ$	Concrete decking NZ$	Steel decking NZ$
Preliminary and general	$38,700	$30,200	$30,200
Earthworks	$9300	$9300	$9300
800mm diameter piles	$113,179	$113,179	$113,179
Abutment and pier crosshead beams	$81,238	$0	$0
Abutment wing walls	$15,097	$0	$0
Settlement slabs	$12,160	$0	$0
Superstructure	$351,097	$390,890	$353,398
Guard rails	$27,620	$27,620	$27,620
Storm water drainage	$1925	$1925	$1925
Asphalt paving	$16,640	$16,640	$16,640
Timber pole retaining walls	$0	$8400	$8400
Contingencies	$10,000	$10,000	$10,000
Concrete segmental retaining wall	$0	$30,000	$30,000
Total cost	**$676,956**	**$638,154**	**$600,662**
Savings		$38,802	$76,294

Weiti Stream Bridge was built and completed in mid-2007. Since then the cost of steel has increased and subsequently decreased, which will affect the costs given in table 3.1. This however does not affect the lessons learned from this case, of designing (or at least considering) the substructure early on in the project and utilising the low superstructure weight of a composite steel-concrete bridge and the high strength to weight ratio of steel.

3.4.2 SH4 Okura Realignment Project

The Okura Realignment Project demonstrates that the design concept outlined in section 3.5, if used correctly, will provide cost savings regardless of the material cost.

Located on State Highway 4, inland from Whanganui, the existing road alignment was subject to on-going subsidence and erosion, coupled with a sub-standard road alignment. To address this issue, realignment of the road was needed with a curved 90m North Bridge (having a constant cross fall of 7.6%) and a straight 96.6m long South Bridge (having a 3% cross fall either wide of the bridge centreline). The conforming design consisted of two three-span bridges carrying the realigned route, using precast prestressed concrete Super T-girders. This choice posed a number of key logistical challenges for the contractor including:

- transporting the over-length and overweight concrete girders to the site
- poor access to each bridge site with steep river banks and a final bridge deck level elevated 15m–20m above river level (this created issues for positioning cranes to enable the girders to be lifted into place)
- large diameter pile foundations up to 2.4m diameter within the river channel (access to the middle of the river channel would have been an issue and the pile sizes were outside the range of the contractor's plant capability).

By utilising the benefits of a steel-concrete composite via ladder deck system, this form of construction addressed the transportation and construction issues encountered with the conforming design; this included eliminating the need for a piled foundation in the river. The reduced weight of the steel ladder deck bridge superstructure allowed considerable savings to be made to the size of the foundations required for each bridge. This also included the elimination of concrete pier head beams, which resulted in further cost savings in both construction time and material. See figure 3.3 for details.

Figure 3.3 SH4 Okura Southern Bridge (image courtesy of Holmes Consulting Group)

Table 3.2 shows the cost for the South Bridge which was a straight 96m long bridge of 30m–36m–30m spans. The total bridge decking area was 936m^2.

Table 3.2 shows similar results to that for the Weiti Stream Bridge outlined in table 3.1; the superstructure cost of both options are comparable, with most of the cost savings found at the substructure. Due to the lighter superstructure weight, using a composite steel-concrete option instead of a concrete option, the cost savings in the foundations and columns amounted to a saving of NZ$334/m^2 of decking area. The total cost savings amounted to NZ$417,456 between the two options.

Table 3.2 Cost comparison between the concrete and a composite steel bridge options

Southern Bridge (with 936m^2 deck area)	Concrete bridge (NZ$/m^2)	Composite bridge (NZ$/m^2)
Foundations and columns	$1054	$720
Abutments/wing walls/pier cap	$183	$121
Beam and deck	$1168	$1118
Total cost ($)	$2,251,080	$1,833,624
Savings		$417,456

3.5 Design concept

To summarise the study results outlined in section 3.3, what was learned in section 3.4.1 and used in section 3.4.2, the following factors should be considered by the designer during the design of a bridge:

- **Superstructure steelwork:** Use the least overall main girder weight, if possible, independent of the number of girders, with minimum numbers of braces. This is found to be the best option to reduce handling and erection cost irrespective of the coating cost. Ladder deck bridges also provide further cost savings in the substructure.
- **Slab construction:** Partial depth precast solutions provide a cost-effective solution when all factors and constraints are taken into account. However, in situ concrete on steel decking as permanent formwork, with the appropriate corrosion protection system may provide additional cost savings.
- **Piers:** The number of piers and size of pier cross head girders can equate to a significant percentage of the total cost of the structure. Utilise the increased composite high weight to strength ratio to maximise the bridge span to reduce the number and size of the piers.
- **Abutments:** Replace the abutments with concrete segmental retaining walls (or similar) and concrete piers or steel piles.

4 Preliminary design

4.1 General

In the preliminary design stage, the bridge designer develops a structural solution based on the client's requirements that are outlined in the design statement in accordance with the *Bridge manual*. It is recommended that consultation with other representatives of the bridge industry, such as fabricators and coating suppliers, should be conducted at an early stage as they will be able to provide advice on optimising the bridge design. This will increase the efficiency and cost effectiveness of the steel-concrete composite bridge option.

4.2 Design for construction

While minimising cost may be the most obvious consideration when embarking on the design of a highway bridge, the health and safety of all those concerned in the construction of the bridge and in its maintenance throughout its life is the responsibility of all decision makers related to the procurement of the bridge. As well as aiming for a structurally efficient solution, the hazards associated with the construction process must be fully appreciated from the outset.

4.2.1 Steelwork fabrication

Clean lines in the overall appearance and minimum use of complex details are most likely to lead to an economic and efficient bridge structure, though external constraints often compromise selection of the best structural solution. The fabrication of the basic I-section is not expensive, especially with the use of modern semi-automatic girder welding machines (T and I machines), such as that shown in figure 4.1.

Overall fabrication cost is of the same order as the cost of the material used. With the widespread use of computers in design and in control of fabrication shop machines, geometrical variations, such as curved soffits, varying super-elevation, plan curvature and precambering, can be readily achieved with reasonably minor cost penalties. Much of the total cost of fabrication is incurred in:

- the addition of stiffeners
- the fabrication of bracing members
- butt welding
- the attachment of ancillary items (such as stud welding)
- local detailing that leads to a significant manual input to the process.

Figure 4.1 Example of a semi-automatic girder welding machine (image courtesy of D&H)

The designer can exercise freedom in the choice of overall arrangement but should try to minimise the number of small pieces that must be dealt with during the fabrication process.

The welding required, not just for forming the plate girder but for all other welded components and connections in the structure, must be considered and designed carefully. Inappropriate details or the specification of an over-designed weld detail may result in a substantial increase in the fabrication cost of the structure.

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

Transportation by road imposes certain limitations on size and weight of fabricated assemblies. The most frequently noted limitation is a maximum length of 20m (including truck and trailer); above which special notification and procedures apply. Nevertheless, bridge fabricators in New Zealand are used to transporting longer loads, where in exceptional cases girders well over this limit have been transported. Further guidance on the transportation limitations is available in *Guide to heavy vehicle management* (NZTA 2006).

Figure 4.2 Transporting a 20.3m, 30.5 tonnes bridge girder (Image courtesy of Eastbridge)

4.2.2 Erection scheme

A scheme for erection of all the major components of the bridge needs to be considered from an early stage. Access, temporary support arrangements, stability of the part-erected structure and the need to minimise work during road or railway closures can all have an effect on the form and detail of the structure. Construction of a composite bridge superstructure usually proceeds by the sequential erection of the steelwork, working from one end to the other. Depending on the type of decking used, this is followed by concreting of the deck slab and removal of false work for in situ decking, or alternatively by the placement of partial depth precast decking followed by an in situ pour of the remainder of the decking (see section 2.3 for details). However, situations vary considerably and constraints on access may well demand a sequence that differs considerably from the usual. In some cases the access constraints will determine which structural configuration can be safely and economically used.

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

It is much quicker to establish a secure connection using bolts than by welding. On-site welded joints are more expensive, with a high risk of delay due to weather and are onerous for quality control on a small job but welding may be considered on larger jobs. Either bolting or welding should be used throughout the bridge construction; it is normally uneconomic to use both methods.

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

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

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

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

General guidance on the erection of bridge steelwork is given in the *BCSA Guide to the erection of steel bridges* (BCSA 2005).

4.2.3 Slab construction

The deck slab of a composite bridge is normally cast in situ, on either temporary or permanent formwork. Traditionally, timber formwork, fitted between the erected girders, was most commonly used for full depth in situ decks. Recently, the use of permanent formwork, such as partial depth precast concrete decking, has become common. The latter avoids the costly and potentially hazardous operation of stripping out temporary formwork after casting.

Partial depth precast decking has been used in a number of ladder deck bridges in New Zealand and can be used on multi-girder bridges as well where the top flanges of the cross girders are all in a common plane. Partial depth precast decking can be used for slab spans up to about 4m. They are placed on the girder flange edges allowing the connecting shear studs to have a solid concrete block to assist in transferring the longitudinal shear force between the steel and concrete. This is the type of partial depth precast decking considered in this guide.

Another form of permanent formwork that could be used is trapezoidal steel decking, (see sections 2.3 and 3.4.1). Decking can span up to 4m, allowing for girder spacing of the same length. Note that the total slab thickness will need to be greater than 250mm (minimum of 300mm thick) to be able to resist punching shear caused by vehicle point loads.

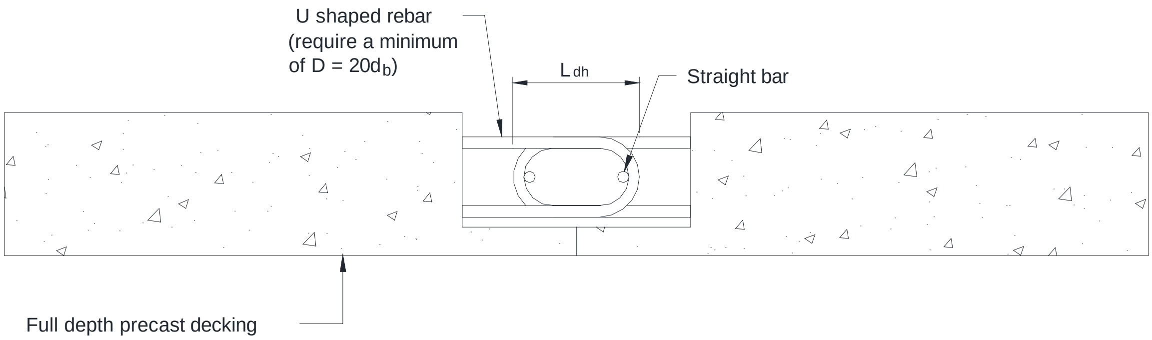
It is common not to pour the concrete over the full length of the bridge at one time but to place concrete over part lengths, in a number of stages. This choice is partly for practical reasons and partly, by concreting mid-span regions first, to minimise hogging moments due to dead load. With integral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

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

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

Finally, full depth precast concrete decking units are available; however, there are concerns about the performance of the in situ joint at the serviceability limit state (SLS). *NZS 3101 Concrete structures standard* (SNZ 2006), also prohibits the use of the common panel-to-panel connection in 'high fatigue' locations (such as traffic live loads, see section 6.4), unless there is performance verification. This connection utilises U-shaped reinforcing bars from each unit extending out from the edge and overlapping with the U-shaped bars from the adjacent unit (straight bars are then passed through the intersecting U bars to complete the reinforcement). This detail is shown in figure 4.3.

Figure 4.3 Full depth precast decking joint detail (not desirable)



4.3 Design for in-service maintenance

All publicly owned bridges are designed for 100-year design life as stated in the *Bridge manual* and are assumed to be provided with periodic maintenance, as outlined in *NZS 3404.1 Steel structures standard NZS 3404.1* (SNZ 2009), *NZS 3101* (SNZ 2006) and the coating standard *AS/NZS 2312* (SNZ 2002). The design and, in particular, the detailing, should recognise the need for durability and facilitate whatever maintenance will be necessary.

Particular issues to be addressed include:

- access for repainting
- detailing requirements of drainage arrangements so that minimal maintenance is required and their failure would not cause durability problems
- facilities for bearing replacement.

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

The Resource Management Act 1991 (RMA) regulations require the assessment of hazards during maintenance work. The design must be such as to avoid or reduce, as far as practicable, risks during maintenance, whether minimising environmental impact or considering the health and safety concerns of the maintenance personal.

4.3.1 Corrosion protection systems

Traditionally, steel bridges have been protected against the effects of corrosion by the application of protective coatings. Coating systems are available that will last up to 40 years before their first maintenance, using products that comply with current health and safety requirements and environmental regulations.

To ensure complete and reliable application, and to maximise the life of protective coatings, the arrangement and detailing of the steelwork should be such as to avoid any features that would limit access for proper application and maintenance or which would trap water and dirt in service. Guidance on detailing and the selection of a coating system is given in *NZS 3404.1* (SNZ 2009), *AS/NZS 2312* (SNZ

2002) and El Sarraf and Clifton (2011). Figure 4.4 shows the application of a coating system onto State Highway 25, Kopu Bridge.

Figure 4.4 Coating application on the Kopu Bridge beam (image courtesy of D&H)

4.3.2 Weathering steel

Weathering steel is a special alloy of carbon steel that forms a stable and tightly adhering oxide layer (or 'patina') when subject to alternate wetting and drying. Unlike ordinary rust, the patina does not fall off the surface and it prevents further oxidation. Weathering steel requires minimal, if any, maintenance, provided that it has been used in the appropriate circumstances. Figure 4.5 shows State Highway 1, Mercer to Longswamp off-ramp, New Zealand's first weathering steel bridge, opened in 2006. The appearance of the steel will darken, with a uniform appearance, after a few years.

Figure 4.5 State Highway 1 Mercer to Longswamp off-ramp, shortly after construction

The use of weathering steel results in a slightly higher material unit cost and the additional cost of a 'corrosion allowance' to the steelwork (a small addition to the thickness is required for design purposes), but saves the cost of applying a protective coating. The savings usually outweigh the extra costs as shown in the net present value example given in section 10.2.7. For guidance on the use of weathering steel refer to El Sarraf and Clifton (2005). Note that weathering steel plate greater than 20mm is only available from overseas; see appendix B for available plate sizes.

4.3.3 Bearing replacement

The design must allow bearings, both mechanical and elastomeric, to be replaced during the life of a bridge, in accordance with section 4.7 of the *Bridge manual* and clause 7.4 in part 4 of *AS 5100 Bridge standard* (SA 2004). It is relatively straightforward to stiffen the web of the main girders to permit jacking to replace bearings but there does need to be sufficient space (on top of columns, etc) on which to sit the jacks. The need for temporary supports adjacent to an intermediate support, off which to jack the structure, should be avoided, because of the substantial costs and hazards that are introduced.

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

4.4 Choice of structural configuration

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

The advantages of a ladder deck configuration are:

- a reduced tonnage of steel, relative to a multi-girder deck as a result of the best possible load share in the case of a ladder deck (although fabrication costs per tonne may be higher)
- well-suited to efficient slab construction (uniformity in thickness, easily detailed to suit precast permanent formwork and full-width placing of concrete)
- cross girders provide regular stiff restraint to cantilever construction, facilitating the use of precast cantilever units
- where horizontally curved girders are needed, the regular spacing of cross girders easily provides the restraint to the bottom flanges
- only two columns needed at each intermediate support, avoiding leaf piers and achieving a more open appearance
- reduced maintenance liabilities (less surface area of steelwork, fewer small bracing elements, and fewer bearings)
- faster construction times provide additional cost savings to the project total cost.

The advantages of a multi-girder configuration are:

- smaller piece sizes (of main girders), thus reducing crane requirements
- braced pairs of girders require no additional temporary bracing to top flanges
- fewer bolted connections on sites
- good load distribution (through transverse bending of the slab)
- readily adaptable to any bridge width
- shallow construction depth can be achieved without resorting to excessively large flange plates.

4.4.1.1 Influence of substructure constraints

As mentioned in section 3.5, it is recommended that the substructure design is considered at an early stage. The form of the substructure at intermediate supports, whether for reasons of appearance or construction, often has a strong influence on the form of the superstructure. For example, a low clearance bridge over poor ground might use multiple main girders on a single broad pier, whereas a high level bridge of the same deck width and span over good ground might use a ladder deck with twin main girders on individual columns.

4.4.1.2 Influence of skew

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

4.4.1.3 Influence of requirements for drainage

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

4.5 Preliminary sizing – material selection

4.5.1 Steel

The main structural steel members in bridges are usually grade G300 or G350 in accordance with *AS 5100* (SNZ 2004) and the relevant material standards. Higher strength steel grades, such as G450L15, are available and can be used in designs but they are more expensive and have not yet been used in New Zealand to any significant extent in bridgework.

When designing bridge girders, the designer should take into account the available steel plate dimensions to minimise wastage and optimise the girder size. For example, by cutting the maximum number of flanges out of one plate to minimise wastage this will assist in keeping the fabrication cost low. Another tip is choosing the web depth based on the available plate width. Therefore, if the designer requires a 1200mm deep beam that spans 18m, with the flanges being 750mm width and 32mm thick, and the web being 16mm, then a 1530mm wide 32mm thick plate is chosen for the flanges. This plate is cut in half for the flanges and a 1532mm wide 16mm plate is chosen for the web. To achieve the required 18m span, a 6m and 12m plate for those dimensions is chosen and butt welded together. Note that when the plate is cut, the designer should allow for 10mm loss for each cut and take into account the precamber allowance. It is recommended that designers discuss this matter with fabricators to ensure the appropriate plate dimensions are used at the detailed design stage.

The available dimensions of New Zealand made plates are given in appendix B.1 and imported plates are given in appendix B.2. Note that the maximum plate length is typically 12m and there is a limit on the weight of an individual plate (i.e. on the combination of length, thickness and width) which may further limit the length for very thick plate.

For bracing members, hot rolled section angles, channels and I sections may be used, especially angles for K- or X- braces. If weathering steel is to be used, there are no available hot rolled sections; however, fabricators can fabricate similar sections from plate; seek advice from a fabricator about what sections are economically feasible. Weathering steel related guidance is found in El Sarraf and Clifton (2005).

Selection of the particular sub-grade (for toughness) does not usually need to be considered in the initial design unless availability is likely to be a problem, such as non-standard thickness or dimensions (see appendix B for details of standard plate thicknesses and dimensions). Guidance on availability and delivery times for specific products and sizes should be sought from fabricators and/or steel suppliers as early as possible to optimise the final bridge girder design. If the construction period is limited, delivery times may affect the choice of components.

4.5.2 Concrete

The choice of concrete grade depends on its function. For above ground substructure components such as abutment beams, pier head beams and piers, 40MPa concrete is common. For slab decking, 50MPa is usually specified for precast decking and when durability aspects govern (ie the bridge is located in a marine environment), while 40MPa concrete is specified for in situ decking and topping.

All concrete components, whether in situ or precast, are manufactured in accordance with *NZS 3104 Specification for concrete production* (SNZ 2003), constructed to *NZS 3109 Concrete construction* (SNZ 1997) and designed to *NZS 3101* (SNZ 2006).

4.6 Preliminary sizing – multi-girder bridges

4.6.1 Girder spacing

The total transverse moments in the slab of a multi-girder deck are not particularly sensitive to girder spacing in the range of 2.5m to 3.8m (the increase in local moment with span is more or less balanced by a reduction in the moment arising from the transfer of load from one girder to the next). It is advantageous to choose the widest girder spacing as possible, consistent with other geometrical considerations and with the appropriate decking option, such as partial depth precast decking outlined in section 2.3. It is preferable to use an even number of girders so that they may be paired during construction, but an odd number can be used, if due provision is made for the erection of single girders.

In selecting a suitable girder spacing, attention must be paid to the cantilevers at the edges of the deck. The cantilever length from the outer girder centreline should normally be restricted to about 1.5m to 2m, including the edge beam.

When designing for accidental traffic loading applied onto the barrier, the length of cantilevers may need to be restricted, and the slab increased in thickness locally.

4.6.2 Girder profile

For simple spans over about 25m, a construction depth (top of slab to underside of beam) of between about 1/18 and 1/30 of the span can be achieved with fabricated beams; although the most economical solution will be toward the deeper end of this range. Note that the cost of applying a protective coating system must be considered as the cost may increase with the increased surface area of a deeper beam in comparison with a shorter beam. For shorter spans, the depth is likely to be proportionately greater, particularly for spans under 20m. This may limit the use of a welded beam due to the available depth in comparison to a fabricated beam.

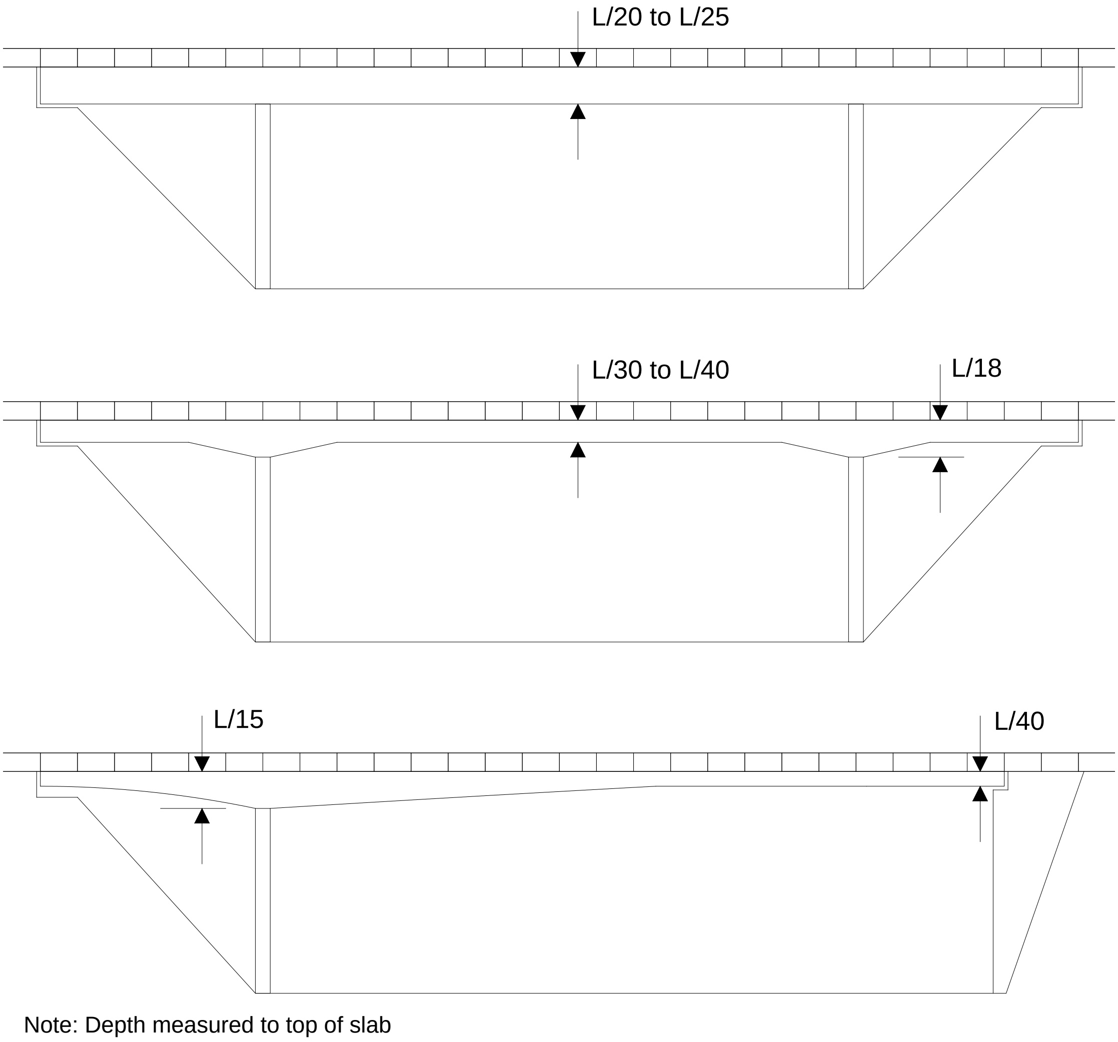
For composite continuous spans with parallel flanges, the construction depth (again, from top of slab to underside of beam) is typically between 1/20 and 1/25 of the major span. The use of curved or tapered haunches can reduce construction depth at mid-span, at the expense of increased depth at the internal supports. A selection of typical arrangements is given in figure 4.6.

4.6.3 Flange and web sizes

Experience and a few rules of thumb can often be used for an initial selection of sizes. As a result of the weight of the steelwork contributing little to overall design moments, the selection can be quickly refined. One such set of simple rules is given in appendix C.

Spans that must be fabricated with more than a single length to each main girder give the opportunity to vary the girder make-up in the different pieces required for each span. Maximum length of the pieces is influenced by transportation (loads over 20m long require special arrangements) and by the length of plate that is available.

Figure 4.6 Typical span/depth proportions for continuous spans in a multi-girder bridge



4.6.4 Slab thickness

For initial design, choose a slab thickness of 250mm. This can be refined in detailed design. If the cantilever barrier is designed for an accidental traffic load, use an initial thickness of 330mm (for a 2m cantilever) at the root of the cantilever, reducing to 250mm at the first internal main girder.

4.7 Preliminary sizing – ladder decks

4.7.1 Girder spacing

The main girder spacing should be such that cantilevers are about 1.2m to 2m long. Main girders are typically spaced between 5.5m and 18m apart to suit the road width. For a wider deck, use either two separate ladder decks or two sets of ladder deck steelwork with a common slab. If very high containment barriers have to be provided at the edge of the deck, choose a shorter cantilever or thicken the slab.

Choose a cross girder spacing between 3.3m and 4m. Spacing up to 4m can be achieved using partial depth precast decking as permanent formwork. Note that if the bridge is curved in plan, the cross girder spacing will be greater on the outside of the curve and formwork lengths have to vary across the width of the deck.

4.7.2 Girder profile

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

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

4.7.3 Flange and web sizes

Choose initial flange sizes on the basis of previous experience or using simple line beam models (the proportion of load carried by each girder is easily determined by a ‘statics’ distribution transversely).

For long spans, the flanges may be quite thick, up to 80mm thick or even 100mm thick. The use of a higher strength grade (higher than G350) may be appropriate in some circumstances but higher strength grades are, at present, significantly more expensive and less readily available; delivery time is also likely to be longer. Grade G350 has been almost exclusively used for ladder deck bridges in New Zealand.

For cross girders, hot rolled sections or welded beams may be used. Choose a plate girder section on the basis of the cross girders acting as simply supported beams.

4.7.4 Slab thickness

Choose a slab thickness of 250mm for a cross girder spacing up to about 4.0m, with due regard to cover required for durability (and to the grade of the concrete, which is normally 40MPa). Where a very high containment level barrier is to be carried, there will be large moments and lateral forces (outward forces, causing tension in the slab) to be sustained, with consequences on slab thickness. To achieve a 2m cantilever while carrying such a barrier, a slab thickness of up to about 330mm may be needed. This thickness will need to be tapered back to the regular slab thickness inboard of the outer girder over a length of 1m to 2m. Consideration must also be given to the means of transfer of the transverse moment into the ends of the cross girders.

5 Design standards

The responsibility for the maintenance of state highway roads and bridges in New Zealand rests with the NZTA, while territorial authorities (such as city and district councils) are responsible for the maintenance of all non-state highway roads and bridges.

The *Bridge manual* defines design loadings, load combinations and load factors, together with criteria for earthquake resistant design. It does not, however, define detailed design criteria for the various materials, but refers to standards such as those produced by Standards New Zealand, Standards Australia and the British Standards Institution. The standards to be used should be the editions referenced in the *Bridge manual*, including all current amendments. For cases when specific portions of these standards are referred to in the *Bridge manual*, any reference in these standards to specific loads or load combinations that might conflict with the provisions in the *Bridge manual* should be disregarded.

For the purpose of assessing probabilistic effects of loading such as wind, earthquake, flood and live load fatigue, and for consideration of long-term effects such as corrosion, creep and shrinkage, the design working life of a bridge or an earth retaining structure is taken in the *Bridge manual* to be 100 years. This may be varied by the controlling authority if circumstances require it. It should be noted that the 100-year design working life exceeds the minimum requirement of the New Zealand Building Code.

This chapter reviews briefly the range of documents that are used in the design and construction of bridges in New Zealand. These documents form the basis of the design guidance given in this publication.

5.1 The *Bridge manual*

The *Bridge manual* is the main bridge design document that outlines the criteria for the design of new bridges and evaluation of existing bridges in New Zealand. It comprises the following principal sections:

- Section 1: Technical approval and certification procedures
- Section 2: Design – general requirements
- Section 3: Design loading
- Section 4: Analysis and design criteria
- Section 5: Earthquake resistant design of structures
- Section 6: Site stability, foundations, earthworks and retaining walls
- Section 7: Evaluation of bridges and culverts
- Section 8: Structural strengthening
- Appendices A to G

All vehicle traffic loading and design loading requirements are based on section 3 of the *Bridge manual*. Earthquake loadings are given in section 5 of the *Bridge manual*, which incorporates parts of the guidance given in part 5 of *NZS 1170.5* (SNZ 2004e). Future amendments and revisions to the *Bridge manual* are planned and the designer should comply with these updates.

For steel and composite construction, section 4.3 of the *Bridge manual* currently refers to *NZS 3404 Steel structures standard* (SNZ 1997b) and *AS 5100.6* (SA 2004). Box girders are outside the scope of this document.

For reinforced concrete and prestressed concrete, section 4.2 of the *Bridge manual* refers to *NZS 3101 Concrete structures standard* (SNZ 2006).

5.2 AS 5100

The Australian *AS 5100 Bridge design standard* (SA 2004) is composed of seven parts and provides a comprehensive design standard specific for bridges, unlike the New Zealand *NZS 3404 Steel structures standard* (SNZ 1997b), which provides guidance for all steel structures. *AS 5100,* part 6 (SA 2004a), is the main design standard that will be used in this guide for the design of steel-concrete composite bridges. The seven parts are:

- Part 1: Scope and general principles
- Part 2: Design loads
- Part 3: Foundation and soil-supporting structures
- Part 4: Bearings and deck joints
- Part 5: Concrete
- Part 6: Steel and composite construction
- Part 7: Rating of existing bridges.

It should be noted that *AS 5100* (2004) references *BS 5400* (all parts), in addition to a number of Australian and international documents. For the reasons stated below and for the sake of consistency, most of the design guidance on composite construction given in this guide will be based on the relevant parts of *AS 5100* (SA 2004) and its 2010 amendment no. 1.

5.3 NZS 3404

There are currently two versions of the New Zealand *Steel structures standard* in print: *NZS 3404.1* and *2* (SNZ 1997) which includes two amendments dated June 2001 and October 2007, and *NZS 3404.1* (SNZ 2009). The latter specifically provides updated guidance on materials, fabrication and construction, which includes corrosion protection and inspection of welding and bolting, while the former also includes general design guidance of steel structures.

Section 4.3 of the *Bridge manual* specifies that *NZS 3404* (SNZ 1997b) should be used for the design of steel componentry of bridge substructures together with any seismic load resisting componentry that is expected to behave inelastically. Steel componentry of bridge superstructures, including seismic load resisting components expected to behave elastically, should be designed in accordance with *AS 5100.6* (2004).

NZS 3404.1 (SNZ 2009) is the first part of seven, which in time will supersede the 1997 version. Designers must be aware of the two versions and the superseding parts that will be developed and published in the coming years. The 2009 version is mainly referenced in this guide for providing guidance on the fabrication of steel in bridges (section 8.1 of the standard).

5.4 NZS 3101

NZS 3101 (SNZ 2006) provides guidance on the design and use of concrete, which includes bridge-related guidance especially for concrete decking, concrete bridge fatigue design, as well as precast concrete components in the bridge superstructure. Important durability design guidance is also given.

5.5 BS 5400

The British standard *BS 5400 Steel, concrete and composite bridges* consists of 10 parts, which were published and revised from 1978 through to 2010. In March 2010, *BS 5400* was formally superseded by the Eurocodes and withdrawn. Although the *Bridge manual* no longer references *BS 5400*, this standard still remains useful as a reference document.

5.6 Eurocodes

The Eurocodes are a set of structural design standards that have been developed by CEN, the European standards body, and adopted throughout Europe. The Eurocodes are published in a total of 58 separate parts, each dealing with either general, material-specific or structure type-specific matters. Each part contains principles and application rules that are common in all the adopting countries, apart from certain aspects that have been left for national choice, as determined by the individual national standards body. The Eurocodes are published unchanged by each national standards body, together with a national annex that implements the Eurocode in the country and gives the national choices for that country.

The Eurocode parts that are relevant to the determination of resistance and serviceability of steel-concrete composite bridges are:

- Eurocode 3 part 1-9 *EN 1993-1-9* (CEN 2005a)
- Eurocode 3 part 2 *EN 1993-2* (CEN 2006)
- Eurocode 4 part 2 *EN 1994-2* (CEN 2005b)

The use of rules from the appropriate Eurocode will be considered in this document where guidance has proved to be insufficient in *AS 5100* and the other stated standards. This is especially important to the fatigue design of the composite bridge structure. Reference is made, for convenience, to the UK publications, for example to *BS EN 1993-2*, and the National Annex to *BS EN 1993-2*. This is especially useful as it is easier to obtain a copy of the UK version of the Eurocode from the British Standards Institution (BSI), in addition to the fact that they are printed in English.

In addition, BSI has issued several ‘published documents’ such as *PD 6695-2* (BSI 2008) that provide non-contradictory complementary information for use with the Eurocodes. Where relevant, such documents are referenced in this guide.

5.7 Basis of design

The design philosophy to be adopted for bridges is set out in clause 2.1.1 of the *Bridge manual*: ‘Highway structures shall be designed to satisfy the requirements of both the ultimate and serviceability limit states when acted upon by any of the combinations of loading defined in this document’. Clause 2.1.2 defines the requirements at the two limit states:

5.7.1.1 Ultimate limit state (ULS)

'The state beyond which the strength or ductility capacity of the structure is exceeded, or when it cannot maintain equilibrium and becomes unstable'.

In AS 5100.6, clause 3.2 expresses the strength requirement as:

$$S^* \leq \phi R_u$$

In which (ϕR_u) is the design capacity of the structure and its component members and (S^*); is the design action effect (ie the internal forces and moments due to the design loading).

5.7.1.2 Serviceability limit state (SLS)

'The state beyond which a structure becomes unfit for its intended use through deformation, vibratory response, degradation or other operational inadequacy.'

In *AS 5100.6* (SA 2004) clause 3.3 sets out more detailed SLS requirements. For actual limits for deflection etc reference must be made to *NZS 3101* (NZS 2006).

5.8 Project-specific requirements

For state highway bridges, the NZTA is responsible for defining the design parameters and endorsing the final design for construction under the NZTA technical approval process. Details of this process are given in section 1 of the *Bridge manual*.

Often owners of private bridges, not managed by either the NZTA or the territorial authority, choose to comply with the provisions given in the *Bridge manual*, since this is the main bridge design document in New Zealand.

5.9 Health and safety regulations

In addition to the technical requirements of the documents discussed above, health and safety considerations, such as those set out in the Health and Safety in Employment Act 1992 (HSE Act) and in-house health and safety policy, lead to the requirements for an assessment of the risks at all stages of the construction. The aim should be to eliminate, or reduce to an acceptable level, the identifiable risks. Designers are therefore required to anticipate how a structure will be built and to assess the risks involved in the construction. Arrangements must be made for all pertinent information to be passed on to others who must work on the structure including the owners.

It is recommended that health and safety consideration and risk assessment during the conception and design of the structure, during its use, repair and its final removal/demolition should also be considered. Guidance on these matters can be found in the HSE Act, *AS 3828* (SA 1998), or the UK CDM Regulations (Health and Safety Executive 2007).

6 Calculation of action effects

6.1 Structural analysis

For steel and composite construction, section 4 of *AS 5100.6* (SA 2004) states that one of the following two methods shall be used to determine the design action effects in a structure:

- elastic analysis
- rigorous structural analysis.

Section 4 also gives additional requirements for buckling analysis, analysis of composite beams, staged construction, connections and determination of longitudinal shear.

6.1.1 Elastic analysis

Clause 4.2 of *AS 5100.6* (SA 2004) describes the requirements for elastic analysis, where individual members are assumed to remain elastic under the action of design loads at both the ultimate and serviceability limit states. Two types of analysis are referred to in clause 4.3: first-order analysis and second-order analysis.

First-order elastic analysis does not take into account any changes in the geometry of the structure or changes to the effective stiffness of members due to compressive forces. Where the change in geometry is sufficient to modify the internal forces and moments, the changes are referred to as second-order effects.

Second-order effects due to compressive axial strains in individual members that lead to joint displacements are commonly referred to as $P\text{-}\Delta$ effects; effects due to bending of individual members are referred to as $P\text{-}\delta$ effects. It should be noted that tensile strains can also cause second order effects and lead to a reduction in first-order moments and shears but this is typically neglected in design, except for triangulated structures such as trusses. Second-order effects are especially important for seismic design; this is briefly discussed in section 6.5.

First-order elastic analysis is adequate for most composite bridge structures.

Where it is necessary to consider second-order effects, clause 4.2.1.2 of *AS 5100.6* states they can be determined either by a simple amplification of the first-order effects or by a second-order elastic analysis. For the first option, first-order elastic analysis gives the values of the maximum calculated design bending moments (M_m*) in the members. These are amplified by the moment amplification factor (δ_b or δ_s), that is determined in accordance with clauses 4.2.2.2 and 4.2.2.3 (and based on member buckling analysis, see section 6.1.3), to give the design bending moments (M*). For the second option, clause 4.2.1.2 refers to a second-order elastic analysis in accordance with appendix C of *AS 5100.6*. A full second-order elastic analysis would determine (M*) directly and avoid reference to clause 4.3.

6.1.2 Rigorous structural analysis

Clause 4.10 of *AS 5100.6* gives general requirements for a rigorous structural analysis (as an alternative to an elastic analysis) but gives very little detailed guidance. Rigorous structural analysis takes account of geometric deformations and non-linear behaviour of materials but would not be used for conventional bridge design.

6.1.3 Member buckling analysis

Clause 4.3 of *AS 5100.6* provides rules for determining the elastic buckling loads of members. This is used to determine the moment amplification factor (δ_m) in clause 4.2.2.3 of *AS 5100.6*. For composite bridges, member buckling analysis is used to determine the effective length of compression members in bracing systems, which is required by clause 10.3.2.

6.1.4 Analysis of composite beams, girders and columns

Clause 4.4 of *AS 5100.6* gives additional requirements for the analysis of composite beams, girders and columns. It provides rules for determining the effective section properties, such as the concrete-related factors and information needed, when analysing a steel-concrete composite member. It includes taking account of shear lag in the concrete slab and the difference in modulus of elasticity of concrete and steel. This is then used with the provisions of *NZS 3101* (SNZ 2006) when designing the concrete slab of the bridge structure, see sections 7.1 and 7.10 for further details.

The methodology to determine the longitudinal moments and associated shear forces and reactions in a continuous composite girder is given in clause 4.4.3; this includes guidance on taking account of concrete that is cracked in tension. Clause 4.4.4 gives requirements for calculating deflections in a composite member: for this purpose, concrete in tension may be assumed to be uncracked.

6.1.5 Staged construction

Clause 4.6 of *AS 5100.6* gives the additional requirements for the analysis of staged construction of the steel-concrete composite bridge. It states that adequacy checks are required for each stage of construction, such as when the steel section initially carries its own self-weight and the weight of the concrete deck but then acts compositely with the concrete slab for subsequently applied loadings. The result is different bending stiffness and load distribution at each stage of construction.

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

Clause 4.6.2 distinguishes between compact and non-compact sections in the treatment at ULS of the effects due to the various construction stages. For compact sections, the total effects at each section (moments and shears) are simply added together for verification of design capacity. For non-compact sections the stresses at each section for each construction stage are added together to give total stresses, for verification against design capacity. Clause 4.6.3 requires that, at SLS, irrespective of the section classification, stresses are determined for each stage and added together (in practice this only needs to be applied for compact sections, to ensure that limiting stresses are not exceeded – for non-compact sections, ULS verification is always more onerous).

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

6.1.6 Connections

Clause 4.7 of *AS 5100.6* gives the additional requirements for the analysis and design of connections whether they are bolted or welded. Each element of the connection shall be designed so that the structure is capable of resisting all the applied design actions. The design capacities of each element shall not be less than the calculated design actions effects. The connection design parameters are given in clause 4.7.1 and subsequent clauses.

6.1.7 Longitudinal shear

Clause 4.8 of *AS 5100.6* gives the requirements for calculating the design longitudinal shear force per unit length on a shear plane. This will be used to determine the number of shear connectors needed. Guidance on calculating the transformed concrete area can be found in Beer et al (2008).

6.1.8 Shrinkage and differential temperature effects

Clause 4.9 of *AS 5100.6* states how the effects of shrinkage modified by creep and differential temperature are to be evaluated. Requirements for evaluating the consequent effects on the longitudinal shear forces under both ULS and SLS are also given.

6.2 Computer modelling

Computer modelling is now commonly used for most bridge global analysis, even for simply supported single span bridges. This is especially useful for multi-girder bridges, where the distribution of action effects between the girders would be difficult to determine accurately by hand. Powerful analytical software is becoming available that not only assists designers in determining the action effects on the structure but also in determining the critical buckling moments of members and elastic buckling values of structures.

It should be noted that no matter how advanced computer modelling programmes become, it is always recommended that the novice bridge designer (or even experienced designers) verify the results given by the programme to ensure that the correct data is calculated. This can be as simple as hand calculation of the overall moment and shear actions acting on the whole bridge structure. The purpose of this exercise is to build up the designer's experience and to acquire a 'feel' of what the correct results should be. This provides a knowledge that would identify errors in the computer model outputs and in working around some of the limitations that the program may impose.

In the end, it is up to the designer to determine the computer modelling program that is appropriate when analysing and designing a bridge structure. A brief introduction of the different available computer modelling options are summarised below.

Note that, if second-order analyses are being used to determine second-order effects directly, then the computer software must be capable of modelling both $P\text{-}\Delta$ and $P\text{-}\delta$ effects. Additional guidance is available in appendix E of *NZS 3404* (1997).

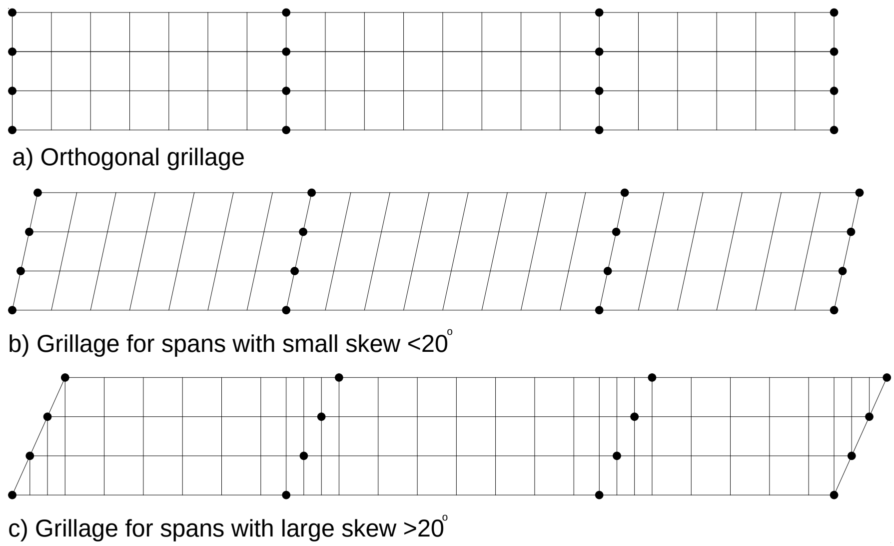
6.2.1 2D grillage models

6.2.1.1 Multi-girder decks

For multi-girder bridge decks, a simple 2D grillage will give adequate results for non-integral bridges (see (Iles 2010) for guidance on modelling integral bridges and (Hambly 1990) for additional guidance). In such models, the structure is idealised as a number of longitudinal and transverse beam elements in a single plane, rigidly interconnected at nodes. The transverse beam elements may be orthogonal or skewed with respect to the longitudinal beams. Each beam element represents either a composite section (eg main girder with associated slab) or a width of slab (eg a transverse element may represent a width of slab equal to the spacing of the transverse elements). Where the supports are square to the main beams, an orthogonal grillage is used. Where the supports have a small skew, the grillage may be skewed (the lines of reinforcement will also probably be skew in these cases). Where the skew exceeds about 20 degrees, a skew grillage has difficulty in modelling the slab behaviour; an orthogonal grillage with skewed ends is used instead (but then a local model may also be needed in the obtuse corners because the grillage model cannot separate the torsional effects carried by the slab from those carried by the beam in warping, which is a particular concern in those regions). Examples of all three configurations are shown in figure 6.1.

A line of elements should be provided along each main girder; intermediate lines (representing slab only) are used to refine the mesh so that the effects of wheel loads can be evaluated (rather than developing a separate model or referring to standard plate influence surfaces to determine local effects). An edge beam is usually provided to facilitate modelling of the cantilevers. Because the transverse beam elements do not represent discrete structural elements, the spacing can be chosen by the designer to facilitate the analysis. Generally, the spacing should not exceed about 1/8 of the span for modelling global effects. Uniform node spacing should be chosen in each direction where possible, though it may be helpful to locate nodes at splice or bracing positions (so that values of moments and shear forces at these positions are available in the output). For multi-girder bridges, shear lag is unlikely to reduce the effective width of the slab below its actual width (see the reference to allowance for shear lag in sections 6.1 and 7.1.1). Models for the bare steel condition (this may be a line-beam model), for the partially composite conditions, for the long-term condition and for the short-term condition are required.

Figure 6.1 2D grillage models for a three-span multi-girder bridge



Section properties for the composite main beams in the sagging moment region should use the full composite second moment of area; if there are intermediate longitudinal lines, the elements should be given only the properties of the slab itself. For continuous composite main beams in the hogging moment region, clause 4.4.3 of *AS 5100.6* specifies that the stiffening effects of the concrete over 15% of the length of the span on each side of the internal support may be neglected. In this case, the properties of the steel section and the longitudinal reinforcement should be used. Section properties for transverse beam elements representing the slab alone should use a width equal to the element spacing.

In regards to the torsional stiffness of the slab, it can be conservatively taken as zero in a 2D grillage when modelling the slabs, thereby designing them for bending effects only. Even though in reality, the torsional stiffness of the slab will assist in distributing the loads between the transverse and longitudinal beams. Intermediate bracing (between beam pairs) should be modelled, because it does affect the local transverse bending stiffness (although it does not significantly affect distribution of load between main beams). A shear-flexible member should be introduced (derived from a local plane frame model of the bracing); this will give rise to local forces that can be used to verify the adequacy of the web stiffener to flange connection. (The inclusion of bracing in the model gives a better distribution of effects than the alternative of applying deformations from an unbraced model to a local plane frame.)

6.2.1.2 Ladder decks

For ladder deck bridges, 2D grillage models can be used, although they are not fully able to model the local effects of the deflections of the cross girders and their interaction with the lateral bending of the main girder bottom flanges. For cross girders, the appropriate width of slab acting with the cross girder is the spacing of the cross girders (ie one half of the distance to the next girder on either side) as specified in clause 4.4.1 of *AS 5100.6*. However, further guidance related to ladder deck cross girders is also found in clause 5.4.1.2 of *EN 1994-2* (CEN 2005) which states that the slab width should also not be more than one quarter of the spacing between main girder webs plus the spacing to the outer studs on the cross girder.

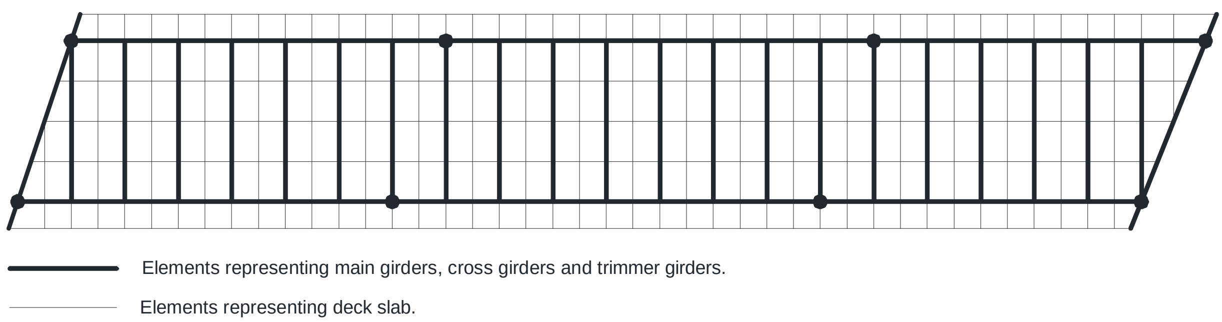
Joints in the grillage model should be rigid connections. This applies not only to the joints between elements along a beam (which must be rigid) but also to the joints between cross girders and main girders (for both the bare steel/wet concrete condition and the composite condition), assuming that either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

The same guidance on torsional stiffness is taken as zero in ladder deck bridges and the slab is designed for bending effects only. In reality the slab and the lateral bending stiffness of the bottom flanges of the main girders provide additional restraint against the twisting of the main girder (in effect this is warping restraint) but this is too complicated to model in a simple 2D grillage.

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

Figure 6.2 shows a typical 2D grillage model suitable for the global analysis of a three-span ladder deck bridge. The slab mesh should be at about 3m spacing transversely and half the cross girder spacing. Cantilever slabs and edge beams should also be modelled. (This figure shows coincident cross girders and trimmer girders at the obtuse corners but this may not be practical, as mentioned in section 2.2.3; careful attention should be given to modelling in these corners.)

Figure 6.2 2D grillage model for a skew three-span ladder deck



6.2.2 3D grillage models

For ladder deck bridges, a 3D grillage skeleton model, essentially the same plane model as a 2D model but with vertical elements at every cross girder connection, connected at the bottom to beam elements representing the bottom flanges, is better able to represent the interaction between cross girders and main girders. The vertical elements have negligible bending stiffness in the plane of the web but have the stiffness of the effective stiffener section out of that plane. Vertical bending stiffness of the main girders is assigned wholly to the upper members and the 'bottom flange' elements represent only the plan bending of those flanges. The differential loadings on adjacent cross girders generate different deformations in the adjacent U-frames and thus plan bending of the bottom flange; additional guidance is available in appendix A of *AS 5100.6*.

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

Finally, designers must be aware of some of the limitations that may be experienced when undergoing a grillage model, whether it is 2D or 3D. Some of these limitations are:

- Eccentricity among the structural elements of a bridge cannot be taken into account in the model. Inevitability, additional internal forces and possible load distributions are ignored.
- It is difficult to take into account torsion and distortional warping effects.
- It is difficult to investigate the buckling phenomena of the steel girders during erection stages and the deck concreting.
- Diaphragms, bracing systems and stiffeners, which are usually installed in order to improve global and local stability and increase torsional rigidity, cannot be adequately represented. These limitations may be addressed by the use of other available software, which may be more complex.

6.2.3 Finite element models

A full linear 3D finite element model can give a more realistic determination of the structural response, particularly for ladder decks. Finite element programs are currently the most powerful and versatile analytical tool available. The main advantage finite element modelling provides is that it can be applied to any structure regardless to the level of complexity. For bridges, models can be built using plate elements and beam elements. Plate elements are used for the deck slab (using the Wood Armer Method to analyse the slab) and for the webs of the main girders (and for the webs of cross girders in ladder decks). Beam elements can be used for the flange plates (aligned with their bending stiffness in the plane of the flange and, for the top flange, having an offset from the slab elements), for web stiffeners (representing the effective Tee section) and for triangulated bracing.

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

Verification of buckling resistance of members sometimes requires an elastic buckling analysis of the structure to determine its critical loading. Software is available that can determine elastic buckling load using finite element models, and these models can be used either to determine elastic critical moments of beams directly or with the general method of verification (see appendix A of *AS 5100.6* for details).

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

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

Finite element modelling does have its limitations. These are:

- Depending on the level of complexity of the model, the quantity of computation and time required to analyse the model may be long. This may result in a delay in the availability of the model output, leading to later confirmation of the design details.
- Modelling of concrete especially when cracked is difficult.
- Greater experience is needed to construct the model and interpret the results.

6.2.4 Alternative analysis options

In addition to the different form of grillage analysis and finite element modelling outlined above, there are other computer software packages that provide similar, possibly less detailed, results. The main difference can be the different modelling 'engine' that the program is based on, its available function and output capabilities. Depending on the level of sophistication that is required or afforded by the engineer, the following options can be considered.

6.2.4.1 Simple design tools

These can either be in-house Excel spread sheets or a specially developed program that performs repetitive or specific functions. Most, if not all, design practices have at least one form of a simple design tool that are especially useful in the preliminary design stage to assist in evaluating the different bridge concepts. Once the final concept has been chosen, then the more detailed grillage analysis or finite element modelling is used.

6.2.4.2 Computer-aided engineering tools

Computer-aided engineering tools (often referred to as CAE) are more sophisticated than simple design tools that provide a wide range of functions to bridge designers. The functions may include analysis, simulation, design and detailing - the CAE can import and export to other programs by the use of building information modelling (BIM). They require additional more detailed inputs than simple design tools, which in turn results in comprehensive outputs that may include detailed design calculations to a specified standard to checking creep and cracking in concrete slabs and even producing construction drawings.

These programs mostly use a finite element analysis engine. However, the main difference from finite element modelling outlined above is that each designed component is taken as a whole or as a very rough mesh (for slabs) in comparison to the more detailed and finer finite element modelling. This results in a faster calculation of the results and is easier to set up the model and analyse, it also does not require a highly specified computer or a specially trained engineer to use the program.

6.3 Deck slab analysis

The *Bridge manual* refers to *NZS 3101* (SNZ 2006) when analysing the concrete deck slab. Two methods exist: the empirical method, based on assumed membrane action; and elastic plate bending analysis method.

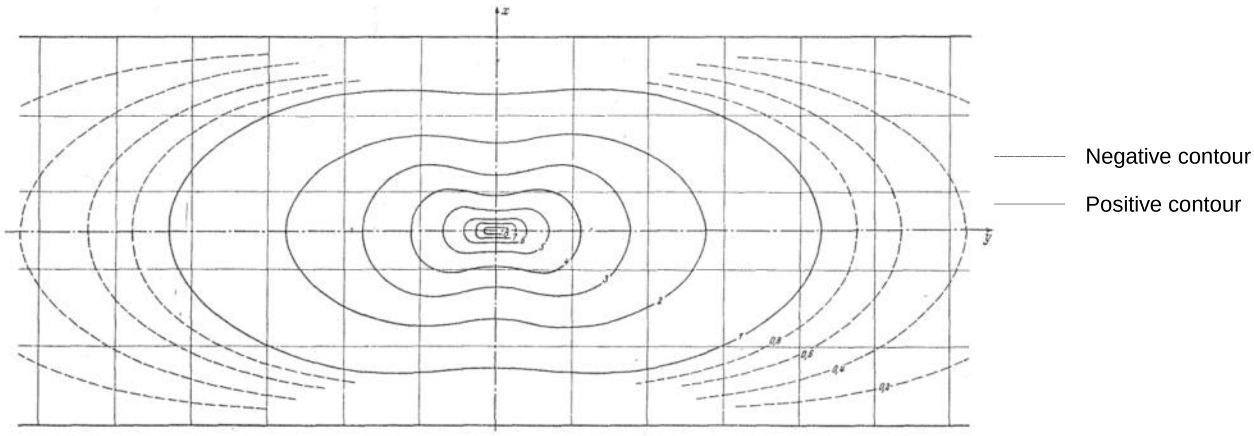
The empirical design method takes account of membrane action in the slab, and is based on test results. Requirements for the use of the empirical method are given in clause 12.8.2.1 of *NZS 3101,* where one of the requirements specifically state that the deck must be fully cast in place. However, as demonstrated in section 3.5, partial depth precast decking provides a more economical option than cast in place decks. In this case, elastic plate bending analysis should be used.

Clause 12.8.3 of *NZS 3101* outlines the design basis when using the elastic plate bending analysis method. It should be noted that any of the following methods can be used in this case; simple analysis, grillage analysis with a fine mesh and finite element analysis. Clause 12.4 of *NZS 3101* outlines the design methodologies that can be used for analysing bridge deck slabs, and also provides guidance on some of the design options given above. Further details on the design of slabs are given in section 7.10.

The choice of any of the above analysis options is dependent on the level of complexity of the slab design. For example, a simple analysis may be sufficient for a single span, simply supported bridge, while a grillage or finite element analysis of the deck may be required for a continuous, multi span, multi-girder bridge. This is also dependant on the level of experience of the bridge designer and the computer modelling programme available. As mentioned in section 6.3, it is always recommended that the designer checks the results, for example, for deck slabs one way is by the use of Pucher Charts (Pucher 1964).

Pucher Charts are a series of contour plots of influence surfaces, as illustrated in figure 6.3. The simplification of support conditions to permit use of standard charts normally leads to a conservative assessment of worst moments. If a fine mesh of shell elements is used in a 3D global model, local bending moments in the slab will be determined directly. A resolution (node spacing) of about 500mm should be adequate in view of the loaded area under a wheel and dispersal through surfacing and slab.

Figure 6.3 Typical influence chart of slab moments using Pucher Charts



Localised dead load effects including self-weight of the slab must also be considered. However, these effects should be able to be simply calculated using manual methods. As deck slabs are usually relatively thin, these effects are normally relatively small.

6.4 Design loading and load combinations

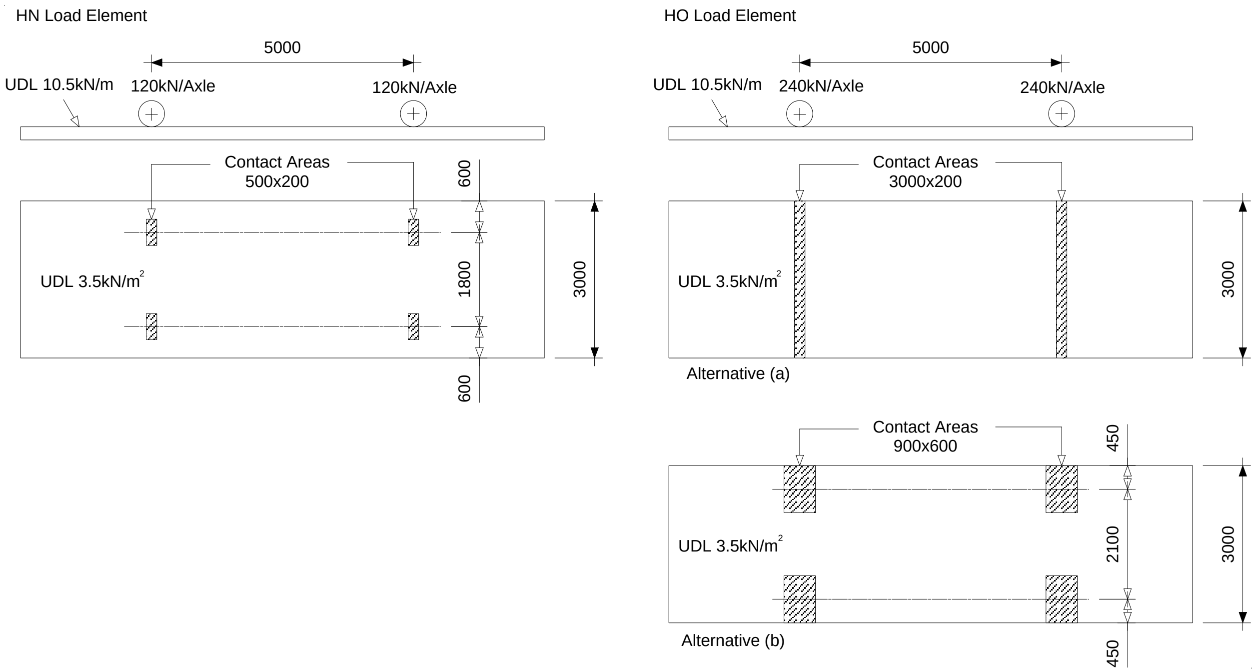
All design loading acting on the bridge structure should be in accordance with section 3 of the *Bridge manual* and in accordance with the load combinations given in section 3.5 of that document. These loadings are summarised below.

6.4.1 Traffic loads – gravity effects

Rules for the placement of these loadings, number of load lanes per bridge width and other required details while designing the bridge structure are given in section 3.2.3 of the *Bridge manual*. Section 3.2.4 provides guidance on the combination of the normal (HN) and overload (HO) loading components and the reduction factors used when there is more than one element of loading (ie more than one notional lane loaded).

For all above ground level components of the bridge, the applied HN and HO loading must be multiplied by the dynamic load factor in accordance with section 3.2.5 of the *Bridge manual*. For below ground level components of the bridge, the dynamic load factor taken is 1.0, to allow for the fact that vibration is damped out by the soil.

Figure 6.4 Diagrammatical representation of normal and overload loading



6.4.2 Fatigue loading spectrum

The design loading used in the fatigue assessment needs to represent the expected frequent loading over the design life of the structure. It is common to represent the varied spectrum of real traffic by a single notional vehicle that traverses the length of the bridge a defined number of times over the life of the bridge.

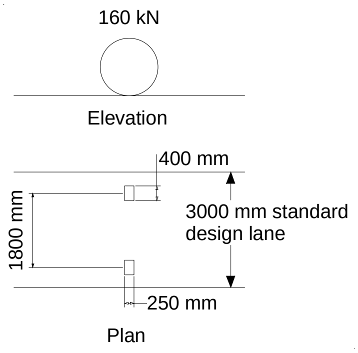
Section 3.2.6 of the *Bridge manual* currently refers to the *Recommended draft fatigue design criteria for bridges* (Clifton 2007a), which is reproduced below. The background to this is given in appendix D of this guide.

6.4.2.1 Basic loadings for fatigue design

There are two basic loads given, in clause 6.9 of *AS 5100.2* (SA 2004), being:

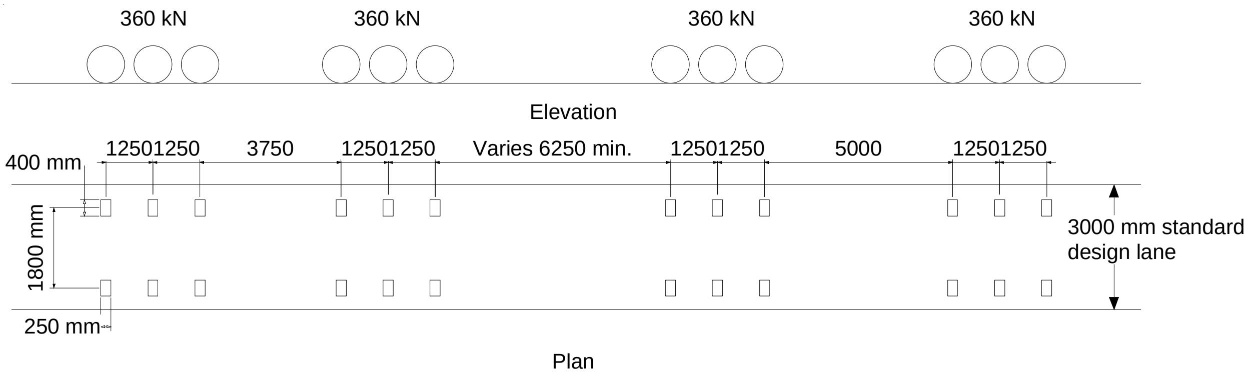
1 The modified individual A160 heavy axle load from *AS 5100.2* as shown in figure 6.5.

Figure 6.5 160 fatigue loading



2 The modified individual M1600 moving traffic load from *AS 5100.2,* without the UDL component, as shown in figure 6.6.

Figure 6.6 M1600 fatigue load



6.4.2.2 Fatigue design loading and number of fatigue stress cycles

It is recommended that the fatigue design traffic load effects be determined from 70% of the effects due to the passage of either a single A160 axle or a M1600 load, whichever gives the more severe range of effects at the location being assessed. In each case, a load factor of 1.0 is used and the load effects are increased by the dynamic load allowance (α), as given below:

- for the A160 axle load $\alpha = 0.4$
- for the M1600 axle load $\alpha = 0.3$.

Each load should be placed within the width of any traffic lane as marked on the bridge such that the fatigue effects for the component under consideration are maximised.

Unless determined otherwise by the relevant authority, the number of fatigue stress cycles to be used for the calculation of the fatigue capacity of the structural element under consideration should be as follows:

1 For the fatigue design load of 0.70 x (A160 axle load) x (1 + α):

(current number of heavy vehicles per lane per day) x 4 x 10^4 x (route factor).

2 For the fatigue design load of 0.70 x (M1600 moving traffic load without UDL) x (1 + α):

(current number of heavy vehicles per lane per day) x 2 x 10^4 ($L^{-0.5}$) x (route factor).

In the above expression:

- The current number of heavy vehicles per lane per day is provided in the bridge project documentation or is determined by project specific traffic counts. The current number of heavy vehicles is based on the year the bridge is to be put into service.

  On rural routes where there are two or more lanes in one direction, the number of heavy vehicles per lane per day shall be the total of the heavy vehicles travelling in that direction. On urban routes where there are two or more lanes in one direction, the number of heavy vehicles per lane per day shall be 65% of the total number of heavy vehicles in that direction.

- The route factor for the class of road shall, unless specified otherwise by the relevant authority, be as follows:
  - for principal state highways = 0.9
  - for major urban roads = 0.65
  - for other rural routes = 0.45
  - for urban roads other than major urban roads = 0.3
- *L* is the effective span in metres and is defined as follows:
  - for positive bending moments, L is the actual span in which the bending moment is being considered
  - for negative moment over interior supports, L is the average of the adjacent spans
  - for end shear, L is the actual span
  - for reactions, L is the sum of the adjacent spans
  - for cross-girders, L is twice the longitudinal spacing of the cross-girders.

AS 5100.2 states that the fatigue design traffic load effects and relevant stress cycles should be applied to each design lane independently. However, current New Zealand practice is to apply this load and relevant stress cycles on each marked lane independently. The rationale being that in reality the traffic load will be acting on the actual marked lane rather than on the hypothetical design lane. Therefore, the number of marked lanes given in the project design statement should be designed for; this includes any future marked lanes that will be added to the bridge structure.

The fatigue stress range shall be taken to be the maximum peak-to-peak stress from the passage of the relevant fatigue design load.

These recommendations do not apply to fatigue design of roadway expansion joints.

6.4.3 Traffic loads - horizontal effects

Section 3.3 of the *Bridge manual* gives the required forces for determining the horizontal effects of traffic loadings, namely braking and traction in section 3.3.1 and centrifugal force in section 3.3.2.

6.4.4 Loads other than traffic

Table 6.1 lists the other non-traffic loads given in section 3.4 of the *Bridge manual* that must be considered when designing a bridge.

Table 6.1 Other non-traffic load cases

Load	Section	Note
Dead	3.4.1	
Superimposed dead	3.4.2	Given as 1.5kN/m^2 for road surfacing, in addition to handrails, guardrails, lamp standards, kerbs and services.
Earthquake	3.4.3	
Shrinkage, creep and pre-stressing effects	3.4.4	Outlines the effects of shrinkage and creep of concrete, and shortening due to pre-stressing (see section 6.4.5).
Wind	3.4.5	
Temperature effects	3.4.6	See section 6.4.5 for further details.
Construction and maintenance	3.4.7	
Water pressure	3.4.8	Outlines the water pressure loading experienced by the bridge piers in waterways.
Groundwater on buried surfaces	3.4.9	
Water ponding	3.4.10	
Snow	3.4.11	Outlines the snow loading requirements that are determined from *AS/NZS 1170.3* (SNZ 2003a). Note this only applied to pedestrian bridges.
Earth	3.4.12	Outlines the provisions for determining the earth loads experienced by the bridge substructure.
Kerbs, guardrails, barriers and handrails	3.4.13	They should be designed in accordance with appendix B of the *Bridge manual*.
Footpaths and cycle tracks	3.4.14	These are designed to the loads given in section 3.2 of the *Bridge manual*.
Vibration	3.4.15	
Settlement, subsidence and ground deformation	3.4.16	The most adverse combination of differential ground movement should be considered. Advice should be sought from the geotechnical engineers.
Forces locked-in by the erection sequence	3.4.17	The guidance given in clause 4.6 of *AS 5100.6* should be followed.
Collision	3.4.18	

6.4.5 Shortening, temperature effects and ground deformation

Section 3.4.6 of the *Bridge manual* outlines two types of temperature effects. The first is the overall temperature change which is taken as ±25˚C for steel and is the value that is used for composite sections. This covers the forces and movements resulting from the variation in the mean temperature of the structure.

The second is the differential temperature change which for non-compact sections is applied at both the ULS and SLS load cases, but only for the SLS case for compact sections. This is due to the compact section being able to redistribute the forces due to the nature of the compactness of the section. This covers the allowable stresses both longitudinal and transverse of the section resulting from the temperature variation through the depth of the structure.

Similar to differential temperature, the effects of shortening and settlement, subsidence and ground deformation should be applied to non-compact sections at both the ULS and SLS load cases, and only for the SLS case for compact sections.

6.4.6 Geotechnical actions in integral bridges

Geotechnical actions on bridge superstructures are the pressures of the backfill at the abutments. These pressures arise initially from the backfilling behind the abutments but increase over time due to the ‘strain ratchetting’ effect from the cyclic expansion/contraction of the deck with temperature variation. Soil pressures due to the movements at the ends of the deck may be calculated using guidance given in section 4.8 in the *Bridge manual* and additional information may be found in *PD 6694-1* (BSI 2011).

6.4.7 Load combinations

Section 3.5 of the *Bridge manual* outlines the parameters and load combination of the load effects in section 3.4 of the *Bridge manual*, for both SLS and ULS.

6.5 Seismic effects

Section 5 of the *Bridge manual* is used to determine the horizontal force generated by the seismic action on the bridge superstructure. These forces in turn are transferred into the supports (piers and abutments), which can be designed to form plastic hinges that resist this horizontal force (as shown in figure 5.3 of the *Bridge manual*). Designers should be aware that the horizontal forces are transferred by the restraints into the supports, which are designed for; this is discussed further in section 7.6.7. Therefore, seismic actions acting on the superstructure are actually minimal as long as the load path is available to transfer these actions into the substructure.

The design of the piers and abutments is outside the scope of this guide.

7 Detailed design

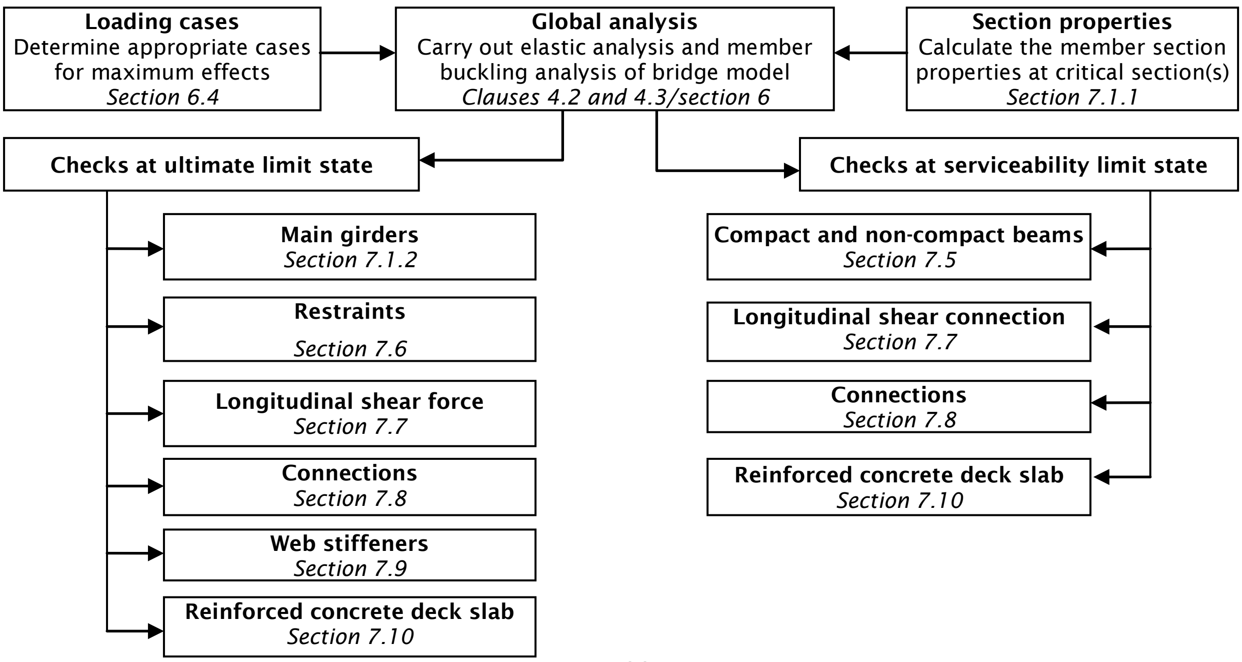
Once the final bridge configuration has been chosen and the preliminary design completed, the designer then has to refine and verify the performance of the main structural components and to detail the connections. The main design steps include:

- The resistance (capacity) of the girders and other structural elements must be verified at the ULS during construction and in service. This requires consideration of the bending resistance of the steel and composite cross sections, the shear resistance of the web panels and the shear connection between the steel and the concrete slab.
- The resistance to buckling needs to be verified, for the bare steel girders during construction and, for the composite girders, for the bottom flanges of composite girders adjacent to intermediate supports, where they are in compression.
- In integral bridges, the effect of coexisting compressive forces must be taken into account.
- The resistance of the deck slab to combined local and global effects must be verified.
- Adequacy at the fatigue limit state must be verified.
- Toughness of the steel against brittle fracture must be ensured by selecting a suitable material sub-grade.

Performance at the SLS requires to be verified; principally the checks are to ensure that inelastic behaviour does not occur under SLS actions, that crack widths in concrete are not excessive and that deformations are within acceptable limits.

The following sections outline the design procedures to conduct a detailed design of the above components, by providing a design flow chart for each component, while referencing the appropriate Clauses in *AS 5100.6* (SA 2004) and the corresponding sections in this document. All clause references are to *AS 5100.6*, unless stated otherwise. Figure 7.1 outlines the overall design procedure that should be undertaken when designing a steel-concrete composite bridge.

Figure 7.1 Overall design flow chart for the design of a steel-concrete composite bridge



7.1 Overall bridge design

7.1.1 Section properties

The section properties of the main members should be determined at key locations. Section properties are required for the bare steel members in the construction stages and for the composite members at the in-service stage. The main section properties required are:

- area of the cross section, *A*
- second moment of area about the major axis, *Ix*
- plastic section modulus, *S*
- elastic section modulus at extreme fibres, *Z*
- torsion stiffness, *J*
- out of plane stiffness, *Iyy*.

For composite members, two sets of properties are needed: short-term properties for effects due to live loads, including wind, temperature and seismic loading; long-term properties, for superimposed dead loads and shrinkage.

The short-term and long-term properties are usually calculated in 'steel units' by using the transformed area of the concrete slab – dividing the area by the modular ratio (*n*), the ratio of the elastic modulus of steel (E_s) to the modulus of elasticity of concrete (E_c). For the short term loading the normal modulus of elasticity of concrete is used, while for long term properties the effect of creep is taken into account, which is given by the creep factor (ϕ_{cc}) given in clause 5.2.1 of *NZS 3101* (SNZ 2006).

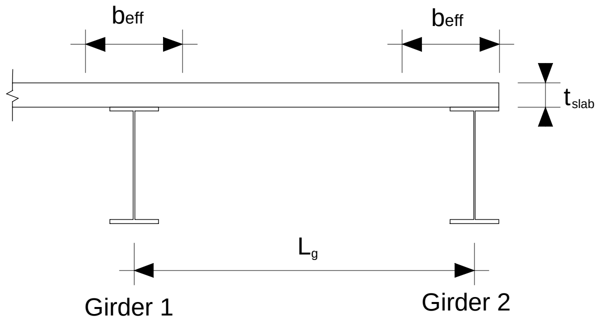
The modulus of elasticity for steel and for concrete is given in clause 2.2.5 of *AS 5100.6* (SA 2004) and clause 5.2.3 of *NZS 3101* respectively. The modular ratio (*n*) values for short and long-term loading derived for typical strength grades of concrete are given in table 7.1, for concrete with a density (ρ) of 2400kg/m^3 (normal weight concrete).

Table 7.1 Modulus of elasticity for steel and concrete

Modular ratio (*n*)	**Standard strength grades of concrete (MPa)**				
	25	32	40	50	65
Short-term loading	8.5	7.8	7.2	6.6	5.9
Long-term loading	44.3	34.3	25.1	19.8	17.8

When the steel girder is acting compositely with the concrete deck slab, the effect of shear lag in the concrete deck slab should be taken into account, in accordance with clause 6.1.7. To calculate the transformed area of concrete, the effective width of the concrete slab is required, which is given in clause 4.4.1 and is represented in figure 7.2.

Figure 7.2 Effective width of concrete



Girder 1: Composite flanges on both sides of the web:

$b_{eff} = \min(L_e/5,\ L_g,\ 12*t_{slab})$

Girder 2: Composite flanges on one side only of the web:

$b_{eff} = \min(L_e/10,\ L_g/2,\ 6*t_{slab})$

Where:

b_{eff} is the effective width of the concrete slab.

L_e is the effective span length of the girder, taken as distance between supports for simply supported beams and girders, and 0.7 times the distance between supports for continuous beams and girders.

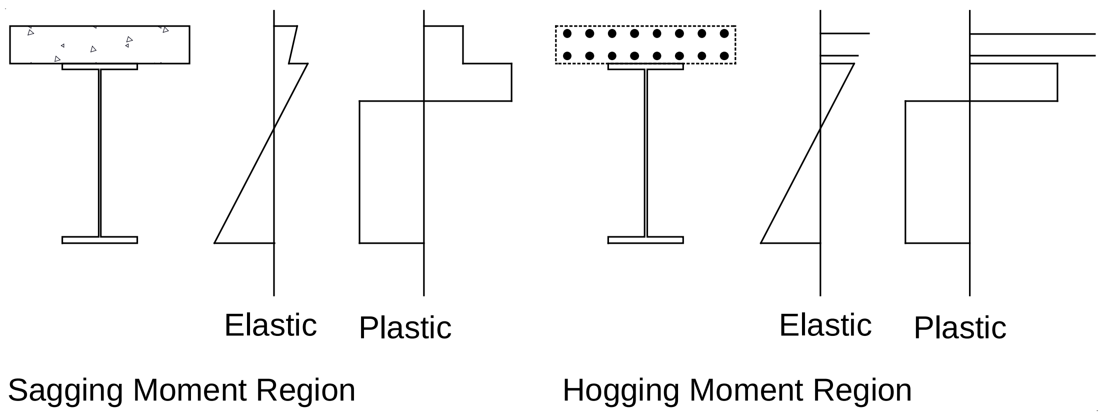
L_g is the distance centre-to-centre of girders.

t_{slab} is the the least thickness of the slab.

The same effective width is used over the whole span, for both hogging and sagging regions (except for the determination of fatigue stresses in hogging regions in continuous girders, where the effective width values are taken as half, see further comment in section 7.10). However, when there is a difference in the decking thickness in different spans, for example at intermediate supports between unequal spans, then the effective width based on that spans thickness will be used. The transformed area of concrete is only taken into account when calculating the properties for the sagging moment region. In this region the concrete is uncracked and in compression, therefore contributing to the composite action between the steel girder and the slab, as stated in clause 6.3.1. In the hogging region, the contribution from the concrete is neglected and instead the contribution from the reinforcing bars is taken into account, as stated in clause 6.3.2.

Examples of the elastic and plastic stress blocks for both the sagging and hogging regions are shown in figure 7.3 below. Calculation of plastic blocks is demonstrated in appendix E of *AS 5100.6* for sagging regions.

Figure 7.3 Stress distribution in composite beams in the sagging and hogging moment region



7.1.2 Section slenderness and the definition of compact and non-compact sections

Cross sections are classified according to the slenderness of the elements of the cross section that are in compression. Two principal classes are defined, compact and non-compact (actually 'not compact' in *AS 5100* but the hyphenated term is used in this guide, for greater clarity). Section slenderness limits for steel cross sections given in clauses 5.1.2 and table 5.1 are used to classify the section as compact or non-compact and these limits also apply to composite sections (see clause 6.1.8) with an extra limitation applied to steel flanges connected to a concrete slab. Compact sections are those where 'the full plastic moment can be developed before, and maintained after, the onset of local buckling' (clause 5.1.3). The web and compression flange possess sufficient stiffness to enable full plasticity and adequate inelastic rotation to be developed without the loss of strength due to local buckling. In a compact section, the section slenderness (λ_s) is less than or equal to the plasticity slenderness limit (λ_{sp}); ie $\lambda_s \le \lambda_{sp}$.

Clause 5.1.4 states that 'a section that is non-compact is one for which local buckling prevents the development of the full plastic moment and which is liable to local buckling before the onset of yielding'. In non-compact sections the section slenderness (λ_s) is greater than the plasticity slenderness limit (λ_{sp}), ie $\lambda_s > \lambda_{sp}$.

Attention is drawn to the sub-division of the non-compact classification, effectively creating a class called 'slender', although this term is only used explicitly in *AS 5100.6*. In a non-compact section that is not slender the section slenderness (λ_s) is greater than the plasticity slenderness limit (λ_{sp}) but less than or equal to the yield slenderness limit (λ_{sy}), ie $\lambda_{sp} < \lambda_s \le \lambda_{sy}$. In a slender section the section slenderness is greater than the yield slenderness limit, ie $\lambda_s > \lambda_{sy}$. By combining the non-compact and slender section definitions, *AS 5100* has created potential confusion. Note that, generally, it would be uneconomic to use slender flange elements in a girder, but web elements may quite often be slender.

Be aware that the term 'slender section' should not be confused with 'slender beam'.

Plastic moment capacity is often expressed in relation to the plastic section modulus (*S*), which is determined by consideration of rectangular stress blocks. For composite sections, the plastic section modulus is usually expressed in 'steel units' and the stress block for the concrete slab depends on the strength of the concrete, relative to the yield stress of the steel.

Elastic moment capacity depends on the stress at the extreme fibre (the furthest from the neutral axis, in either tension or compression, expressed as the elastic section modulus *(Z_e)* but for summation of stresses in composite sections (see discussion in section 7.3.2) the section moduli will be needed at several locations in the cross section. When a cross section is slender, part of the cross section is deemed ineffective and this reduces the value of the second moment of area and thus the elastic section moduli are also reduced.

It should also be noted that although *AS 5100.6* allows plastic theory for the determination of section capacity when the section is compact (and this is particularly applicable for composite girders, see clause 6.3.3 and appendix E of *AS 5100.6*) elastic analysis is still required for determining action affects (clause 4.1), apart from the rare situations where a rigorous analysis of the whole structure is carried out.

7.2 Main girders at ULS

The main longitudinal girders must be designed to provide adequate strength in bending and shear, to resist the combined effects of global bending, the local effects such as compression over bearings, and structural participation with the bracing systems. Adequacy must be demonstrated during construction (at the bare steel stage and the partly composite stage, when the deck is concreted in stages) and in service (when the girder is composite). Generally, the design procedure for both situations is similar but, for clarity, the sequence is shown separately below for the non-composite bare steel girders and for composite girders, in figures 7.4 and 7.5 respectively.

Cross girders in ladder deck bridges follow the same sequences; bracing members not acting compositely with the deck follow the sequence in figure 7.4.

Figure 7.4 Non-composite bare steel girder design flow chart

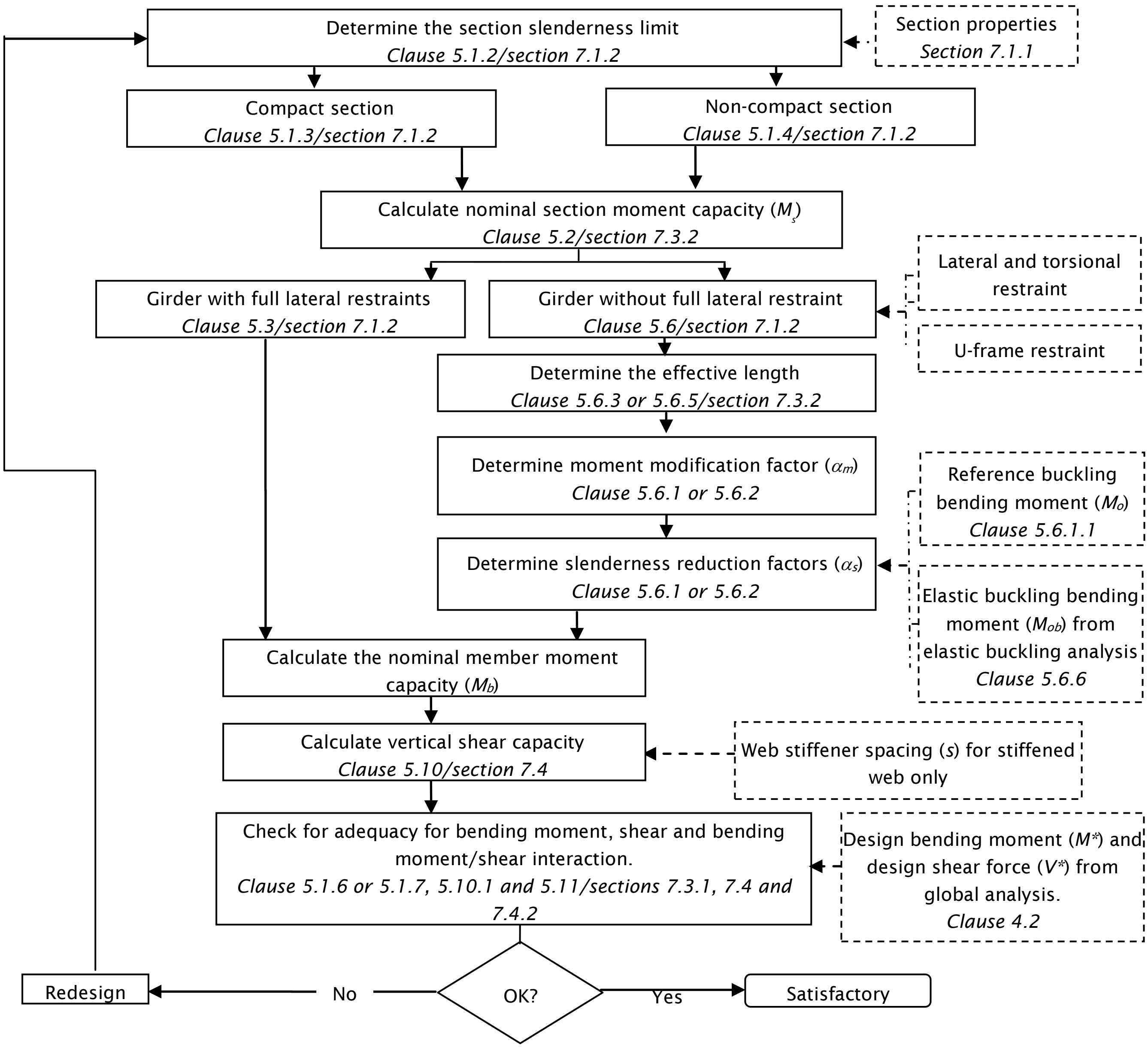
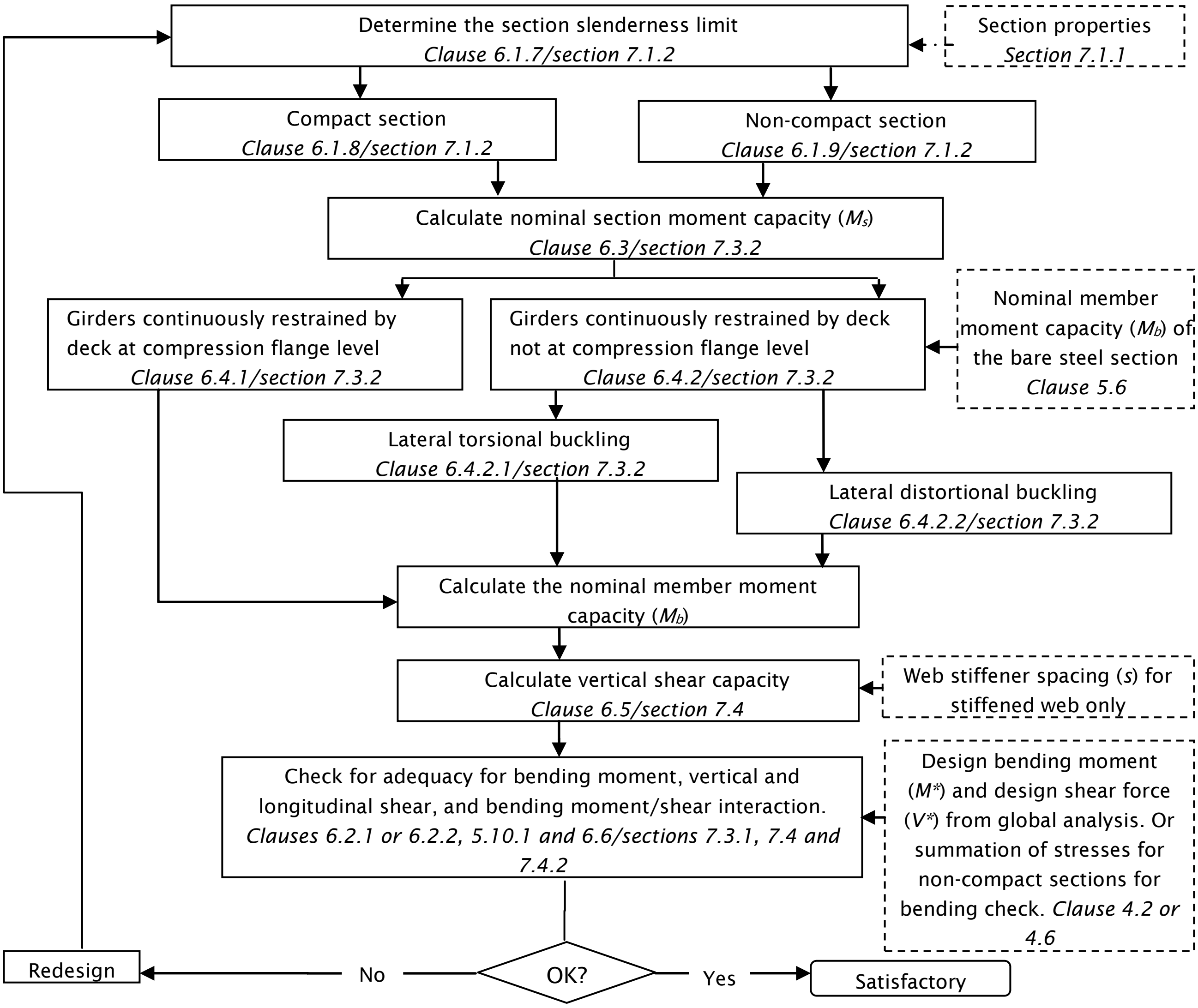


Figure 7.5 Composite main girder design flow chart



7.3 Design for bending

7.3.1 Bending capacity

7.3.1.1 Compact sections

For a member with a compact cross section bent about the major (x-x) axis, the requirement for bending capacity is expressed, for steel sections, in clause 5.1.6 as:

$$M_x^* \leq \emptyset M_{sx} \quad \text{(Equation 7.1)}$$

$$M_x^* \leq \emptyset M_{bx} \quad \text{(Equation 7.2)}$$

For composite sections, the requirement is expressed in clause 6.2.1 as:

$$M^* \leq \emptyset M_{sx} \quad \text{(Equation 7.3)}$$

$$M^* \leq \emptyset M_{bx} \quad \text{(Equation 7.4)}$$

The capacity reduction factor (ϕ) is given in table 3.2 of *AS 5100.6* (SA 2004).

For both sets of requirements, the right-hand side of the first expression represents the bending capacity of the cross section and the right-hand side of the second represents the bending capacity of the beam, although those terms are not actually used in *AS 5100.6*.

Note that, since the plastic moment can be developed and maintained in a compact section, the requirement is simply that the total design bending moment (M^*_x) shall not exceed the bending capacity at ULS; there is no need to check the summation of the separate ULS stress distributions for each stage of construction.

Also, steel beams that are compact when acting compositely with the slab may not be compact when acting alone during construction. In such a case, the checks for the construction condition must be made on the basis of non-compact sections, as stated in clause 6.1.2.

Another point of interest is the difference in the acting design moments at the sagging and hogging moment regions, which are typically located in the mid-span and intermediate supports respectively. This may result in different section classifications for those regions, ie the girder may be compact in the sagging region and non-compact in the hogging region. This is especially important for simply supported bridges that may use the same section in both regions. In this instance, the girder as a whole (in both sagging and hogging moment regions) should be treated as a non-compact member and summation of stresses apply.

In most cases of composite beam and slab construction the hogging moment regions over intermediate supports are unlikely to be compact. To achieve compact classification, the web would need to be much thicker than what is required for shear capacity and therefore would be uneconomic.

7.3.1.2 Non-compact sections

For a member with a non-compact cross section bent about the major (x-x) axis, the requirement for bending capacity is expressed, for steel sections, in clause 5.1.7 as:

$$f_x^* \leq \emptyset f_y \qquad \text{(Equation 7.5)}$$

$$f_x^* \leq \emptyset f_b \qquad \text{(Equation 7.6)}$$

For composite sections, the requirement is expressed in 6.2.2 as:

$$f_s^* \leq \emptyset f_y \qquad \text{(Equation 7.7)}$$

$$f_s^* \leq \emptyset f_b \qquad \text{(Equation 7.8)}$$

$$f_c^* \leq 0.62 f_c' \qquad \text{(Equation 7.9)}$$

The capacity reduction factor (ϕ) is given in table 3.2 of *AS 5100* (SA 2004).

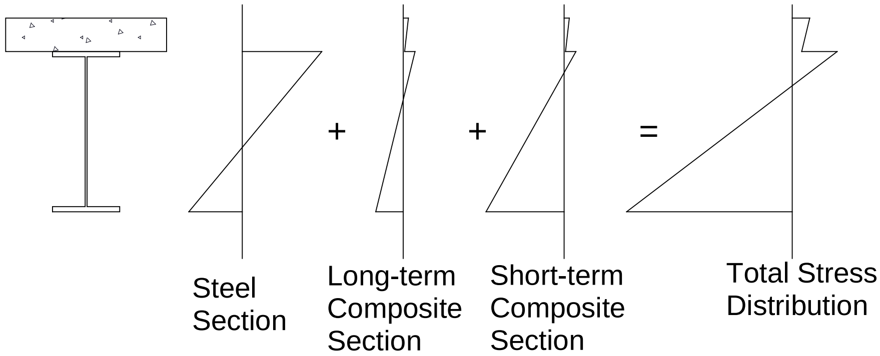
For both sets of requirements, the right-hand side expressions represents the ‘design value’ of the material strength, as limited by buckling bending capacity of the beam, although the term design value is not used in *AS 5100.6*.

The design capacity of non-compact beams in bending is determined by elastic stress distributions and limiting stresses and in most cases checks need only be made at ULS (since the design bending moments at SLS will usually be lower). Checks at SLS are required in only a few circumstances, such as when checking the effects of shrinkage and differential temperature (see section 7.5.3 for further details).

A non-compact section is not able to redistribute stresses once yield is reached in the compression flange and therefore the verification must be made on the summation of the elastically determined stresses at each construction stage.

The total stresses and strains in the fibres of a composite beam where the deck slab is added in stages are determined as the summation of the distributions for each stage (and similarly for short and long-term loads), as shown diagrammatically in figure 7.6. The position of zero stress at ULS will therefore not necessarily correspond with any particular neutral axis level.

Figure 7.6 Summation of stresses for staged construction



Where SLS must be checked as well as ULS, the stress distributions for SLS and ULS must be calculated separately, each using its appropriate set of partial factors for the various loads.

7.3.2 Section moment capacity

7.3.2.1 Compact sections

For steel compact sections, the nominal section moment capacity (M_s) is given by clause 5.2.1 as:

$$M_s = f_y Z_e \qquad \text{(Equation 7.10)}$$

Where the effective section modulus (Z_e), is equal to the lesser of the plastic section modulus (S) and 1.5 times the elastic section modulus (Z) (clause 5.1.3), ie:

$$Z_e = S \leq 1.5Z \qquad \text{(Equation 7.11)}$$

For composite compact cross sections the nominal section moment capacity (M_s) is given by clause 6.3.3 as:

$$M_s = M_p \qquad \text{(Equation 7.12)}$$

Guidance on determining (M_p) is given in appendix E of AS 5100.6 (2004) for sagging moment regions.

7.3.2.2 Non-compact sections

In the verification criteria, reference is made only to stresses; although the section moment capacity (M_s) is defined in clauses 5.2.2 and 6.3.4 and reference to it is only made in relation to interaction between bending and shear (see section 7.4.2).

For a steel non-compact section, the nominal section moment capacity (M_s) is given by clause 5.2.2 as:

$$M_s = f_y Z_{en} \qquad \text{(Equation 7.13)}$$

The designer should be aware that the effective section modulus (Z_e) is the elastic section modulus of the effective section; this in turn is affected by the section classification. As discussed in section 7.1.2, *AS 5100.6* combined the non-compact and slender classifications and the calculation of the effective section modulus is not clear. Based on the guidance given in *AS 5100.6* and Gorenc et al (2005), the effects of the section classification on the effective section modulus, are summarised below:

- For non-compact sections, where $\lambda_{sp} < \lambda_s \leq \lambda_{sy}$ (clause 5.1.4), the effective section modulus (Z_{en}) is between the plastic section modulus (S) and the elastic section modulus (Z). This can be calculated by using;

$$Z_{en} = Z + c_z\,(Z_c - Z) \qquad \text{(Equation 7.14)}$$

  Where:

  Z_c is the effective section modulus assuming the section is compact (ie given by equation 7.11)

  c_z is the $\frac{\lambda_{sy}-\lambda_s}{\lambda_{sy}-\lambda_p}$

- For slender sections, where $\lambda_s > \lambda_{sy}$ (clause 5.1.4), the section is less effective, therefore the effective section modulus (Z_{en}) is less than the elastic section modulus (Z). There are two situations that should be considered in bridges, these are:
  - section elements having uniform compression (ie no stress gradient), such as flanges of a UB bent about the major axis.
    - method 1: (simple method)

$$Z_{en} = Z\left(\frac{\lambda_{sy}}{\lambda_s}\right) \qquad \text{(Equation 7.15)}$$

    - method 2: (clause 5.1.4)

      The effective section modulus is calculated for the effective cross section determined by removing the excess width of plates whose (*b/t)* exceeds the λ_{ey} for that element. See clause 5.1.5 for further information.
  - sections with slenderness determined by a stress gradient in plate elements with one edge unsupported in compression, such as a UB bent about its minor axis.

$$Z_{en} = Z\left(\frac{\lambda_{sy}}{\lambda_s}\right)^2 \qquad \text{(Equation 7.16)}$$

7.3.2.3 Composite stage

Clause 6.3 covers the calculation of the nominal section capacity in the composite stage. The main difference is when determining the effective section modulus (Z_e), the concrete deck is taken into account. As explained in section 7.1.1 and demonstrated in figure 7.3, the transformed area of the concrete is taken in the sagging moment region, while in the hogging moment region only the contribution from the steel reinforcement is taken into account.

7.3.3 Member moment capacity

The moment capacity of a member, whether the girder is non-composite or composite, depends on both the moment capacity of the cross section and on the degree of lateral restraint provided to the critical flange, which for a member supported at both ends is the flange in bending-induced compression.

A steel bridge beam in bending is required to be provided with effective lateral and torsional restraint (about its longitudinal axis) at its supports. Between supports it may be unrestrained, especially during the construction stage. For such a configuration, the member can buckle in a lateral-torsional buckling mode (see section 7.3.4) and the moment capacity of the member depends on both the section moment capacity and the elastic critical buckling moment (referred to in *AS 5100.6* as the reference buckling moment M_o).

The value of the elastic critical moment can be increased by providing intermediate restraints between the supports; the beam can then be considered as a series of segments, each with full or partial restraint at the ends of the segments. In a composite bridge, the presence of the deck slab provides continuous restraint to the top flanges, but, adjacent to intermediate supports, it is the bottom flange that is in compression. Therefore, buckling of the segments between the bracing positions to the cross section for that bottom flange must be considered.

The forms of restraint considered by *AS 5100.6* are discussed in section 7.3.6 below. The type and spacing of the restraints determines the member moment capacity, as discussed below. However, one form of buckling for bare steel girders is not recognized by *AS 5100.6*; this is discussed separately in section 7.3.7.

7.3.3.1 Segments with full lateral restraint

Where the ends of a segment of steel beam are fully or partially restrained and the length is sufficiently short that the slenderness (expressed in terms of the ratio L/r_y, where L is the length of the segment and r_y is the radius of gyration of the beam about the minor axis) does not exceed a limiting value, the segment is considered to have full lateral restraint. Clause 5.3.1 states that, with full lateral restraint, the nominal member moment capacity (M_b) is taken as equal to the nominal section moment capacity (M_s). The slenderness limits are given in clause 5.3.2.4.

For a composite beam in sagging, the compression flange is continuously restrained and, again, $M_b = M_s$ (see clause 6.4.1).

7.3.3.2 Segments without full lateral restraint

When the slenderness exceeds the relevant limit for considering the segment as having full lateral restraint, a slenderness reduction factor needs to be applied to the section moment capacity to determine the member moment capacity. The value of the reduction factor and the means for its determination depend on the type of cross section and type of end restraint. The rules for the various situations covered in clause 5.6.1 are discussed below.

Segments fully or partially restrained at both ends

For a non-composite girder with open sections with equal flanges, the nominal member moment capacity (M_b) is given by clause 5.6.1.1(a) as:

$$M_b = \alpha_m \alpha_s M_s \leq M_s \qquad \text{(Equation 7.17)}$$

Where α_m is a moment modification factor and α_s is the slenderness reduction factor, which depends on the elastic buckling moment (M_{oa}) (either given by clause 5.6.1.1 or determined by an elastic buckling analysis).

For non-composite girders with open I-sections with unequal flanges, the nominal member moment capacity (M_b) is given by clause 5.6.1.2, which uses the same expression as clause 5.6.1.1(a). However, to take into account the unequal flanges a different reference buckling moment (M_o) expression is used. Both cases use the value of effective length given by clause 5.6.5.

For the support regions of continuous composite beams, clause 6.4.2.2 does provide rules for determining a reduction factor for distortional buckling but this relies either on a buckling analysis of the structure or on a simplified evaluation that considers only the whole span and does not take any account of bracing near the support. Although clause 6.4.2.1 applies to situations where there are discrete U-frame restraints, it is not appropriate for the hogging moment regions (the evaluation of effective length in clause 5.6.4 has been derived for situations where the moment is relatively constant).

Segments unrestrained at one end (ie cantilever segments)

For a non-composite girder with a segment unrestrained at one end, the nominal member moment capacity (M_b) is given by clause 5.6.2, again using the value of effective length given by clause 5.6.5.

Beams restrained by U-frames

For non-composite girders restrained by U-frames, the nominal member moment capacity (M_b) is again given by clause 5.6.1.1(a) but the effective length is now given by clause 5.6.3.

7.3.3.3 Effective length

The different calculations for effective length are summarised below:

Clause 5.6.3: Effective length for beams restrained by U-frames

U-frame restraints are created by stiff transverse beams, connected close to the tension flange, acting in conjunction with either plan bracing or deck slab at tension flange level, and with web stiffeners on the webs of the beams. Intermediate U-frames are designed in accordance with clause 8.4.6

For a ladder deck bridge to provide restraint to the bottom compression flange (in the hogging moment region), a moment connection is required between cross-beam and stiffener in order for this connection to provide P or F cross section restraint.

Clause 5.6.4: Effective length for beams continuously restrained by a deck not at compression flange level

This clause is mainly applicable to half-through deck bridges, and is outside the scope of this document.

Clause 5.6.5: Effective length (of a segment)

This clause gives rules for the effective length of segments and sub-segments (between intermediate restraints) for full partial and unrestrained conditions, as defined by clause 5.4 (see section 7.3.6). Allowance for restraint of the compression flange in plan at the ends is included, though such restraint is not normally present.

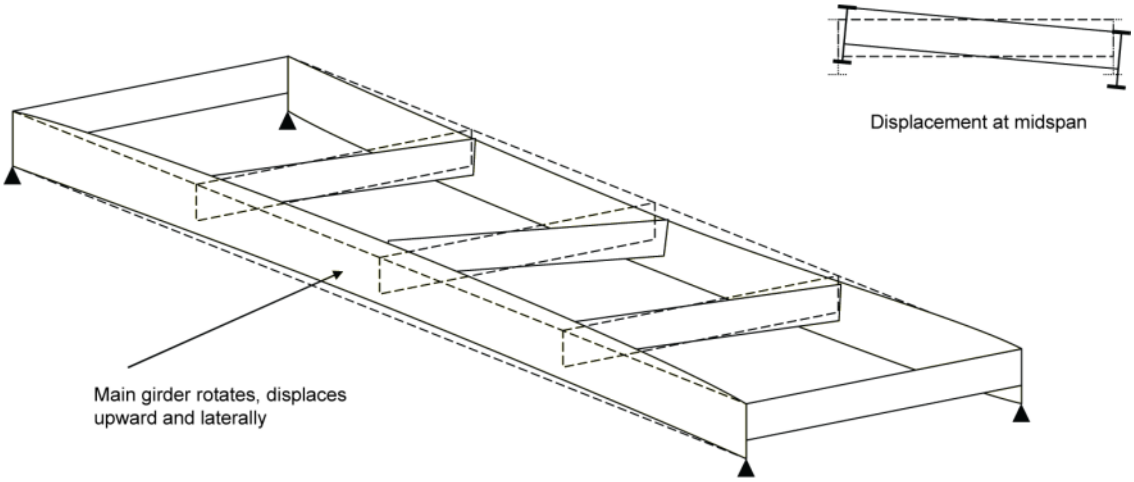
7.3.4 Lateral buckling

The buckling of a length of beam without intermediate restraint under bending is generally referred to as lateral-torsional buckling[3].

Lateral-torsional buckling of a beam occurs when a beam is loaded in flexure (bending) and the critical flange (which is the compression flange) is not restrained in the lateral direction (ie perpendicular to the plane of bending) over a length between positions of effective cross-section restraint. At a certain critical limit, the beam will fail in a mode where the critical flange displaces laterally and the beam twists, as shown in figure 7.7.

[3] Lateral-torsional buckling is a term that is widely used internationally and is the term for this form of instability in the Eurocodes. *AS 5100.6* (SA 2004) does not use the term, although it does refers to ‘elastic flexural-torsional buckling analysis’ in clause 5.6.6. The term flexural-torsional buckling is used in the Eurocodes for a form of instability in members subject to compression. Three types of instability are possible for members in compression: flexural buckling (the most common type of buckling), torsional buckling (involving twist but no lateral displacement) and flexural-torsional buckling (a combined mode). However the first two of these relate to members subject to axial compression which is not the case for bridge beams.

Figure 7.7 Lateral-torsional buckling of an I-girder's critical flange between adjacent points of restraint



The critical load at which the beam buckles may be increased by providing intermediate restraints and its value then depends on the types of restraints and their strength and stiffness. If torsional restraint at the support is not effectively rigid, the critical load will be reduced; this too must be taken into account.

A composite continuous beam (with the slab connected to the top flange) may also be limited by buckling, when the bottom flange buckles laterally adjacent to an intermediate support. This is actually a distortional buckling mode but is conservatively treated in the same manner as lateral-torsional buckling; the critical load depends on the beam geometry and any restraints to the bottom flange.

7.3.5 Restraints

A restraint system (bracing) is required at the supports of the main girders to provide a load path for transferring horizontal forces (transverse to the main girders) to the bridge supports and to restrain the main girder against lateral deflection or twist rotation at different stages during construction of the bridge and throughout its design life. Such restraints are designed in accordance with clause 8.4, which is discussed in section 7.6 below.

Restraints are classified in clause 5.4 according to the degree of restraint that they provide to the cross section and the critical flange. The critical flange is the flange that would deflect the furthest laterally under lateral-torsional buckling (clause 5.5.1). For members with both ends supported (ie not cantilevers) the critical flange is the bottom flange, which is in compression. For a cantilever which is only restrained at the support, the critical flange is the top flange (which is in tension). Cantilevers which are restrained against cross section twist at the unsupported end will then have the critical flange become the bottom flange, which is in compression, which is why both flanges in such a cantilever are considered critical in design.

7.3.6 Types of restraints

Clause 5.4 outlines the different restraint types and the conditions they must satisfy. The different types of restraints and other useful terms are summarised below.

7.3.6.1 Fully restrained (F)

There are three cases where a cross section can be considered fully restrained:

1 A restraint that prevents the lateral displacement of the critical flange of the cross section and prevents twisting of the section. This case occurs at the support of the main girder provided that the restraint is designed to resist the appropriate restraining forces from section 7.3.7.

2 A restraint that prevents lateral displacement of the critical flange of the cross section and partially prevents twisting of the section. This case occurs in the sagging moment region (top flange critical) and where the restraint is designed to resist the appropriate restraining forces from section 7.3.7.

3 A restraint that prevents lateral displacement at a point of the cross section other than the critical flange and prevents twisting of the section. This case occurs in the hogging moment region (bottom flange critical) when the deck is in place and where the web at the cross section has a full height transverse stiffener.

7.3.6.2 Partially restrained (P)

There are two cases where a cross section can be considered to be partially restrained.

1 A restraint that prevents lateral displacement at a point of the cross section other than the critical flange and partially prevents twisting of the section. This case occurs in the hogging moment region when the deck is in place and where the web is unstiffened, provided that the unstiffened web can resist the restraining force. For many bridge beams this will not be the case. Guidance on how to determine the adequacy of an unstiffened web is given in Clifton (1997).

2 A restraint that prevents twist rotation of the cross section and provides partial restraint against lateral deflection of the cross section. This case applies during construction when neither the deck nor any horizontal bracing system is in place and the cross section is stiffened with a moment connection to the cross beams and both the connection and the cross beams designed to the restraining forces from section 7.6.

7.3.6.3 Rotationally restrained (R)

A restraint that prevents rotation of the critical flange in its plane. This condition is not usually achieved and it is conservative to ignore rotational restraint

7.3.6.4 Laterally restrained (L)

The restraint effectively prevents lateral displacement of the critical flange of the cross section without preventing twisting of the section. This is found in the sagging moment region, where the top flanges are tied together by the web members of a horizontal truss used for stability during erection.

7.3.6.5 Unrestrained (U)

This is a cross section that does not comply with types fully (F), partially (P) or laterally (L) restrained, ie does not prevent lateral displacement or twisting of the section.

7.3.6.6 Segment

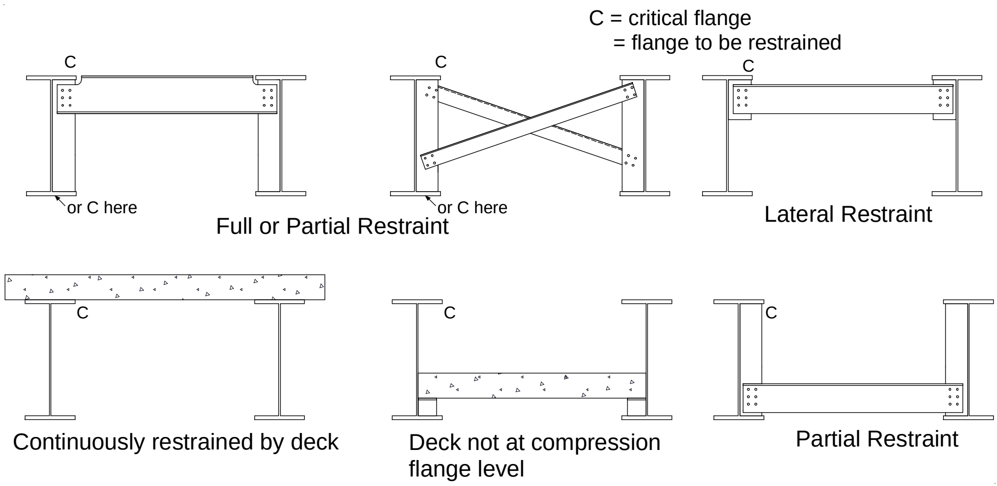
The length of a beam between adjacent points of full or partial cross section restraint, (F or P) or between a full or partial cross section restraint and an unrestrained end in the case of a cantilever. The cantilever case is rare in bridge girders. For a beam having FF or PP end restraints and no mid-span restraints, the segment length is equal to the beam span. An additional lateral restraint at mid-span would result in a sub-segment length of one-half span with end restraints of FL or PL.

7.3.6.7 Sub-segment

A segment can be subdivided into portions having at least the lateral (L) restraints to the critical flange at their ends. Restraint combinations can be FL, PL or LL. Note that sub-segments are used in *AS 5100.6* (SA 2004) and this is the notation used in this guide. This is different from *NZS 3404* (SNZ 1997b) where the sub-segment concept has been deleted. It is an unnecessary complication but for consistency of notation with *AS 5100.6* is retained in this guide. It also has more relevance to bridges as pairs of beams restrained only by cross beams cannot be considered laterally restrained even when the cross beams are located near the critical flange. They have to be designed for the restraining moment and hence provide partial restraint.

Figure 7.8 shows the type of restraint offered to the girder and the potential location of the critical flange in different restrained girders. Note that the type of restraint in some cases is dependent on which flange is critical, ie whether the bending moment at that point along the beam is hogging or sagging. Further details on restraints and their design are given in section 7.6.

Figure 7.8 Examples of restraining systems for prevention of lateral-torsional buckling failure of simply supported beams



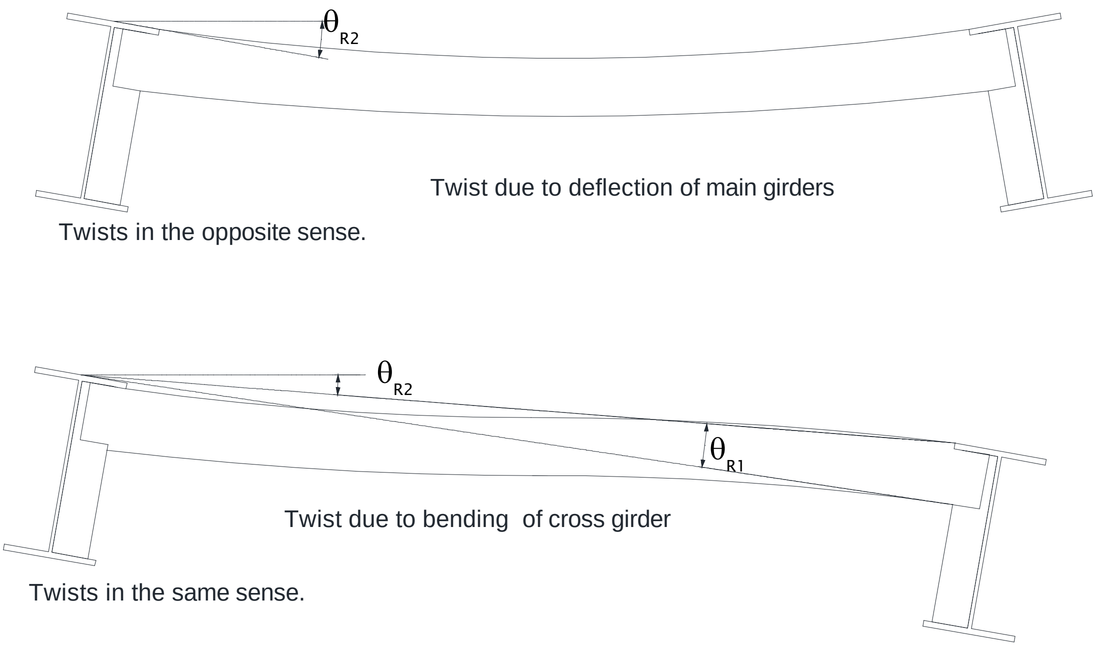
One form of restraint to bare steel beams that is not covered by *AS 5100.6* is the intermediate torsional restraint (without any lateral restraint), shown in figure 7.9. This is achieved by pairing beams together with transverse planes of triangulated bracing or by stiff cross beams that are rigidly connected to web stiffeners on the main girders.

Triangulated bracing is often used in multi-girder construction (see section 2.1.3) though cross girders are used when the main girders are relatively shallow (up to about 1200mm deep). Cross beams are usually lapped and bolted to the web stiffeners; often the ends of the beams have deep gusset plates welded to them so that the bolt group has a higher moment capacity. Once the deck slab has been cast, torsional bracing effectively becomes part of a lateral bracing system.

The ladder deck form of construction, in which the cross beams are rigidly connected to stiffeners on the main girders, provides a series of closely spaced torsional restraints. Once the deck slab has been cast the cross girders effectively form inverted U-frames and these may be used to provide some restraint to the bottom flanges adjacent to intermediate supports. An alternative design for lateral restraint to that offered by *AS 5100.6* has been developed in the UK for ladder deck bridges. Design provisions are given in *PD 6695-2* (BSI 2008) and that guidance is reproduced, in a format compatible with *AS 5100.6* terminology, in appendix E.

The torsional flexibility can be evaluated for a unit torque at each beam, in the same sense. There are two components to the resulting displacement, one due to the vertical deflection of the main girders and one due to the double curvature flexure of the cross beam. The presence of this torsional restraint will increase the elastic buckling moment (and thus decrease the slenderness) relative to that for the span without any intermediate restraint but it is not as effective as plan bracing.

Figure 7.9 Determination of torsional flexibility



7.3.7 Design of restraints to AS 5100

Section 7.6 covers the forces required for design of restraints, note that *AS 5100.6* (2004) does not require a stiffness check for restraints that are designed to the specified level of force or moment. This is the same approach taken in *NZS 3404* (SNZ 1997b) and follows from research referenced in Mutton and Trahair (1975) and Clifton (1994). It is one reason why the restraining force is set at 2.5% of the maximum critical flange force. Other standards specify a lower level of restraining force, typically 1% in conjunction with a stiffness requirement.

When the main girders are F or P restrained by cross girders with a moment connection then the connection and the cross girder must be designed to resist the following:

1 When the bottom flange is critical, the moment is that determined from 2.5% of the moment induced compression force in the critical flange of the main girder acting at a lever arm corresponding to the vertical distance between the centroid of the bottom flange and the centroid of the cross girder. For pairs of main girders so connected and without a deck or plan bracing in place, this provides P restraint and the displaced shape will be as shown in the top diagram of figure 7.9, involving the two critical flanges moving away from each other. The major principal axis bending moment induced in the cross girder is additional to the moment on that girder from the direct loading.

2 When the top flange is critical, the girders are anchored by the bottom flange which is in tension and so, for a pair of main girders, the displaced shape will be as shown in the bottom diagram of figure 7.9, involving the two critical flanges moving in the same direction as they are rigidly held apart by the cross girder. The moment generated in the restraint is that determined from 2.5% of the moment induced compression force in the critical flange of the main girder acting at a lever arm corresponding to the vertical distance between the centroid of the bottom flange (ie the tension flange) and the centroid of the cross girder. This moment has to be included in the connection design in conjunction with the vertical shear on the cross girder from applied loading. It also has to be considered in the cross girder design; however, where this is designed to resist the applied loading as simply supported, the maximum positive

bending moment from applied loading will be near mid-span, which is where the restraint induced moment will be zero, so the restraining actions will not increase the maximum design bending moment on the cross girder. There is also the opposite case to consider, in which the two critical flanges try to move towards each other and put the cross girder into compression; however, that case is less severe than the above case for the connections between the main girder and cross girder and will only govern the cross girder design if the slenderness ratio for compression buckling about its minor principal axis is high, generating a low member compression capacity.

3 Note that as described in section 7.3.6 for a pair of main girders restrained by cross girders this detail provides partial (P) cross section restraint only when there is no slab or plan bracing between the two beams present.

When the cross girders are more closely spaced than is required to ensure that $M^* \leq \phi M_{bx}$, then a lesser restraining force and moment may be designed for in accordance with clause 8.4.2. Further guidance on the design of restraint is given in section 7.6 below.

7.4 Shear resistance

7.4.1 Shear capacity

The girder web is assumed to be the component that resists the shear force acting on the given member; in a composite section, the slab is assumed to resist no shear (see clause 6.5). The calculation of the nominal shear capacity of the web (V_v) is given by clause 5.10 as:

$$V^* \leq \emptyset V_v \qquad \text{(Equation 7.18)}$$

The nominal shear capacity (V_v) is governed by the shear stress distribution in the web, which can be either approximately uniform or non-uniform.

For approximately uniform shear stress distribution, $V_v = V_u$ and the value of V_u is dependent on the web panel depth to thickness ratio (clause 5.10.2). Approximately uniform shear stress distribution occurs in girders with equal flanges, such as in Universal Beam and welded beam sections.

If the web depth to thickness ratio does not exceed a limiting value (given by clause 5.10.2(2)) then the nominal shear capacity (V_u) is provided by the gross section area of the web and the value is equal to the nominal shear yield capacity (V_w). However, if the ratio exceeds the limit then the nominal shear capacity depends on the nominal shear buckling capacity (V_b).

An unstiffened slender web is unable to develop full shear yield resistance because its capacity is limited by shear buckling. By introducing transverse stiffeners, the buckling resistance of the web can be increased. The increase arises firstly from the constraint of the rectangular panel (between stiffeners and flanges) and secondly because in very slender panels, some of the shear is carried by tension field action.

The nominal shear yield capacity (V_w) is given by clause 5.10.4 and the nominal shear buckling capacity (V_b) is given by clause 5.10.5.

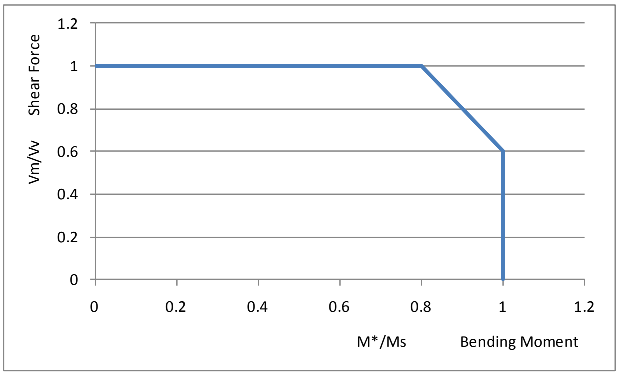
The same rules for calculating shear resistance apply to webs in both compact and non-compact sections but the webs in compact sections are likely to be less slender, since the depth in compression is limited. However, although mid-span sections of composite beams may well be compact, since there is very little depth of web in compression, the web may be sufficiently slender that the shear capacity must be based on the shear buckling capacity.

7.4.2 Moment/shear interaction

Clause 5.11 gives two methods for taking account of combined moment and shear in a beam. The first is the proportioning method (clause 5.11.2); this method says that if the bending moment is assumed to be resisted by the flanges alone, then full shear capacity of the web is available to resist the design shear force.

The second method, the shear and bending interaction method (clause 5.11.3) says that when the design bending moment is assumed to be resisted by the whole cross section, then the nominal shear capacity in the presence of bending moment is modified. The modified nominal shear capacity (V_{vm}) depends on the ratio (M^*/M_s), where (M^*) is the design bending moment and (M_s) is the nominal section moment capacity (note that this is the only place where the value of (M_s) is required for non-compact sections and for a composite section built in stages the value for the section at the stage considered should be used). An illustration of the limiting envelope for interaction of shear and bending is shown in figure 7.10. Additional guidance on stiffened web panels that are designed to resist a combination of bending moment, shear, axial and transverse loading is given in appendix B of *AS 5100.6* (SA 2004).

Figure 7.10 Moment/shear interaction



The moment/shear interaction should be checked for the worst moment and worst shear anywhere within the web panel length (ie between intermediate web stiffeners), rather than at a single section. This is only slightly conservative for web panels adjacent to internal supports but seems rather onerous for mid-span regions of beams of compact section, where the panel could extend between stiffeners at the points of contraflexure (mid-span moment is then considered in combination with shear at the zero moment position).

For beams constructed in stages, the moment acting on the section should be taken as the total moment for sections designed as compact (clause 5.1.6), but for sections designed as non-compact an effective bending moment must be derived for use in the interaction formulae. This is obtained by multiplying the extreme fibre total stress by the modulus for that fibre in the section that is appropriate to the stage of construction being checked (clause 6.3.4). The designer should take care to ensure that the fibre for which total design stress is used to determine the equivalent bending moment is the same fibre that determines the bending resistance.

7.5 SLS checks for compact and non-compact beams

Clause 3.3 outlines the different SLS checks that must be considered when designing the different components in the bridge structure and in this case the main girders are being checked for whether they are non-composite bare steel or composite, compact or non-compact. This guidance is also applicable to other structural members, such as beams used in restraints. The SLS checks required are:

- deflection limits for girders (clause 3.3.1)
- vibration of girders (clause 12 of *AS 5100.2* (SNZ 2004))
- stress limits during staged construction (clause 4.6.3)
- crack control of slabs in tension (clause 2.4.4 of *NZS 3101* (SNZ 2006)).

The common factor between all the above SLS checks is the applied load acting on the structure. When the SLS is being considered, most of the applied loads are un-factored compared with the ULS case where load factors are included. The SLS load combinations are given in section 6.4.7.

Other SLS checks for bridge components such as deck slab, bolts and longitudinal shear connections will be given under their relevant section.

7.5.1 Deflection limits for girders

The deflection limit for girders under traffic loading for SLS is given in clause 6.11 of *AS 5100.2,* where it is stated that the deflection shall not be greater than L/600 of the span or L/300 for the cantilever projection. In addition to the above requirements, the following criteria should be observed which are also given in clause 6.11:

- Deflections do not infringe on clearance diagrams.
- The deflection in the hogging moment region (between the supports and point of contraflexure) should not exceed L/300 of the span.
- No deflection is to occur under permanent loading in the sagging moment region.

The deflection is determined by the use of elastic analysis, in accordance with clause 4.2 and as summarised in section 6.1, for the appropriate SLS loading combination.

7.5.2 Vibration of girders

Section 3.4.15 of the *Bridge manual* requires that all highway bridges should be checked for vibration. Although the *Bridge manual* presents acceptability criteria for vibrations it does not, however, provide guidance on how this criteria should be verified. To remedy this situation, some recommendations are presented below.

7.5.2.1 Vehicular traffic

When considering vibrations occasioned by vehicular traffic, although most international standards describe the severity of vibrations in terms of acceleration, the *Bridge manual* specifies that the maximum vertical velocity during a cycle of vibration should be limited to 0.055m/s when the design load is taken as two 120kN axles of one HN load element. From vibration theory, the velocity of a structure can be calculated from the following equation:

$$v = 4\pi f_o \tilde{y} \qquad \text{(Equation 7.19)}$$

Where:

f_o is the natural frequency of the structure

$\tilde{y}$ is the displacement amplitude, which is defined as:

$$\tilde{y} = \frac{P}{k}\frac{1}{2\zeta} \qquad \text{(Equation 7.20)}$$

Where:

P is the amplitude of the force

k is the stiffness of the structure under consideration

ζ is the damping ratio.

Some typical damping ratios, from *BS 5400-2* (BSI 2006), for bridges are presented in table 7.2 below:

Table 7.2 Typical damping ratios for bridges

Bridge superstructure	Logarithmic decrement δ	Damping ratio ζ
Steel with asphalt or epoxy surfacing	0.03	0.477%
Composite steel-concrete	0.04	0.637%
Prestressed and reinforced concrete	0.05	0.796%

7.5.2.2 Pedestrian and cycle traffic

When pedestrian and cycle bridges are considered, the *Bridge manual* refers to appendix B of *BS 5400-2* and appendix A of *BD 37/01* (Highways Agency 2010); the latter is available as a free download. Should the fundamental frequency of horizontal vibration of the bridge be found to be less than 1.5Hz limit, a dynamic analysis should be undertaken to evaluate the maximum horizontal acceleration in accordance with *BS EN 1991-2 Eurocode 1: Actions on structures part 2: Traffic loads* (CEN 2003). According to the *Bridge manual*, suitable limits for lateral acceleration are given in *Guidelines for design of footbridges* (fib 2005).

For cases when the spans of pedestrian and cycle bridges exceed 30m, aerodynamic effects may be critical. In these cases, wind vibration effects as detailed in *BD 49/01* (Highways Agency 2001) should be considered.

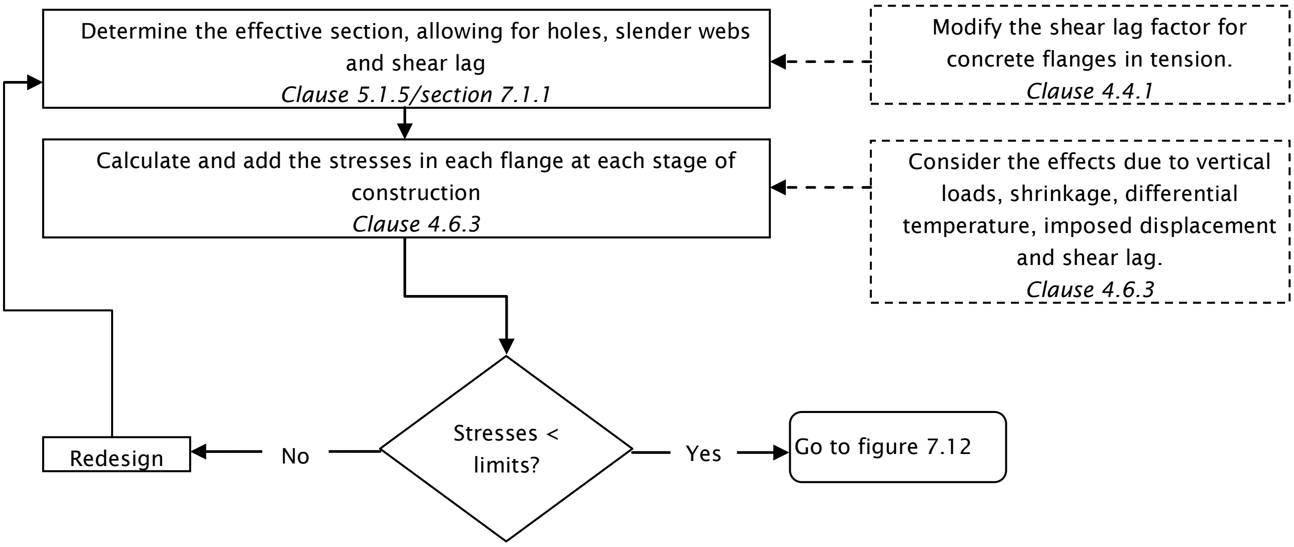
7.5.3 Stress limits during staged construction

Clause 4.6.3 requires that the SLS stresses at each stage shall be calculated elastically and added together.

In sections 5 and 6 of *AS 5100.6*, no limits are given for total stresses at SLS but it may be presumed that the limit is the yield strength for steel elements. For non-compact sections, the ULS limit is always more onerous. For compact sections, the SLS limit may govern in rare situations.

The design steps for this verification are summarised in figure 7.11.

Figure 7.11 Verification of stress limits at SLS during staged construction



7.5.4 Crack control of slabs in flexure and tension

The requirements for controlling crack width for the concrete deck slab in tension are given in clause 2.4.4 of *NZS 3101*. The calculated crack width shall not exceed those specified in section 4.2.1(a) of the *Bridge manual* unless, alternatively, the requirements of clause 2.4.4.1(a) of *NZS 3101* are satisfied. Notwithstanding this, experience has shown that due to the steel girders preventing the concrete from shrinking, large restrained shrinkage cracking occurs. In this case, it is recommended that the minimum reinforcement requirements of clause 6.1.4 of *AS 5100.6* are provided to minimise this effect.

7.6 Design of restraints to beams

The restraints to beams need to be designed for the forces generated by their restraining function in providing restraint against buckling and any directly applied forces. The forces associated with the provision of restraint are determined in accordance with section 8 of *AS 5100.6* and are discussed below. Directly applied forces are discussed in section 7.6.7.

The forces in the individual bracing members (at a particular location) due to the restraint forces and applied forces may be determined from a simple frame analysis. The members should be verified for the bending and axial forces: requirements for tension and compression members are given in sections 9 and 10 of *AS 5100.6* respectively and are not discussed in this guide. Advice on the design of tension and compression members is available in the *Steel designers handbook* (Gorenc et al 2005) and *Simplified design of steel members* (Bird and Feeney 1999).

It should be noted that the restraint stiffness requirement is not given in *AS 5100.6*. Commentary clause C8.4.2 of *AS 5100.6* states that research conducted by Mutton and Trahair (1975) found that the stiffness requirements of restraints are satisfied when the strength requirements are met. Similar results were found by Medland and Segedin (1979). It is concluded that the strength requirements given below will achieve adequate stiffness for the types of restraint to the main girders.

7.6.1 Restraints against lateral deflection

For restraints that prevent lateral deflection in the main girder, clause 8.4.2 states that the transverse force acting on that restraint shall be taken as 0.025 times (ie 2.5% of) the maximum force in the critical flange in the main girder (ie for the most onerous ULS load combination) (see section 6.4.7). Where the restraints are more closely spaced than is required, then clause 8.4.2 allows the forces on individual restraints to be reduced. However, it is much easier (and not significantly more onerous) to design all restraints for the 2.5% of the force in the flange.

7.6.2 Restraints against twist rotation

The requirements for restraint against twist about the longitudinal axis of the main girder are given in clause 8.4.3. For a restraint to be considered as fully effective, the clause requires it to be able to transfer 2.5% of the maximum force in the critical flange in the main girder. A torsional restraint is actually required to provide a restraining moment, rather than a force. It is presumed that the restraining moment is due to a couple of the horizontal forces at each flange, the magnitude of the force being 2.5% of the force in the critical flange. (Again, this is for the most onerous ULS load combination for the location considered).

For restraints to be considered as providing partial restraint against twist rotation, they must be able to provide an elastic restraint against twist rotation without rotational slip. Flexible elements, such as unstiffened webs, may form part of such a restraint provided they are designed and connected to prevent rotational slip. No advice is given in clause 8.4.2 about the design force for such restraints but it would be sensible to design them for the same restraint forces given above.

Restraints that permit rotational slip are deemed to be ineffective in restraining twist rotation of the main girder.

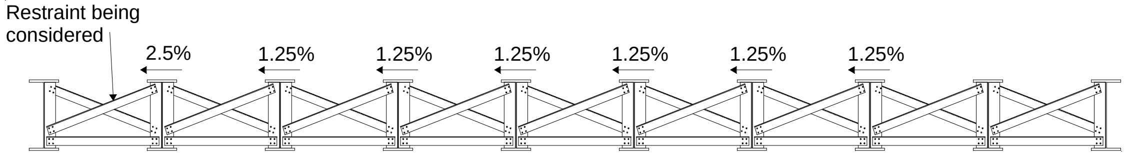
Guidance on the effects of the stiffness of torsional restraints at supports on the elastic buckling moment is given in appendix A.5 of *AS 5100.6*.

For intermediate torsional restraints, as discussed in section 7.3.6, their stiffness is taken into account in determining the slenderness; the design forces associated with provision of that restraint are given in *PD 6695-2* (BSI 2008) and appendix D of this guide.

7.6.3 Parallel restrained members

Clause 8.4.4 gives the requirements for the design forces in restraints to a series of parallel members. This is specifically for multi-girder bridges where the restraints are connecting all the main girders. In this case, each restraint shall be designed to transfer transverse force with the sum of 2.5% of the flange force from the connected member and 1.25% of the flange force from the adjacent connected member. However, no more than seven members need to be considered. Note that shallow bridges and/or very wide (>15m) decks may have a large number of main girders, depending on their requirements, in comparison with typical bridges with three to four main girders with a deck with less than 15m. See figure 7.12 below for details.

Figure 7.12 Parallel restrained members transverse force transfer



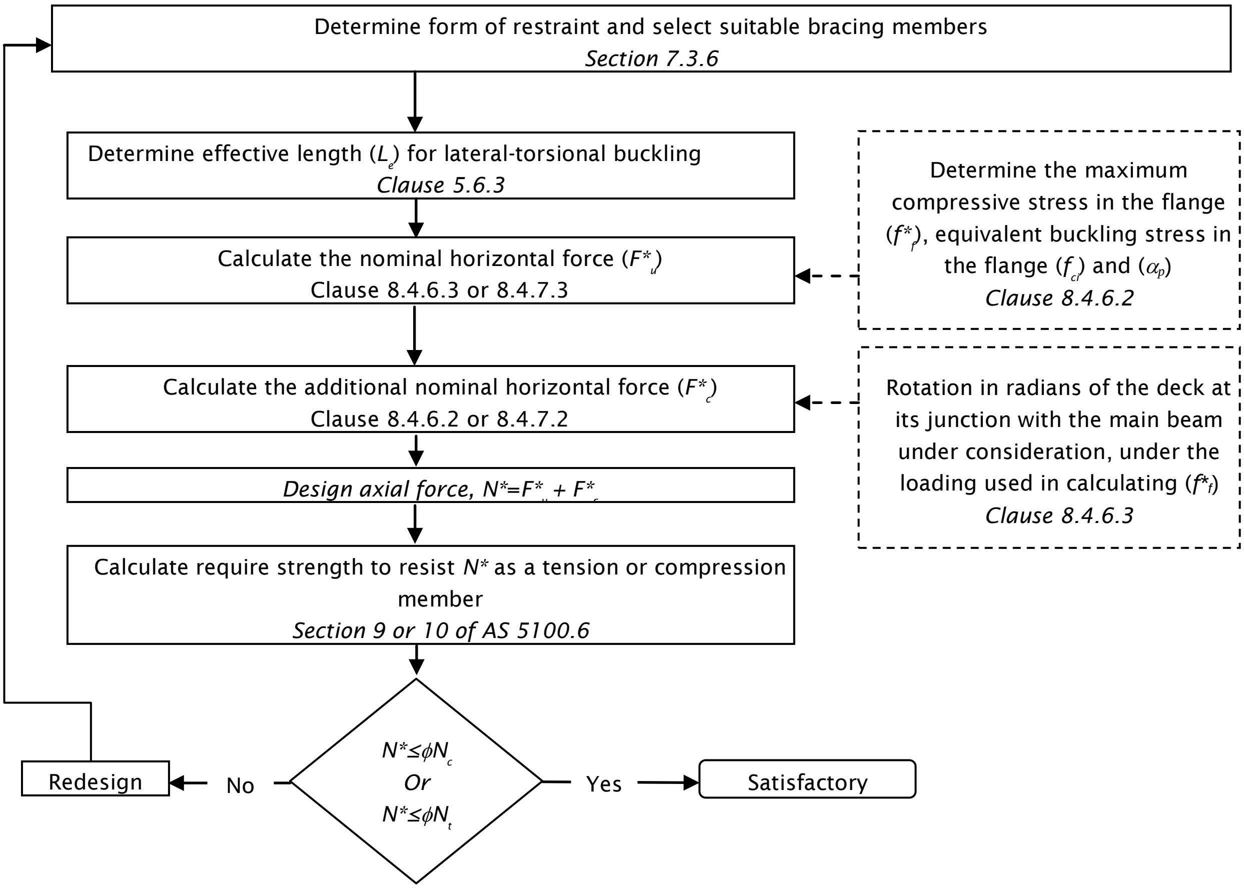
7.6.4 Restraints against lateral rotation

The requirements for effective restraints against lateral rotation (in the plane of the restrained flange) are given in clause 8.4.5. Stiffness requirements for such restraints are given in appendix A of *AS 5100.6* but no requirements are given in clause 8.4.5 for the values of design moment at the restraint position. As a rule, a transverse member providing rotational restraint in plan should have the same stiffness as the stiffness measured about the strong axis of the flange of the girder to which the rotational restraint in plan is being provided, and should have a rigid connection to that flange. These conditions will seldom be provided, which is why the influence of rotational restraint in plan is usually ignored. Where restraint against plan bending is assumed, the restraining members will need to be designed for bending in plan as well as for axial force due to the lateral restraint provided at the location.

7.6.5 U-frame restraint and decks not at compression flange level

Clauses 8.4.6 and 8.4.7.2 outline the design procedure to determining the horizontal forces acting in the restraints, in U-frame restraints and for decks not at the compression flange level. These horizontal forces are the nominal horizontal force (F^*_u) and the additional nominal horizontal force (F^*_c). Both types of restraints follow the same design steps which are summarised in figure 7.13 below.

Figure 7.13 U-frame and decks not at compression flange level design flow chart



7.6.6 Deck at compression flange level

Clause 8.4.7.1 states that if the concrete deck is at the compression flange level and complies with clause 5.4.2, then it can be considered to provide continuous restraint to the critical flange. Therefore, the transverse force is taken as 2.5% (or 0.025) of the maximum force in the critical flange and shall be distributed uniformly along the span of the main girder. This in turn will be resisted by the shear connectors between the steel girder and the concrete deck. The design of the shear connectors is given in section 7.7, using the calculated transverse force to verify this case.

7.6.7 Directly applied forces

Intermediate restraints may be required to transfer forces due to wind (usually very modest magnitude for medium span bridges) and possibly due to impact forces due to collision on a superstructure. In the event of seismic action, they may also be required to transfer forces due to horizontal acceleration. If girders are curved in plan, they will also be required to transfer 'radial' forces due to the curvature.

Restraints at supports are required to transfer all the effects of horizontal forces to laterally restrained bearings or shear keys.

The effects of seismic action on restraints are not covered in *AS 5100.6.* However, consideration of seismic action is required in New Zealand and for restraints the check is straightforward. In this case, the horizontal force due to seismic action is determined from global analysis (using computer modelling described in section 6.3 for the seismic load combination (section 6.5) acting on the restraint. This in turn is compared with either the transverse force or the nominal horizontal force, as described above, and the maximum calculated force is used to design the restraint.

7.6.8 Restraints at supports

Bracing or pier diaphragms at supports provide torsional restraint to the main girders and a load path for transferring lateral forces to restraint bearings and/or shear keys. The same 2.5% transverse force and directly applied forces should be considered at the supports when designing restraints at supports.

7.7 Longitudinal shear connection

To provide the necessary shear transfer between the steel girder and the concrete slab that is required for composite action, shear connectors are required on the top flange. The shear flow varies along the length of the beam, being highest near the supports, and it is customary to vary the number and spacing of connectors to provide just sufficient shear resistance for economy. The most commonly used form of connector is the headed stud, though channel connectors and high strength structural bolts may be used (clause 6.6.4.1).

The SLS, fatigue and ULS design procedures for longitudinal shear connection are set out in figures 7.14 to 7.16.

Figure 7.14 Longitudinal shear connection SLS design flow chart

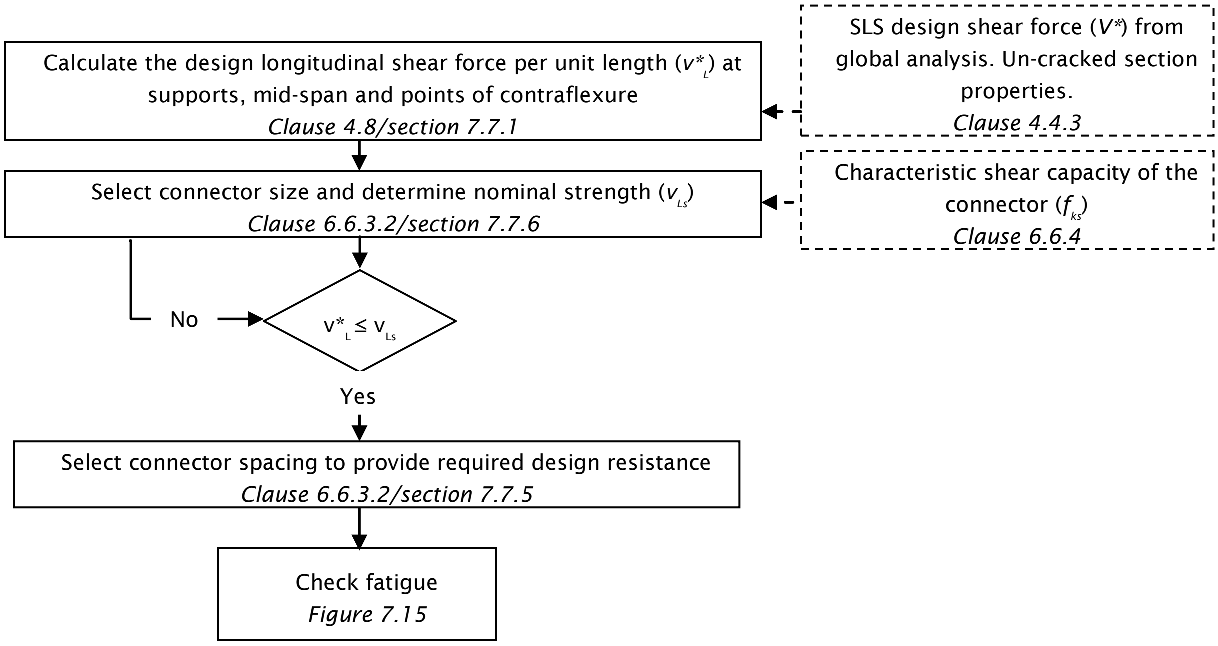


Figure 7.15 Longitudinal shear connection fatigue design flow chart

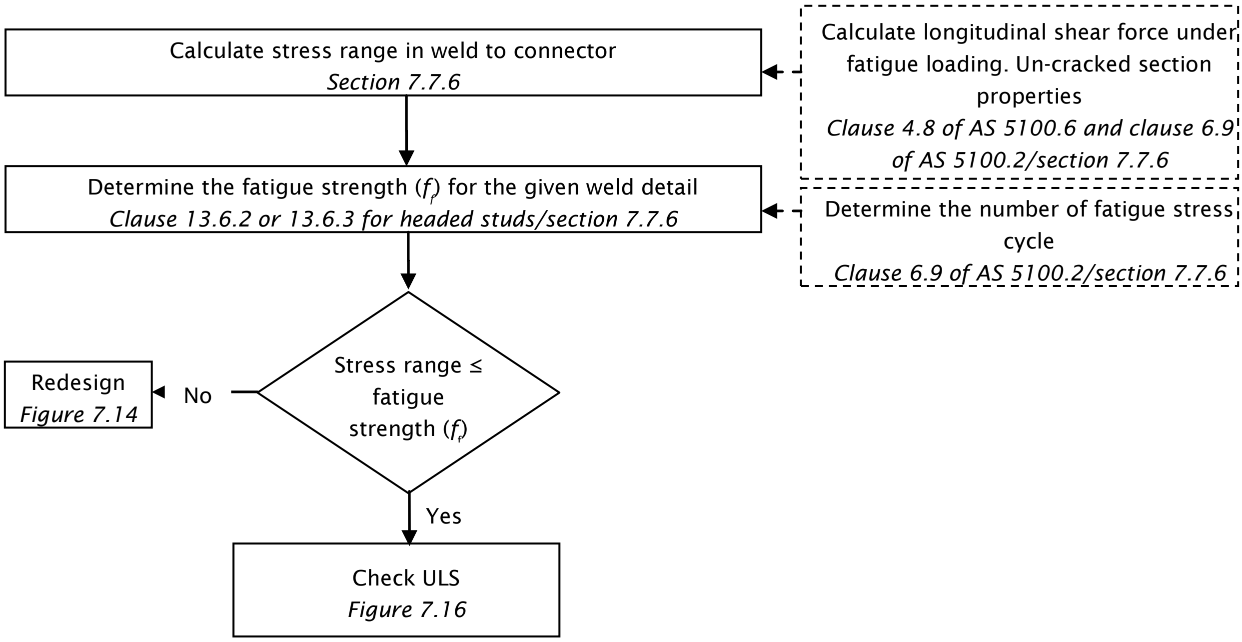
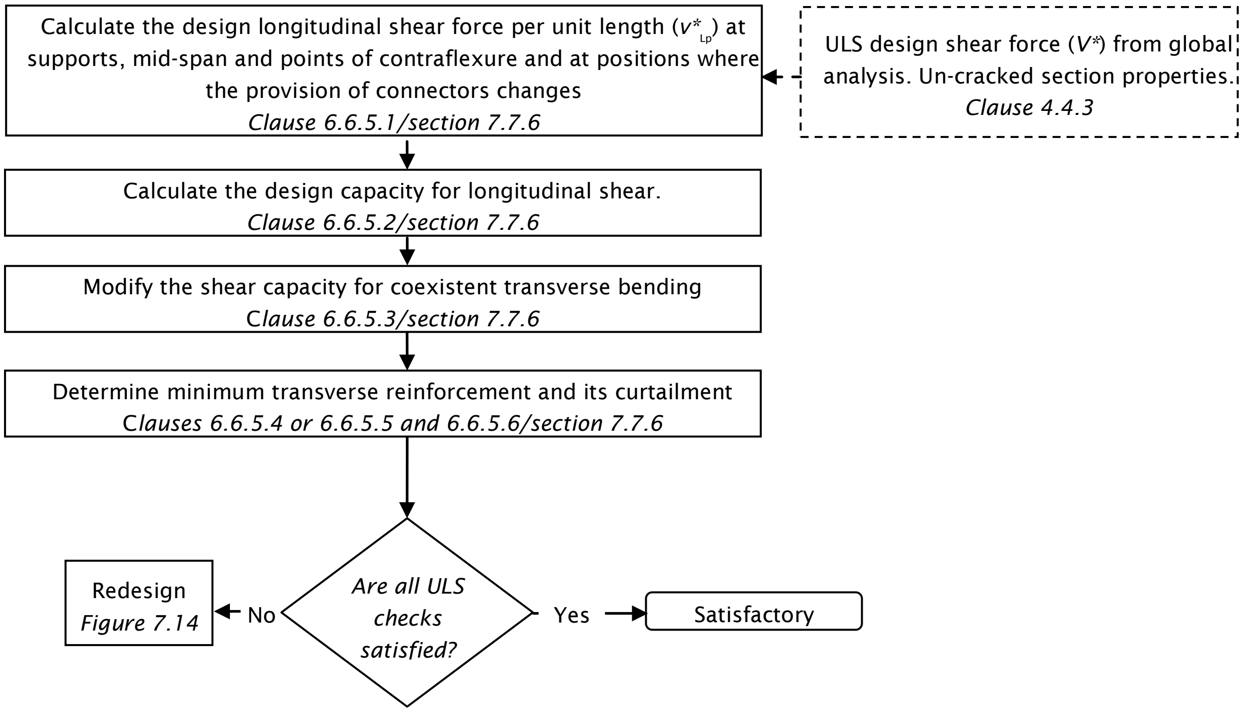


Figure 7.16 Longitudinal shear connection ULS design flow chart (or design of transverse reinforcement at shear studs)



7.7.1 Longitudinal shear

The longitudinal shear is the means by which axial force is transferred from the girder into the slab. The design longitudinal shear force per unit length (v^*_L) is given in clause 4.8, based on simple elastic theory of bending, by the following expression.

$$v_L^* = \frac{V^* A_t y_c}{I_t} \qquad \text{(Equation 7.21)}$$

Where:

V^* = design shear force at the cross section.

Clause 4.8 does not distinguish between design shear forces at ULS and SLS; the expression applies to both situations. The expression is presumed to apply at ULS even when the cross section is compact (and thus the plastic bending capacity is mobilised).

Where the beam cross section varies significantly (eg for a tapered section or for different girder dimensions at the hogging and sagging moment region) the variation of section properties must be taken into account (use the properties at the cross section being considered).

Clause 4.8 states that the design longitudinal shear force is to be calculated using the un-cracked section properties in sagging moment region with the short-term concrete modular ratio, while for the hogging moment region it allows either the use of the area of the embedded steel and the un-cracked concrete area or the area of the embedded steel on its own. This gives a conservative value without the need for more complex calculation, even when the plastic resistance of the cracked section is relied upon. Short term un-cracked properties may be used for this purpose. This force is calculated for both the SLS and ULS cases using the relevant design shear force (V^*) for each case. This is based on the load combination summarised in section 6.4.7, which not only includes the traffic loading but other loading such as temperature effects and shrinkage, especially at the hogging moment region.

In the positive moment regions, the design (v^*_L) varies from a maximum at the point of contraflexure to zero at the point of maximum moment. This can be used to reduce the shear stud numbers (by increasing the spacing) accordingly.

7.7.2 General requirements

Clause 6.6.1 requires the shear connection to be designed to satisfy SLS requirements and for fatigue resistance (section 7.7.4). No checks are required at ULS, except when the connectors are also subject to tensile forces (note that the reference to clause 6.6.3.3 in clause 6.6.1 is erroneous). The clause also requires the transverse reinforcement to be designed to resist the longitudinal shear force but for this check, ULS values of design longitudinal shear force must be used.

The values of characteristic shear resistance given by clause 6.6.4 are only valid when certain dimension limitations on spacing and height of the shear connectors are met. These limitations are set out in clause 6.6.2.

7.7.3 Design for shear at SLS

The basic requirement for shear connection is expressed in clause 6.6.3.2 as:

$$v_L^* \leq \emptyset v_{ls} \qquad \text{(Equation 7.22)}$$

Where:

v^*_L = design shear force per unit length,

v_{ls} = permissible shear force per unit length,

ϕ = capacity reduction factor (given by table 3.3 of *AS 5100.6* as 1.0).

Over lengths where the shear connector spacing is constant, clause 6.6.3.2 allows the SLS value of shear per unit length at any particular location to exceed the capacity per unit length (ϕv_{Ls}) by up to 10%, provided that the total shear over the length where the spacing is constant does not exceed the total design shear resistance over that length.

For negative (hogging) moment regions, clause 6.6.3 imposes a further requirement that the adequacy to resist a total horizontal shear force (F^*_h) should also be verified. This force will be resisted by the shear connectors between the position of maximum negative bending moment and the adjacent position of zero moment (the point of contraflexure), at the SLS case. Therefore, in the hogging moment region the more onerous of the longitudinal shear force (v^*_L) (as it and, potentially, the capacity vary along this length) and the total horizontal shear (F^*_h) shall be used when designing for the SLS case. The total horizontal shear (F^*_h) expression is given in clause 6.6.3.3 which is outlined below:

$$F_h^* = 0.55 A_{rs} f_{sy} \qquad \text{(Equation 7.23)}$$

Where:

A_{rs} = area of slab reinforcement within the effective width of the slab

f_{sy} = yield stress of tensile reinforcement.

7.7.4 Design for shear at ULS

In certain cases, shear connectors may be subject to tensile force as well as to longitudinal shear. This situation arises when forces tend to separate the slab from a girder; or when there are transverse moments on a group of connectors resulting from the restraint to transverse bending of the slab, particularly in the region of diaphragms or transverse cross-bracing as stated in clause 6.6.3.4. In the latter case, additional ties, suitably anchored, are required to resist these forces.

For such situations, the requirement at ULS is expressed as a limit to the reduced shear capacity:

$$v_L^* < \emptyset n\left[f_{ks} - \frac{N_u^*}{\emptyset\sqrt{3}}\right] \qquad \text{(Equation 7.24)}$$

Where:

ϕ = capacity reduction factor (given by table 3.2 of *AS 5100.6*)

n = number of shear connectors per unit length

f_{ks} = characteristic shear capacity of the connector specified in clause 6.6.4

N^*_u = design axial tension on the shear stud at the strength limit state.

The requirement for fatigue resistance is also modified in these situations. See clause 6.6.3.4(b)(ii).

7.7.5 Positioning of shear connectors

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

The spacing of studs (for main girders and cross girders) needs to be coordinated with the spacing of transverse reinforcement, to avoid potential clashes in positions. Detailed requirements for the placement of the shear connectors are given in clause 6.6.2.

7.7.6 Fatigue design of shear connectors

Section 13 of *AS 5100.6* covers the fatigue design requirements for all welded components in the bridge structure, which is discussed in section 7.11. However, the fatigue design of shear connectors is given below.

It should be noted that other than the specific tension case outlined in section 7.7.4 above, the design of shear connectors will be undertaken for the SLS case. However, as seen in figure 7.16, if the shear connector did not satisfy the fatigue requirements, then the connector will have to be redesigned accordingly. This can either be done by changing the stud spacing first and if needed increasing the number of shear connectors or using bigger headed studs or bolts.

7.7.6.1 Fatigue loading and number of fatigue stress cycles

Clause 6.9 of *AS 5100.2* gives the requirements for determining the fatigue loading on shear connectors and the number of fatigue stress cycles. This data is required for determining the stress range in the weld to the shear connector. Further information on the use of the fatigue loading is given in section 6.4.2.

7.7.6.2 Calculation of the stress range (f^*)

The stress range (f^*) in the weld to the shear connector is calculated as follows:

1 Calculate the longitudinal shear force per unit length (v^*_L) (clause 4.8), using the fatigue loading (clause 6.9 of *AS 5100.2* as modified by Clifton (2007)), at supports, mid-span and points of contraflexure. Further guidance on the fatigue loading is given in section 7.11.

2 Take the longitudinal shear force as per metre length for the considered region and divide it by the number of shear connectors in a row (this can be one for a channel connector to up to four for headed studs or bolts) and the cross sectional area of the shear connector in contact with the girder. The result is the design stress range, in this case of shear stresses (f^*_s), of the weld to the shear connectors, given as MPa (N/mm^2). This expression is summarised below:

$$f_S^* = \frac{v_L^*}{nA_{sc}} \qquad \text{(Equation 7.25)}$$

Where:

v^*_L is the longitudinal shear force per unit length

n is the number of shear connectors

A_{sc} is the cross sectional area of the shear connector.

7.7.6.3 Determine the fatigue strength (f_f) for the given weld detail

1 Determine the weld detail for the specified shear connector. This is taken as detail category 80 for headed studs, detail category 80 for channel connectors (for $t \leq 12$mm) and detail category 100 for bolts.

2 Determine the number of fatigue stress cycle (n_{sc}), see section 7.11 for further details.

3 Determine the fatigue strength (f_f) for the given weld detail and number of fatigue stress cycle, which is plotted on the S-N curve for shear stress in clause 13.6.2 or use the expression given in clause 13.6.3 for headed studs.

4 If the stress range of the weld to the shear connector is less than or equal to the fatigue strength (f_f), then fatigue is satisfied. If not then, the shear connector will need to be redesigned, ie:

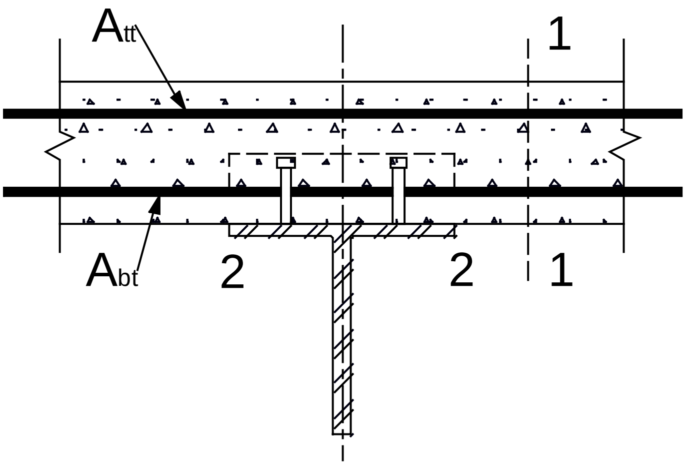
$$f_S^* \leq f_f \qquad \text{(Equation 7.26)}$$

7.7.7 Design of transverse reinforcement

The transverse reinforcement in the slab is required to transfer the longitudinal shear forces from the zone around the connectors to the rest of the slab. The requirements are expressed in clause 6.6.5. The longitudinal shear force is calculated in a similar manner to that for the shear connectors but the forces are evaluated on possible planes of failure, called shear planes.

An example of typical shear planes is illustrated in figure 7.17. For a shear plane around the studs, the design longitudinal shear force at ULS (note the difference that the capacity of the connectors is normally verified only at SLS) but for shear planes either side of the beam, the shear force additionally depends on the relative magnitude of the effective width of slab on that side. For a ladder deck bridge, most of the slab and thus the greater part of the shear force, is on one side.

Figure 7.17 Typical shear planes



Once the design longitudinal shear force is calculated for the particular shear plane, the design capacity of the transverse reinforcement must satisfy the criteria in clause 6.6.5.2 and, for certain shear planes, clause 6.6.5.3 must be satisfied. In both cases, guidance on the use of haunches and their design is given in these clauses. Note that, for cross girders in ladder decks, 'transverse' refers to the cross girder and thus the reinforcement is the longitudinal reinforcement in the slab; in hogging moment regions that reinforcement will be under significant tensile stress. Transverse reinforcement is required to provide the tensile resistance to global bending, local resistance to longitudinal bending and the transfer of longitudinal shear from the cross girder. Rules for interaction are, however, not given in *AS 5100.6* but it would be pragmatic to provide an area of reinforcement that is at least the sum of the areas needed to resist each of the effects separately.

Finally, clauses 6.6.5.4, 6.6.5.5 and 6.6.5.6 conclude the transverse reinforcement design by providing limiting values of minimum transverse reinforcement and its curtailment. This minimum transverse reinforcement shall also not be less that the value given in clause 9.3.9.4.15 of *NZS 3101*. This provision relates to the design of the deck slab, which is discussed in section 7.10 below.

See section 8.2.3 for guidance on detailing the deck slab in the region of the shear connectors.

7.8 Connections

Connections between steelwork components are made by welding or bolting (rivets are rarely used nowadays). Generally, welding is used for connections made in the fabrication shop (butt welds in flange plates, web/flange welds, connection at stiffeners etc) and bolting is used for connections on site (using cover plates, gusset plates, cleats etc). Welding is rarely used on site because of the extra control processes that have to be made available, the need for weather protection and the additional time involved, although for large projects it might prove beneficial.

The quantity of weld metal is small in comparison to the mass of the structural components but the cost of welding is comparable with that of the material. The splice plates and bolts are also small in mass relative to the structural members but the process of connecting them is a labour-intensive process that is a major part of the cost of erection. Economy is therefore achieved through simplicity, minimising the number of components and making the connection process as straightforward as possible. The designer must keep in mind the following considerations:

- Design for strength:

  - do not overdesign the connections
  - arrange the connections for direct force-transfer, with minimal eccentricities.
- Design for fatigue resistance:
  - avoid 'poor' fatigue details in highly stressed locations
  - avoid creating notches and other stress-raising details.
- Design for serviceability:
  - avoid details that might cause local yielding under working load
  - avoid details that would cause difficulty in of application of protection (and other) coatings
  - avoid features that would cause collection of water and debris.

7.8.1 Requirements for the design of connections

The requirements for the design of connections are given in section 12 of *AS 5100.6*. The general requirement stated in clause 12.1 is that connections shall be capable of transmitting the calculated design action effects. It may be noted that this means that a connection does not necessarily need to be as strong as the components that it connects. However, clause 12.3.1 sets out minimum values of design actions for connections, many of which are related to the resistance of the connected member (typically 50% or 75% of the nominal member capacity).

Detailed rules for the design of bolted and welded connections are given in clauses 12.4 to 12.6. Guidance on bolting and welded design procedures is available in Gorenc et al (2005); Clifton (1994); or SCNZ (2007); the first of these references refers to *AS 5100,* the second and third references refer to *NZS 3404.1* (2009) although the design procedures are similar in both standards. The following sections summarise the key aspects of the bolting and welding design procedures.

7.8.2 Bolting

Most bolted connections in bridges transfer forces between parts by means of bolt shear. Tension connectors are rarely used to transmit loads between primary components.

Clause 12.5.1 defines four types of bolting category as shown in table 7.3.

Table 7.3 Bolting category

Bolting category	Bolt grade	Method of tensioning	Means of load transfer
4.6/S	4.6	Snug-tight	Bearing/shear
8.8/S	8.8	Snug-tight	Bearing/shear
8.8/TB	8.8	Full tensioning	Bearing/shear
8.8/TF	8.8	Full tensioning	Friction at SLS Bearing/shear at ULS

Clause 12.3.2 states that all connections and splices in the main girder shall use bolting category 8.8/TF (also known as fully tensioned property class 8.8 bolt to *AS/NZS 1252* (SNZ 1996)). Although this would seem to permit the use of other bolting categories for connection of bracing members it is preferable to use only bolting category 8.8/TF bolts for all connections of structural members in bridges.

Bolts should be positioned in accordance with the limits given in clause 12.5.2.

Where weathering steel is used, it is common practice to design the connection for M24 bolts but to set out the bolt positions according to minimum spacing and edge distance values for 1 inch size bolts because the bolts are likely to be supplied from the USA in that size. Additional guidance on the use of weathering steel is found in El Sarraf and Clifton (2005).

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

The design resistance of a bolted connection is based on the resistances for individual bolts. Bolt capacity at ULS must satisfy the provisions in clause 12.5.3 and slip resistance at SLS must satisfy clause 12.5.4.

Bolting category 8.8/TF bolts are usually used in normal sized holes but the designer may wish to allow the use of oversize holes in some locations, for constructional reasons. If oversize holes are to be accepted, this needs to be recognised in design, since the design resistances are reduced and the minimum hole spacing is increased (because it is based on hole diameter).

Additional guidance on the design of a bolted splice is given in section 7.8.4 below. Note that property class 10.9 bolts are also available and can be considered in connections that require additional bolt capacity where space is limited for instance. However, this type of bolt is susceptible to hydrogen pickup, possibility leading to delayed brittle fracture. Specialist advice should be sought before specifying these bolts.

7.8.3 Welding

Welded connections are made using either fillet welds or butt welds and clause 12.6.1 states that all welding shall be carried out in accordance with *AS/NZS 1554* (SNZ 2004), specifically part 5.

The strength of a fillet weld depends on its throat thickness (see figure 12.6.2.4(a) of *AS 5100.6*) and for some welding processes this throat thickness can be increased by extending the fusion zone: this is then referred to as a deep penetration fillet weld (see figure 12.6.2.4(b) of *AS 5100.6*). Butt welds are categorised as either complete penetration butt welds or incomplete penetration butt welds, depending on whether the fusion zone extends through the complete or only partial depth of the joint. Clause 12.6.3.1 only permits incomplete penetration welds for longitudinal welds such as the web/flange welds. All welds shall be designed to clause 12.6.7 and shall comply with the requirements of *AS/NZS 3679.2* (SNZ 2010b), especially table 3 of that standard.

The inspection and testing of welds is an important aspect of quality control in fabrication; *see NZS 3404.1* (SNZ 2009) for further advice.

7.8.4 Splices in main girders

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

7.8.4.1 Bolted splices

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

In a bolted splice, a key design task is to determine the distribution of forces between the individual bolts. For each flange, the number of bolts in the connection can be determined on the basis of the flange force, calculated on the basis of an elastic stress distribution in the girder section. Therefore, at ULS, the resistance of the bolt group is the sum of the resistances of all the fasteners.

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

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

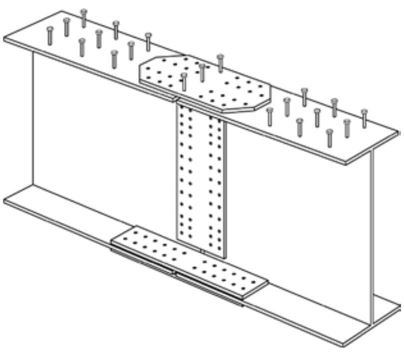
The force on the outermost bolt determines the design of the group. In this case, at ULS the bearing resistance may govern, rather than slip resistance, because webs and their cover plates are usually thin.

In the flange and web cover plates, stresses should be checked where the plate is in tension or where the holes are oversize or slotted, allowing for holes for fasteners in determining net sections (see clauses 12.3.6 and 12.4).

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

For further advice on the design of bolted connections, see Gorenc et al (2005), Clifton (1994) or SCNZ (2007). A typical bolted splice is shown in figure 7.18.

Figure 7.18 Typical bolted splice



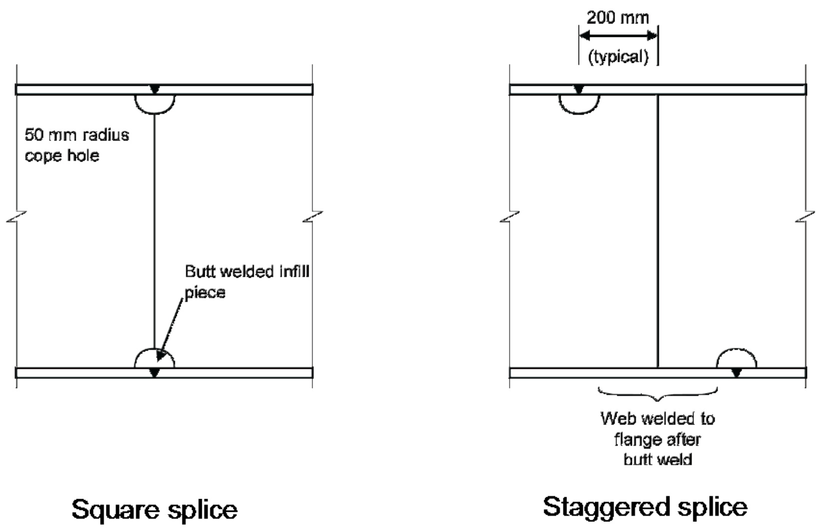
7.8.5 Welded splices

Welded splices in girders usually involve full penetration butt welds in webs and flanges. As noted above, full penetration welds should always be used: this makes the splice capable of the same bending resistance as the girder section.

The butt weld in the web is often staggered relative to that in the bottom flange, to assist in locating the web during erection (the web can sit on the projecting length of flange). A semi-circular cope hole is usually provided in the web above the flange weld, to facilitate the butt welding of the flange. Where the splice is not staggered, the cope hole is usually filled after the splice has been welded, to avoid the stress concentration (around the open hole) at the end of the butt weld in the web.

Details are shown in figure 7.19. Where the cope hole does not have the termination of a butt weld it may be left open.

Figure 7.19 Arrangement at welded splice



7.8.6 Cross girder end connection

7.8.6.1 Design basis of intermediate cross girders

During construction, there will be very little end moment on the cross girder due to gravity loads (ie there is very little end fixity from the main girder) and the single lap connection is assumed to transmit the vertical shear and a sagging moment equal to the shear multiplied by the distance from the web to the centroid of the bolt group.

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

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

The bolted connections provide restraint to the main girder against lateral-torsional buckling in regions adjacent to the intermediate support and can be designed for bolting categories 8.8/TB using the ULS restraining forces. This is also applicable to the mid-span regions.

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

See section 9.2 for further details.

7.8.6.2 Design basis of pier diaphragms

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

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

The use of a stub flange, with bolts in the cover plates at maximum distance from the slab, is more effective in transmitting moment; the stub flange then has to be designed to transfer the forces into the stiffener. See section 9.2.2 for further details.

7.9 Stiffeners

Web stiffeners are required to improve the shear resistance of the web, for the attachment of transverse bracing or cross girders and as support over bearings. In some cases, they can also be used to improve the compression resistance of a slender web, as well as, the effectiveness of the cross section. Three different types of web stiffeners are covered in *AS 5100.6*, they are:

- load-bearing stiffeners
- intermediate transverse web stiffeners
- longitudinal web stiffeners

The main considerations in the design of web stiffeners, which must be satisfied, are discussed below. Further detailed guidance on the design of stiffeners is available in Gorenc et al (2005) and Clifton (1994).

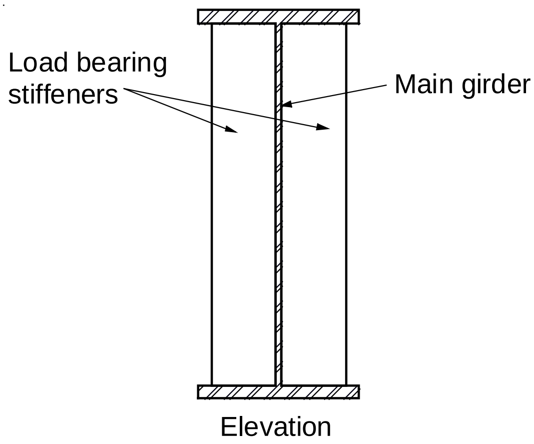
7.9.1 Load-bearing stiffeners

Clause 5.13 gives the requirements for the design of load-bearing web stiffeners.

Load-bearing stiffeners are used to strengthen the web to carry concentrated forces such as the reactions at supports. Bearing stiffeners are usually flat plates provided on both faces of a web, and are usually positioned symmetrically about the web, so that the centroid of the effective section is on the line of the web. In some cases, this would give resistance to bending about a transverse axis while allowing a single lapped connection to transverse bracing. See figure 7.20 for typical details. The ends of the stiffeners are normally fitted to bear against the flange where the concentrated load is introduced and fillet welded to the web and flanges.

Clause 5.13.4 requires that the stiffeners 'shall be provided with sufficient welds [or bolts] to transmit the entire bearing force or design reaction (R^*) to the web'. This requirement does not preclude the transmission of (part or all of) the reaction to the end of the stiffener in bearing and it is common to design that connection for direct bearing at ULS, when the stiffener has been fitted. However, it should be noted that for fatigue design the variation in reaction due to the passage of the fatigue load should be assumed to be transmitted through the welds, not in bearing. Note that clause 5.13.4, despite its heading, does not make any statement about the fitting (as opposed to attachment) of the stiffeners.

Figure 7.20 Typical bearing stiffeners



The stiffener outstands from the face of the web (b_{es}) need to comply with the geometric limit given in clause 5.13.3.

$$b_{es} \leq \frac{15t_s}{\sqrt{\frac{f_{ys}}{250}}} \quad \text{(Equation 7.27)}$$

Where:

b_{es} is the stiffener outstand from the face of the web

t_s is the flat stiffener thickness

f_{ys} is the design yield stress of the flat stiffener without the outer edge continuously stiffened (eg the stiffener is from a flat bar or plate and not an angle section).

Load bearing stiffeners must be designed at ULS for yield capacity (clause 5.13.1) and buckling capacity (clause 5.13.2); the requirements are expressed respectively as:

- yield capacity:

$$R^* \leq \emptyset R_{sy} \quad \text{(Equation 7.28)}$$

- buckling capacity:

$$R^* \leq \emptyset R_{sb} \quad \text{(Equation 7.29)}$$

Where:

R^* is the design bearing force or design reaction, including the effects of any shear forces applied directly to the stiffener.

ϕ is the capacity reduction factor (given in table 3.2 of *AS 5100.6*)

R_{sy} is the nominal yield capacity of the stiffened web, given in equation 5.13.1(2) of *AS 5100.6*.

R_{sb} is the nominal buckling capacity of the stiffened web and stiffener.

The calculation of the nominal buckling capacity of the stiffened web and stiffener (R_{sb}), given in clause 5.13.2 requires further guidance. The recommended procedure is:

1 Determine the effective section (A_s) of the compression member is taken as the stiffener area plus an effective length of the web, taken as the lesser of:

$$\frac{17.5t_w}{\sqrt{\frac{fy}{250}}}\ or\ \frac{s}{2} \qquad \text{(Equation 7.30)}$$

Where:

t_w is the web thickness.

s is the web panel width or spacing to the next web stiffener, if present.

2 Determine the effective length (L_e) of the compression member, is given in equation 5.13.2(2) and (3), see below:

a where the flanges are restrained by other structural elements against rotation in the plane of the stiffener, or

$$L_e = 0.7d_1 \qquad \text{(Equation 7.31)}$$

if either of the flanges is not so restrained

$$L_e = d_1 \qquad \text{(Equation 7.32)}$$

Where:

d_1 = depth between flanges.

3 Calculate the compression member slenderness reduction factor (α_c) given in clause 10.3.3, taking α_b as 0.5 and k_f as 1.0. Note that the radius of gyration (r_s) is taken as:

$$r_s = \sqrt{\frac{I_s}{A_s}} \qquad \text{(Equation 7.33)}$$

Where:

I_s = the web stiffener second moment of area about the axis parallel to the web.

4 Calculate the nominal buckling capacity of the stiffened web and stiffener (R_{sb}) for the parameters given in clause 5.13.2:

$$R_{sb} = \propto_c A_s f_y \qquad \text{(Equation 7.34)}$$

The requirement in clause 5.13.5 for when load-bearing stiffeners are the sole means of providing torsional end restraint at the support(s) only applies in circumstances such as half-through bridges. For the forms of bridge described in this guide, bracing would normally be provided at supports even during the bare steel construction stage.

7.9.2 Intermediate transverse web stiffeners

Clause 5.14 gives the requirements for the design of intermediate transverse web stiffeners, which are used to increase the buckling resistance of the web and for the attachment of bracing between beams.

Intermediate transverse web stiffeners are usually flat plates fillet-welded to one face of the web and to one or both flanges. For deep slender webs angle stiffeners, with one leg outstanding, are sometimes used, for greater stiffness.

Generally, on the outermost beams, the stiffeners should be on the hidden face, rather than the exposed face, for better appearance; any stiffeners to which bracing are attached would be on the inner face in any case. To restrain the web against buckling, the stiffener does not need to be connected to either flange;

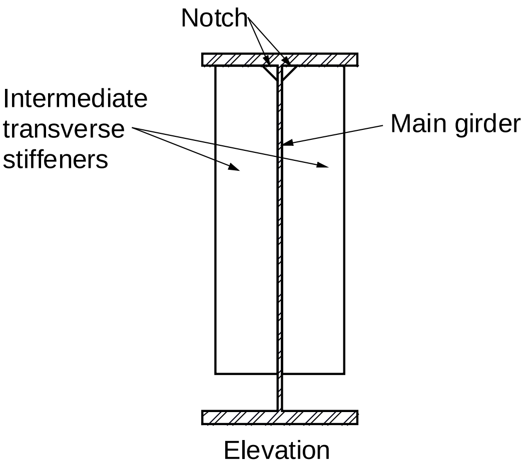
although it is usual to connect it to the top flange (it avoids potential fatigue at the flange/web weld due to transverse flexure of the slab). At the bottom, the stiffener can be stopped short (subject to a maximum clearance of four times the web thickness, according to clause 5.14.1); this simplifies fabrication and is thus more economic.

Where the stiffener acts as a connection for transverse bracing, it should be connected to both flanges. Practical experience indicates that failure to provide such attachment may lead to fatigue cracking in the web at the point of curtailment of the stiffener.

Where bracing is attached to a web stiffener, the stiffener may need to be shaped to provide sufficient lap to connect the bracing members, see section 9.2 for further details.

The fillet welding of the end of a stiffener to a flange does not introduce a lower class of fatigue detail than is likely to be present already, provided that the toe of the weld is at least 10mm from the edge of the flange. Web stiffeners should be proportioned such that they are narrower than the flange outstand; shaped stiffeners for bracing may need to be notched at the end (see figure 7.21) to ensure that the welds are not too close to the edge of the web.

Figure 7.21 Example of an intermediate transverse web stiffener with a notched end



The outstands of intermediate transverse web stiffeners need to comply with the same geometric limit as load-bearing stiffeners (see clause 5.14.2) and to have a minimum area (clause 5.14.4), given by:

$$A_s \geq 0.5\gamma A_w(1-\alpha_v)\left(\frac{V^*}{\emptyset V_u}\right)\left[\left(\frac{s}{d_p}\right) - \frac{\left(\frac{s}{d_p}\right)^2}{\sqrt{1+\left(\frac{s}{d_p}\right)^2}}\right] \quad \text{(Equation 7.35)}$$

Where:

A_s is the area of stiffener,

γ is the factor for transverse stiffener arrangement, taken as:

= 1.0 for a pair of stiffeners

= 1.8 for a single angle stiffener

= 2.4 for a single plate stiffener

A_w is the area of web

α_v is the shear buckling coefficient determined in accordance with clause 5.10.5.2

Stiffeners are also required to have a minimum stiffness, given by clause 5.14.5 as:

$$I_s \geq 0.75 d_1 t_w^3 \quad for\ \frac{s}{d_1} \leq \sqrt{2}$$ (Equation 7.36)

$$I_s \geq \frac{1.5 d_1 t_w^3}{s^2} \quad for\ \frac{s}{d_1} > \sqrt{2}$$ (Equation 7.37)

At ULS, stiffeners not attached to bracing are required only to have a buckling capacity greater or equal to that calculated to clause 5.14.4 as:

$$V^* \leq \emptyset(R_{sb} + V_b)$$ (Equation 7.38)

Where

ϕ is the capacity reduction factor (see table 3.2 of *AS 5100.6*.

R_{sb} is the nominal buckling capacity of the web and intermediate stiffener determined in accordance with clause 5.13.2.

V_b is the nominal shear buckling capacity specified in clause 5.10.5.2 for a stiffened web using α_d = 1.0 and α_l = 1.0.

Additionally, clause 5.14.8 states that the web connections of intermediate transverse stiffeners not subject to external loading shall be designed to resist a minimum design shear force per unit length (kN/mm), of not less than:

$$\frac{0.0008 t_w^2 f_y}{b_{es}}$$ (Equation 7.39)

Stiffeners subject to external loads or moments are also required to be designed for Increase in strength (clause 5.14.7.2) and increase in stiffness, given by clause 5.14.7.1 as a minimum value of the second moment of area (I_s):

$$I_s = d_1^3 \frac{\left(2F_n^* + \frac{\left(M^* + F_p^* e\right)}{d_1}\right)}{\emptyset E t_w}$$ (Equation 7.40)

Where:

F^*_p is the transfer design forces

e is the distance between the end plate and load-bearing stiffener.

It should be noted that due to the nature of loading on intermediate transverse stiffeners, the yield capacity checks are not required.

7.9.3 Longitudinal stiffeners

Clause 5.15 outlines the design requirements for the design of longitudinal stiffeners. These stiffeners are sometimes used in very deep girders, as they have a significant effect on the compressive buckling of a slender web.

For best results, longitudinal stiffeners should be placed at a distance of about 0.2d from the compression flange; clause 5.15.2 refers only to stiffeners at this location and at the neutral axis. Clause 5.15.1 requires these stiffeners to be continuous or extend between and be attached to transverse stiffeners. Longitudinal stiffeners at 0.2d are required to satisfy the following stiffness requirement only:

- minimum stiffness (clause 5.15.2)

$$I_s \geq 4 d_2 t_w^3 \left[1 + \frac{4A_s}{d_2 t_w}\left(1 + \frac{A_s}{d_2 t_w}\right)\right]$$ (Equation 7.41)

Where:

A_s is the area of the stiffener

d_2 is twice the clear distance from the neutral axis to the compression flange area of the stiffener.

Clause 5.15.2 states that, when a second longitudinal stiffener is required at the neutral axis of the section, it shall have a second moment of area (I_s) about the face of the web such that:

$$I_s \geq d_2 t_w^3 \qquad \text{(Equation 7.42)}$$

7.10 Deck slab design

The design of the reinforced concrete deck slab is carried out in accordance with *NZS 3101*, rather than with *AS 5100.5* (SA 2004), as discussed in sections 5.3 and 6.3. In this section, references to specific clauses are to *NZS 3101*, unless noted otherwise.

Clause 12.8 sets out two acceptable methods of design for reinforced concrete bridge decks:

- empirical design, based on membrane action
- elastic plate bending analysis.

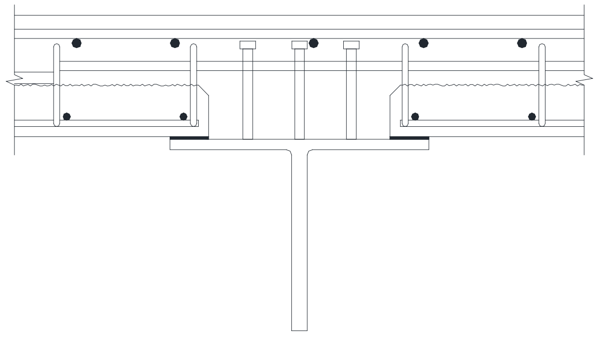
Due to the stringent acceptance criteria outlined in clause 12.8.2.2, the use of empirical design is not commonly used. Therefore, the second method is normally adopted and is the only method discussed here.

As noted in chapter 6, the slab participates with the main girders and (in ladder decks) with cross girders and the effects due to this composite action are revealed by the global analysis. A separate local analysis is often used to determine the local effects in the slab due to the individual wheel loads. The slab must then be verified for the different loading combinations (see section 6.4). Although the slab is predominantly one-way spanning for global effects, it is two-way spanning for local effects and the rules in section 12 of *NZS 3101* apply (with appropriate references to other clauses for detailing requirements).

7.10.1 Slab reinforcement

It is usual to provide two orthogonal layers of reinforcement on each face of the deck slab over the whole area of the deck. For a slab cast wholly in situ, one layer is usually laid on the bottom; for a deck cast with partial depth precast units, some of the bars are within the units and the arrangement is shown in figure 7.22. Further information is given in section 9.4.

Figure 7.22 Partial depth precast units reinforcement details



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

In a ladder deck bridge, the greatest bending moments in the slab due to traffic loading occur in the longitudinal direction: both hogging and sagging moments are generated. Transverse moments (due to the local effect of wheel loads) are smaller and mainly sagging in nature – the exception is over the length of the cantilever and immediately inboard of the main girders. Consequently, the longitudinal reinforcement is considered as the principal reinforcement and usually placed as the outer layer. However, the reduced lever arm for transverse bending resistance is a disadvantage at the root of the cantilever but the penalty is normally accepted.

7.10.2 Design for flexure

The basic requirement for design for flexure in slabs is given in clause 7.4.1:

$$M^* \leq \emptyset M_n \qquad \text{(Equation 7.43)}$$

Where:

M^* is the design bending moment

M_n is the nominal flexural strength of that section of the slab

ϕ is the capacity factor, given by clause 2.3.2.2 of *NZS 3101*.

It should be noted that the design for flexure for two-way slab, is according to the requirements outlined in section 9 of *NZS 3101* and clause 12.5.1. The main difference in the two outlined design methods is in the shear design of the slab, which is given in section 7.10.3 below.

Using one of the analysis models described in section 6.3, such as the commonly used 2D grillage, the slab and reinforcement are designed as a beam as well. In this case, the slab, or the idealised beam, can be designed as a one-way slab in accordance with the provisions of section 9 of *NZS 3101*. To take advantage of the larger lever arm, the layer of reinforcement that resists the largest local moment is placed as the outer layer, while the inner layer then resists the lesser orthogonal moment. The positioning of the principal reinforcement is dependent on the type of bridge being used, which is clarified below.

In either case, the following design requirements must be satisfied when designing for flexure:

- Clauses 9.3.6.2 and 9.3.8.1 require that all flexural tension reinforcement shall be well distributed across the zone of maximum tension and there is sufficient reinforcement to meet crack control requirements. The amount and distribution of longitudinal reinforcement shall be such that, at every section, the distance of the extreme compression fibre to the neutral axis is less than 0.75 of the corresponding distance for balanced strain conditions (c_b). Designers should also note the provisions of clause 6.1.4 of *AS 5100.6* on the minimum longitudinal reinforcement (this provides the minimum requirements in the sagging moment regions but it is common practice to provide for this minimum requirement in the hogging moment region also).
- Clause 9.3.8.2.1, outlines the minimum reinforcement area (A_s) needed in the idealised beam (ie slab), which shall be greater than that given in the expression below:

  $$A_s > \frac{\sqrt{f_c'}}{4f_y} b_w d \qquad \text{(Equation 7.44)}$$

  But equal or greater than:

$$\frac{1.4 b_w d}{f_y} \quad \text{(Equation 7.45)}$$

- Clause 9.3.8.3 outlines the reinforcement spacing in the slab. For the principal reinforcement (as outlined below for the different bridge types) the spacing is given as not exceeding the lesser of twice the slab thickness or 300mm. For reinforcement perpendicular to the principal reinforcement the spacing shall not exceed the lesser of three times the slab thickness or 300mm.
- Clauses 9.3.8.2.4 and 8.8.1 specify that distribution reinforcement shall satisfy the requirements for temperature and shrinkage but with a ratio of reinforcement area to gross cross sectional area of 0.7/fy but equal to or greater than 0.0014.

When designing deck slabs that utilise partial depth precast decking (taken as 100mm thickness), the transverse reinforcement can be conservatively designed as being located in the remaining 150mm concrete topping. When designing for the longitudinal reinforcement, the total deck thickness of 250mm is used. Note that all such slabs composing of partial depth precast decking with an in situ topping shall be designed in accordance with section 18 of *NZS 3101*, to ensure composite action between the components through detailing the appropriate reinforcement.

Positioning of the reinforcement in the slab is dependent on the specified cover thickness to the outer layer, in accordance with section 3 of *NZS 3101*. Specifying the correct concrete cover is crucial to ensuring the performance of the slab over the design life of the bridge structure. The cover is dependent on the exposure classification according to its location (given in table 3.1 of *NZS 3101*) and the compressive strength of the concrete. Table 3.7 of *NZS 3101* provides the different concrete covers based on these factors. Bridge deck slabs are usually specified with 40MPa concrete that has a minimum cover ranging between 35mm and 50mm for the different exposure classifications of inland (A2), coastal perimeter (B1) and coastal frontage (B2) respectively. Note that the correct curing of the concrete is another important factor in ensuring the performance of the concrete member. See section 8.1 for further details.

7.10.2.1 Slabs in multi-girder decks

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

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

7.10.2.2 Slabs in ladder decks

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

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

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

Slabs in ladder deck bridges are designed as two-way slabs, due to the above moment actions and the punching shear requirements. This is clarified in section 7.10.3 below.

7.10.2.3 Effect of the axial load

It was noted that the effect of axial loads on the bending resistance of the composite section is not covered in either *NZS 3101* or *AS 5100*.

Where the slab is in tension, ie in the hogging moment region, then additional reinforcement may be required at the top of the slab that is designed to carry the tensile force acting on the slab at that point (See clause 6.1.4 of *AS 5100.6* for additional guidance on minimum reinforcement). When the slab is in compression, ie in the sagging moment region, the compression forces are typically resisted by the concrete which is assumed to be fully effective as stated in clause 7.4.2.1 of *NZS 3101*.

However, where a ladder deck slab is in axial compression, a question then arises about its slenderness, over the lengths between cross girders. There is no guidance available on the maximum spacing between the main girders where the slab is considered to be fully effective in compression. Clause 7.4.2.1 of *NZS 3101* states that it can either be assumed to be fully effective or a complete stress-strain analysis must be undertaken. To address this issue, Iles (2010, section 6.3.2) states that if the spacing of the main girders does not exceed about 30 times the thickness of the slab, the slab may be considered as fully effective in compression (it acts as a plate supported on four sides). For a wider spacing of the main girders, the slab tends to act as a wide slender strut in compression and its slenderness reduces its axial resistance, because second order effects are introduced. However, the slab is not usually fully utilised in compression and the reduction is acceptable.

7.10.3 Design for shear

The basic requirement for design for flexure in slabs is given in clause 7.5:

$$V^* \leq \emptyset V_n \qquad \text{(Equation 7.46)}$$

Where the design shear action (V^*) derived from the ULS loads is less than or equal to the factored nominal shear strength (V_n) of that section of the slab.

The requirements for the design for shear of two-way slabs are given in clause 12.7.2. In that clause it is noted that the shear strength is governed by the more severe of beam action and two-way action: for a deck slab supporting wheel loads, the two-way action will be more severe, due to the concentrated nature of the loading. However, it is recommended that both cases are considered when designing for shear, as the slab shear capacity is reduced when the slab is in tension, which may affect the slab overall capacity.

Clause 12.7.2 refers to clause 12.7.4 for determination of shear strength and notes that this is based on the general requirements in clause 7.5.3; the basic requirement is expressed as:

$$V_n = V_s + V_c \qquad \text{(Equation 7.47)}$$

Where:

V_n = nominal shear strength of the section

V_s = nominal shear strength of shear reinforcement

V_c = nominal shear strength from the concrete.

The values of the minimum shear strength of shear reinforcing and concrete are given in the accompanying clauses 12.7.3 and 12.7.4.

Clause 12.7.3.4 limits the maximum nominal shear stress for punching shear on any part of the perimeter to $0.5\sqrt{f_c'}$. When the nominal shear stress (v_n) on any part of the critical perimeter exceeds the critical shear stress (v_c), then (v_c) is reduced to $\frac{\sqrt{f_c'}}{6}$ around the complete perimeter according to clause 12.7.3.5.

Where the applied shear stress exceeds that provided by the concrete, shear reinforcement in the form of anchored bars, wires, single or multiple leg stirrups can be provided in slabs in accordance with clause 12.7.4.1, where the effective depth of the slab is greater than or equal to:

- 150mm and
- 16 times the diameter of the shear reinforcement.

An additional and important requirement in New Zealand is related to the minimum area of shear reinforcement that is given in clause 9.3.9.4.13. This clause states that when highly repetitive loads (ie vehicle loading) are acting on the slab, shear reinforcement is required. The minimum area of shear reinforcement is given in clause 9.3.9.4.15, which is:

$$A_v = \frac{1}{16}\sqrt{f_c'}\frac{b_w s}{f_{yt}} \qquad \text{(Equation 7.48)}$$

The spacing limits for shear reinforcement are given in clause 9.3.9.4.12.

Note that in the hogging moment region, the deck slab is in longitudinal tension, which reduces the concrete shear capacity, thereby the nominal shear strength of concrete (V_c) is taken as zero.

7.11 Fatigue design

7.11.1 Introduction to fatigue

Fatigue is one of the most important design checks that must be undertaken when designing a steel-concrete composite bridge. Fatigue cracks grow from very tiny imperfections when there is a fluctuation of stress across the imperfection that tends to open it. However, this does not mean that only an overall tensile stress in the part is relevant to crack growth, because any stress range will tend to open and close the initiating imperfection; the reference stress level for classification is therefore the stress range, which in this case is taken as the maximum peak-to-peak stress level due to the application of the cyclic load.

Fatigue failure of steel depends on the propagation of these imperfections (usually in the welds) in regions that are subject to fluctuating stress. The fatigue life therefore depends on the detail category (which accounts for the size of the initial imperfection and, to some extent, the local stress concentration) and on the range of the stress variation (number and magnitude of cycles).

Fatigue assessment is therefore a two-stage process:

1 Determination of the fatigue design spectrum – the number and magnitude of the cycles of stress variation as a result of the frequent loading on the bridge.

2 Determination of the fatigue life of particular details for the given fatigue spectrum.

7.11.2 Fatigue design spectrum

The basic requirements for determining the design spectrum are given in clause 13.3. It requires that stresses are calculated using an elastic analysis of the structure (which implicitly requires the effects of shear lag to be taken into account). Clause 13.3.2 states that the stress spectrum shall be obtained by a stress cycle counting method.

In general, a bridge will be subject to numerous different stress ranges, each with its number of cycles over the design life. Fatigue assessment would then need to take account of the damage caused by each range to obtain a cumulative assessment for the entire loading. This is a rather time consuming and difficult exercise and for that reason a simplified fatigue loading model is normally used. The choice of a suitable loading model for New Zealand traffic is discussed in section 6.4.2. This simplified model is used to determine an equivalent constant stress range for a specified number of cycles.

7.11.3 Fatigue strength

Fatigue strength is conventionally expressed in relation to the endurance (number of cycles to failure) of a particular structural detail subject to constant amplitude stress. The relationship between fatigue strength and number of stress cycles is referred to as an S-N curve (plotted on a log-log scale); examples for normal stress and shear stress are given in clause 13.6 for a range of detail categories. The detail categories are discussed below. Note, however, that the fatigue strength for transverse fillet or butt welded connections needs to be reduced or 'corrected' when the connected plate thickness exceeds 25mm (see clause 13.1.7 and section 7.11.5).

7.11.4 Classification

Fatigue detail classifications relate to the potential imperfections at welds, holes or other discontinuities, and their relationship to the stress direction. The greater the imperfection, the lower the fatigue strength is for a given number of cycles. The complete range of classifications is shown in table 13.5.1 of *AS 5100.6*. This table shows a range of typical details in detail category classification group 1 to 4, which includes welds and shear connectors.

Generally, lower category details are introduced by making attachments to the steelwork component. Fatigue assessment therefore needs to be carried out chiefly in regions of significant variations of stress and at the locations of the attachments. Typical locations requiring detailed assessment are: webs and flanges over internal supports; at the attachment of web stiffeners, flange or web reinforcing plates; all connections in transverse bracing and at splices (welded or bolted).

The attachment of web stiffeners or other elements not carrying load in the stressed direction usually produces a category 71 or 80 detail. Reinforcing plates and bearing plates welded to the underside of the flange usually introduce a category 45 or 50 detail. Shear stud connectors introduce a category 80 detail in the plate to which they are attached (see section 7.7.6 for further details). Preloaded bolted splices introduce category 140 details for double covers or single covers. Transverse butt welds create category 50, 80, 90 or 112 details and a size effect reduction factor applies as specified in table 13.5.1 of *AS 5100.6*.

The reliability of the classification of a particular detail depends on a presumption about the quality of workmanship. To ensure that the workmanship will be appropriate to the fatigue life and detail class assumed by the designer, clause 13.1.5 states that the weld details given in table 13.5.1(B) and 13.5.1(D) for detail category 112 shall conform with category SP (structural purpose) to *AS/NZS 1554.1* (SNZ 2004c). While the weld details in table 13.5.1(B) for detail category 125 shall have a weld quality conforming to that defined in *AS/NZS 1554.5* (SNZ 2004d). Workmanship levels, which are specified in *AS/NZS 1554* (SNZ 2004) and *NZS 3404.1* (SNZ 1997b), relate the level of inspection and the acceptance criteria for imperfections to the minimum fatigue class required by the designer.

7.11.5 Fatigue assessment

Clause 13.7 sets out how detail categories in a particular structure are to be assessed for acceptability for fatigue design. In making this assessment, the design fatigue capacity is determined by applying a capacity reduction factor (ϕ), as given by clause 13.1.6. Although the value of (ϕ), is generally to be taken as 1.0, for non-redundant load paths $\phi = 0.7$ is used. It should be noted that the non-redundant load path is taken as the path in a member where failure would lead to the instability resulting in partial or complete collapse of the structure.

Clause 13.7.1 gives an exemption from the need for detailed fatigue assessment for situations where the stress range is less than the corrected detail category fatigue strength at the constant amplitude fatigue limit. This is expressed by:

- for normal stresses:

$$f_n^* \leq \emptyset f_{5nc} \qquad \text{(Equation 7.49)}$$

- for shear stresses:

$$f_s^* \leq \emptyset f_{5sc} \qquad \text{(Equation 7.50)}$$

Where:

f^*_n = design stress range for normal stresses

f_{nc} = corrected detail category fatigue strength at constant amplitude fatigue limit for normal stresses

f^*_s = design stress range for shear stresses

f_{5sc} = corrected detail category fatigue strength at constant amplitude fatigue limit for shear stresses.

Clause 13.1.7 states that the thickness correction factor (β_{tf}) that is needed to determine (f_{5nc}) and (f_{5sc}) shall be taken as 1.0, except for a transverse fillet or butt-welded connection with a plate thickness (t_p) greater than 25mm. In that case the uncorrected fatigue strength for the different detail categories is multiplied by the following expression, given in equation 13.1.7(1) of *AS 5100.6*.

$$\beta_{tf} = \left(\frac{25}{t_p}\right)^{0.25} \qquad \text{(Equation 7.51)}$$

Clause 13.7.2 gives the requirements for assessing the detail category in the constant stress range, or when the design stress range (f^*_n) or (f^*_s), is calculated using the simplified fatigue loading (see section 6.4.2) and the effective number of fatigue cycles (n). Then the detail being assessed for normal and/or shear stresses is deemed to comply if it satisfies the expression given in this clause.

Clause 13.7.3 gives requirements for assessment for variable stress range. This method is used where the loading such as vehicle or wind loadings acting on a bridge are variable but the magnitude and frequencies of each range are known. The variable stress range, is based on the damage accumulation method (commonly known as Miner's summation), as the design stresses and their damage is accumulated when assessing whether the detail complies with the clause requirements. The variable stress range must satisfy the following expression:

$$\frac{\Sigma_i n_i (f_{in}^*)^3}{5\times10^6(\emptyset f_{3nc})^3} + \frac{\Sigma_j n_j \left(f_{jn}^*\right)^5}{5\times10^6(\emptyset f_{3nc})^5} \leq 1.0 \qquad \text{(Equation 7.52)}$$

To determine the variable stress range on a multilane bridge, assuming two heavy vehicles are acting on the bridge, the stress range (f^*) for each heavy vehicle is first determined as an independent load, as described in section 6.4.2. The stress range for each vehicle is then compared with the constant stress range fatigue limit (f_3) for the detail category (f_{rn}) being considered given in figure 13.6.1 of *AS 5100.6*. If the stress range for the vehicle is less than the constant stress range fatigue limit, then the left hand part of the equation is used. Conversely if the stress range of the vehicle is greater than the constant stress range then the right hand part of the equation is used. In addition to the above requirements, an accompanying lane factor shall be added to the second lane vehicle in accordance to clause 6.6 of

AS 5100.2. The summation of stresses for both vehicles must be less than or equal to (1.0) to satisfy this expression.

Clause 13.7.4 gives a more general version of the variable stress range method, expressed in a manner suited to assessment of existing structures or when the bridge design life is other than 100 years.

If designers find that fatigue assessment has an unacceptable outcome, they may choose to specify a better class of detail, to modify the design to reduce the stress range or, if the simplified model has been used, to re-assess by the variable stress method.

Finally, as briefly discussed in section 4.2.3, clause 8.4.2.2 of *NZS 3101* prohibits the use of bent reinforcement bars less than 20 times the bar diameter for concrete members subject to frequently repetitive loads. This is applicable to vehicle loads that expose the reinforcing bars to potentially high levels of fatigue. The fatigue design requirements and the permissible stress range for reinforcement bar is given in clause 2.5.2 of *NZS 3101*. While the permissible stress range for concrete is given in clause 2.5.2 of *AS 5100.5*.

7.12 Selection of steel sub-grade

The following guidance is based on section 2 of *NZS 3404.1* (SNZ 2009), as it contains New Zealand specific steel selection requirements, which include temperature and seismic requirements.

All parts of structural steelwork are required to have adequate notch toughness, to avoid the possibility of brittle fracture. Brittle fracture can initiate from a stress concentration when loading is applied suddenly, if the material is not sufficiently 'tough'. The degree of toughness required is expressed as a Charpy impact value (determined from a test carried out on a sample of material) and the requirement depends on the thickness of the material, its minimum temperature in service, seismic requirements, the stress level and rate of loading.

The design requirements for bridges are given in clause 2.2.5 and table 5 of *NZS 3404.1,* in terms of the steel type and its limiting thickness for the given conditions and specified material. If steel type and its thickness comply with the requirements, then it is considered to be sufficiently tough.

The most obvious condition that needs to be considered is the lowest temperature that the steel will experience. The basic design temperature is specified in clause 2.6.3 of *NZS 3404.1* as the lowest one-day mean ambient temperature (LODMAT), which depends on the bridge location around New Zealand. According to figure 1 of that standard, the lowest temperatures in New Zealand range from -10°C in the South Island to 7.5°C in Northland. However, for bridges, site-specific low temperatures are required, especially in the alpine areas in the South Island. In some cases the basic design temperature is reduced by 5ºC (see clause 2.6.3.2 of *NZS 3404.1*).

Clause 2.2.5 of *NZS 3404.1* gives the steel type chosen for bridges where they are fracture critical members (FCM). These are components in the bridge, the failure of which would result in the collapse of the bridge. FCM are specifically required for all railway bridges or highway bridges with significant overloads and with high fatigue loading.

In most cases, non-FCM bridges will commonly use steel type 4, either grade 350 to *AS/NZS 3679.1* (SNZ 2010a) or WR350 to *AS/NZS 3679.2* (SNZ 2010b). Alternatively, for FCM bridges steel type 5 will commonly be used, either grade 350L0 to *AS/NZS 3679.1* or WR350L0 to *AS/NZS 3679.2*, both of which have a Charpy impact value of 27J at 0°C.

Other steel grades with higher Charpy impact, such as L15 grade with 27J at -15°C or the seismic grade S0 with 70J at 0°C, are available. However, the seismic grade is rarely used in bridges, unless it is used in the substructure (ie piers).

Table 5 of *NZS 3404.1* gives the permissible service temperatures according to the steel type and thickness, which must be considered by the designers when specifying the steel type for the bridge.

8 Construction considerations

8.1 Steel fabrication and concrete construction

The design rules in *AS 5100.6* (SA 2004) are valid for steel components that are fabricated in accordance to *NZS 3404.1* (SNZ 2009), which includes specification of materials and of fabrication workmanship.

All concrete components that are designed to *NZS 3101* (SNZ 2006) shall be constructed in accordance to *NZS 3109* (1997a). Note that proper curing of the concrete is crucial to the structural and durability performance of the concrete member. Therefore, the designer should ensure that the contractor meets the minimum requirements listed in *NZS 3109*, which can be achieved by providing a financial incentive by stipulating the curing of the concrete as a separate item on the project contractual documents.

8.2 Construction sequence

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

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

The results of the analytical model for the chosen construction sequence will then be used to design the structural components for that construction stage, following the guidance given in chapter 7.

8.2.1 Girder erection

Girder lengths are usually chosen to suit transportation (see section 4.2.1 and *Guide to heavy vehicle management* (NZTA 2006)), although the weight of individual pieces may limit the sizes where crane access is restricted. Strength verification at this stage is unlikely to require detailed evaluation but stability and buckling resistance do require careful consideration, particularly before bracing or cross girders are fully installed.

8.2.2 Bracing

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

8.2.3 Slab construction

Although deck slabs have traditionally been cast on temporary timber false work, the use of permanent formwork notably partial depth precast decking that forms part of the final slab is now very common (section 2.3). Timber false work is often supported off the bottom flanges of the girders; precast permanent formwork sits on the top flanges and thus needs to be considered as a destabilising load. Whichever type of formwork, the weight at the wet concrete stage imposes quite high stresses in the top flanges of the girders. Their strength and stability at this stage require a detailed evaluation of the progressive changes in structural behaviour as load is added.

The weight of the concrete cantilevers needs particular attention, because of the moment (about the longitudinal axis) that is imposed on the outer girder. See further comment in section 8.3.1.

8.2.4 Patch loading on webs

For girders that are erected by launching, reactions under the girder as it is progressively launched impose local 'patch loading' on the unstiffened portions of the main girder webs. The web will need to be checked for the effects of combined stresses and for buckling. See section 7.4 for details of web design.

8.3 Cantilever edge slabs

8.3.1 Loading from cantilevers

The use of precast units, especially partial depth precast decking, is the current popular method of cantilever construction.

The moment due to weight of cantilevered false work and wet concrete is transferred to the main girder as a couple of horizontal forces at top and bottom flange levels; these forces cause horizontal bending of the flanges between restraint positions. This is in effect warping torsion, rather than St Venant torsion. In ladder deck bridges, the effects of warping are modest, because the cross girders provide restraint at close regular intervals; in multi girder decks the restraint positions are further apart and the effects are greater. Deflection at the restraint positions (due to the bending of cross girders or the vertical displacement of the main girders due to the eccentric moment) adds twisting effects. Warping stresses, distortional displacements and twists all need to be determined.

Although the warping stresses (transverse bending stresses) in the top flanges are locked in once the concrete hardens, it is not necessary to include these effects for the in-service condition because at ULS they will redistribute and at SLS any relaxation would be unlikely to lead to any noticeable permanent deformation.

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

8.3.2 Design of cantilever edge slabs

For multi-girder bridges, cantilever edge slabs are usually the last part of the deck slab to be concreted, in order to achieve a good alignment along this very visible feature. Their contribution to structural behaviour of the cantilevers cannot therefore be relied upon until a late stage during construction. The edge slabs of ladder deck bridges are usually concreted at the same time as the main deck slab.

8.4 Allowance for permanent deformations

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

A residual hogging profile is often specified, for aesthetic reasons, even when not needed to meet a clearance requirement at SLS. For the calculation of deflections of composite sections, it is necessary to assume an age at first loading, so that the appropriate parameters for concrete can be determined. The steelwork should normally be pre-cambered to offset the predicted deflection at the end of construction.

9 Detailing considerations

9.1 Geometric configurations

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

9.1.1 Multi-girder bridges

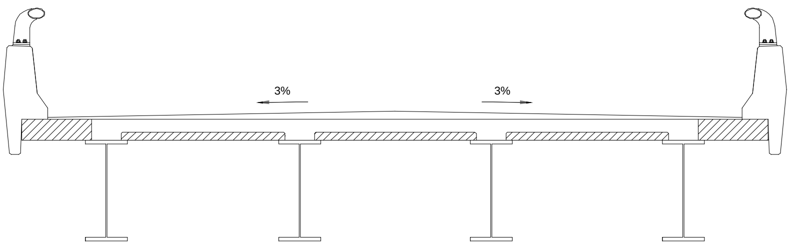
9.1.1.1 Road camber and crossfall

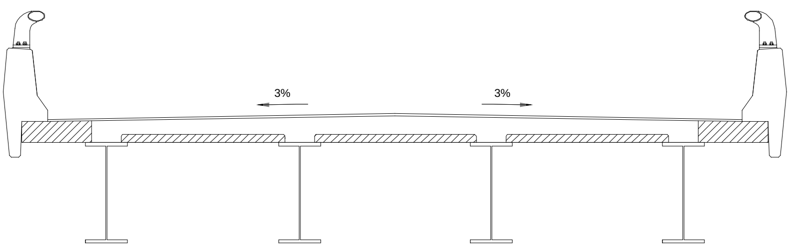
Usually the top flanges of the main girders are square to the vertical webs. The relationship with a deck slab that follows the road camber or a transverse crossfall then needs to be considered carefully. There are four main options:

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

The first and second options are only appropriate for two- or three-lane bridges with no super-elevation, where the weight penalty is modest. Variation of surfacing thickness is preferred to variation of slab thickness, for economy in construction. The first three options all suit the use of permanent formwork; option 3 is perhaps the most common and is the arrangement shown in figure 2.1. Options 1 and 2 are illustrated in figure 9.1.

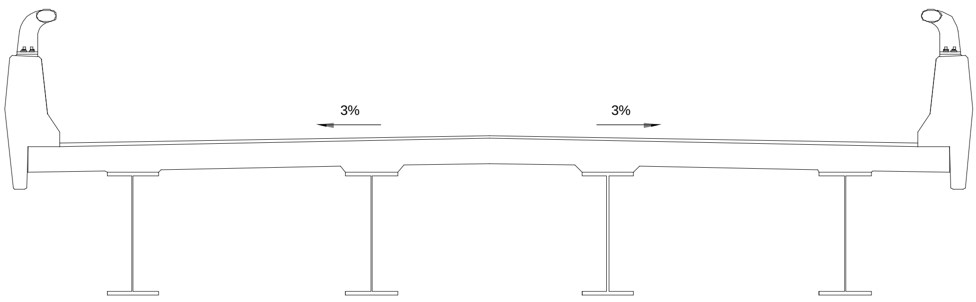
Figure 9.1 Two options for dealing with road camber in single carriageway multi-girder bridges





Option 4 is suited to the use of timber temporary formwork and, until the increased use of permanent formwork, was the most common arrangement. A typical arrangement for option 4 is shown in figure 9.2. The haunches can be formed relatively easily and the resulting slab is of uniform thickness. The use of haunches also makes it easier to accommodate any unintended differences in relative level between adjacent girders that are found after steelwork erection.

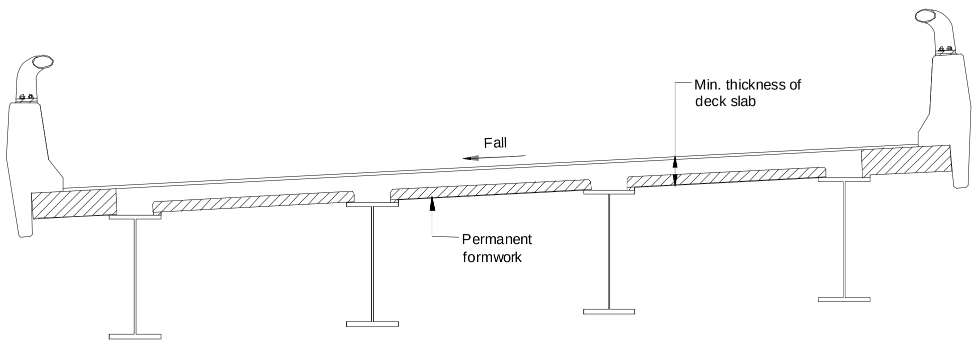
Figure 9.2 Use of haunches with a multi-girder slab constructed on temporary formwork

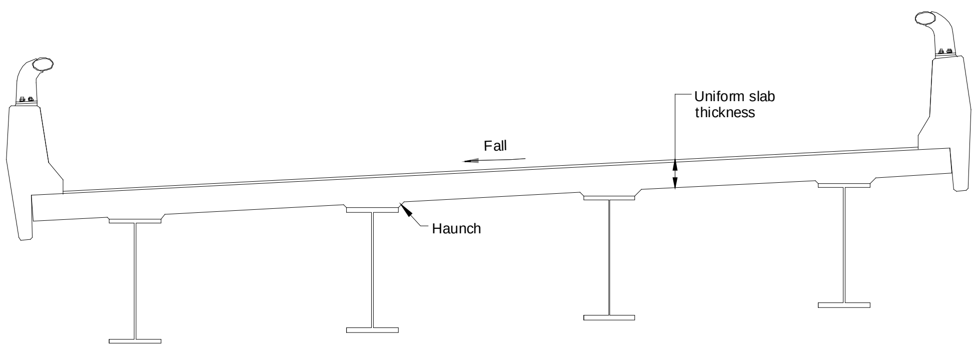


For all four options, designers usually choose the same girder depth for all girders. For options 3 and 4, the girders are at slightly different levels.

Where the deck is wider, or where there is super-elevation (uniform gradient across the full width of the carriageway), the arrangements are then similar to those illustrated in figure 9.3. See further comment on the effect of a varying slab thickness in section 9.4.

Figure 9.3 Two options for dealing with deck slab with super-elevation in multi-girder bridges





9.1.1.2 Bracing planes

Planes of bracing are usually square to the top flange, rather than vertical. As noted in section 2.1.3, the planes are usually square to the main girders in plan.

9.1.2 Ladder decks

9.1.2.1 Plan layout

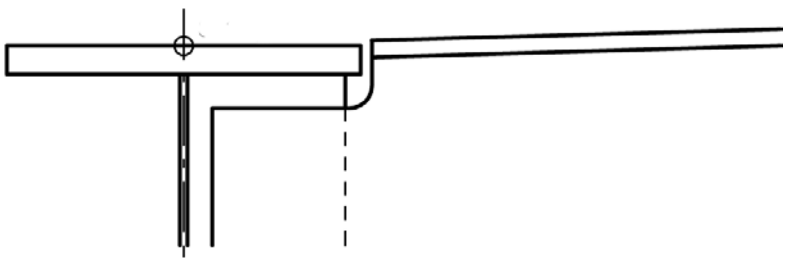
The position or spacing of cross girders should be defined in relation to their centrelines (ie the mid-thickness of the web). This is particularly important where a lapped connection is used, to avoid confusion if the position were defined to one face of the web or to the centreline of the stiffener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

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

9.1.2.2 Road camber, crossfall and longitudinal gradient

Transversely, the top flange of the cross girder will usually follow the camber of the roadway or the super-elevation of the roadway. The alignment of the flange in relation to the flanges of the main girders needs to be considered: if the main girder flanges are horizontal (across the bridge), variations in slab thickness and the consequences of any variation in width of main girder flange need to be taken into account. For the usual crossfall (3%), or a modest crossfall to provide super-elevation, the top flanges of the cross girders can be aligned as shown in figure 9.4; the small step at the edge of the main girder flange does not introduce construction difficulties, although care will be needed in sealing between permanent formwork and the main girders.

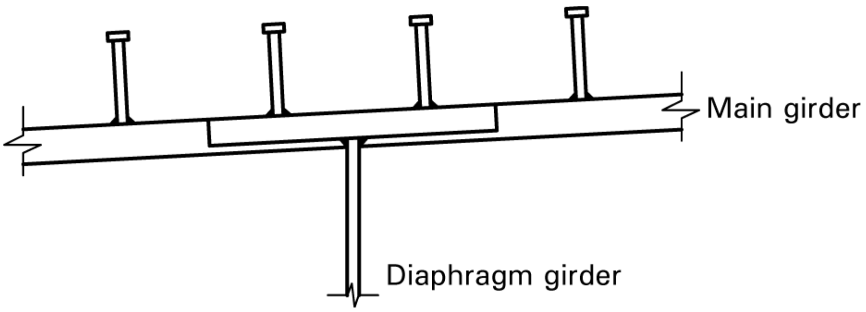
Figure 9.4 Alignment of flanges



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

There is no explicit requirement for bearing stiffeners at intermediate or end supports to be truly vertical; although designers usually prefer to detail them to be vertical under dead load (it is visually better for the bearing stiffeners on the outer faces to be vertical). Where there are integral crossheads, it is structurally better to make their webs vertical, to minimise the twisting effects from the bearing reaction. If a pier cross girder, diaphragm or integral crosshead is detailed with the web vertical, its top flange should still follow the longitudinal profile of the main girders and the flange will then be slightly tilted relative to the web, as shown in figure 9.5.

Figure 9.5 Alignment of diaphragm girder flange when its web is vertical



9.1.3 Allowances for permanent deformation

The steelwork is normally fabricated with allowances for permanent deformation due to the weight of the complete structure (see section 8.4) and for cutting and welding during fabrication. The fabricator can deal with these allowances in determining the shape of all the elements that are to be cut from steel plate, provided the designer advises the allowances for the intended construction sequence (see section 8.4). However, there are still some questions that arise with a composite structure, such as at what stage are the webs at supports to be truly vertical: under the weight of steelwork alone or after completion? Although designers might wish to select the latter option, it is well known that it is very difficult to predict rotations of main girders (about their longitudinal axes) at supports, particularly for skew bridges. It is therefore commonly arranged that the webs are fabricated to be vertical under the weight of bare steelwork and the girders are designed for the out-of plumb that would result under the weight of concrete and superimposed load (assuming that rotations occur as predicted).

Guidance on fabrication and erection tolerances is given in appendices G and H of *AS 5100.6* (SA 2004), which is similar to the guidance given in sections 3 and 4 of *NZS 3404.1* (SNZ 2009).

9.2 Cross girder end connections

9.2.1 Intermediate cross girders

Intermediate cross girders (ie away from supports) are connected to web stiffeners by simple lapping of the web plate onto the stiffener, as shown in figure 9.6. Both flanges are stopped short of the end of the web; this is easily achieved with a fabricated girder. Double cover plate splice connections are inappropriate because the same number of bolts is required as with a single lapped connection, and additional cover plates have to be fabricated, making the detail more expensive. However, if rolled sections are chosen for the cross girders, a double cover detail may be preferable (allowing the section to be cut with a plane end) because of the cost of the work that would otherwise be needed in the cutting back of flanges of a rolled section.

Figure 9.6 Lapped connection of intermediate cross girder

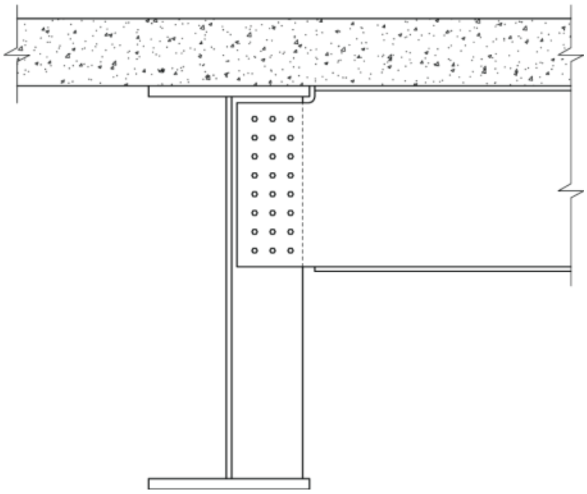
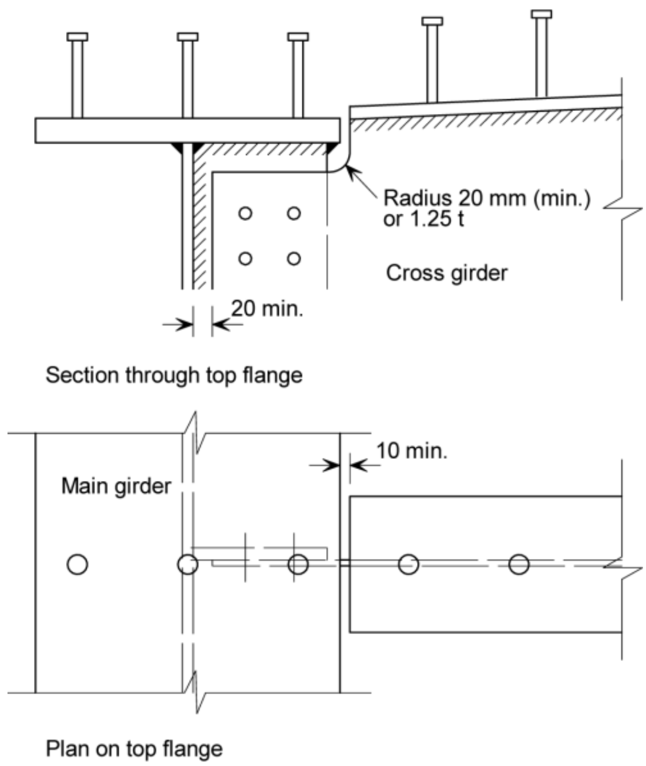


Figure 9.7 Details of lapped connection



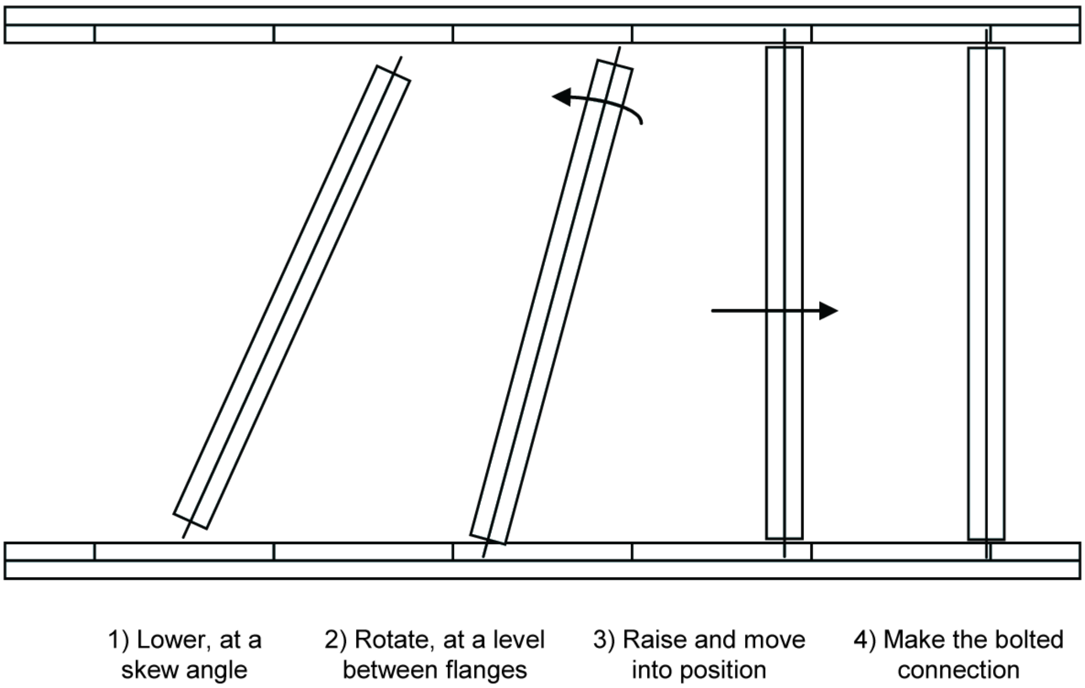
In most cases, this simple lap detail, with no connection of the flanges, will be sufficient to transmit the moments in the U-frame (see section 7.8.6). If the moments are greater than can be transmitted this way (perhaps because a very shallow cross girder is chosen) a connection of the bottom flange similar to that sometimes used for pier diaphragms (see figure 9.11) may be effective.

With a lap detail, the cross girders are longer (over the length of the web) than the clear gap between the main girder flanges. Consequently, during erection they are lifted at a skew (in plan) so that they can be lowered past the top flanges and then rotated and brought into lapping contact with the main girder stiffeners (see figure 9.8). Ideally, plan rotation as shown should be possible with both the adjacent cross girders in position. It may be necessary to keep the end of the cross girder web sufficiently clear of the face of the main girder web (and this may require a wider web stiffener) so that the cross girder can be erected in this manner even when the cross girders on both sides are already in place. The laps at the two ends of the cross girder should be to the same face of the web, to avoid the risks of confusion and error in setting out and installation.

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

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

Figure 9.8 Schematic arrangement for erecting intermediate cross girders



9.2.2 Pier diaphragms

Lap type connections cannot readily be made at intermediate supports as, once the main girders are in place at the required spacing, the pier diaphragm girder will foul on the jacking stiffeners as it is swung into place. Hence, cross girders framing into the bearing stiffeners will normally be connected using double cover plates to the web. See figures 9.9 and 9.10 for further details.

Figure 9.9 Spliced connection of pier diaphragm

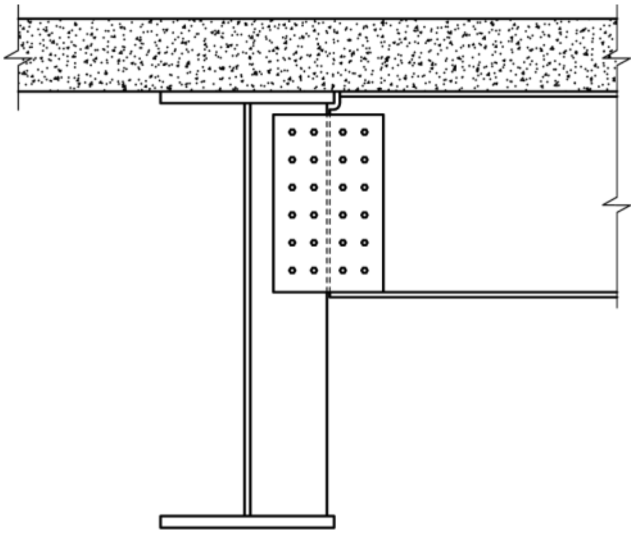
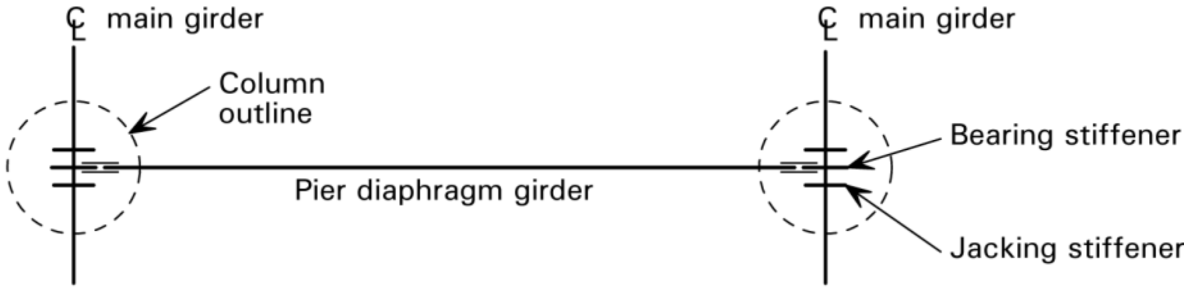


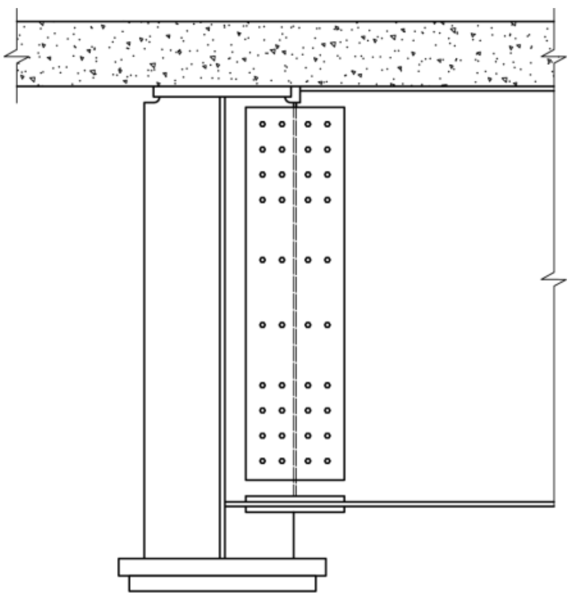
Figure 9.10 Arrangement of double lap splice connection of pier diaphragm



(Flanges omitted for clarity)

To transfer larger moments at supports, a cover plate connection to a ‘stub flange’ attached to the bearing stiffener may be needed (see figure 9.11). A double cover plate connection should be provided. Connection of the top flange should not be necessary. The cross girder should be less deep than the main girder, to avoid any need to connect to the main girder bottom flange.

Figure 9.11 Elevation on splice connection of pier diaphragm with stub flange

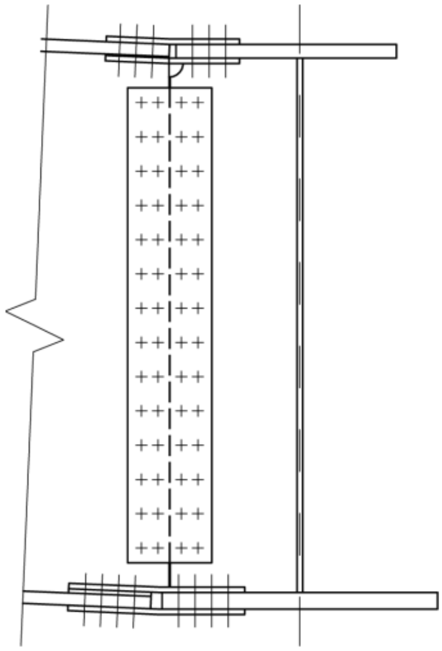


9.2.3 Integral cross heads

Where the bridge is supported on bearings under an integral crosshead, the load on the connection is clearly much greater and more bolts will be required. A typical connection arrangement is shown in figure 2.12 and some of the details are shown at larger scale in figure 9.12.

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

Figure 9.12 Connection of integral crosshead to main girder



9.2.4 Crosshead girders in multi-girder bridges

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

9.3 Shear connection

Even though clause 6.6 references three types of shear connectors (see section 7.7), the most commonly used shear connectors are the headed stud. Headed studs, also known as shear studs, are available as 15.9mm, 19.0mm and 22.2mm diameter, of which the 19mm diameter is the most commonly used. They are readily available and can easily be welded using a special semi-automatic welding tool. Stud dimensions and minimum spacing limits are given in clause 6.6.2.

Because studs are required to prevent separation, the undersides of the heads of the studs need to be a minimum distance above the bottom layer of reinforcement. Requirements for haunched and unhaunched configurations are shown in figure 6.6.5.2 of *AS 5100.6* (SA 2004).

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

The stud longitudinal spacing shall not be greater than the minimum of 600mm, three times the slab thickness or four times the height of the connector (which leads to the need for studs on cover plates on the top flange, as shown in figure 7.18). They should also not be closer than 25mm to the edge of a flange (50mm if the slab is haunched); larger edge distances are needed when precast permanent formwork is used, to ensure secure seating of units.

9.4 Deck slab

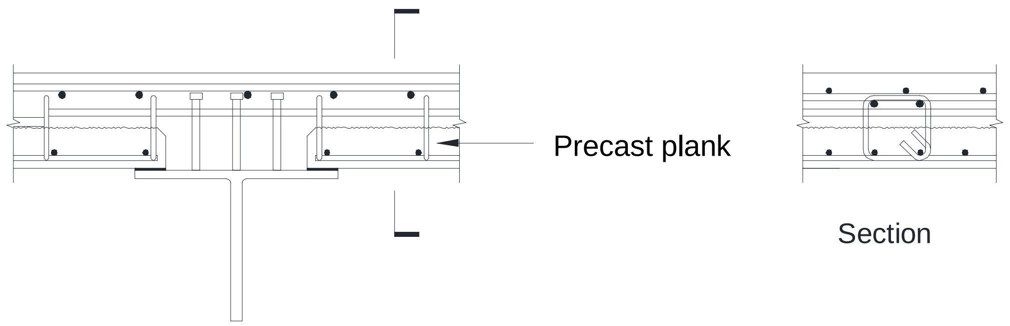
The detailing issues related to deck slabs concern chiefly the location of reinforcement.

9.4.1.1 Multi-girder decks

In multi-girder decks, the transverse reinforcement is normally placed as the outer layers and the longitudinal reinforcement is placed as the inner layers. Where precast permanent formwork is used between the main girders, only the in situ lower transverse rebars are effective in transferring shear to the main girders; the adequacy of these bars needs to be verified. The location of the upper longitudinal rebars in the inner layer suits fixing of reinforcement, as they can sit on the top of the protruding hoops of the precast units (although see note below about varying slab thickness).

Where permanent formwork is not horizontal, the thickness of the slab varies, as noted in figure 9.3. The arrangement of the transverse reinforcement at a main girder is illustrated in figure 9.13. It is not usual (or desirable for fatigue reasons) to crank the transverse bars in either the top or bottom mat, although they will bend a little. In the top, the bars may lift off slightly from the plank reinforcement on the 'higher' side and the cover may be slightly greater. The bottom bars may also be slightly higher at the same location.

Figure 9.13 Local cross section at main girder in multi-girder deck with precast permanent formwork



Permanent formwork needs to be constrained by the positions of the studs at each end so that it cannot displace and fall through, between the girders, during construction. The nominal bearing length for precast decking is typically 75mm and studs should be no more than 25mm inside the nominal position of the decking ends; this will ensure that the decking cannot be displaced along their length and then fall through. A seating rubber strip is recommended to take into account any height differences in the decking placement. The designer should also consider the positioning of the reinforcement and available flange width to ensure the above dimensions are met.

Any protective treatment to the steelwork should be continued inward from the top edges of the flanges to a minimum of 100mm. A sealant will also be required where permanent formwork is sloped, relative to the flange and in all cases where the girder is of weathering steel.

9.4.1.2 Ladder decks

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

Where precast permanent formwork is used, the transverse reinforcements in the main body of the slab are even higher. To achieve the necessary clearance below the head of the studs bars need to be cranked, or additional U-bars provided, see figures 9.14 and 9.15. Alternatively, taller studs can be used.

Figure 9.14 Cranking of transverse reinforcement

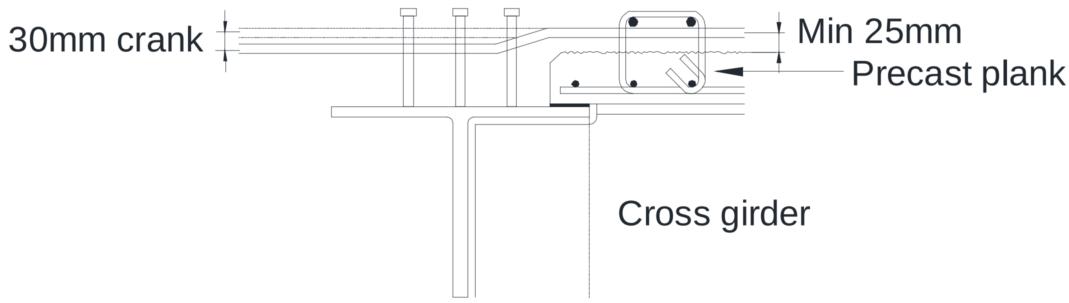
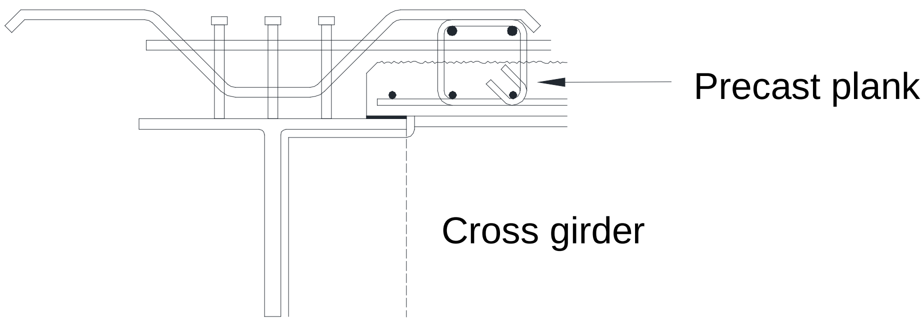


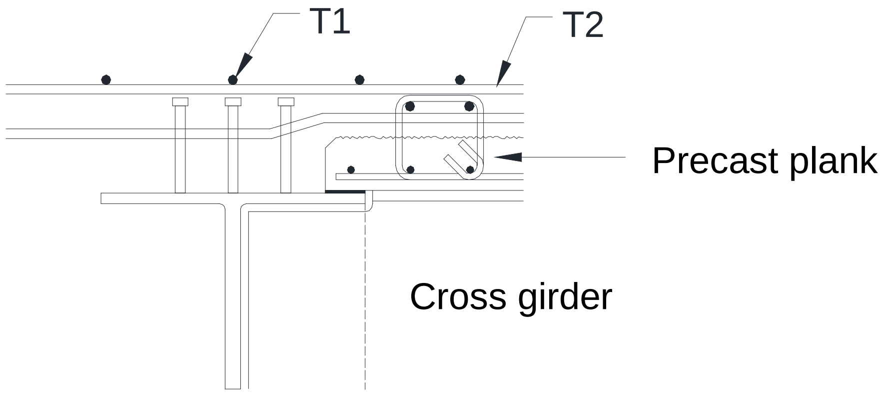
Figure 9.15 Alternative use of U-bars



Consideration should also be given to ease of fixing the top mat of the slab reinforcement. Usually the transverse bars would be detailed in the top layer as this gives maximum lever arm for the tension reinforcement at the root of the deck cantilever. However for steel fixing it is easier to place the transverse bars as the lower layer, directly onto the precast plank lattice (layer T2 in figure 9.16), and then place the longitudinal bars in the top layer (ie layer T1 in figure 9.16).

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

Figure 9.16 Positioning of top reinforcement in slab



As for multi-girder decks, protective treatment should be returned along the top surface of the top flanges edges by a minimum of 100mm and sealants used where appropriate.

9.5 Bearing specification

Bearing design and construction are covered by *AS 5100.4* (SA 2004).

Bearing design is usually the responsibility of the bearing manufacturer; the bridge designer should provide a specification for each bearing, listing the range of reaction forces and movements (translational and rotational).

The most commonly used type of bearing for highway bridges in New Zealand are elastomeric bearings (section 12 of *AS 5100.4*); consisting of a single unbounded layer or laminated elastomeric bearings. The other is the pot bearing (section 13 of *AS 5100.4*); consisting of a disk of elastomer confined in a short cylinder, onto which the reaction is transferred by a 'piston'. Both types of bearings accommodate moderate rotations in any direction but are relatively stiff vertically. If a sliding surface is provided within the bearing, translational movement can be accommodated; freedom can be provided in any direction, or guides may be provided to confine movement to one direction. Without a sliding surface, full translational restraint is provided.

Displacements due to permanent actions in heavily skewed decks may include large rotations (about each main girder axis) at both intermediate and end supports, and thus large transverse displacements at the bottom flange level.

This is particularly true for ladder deck bridges. These effects are a function of the plan geometry of the deck and are related to the magnitude of the dead load precamber required; they cannot be avoided. Due allowance for these rotations should be made in the design of the bearings (include the rotations in the bearing schedule) and the detailing of tapered plates to which the bearings are attached.

The increased flexibility of the cross section in decks with shallow cross-girders can lead to significant splaying of the main girders under both permanent and variable actions, causing relatively large transverse translations in the bearings.

The use of deep cross-girders at intermediate supports and concrete diaphragm beams at end supports will greatly reduce these movements.

As well as specifying the bearing, the designer should consider the requirements for attaching the bearing, both to the steelwork and to the substructure. The attachment or anchorage, usually with a tapered plate between the flange soffit and the bearing, should be designed for both the vertical and horizontal forces involved (see section 10 of *AS 5100.4*). The alignment of the bearings and the identification of the direction of the principal movement should be made clear on the drawings.

9.6 Deck joints

Deck joints are usually designed and supplied by specialist manufacturers. The bridge designer is required to provide a specification, giving the displacements and design actions on the joint. Guidance on the design requirements and specification of deck joints is given in section 17 of *AS 5100.4*, with additional guidance and modifications outlined in section 4.7 of the *Bridge manual*, which the designer shall design to.

10 Design of durable composite structures

Durability design of steel and concrete must be considered by designers and is an important aspect of a successful cost-effective design of any bridge structure.

10.1 Durability design of steel and concrete

To complement the numerous benefits of steel-concrete composite construction and extend its life, suitable protective measures must be taken to ensure structural performance over its design life. This is especially the case for the wide range of corrosion environments for either steel or concrete in New Zealand.

New Zealand is a long, thin mountainous country lying in the prevailing westerly wind belt of the Southern Hemisphere. Although surrounded by sea, its two nearest significant land masses are one of the world's hottest continents, Australia, and the world's coldest, Antarctica. All these conditions give New Zealand a diverse climate and very wide range of corrosion conditions, which if not designed for properly, will prohibit steel and concrete from reaching their full sustainability potential by premature failure. The following sections provide a summary of the available recommended publications that will provide information to design engineers on different durability issues and their solutions.

10.1.1 Durability design of steel

Any steel structure exposed to a corrosive environment must be designed to provide optimum long-term performance with a minimal level of normal maintenance. Durability design will require either the use of self-protecting stainless or weathering steel or conventional carbon steel with a corrosion protection system utilising a corrosion protection coating. When conducting a durability design, the bridge designer is recommended to determine the optimum solution, ie one that will achieve the most economic time to first maintenance based on the structure's performance and aesthetic requirements, design life and location. The optimally designed structure, whether coated or uncoated, will minimise the initial material and energy inputs, provide cost savings from reduced future maintenance, provide health and safety benefits and, for coated structures, less on-site debris to be contained and disposed of.

HERA report R4-133 'New Zealand steelwork corrosion and coatings guide' (El Sarraf and Clifton 2011), which is used in conjunction with the joint Australian/New Zealand standard *AS/NZS 2312* (SNZ 2002) *Guide to the protection of structural steel against atmospheric corrosion by the use of protective coatings* and the New Zealand standard *NZS 3404.1* (SNZ 2009) *Steel structures standard, part 1*. All three provide guidance to allow an appropriate and cost-effective coating system for any type of structural steelwork to be selected and then specified in a generic manner. They cover the different types of protective coatings available, the calculation of design corrosion rates for any steel surface, interior or exterior, above or below ground, as well as coating inspection and maintenance, among other topics. The calculation method, outlined in section 10.2.2, is necessary if coatings are to be appropriately specified for the specific range of environments to be encountered.

Weathering steel provides an alternative to the use of protective coatings especially for bridges. *HERA report R4-97* 'New Zealand weathering steel guide for bridges' (El Sarraf and Clifton 2005) provides the necessary guidance to ensure that dependable performance is realised for applications of weathering steel for New Zealand bridges. It covers the limitations on the use of weathering steel, design issues such as standards and detailing, construction issues such as bolting, welding and handling, as well as what to look for when inspecting and maintaining a weathering steel bridge.

These publications provide improved information to design engineers on different durability issues and their solutions. By specifying a protective coating system or utilising the benefits of weathering steel a more sustainable and durable solution can be found.

10.1.2 Durability design of concrete

Concrete structures (and elements in a composite bridge) are robust. However, this robustness, even with increased concrete quality and cover to reinforcement, will not compensate for gross errors in design or construction. Design, detailing, specification, execution and maintenance all influence the durability of a structure, regardless of the materials used in its construction.

Section 3 of *NZS 3101* (SNZ 2006) provides comprehensive guidance on how to select the appropriate concrete quality in relation to the cover to reinforcement to provide a structure that is required to be durable in relation to the identified exposure classes throughout its design life. To achieve a durable concrete structure, other aspects of the process of design, specification and construction are equally important and should not be overlooked: in particular, achieving the minimum cover, attention to detailing, crack width control, care during the execution of the works and the correct curing of the concrete. These requirements are given in *NZS 3109* (SNZ 1997a).

Therefore, for any concrete structure (such as the piles and abutments) and concrete elements (such as the concrete decking in a composite bridge), these components will be durable for their intended design life, if made from properly compacted concrete which is in compliance with the compressive strength and other specified requirements, and in which the achieved cover to reinforcement meets the minimum levels specified.

10.2 An example of the durability design of steel

10.2.1 Background information

10.2.1.1 General steps to determining an appropriate coatings system using *AS/NZS 2312*

The procedure outlined in El Sarraf and Clifton (2011) and *AS/NZS 2312* (SNZ 2002) to determine an appropriate coatings system is as follows:

1. Determine the design service life of the element being coated.
2. Determine the site-specific corrosivity category (ACC), which is derived from the first-year steel corrosion rate. This rate is determined from the combination of:
    a. macroclimate, plus
    b. microclimate.
3. Determine the time to first (major) maintenance required for the coatings system.
4. Select an appropriate corrosion protection system to meet the environmental requirements of 2 and 3 stated above based on cost, performance and any owner-specified factors such as colour and appearance.

In accordance with the *Bridge manual*, the design service life for bridges is 100 years, which is much longer than the 50-year design life typically used for buildings. Note that the design life of the structure is not usually the same as its durability rating, ie the years to first major maintenance. This point is made in clause 1.6 of *AS/NZS 2312* and clause C5.1.1 of *NZS 3404.1* (SNZ 2009), where it is noted that the protection offered by the coatings systems is usually shorter than the design service life of the structure, which means due consideration must be given to its maintenance or renewal requirements at the planning and design stage. It is only when components of the structure are not accessible for maintenance after assembly that the corrosion protection system must remain effective for the design service life of the structure. This important distinction must be recognised by designers and specifiers.

The process of selecting an appropriate coatings system is always site specific, and will typically be surface specific where microclimate effects are important and different surfaces have different exposures.

10.2.1.2 Determining the first-year corrosion rate

Macroclimate

An important part of this guide is the maps of first-year carbon steel corrosion rate in New Zealand. These maps are based on equations developed by (Hyland and Enzensberger 1998), using BRANZ corrosion data published by (Duncan and Cordner 1991) and climate data from the National Institute of Water and Atmospheric Research Ltd. The equations were determined as a function of the following variables which are needed to determine the macroclimate:

- distance from seacoast (0.5, 5, ≥20km)
- average annual daily temperature
- 9am time of wetness (RH ≥80%)
- annual rainfall
- upper bound results used.

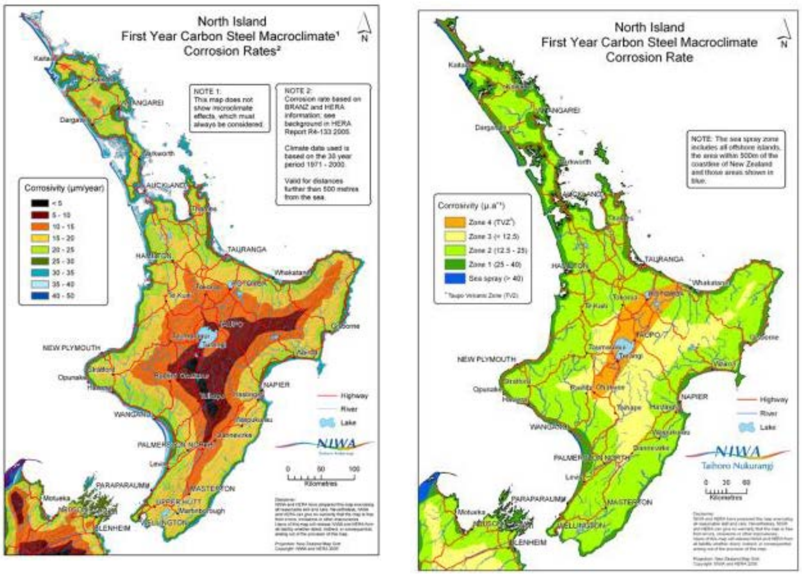
The equations were used to produce the macroclimate corrosion rate maps which greatly simplified the process of determining the atmospheric corrosivity category, as seen in figure 10.1, which provides the North Island first-year corrosion rates. The corrosion maps have recently been updated and simplified in *NZS 3404.1*.

Microclimate

The other factor needed to determine the corrosion rate is the microclimate effects on the steel surface. These depend on whether the steel surface is shaded, in a wet location, and whether the steel is in contact with timber or concrete. The most significant microclimate effect is if the steel surface is sheltered from rain washing but exposed to the windblown marine salts as this greatly influences the corrosion rate, depending on the distance from the sea and its position in relation to the prevailing wind.

Each one of these factors will affect the corrosion rate by multiplying or adding to the macroclimate corrosion rate determined above. Figure 10.1 demonstrates the effects of the microclimate on determining the atmospheric corrosivity category.

Figure 10.1 Detailed (left) and simplified (right) North Island first-year carbon steel macroclimate corrosion rates



10.2.2 Waikato River bridge example

The following example is given to demonstrate the process of determining the life cycle costing methodology and the differences between a standard durability design and a sustainable durability design. A steel bridge is to be built for the NZTA, which is located on the Waikato River in Hamilton, 40km from the sea. Section 10.2.3 starts with the methodology of determining the actual atmospheric corrosion category that will be used to specify a coating system and calculate the corrosion rate of weathering steel.

10.2.3 Determine the atmospheric corrosion category

10.2.3.1 Determine the design service life of the element being coated

The bridge is to have a design life of 100 years as stated in the *Bridge manual*.

10.2.3.2 Determine the site-specific corrosivity category (ACC), which is derived from the first-year steel corrosion rate

This rate is determined from the combination of the macroclimate and microclimate effects.

Site macroclimate effect:

The site macroclimate effect is determined from the North Island first-year carbon steel macroclimate corrosion rate found in El Sarraf and Clifton (2011, appendix A.1), as 20 μm/year.

Site microclimate effect:

Shaded location: microclimate effect of a shaded location is 5μm/y which is added to the macroclimate effect, from El Sarraf and Clifton (2011, section 4.3.1). Therefore, the design corrosion rate is now 25μm/y.

Unwashed effects: taking into account the unwashed area C_{uw} multiplier from section 4.3.2, the multiplier is:

C_{uw} = 1.2 for sites greater than 5km from the seacoast (option 5 of El Sarraf and Clifton 2011, section 4.3.2).

Therefore, the first-year corrosion rate is 25 x 1.2 = 30 μm/y.

10.2.3.3 Corrosivity category for the site, including microclimate effects

This is obtained by using the first-year corrosion rate calculated in table 10.1 as 30 μm/y.

From table 10.1, the corrosivity category C20%D applies; this designation means that the determined atmospheric corrosivity category is within C and 20% of the way towards category D.

Table 10.1 Corrosivity categories

Corrosion rate for steel μm/year	AS/NZS 2312	ISO 9223
<1.3	A: Very low	C1
1.3 to 25	B: Low	C2
25 to 50	C: Medium	C3
50 to 80	D: High	C4
80 to >200	E: Very high	C5

Note: This is based on table B1 of AS/NZS 2312 (SNZ 2002).

10.2.3.4 Determine the time to first (major) maintenance required for the coatings system

For a more sustainable durability design option a time to first maintenance of 35 years or longer time is sought, while a time to first maintenance of 25 years is considered for the 'standard' durability design.

10.2.4 Select an appropriate corrosion protection system

A single coat inorganic zinc silicate (IZS) solvent borne (SB), IZS3-SB which has 125μm dry film thickness will meet the 35 years requirement. Based on El Sarraf and Clifton (2011) and *AS/NZS 2312*, for the determined corrosivity category of C20%D, the time to first maintenance for the IZS3-SB system is 37 years.

Note that inorganic zinc silicate (IZS) coatings are available in either solvent-borne (SB or water-borne (WB) form. *AS/NZS 2312* gives lower performance levels for the solvent-borne (SB coatings than for the water-borne (WB coatings. However, this is misleading for the SB systems for the reasons given in section 7.3.5 of El Sarraf and Clifton (2011). The sacrificial protection offered by both products is similar; hence the only difference in design life is due to the slightly increased quantity of active ingredient (powdered metallic zinc) in the WB systems.

For the 25 years' time to first maintenance, a polyurethane (PUR3) three coat thickness with a total 250μm DFT is chosen.

10.2.5 Determination of life cycle cost of IZS3-SB and PUR3

This section gives the life cycle cost estimate for the IZS3-SB and PUR3 coating systems over the proposed 100-year life determined above. The information is presented in figures 3.2 and 3.3 (see El Sarraf and Clifton 2011 for further guidance). The estimated time to next maintenance is determined in accordance with section 10.2.4 above. The coating and labour rates are provided from the coating supplier, International Protective Coatings. The labour costs are based on shop application of large areas, with adjustments made for site conditions as specified in note 6 in figures 3.2 and 3.3. The net present value is then calculated to provide a more realistic associated cost.

10.2.5.1 Estimated time to next maintenance

The estimated time to next maintenance is assessed in accordance with *AS/NZS 2312* and (Reina et al 1998) as follows:

After each painting (initial coat and maintenance recoats), 2% of the area is assumed to require touch up after three years and another 5% of the area is assumed to require touch up at 75% of the time for which the next full coating is required.

The time to first full coating is taken as 35 years in figure 3.2 and 25 years in figure 3.3. The time to subsequent touch-up repair is taken as 75% of 35 years (26 years) and 75% of 25 years (18 years).

The two touch-up coats between full recoats are in accordance with the recommendations for bridge coatings, based on experience. In practice, one or both may not be necessary. It is assumed that both coatings thickness loss will be reasonably consistent over the exposed surface area so that all areas will require reinstatement when the durability time to first maintenance is reached. For that reason, a full recoat is specified when that time is reached. In practice, using the criteria for assessing when to paint or repair from clause 10.2(a) of *AS/NZS 2312*, a longer interval between recoats than that used in figures 3.2 and 3.4 might be obtained.

Figure 10.2 Life cycle cost estimate for IZS3-SB including inspection

Operation Number	Year (1)	System Designation	%Area Maintained	Current Cost (2) $/total m^2	NPV (3) for DR (4) 8% $/total m^2
0	0	A1+A2+F	100%	44.00	44.00
1	0	B+F	2%	1.78	1.78
2	3	C+F	2%	1.78	1.42
3	26	E	100%	covered	client
4	26	C+F	5%	4.46	0.60
5	35	E	100%	covered	client
6	35	D+F	100%	57.00	3.86
7	38	C+F	2%	1.78	0.10
8	61	E	100%	covered	client
9	61	C+F	5%	4.46	0.04
10	70	E	100%	covered	client
11	70	D+F	100%	57.00	0.26
				Total $	52.06

Coating System Specification:
Based on 125 micron IZS3-SB, paint reference number CO1a from (AS/NZS2312 2002).
Time to first maintenance (35, 40, 50 years) 35
System Designation:

A1 Washdown	High pressure washdown to remove fabrication oil etc. Prior to blast cleaning
A2 Initial Coat:	Shop: Grit Blast to Class 2.5; 125 micron nominal DFT IZS3-SB.
B 2% Erection Touch-up:	Field: Abrasive blast of damaged area to Class 2.5; 125 micron nominal DFT IZS3-SB
C 2% or 5% Touch-up Repair:	Field: Abrasive blast of degraded area to Class 2.5; 125 micron nominal DFT IZS3-SB
D 100% Recoat:	Field: Abrasive blast of whole area to Class 2.5; 125-150 micron nominal DFT IZS3-SB
E Inspection of surface to determine condition [9]:	Field: Independent inspection of existing surface to determine extent of maintenance required at 75%, 100% tfm. Cost covered by Transit NZ/ONTRACK general bridge inspection regime and not part of specific steel coating cost
F Independent inspection of coating [10]:	Field: Independent inspection of applied coating for designations A2, B, C and D

Base costings used are as follows (these are for shop application of large areas, with small area and site access factors included for the on site work as described below)

Operation	Cost $/m^2 steel surface
Initial washdown in shop	4.00
Blast clean and apply single coat (labour cost)	18.00
Shop inspection of surface or coating	4.00
Site inspection of surface or coating	5.00

Paint/Inspection System Designation	Base Rate Paint $/m^2	Base Rate Labour $/m^2	Area Factor Labour(5) $/m^2	Initial Rate(6) $/m^2	Site Access Rate(7) $/m^2	Current Cost(8) $/m^2
A1	0.00	4.00	1.00	4.00	0.00	4.00
A2+F	18.00	22.00	1.00	40.00	0.00	40.00
B+F	19.00	23.00	2.40	74.20	15.00	89.20
C+F	19.00	23.00	2.40	74.20	15.00	89.20
D+F	19.00	23.00	1.00	42.00	15.00	57.00
E	0.00	5.00	1.00	5.00	15.00	20.00

Summary:

Initial Cost/total m^2	$ 47.20	This covers operations 0, 1 and 2 above which are part of the construction contract
NPV Maintenance Cost 100 year/total m^2	$ 4.86	This covers operations 3 to 11
Full life cycle NPV cost 100 year/total m^2	$ 52.06	

Notes:
1. Year after commissioning bridge.
2. Current Cost/total m^2 (% Area Maintained x System Cost $/m^2)
3. NPV Net Present Value: NPV = Cost/(1+DR/100)^{Year}
4. Recommended discount rate for NZTA NZ is 8% for 30 years. (Economic Evaluation Manual (Volume 1) 2010)
5. Area factor for labour is obtained from Section 10.5 of (El Sarraf and Clifton 2011). It allows for higher setup and wastage costs due to the small areas involved.
6. The initial rate is the materials + labour including area factor.
7. The Site access rate is taken from Section 10.5 of (El Sarraf and Clifton 2011), using *moderate* for touch-up, small area maintenance and for recoat. This reflect the remotness of the site.

8. The current cost/m^2 is the initial rate plus the site access rate.
[illegible]
10. Inspection costs for the painted surface cover independent inspection of the area that has been painted.

DISCLAIMER: The above costings are approximate and should be confirmed with a coating supplier and applicator.

Source: El Sarraf and Clifton (2011).

Figure 10.3 Life cycle cost estimate for PUR3 including inspection

Operation Number	Year (1)	System Designation	%Area Maintained	Current Cost (2) $/total m^2	NPV (3) for DR (4) 8% $/total m^2
0	0	A1+A2+F	100%	60.50	60.50
1	0	B+F	2%	2.52	2.52
2	3	C+F	2%	2.52	2.00
3	18	E	100%	covered	client
4	18	C+F	5%	6.30	1.58
5	25	E	100%	covered	client
6	25	D+F	100%	73.50	10.73
7	28	C+F	2%	2.52	client
8	43	E	100%	covered	client
9	43	C+F	5%	6.30	0.23
10	50	E	100%	covered	client
11	50	D+F	100%	73.50	1.57
12	53	C+F	2%	2.52	0.04
13	68	E	100%	covered	client
14	68	C+F	5%	6.30	0.03
15	75	E	100%	covered	client
16	75	D+F	100%	73.50	0.23
				Total $	79.43

Coating System Specification:

Based on a 3 coat total DFT of 250 micron PUR3, paint reference number C06+C13+C26 from (AS/NZS2312 202) .

Time to first maintenance (35, 40, 50 years) 25

System Designation:

A1 Washdown	High pressure washdown to remove fabrication oil etc. Prior to blast cleaning
A2 Initial Coat:	Shop: Grit Blast to Class 2.5; 250 micron nominal DFT PUR3
B 2% Erection Touch-up:	Field: Abrasive blast of damaged area to Class 2.5; 250 micron nominal DFT PUR3
C 2% or 5% Touch-up Repair:	Field: Abrasive blast of degraded area to Class 2.5; 250 micron nominal DFT PUR3
D 100% Recoat:	Field: Abrasive blast of whole area to Class 2.5; 250 micron nominal DFT PUR3
E Inspection of surface to determine condition 9:	Field: Independent inspection of existing surface to determine extent of maintenance required at 75%, 100% tfm Cost covered by Client general bridge inspection regime and not part of specific steel coating cost
F Independent inspection of coating 10:	Field: Independent inspection of applied coating for designations A2, B, C and D

Base costings used are as follows (these are for shop application of large areas, with small area and site access factors included for the on site work as described below)

Operation	Cost $/m^2 steel surface
Initial washdown in shop	4.00
Blast clean and apply three coat system(labour cost)	32.50
Shop inspection of surface or coating	4.00
Site inspection of surface or coating	5.00

Paint/Inspection System Designation	Base Rate Paint $/m^2	Base Rate Labour $/m^2	Area Factor Labour(5) $/m^2	Initial Rate(6) $/m^2	Site Access Rate(7) $/m^2	Current Cost(8) $/m^2
A1	0.00	4.00	1.00	4.00	0.00	4.00
A2+F	20.00	36.50	1.00	56.50	0.00	56.50
B+F	21.00	37.50	2.40	111.00	15.00	126.00
C+F	21.00	37.50	2.40	111.00	15.00	126.00
D+F	21.00	37.50	1.00	58.50	15.00	73.50
E	0.00	5.00	1.00	5.00	15.00	20.00

Summary:

Initial Cost/total m^2	65.02	This covers operations 0, 1 and 2 above which are part of the construction contract.
NPV Maintenance Cost 100 year/total m^2	14.41	This covers operations 3 to 11.
Full life cycle NPV cost 100 year/total m^2	79.43	

Notes:

1. Year after commissioning bridge.
2. Current Cost/total m^2 (% Area Maintained x System Cost $/m^2)
3. NPV Net Present Value: NPV = Cost/(1+DR/100)^{Year}
4. Recommended discount rate for NZTA NZ is 8% for 30 years. (Economic Evaluation Manual (Volume 1) 2010)
5. Area factor for labour is obtained from Section 10.5 of (El Sarraf and Clifton 2011). It allows for higher setup and wastage costs due to the small areas involved.
6. The initial rate is the materials + labour including area factor.
7. The Site access rate is taken from Section 10.5 of (El Sarraf and Clifton 2011), using *moderate* for touch-up, small area maintenance and for recoat. This reflect the remotness of the site.

8. The current cost/m^2 is the initial rate plus the site access rate.

10. Inspection costs for the painted surface cover independent inspection of the area that has been painted.

DISCLAIMER: The above costings are approximate and should be confirmed with a coating supplier and applicator.

Source: El Sarraf and Clifton (2011).

10.2.6 Determining the corrosion rate of weathering steel for a 100-year design life

The calculation below will estimate the corrosion rate of the weathering steel based on El Sarraf and Clifton (2011) and El Sarraf and Clifton (2005) and *ISO 9224* (ISO 2012). It uses the corrosion rate of mild steel determined in section 10.2.3 herein and that given in *ISO 9224* to interpolate the corrosion rate of weathering steel in comparison to mild steel. Therefore, the corrosion rate of weathering steel is:

10.2.6.1 Determine the corrosion rate of weathering steel for the first 10 years of the 100-year design life from ISO9224 (ISO 1992)

From *ISO 9224*, the corrosion rate (r_{av}) for the first 10 years for weathering steel for an atmospheric corrosivity category (ACC) of C20%D is given as 2 to 8. Therefore, taking 20% into that range the first 10-year corrosion rate is given as:

r_{av} = 3.2 μm/y

10.2.6.2 Determine the corrosion rate of weathering steel for the remaining 90 years of the 100-year design life from *ISO 9224*

From *ISO 9224*, the corrosion rate (r_{lin}) for the remaining 90 years for weathering steel for an ACC of C20%D is given as 1 to 5. Therefore, taking 20% into that range the remaining 90-year corrosion rate is given as:

r_{lin} = 1.8 μm/y

Therefore, the 100-year design life total corrosion per exposed face is:

3.2(10) + 1.8(90) = 194 μm/exposed face or 0.2mm/exposed face

A factor of 2 is recommended to be applied to allow for localised increased rates of corrosion. Therefore, when designing the main girders, a loss of 0.4mm/exposed face should be taken into account when determining the section capacity of the girder.

10.2.6.3 Notes on using weathering steel

The calculated corrosion rate stated above is based on the protective patina layer forming, if this does not form properly then continuous corrosion of the steel will occur which will be higher than the estimated corrosion rate above. Weathering steel will start out as a rusty red colour but with time it will become a darker earthy tone (nearly black) but only if the conditions are favourable (eg not subject to salt contamination and low 'times of wetness'). For the bridge in this example, the proposed site is an ideal location for the protective patina layer to form.

10.2.6.4 Costs of using weathering steel

On average the cost of weathering steel is $300/tonne more than that of mild steel. In this case, the cost (as at October 2008) of uncoated mild steel plate is $2200/tonne, this equates to an average cost of $2500/tonne for weathering steel (the designer should contact the steel supplier for the latest prices of mild and weathering steel when undertaking a cost comparison).

In this example, the cost per square metre of using weathering steel equates to \$14.27/m^2 to \$31.2/m^2 for beams ranging from an 800WB122 to 1200WB455 respectively.

Once the patina forms on the weathering steel, the greatly reduced corrosion rate will allow the steel to meet the required performance requirements with negligible, if any, maintenance. These figures are applicable for the net present value minus the inspection cost, as they equate to the initial cost of a protective corrosion system. Even though weathering steel has this cost premium in comparison to uncoated mild steel, the future cost savings on maintenance makes this option cost competitive.

10.2.7 Results discussion

A summary of the results is given in table 10.2. By comparing both coating systems net present value, the use of a single coat coating with a longer time to first maintenance provides the most cost-effective solution. This solution also has less on-site wastage with the fact that only one coating layer is needed to be reinstated while the multi-coat option requires up to three coats. Also, with a longer time to first maintenance the IZS3 option requires only two recoats in comparison with the three recoats for the PUR3, this equates to a reduced possibility of accidents and other health and safety issues occurring.

However, when comparing the costs between the inorganic zinc silicates (IZS) single-coat system with the weathering steel option, the results show that the latter provides a more economical solution. Also, with the negligible maintenance, on-site wastage is further reduced to a minimum, while the health and safety benefits have been greatly improved due to that negligible maintenance.

Table 10.2 Cost difference between the different corrosion protection systems

Corrosion protection system	**Cost ($/m^2)* for a 100-year design life**	**Notes**
	Net present value	
IZS3	$52.06	Single-coat system
PUR3	$79.43	Multi-coat system
Weathering steel	$31.2	Assuming a 1200WB 455

Note: Cost comparison is based on October 2008 prices.

10.2.8 Durability design example conclusion

Even though weathering steel produced the most economical solution, other factors must be considered by the designer when specifying the optimum sustainable durability option. The designer must not only consider achieving the most economic time to first maintenance option, but must also consider the required structure's performance, aesthetics, design life and location, which includes future maintenance accessibility. These factors may govern the design and a more expensive option may be chosen to meet requirements. Weathering steel starts as a layered rust colour which gets darker with age. Aesthetically this may not be desirable in high-profile projects. IZS provides a single colour option of grey, while the polyurethane option provides a wide range of colours and graffiti protection. The aesthetic factor is one of many factors considered when carrying out a sustainable durability design. All these factors govern the chosen corrosion protection system based on the requirement stated by the client. Whether it is a major highway bridge project, a road bridge or a back country overpass, each structure has its challenges and requirements which the designer must consider to produce the optimum solution for that structure.

10.3 Coating application and inspection

The correct application of a specified corrosion protection coating system is one of the crucial steps in the protection of structural steelwork. It is recommended that the protective coatings are applied by a qualified applicator who has acquired the required level of training on the application of coating systems by a recognised organisation, such as Extractive Industries Training Organisation. Furthermore, regular inspection throughout the application process by a certified independent third party inspector, either by a Certified Board for Inspection Personnel, Australasian Corrosion Association or NACE International coating inspector, is recommended to ensure the appropriate quality control and health and safety procedures are followed. Most importantly, inspectors will also supervise the actual application of the coatings to approve the correct coating with its resultant dry film thickness has been applied in accordance with the relevant standards and coating supplier's guidelines.

Guidance on the specification, application and inspection of coating systems is available through Steel Bridge Development Group on www.steelbridges.org.nz. Designers should be familiar with these documents to ensure that the specified protective coating system will provide the required corrosion protection, which includes its maintenance, throughout the design life of the structure.

10.4 Maintenance management

Another important topic is the maintenance management of coating systems and structures. All structures require regular maintenance to guarantee their performance over the design life of the structure, as specified in the *Building code* (DBH 1992) and the *Bridge manual*. However, this is sometimes not adequately addressed at the design stage. Considerations for maintenance must be conducted as part of the design process; this includes accessibility for future maintenance and the establishment of a maintenance regime as required for all of the components during the life of the structure, from reinstatement of the coating system to the regular cleaning of the steelwork of chemical and biogenic contamination. This also applies to concrete elements of the composite bridge.

In reality, both steel and concrete bridges require periodic maintenance to attain a 100-year life. Where the maintenance programmes will differ is when the work is required.

For example, concrete bridges in New Zealand have not been maintenance free. Significant remedial work on many bridges, including high-profile bridges, has been required after 30 to 60 years with some cases requiring a complete rebuild. Examples are the Newmarket Viaduct, for which the distortional effects of temperature were under recognised by the design procedure at that time, resulting in the initial post tensioning being insufficient to prevent tension on lower regions of the section near the supports. Remedial work to suppress this, in the form of additional post tensioning, led to over-compression and resulting deterioration of the concrete at the supports. Another bridge is the Victoria Park Bridge flyover, which is suffering alkali-aggregate reaction and needs replacement and a number of bridges close to the sea that have accelerated rebar corrosion due to improper concrete cover thickness (Bruce et al 1999; Bruce et al 2006; Rogers et al 2009).

Steel bridges in New Zealand have also been adversely affected by shortcomings in the design procedures of the day. The best-known example is the Auckland Harbour Bridge, which suffers from problems with fluctuating loads in the deck of the clip-on box girders due to the effect of fatigue on the light-weight steel deck not being adequately accounted for in the design procedures of the 1960s. However, steel bridges can generally be repaired rather than needing to be replaced in this situation. In most cases, regular inspection of the structure can determine potential structural issues more easily than concrete structures, which allows for remedial actions to be undertaken before the issues become irreparable.

11 Other types of composite construction

11.1 Composite box girders

Steel-concrete composite box girders offer an attractive and economic form of construction for medium span bridges ranging between 50m and 100m. They are commonly used for spans where plate girder sizes may be excessive or where torsion, curvature or wind and seismic forces demand greater torsional stiffness. Torsional stiffness is provided by the hollow box shape of the girder that is either rectangular or trapezoidal in cross section. Basically, the box girder comprises two 'webs' that are connected by a single bottom flange, while each web has its own top flange for an open box girder or another single top flange for a closed box girder.

Stiffeners are evenly spaced along the length of the girder and usually a single wide box girder is sufficient for a single or double lane bridge. For wider bridges, two or more box girders can be used, with additional restraints provided, if required at supports, between them.

The benefits of a steel-concrete composite girder are:

- longer spans possible providing cost effect spans
- span-long girder sections can be erected by mobile cranes or launched from one end of the bridge
- enhanced torsional performance may reduce bearing requirements in comparison to a plate girder bridge
- smooth clean lines with clean surfaces provide an aesthetically pleasing structure
- sloping webs and lack of bottom flange at the outer face reduced potential durability issues due to water entrapment.

Disadvantages of box girder are:

- difficult to fabricate thereby adding to the cost
- more difficult to maintain due to the access to confined spaces inside the girder.

An example of a box girder is shown in figure 11.1.

Figure 11.1 Steel-concrete composite box girder

11.2 Network arch bridges

A network arch is a tied arch structure where the hangers supporting the tie, which also acts as the deck of the bridge, are inclined and arranged in such a way that they cross one another at least twice. Such structures are used to carry a roadway, railway or footpath.

Like any tied arch, the load on the deck is carried principally as compression in the arch and tension in the tie. Increasing the rise of the arch reduces the axial forces in both the arch and the tie. The majority of the shear force is taken by the vertical component of the arch top chord force, with any variation in the shear force taken by the hangers.

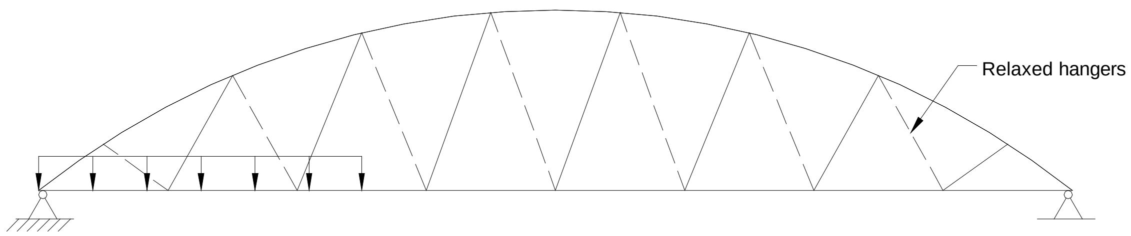
This form of structure was developed in Norway during the 1950s by Per Tviet with the first network arch bridge constructed at Steinkjer, Norway in 1963 (Tviet 2008). Although many examples of network arch bridges have been constructed in Japan, where they are often (incorrectly) referred to as Neilsen-Lhose bridges, network arch bridges have not been adopted widely around the world until recently. This is being changed now with a number of network arches in design and construction in recent years across Europe, the USA and now in New Zealand with the Mangamuhu Bridge on the Mangawhero River north-east of Wanganui, and the Waikato River Bridge near Taupo (figure 11.3).

Figure 11.2 Waikato Network Arch Bridge near Taupo (image courtesy of Holmes Consulting Group)

Network arch bridges tend to act like a truss with a light web. With inclined hangers, the effects of concentrated or non-symmetric loading are distributed better than with vertical hangers and the bending moments and shear forces in the arch and deck due to such loading are reduced.

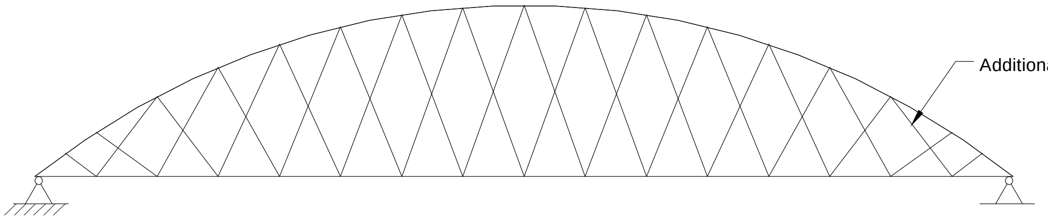
The benefits of using an inclined overlapping hanger system may be seen in figures 11.3 and 11.4. When the span is subjected to a distributed load on only part of the span length, some of the hangers will tend to relax as can be seen in figure 11.3. Increasing the distance: between hanger nodes reduces the tendency for the hangers to relax but this creates increased bending in the arch chords requiring larger chord elements.

Figure 11.3 Partial loading causing relaxation of hangers (Tviet 2008)



As the arch top chord tends to deflect upward and the bottom chord downwards between the arch hanger points under loading, introducing a second set of inclined hangers between the first set helps to minimise the arch chord deflections and bending actions as seen in figure 11.4. Several sets of overlapping hangers can be introduced to decrease the distance between hanger points further, thus creating the network arch configuration.

Figure 11.4 Additional hangers added to original layout (Tviet 2008)



By decreasing the distance between hanger locations, bending actions in the arch's top and bottom chord elements are reduced. The shorter length of the arch top chord also increases the buckling strength of the top chord element. By minimising the bending component of action in the arch chords, greater efficiency can be realised from a given chord element section. Compared with similar span arches using vertical hangers, reported savings of up to 50% of the structural steel weight can be realised through a network arch structure (Tviet 2008).

The arch top chord can be profiled to follow the line of thrust of forces within the arch. For simplicity of fabrication, it is easy to adopt a circular geometry for the arch top chord profile. While compressive actions dominate in the arch top chord, good support is offered by the hangers reducing the tendency of the chord to buckle in the vertical plane. Therefore greater stiffness is provided in the transverse direction, leading to the adoption of UC or H-profile sections for these elements in a number of bridges, where heavy rolled UC or H-profile sections are available, such as in Europe and the USA, these are very efficient choices for top chord elements. As heavy hot-rolled UC sections (upwards of 400 – 600kg/m) are not generally available in New Zealand, some fabrication of the top chord may be required.

The original concept prefers that the bottom tie is constructed from post-tensioned concrete – the tensile force carried in longitudinal post-tensioning tendons. The compression provided to the concrete slab limits cracking of the bottom tie member. Depending on the width of the bridge between arches, the bottom concrete deck tie may be transversely post-tensioned as well.

Depending on construction methodology, some form of temporary bottom tie may be required (for example, if the steelwork tie is erected and lifted into place prior to the bottom concrete tie being completed). For Mangamuhu Bridge, it was found that it was appropriate to utilise a ladder form of deck construction with a steel UC longitudinal tie and transverse transoms at regular centres (Chan and Romanes 2008). The precast deck slabs were made composite with the transoms and bottom tie with an in situ topping. Longitudinal post-tensioning is provided within the deck slab to provide a residual compressive stress and control the variable tensile stresses due to live loading.

11.3 Stainless steel in bridge construction

Stainless steel is becoming a more common choice for bridge construction around the world and especially in Europe, as engineers begin to recognise the benefits that the material's durability can offer. A number of bridges incorporating stainless steel for the main structural elements have been built in the past eight years, such as the recently opened Añorga Railway Bridge in Spain (figure 11.5), Likholefossen Bridge in Norway (figure 11.6), the Celtic Gateway Bridge in Holyhead, Wales (figure 11.7), and the Passarella Ruffolo in Sienna, Italy.

Figure 11.5 Añorga Railway Bridge in Spain (photo courtesy of Outokumpu)

Figure 11.6 Likholefossen Bridge in Norway

Figure 11.7 The Celtic Gateway Bridge in Holyhead, Wales

Although there is an enormous variety in stainless steel types, their common denominator is the presence of at least 11% of chromium. In combination with other elements such as nickel, molybdenum or nitrogen, this produces a steel alloy that has high corrosion resistance, malleability, ductility and mechanical strength, even when exposed to high temperatures, as well as excellent aesthetics and easy maintenance and cleaning. The chromium in the stainless steel forms a stable and transparent layer of chromium oxide on the surface - the so-called passivation layer - that prevents corrosion. Four main types of stainless steel exist according to their metallurgical structure: ferritic, austenitic, duplex and martensitic - of these, the high strength of duplex steel makes it suitable for applications such as bridges.

Duplex stainless steel is an austenitic-ferritic alloy which has a microstructure of high corrosion resistance, excellent ductility and mechanical characteristics superior to the great majority of carbon steels. With the existence of such a wide range of duplex steel grades, the selection of the most suitable type clearly depends on the ambient aggressiveness, mechanical properties, types of surface finish and so forth.

Unlike conventional carbon steel, stainless steel exhibits a nonlinear mechanical behaviour, even under lower stress values, without having a clearly defined elastic limit strength. However, it is being designed with a conventional yield stress with the value associated to a strain of 0.2%. The construction procedures for stainless steel are similar, but not identical to, those used for carbon steel. Austenitic steel shows excellent possibilities for bending, but it requires 50% more energy than carbon steel. It has a similar energy requirement when being welded, making it difficult to weld duplex steel grades. In addition, contact between stainless steel and other metals during manufacture can cause iron particles to be embedded in the stainless steel surface, thereby penetrating the protective passivation layer and initiating corrosion. Galvanic corrosion can also occur due to iron contamination at any stage, for example, the use of carbon steel strapping on stainless steel components or the bolting of carbon steel onto stainless steel. For this reason, manufacturing and assembly of the pieces must be carried out in areas that will not come in contact with carbon steel, and has to be done using specific tools.

In 2005, stainless steel was chosen for the Cala Galdana Bridge on the island of Menorca - the location of the bridge, in a harsh and corrosive environment, made stainless steel, with its resistance to corrosion, an attractive choice. The Algendar river runs into the sea at Cala Galdana and its channel has for the last 30 years been crossed via a reinforced concrete bridge approximately 18m long, but the concrete bridge reinforcement bars were in an advanced state of corrosion, induced by the marine atmosphere and serious settlement of one of the abutments. This promoted the owner Consell Insular de Menorca to replace it with a new bridge. The new bridge had to span the entire width of the old river channel, more than 40m, fit in with the natural surroundings, demonstrate great durability and require minimum maintenance. The final choice of a duplex stainless steel arch structure was mainly due to its high resistance to corrosion. In addition, its clear span restored the full width of the river channel, offered minimum maintenance and was regarded as a symbol of modern technology.

The overall length of the bridge is 55m with a 13m wide deck, two lanes of road traffic and two footways each 2m wide, to allow pedestrians to enjoy the panoramic views. The main structure consists of two parallel abutments by means of an inclined strut that takes the horizontal component of the arch axial force and, consequently, does not transmit significant horizontal forces to the abutments. The weight of the stainless steel is approximately 165 tonnes and the total cost of the bridge, including its accesses, was approximately €2.6 million.

Figure 11.8 The underside of the Cala Galdana Bridge

11.3.1 Are stainless steel bridges an option for New Zealand?

The answer is yes. The excellent durability performance and aesthetic properties of stainless steel lends its use for certain areas around New Zealand, specifically the areas within 5km from the sea. Initial construction cost is relatively higher than other materials; whether it is mild steel, weathering steel or concrete; however, by taking the life cycle cost of a bridge including the maintenance cost over the 100-year design life, stainless steel can provide a competitive solution in those specific locations. Stainless steel does lend itself to iconic projects and especially for pedestrian bridges. Lean duplex grade LDX 2101 is a cost-effective option in comparison with normal duplex stainless steel, for pedestrian bridges, while Duplex Grade 2205 is preferred for road or rail bridges.

As with other materials, the design must specify the correct specification which is crucial for the success of the project. One of the concerns with stainless steel is the discolouration of the surface of the steel known as 'tea staining'. This is caused by the use of a rougher surface finish than that required, especially in areas close to the sea. To take maximum advantage of the benefits of stainless steel, the correct grade and surface finish must be specified depending on the location and use of the bridge structure.

Guidance on the structural design of stainless steel is available via New Zealand Heavy Engineering Research Association (HERA), the New Zealand Stainless Steel Development Association and excellent guidance is available from the American Nickel Institute and the British Stainless Construction websites. Also guidance on the welding of structural stainless steel is given in *AS/NZS 1554.6* (SNZ 1994).

12 References

American Iron and Steel Institute (AISI) (2001) *Designing short span bridge.* Washington DC: AISI.

Australian Steel Institute (ASI) (1999) *Design capacity tables for structural steel.* Sydney: Australian Steel Institute.

Beamish, M, P Tindall and P Billing (2006) Auckland Harbour Bridge fatigue assessments. In *Proceedings of 2006 Austroads Conference*. Perth, Australia, 2006.

Beer, FP, R Johnston Jr, J DeWolf and D Mazurek (2008) *Mechanics of materials.* McGraw-Hill. New York.

Bird, G and M Feeney (1999) *Simplified design of steel members*. Auckland: SESOC.

British Constructional Steelwork Association (BCSA) (2005) *BCSA guide to the erection of steel bridges.* (ref 38/05).

Bruce, SM, SA Freitag and WE Hickman (1999) Durability of concrete road bridges in New Zealand. *Transfund NZ research report 129*. 87pp.

Bruce, SM, PS McCarten, SA Freitag and LM Hasson (2006) Deterioration of prestressed concrete bridge beams. *Land Transport NZ research report 337*.

Cement and Concrete Association of New Zealand (CCANZ) *Concrete^3 economic, social and environmental.* Wellington: CCANZ.

Chan, M and M Romanes (2008) New Zealand's first network arch bridges. *2008 Metals Industry Conference Proceedings, HERA*, Manukau City, New Zealand.

Clifton, GC (1997) Restraint classifications for beam member moment capacity determination to NZS 3404:1997. *HERA report R4-92*.

Clifton, GC (2007a) *Recommended draft fatigue design criteria for bridges*. Manukau City: HERA.

Clifton, GC (2007b) New Zealand structural steelwork limit state design guides volume 1. *HERA report R4-80*.

Corus (2006) *Sustainable steel construction.* Scunthorpe, UK: Corus.

CSC World (2006) *S-Frame software*. Leeds, UK: CSC.

Department for Transport (2006) *TD19/06: Requirement for road restraint systems. Design manual for roads and bridges.* Accessed 27 April 2013. www.dft.gov.uk/ha/standards/dmrb/vol2/section2/td1906.pdf

Department of Building and Housing (DBH) (1992) *New Zealand building code.* Wellington: Building Industry Authority, DBH.

Duncan, J and R Cordner (1991) Atmospheric corrosion rates over two years' exposure at 156 sites in New Zealand. *IPENZ transactions 18 (1/GEN)*, 1991, pp37–49.

El Sarraf, R and GC Clifton (2005) New Zealand weathering steel guide for bridges. *HERA report R4-97*.

El Sarraf, R (2008) Economical steel bridge solutions for New Zealand. *SESOC Journal 21*, no.2.

El Sarraf, R and GC Clifton (2011) New Zealand steelwork corrosion and coatings guide. *HERA report R4-133*.

Fisher, J (1977) *Bridge fatigue guide: design and details*. Chicago: American Institute of Steel Construction.

Gorenc, B, R Tinyou and A Syam (2005) *Steel designers handbook*. Sydney: UNSW Press.

Hambly, EC (1990) *Bridge deck behaviour*. Spon Press. UK.

Hayward, A (2002) *Composite steel highway bridges*. Scunthorpe, UK: Corus Construction Centre.

Heath and Safety Executive (2007) Managing *health and safety in construction*. Construction (Design and Management) Regulations. L144. UK: HSE Books.

Hendy, CR and DC Iles (Eds) (2009) *Steel bridge group: guidance notes on best practice in steel bridge construction*. SCI P185 (5th issue). Ascot, UK: The Steel Construction Institute.

HERA (2006); BRIDGENZ. Manukau City. Now withdrawn.

Hyland, CWK and M Enzensberger (1998) Prediction of site-specific steel corrosion rates in New Zealand to assist coatings selection. In *Proceedings of Australasian Structural Engineering Conference, volume 2,* pp835–842, Auckland, New Zealand.

Iles, DC (2010) *Composite highway bridge design*. SCI P356. Ascot, UK: The Steel Construction Institute.

International Federation for Structural Concrete (fib) (2005) *Guidelines for the design of footbridges*. Lausanne: fib.

Medland, IC and CM Segedin (1979) Brace forces in interbraced column structures. *Journal of the Structural Division 105*, no.7: 1543–1556.

Ministry of Works (1981) *Standard bridge design*. Wellington: Ministry of Works.

Mutton, BR and NS Trahair (1975) *Design requirements for column braces*. CE17, no.1. Australia: Civil Engineering Transactions, Institution of Engineers, pp30–35.

National Roads Board (1984) *Road Research Unit bulletin 70* (RRU 70). Wellington: National Roads Board.

New Zealand Steel (2003) *MemDes version.2.1*. Auckland: New Zealand Steel.

New Zealand Transport Agency (NZTA) (2001) *Bridge inspection and maintenance manual*, SP/M/016 (rev ed). Wellington: NZTA.

New Zealand Transport Agency (NZTA) (2003) *Bridge manual,* SP/M/022. Wellington: NZTA.

New Zealand Transport Agency (NZTA) (2004) *Provisional amendment to section 5 of the Bridge manual*. Wellington: NZTA.

New Zealand Transport Agency (NZTA) (2006) *Guide to heavy vehicle management*. RTS16. Wellington: NZTA.

New Zealand Transport Agency (NZTA) (2011) *Economic Evaluation manual (Volume 1)*. Wellington: NZTA.

New Zealand Transport Agency (NZTA) (2012) *Bridge manual*. 3rd ed. Wellington: NZTA.

Pucher, A (1964) *Influence surfaces of elastic plates*. Vienna: Springer-Verlag.

Rapattoni, F (1998) *Composite steel road bridges concepts and design charts*. Port Kembla, Australia: BHP.

Reina, MP, M Melampy and KR Shields (1998) Costing considerations for maintenance and new construction coating work. *Materials Performance*, November 1998: 25–30.

Rogers, M, MA Al-Ani and JM Ingham (2009) Prevalence and distribution of pre-tensioned prestressing steel corrosion in the New Zealand concrete bridge stock. *The New Zealand Concrete Industry Conference, Rotorua*, 8–10 October 2009.

Steel Construction New Zealand (SCNZ) (2007) *Steel connect*, *SCNZ 14.1*. Manukau City, New Zealand: SCNZ.

Tata Steel International (2005) *ComFlor 80.* Accessed 8 June 2013. www.tatasteelnz.com/ComFlor_80.asp

Tviet, P (2008) *The network arch – bits of manuscript in September 2008 after lectures in 50 countries.* Accessed 27 April 2013. http://home.uia.no/pert/backup/documents/The_Network_Arch_-_Fall_2008.pdf

Vianna, J, L Costa-Neves, P Vellasco and S Andrade (2008) *An experimental evaluation of composite structure perforated shear connectors*; Steel Bridges Advanced Solutions and Technologies. 7th International Conference on Steel bridges. ECCS. Europe. II-125-II-132

Yandzio, E and DC Iles (2004) *Precast concrete decks for composite highway bridges, SCI P316*. Ascot, UK: The Steel Construction Institute.

Standards cited in this report

British Standards Institution (BSI) (1980) *BS 5400-10 Steel, concrete and composite bridges. Code of practice for fatigue*. London: BSI.

British Standards Institution (BSI) (2000) *BS 5400-3 Steel, concrete and composite bridges. Code of practice for design of steel bridges*. London: BSI.

British Standards Institution (BSI) (2005) *BS 5400-5 Steel, concrete and composite bridges. Code of practice for design of composite bridges.* London: BSI.

British Standards Institution (BSI) (2006) *BS 5400-2 Steel, concrete and composite bridges part 2: specification for loads*. London: BSI.

British Standards Institution (BSI) (2008) *PD 6695-2 Recommendations for the design of bridges to BS EN 1993.* London: BSI.

British Standards Institution (BSI) (2011) *PD 6694-1 Recommendations for the design of structure subject to traffic loading to BS En 1997-1*. London: BSI.

European Committee for Standardization (CEN) (2003) *BS EN 1991–2 Eurocode 1: Actions on structures.* Brussels: CEN.

European Committee for Standardization (CEN) (2005a) *EN 1993-1–9 Eurocode 3: Design of steel structures – part 1–9: fatigue*. Brussels: CEN.

European Committee for Standardization (CEN) (2005b) *EN 1994-2 Eurocode 4: Design of composite steel and concrete structures – part 2: general rules and rules for bridges*. Brussels: CEN.

European Committee for Standardization (CEN) (2006) *EN 1993-2 Eurocode 3: Design of steel structures – part 2: steel bridges*. Brussels: CEN.

Highways Agency (2001) *BD 49/01 Design rules for aerodynamic effects on bridges*. London: Highways Agency, Department for Transport. Accessed 27 April 2013. www.dft.gov.uk/ha/standards/dmrb/vol1/section3/bd4901.pdf

Highways Agency (2006) *TD 19/06 Requirement for road restraint systems.* London: Highways Agency, Department for Transport. Accessed 27 April 2013. www.dft.gov.uk/ha/standards/dmrb/vol2/section2/td1906.pdf

Highways Agency (2010) *BD 37/01 Loads for highway bridges*. London: Highways Agency, Department for Transport. Accessed 27 April 2013. www.standardsforhighways.co.uk/dmrb/vol1/section3/bd3701.pdf

International Organization for Standardization (ISO) (2012) *ISO 9224 Corrosion of metals and alloys – corrosivity of atmospheres – Guiding values for the corrosivity categories*. Geneva: ISO.

Standards Australia (SA) (1998) *AS 3828 Guidelines for the erection of building steelwork*. Sydney: Standards Australia.

Standards Australia (SA) (2004) *AS 5100 Bridge standard. Including amendment no.1 (2010).* Sydney: Standards Australia. .

Standards New Zealand (SNZ) (1994) *AS/NZS 1554.6 Structural steel welding – welding stainless steels for structural purposes.* Wellington: SNZ.

Standards New Zealand (SNZ) (1996) *AS/NZS 1252 High-strength steel bolts with associated nuts and washers for structural engineering.* Wellington: SNZ.

Standards New Zealand (SNZ) (1997a) *NZS 3109 Concrete construction*. Wellington: SNZ.

Standards New Zealand (SNZ) (1997b) *NZS 3404 Steel structure standard, parts 1 and 2*. Wellington: SNZ.

Standards New Zealand (SNZ) (2002) *AS/NZS 2312 Guide to the protection of structural steel against atmospheric corrosion by the use of protective coatings, incorporating amendment no.1:2004*. Wellington: SNZ.

Standards New Zealand (SNZ) (2003a) *AS/NZS 1170.3 Structural design actions – snow and ice actions*. Wellington: SNZ.

Standards New Zealand (SNZ) (2003b) *NZS 3104 Specification for concrete production*. Wellington: SNZ.

Standards New Zealand (SNZ) (2004a) *AS/NZS 1170.5 Earthquake actions – New Zealand*. Wellington: SNZ.

Standards New Zealand (SNZ) (2004b) *AS/NZS 1554 Structural steel welding set*. Wellington: SNZ.

Standards New Zealand (SNZ) (2004c) *AS/NZS 1554.1 Structural steel welding – welding of steel structures*. Wellington: SNZ.

Standards New Zealand (SNZ) (2004d) *AS/NZS 1554.5 Structural steel welding – welding of steel structures subject to high levels of loading.* Wellington: SNZ.

Standards New Zealand (SNZ) (2004e) *NZS 1170.5 Structural design actions – earthquake actions – New Zealand*. Wellington: SNZ.

Standards New Zealand (SNZ) (2006) *NZS 3101 Concrete structures standard*. Wellington: SNZ.

Standards New Zealand (SNZ) (2009) *NZS 3404 Steel structure standard, part 1.* Wellington: SNZ.

Standards New Zealand (SNZ) (2010a) *AS/NZS 3679.1 Structural steel - hot-rolled bars and sections*. Wellington: SNZ.

Standards New Zealand (SNZ) (2010b) *AS/NZS 3679.2 Structural steel – welded I sections*. Wellington: SNZ.

Appendix A: Two-lane, three and four girder bridge costs

A1 How to achieve a cost-effective steel-concrete composite bridge

While the following guidance is for short span bridges, the design philosophy is applicable to any span length. This section starts by providing a summary of current international preliminary design methods and the current design and construction practice in New Zealand, followed by a study on the cost breakdown of a steel-concrete composite bridge. A cost-effective superstructure configuration is determined and a cost comparison is given in an example from two real bridge projects where a steel-concrete composite bridge option was found to be more cost effective than a conventional concrete bridge. Costs are given for the superstructure and substructure as well as the coating maintenance.

A2 Current short span design practice

The American *Designing short bridges* (AISI 2001) provides complete guidance on the number of girders, girder section size, spacing and other required data for spans from 6m to 36m and decking width from 7m to 13m in a table format. While, the British Tata Steel publication (Hayward 2002) provides the guidance in a chart format for spans up to 60m and decking width up to 16m.

Previous New Zealand guidance *Standard bridge design* (Ministry of Works 1981) followed the American format; however, it specifies structural forms and solutions which are either now not available or cost-effective and it is currently out of print, while Australia's (Rapattoni et al 1998) follows the British format.

The main difference in scope and format is that the American solution provides a complete design solution while the British charts provide preliminary sizing and require detailed design. The common aspect of both these publications is that the substructure and the different decking options are not considered. Work undertaken by El Sarraf and Clifton (2008) has shown that these aspects have a major effect on the cost (as shown in the next section) and should be considered early in the design stage.

A3 Cost breakdown through an example

A3.1 Design parameters

The following design parameters and assumptions were used for the design example. Costs are also included.

A3.1.1 Deck widths and different deckings

The deck width was determined in accordance with the *Bridge manual* therefore, for a two-lane bridge with a medium volume of average annual daily traffic (AADT) of 4000 vehicles, the width is 9.7m. A single lane 6.2m wide option was also researched but the results are not included in this guide due to space limitations. The full research findings are found in El Sarraf and Clifton (2008).

Three types of decking types were considered, namely:

- in situ concrete deck
- full depth precast concrete deck
- partial depth precast concrete deck.

During the analysis of the bridge, each deck type incorporated the appropriate construction live load, as shown in table A.1. For the precast decking construction cases, two deck placement options were used, with the corresponding construction live loads. The first was based on the units being installed by crane and the second by mobile excavator with lifting arm, requiring the excavator to drive across the bridge to place the units. For the shorter spans especially, this second case is relatively common and imposes higher constructional live loading, resulting in the need for larger beam sizes.

A3.1.2 Span

A total of five spans were considered to determine the difference in the main girder dimensions. The spans chosen were 12m, 15m, 21m, 25m, 27m and 30m, which were selected based on feedback from designers and fabricators. These are the most common spans and come in increments of 3m, as this matches the available lengths of plate and avoids over-sizing.

A4 Load cases

All the bridge options have the same imposed vertical loadings, which are presented in table A.1.

Table A.1 Imposed vertical loadings on bridge

Item	Dimensions	Load
Steel girders	Dependent on the span	
Deck	200mm thick	4.8kN/m2
Superimposed load	-	1.5kN/m2
Construction live load (in situ decking)	-	0.75kN/m2
Construction live load (full depth precast)	-	0.5kN/m2
Construction live load (partial depth precast)		0.75kN/m2
Live load UDL	-	3.5kN/m2
HN point load	-	120kN/axle
HO point load	-	240kN/axle
Excavator point load	-	80kN/axle

Note: The excavator used was a Caterpillar M313D wheel excavator.

A5 Design procedure

Once the factored applied load on each beam was determined, calculation of moments and shears was straightforward. The maximum moment involved the axle loads symmetrically placed about the mid-span, while maximum shear involved the axle loads close to the supports. A number of computer programs were used to assist in the design of the different bridge configurations.

For the three and four girder options, the distribution of deck dead and live loads into the supporting girders had to be considered. This was determined taking into account the position of the load, the longitudinal stiffness of the girders and the transverse stiffness of the deck. The computer analysis program S-Frame (CSC 2006) was used to model the bridge deck and determine the load distribution factors, which are dependent on the load case and the location of the live load point loads and uniformly distributed loads. Once the distribution factors were determined, the loads and moments affecting each girder were calculated and the girder section size was chosen using the New Zealand produced design programs MemDes (New Zealand Steel 2003) for the construction loading and BRIDGENZ (HERA 2006) for the composite loading case (which is now withdrawn).

The section sizes were chosen based on a number of conditions:

1 The strength requirements, both during the construction and when the girders were acting compositely with the decking, were considered, with the larger size chosen for the costing stage.

2 As the number of braces decreases as girder section size increases, this resulted in different options being considered.

3 If the ratio of section design capacity/design action was ≤1.03, that section was chosen

4 In general, a lighter but deeper beam was chosen over a shallower but heavier beam.

A6 Costing used

Table A.2 shows the costing used for the different components. Note that the substructure cost and other related costs other than those outlined in table A.2 were not considered in this study. Costing was obtained from the bridging industry, ie a precast concrete manufacturer, a steel fabricator, coatings supplier and contractor.

Table A.2 Costing used (in August 2008)

Item	Cost
In situ concrete deck	$1800/m^3
Full depth precast concrete deck	$1650/m^3
Partial depth precast concrete deck	$1320/m^3
Coating system (IZS3SB)	$29/m^2
Fabrication cost	$4700/tonne
Delivery, handling and erection cost	$700/tonne
Excavator	$1000/day
Craneage	$4000/day

The in situ concrete decking cost comprised: installed in place concrete, reinforcing bar, formwork and temporary support. The full depth precast cost comprised the installed in-place decking and stitching installation (connecting the decking together) and, finally, the partial depth precast decking cost comprised the installed in-place decking units together with the concrete and the rebar. Rebar quantities were based on 0.3% each way each face, being the minimum required by the *Bridge manual*.

The fabrication cost included the steel plate, shop drawing, welding, cutting and other related fabrication costs; it did not include the protection coating cost, which was calculated separately. The excavator (for installing the precast decking for the 12m and 15m span bridges) and craneage cost assumed a 10m span use per day. The fabrication and delivery, handling and erection cost were added together to keep the number of variables in table A.3 to a minimum. The concrete decking and fabricated steelwork pricing was determined in accordance with common industry practice, which uses the per cubic metre cost for concrete decking and per tonne cost for fabricated steelwork.

An X-brace set consists of two unequal angle sections, four pairs of M20 bolts and a welded plate where the angles meet at the X-point. The assumed coating, selected to give a time to first maintenance of at least 35 years for a location of atmospheric corrosivity category C, was a 125μm of factory applied inorganic zinc silicate (IZS3SB), with cost of NZD$29/$m^2$ (in August 2008). The concrete decking type and X-brace sizes are shown in table A.3.

Table A.3 Concrete deck and X-brace numbering system

Number	Item
1	In situ concrete deck
2	Full depth precast concrete deck with excavator
2*	Full depth precast concrete deck without excavator
3	Partial depth precast concrete deck with excavator
3*	Partial depth precast concrete deck without excavator
1	125x75x8 UA
2	125x75x10 UA
3	125x75x12 UA
4	150x90x12 UA
5	150x100x10 UA
6	150x100x12 UA
7	150x90x10 UA

Table A.4 Two lane, three and four girder bridge costs

Span	Girders	Deck Type	Deck Cost	No of Braces Set	Bracing Size	Choosen Girder Size	Coating Cost	Fab/erecction cost	Decking erection cost	Total cost without deck (incl steelwork)	Cheapest option no deck	Total cost with deck	Cheapest option with deck	Deck % of total cost
12	3	1	$41,904.0	8	1	800 WB 146	$3,513	$32,391	$0	$35,904.54		$77,808.54		53.9%
12	3	2	$38,412.0	8	1	800 WB 146	$3,513	$32,391	$4,800	$35,904.54		$79,116.54		54.6%
12	3	2*	$38,412.0	8	1	800 WB 146	$3,513	$32,391	$1,200	$35,904.54		$75,516.54		52.5%
12	3	3	$30,729.6	8	1	800 WB 146	$3,513	$32,391	$4,800	$35,904.54		$71,434.14		49.7%
12	3	3*	$30,729.6	8	1	800 WB 146	$3,513	$32,391	$1,200	$35,904.54	$32,361.38	$67,834.14	$64,490.98	47.1%
12	3	1	$41,904.0	2	1	800 WB 146	$2,977	$29,385	$0	$32,361.38		$74,265.38		56.4%
12	3	2	$38,412.0	2	1	800 WB 168	$2,998	$33,662	$4,800	$36,659.67		$79,871.67		54.1%
12	3	2*	$38,612.0	2	1	800 WB 146	$2,977	$29,385	$1,200	$32,361.38		$72,173.38		55.2%
12	3	3	$30,729.6	2	1	800 WB 168	$2,998	$33,662	$4,800	$36,659.67		$72,189.27		49.2%
12	3	3*	$30,929.6	2	1	800 WB 146	$2,977	$29,385	$1,200	$32,361.38		$64,490.98		49.8%
12	4	1	$41,904.0	12	1	610 UB 113	$3,648	$33,324	$0	$36,971.40		$78,875.40		53.1%
12	4	2	$38,412.0	12	1	610 UB 113	$3,648	$33,324	$4,800	$36,971.40		$80,183.40		53.9%
12	4	2*	$38,612.0	12	1	610 UB 113	$3,648	$33,324	$1,200	$36,971.40		$76,783.40		51.8%
12	4	3	$30,729.6	12	1	610 UB 113	$3,648	$33,324	$4,800	$36,971.40		$72,501.00		49.0%
12	4	3*	$30,929.6	12	1	610 UB 113	$3,648	$33,324	$1,200	$36,971.40	$33,406.21	$69,101.00	$65,535.81	46.5%
12	4	1	$41,904.0	3	1	610 UB 113	$3,108	$30,298	$0	$33,406.21		$75,310.21		55.6%
12	4	2	$38,412.0	3	1	700 WB 115	$3,472	$30,824	$4,800	$34,296.48		$77,508.48		55.8%
12	4	2*	$38,612.0	3	1	610 UB 113	$3,108	$30,298	$1,200	$33,406.21		$73,218.21		54.4%
12	4	3	$30,729.6	3	1	700 WB 115	$3,472	$30,824	$4,800	$34,296.48		$69,826.08		50.9%
12	4	3*	$30,929.6	3	1	610 UB 113	$3,108	$30,298	$1,200	$33,406.21		$65,535.81		49.0%
15	3	1	$52,380.0	8	1	1000 WB 215	$4,858	$56,300	$0	$61,158.14		$113,538.14		46.1%
15	3	2	$48,015.0	8	1	1000 WB 215	$4,858	$56,300	$6,000	$61,158.14		$115,173.14		46.9%
15	3	2*	$48,215.0	8	1	1000 WB 215	$4,858	$56,300	$1,500	$61,158.14		$110,873.14		44.8%
15	3	3	$38,412.0	8	1	1000 WB 215	$4,858	$56,300	$6,000	$61,158.14		$105,570.14		42.1%
15	3	3*	$38,612.0	8	1	1000 WB 215	$4,858	$56,300	$1,500	$61,158.14	$57,573.97	$101,270.14	$97,685.97	39.6%
15	3	1	$52,380.0	2	1	1000 WB 215	$4,315	$53,259	$0	$57,573.97		$109,953.97		47.6%
15	3	2	$48,015.0	2	1	1000 WB 215	$4,315	$53,259	$6,000	$57,573.97		$111,588.97		48.4%
15	3	2*	$48,215.0	2	1	1000 WB 215	$4,315	$53,259	$1,500	$57,573.97		$107,288.97		46.3%
15	3	3	$38,412.0	2	1	1000 WB 215	$4,315	$53,259	$6,000	$57,573.97		$101,985.97		43.5%
15	3	3*	$38,612.0	2	1	1000 WB 215	$4,315	$53,259	$1,500	$57,573.97		$97,685.97		41.1%
15	4	1	$52,380.0	12	1	800 WB 146	$5,397	$51,416	$0	$56,812.67		$109,192.67		48.0%
15	4	2	$48,015.0	12	1	800 WB 146	$5,397	$51,416	$6,000	$56,812.67		$110,827.67		48.7%
15	4	2*	$48,215.0	12	1	800 WB 146	$5,397	$51,416	$1,500	$56,812.67		$106,527.67		46.7%
15	4	3	$38,412.0	12	1	800 WB 146	$5,397	$51,416	$6,000	$56,812.67		$101,224.67		43.9%
15	4	3*	$38,612.0	12	1	800 WB 146	$5,397	$51,416	$1,500	$56,812.67	$53,178.57	$96,924.67	$93,290.57	41.4%
15	4	1	$52,380.0	3	1	800 WB 146	$4,847	$48,332	$0	$53,178.57		$105,558.57		49.6%
15	4	2	$48,015.0	3	1	900 WB 175	$5,364	$57,740	$6,000	$63,103.77		$117,118.77		46.1%
15	4	2*	$48,215.0	3	1	800 WB 146	$4,847	$48,332	$1,500	$53,178.57		$102,893.57		48.3%
15	4	3	$38,412.0	3	1	900 WB 175	$5,364	$57,740	$6,000	$63,103.77		$107,515.77		41.3%
15	4	3*	$38,612.0	3	1	800 WB 146	$4,847	$48,332	$1,500	$53,178.57		$93,290.57		43.0%
21	3	1	$73,332	8	2	1200 WB 278	$7,506	$99,512	$0	$107,018.79		$180,350.79		40.7%
21	3	2	$67,221	8	2	1200 WB 278	$7,506	$99,512	$8,400	$107,018.79		$182,639.79		41.4%
21	3	3	$53,777	8	2	1200 WB 278	$7,506	$99,512	$8,400	$107,018.79	$105,601.62	$169,195.59	$167,778.42	36.7%
21	3	1	$73,332	6	2	1200 WB 278	$7,323	$98,278	$0	$105,601.62		$178,933.62		41.0%
21	3	2	$67,221	6	2	1200 WB 278	$7,323	$98,278	$8,400	$105,601.62		$181,222.62		41.7%
21	3	3	$53,777	6	2	1200 WB 278	$7,323	$98,278	$8,400	$105,601.62		$167,778.42		37.1%
21	4	1	$73,332	12	1	1200 WB 249	$9,072	$117,261	$0	$126,332.16		$199,664.16		36.7%
21	4	2	$67,221	12	1	1200 WB 249	$9,072	$117,261	$8,400	$126,332.16		$201,953.16		37.4%
21	4	3	$53,777	12	1	1200 WB 249	$9,072	$117,261	$8,400	$126,332.16	$125,061.19	$188,508.96	$187,237.99	33.0%
21	4	1	$73,332	9	1	1200 WB 249	$8,879	$116,182	$0	$125,061.19		$198,393.19		37.0%
21	4	2	$67,221	9	1	1200 WB 249	$8,879	$116,182	$8,400	$125,061.19		$200,682.19		37.7%
21	4	3	$53,777	9	1	1200 WB 249	$8,879	$116,182	$8,400	$125,061.19		$187,237.99		33.2%
27	3	1	$94,284	8	7	1200 WB 423	$10,956	$191,044	$0	$202,000.29		$296,284.29		31.8%
27	3	2	$86,427	8	7	1200 WB 423	$10,956	$191,044	$10,800	$202,000.29		$299,227.29		32.5%
27	3	3	$69,142	8	7	1200 WB 423	$10,956	$191,044	$10,800	$202,000.29	$200,310.98	$281,941.89	$280,252.58	28.4%
27	3	1	$94,284	6	7	1200 WB 423	$10,773	$189,538	$0	$200,310.98		$294,594.98		32.0%
27	3	2	$86,427	6	7	1200 WB 423	$10,773	$189,538	$10,800	$200,310.98		$297,537.98		32.7%
27	3	3	$69,142	6	7	1200 WB 423	$10,773	$189,538	$10,800	$200,310.98		$280,252.58		28.5%
27	4	1	$94,284	12	1	1200 WB 278	$12,383	$166,444	$0	$178,826.93		$273,110.93		34.5%
27	4	2	$86,427	12	1	1200 WB 278	$12,383	$166,444	$10,800	$178,826.93		$276,053.93		35.2%
27	4	3	$69,142	12	1	1200 WB 278	$12,383	$166,444	$10,800	$178,826.93	$177,555.96	$258,768.53	$257,497.56	30.9%
27	4	1	$94,284	9	1	1200 WB 278	$12,191	$165,365	$0	$177,555.96		$271,839.96		34.7%
27	4	2	$86,427	9	1	1200 WB 278	$12,191	$165,365	$10,800	$177,555.96		$274,782.96		35.4%
27	4	3	$69,142	9	1	1200 WB 278	$12,191	$165,365	$10,800	$177,555.96		$257,497.56		31.0%
30	3	1	$104,760	8	2	1200 WB 636	$14,115	$313,462	$0	$327,576.62		$432,336.62		24.2%
30	3	2	$96,030	8	2	1200 WB 636	$14,115	$313,462	$12,000	$327,576.62		$435,606.62		24.8%
30	3	3	$76,824	8	2	1200 WB 636	$14,115	$313,462	$12,000	$327,576.62	$326,323.36	$416,400.62	$415,147.36	21.3%
30	3	1	$104,760	6	2	1200 WB 636	$13,953	$312,370	$0	$326,323.36		$431,083.36		24.3%
30	3	2	$96,030	6	2	1200 WB 636	$13,953	$312,370	$12,000	$326,323.36		$434,353.36		24.9%
30	3	3	$76,824	6	2	1200 WB 636	$13,953	$312,370	$12,000	$326,323.36		$415,147.36		21.4%
30	4	1	$104,760	12	4	1200 WB 392	$16,181	$256,090	$0	$272,270.21		$377,030.21		27.8%
30	4	2	$96,030	12	4	1200 WB 392	$15,944	$262,785	$12,000	$278,728.58		$386,758.58		27.9%
30	4	3	$76,824	12	4	1200 WB 392	$15,944	$262,785	$12,000	$278,728.58	$272,270.21	$367,552.58	$365,146.75	24.2%
30	4	1	$104,760	9	4	1200 WB 392	$15,730	$260,593	$0	$276,322.75		$381,082.75		27.5%
30	4	2	$96,030	9	4	1200 WB 392	$15,730	$260,593	$12,000	$276,322.75		$384,352.75		28.1%
30	4	3	$76,824	9	4	1200 WB 392	$15,730	$260,593	$12,000	$276,322.75		$365,146.75		24.3%

Appendix B: Available plate dimensions

B1 New Zealand-made plates

□ AS/NZS 1594 Coil plate ✓ AS/NZS 3678 Plate (shaded) NZS Non Standard Thickness

Grade	LengthxWidth (mm)	Thickness															
		5	6	8	10	12	14	16	18	20	22	25	28	32	40	45	50
G300Mod*	6000x1230	□	□	□	□	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
	6000x1530	□	□	□	□	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
	9000x1230	□	□	□	□	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
	9000x1530	□	□	□	□	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
	12000x1230	NA	NA	NA	NA	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
	12000x1530	NA	NA	NA	NA	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
Mill Lead Time 6 weeks, all thicknesses																	
G350	6000x1230	□	□	□	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	NA	NA
	6000x1530	□	□	□	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	NA	NA
	9000x1230	□	□	□	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	NA	NA
	9000x1530	□	□	□	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	NA	NA
	12000x1230	NA	NA	NA	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	NA	NA
	12000x1530	NA	NA	NA	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	NA	NA
Mill Lead Time 6 weeks, all thicknesses																	
G300Mod L0 and L15	6000x1230	□	□	□	□	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
	6000x1530	□	□	□	□	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
	9000x1230	□	□	□	□	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
	9000x1530	□	□	□	□	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
	12000x1230	NA	NA	NA	NA	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
	12000x1530	NA	NA	NA	NA	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
Mill Lead Time 6 weeks, all thicknesses																	
G350 L0 and L15	6000x1230	□	□	□	□	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	NA	NA
	6000x1530	□	□	□	□	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	NA	NA
	9000x1230	□	□	□	□	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	NA	NA
	9000x1530	□	□	□	□	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	NA	NA
	12000x1230	NA	NA	NA	NA	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	NA	NA
	12000x1530	NA	NA	NA	NA	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	NA	NA
Mill Lead Time 6 weeks, all thicknesses																	

NOTES: Lengiths in range <10m for coil plate and between 2.4m and 13m for AS/NZS 3678 plate, in minimum order quanity.
Mill minimun order item quantity for AS/NZS 1594 Coil Plate is the outturn of a slab (approx 14t, 1230 wide and 18t, 1530 wide)
Mill minimun order item quantity for AS/NZS 3678 Plate is approximately 6 tonne.
≤10mm L0 and L15 tested on subsize Charpy test piece
*When the new AS/NZS 3678 standard is published the mod requirement due to vanadium content will not be required

B2 Commonly imported plates

Grade	LengthxWidth (mm)	Thickness																		
		5	6	8	10	12	16	20	25	32	40	50	60	65	80	100	120	130	150	200
G250/ G300	2400x1220	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓								
	2700x2400																		✓	
	3000x1520	✓	✓	✓	✓	✓														
	3100x2400																	✓		
	3400x2400																✓			
	3600x1520	✓	✓	✓	✓	✓	✓	✓	✓											
	3600x1800	✓		✓	✓															
	4000x2400															✓				
	5200x2400														✓					✓
	6000x1520			✓	✓															
	6000x1800		✓																	
	6000x2400			✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓						
	7600x2400				✓															
	9000x2400			✓	✓	✓	✓	✓	✓											
Contact the Steel Merchant for expected Lead Time																				
G350	6000x2000		✓	✓	✓	✓	✓	✓	✓	✓	✓									
	6000x2400	✓	✓									✓								
Contact the Steel Merchant for expected Lead Time																				

Note that other plate thicknesses (up to 400 mm), dimensions (width in 100 mm increments up to 5m wide and length up to 25m) and types (weathering steel, L0, L5 and L15, etc) are available. Contact the steel merchant and/or fabricator for expected lead times. Minimum tonnages may apply.

Appendix C: Initial sizing of main girders

The following very basic guidance is offered to give a rough first estimate of girder size for a multi-girder bridge using plate girders. It is intended only for use where the basic configuration has already been selected and does not give any indication of the bracing needed to stabilise the girders during construction or in the final condition. It allows the process of analysis, verification and refinement to commence; it is not intended for determining quantities for cost estimation.

Reference to loads should be taken to be the design values, ie after the appropriate load factors on actions have been applied.

At an intermediate support, the maximum shear will occur when a normal load (HN) axle load is positioned in the lane directly over a girder, with the other axle load located on the longer span side. The adjacent lane should have the overload (HO) axles in a similar position. The shear can then be estimated as the sum of components from the HN and HO loads and accompanying UDL load over the span (on the longer side). For the first two axle loads, use the following factors representing the proportion of load carried at the end support of a simply supported span, both longitudinally and transversely (eg for a HN axle load 60% for its position along the span and 90% for a lane that is almost directly above the girder, giving a shear of 54% of the weight of the HN axle load being carried by the girder). For the UDL loads, use 70% of the load in the span times the proportion for lane position.

The maximum moment at an intermediate support occurs with a different distribution of load but the worst values can be assumed to coexist. Position the HN axle loads straddling the support and determine the moment at the end of the longer span (with half the HN axle load on it) assuming it to be a fixed ended span; multiply by a lane position factor, as above. For the HO axle load, position it at ¼ way into the longer span and determine the fixed end moment; multiply by the lane factor. For the accompanying UDL, determine the fixed end moment in the longer span and multiply by the lane position factor.

Size the web adjacent to the support so that it can carry 150% of the governing shear (the reserve is valuable in carrying dead load and contributing to bending resistance). If the bottom flange is inclined, it will carry some of the shear and the web can be reduced in thickness accordingly.

Determine a force in the bottom flange adjacent to the support by dividing the total moment by the distance between the flange and the slab. Size the bottom flange so that the stress is 80% of the yield strength (the 20% allows for dead load and a small reduction for slenderness). Choose a top flange that has an area of 60% of the bottom flange.

In mid-span, determine moments due to the same three components, with the HN and HO axle loads at mid-span; consider the span as simply supported. Consider that 40% of the total moment is carried by a single girder (this makes some allowance for continuity and assumes transverse sharing between girders).

In mid-span, provide a web that has an area of 60% of that at supports and size the bottom flange to carry a force of the total moment divided by the depth between the flange and the middle of the slab, at a stress level of 95% of the yield strength. Choose a top flange area that is 80% of that of the bottom flange (this will be needed for stability during construction).

Compression flanges should always be proportioned so that they are at least non-compact (for grade 350 steel, limit the outstand/thickness ratio to 11.2). Tension flanges should be limited to an outstand/thickness ratio of 20, for robustness (but if they could go into compression during construction, comply with the non-compact limit).

Appendix D: Background to the fatigue design criteria for bridges

Taken from *Recommended draft fatigue design criteria for bridges* (Clifton 2007a).

The fatigue loading in *AS 5100.2* (SA 2004) has been developed to be consistent with the M1600 traffic design live loading. It is based on the same family of future vehicles determined for Australia. In applying it to New Zealand, expert assessment by Dr Clifton has been made to amend the provisions suitable for New Zealand conditions.

Fatigue is the process of cumulative local structural deterioration of a component or connection when subjected to repeated, fluctuating stresses. In bridges, it is normally caused by the repeated loading of a structure by moving vehicles and occurs over an extended period of time. While fatigue damage in main structural members is of principal concern, distortion or distortion restraint induced fatigue damage has also been reported in a number of instances in secondary bracing members or their connections (Fisher 1977).

Fatigue damage is commonly exhibited as cracks or fracture of the component material and usually occurs at locations of stress concentration. The fatigue criteria presented below and discussed in *AS 5100.5* and *AS 5100.6* (SA 2004), have been developed with the intention of limiting the development and propagation of cracks under repeated loading to prevent fracture during the life of the structure. The fatigue behaviour of steel structures, including composite steel and concrete structures, is relatively well understood and direct reference may be made to current international best practice and standards. The fatigue behaviour of concrete is not as well defined. Reference has been made to Eurocode and recent Swiss research. Reference should be made to the *AS 5100.5* and *AS 5100.6* commentaries for further details of materials aspects of the fatigue criteria.

In developing the fatigue design criteria, a fatigue life of 75 years has been assumed. This approach is compatible with a 100-year design life on the basis that:

- Bridges will be inspected regularly and that intervention will occur when fatigue damage is detected. This process should ensure that fatigue damage is controlled and this has been shown to be the case in practice.
- The uncertainty associated with the prediction of the fatigue life is such that only a relatively low percentage of bridges for which the theoretical design fatigue life has been reached actually exhibit fatigue damage.

Initial work on developing fatigue loading criteria concentrated on the use of specific vehicles to represent the range of typical future freight vehicles. This approach was replaced in *AS 5100.2* by the use of the A160 and M1600 truck load for Australia and New Zealand to remove the requirement to introduce further design vehicle loads. In the New Zealand application this involves having to use an axle load case specific for fatigue assessment. This is unavoidable and is consistent with future intentions to align the New Zealand and Australian bridge loadings and design standards more closely. In both instances, assumptions have been made about the rate of increase in freight task and corresponding increase in heavy vehicle volumes and mass together with assumptions about commercial vehicle configurations on four principal route types.

When determining the basic fatigue design loads for New Zealand, the recommendations from Beamish et al (2006) and subsequent discussions with the authors is that the Australian fatigue design provisions are appropriate for New Zealand use (based on a limited number of weigh stations) but with the route factors given for the closest equivalent Australian class of road used and multiplied by 0.9. This has been done below.

This approach has removed the need for the designer to determine the number and amplitude of the stress cycles due to the passage of actual trucks. In reality the passage of a single vehicle over a bridge produces a number of cycles of stress of varying amplitude. The cumulative fatigue damage of this stress history is converted to the equivalent damage of one cycle of amplitude $(f^*_{equiv})^m$, where:

$$\Sigma n_i \left(f_i^*\right)^m = n_{equiv}\left(f^*_{equiv}\right)^m \qquad \text{(Equation D.1)}$$

Where:

n_{equiv} = 1.0.

The design cumulative damage caused by the passage of the assumed mix, mass and number of vehicles is expressed as an equivalent number of cycles of 0.70×(A160 axle load) × (1 + α), in the case of very short spans or 0.95×(HN moving traffic load without UDL) × (1 + α).

The corresponding damage caused by a single passage of the HN truck without UDL or the A160 axle load is $(\phi M1600)^m$ or $(\phi A160)^m$.

Thus the design fatigue damage can be expressed as $\Sigma nA160\times(f^*_{0.7A160})^m$ or $\Sigma nM1600\times(f^*_{0.7M1600})^m$, where:

nA160 is the number of fatigue stress cycles for the A160 axle load

nM1600 is the number of fatigue stress cycles for the M1600 lane load

$(f^*_{0.7A160})^m$ = equivalent cumulative fatigue function for 0.70×(A160 axle load) × (1 + α)

$(f^*_{0.7M1600})^m$ = equivalent cumulative fatigue function for 0.70×(M1600 moving traffic load without UDL) × (1 + α).

The design number of fatigue cycles is applicable to both simply supported, continuous and cantilever spans.

In determining the maximum stress range in the component under consideration, only 70% of the effect of the basic load is used. This reduction has been introduced to take the following into consideration:

- The actual stresses in a component are generally less than the theoretically calculated values because of alternative load paths, (such as bridge barriers) and the magnitude of actual components in comparison with line elements used to represent them in analysis;
- The actual lateral position of heavy vehicles varies and does not generally coincide with the critical lateral position.

In general terms for Australia, the mass of heavy vehicles is assumed to increase from current values to that equivalent to SM1600 vehicles over a period of about 50 years. Emphasis is placed on the average mass of vehicles rather than maximum allowable mass as the percentage of vehicles loaded to maximum allowable mass may be expected to decrease as the allowable mass is increased over time and volume tends to control. The compound rate of increase of freight is assumed to be about 5% on major interstate routes down to only about 1% on lesser routes. A saturation volume of between 4000 and 4500 heavy vehicles per lane per day is also assumed. These assumptions are made for New Zealand.

For the determination of the number of fatigue stress cycles, a route factor has been introduced to take into consideration the average mass of heavy freight vehicles on different types of routes. In adapting this for New Zealand, the classes of roads used here are specified from the *Bridge manual* and the route factors used in *AS 5100* (SA 2004) are reduced by a factor of 0.9. Vehicles travelling on national highways and motorways are generally maximum sized legal vehicles, fully laden and include a high proportion of medium vehicles. Vehicles travelling on major urban roads are more likely not to be fully loaded, with many empty. Vehicles on other regional roads are less likely to be maximum sized than those on the national road network. Vehicles on urban roads other than motorways are likely to be shorter and lighter than heavy vehicles on other types of roads. Considerable use has been made of weigh-in-motion data from Australia in determining the different route factors and limited calibration made with weigh station data from New Zealand.

General traffic data has been used for Australia to determine the presence of heavy vehicles in different lanes of multi-lane roads. This data indicates that for rural roads more than 95% of all heavy vehicles travelled in the slow lane. For urban roads, particularly for freeways with frequent on and off ramps, a maximum of about 65% of heavy vehicles travelled in a single lane. This data has been used in specifying the number of heavy vehicles per day per lane to be used based on the total number of heavy vehicles travelling in that direction. The applicability of this for New Zealand needs to be confirmed or recommended modifications made.

Guidance on the number of trucks per lane for the different classes of road are available from the NZTA in New Zealand for the national roads.

The final aspect is the source of the dynamic load allowance. This is specified in table 6.7.2 of *AS 5100.2* for the Australian loads.

Appendix E: Single span ladder deck bridge

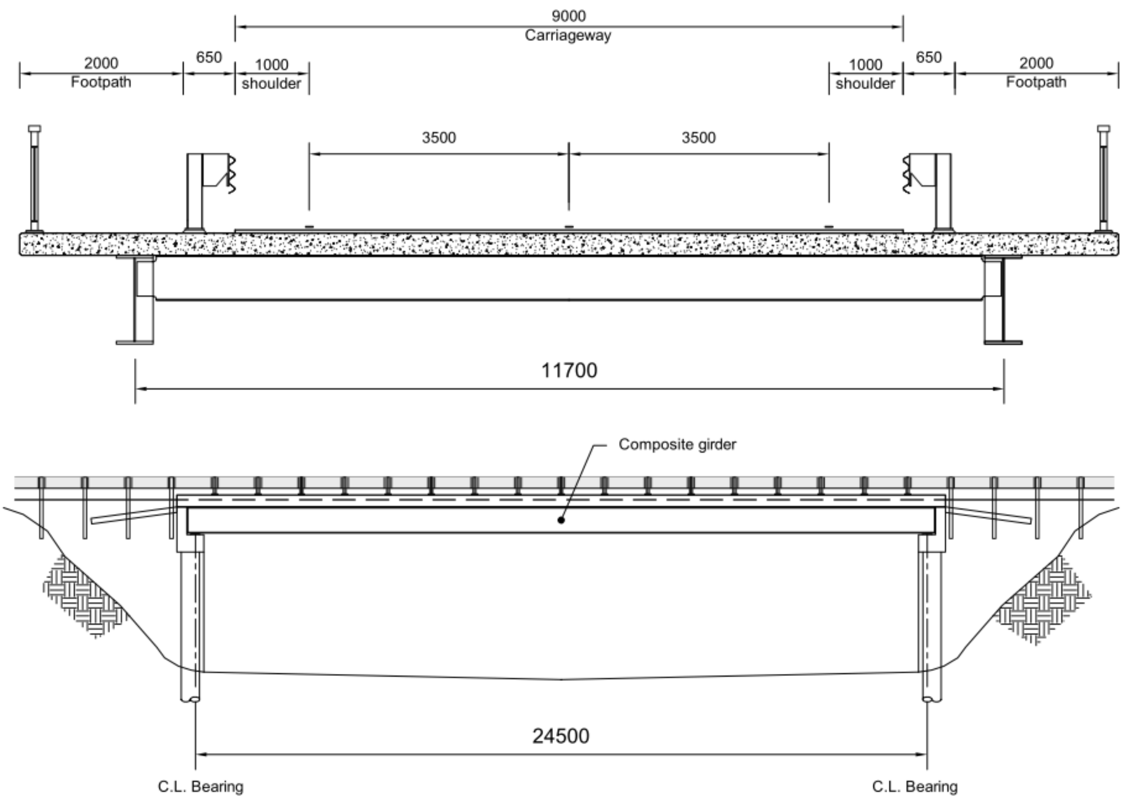
E1 Design statement

- The bridge carries a two-lane (each 3.5m wide) single carriageway road over a flood plain.
- The carriageway has two 1m wide shoulders and has 2m wide footways on either side.
- Speed environment is signed as a maximum of 50km/h but a design threshold of 60km/h is used.
- For barrier design the average annual daily traffic (AADT) is 15,000 vehicles per day with 20% commercial vehicles. An offset of 1.2m is assumed. Thus, a G9 modified TL4 Thrie beam is considered.
- The bridge clearance is minimum 1.5m from the soffit to design flood level, therefore flood load does not need to be considered.
- The bridge is to be designed in accordance with following standards:
  - *Bridge manual* (NZTA 2012)
  - *AS 5100.6* (SA 2004)
  - *NZS 3101* (NZS 2006).
- Structural material properties:
  - structural steel: grade 300
  - concrete: C40
  - steel reinforcement: grade 500.
- Environmental exposure classification for superstructure = A2 – inland, exterior.
- Density of steel = 78.5kN/m^3.
- Density of reinforced concrete = 25kN/m^3.
- Nominal thickness of surfacing = 130mm.
- Density of surfacing = 23kN/m^3.
- Steel parapet selfweight = 2kN/m for both TL4 and pedestrian barriers.
- The construction load is 0.75kN/m^2 and the weight of the temporary formwork is assumed to be 0.5kN/m^2. Additionally, wet concrete is assumed to have a density of 1kN/m^3 greater than that of hardened concrete.
- Pedestrian live load = 5kPa. This value can be reduced when considered in conjunction with road traffic loading.
- *Draft fatigue design criterion for bridges* (Clifton 2007a) is recommended for fatigue assessment.
- Coefficient of linear thermal expansion = 11.7x10^{-6} per °C.
- Characteristic value of shrinkage is $\varepsilon_{sh} = 0.0002$

E2 Structural arrangement

The bridge carries a two-lane single carriageway road over a flood plain. The carriageway has 1m wide shoulders and has a 2m wide footway on either side. A ladder deck girder arrangement has been chosen and a deck slab of 250mm has been assumed. The deck cantilevers 1.3m outside the centrelines of the outer girders. The bridge is assumed to have 24.5m span with pinned end supports.

Figure E.1 Ladder deck bridge structural arrangement



E3 Design basis

The bridge is to be designed in accordance with the *Bridge manual*, *AS 5100.6* and *NZS 3101*.

The fatigue limit state is verified for the reference stress range due to the load application based on Clifton (2007a).

Crack widths in the deck slab are verified at the serviceability limit state (SLS) based on *NZS 3101*.

E3.1 Load combinations

Factors and load combinations of actions are given in the *Bridge manual*, tables 3.1 and 3.2 for the SLS and ultimate limit state (ULS) respectively.

E3.2 Factors on strength

The values of various capacity reduction factors (ϕ) for strength limit states are given by *AS 5100.6*, table 3.2 for steel, and *NZS 3101*, clause 2.3.2.2 for concrete.

E3.3 Structural material properties

It is assumed that the following structural material grades will be used:

- structural steel: grade 300
- concrete: C40
- steel reinforcement: grade 500.

For structural steel, the value of f_y depends on the product material

For rolled sections use:

ASI design capacity tables

320MPa for t ≤11mm; 300MPa for 11mm <t ≤17mm; and 280MPa for t >17mm.

For plates use:

320MPa for t ≤8mm; 310MPa for 8mm <t ≤12mm; 300MPa for 12mm <t ≤20mm; and 280MPa for t >20mm.

Note that designers may wish to use grade 350 steel for better economy.

For concrete, $f_c' = 40\text{MPa}$

For steel reinforcement, $f_y = 500\text{MPa}$

The modulus of elasticity for both structural steel and steel reinforcement is taken as $E_s = 200\text{GPa}$

The modulus of elasticity of the concrete is given by *NZS 3101* as:

NZS 3101, clause 5.2.3

$$E_c = \left(3320\sqrt{f_c'} + 6900\right)\left(\frac{\rho}{2300}\right)^{1.5}$$

$$E_c = 32\text{GPa}$$

This 28-day value will be used for determination of all short-term effects and resistance and the modular ratio is thus:

$$n_s = 200/32 = 6.25$$

For long-term effects, the modular ratio is:

$$E_{c,L} = \frac{E_c}{1+\phi_{cc}} = \frac{32}{1+2} = 10.50$$

$$n_L = 200/10.50 = 19$$

The conservative value of ϕ_{cc} is given in National Roads Board (1984) *Road Research Unit bulletin 70* (RRU 70) as 2.00.

E3.4 Durability requirements

E3.4.1 Concrete

Environmental exposure classification

Superstructure: A2 – inland, exterior

Minimum cover to reinforcement

Surfaces in contact with the ground = 75mm

Surfaces with a damp-proof membrane between the ground and the concrete = 50mm

Elsewhere = 40mm.

E3.4.2 Steel

For an appropriate and cost-effective coating system for structural steelwork, the *New Zealand steelwork corrosion and coatings guide* (El Sarraf and Clifton 2011), is used in conjunction with *AS/NZS 2312* (SNZ 2002) *Guide to the protection of structural steel against atmospheric corrosion by the use of protective coatings* and *NZS 3404.1* (SNZ 2009).

E4 Action on the bridge

E4.1 Permanent actions

E4.1.1 Self-weight of structural elements

The density of steel is taken as 78.5kN/m^3 and the density of reinforced concrete as 25kN/m^3. Self-weights are based on nominal dimensions.

E4.1.2 Self-weight of surfacing

The nominal thickness of the surfacing is 130mm. Assume that the density is 23kN/m^3. The self-weight generally produces adverse effects and as a result is based on nominal thickness + 55%. This follows international practice; however, New Zealand practice differs and reference should be made to the *Bridge manual*, section 3.4.2.

$$\mathrm{DL} = 1.55 \times 0.13 \times 23 = 4.63\mathrm{kPa}$$

E4.1.3 Self-weight of footway construction

The nominal thickness of the footway comprising a concrete slab is 250mm and a uniform density of 25kN/m^3 is assumed. The self-weight is based on the nominal dimensions.

E4.1.4 Self-weight of parapets

A nominal value of 2kN/m is assumed for each parapet.

Note that this value will be increased if a solid concrete crash barrier is used.

E4.2 Construction loads

For global analysis, a uniform construction load of 0.75kN/m^2 is assumed during casting. The use of permanent precast planks is assumed and thus there is no extra load for formwork. Additionally, wet concrete is assumed to have a density of 1kN/m^3 greater than that of hardened concrete; for a slab thickness of 250mm this adds 0.25kN/m^2.

The total construction load is thus: $\mathrm{CN} = 0.75 + 0.25 = 1.0\mathrm{kPa}$.

E4.3 Traffic loads

E4.3.1 Road traffic

For the road carried by this bridge, the *Bridge manual* specifies the HN-HO-72 traffic loading but only HN72 has been considered here for simplicity.

E4.3.2 Pedestrian traffic

Pedestrian traffic is represented by the value in the *Bridge manual,* section 3.4.14(b).

$$\mathrm{FP} = 5\mathrm{kPa}$$

This value can be reduced when considered in conjunction with road traffic loading.

E4.3.3 Fatigue load

Clifton (2007a) is recommended for fatigue assessment.

Refer to section 6.4.2 in the main part of the guide

E4.4 Thermal actions

E4.4.1 Overall temperature change

For a change of length in composite sections, the coefficient of linear thermal expansion is 11.7×10^{-6} per °C. This is then used for determining soil pressure on integral bridges or determining the expansion length in the case of simply supported abutments.

E4.4.2 Differential temperature change

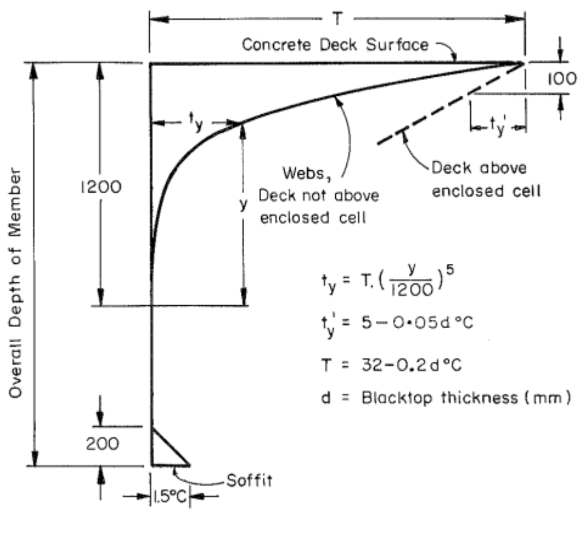
Refer to *Bridge manual*, section 3.4.6

The vertical temperature difference given in the *Bridge manual,* section 3 is used and the temperature difference will be considered to act simultaneously with the overall temperature change.

The effects of vertical temperature gradients are derived for the positive differential temperature conditions (where the top surface is hotter than the average temperature of the superstructure) and for he negative temperature differential conditions (where the top surface is colder than the average temperature of the superstructure).

Note: The negative temperature variation to be considered is the same as that for bridge type 1 from figure 17.3 of *AS 5100.2*.

Figure E.2 Temperature variation with depth



E4.5 Seismic actions

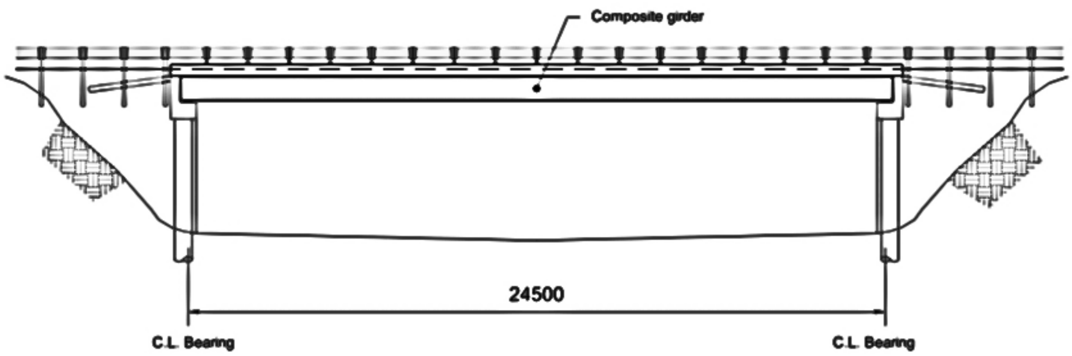
Refer to section 6.5 in the main part of the guide

Section 5 of the *Bridge manual* is used to determine the horizontal force generated by the seismic action on the bridge superstructure. These forces are transferred into the supports (piers and abutments), which are designed to resist this horizontal force (as shown in figure 5.4 of the *Bridge manual).* Therefore, seismic actions acting on the superstructure are ignored as they are minimal as long as the load path is available to transfer these actions into the substructure.

E5 Girder makeup and slab reinforcement

E5.1 Main girder

Figure E.3 Main girder arrangement



	24.50m span girder
Top flange	600 x 25
Web	20
Bottom flange	800x 50
Top rebar	HD20-150
Bottom rebar	HD20-150

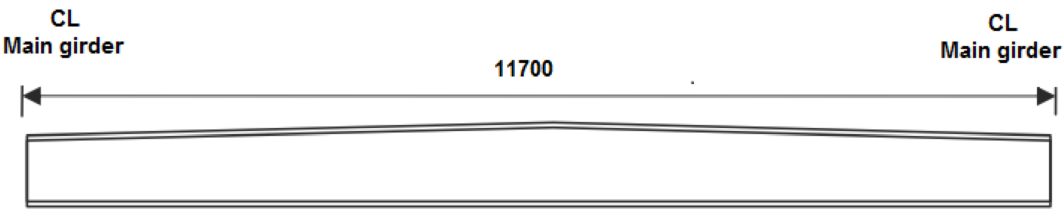
The overall girder depth is 1350mm. Cover to the centroid of the top and bottom reinforcement is 55mm. This complies with *NZS 3101*.

Cross girders are positioned at 3500mm centres and are connected to the main girders by bolting to flat transverse web stiffeners.

One alternative configuration to an in-situ concrete slab is to include precast planks, which typically have prestressing in the bottom, and an in-situ slab with lighter reinforcement.

E5.2 Cross girders

Figure E.4 Cross girder arrangement



Overall depth 750mm at the ends, 896mm at the centre

Flanges: 300 x 25

Web: 16mm

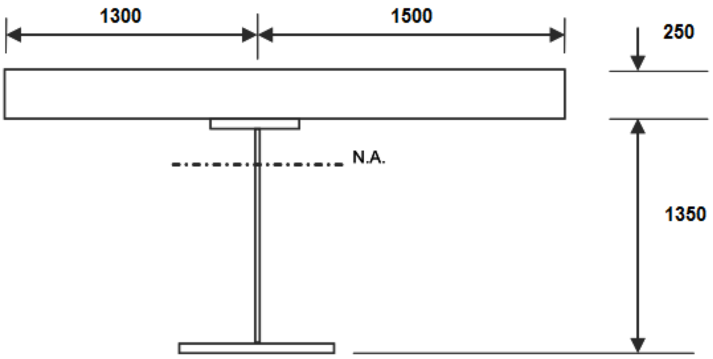
The web is unstiffened, except possibly at the cross girder mid-span if the cross girders need to be braced.

E6 Beam cross section

Refer to section 7.1.1 in the main part of the guide

E6.1 Section properties – main girders

Figure E.5 Slab effective width



For determination of stresses in the cross section and resistances of the cross section, the effective width of the slab, allowing for shear lag is needed. The following calculations summarise the effective section properties for the section considered.

The equivalent span for effective width is:

Refer to AS 5100.6, clause 6.1.7

$$L_C = 24500\text{mm}$$

$$b_{eff} = 1300 + \min\left(\frac{L_C}{10}, \frac{S}{2}, 6t_S\right) = 1300 + \min\left(\frac{24500}{10}, \frac{11700}{2}, 6 \times 250\right) = 2800\text{mm}$$

Refer to AS 5100.6, clause 4.4.1

Properties for the gross section in the mid-span are tabulated below.

Table E.1 Bare steel cross sections

		Span girder	Unit
Area	A	80,500	mm^2
ENA height	ENA	479	mm
Second moment of area	I_x	2.4E+10	mm^4
Section modulus, top flange	$Z_{x,\,tf}$	27E+06	mm^3
Section modulus, bottom flange	$Z_{x,\,bf}$	50E+06	mm^3
Section class		Not compact	
Bending moment capacity	M_{sx}	7560	kN.m

Slenderness check:

Refer to AS 5100.6, clause 5.1.2

$$\lambda_{e,t\_flange} = \frac{b_f - t_w}{2t_f}\sqrt{\frac{f_{y,flange}}{250}}$$

$$\lambda_{e,t\_flange} = \frac{600-20}{2 \times 25}\sqrt{\frac{280}{250}} = 12.28$$

$$\lambda_{e,web} = \frac{d_1}{t_w}\sqrt{\frac{f_{y,web}}{250}}$$

$$\lambda_{e,web} = \frac{1275}{20}\sqrt{\frac{300}{250}} = 69.8$$

$$\lambda_{e_{y,flange}} = 14$$

Refer to AS 5100.6, table 5.1

$$r_e = \frac{(1350 - 479 - 25)}{1275} = 0.66$$

$$\lambda_{e_{y,web}} = \frac{60}{r_e} = \frac{60}{0.66} = 91$$

$$\frac{\lambda_{e,t\_flange}}{\lambda_{e_{y,flange}}} = \frac{12.28}{14} = 0.88$$

$$\frac{\lambda_{e,web}}{\lambda_{e_{y,web}}} = \frac{69.8}{91} = 0.77$$

$$\frac{\lambda_{e,flange}}{\lambda_{e_{y,flange}}} \geq \frac{\lambda_{e,web}}{\lambda_{e_{y,web}}} \rightarrow \text{flange governs}$$

$$\lambda_{e_S} = \lambda_{e_{flange}} = 12.28$$

$$\lambda_{e_p} = 8$$

$$\lambda_{e_S} \geq \lambda_{e_p}$$

Therefore, the bare steel section is **not compact**.

Table E.2 Composite cross section (short term) – sagging (*n_S*=6.25)

		Span girder	Unit
Area	A	192,500	mm^2
ENA height	ENA	1059	mm
Second moment of area	I_x	7.1E+10	mm^4
Section modulus, top of slab	$Z_{x,\ c}$	131E+06	mm^3
Section modulus, top flange	$Z_{x,\ tf}$	243E+06	mm^3
Section modulus, bottom flange	$Z_{x,\ bf}$	67E+06	mm^3
Section class	-	Compact	-
Bending moment capacity	M_{p_l}	22,596	kN.m

The cross section of the span girder is compact, provided the top flange is restrained by shear connectors within the spacing limits given in *AS 5100.6*, clause 6.1.8. (the spacing of the shear stud is more likely to be 150mm).

Plastic moment capacity for a compact composite section can be calculated using the formula given in *AS 5100.6*, appendix E.

$$d_h = \frac{f_y A}{0.85 f_c^{'} b}$$

$$d_h = \frac{280 \times 80500}{0.85 \times 40 \times 2800} = 237\text{mm}$$

$$d_h < d_s$$

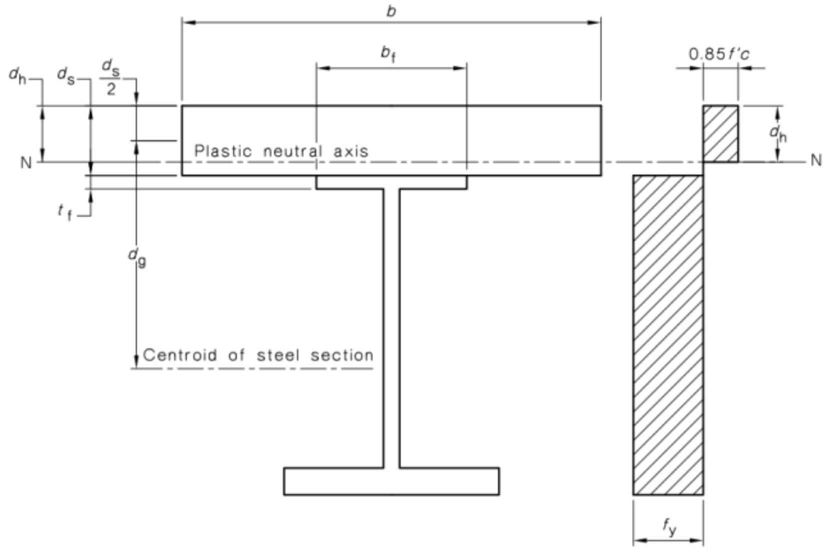
$$M_p = Af_y \left[d_g + \frac{(d_s - d_h)}{2} \right]$$

$$d_g = 1350 + \frac{250}{2} - 479 = 996\text{mm}$$

$$M_p = 280 \times 80500 \left[996 + \frac{(250 - 237)}{2} \right] \times 10^{-6} = 22{,}596\text{kN.m}$$

Refer to AS 5100.6, appendix E

Figure E.6 Main girder and slab stress blocks



Note: During the construction of a composite bridge, it is quite likely a beam will change its section class, because the addition of the deck slab both prevents local buckling of the top flange and significantly shifts the neutral axis of the section. Typically, a mid-span section could be compact after casting the slab but not prior to this. As a consequence, checks at intermediate stages of construction should be based on the relevant classification at the stage being checked.

Table E.3 Composite cross sections (long term) – sagging (n_L=19)

		Span girder	Unit
Area	A	117,250	mm^2
ENA height	ENA	791	mm
Second moment of area	I_x	4.9E+10	mm^4
Section modulus, top of slab	$Z_{x,c}$	61E+06	mm^3
Section modulus, top flange	$Z_{x,tf}$	88E+06	mm^3
Section modulus, bottom flange	$Z_{x,bf}$	62E+06	mm^3

E6.2 Effects of temperature difference and shrinkage

The primary effects of differential temperature through the depth of the cross section of a member are considered in the design. In addition, the secondary effects in continuous members, due to redistribution of the moments and support reactions caused by the primary effects are also considered.

Refer to AS 5100.6, clause 4.9.1.1

Longitudinal stresses and shear forces due to differential temperature effects are calculated by elastic theory assuming full interaction between the concrete slab and the steel beam. The stiffness is based on the transformed composite cross section using a modular ratio appropriate to short-term loading and assuming the concrete slab has an effective width calculated in accordance with AS 5100.6, clause 4.4.1.

When the effects of shrinkage modified by creep adversely affect the structure, they are calculated in the manner described for differential temperature effects, but using a modular ratio appropriate to long-term loading. The beneficial effects of the creep of concrete are taken into account.

Refer to AS 5100.6, clause 4.9.1.2

The effects of the temperature difference and shrinkage on design of the composite section are neglected as they are not adverse in the case of a simply supported bridge. However, designers may wish to check this. The longitudinal forces due to the primary effects of shrinkage and differential temperature are both considered in the design of the stud shear connector and transverse reinforcement.

E6.2.1 Temperature difference

For the calculation of primary effects, the short-term modulus of concrete is used.

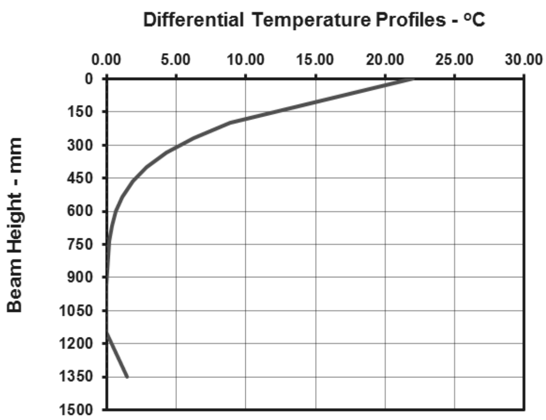
$E_C = 32\text{GPa}$ $\qquad$ $E_S = 200\text{GPa}$

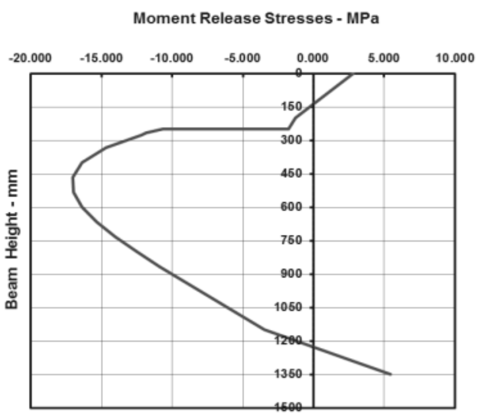
For each element of the section, calculate stress as the strain time's modulus of elasticity, and then determine force and centre of force for that area. The restraint moment in the inner beam, due to the characteristic values of temperature difference is noted in the *Bridge manual* as:

$T = (32 - 0.2d)^{O}C$ $\qquad$ $d = 50\text{mm}$ $\qquad$ $T = 22^{O}C$

The moment release stress is shown diagrammatically in figure E.7.

Figure E.7 Moment release stress over the depth of the span-girder section





E6.2.2 Shrinkage

The effects of shrinkage are calculated for the long-term situation.

The characteristic value of shrinkage is ε_{sh} = 0.0002 and the long-term modular ratio is used.

Table E.4 Temperature and shrinkage axial force

	Force	
Differential temperature axial fixity force	4053	kN
Shrinkage axial fixity force	3754	kN

E6.3 Section properties – cross girders

The gross composite section of the cross girder includes half of the width of the slab to the adjacent cross girder.

For the effective composite section, taking account of shear lag, the cross girder is effectively simply supported and thus $L_c = 11700\text{mm}$ and the effective width of slab is:

Refer to AS 5100.6, clause 6.1.7

$$b_{eff} = \min\left(\frac{L_C}{5}; S; 12t_S\right) = \min\left(\frac{11,700}{5}; 3500; 12\times 250\right) = 2340\text{mm}$$

Assume there is only a single row of connectors on the beam centreline.

Refer to AS 5100.6, clause 4.4.1

The effective section properties of the cross girder at mid-span are tabulated below.

Figure E.8 Cross girder and slab effective width

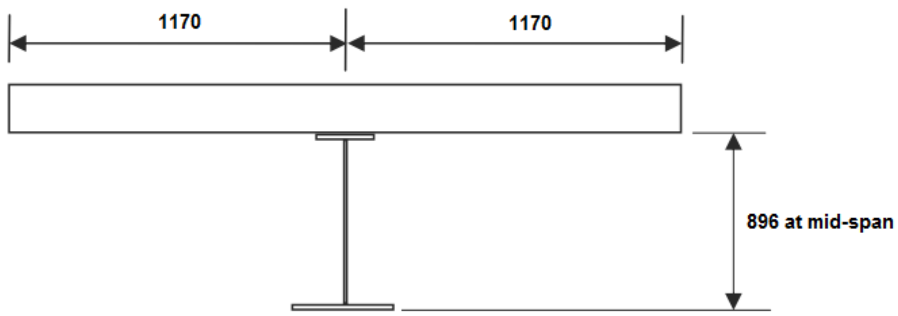


Table E.5 Cross girder section properties

		Bare steel	Short-term	Long-term	Unit
Area	A	28,536	122,136	59,249	mm²
ENA height	ENA	448	887	745	mm
Second moment of area	I_x	3.6E+09	1.1E+10	8.7E+09	mm⁴
Section modulus, top of slab	$Z_{x,c}$	-	44E+06	22E+06	mm³
Section modulus, top flange	$Z_{x,tf}$	8E+06	1256E+06	57E+06	mm³
Section modulus, bottom flange	$Z_{x,bf}$	8E+06	13E+06	12E+06	mm³
Section class	-	Compact	Compact	Compact	-
Bending moment capacity	M_{sx}	2688	5176	-	kN.m

The cross section of the span girder is compact, provided that the top flange is restrained by shear connectors within the spacing limits given in *AS 5100.6*, clause 6.1.8.

E7 Global analysis

E7.1 Model

Since the bridge is simply supported at its ends, a hand calculation effectively provides sufficient design actions at the main and cross girders.

Note, if a sophisticated computer model is used, torsional properties need to be considered.

E7.2 Construction stages

The whole deck span will be concreted in one stage. The edge beams will be concreted afterward.

Separate analytical models are therefore provided for:

Stage 1: All steelwork, wet concrete

Stage 2: Composite structure (long-term properties), the weight of the edge beams is applied

Stage 3: Composite structure (short term properties).

E7.3 Analysis results

The following results are for design values of actions, ie after application of appropriate factors on the characteristic values of actions.

For construction loading, results are given for the total effects at each construction stage. For traffic loading the results are given for the combination of traffic and pedestrian loading for worst bending effects at the central span location.

E7.3.1 Stage 1

1 Selfweight of steelwork

2 Selfweight of concrete

3 Construction load

Table E.6 Stage 1 design actions

Distance from support (m)	ULS		SLS	
	M_y (kN.m)	F_z (kN)	M_y (kN.m)	F_z (kN)
0	0	1046	0	766
12.25	6410	0	4690	0

Load combination 5B is assumed to be critical

E7.3.2 Stage 2

1 Selfweight of parapets

2 Selfweight of carriageway surfacing

3 Selfweight of footpath construction

4 Removal of construction loading

Table E.7 Stage 2 design actions

Distance from support (m)	ULS		SLS	
	M_y (kN.m)	F_z (kN)	M_y (kN.m)	F_z (kN)
0	0	521	0	386
12.25	3190	0	2362	0

Note: Load combination 5B is assumed to be critical

E7.3.3 Stage 3

Table E.8 HN72 traffic loads for worst sagging at mid-span and worst shear at supports

Distance from support (m)	ULS		SLS	
	M_y (kN.m)	F_z (kN)	M_y (kN.m)	F_z (kN)
0	0	1182	0	703
12.25	6836	0	4063	0

Note: Load combination 1A is assumed to be critical

Please note that in this case the maximum sagging moment at mid-span occurs when the HN72 axle loads are positioned at 8.5m and 13.5m respectively from the supports.

E8 Design values of the effects of combined actions

Design values are given in section E8 for certain situations in the design of the main girder and cross girder beams. In practice, the design of other parts of the structure would also need to be considered.

E8.1 Effects of construction loads (ULS)

Generally, the effects of construction loads apply to different cross-section properties. The cross sections for the main girder beams are different at stages 1 and 2. The following tabulations summarise the forces and moments at each stage and the stresses due to those effects, for selected cross sections. According to the *Bridge manual* the load combination 5B is the most critical during the stages of construction.

Table E.9 Stress at mid-span due to construction loads

			Bottom flange		Top flange		Top of slab	
	M_y	F_z	$Z_{x, bf}$	σ	$Z_{x, tf}$	σ	$Z_{x, c}$	σ
	kN.m	kN	10^6mm^3	MPa	10^6mm^3	MPa	10^6mm^3	MPa
Stage 1	6410	1046	50	128	27	-237		
Stage 2	3190	521	62	51	88	-36	61	-52
	9600	1567		179		-273		-52

Load combination 5B is assumed to be critical

E8.2 Effects of traffic load plus construction loads (ULS)

Effects due to traffic actions are determined from the short-term composite section in mid-span.

Table E.10 Stress at mid-span due to traffic load plus construction loads

			Bottom flange		Top flange		Top of slab	
	M_y	F_z	$Z_{x, bf}$	σ	$Z_{x, tf}$	σ	$Z_{x, c}$	σ
	kN.m	k	10^6mm^3	MPa	10^6mm^3	MPa	10^6mm^3	MPa
Construction	9600	1567		179		-273		-52
Traffic HN72	6836	1182	67	102	243	-28	133	-51
	16,436	2749		281		-301		-103

The resulting stresses in the tables above indicate that the steel section remains elastic under construction loading and is plastic under a full service load. These results indicate an economical section size.

E8.3 Effects in intermediate cross girders

With the traffic load positioned for the worst effects on the cross girders, the worst sagging occurs in the middle cross girder.

E8.3.1 Worst sagging on cross girder (ULS)

The following stresses are elastic stresses on the effective cross section allowing for shear lag.

Table E.11 Cross girder stress at mid-span

			Bottom flange		**Top flange**		**Top slab**	
	M_y	F_z	$Z_{x, bf}$	σ	$Z_{x, tf}$	σ	$Z_{x, c}$	σ
	kN.m	**kN**	10^6mm^3	**MPa**	10^6mm^3	**MPa**	10^6mm^3	**MPa**
Stage 1	650	222	8	79	8	-79		
Stage 2	287	98	12	24	57	-5	22	-13
Construction	937	320		103		-84		-13
Traffic HN72	1998	460	13	154	1256	-2	44	-46
	2935	780		257		-86		-59

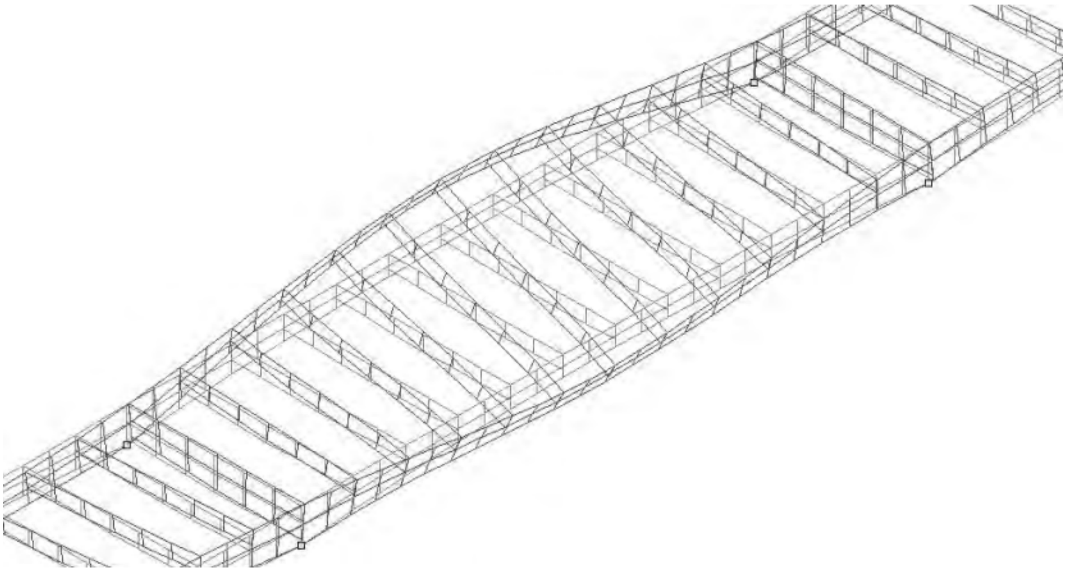
Values for SLS would be needed if the total stresses exceeded the design elastic values but, by inspection, they are not exceeded.

E9 Verification of bare steel girder during construction

The two main girders are susceptible to lateral torsional buckling under the weight of the wet concrete (ie before it hardens and provides restraint to the top flanges). The beams are partially restrained against buckling by the presence of the cross girders. The cross girders provide flexible torsional restraint to the beams.

Refer to figure 7.4 in the main part of the guide

Figure E.9 Lateral torsional buckling of the main girders



E9.1 Restraint to main girders

To provide restraint to the main girders, the cross girders should be designed for a situation where the two critical flanges try to move towards each other and put the cross girder into compression. Therefore, AS 5100.6, clause 8.4.2 states that the transverse force acting on the lateral restraint is taken as 2.5% of the most onerous ULS force in the critical flange in the main girder. However, for ladder deck bridges the transverse member loads may be more significant. Hence the following check may be undertaken

Stress at top flange under construction load = 237MPa

Maximum flange force (under construction loading) = $237 \times 600 \times 25 \times 10^{-3} = 3555\text{kN}$

2.5% flange load = $0.025 \times 3555 = 89\text{kN}$

This load is resisted by the cross girder axially. The following calculation is conservatively based on the minimum height of the cross girder section.

$N^* \leq N_{us}$

$N^* \leq N_{uc}$

$N_{us} = \phi N_s$

$N_s = k_f A_n f_y$

$k_f = A_e / A_g$

Refer to AS 5100.6, clause 10.1

Refer to AS 5100.6, clause 10.2.1

$$\lambda_{e,flange} = \frac{b_f - t_w}{2t_f}\sqrt{\frac{f_{y,flange}}{250}}$$

$$\lambda_{e,flange} = \frac{300-16}{2 \times 25}\sqrt{\frac{280}{250}} = 6.0$$

$$\lambda_{e,web} = \frac{d_1}{t_w}\sqrt{\frac{f_{y,web}}{250}}$$

$$\lambda_{e,web} = \frac{700}{16}\sqrt{\frac{300}{250}} = 48.0$$

From *AS 5100.6*, table 10.2.4

$$\lambda_{ey,flange} = 14$$

$$\lambda_{ey,web} = 115$$

Therefore,

$$b_{e,flange} = \frac{b_f - t_w}{2\lambda}\left(\frac{\lambda_{ey,flange}}{e,flange}\right) \leq \frac{b_f - t_w}{2}$$

$$b_{e,flange} = \frac{300-16}{2}\left(\frac{14}{6}\right) \leq \frac{300-16}{2}$$

$$b_{e,flange} = \frac{300-16}{2} = 142\text{mm}$$

$$b_{e,web} = d_1\left(\frac{\lambda_{ey,web}}{\lambda_{e,web}}\right) \leq d_1$$

$$b_{e,web} = 700\left(\frac{115}{48}\right) \leq 700$$

$$b_{e,web} = 700\text{mm}$$

$$A_e = \left[(2\times300\times25)+(700\times16)\right] = 26{,}200\text{mm}^2$$

$$A_g = \left[(2\times300\times25)+(700\times16)\right] = 26{,}200\text{mm}^2$$

$$k_f = A_e / A_g = 26{,}200/26{,}200 = 1.0$$

$$N_s = k_f A_n f_y = 26{,}200\times280\times10^{-3} = 7336\text{kN}$$

$$N_{us} = \phi N_s = 0.9 \times 7336 = 6600\text{kN}$$

Also,

$$N_{uc} = \phi N_c$$

$$N_c = \alpha_c N_s \leq N_s$$

$$I_y = 113\times10^6\text{mm}^4$$

$$r_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{113\times10^6}{26200}} = 66\text{mm}$$

$$\lambda_\eta = \left(\frac{L_e}{r_y}\right)\sqrt{k_f\left(\frac{f_y}{250}\right)} = \left(\frac{11700}{66}\right)\sqrt{1.0\times\left(\frac{300}{250}\right)} = 194$$

Refer to AS 5100.6, clause 10.3.3

$$\alpha_a = \frac{2100(\lambda_\eta - 13.5)}{\lambda_\eta^2 - 15.3\lambda_\eta + 2050} = \frac{2100(194-13.5)}{194^2 - 15.3\times194 + 2050} = 10.32$$

$\alpha_b = 0.5$

$\lambda = \lambda_\eta + \alpha_a \alpha_b = 194 + 10.32 \times 0.5 = 199$

$\eta = 0.00326\left(199 - 13.5\right) \geq 0$

$\eta = 0.6$

$$\xi = \frac{\left(\frac{\lambda}{90}\right)^2 + 1 + \eta}{2\left(\frac{\lambda}{90}\right)^2} = \frac{\left(\frac{189}{90}\right)^2 + 1 + 0.6}{2\left(\frac{189}{90}\right)^2} = 0.68$$

$$\alpha_c = \xi\left[1 - \sqrt{1 - \left(\frac{90}{\xi\lambda}\right)^2}\right] = 0.68\left[1 - \sqrt{1 - \left(\frac{90}{0.68 \times 189}\right)^2}\right] = 0.195$$

α_c can also be directly read off table 10.3.3 in AS 5100.6

$N_s = k_f A_n f_y = 1.0 \times 26,200 \times 280 \times 10^{-3} = 7336\text{kN}$

$N_c = \alpha_c N_s = 0.195 \times 7336 = 1431\text{kN}$

$N_{uc} = \phi N_c = 0.9 \times 1431 = 1288\text{kN}$

$N^* \leq N_{uc} - \text{OK}$

$N^* \leq N_{us} - \text{OK}$

Also,

Refer to AS 5100.6, clause 11.3.2

$M_x^* = 6410\text{kN.m}$

$M_x^* \leq \phi M_{rx}$

$$\phi M_{rx} = \phi M_{sx}\left(1 - \frac{N^*}{\phi N_s}\right)$$

$$\phi M_{rx} = 0.9 \times 7560\left(1 - \frac{89}{0.9 \times 7336}\right) = 6712\text{kN.m}$$

$M_x^* = 6410\text{kN.m} \leq \phi M_{rx} = 6712\text{kN.m - OK}$

Refer to AS 5100.6, clause 5.6.1

E9.2 Verification

Maximum sagging bending moment at the mid-span under construction load is 6410kN.m.

The flexural capacity of the main girder is given in AS 5100.6, clause 5.6 as follows:

$$M_b = \alpha_m \alpha_s M_s \leq M_s$$

At the mid-span the bending moment assumed to be constant, therefore, $\alpha_m = 1.13$

$L_e = k_t k_L k_r L$

$k_t = 1.0 \quad \text{(LL)}$

$k_L = 1.4$

$k_r = 1.0$

$L_e = 3.5 \times 1.4 = 4.9\text{m}$

$E = 200{,}000\text{MPa}$

$G = 80{,}000\text{MPa}$

$I_y = 2.58 \times 10^9 \text{mm}^4$

$I_w = 0$

$J = 3.92 \times 10^7 \text{mm}^4$

$M_s = 7560\text{kN.m}$

$$M_o = \sqrt{\left(\frac{\pi^2 EI_y}{L_e^2}\right)\left(GJ + \frac{\pi^2 EI_w}{L_e^2}\right)}$$

$$M_o = \sqrt{\left(\frac{\pi^2 \times 200{,}000 \times 2.58 \times 10^9}{4900^2}\right)\left(80{,}000 \times 3.92 \times 10^7\right)} \times 10^{-6}$$

$M_o = 25{,}791\text{kN.m}$

$$\alpha_s = 0.6\left[\sqrt{\left[\left(\frac{M_s}{M_o}\right)^2 + 3\right]} - \left(\frac{M_s}{M_o}\right)\right]$$

$$\alpha_s = 0.6\left[\sqrt{\left[\left(\frac{7560}{25{,}791}\right)^2 + 3\right]} - \left(\frac{7560}{25{,}791}\right)\right] = 0.88$$

$M_b = 1.13 \times 0.88 \times 7560 = 7518\text{kN.m} \le M_s = 7560\text{kN.m}$

$\phi M_b = 0.9 \times 7518 = 6766\text{kN.m}$

$M^* = 6410\text{kN.m} < \phi M_b = 6766\text{kN.m} - \text{OK}$

Refer to AS 5100.6, clause 5.10.2

Also,

$V^* \le \phi V_v$

$$\frac{d_p}{t_w} = \frac{1275}{20} = 63.75 \le \frac{82}{\sqrt{\frac{f_y}{250}}} = \frac{82}{\sqrt{\frac{300}{250}}} = 74.8$$

$V_v = V_w$

$A_w = 1275 \times 20 = 25{,}500\text{mm}^2$

$V_w = 0.6 f_y A_w = 0.6 \times 300 \times 25{,}500 \times 10^{-3} = 4590\text{kN}$

$\phi V_v = 0.9 \times 4590 = 4131\text{kN}$

$V^* = 1046\text{kN} \le \phi V_v = 4131\text{kN} - \text{OK}$

E10 Verification of composite girder

E10.1 Sagging bending in main girder

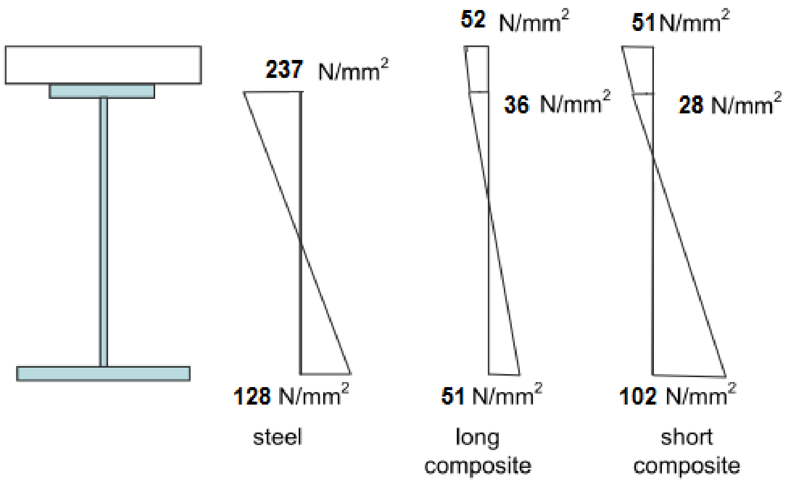
The elastic design bending resistance for a beam constructed in stages depends on the design effects at the stages.

The bare steel section is not compact in bending and the composite cross section is compact. The effects (stresses) in the cross section have been calculated on the basis of gross section properties for effects on the steel beam plus effective section properties on the composite section.

The composite cross section is compact (PNA in the top flange) so the plastic resistance can be utilised.

The plastic bending resistance of the short-term composite section is 0.9 x 22,596=20,336kNm and the total design value of bending effects is 16,436kNm. The cross section is satisfactory by inspection. It can also be seen that the stresses calculated elastically, taking account of construction in stages, are satisfactory, as follows:

Figure E.10 Elastic stresses in the girder



E10.2 Verification of cross girders

As previously noted, the stresses in the cross girders are within elastic limits for the loads considered in the analysis. However, these transverse girders are required to prevent buckling of the slab where it is in compression. The cross girders need to be both stiff enough and strong enough to perform this function in addition to the resistance to the effects already calculated.

Note that the Intermediate cross girders effectively act as simply supported beams in carrying the loading from the slab

The plastic bending resistance of the short term composite section is 0.9 x 5515=4964kNm and the total design value of bending effects is 2935kNm. The cross section is satisfactory by inspection. In addition, the combined axial and bending actions should be checked. This would be derived from a detailed deck analysis.

Note: This design is very conservative, therefore, designers may wish to refine it for economy.

E11 Longitudinal shear

Refer to section 6.1.7 in the main part of the guide

The resistance to longitudinal shear is verified for the web/flange weld, the shear connectors and the transverse reinforcement at the supports and at mid-span. (In practice, intermediate values would also be verified, to optimise the provision of shear connectors.)

E11.1 Effects for maximum shear

Table E.12 ULS values at supports

	Supports	
Shear on steel section (stage 1)	1046	kN
Shear on long term composite section	521	kN
Shear on short term composite section (worst effects)	1182	kN

Table E.13 SLS values

	Supports	
Shear on steel section (stage 1)	766	kN
Shear on long term composite section	386	kN
Shear on short term composite section (worst effects)	703	kN

E11.2 Section properties

To determine shear flows the parameter $A_t y_c / I_t$ is needed for each section and stage.

Table E.14 Section properties

	Web/top fl	Top fl/slab
	$A_t y_c / I_t$	$A_t y_c / I_t$
	m^{-1}	m^{-1}
Steel section	0.537	-
Long term section	0.682	0.514
Short term section	0.715	0.656

E11.3 Shear flow at ULS

Force at the web/top flange junction

At abutment $1046 \times 0.537 + 521 \times 0.682 + 1182 \times 0.715 =$ 1762 kN/m

Force at the flange/slab junction

At abutment $521 \times 0.514 + 1182 \times 0.656 =$ 1043 kN/m

E11.4 Shear flow at SLS

Force at flange/slab junction

At abutment $386 \times 0.514 + 703 \times 0.656 =$ 660 kN/m

The shear flow at SLS is required for verification of the shear connectors.

E11.5 Web/flange weld

Design weld resistance is given in tables 9.8 and 9.9 of ASI (1999) design capacity tables.

For 6mm throat SP fillet weld t_w=8mm.

$n_{w_{6mm}} = 1.11kN/mm$

Resistance of two welds = 2220kN/m > 1762kN/m, shear flow at top flange – OK

Shear flows at bottom flange are slightly less and are OK by inspection.

Refer to section 7.7.3 in the main part of the guide

E11.6 Shear connectors

Shear stud connectors 19mm diameter, 150mm long are assumed, with f_u=410MPa.

The resistance of a single stud is given by *AS 5100.6*, clause 6.6.4.4 as the lesser of:

$$f_{ks} = 0.63 d_{bs}^2 f_{uc}$$

or

$$f_{ks} = 0.63 d_{bs}^2 \sqrt{f'_{cy} E_c}$$

$f'_{cy} = 40MPa$ $\qquad f_{cu} = 410MPa$ $\qquad d_{bs} = 16mm$

$$E_c = 5050\sqrt{f'_{cy}} = 5050\sqrt{40} = 32{,}000MPa$$

Note: For the purpose of evaluating the resistance of headed stud connectors embedded in solid slabs, the modulus of elasticity for concrete may be taken to be $E_c = 5050\sqrt{f'_{cy}}$.

From structural reliability analyses conducted by HERA, it has been shown that the use of this value produces predictions that better reflect the resistance of studs in physical tests.

Refer to AS 5100.6. clause 6.6.3.2

Therefore the design resistance of a single-headed shear connector is:

$$f_{ks} = \min\left[0.63 \times 19^2 \times 410; 0.63 \times 19^2 \sqrt{40 \times 32{,}000}\right] = 93.25kN$$

$u_L^* \le \phi u_{Ls}$ $\qquad u_{Ls} = 0.55 n f_{ks}$ $\qquad \phi = 1.0$

$$u_L^* = 660kN/m + \left(\frac{3754 \times 5}{24.5 \times 2} kN/m\right)_{\text{shrinkage PE}} = 1043kN/m$$

If studs are grouped and spaced at 150mm spacing along the beam (to suit transverse reinforcement), then a row of three studs has a design resistance of:

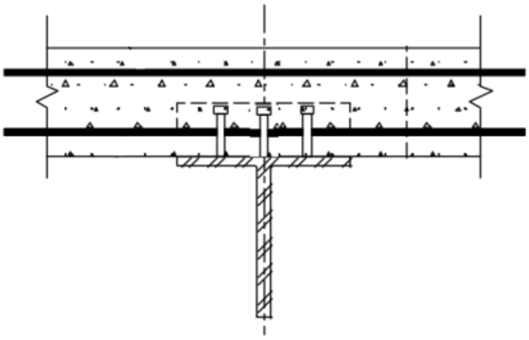
$$\phi u_{Ls} = \frac{0.55 \times 93.25 \times 3}{0.15} = 1025.75kN/m$$

Refer to section 7.7.7 in the main part of the guide

E11.7 Transverse reinforcement

Consider the transverse reinforcement required to transfer the full shear resistance of the studs, ie 1025.75kN/m as well as the maximum ULS shear flow, ie 1046kN/m.

Figure E.11 Transverse reinforcement



For a critical shear plane around the studs, shown dotted above, the shear resistance is provided by twice the area of the bottom bars. The design shear resistance of the transverse reinforcement is given by *AS 5100.6*, clause 6.6.5.2 as follows:

$$\upsilon^*_{Lp} \leq \phi\left(0.9us + 0.7A_{ts}f_{ry}\right)$$

and

$$\upsilon^*_{Lp} \leq \phi\left(0.15uf'_c\right)$$

Assumed HD20-150mm:

$$s = 1\text{MPa}$$

$$u = 2\times150+600 = 900\text{mm}$$

$$A_{st} = \left(2\times314\times1000\right)/150 = 4188\text{mm}^2/\text{m} \qquad f_{ry} \leq 450\text{MPa}$$

$$\upsilon_L = \min\left[1.0\left(0.9\times900\times1.0+\frac{0.7\times4188\times450}{10^3}\right);1.0\left(0.15\times900\times40\right)\right] = 2130\text{kN/m}$$

$$u^*_{Lp} = 1046\text{kN/m}+\left(\frac{1.35\times3754\times5}{24.5\times2}\right)_{\text{shrinkage PE}} = 1563\text{kN/m}$$

$$u^*_{Lp} = 1563\text{kN/m} \leq u_L = 2130\text{kN/m - OK}$$

The transverse bars are adequate.

E12 Cross girder to main girder connection

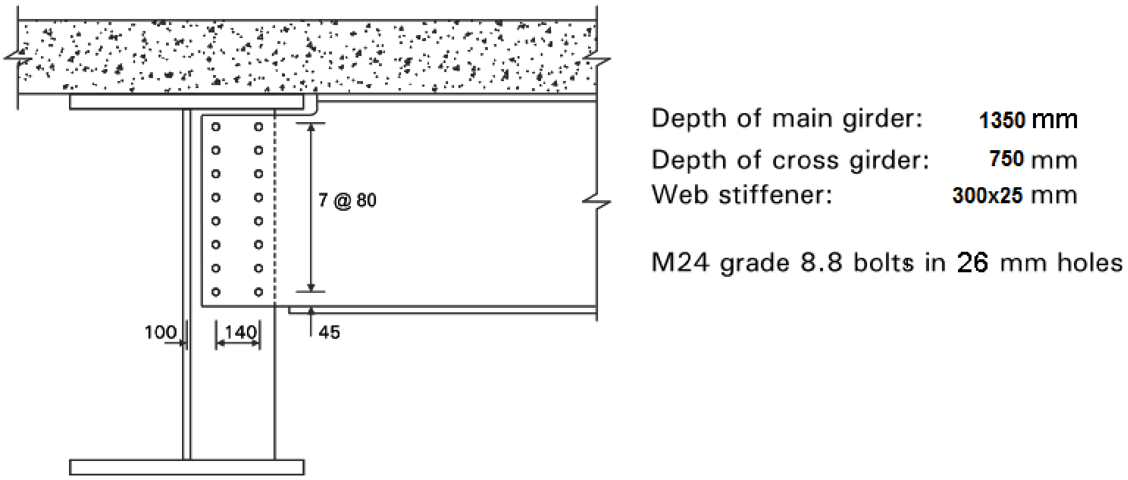
Consider the cross girders at mid-span and adjacent to the support.

E12.1 Structural arrangement of connection

The connection between main girders and cross girder is designed for the bending moment determined from 2.5% of the moment-induced compression force in the top (critical) flange of the main girder. This acts at a lever arm corresponding to the vertical distance between the centroid of the bottom flange (ie the tension flange) and the centroid of the bottom flange. This moment has to be induced in the connection design in conjunction with the critical shear on the cross girder from applied loading.

The structural arrangement of the connection between an intermediate cross girder and a main girder is shown in figure E.12. Only the web of the cross girder is connected to the stiffener on the main girder; the flanges are not connected.

Figure E.12 Main girder and cross girder connection arrangement



$d = 24mm$ $\qquad$ $d_h = 26mm$

$f_{uf} = 830MPa$

$A_s = 353mm^2$ $\qquad$ $A_c = 324mm^2$

$\mu = 0.35$ $\qquad$ $k_h = 1.0$

$N_{ti} = 210kN$

Refer to AS 5100.6, clause 12.5.4.1

SLS slip resistance in double shear:

$V^*_{sf} \le \phi V_{sf}$ $\qquad$ $\phi = 0.7$

$V_{sf} = \mu n_{ei} N_{ti} K_h$ $\qquad$ $V_{sf} = 0.35 \times 210 \times 1.0 = 73.5kN$

$\phi V_{sf} = 51.50kN$

Refer to AS 5100.6, clause 12.5.3.1

E12.2 Shear resistance of bolts

ULS shear resistance of bolts (assuming shear through threads):

$V^*_f \le \phi V_f$ $\qquad$ $\phi = 0.8$

$V_f = 0.62 f_{uf} k_r \left(n_n A_c + n_x A_o\right)$

$k_r = 1.0$ $\qquad$ $n_n = 1.0$

$V_f = 0.62 \times 830 \times 1.0 \left(1 \times 324 + 0\right) = 166kN$

$\phi V_f = 133kN$

$\Sigma r_n^2 = \Sigma\left(x_n^2 + y_n^2\right)$

$\Sigma r_n^2 = 616{,}000mm^2$

$r_{max} = \sqrt{70^2 + 280^2} = 288.62mm$

$M^*_{buckling} = 2.5\%\ \text{flange compression force} \times \text{distance between the centroid of the top and bottom flanges}$

$$M^*_{buckling} = 89 \times \left(1350 - \frac{50}{2} - \frac{25}{2}\right) \times 10^{-3} = 117kN.m$$

$$M^*_{shear} = 222 \times \left(100 + \frac{140}{2}\right) \times 10^{-3} = 38kN.m$$

$$M^* = M^*_{buckling} + M^*_{shear} = 117 + 38 = 155kN.m$$

$$V_{b,m} = \frac{155 \times 288.62}{616,000} \times 10^3 = 73kN$$

$$V_{m_x} = 73 \frac{280}{288.62} = 71kN$$

$$V_{m_y} = 73 \frac{70}{288.62} = 18kN$$

$$V_s = \frac{222}{16} = 13.87kN$$

$$V_{b,max} = \sqrt{\left(V_{m_x}\right)^2 + \left(V_{m_y} + V_s\right)^2} = \sqrt{71^2 + (18 + 13.87)^2} = 78kN$$

$$V_{b,max} \leq \phi V_f$$

E13 Fatigue assessment

E13.1 Basic loadings for fatigue design

Clause 6.9 of AS 5100.2 gives two basic loads as shown in figures E.13 and E.14

Figure E.13 Modified individual A160 heavy axle load

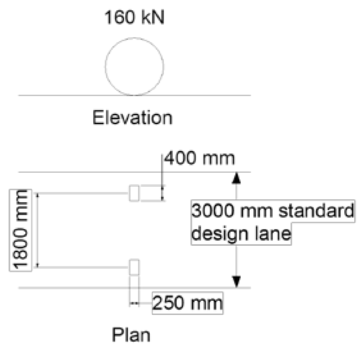
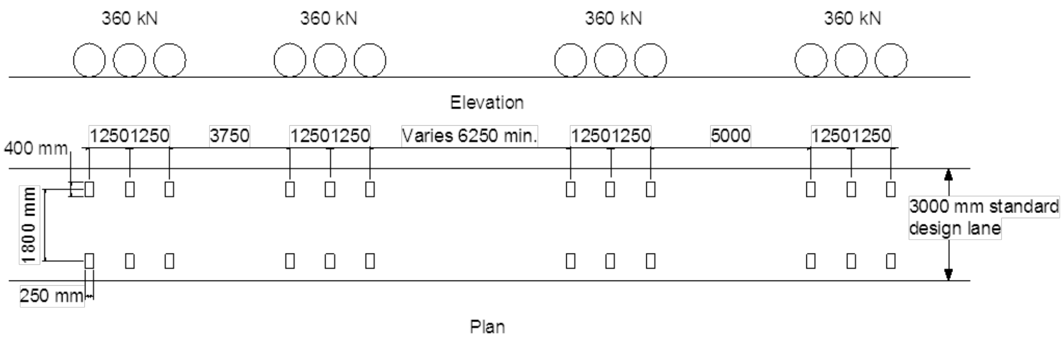


Figure E.14 Modified individual M1600 moving traffic load without the UDL component



The number of fatigue stress cycles to be used for the calculation of the fatigue capacity of the structural element under consideration should be as follows:

- For the fatigue design load of 0.70 x (A160 axle load) x (1 + α):
  - (current number of heavy vehicles per lane per day) x 4 x 10^4 x (route factor)
- For the fatigue design load of 0.70 x (M1600 moving traffic load without UDL) x (1 + α):
  - (current number of heavy vehicles per lane per day) x 2 x 10^4 ($L^{-0.5}$) x (route factor).

Dynamic load allowance α is given as follows:

- $\alpha = 0.4$ for the A160 axle load
- $\alpha = 0.3$ for the M1600 axle load.

The route factor for the urban road specified as 0.3.

L is the effective span in metres and is defined as:

- for positive bending moments, L is the actual span in which the bending moment is being considered
- for reactions, L is the sum of the adjacent spans
- for cross-girders, L is twice the longitudinal spacing of the cross-girders.

E13.2 Range of effects due to passage of fatigue vehicle

Through inspection it is clear that the fatigue loading of M1600 is more critical, therefore the fatigue loading is:

$$0.70 \times (\text{M1600 without UDL}) \times (1+\alpha)$$

The corresponding total number of cycles is:

$$n = NHVD_{\text{per lane}} \times 2 \times 10^4 \left(L^{-0.5}\right)(\text{route factor})$$

$NHVD = 1500$ route factor $= 0.3$

Table E.14 Worst bending effects

	Span
	M_y (kN.m)
Range	2028

Table E.15 Worst shear effects

	Span
	F_z (kN)
Range	390

E13.3 Assessment of structural steel details

E13.3.1 Design stress range mid-span

For the mid-span the length of the effective span is $L = 24.50\text{m}$.

Therefore, the total number of cycles is:

$n = 1500 \times 2 \times 10^{4} \times 24.5^{-0.5} \times 0.3 = 1.82 \times 10^{6}$

At mid-span there is negligible stress range in the top flange. The stress range in the bottom flange is:

$$f^* = \frac{2028}{67} = 30\text{MPa}\ .$$

As $f^* = 30\text{MPa} \geq 27\text{MPa}$ fatigue assessment is required.

The worst detail category that might apply is for the weld between the bottom flange and the web, which, for a manual continuous weld, is category 100 (*AS 5100.6*, table 13.5.1-B).

Design value of fatigue strength ϕf_{fc} is:

$n = 1.82 \times 10^{6} \leq 5 \times 10^{6}$

$\phi = 1.0$

$\phi f_{rnc} = 100$

$$\phi f_{fc} = \phi f_{rnc} \left(\frac{2 \times 10^{6}}{n} \right)^{1/3} = 100 \left(\frac{2}{1.82} \right)^{1/3} = 103\text{MPa}$$

$f^* \leq \phi f_{fc}$ - OK

E13.4 Assessment of shear connection

The design value of the stress range in shear studs is given 425MPa times the ratio of the longitudinal shear load on the stud to the nominal static strength specified in *AS 5100.6*, clause 6.6.3.

E13.4.1 Shear at supports

The range of vertical shear force at the pier is 390kN.

At the pier, the studs are 19mm diameter, in rows of two at 150mm spacing.

$u_L^* = 390 \times 0.656 = 256\text{kN/m}$

$$\frac{A_t y_c}{I_t} = 0.656\text{m}^{-1}$$

$\phi u_{Ls} = 684\text{kN/m}$

$\phi = 1.0$

Therefore,

$$f_s^* = \frac{u_L^*}{u_{Ls}} \times 425 = \frac{\frac{256 \times 0.15}{2}}{684} \times 425 = 12\text{MPa}$$

The fatigue strength of the shear stud is given in *AS 5100.6*, clause 13.6.3 as follows:

$$\phi f_f = \phi \left(\frac{2.08 \times 10^{22}}{n} \right)^{1/8} = \left(\frac{2.08 \times 10^{22}}{1.82 \times 10^{6}} \right)^{1/8} = 102\text{MPa}$$

$f_s^* \leq \phi f_f$ - OK

Therefore, the number of the shear stud needs to be increased to three.

Appendix F: Two span multi-girder bridge

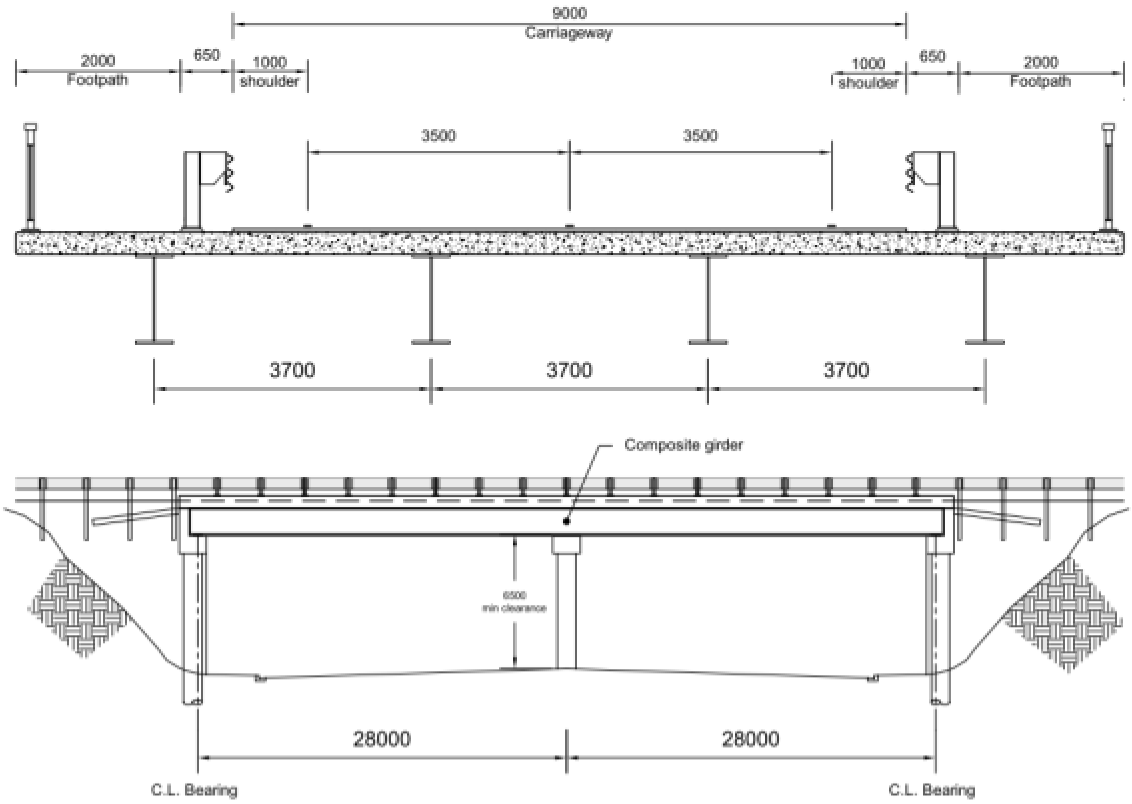
F1 Design statement

- The bridge carries a two-lane (each 3.5m wide) single carriageway road over a flood plain.
- The carriageway has two 1m wide shoulders and has 2m wide footways on either side.
- Speed environment is signed as a maximum of 50km/h but a design threshold of 60km/h is used.
- For barrier design the annual average daily traffic (AADT) is 15,000 vehicles per day with 20% commercial vehicles. An offset of 1.2m is assumed. Thus, a G9 modified TL4 Thrie beam is considered.
- The bridge clearance is 6.5m from the soffit to the road surface, therefore the impact load does not need to be considered.
- The bridge is being designed in accordance with following standards:
  - NZTA (2012) *Bridge manual*
  - *AS 5100.6* (SA 2004)
  - *NZS 3101* (SNZ 2006).
- Structural material properties:
  - structural steel: grade 300
  - concrete: C40
  - steel reinforcement: grade 500.
- Environmental exposure classification for superstructure = A2 – inland, exterior.
- Density of steel = 78.5kN/m^3.
- Density of reinforced concrete = 25kN/m^3.
- Nominal thickness of surfacing = 130mm.
- Density of surfacing = 23kN/m^3.
- Steel parapet selfweight = 2kN/m for both TL4 and pedestrian barriers.
- The construction load is 0.75kN/m^2 and the weight of the temporary formwork is assumed to be 0.5kN/m^2. Additionally, wet concrete is assumed to have a density of 1kN/m^3 greater than that of hardened concrete.
- Pedestrian live load = 5kPa. This value can be reduced when considered in conjunction with road traffic loading.
- *Draft fatigue design criterion for bridges* (Clifton 2007a) is recommended for fatigue assessment.
- Coefficient of linear thermal expansion = 11.7x10-6 per ºC.
- Characteristic value of shrinkage is $\varepsilon_{sh} = 0.0002$.

F2 Structural arrangement

The bridge carries a two-lane single carriageway road over another road. The carriageway has 1m wide shoulders and has a 2m wide footway on either side. A four-girder arrangement has been chosen and a deck slab of 250mm has been assumed. The deck overhangs 1.6m outside the centrelines of the outer girders.

Figure F.1 Two lane bridge structural arrangement



F3 Design basis

The bridge is designed in accordance with the *Bridge manual*, *AS 5100.6* and *NZS 3101*.

The fatigue limit state is verified for the reference stress range due to the load application based on Clifton (2007a).

Crack widths in the deck slab are verified at the serviceability limit state (SLS) based on *NZS 3101*.

F3.1 Load combinations

Factors and load combinations of actions are given by the *Bridge manual*, tables 3.1 and 3.2 for the SLS and ultimate limit state (ULS) respectively.

F3.2 Factors on strength

The values of various capacity reduction factors (ϕ) for strength limit states are given by *AS 5100.6*, table 3.2 for steel, and *NZS 3101*, clause 2.3.2.2 for concrete.

F3.3 Structural material properties

It is assumed that the following structural material grades will be used:

- structural steel: grade 300
- concrete: C40
- steel reinforcement: grade 500

For structural steel, the value of f_y depends on the product material.

ASI design capacity tables

For rolled sections use:
320MPa for t ≤11mm; 300MPa for 11mm <t ≤17mm; and 280MPa for t >17mm.

For plates use:
320MPa for t ≤8mm; 310MPa for 8mm <t ≤12mm; 300MPa for 12mm <t ≤20mm; and 280MPa for t>20mm.

Note that designers may wish to use grade 350 steel for better economy.

For concrete, $f_c^{'} = 40\text{MPa}$

For steel reinforcement, $f_y = 500\text{MPa}$

The modulus of elasticity for both structural steel and steel reinforcement is taken as $E_s = 200\text{GPa}$

The modulus of elasticity of the concrete is given by *NZS 3101*, section 5.2.3 as:

$$E_c = \left(3320\sqrt{f_c^{'}} + 6900\right)\left(\frac{\rho}{2300}\right)^{1.5}$$

$$E_c = 32\text{GPa}$$

This 28-day value is used to determine all short-term effects and resistance, and the modular ratio is thus:

$$n_s = 200/32 = 6.25$$

For long-term effects, the modular ratio is:

$$E_{c,L} = \frac{E_c}{1+\phi_{cc}} = \frac{32}{1+2} = 10.50$$

$$n_L = 200/10.50 = 19$$

The conservative value of ϕ_{cc} is given in RRU 70 as 2.00.

F3.4 Durability requirements

F3.4.1 Concrete

Environmental exposure classification

Superstructure: A2 – inland, exterior

Minimum cover to reinforcement

Surfaces in contact with the ground = 75mm

Surfaces with a damp proof membrane between the ground and the concrete = 50mm

Elsewhere = 40mm

F3.4.2 Steel

For appropriate and cost-effective coating system for structural steelwork 'New Zealand steelwork corrosion and coatings guide' (El Sarraf and Clifton 2011), should be used in conjunction with *AS/NZS 2312* (SNZ 2002) and *NZS 3404.1* (SNZ 2009).

Table F.1 Load combinations for the serviceability limit state

Group	Loads
1A	DL + EL + GW + EP + OW + SG + ST + CF + 1.35LLxI + FP
1B	DL + EL + GW + EP + OW + SG + ST + TP
2A	DL + EL + GW + EP + OW + SG + ST + CF + 1.35LLxI + FP + HE + TP
2B	DL + EL + GW + EP + OW + SG + ST + CF + 1.35LLxI + FP + HE + WD
2C	DL + EL + GW + EP + FW + PW + SG + ST + CF + 1.35LLxI + FP + HE
3A	DL + EL + GW + EP + OW + SG + ST + EQ + 0.33TP
3B	DL + EL + GW + EP + FW + PW + SG + ST + WD
3C	DL + EL + GW + EP + OW + SG + ST + CO + 0.33 TP
4	DL + EL + GW + EP + OW + SG + ST + OLxI + 0.5FP + 0.33TP
5A	DL + EL + GW + EP + OW + SG + 0.33WD + CN
5B	DL + EL + GW + EP + OW + SG + 0.33TP + CN
5C	DL + EL + GW + EP + OW + SG + 0.33EQ + CN

Table F.2 Load combinations for the ultimate limit state

Group	Loads and Load Factors
1A	U = 1.35 (DL + EL + 1.35EP + OW + SG + ST + 1.67(CF + LLxI) + 1.30FP) + GW
1B	U = 1.35 (DL + EL + 1.35EP + OW + SG + ST + 1.25TP) + GW
2A	U = 1.20 (DL + EL + EP + OW + SG + ST + CF + LLxI + FP + HE + TP) + GW
2B	U = 1.35 (DL + EL + EP + OW + SG + ST + CF + LLxI + FP + HE) + GW + WD
2C	U = 1.35 (DL + EL + EP + SG + ST + CF + LLxI + FP + HE) + GW + FW + PW
3A*	U = 1.00 (kDL + EL + 1.35 (EP + OW) + SG + ST + EQ + 0.33TP) + GW
3B	U = 1.10 (DL + EL + 1.25 EP + SG + ST) + GW + FW + PW + WD
3C	U = 1.00 (DL + EL + 1.35 (EP + OW) + SG + ST + 2.00CO + 0.33TP) + GW
3D	U = 1.20 (DL + EL + EP + OW + SG + ST + TP) + GW + PW + SN + 0.33WD
4	U = 1.35 (DL + EL + EP + OW + SG + ST + 1.10(CF + OLxI) + 0.70FP + 0.33TP) + GW
5A	U = 1.35 (DL + EL + EP + OW + SG + 1.10CN) + GW + 0.33WD
5B	U = 1.35 (DL + EL + EP + OW + SG + 0.33TP + 1.10CN) + GW
5C	U = 1.35 (DL + EL + EP + OW + SG + 0.33EQ + 1.10CN) + GW

* k = 1.3 or 0.8 whichever is more severe, to allow for vertical acceleration. k=1.0 when considering the vertical earthquake response specified by clause 5.2.6

F4 Action on the bridge

F4.1 Permanent actions

F4.1.1 Self-weight of structural elements

The density of steel is taken as $78.5kN/m^3$ and the density of reinforced concrete as $25kN/m^3$. Self-weights are based on nominal dimensions.

Self-weight of surfacing

The nominal thickness of the surfacing is 130mm. Assume that the density is $23kN/m^3$. The self-weight generally produces adverse effects and international practice is to base it on nominal thickness + 55%. However, New Zealand practice differs and reference should be made to the *Bridge manual*, section 3.4.2.

$$DL = 1.55 \times 0.13 \times 23 = 4.63kPa$$

F4.1.2 Self-weight of footway construction

The nominal thickness of the footway comprising a concrete slab is 250mm and a uniform density of $25kN/m^3$ is assumed. The self-weight is based on the nominal dimensions.

F4.1.3 Self-weight of parapets

A nominal value of 2kN/m is assumed for each steel parapet.

Note that this value will increase if a solid concrete crash barrier is used.

F4.2 Construction loads

Refer to section 6.1.5 in the main part of the guide

For global analysis, a uniform construction load of $0.75kN/m^2$ is assumed during casting and the weight of the temporary formwork is assumed to be $0.5kN/m^2$. Additionally, wet concrete is assumed to have a density of $1kN/m^3$ greater than that of hardened concrete; for a slab thickness of 250mm this adds $0.25kN/m^2$.

The total construction load is thus: $CN = 0.75 + 0.50 + 0.25 = 1.50kPa$

F4.3 Traffic loads

F4.3.1 Road traffic

Refer to section 6.4.1 in the main part of the guide

For the road carried by this bridge, the NZTA specifies the HN-HO-72 traffic loading but only HN72 has been considered here for simplicity.

F4.3.2 Pedestrian traffic

Pedestrian traffic is represented by the value in the *Bridge manual*.

$$FP = 5kPa$$

This can be reduced to 4.0kPa when applied with traffic loading

F4.3.3 Fatigue load

Refer to section 6.4.2 in the main part of the guide

Clifton (2007a) is recommended for fatigue assessment

F4.4 Thermal actions

F4.4.1 Overall temperature change

For change of length in composite sections, the coefficient of linear thermal expansion is 11.7×10^{-6} per °C. This is then used for determining soil pressure on integral bridges.

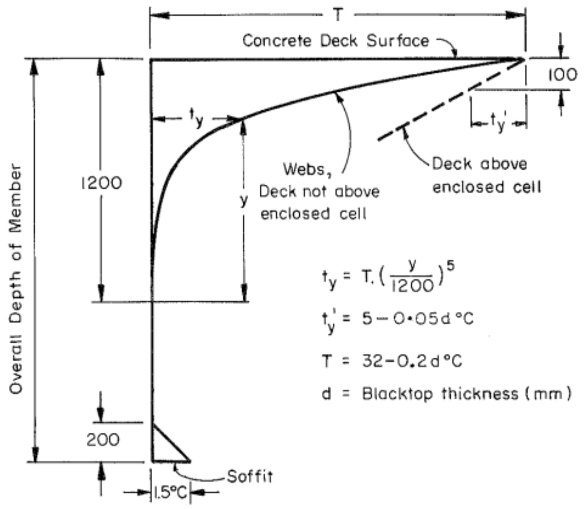
F4.4.2 Differential temperature change

The vertical temperature difference given in chapter 3 of the *Bridge manual* is used and the temperature difference will be considered to act simultaneously with the overall temperature change.

The effects of vertical temperature gradients are derived for the positive differential temperature conditions (where the top surface is hotter than the average temperature of the superstructure) and for the negative temperature differential conditions (where the top surface is colder than the average temperature of the superstructure).

Note: The negative temperature variation is the same as for bridge type 1 in *AS 5100.2*, figure 17.3.

Figure F.2 Temperature variation with depth



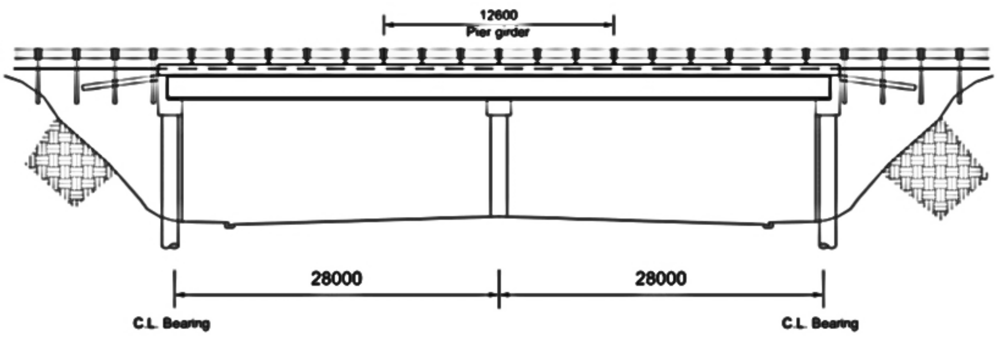
F4.5 Seismic actions

> Refer to section 6.5 in the main part of the guide.

Section 5 of the *Bridge manual* is used to determine the horizontal force generated by the seismic action on the bridge superstructure. These forces are transferred into the supports (piers and abutments), which are designed to resist this horizontal force (as shown in figure 5.3 of the *Bridge manual*). Therefore, seismic actions acting on the superstructure are ignored as they are minimal as long as the load path is available to transfer these actions into the substructure.

F5 Girder makeup and slab reinforcement

Figure F.3 Girder makeup and slab reinforcement



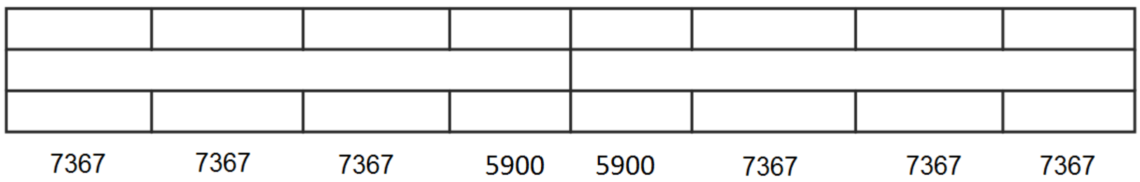
	21.7m span girder	12.6m pier girder	21.7m span girder
Top flange	500 x 25	600 x 50	500 x 25
Web	20	20	20
Bottom flange	600x 40	700x 60	600x 40
Top rebar	HD16-150	HD25-150	HD16-150
Bottom rebar	HD16-150	HD25-150	HD16-150

The overall girder depth is 1100mm. Cover to the centroid of the top and bottom reinforcement is 55mm. This complies with *NZS 3101*.

Note: The above section sizes are based on an initial few iterations to determine the section properties for this worked example. For an actual bridge, the designer may wish to undertake further iterations in order to optimise the cross sections used and gain economy.

F5.1 Bracing arrangements

Figure F.4 Bracing arrangements



The above bracing arrangements are assumed for this example.

F6 Beam cross sections

Refer to section 7.1.1 in the main part of the guide.

F6.1 Section properties – internal main girders

Section properties are required for global analysis. For section analysis, consider the effective section, allowing for shear lag.

The equivalent spans for effective width are:

Refer to AS 5100.6, clause 6.1.7.

Abutment and mid-span sections: $L_c = 0.7 \times L_1 = 0.7 \times 28 = 19.6\text{m}$

Hogging section: $L_c = 0.25(L_1 + L_2) = 0.25 \times 56 = 14.0m$

At mid-span, $b_{eff} = \min\left(\frac{L_c}{5};S;12t_s\right) = \min\left(\frac{19600}{5};3700;12\times 250\right) = 3000mm$

At central pier, $b_{eff} = \min\left(\frac{14000}{5};3700;12\times 250\right) = 2800mm$

Refer to AS 5100.6, clause 4.4.1

Properties for gross sections at the central pier and in the span are tabulated below.

Figure F.5 Internal main girders effective width

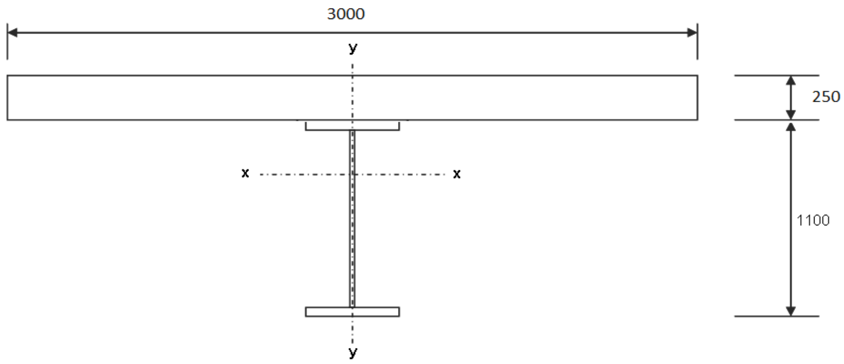


Table F.4 Bare steel cross sections

		Span girder	Pier girder	Unit
Area	A	57,200	91,800	mm^2
ENA height	ENA	448	485	mm
Second moment of area	I_x	1.2E+10	2.1E+010	mm^4
Section modulus, top flange	$Z_{x,\,tf}$	18E+06	34E+06	mm^3
Section modulus, bottom flange	$Z_{x,\,bf}$	26E+06	43E+06	mm^3
Section class		Not compact	Compact	
Section bending capacity	M_{sx}	5040	11480	kN.m

Slenderness check for span girder:

$$\lambda_{e,flange} = \frac{b_f - t_w}{2t_f}\sqrt{\frac{f_{y,flange}}{250}} \qquad \lambda_{e,web} = \frac{d_1}{t_w}\sqrt{\frac{f_{y,web}}{250}}$$

$$\lambda_{e,t\_flange} = \frac{500-20}{2\times 25}\sqrt{\frac{280}{250}} = 10.20 \qquad \lambda_{e,web} = \frac{1035}{20}\sqrt{\frac{300}{250}} = 56.68$$

Refer to AS 5100.6, section 5.1.2.

$$\lambda_{e_{y,flange}} = 14$$

$$r_e = \frac{(1100 - 448 - 25)}{1035} = 0.61$$

$$\lambda_{e_{y,web}} = \frac{60}{r_e} = \frac{60}{0.61} = 98$$

$$\lambda_{e_S} = \lambda_{e_{flange}} = 10.20$$

$$\lambda_{e_p} = 8$$

$$\frac{\lambda_{e,t\_flange}}{\lambda_{e_{y,flange}}} = \frac{10.20}{14} = 0.73$$

$$\frac{\lambda_{e,web}}{\lambda_{e_{y,web}}} = \frac{56.68}{98} = 0.58$$

$$\frac{\lambda_{e,flange}}{\lambda_{e_{y,flange}}} > \frac{\lambda_{e,web}}{\lambda_{e_{y,web}}} \rightarrow \text{flange governs}$$

$$\lambda_{e_S} \geq \lambda_{e_p}$$

Refer to AS5100.6, table 5.1

Therefore, the bare steel section at span girder is **not compact**.

Slenderness check for bare steel pier girder:

$$\lambda_{e,flange} = \frac{b_f - t_w}{2t_f}\sqrt{\frac{f_{y,flange}}{250}}$$

$$\lambda_{e,b\_flange} = \frac{700 - 20}{2 \times 60}\sqrt{\frac{280}{250}} = 6.00$$

$$\lambda_{e,web} = \frac{d_1}{t_w}\sqrt{\frac{f_{y,web}}{250}}$$

$$\lambda_{e,web} = \frac{990}{20}\sqrt{\frac{300}{250}} = 54.22$$

$$\lambda_{e_{y,flange}} = 14$$

$$r_e = \frac{485 - 60}{990} = 0.43$$

bottom flange is in compression in the pier girder

$$\lambda_{e_{y,web}} = \frac{60}{r_e} = \frac{60}{0.43} = 139.5$$

$$\lambda_{e_S} = \lambda_{e_{flange}} = 6.00$$

$$\lambda_{e_p} = 8$$

$$\lambda_{e_S} \leq \lambda_{e_p}$$

$$\frac{\lambda_{e,t\_flange}}{\lambda_{e_{y,flange}}} = \frac{6.0}{14} = 0.43$$

$$\frac{\lambda_{e,web}}{\lambda_{e_{y,web}}} = \frac{54.22}{139.5} = 0.39$$

$$\frac{\lambda_{e,flange}}{\lambda_{e_{y,flange}}} > \frac{\lambda_{e,web}}{\lambda_{e_{y,web}}} \rightarrow \text{flange governs}$$

Therefore, the bare steel section at pier girder is **compact**.

Table F.5 Composite cross section (short term) – sagging (n_S=6.25)

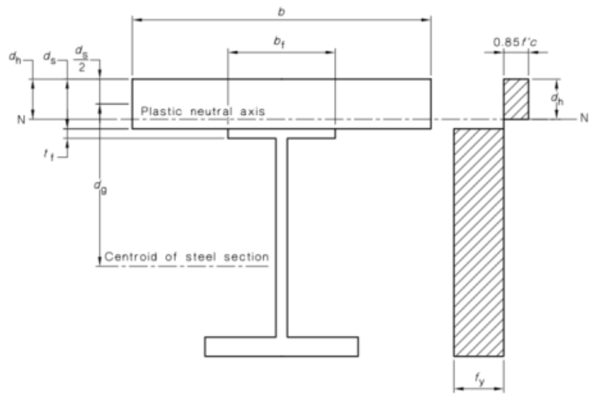
		Span girder	Pier girder	Unit
Area	A	177,200	203,800	mm^2
ENA height	ENA	974	892	mm
Second moment of area	I_x	3.6E+10	4.9E+10	mm^4
Section modulus, top of slab	$Z_{x,c}$	95E+06	107E+06	mm^3
Section modulus, top flange	$Z_{x,tf}$	286E+06	236E+06	mm^3
Section modulus, bottom flange	$Z_{x,bf}$	36E+06	55E+06	mm^3
Section class		Compact	-	-
Plastic bending capacity	M_{pl}	13,220	-	kN.mm

The cross section of the span girder is compact, provided the top flange is restrained by shear connectors within the spacing limits given in *AS 5100.6*, clause 6.1.8

Plastic moment capacity for a compact composite section can be calculated using the formula given in *AS 5100.6*, appendix E.

Refer to AS 5100.6, appendix E

Figure F.6 Section moment capacity compression zone entirely within the concrete slab



$$d_h = \frac{f_y A}{0.85 f_c' b} = \frac{280 \times 57{,}200}{0.85 \times 40 \times 3000} = 157\text{mm}$$

$$d_h \le d_s$$

$$M_P = Af_y\left(d_g + \frac{d_s - d_h}{2}\right) = 57{,}200 \times 280\left(779 + \frac{250 - 157}{2}\right) \times 10^{-6} = 13{,}220\text{kN.m}$$

Note: Uncracked pier girder section properties are needed for the calculation of shear flow.

Note: During the construction of a composite bridge, it is quite likely that a beam will change its section class, because the addition of the deck slab both prevents local buckling of the top flange and significantly shifts the neutral axis of the section. Typically, a mid-span section could be compact after casting the slab but not prior to this. As a consequence, checks at intermediate stages of construction should be based on the relevant classification for the stage being checked.

Table F.6 Composite cross sections (long term) – (n_L=19)

		Span girder	Pier girder	Unit
Area	A	96,575	128,550	mm^2
ENA height	ENA	765	696	mm
Second moment of area	I_x	2.6E+10	3.5E+10	mm^4
Section modulus, top of slab	$Z_{x,c}$	44E+06	54E+06	mm^3
Section modulus, top flange	$Z_{x,tf}$	78E+06	87E+06	mm^3
Section modulus, bottom flange	$Z_{x,bf}$	34E+06	51E+06	mm^3

Table F.7 Composite cross sections – hogging (cracked)

		Span girder	Pier girder	Unit
Area	A	77,817	110,453	mm^2
ENA height	ENA	654	610	mm
Second moment of area	I_x	2.1E+10	2.9E+10	mm^4
Section modulus, top rebars	$Z_{x,c}$	33E+06	42E+06	mm^3
Section modulus, top flange	$Z_{x,tf}$	47E+06	60E+06	mm^3
Section modulus, bottom flange	$Z_{x,bf}$	32E+06	48E+06	mm^3
Section class		-	Not compact	-

Slenderness check for composite pier girder:

Refer to AS 5100.6, clause 5.1.2

$\lambda_{e,t\_flange} = 6.14$

$\lambda_{e,b\_flange} = 6.00$

$\lambda_{e,web} = 54.22$

$\lambda_{e_{y,flange}} = 14$

$r_e = \frac{610 - 60}{990} 0.56$

$\lambda_{e_{y,web}} = \frac{60}{r_e} = \frac{60}{0.56} = 108$

Refer to AS 5100.6, table 5.1

$\frac{\lambda_{e,t\_flange}}{\lambda_{e_{y,flange}}} = \frac{6.14}{14} = 0.44$

$\frac{\lambda_{e,web}}{\lambda_{e_{y,web}}} > \frac{\lambda_{e,flange}}{\lambda_{e_{y,flange}}} \rightarrow$ web governs

$\frac{\lambda_{e,web}}{\lambda_{e_{y,web}}} = 108 = 0.50$

$\lambda_{e_S} = \lambda_{e_{web}} = 54.22$

Plastic neutral axis =1050mm

$r_p = \frac{1050-60}{990} = 1.0$ $\quad \lambda_{e_p} = \frac{111}{4.7r_p-1} = \frac{111}{4.7\times1.0-1} = 30$ $\quad \lambda_{e_S} \geq \lambda_{e_p}$

Therefore, the composite steel section is **not compact**.

F6.2 Primary effects of temperature difference and shrinkage

F6.2.1 Temperature difference

Refer to section 7.3.1 in the main part of the guide.

For calculation of primary effects, the short-term modulus of concrete is used.

$E_c = 32GPa$ $\quad E_s = 200GPa$

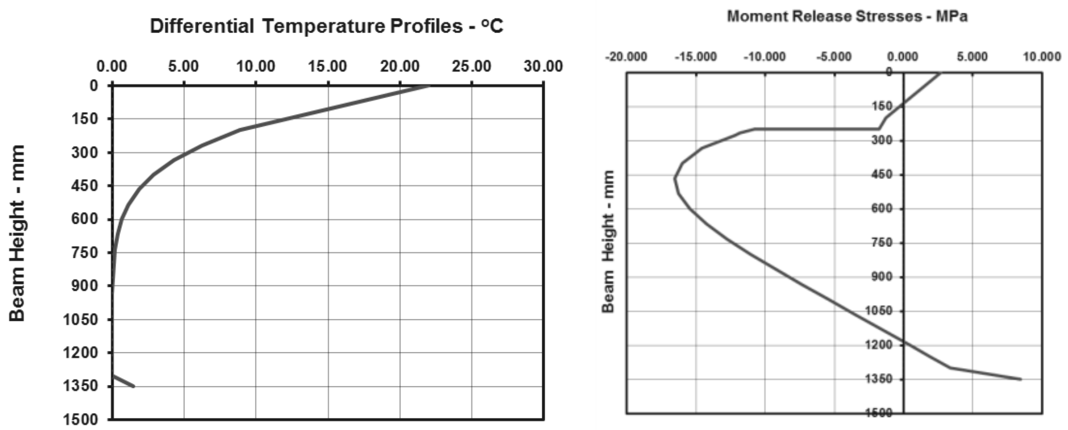
For each element of the section, calculate stress as strain times modulus of elasticity, then determine the force and centre of force for that area. The restraint moment in the inner beam, due to the characteristic values of temperature difference noted in the *Bridge manual,* section 3.4.6 is:

Assume 50mm blacktop thickness for this case only:

$T = (32 - 0.2d)^{\circ}C$ $\quad d = 50mm$ $\quad T = 22^{\circ}C$

The moment release stress is shown diagrammatically below.

Figure F.7 Moment release stress over the depth of the span-girder section



The effects of the negative vertical temperature variation are insignificant and therefore have been ignored.

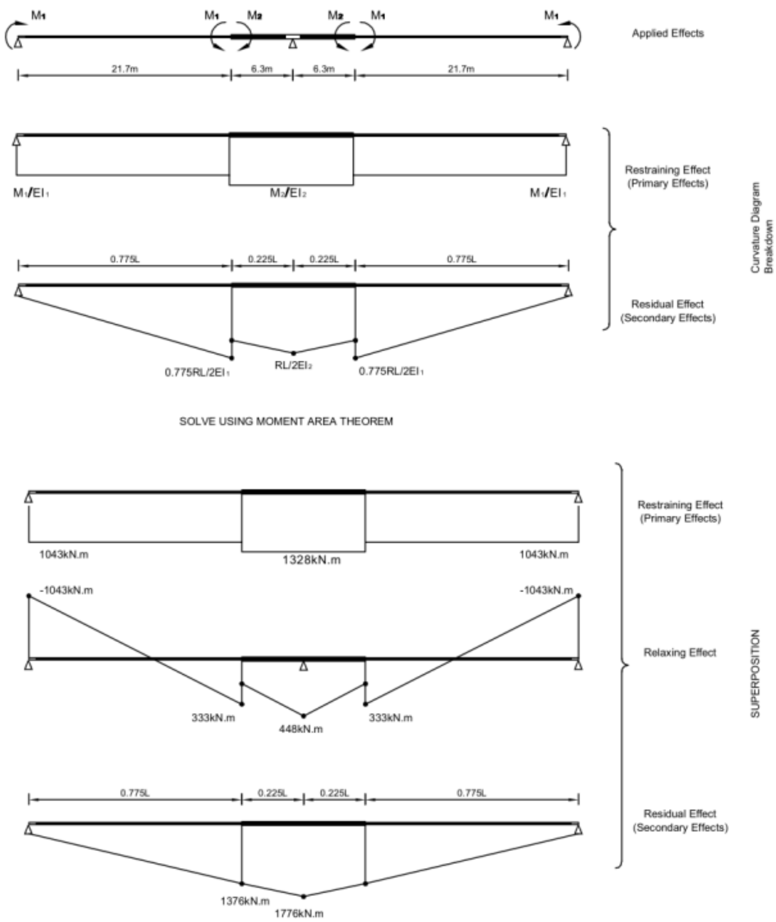
Table F.8 Temperature induced force and moment effects

		Span girder	Pier girder
Axial fixity force	(kN)	4196	4182
Fixity moment	(kN.m)	-1043	-1328

The release of the restraint moments is applied along the span to determine the secondary effects of the vertical temperature difference.

Note: The following procedure shows how to calculate the secondary moment on the main girders.

Figure F.8 Method to calculate main girder secondary moments



The primary effects of differential temperature through the depth of the cross section of a member are considered in the design. In addition, the secondary effects in continuous members, due to redistribution of the moments and support reactions caused by the primary effects are also considered.

Refer to AS 5100.6, clause 4.9.1.1.

F6.2.2 Shrinkage

The effects of shrinkage are calculated for the long-term situation, and where the total effects of shrinkage are advantageous, they are ignored.

The characteristic value of shrinkage is $\varepsilon_{sh} = 0.0002$ and the long-term modular ratio is used.

For a fully restrained section, the restraint force and moment in the span and pier girders, and inner beams, due to the characteristic value of shrinkage strain, are given by:

Table F.9 Shrinkage induced restraint force and moment

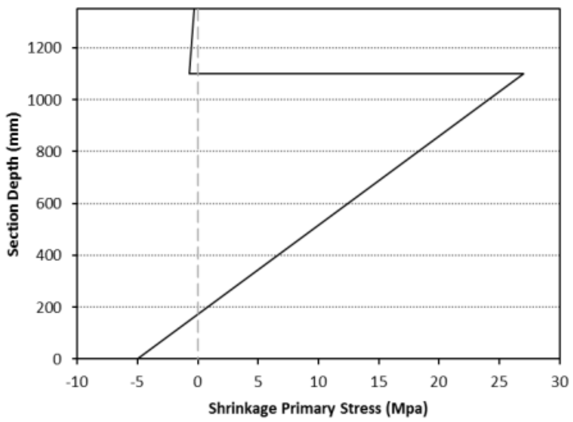
	Strain	Force kN	Centre of force above NA mm	Moment kN.m
@ span girder	0.0002	1943	460	894
@ pier girder	“	“	529	1028

Hence the primary effects for the pier girder are:

Table F.10 Primary effects for the pier girder

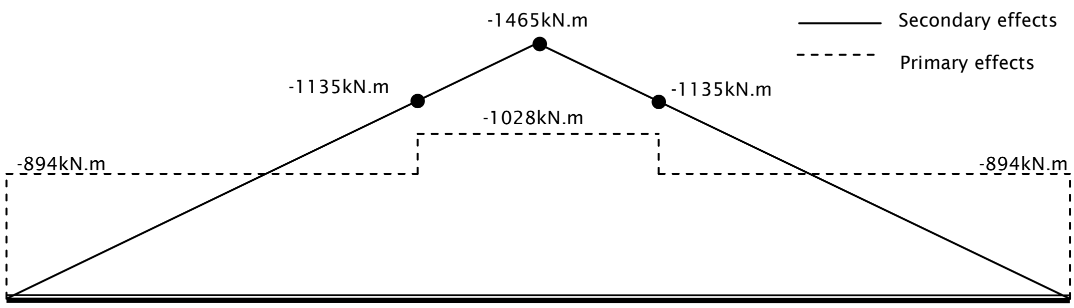
Pier girder	Zx (mm3)	Restraint (MPa)	Release of restraint		Total (MPa)
			Bending (MPa)	Axial (MPa)	
Top of concrete slab	54E+06	-2.1	1	0.8	-0.3
Bottom of concrete slab	87E+06	-2.1	0.6	0.8	-0.7
Steel top flange	87E+06	0	12	15	27
Steel bottom flange	51E+06	0	-20	15	-5
Top reinforcement	58E+06	0	18	15	33
Bottom reinforcement	76E+06	0	13.5	15	28.5

Figure F.9 Primary effect stresses over the depth of the pier girder section



The release of restraint moments is applied along the girder to determine the effects of shrinkage.

Figure F.10 Primary and secondary effects of shrinkage



Refer to AS 5100.6, clause 4.9.1.2

Note: The previously-shown procedure can also be used to calculate the secondary effect of shrinkage on the main girders.

Refer to AS 5100.6, clause 4.9.2.1

When the effects of shrinkage modified by creep adversely affect the structure they are considered in the manner described for differential temperature effects.

The effects of shrinkage and differential temperature are considered at the SLS for composite beams with sections that are not compact at internal supports.

Account is taken of the longitudinal shear forces arising from shrinkage and differential temperature effects in the design of all composite beams for the serviceability.

Refer to AS5100.6, clause 4.9.3.2

The longitudinal shear forces arising from the effects of shrinkage and differential temperature are considered in the design of the longitudinal and transverse reinforcement in the concrete slab.

Refer to AS 5100, clauses 6.4.2 and 6.4.3

F7 Global analysis

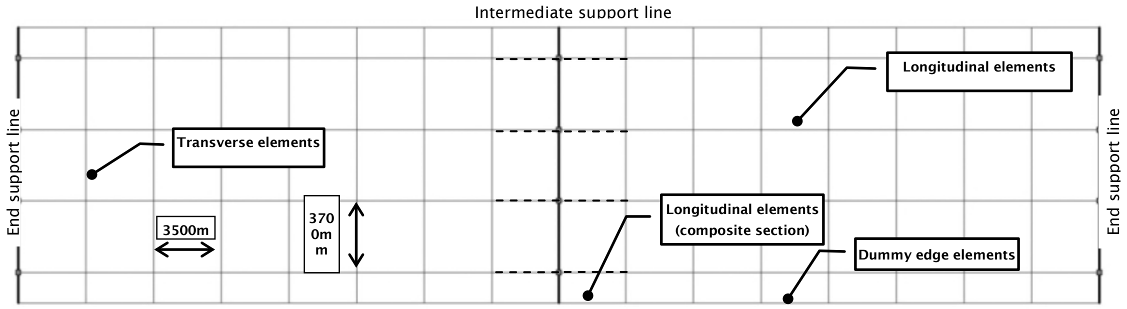
F7.1 Grillage model

A grillage model of the superstructure was created, with longitudinal elements for composite girder and transverse elements for the concrete slab.

For the crack region over the intermediate support (15% of each span) the composite elements were given cracked properties but transverse elements remained uncracked.

As the bridge is simply supported at end supports, no concrete diaphragms are provided at both abutments.

Figure F.11 Grillage model of the superstructure



Note: The grillage model utilised a transverse elements comprising a rectangular slab 3500mm by 250mm. Modern computer models automatically generate section properties; however, these should be checked manually at least for torsional properties to suit the designer's assumptions.

F7.2 Construction stages

It is assumed that the deck will be concreted in two stages: first the whole of span 1, followed by the whole of span 2. The edge beams will be concreted after span 2. Separate analytical models are therefore provided for:

Stage 1 All steelwork, wet concrete in span 1

Stage 2 Composite structure in span 1 (long-term properties), wet concrete in span 2

Stage 3 Composite structure in both spans (long-term properties)

Stage 4 Composite structure (short-term properties).

F7.3 Analysis results

The following results are for design values of actions, ie after application of appropriate load factors on nominal values of actions.

Note: the following results may not correlate with a slightly different model and hence should not be viewed as absolute. They do, however, provide an indication of the order of magnitude expected.

F7.3.1 Stage 1

Selfweight of steelwork

Selfweight of concrete on span 1

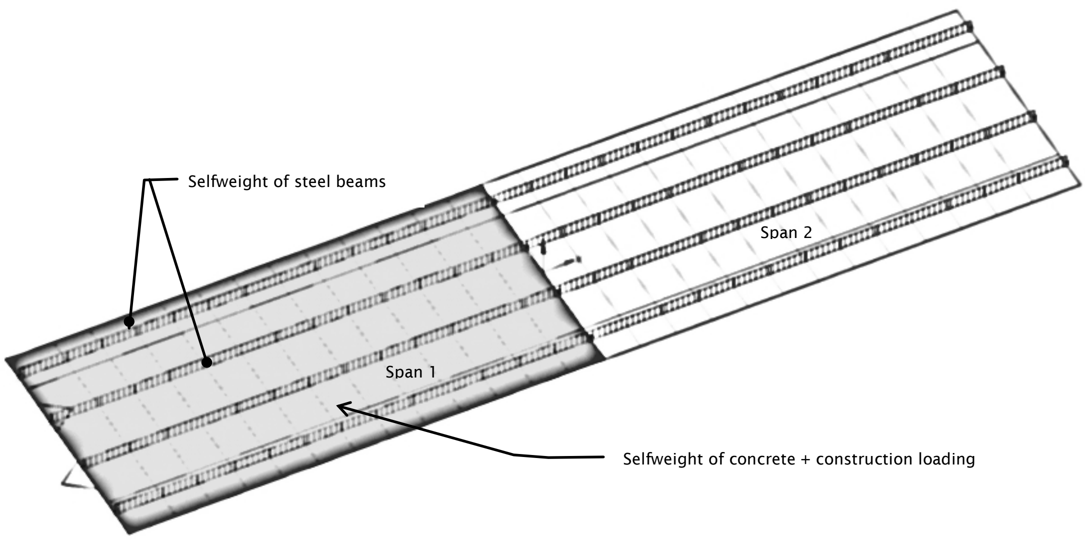
Construction load on span 1

Table F.11 Stage 1 design actions

Distance from pier (m)	ULS		SLS	
	M_y (kN.m)	F_z (kN)	M_y (kN.m)	F_z (kN)
0	-2580	685	-1960	523
6.3	1083	478	834	365
16.8	3211	-73	2454	-57
28	0	-488	0	-373

Load combination 5B is assumed to be critical

Figure F.12 Stage 1 loading



F7.3.2 Stage 2

1 Selfweight of concrete on span 2

2 Construction load on span 2

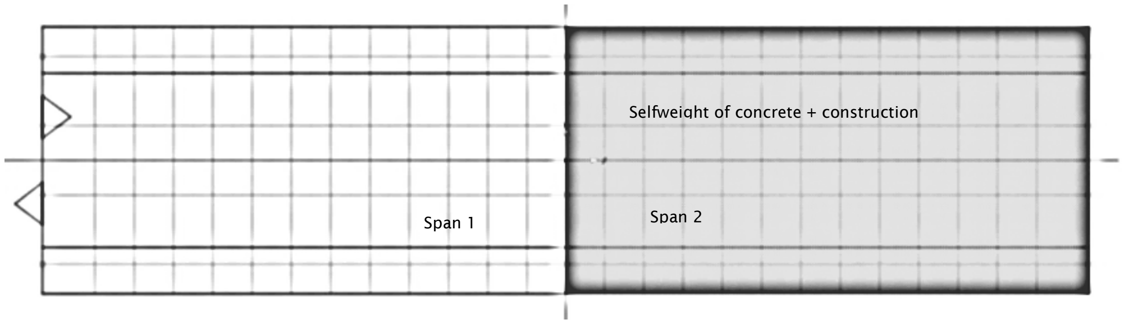
3 Removal of construction load on span 1.

Table F.12 Stage 2 design actions

Distance from pier (m)	ULS		SLS	
	M_y (kN.m)	F_z (kN)	M_y (kN.m)	F_z (kN)
0	-2231	-15	-1714	-12
6.3	-1926	87	-1480	67
16.8	-949	90	-729	69
28	0	87	0	67

Load combination 5B is assumed to be critical

Figure F.13 Stage 2 loading



F7.3.3 Stage 3

1 Selfweight of parapets

2 Selfweight of carriageway surfacing

3 Selfweight of footpath construction

4 Removal of construction loads on span 2.

Figure F.14 Stage 3 loading

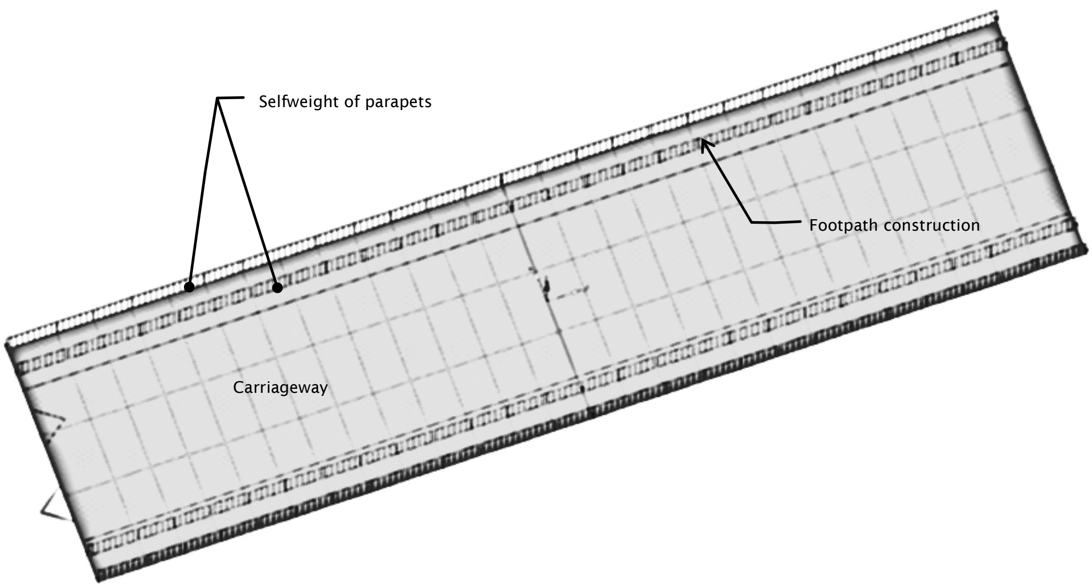


Table F.13 Stage 3 design actions

Distance from pier (m)	ULS		SLS	
	M_y (kN.m)	F_z (kN)	M_y (kN.m)	F_z (kN)
0	-1696	286	-1256	212
6.3	-76	220	-56	163
16.8	1120	-14	829	-10
28	0	-158	0	-117

Load combination 5B is assumed to be critical

F7.3.4 Long-term shrinkage

The nominal value of secondary moment is calculated using the procedures in figure F.8. These need to be factored depending on the ULS and SLS load effects

Table F.14 Secondary design actions

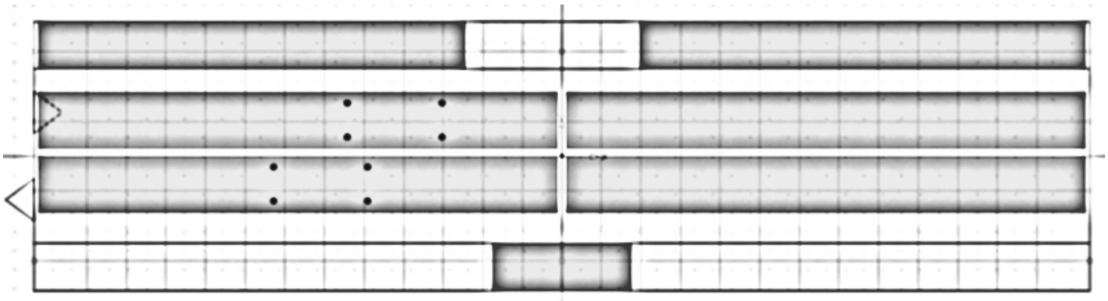
Distance from pier (m)	ULS/SLS	
	M_y (kN.m)	F_z (kN)
0	-1465	52
6.3	-1135	52
16.8	-586	52
28	0	52

F7.3.5 Stage 4

Table F.15 HN72 traffic loads for worst hogging at intermediate support

Distance from pier (m)	ULS		SLS	
	M_y (kN.m)	F_z (kN)	M_y (kN.m)	F_z (kN)
0	-3089	754	-1844	451
6.3	1262	648	757	387
16.8	2720	-151	1623	-90
28	0	-354	0	-212

Load combination 1A is assumed to be critical

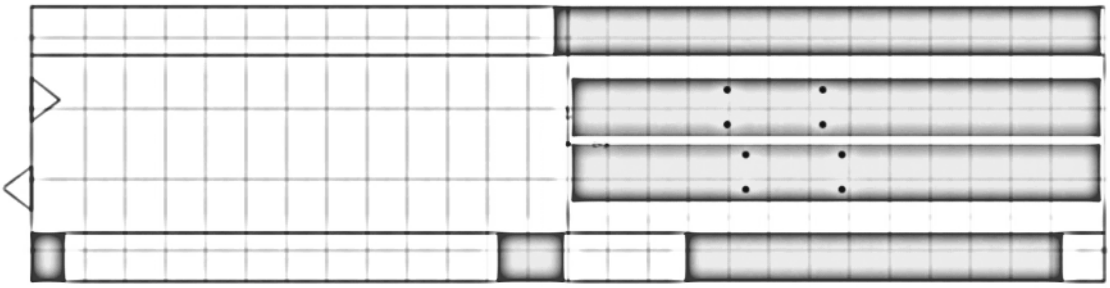


Relative position of vehicle axle loads and pedestrian live load for the worst hogging at intermediate support

Table F.16 HN72 traffic load for worst hogging at splice position

Distance from pier (m)	ULS		SLS	
	M_y (kN.m)	F_z (kN)	M_y (kN.m)	F_z (kN)
0	-2052	94	-1226	56
6.3	-1465	88	-873	53
16.8	-684	66	-406	39
28	0	62	0	37

Load combination 1A is assumed to be critical

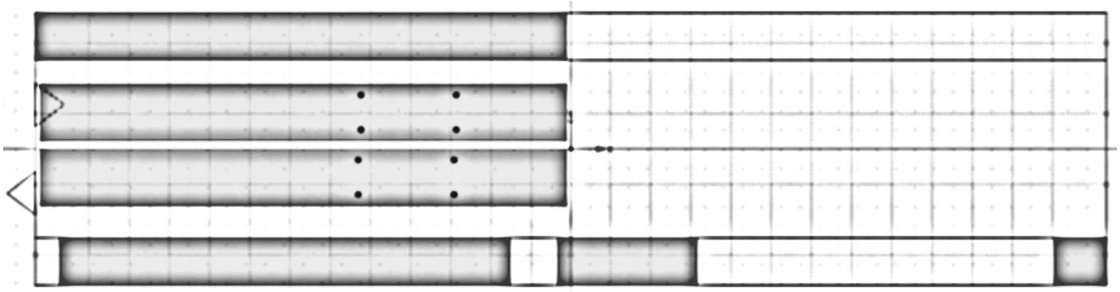


Relative position of vehicle axle loads and pedestrian live load for the worst hogging at splice

Table F.17 HN72 traffic load for worst sagging at splice position

Distance from pier (m)	ULS		SLS	
	M_y (kN.m)	F_z (kN)	M_y (kN.m)	F_z (kN)
0	-2032	729	-1215	436
6.3	2104	583	1256	348
16.8	2820	-166	1680	-99
28	0	-352	0	-210

Load combination 1A is assumed to be critical

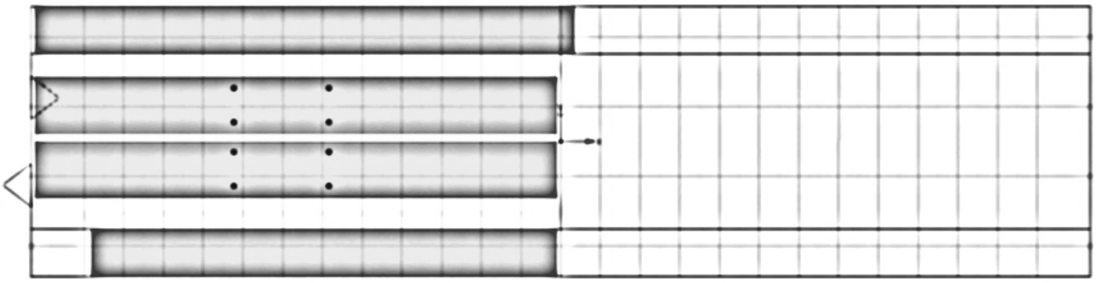


Relative position of vehicle axle loads and pedestrian live load for the worst sagging at splice

Table F.18 HN72 traffic load for worst sagging in span

Distance from pier (m)	ULS		SLS	
	M_y (kN.m)	F_z (kN)	M_y (kN.m)	F_z (kN)
0	-1907	571	-1139	341
6.3	1318	471	785	281
16.8	3798	-217	2262	-130
28	0	-447	0	-267

Load combination 1A is assumed to be critical

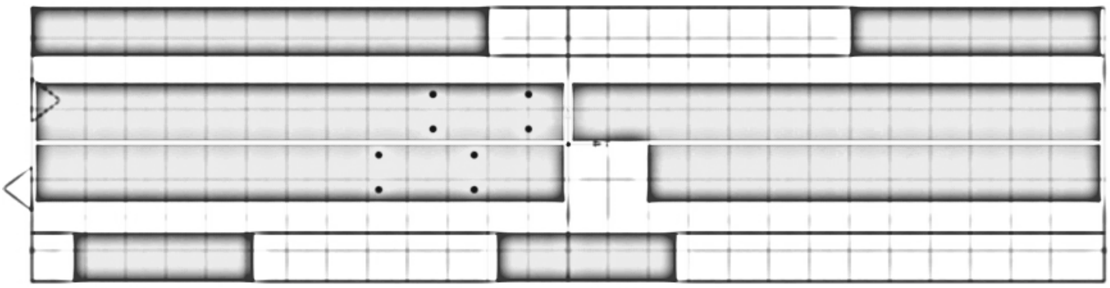


Relative position of vehicle axle loads and pedestrian live load for the worst sagging at mid-span

Table F.19 HN72 traffic load for maximum shear forces

Distance from pier (m)	ULS		SLS	
	M_y (kN.m)	F_z (kN)	M_y (kN.m)	F_z (kN)
Pier	-2805	877	-1675	525
Splice/pier side	765	583	458	348
Splice/span side	755	72	454	44
Span	3271	-321	1950	-192
Abutment	0	-670	0	-401

Load combination 1A is assumed to be critical



Relative position of vehicle axle loads and pedestrian live load for the worst shear at intermediate support

Table F.20 Effects of thermal actions

Distance from pier (m)	ULS/SLS	
	M_y (kN.m)	F_z (kN)
0	1776	-63
6.3	1376	-63
16.8	710	-63
28	0	-63

F8 Design values of the effects of combined actions

This section provides design values for certain situations in the design of the inner beams. In practice, the design of other parts of the structure would also need to be considered.

F8.1 Effects of construction loads

Generally, the effects of construction loads apply to different cross-section properties, although for span 1, the cross sections for the inner beam are the same at stages 2 and 3. The following tabulations summarise the forces and moments at each stage and the stresses due to those effects, for selected cross sections. According to the *Bridge manual* the load combination 5B is the most critical load case for construction stages.

Table F.21 Stress at pier (using steel and cracked concrete section properties)

			Bottom flange		Top flange		Top rebars	
	M_y	F_z	$Z_{x,\,bf}$	σ	$Z_{x,\,tf}$	σ	$Z_{x,\,c}$	σ
	kN.m	kN	10^6mm^3	MPa	10^6mm^3	MPa	10^6mm^3	MPa
Stage 1	-2580	685	43	-60	34	76	-	-
Stage 2	-2231	-15	48	-46	60	37	42	53
Stage 3	-1696	286	48	-35	60	28	42	40
Shrinkage - PE	-	-	-	-7	-	36	-	45
Shrinkage - SE	-1978	70	48	-41	60	33	42	47
Temperature	(not adverse)							
	-8485	1026		-189		210		185

Load combination 5B is assumed to be critical

Table F.22 Stress at splice (using steel and cracked concrete section properties)

			Bottom flange		Top flange		Top rebars	
	M_y	F_z	$Z_{x,\,bf}$	σ	$Z_{x,\,tf}$	σ	$Z_{x,\,c}$	σ
	kN.m	kN	10^6mm^3	MPa	10^6mm^3	MPa	10^6mm^3	MPa
Stage 1	1083	478	43	25	34	-32	-	-
Stage 2	-1926	87	48	-40	60	32	42	46
Stage 3	-76	220	48	-2	60	1	42	2
Shrinkage - PE	-	-	-	-7	-	36	-	45

			Bottom flange		Top flange		Top rebars	
Shrinkage - SE	-1532	70	48	-32	60	26	42	36
Temperature	(not adverse)							
	-2451	855		-56		63		129

Load combination 5B is assumed to be critical

Table F.23 Stress at span girder (using steel and long-term concrete section properties)

			Bottom flange		Top flange		Top rebars	
	M_y	F_z	$Z_{x,\ bf}$	σ	$Z_{x,\ tf}$	σ	$Z_{x,\ c}$	σ
	kN.m	kN	10^6mm^3	MPa	10^6mm^3	MPa	10^6mm^3	MPa
Stage 1	3211	-73	26	124	18	-178	-	-
Stage 2	-949	90	34	-28	78	12	44	22
Stage 3	1120	-14	34	33	78	-14	44	-25
Shrinkage	(not adverse)							
Temp - PE	-	-	-	4	-	-4	-	1
Temp - SE	316	-28	34	9	78	-4	44	-7
	3698	-25		142		-186		-9

Load combination 5B is assumed to be critical

As the support at each abutment is assumed to be simply supported, the stress at the abutment will be zero.

F8.2 Effects of traffic load plus construction stages loads (ULS)

The worst effects are due to HN72 traffic loads.

Table F.24 Loading for maximum hogging at pier (using cracked concrete section properties)

			Bottom flange		Top flange		Top rebars	
	M_y	F_z	$Z_{x,\ bf}$	σ	$Z_{x,\ tf}$	σ	$Z_{x,\ c}$	σ
	kN.m	kN	10^6mm^3	MPa	10^6mm^3	MPa	10^6mm^3	MPa
Stages 1, 2, 3	-6507	956	-	-141	-	141	-	93
HN72 traffic	-3089	754	48	-64	60	51	42	74
Shrinkage - PE	-	-	-	-7	-	36	-	45
Shrinkage - SE	-1978	70	48	-41	60	33	42	47
	-11,574	1780		-254		219		259

Load combination 1A is assumed to be critical

The maximum hogging moment at the splice position is much greater than the maximum sagging moment, so this will govern the design of the splice.

Table F.25 Effects at splice position (using cracked concrete section properties)

			Bottom flange		Top flange		Top rebars	
	M_y	F_z	$Z_{x, bf}$	σ	$Z_{x, tf}$	σ	$Z_{x, c}$	σ
	kN.m	kN	10^6mm^3	MPa	10^6mm^3	MPa	10^6mm^3	MPa
Stages 1, 2, 3	-919	785	-	-17	-	-1	-	48
HN72 traffic	-1465	88	48	-31	60	24	42	35
Shrinkage - PE	-	-	-	-7	-	36	-	45
Shrinkage - SE	-1532	70	48	-32	60	26	42	36
	-3916	943		-86		88		164

Load combination 1A is assumed to be critical

Table F.26 Loading for maximum sagging bending (using short-term composite section properties)

			Bottom flange		Top flange		Top rebars	
	M_y	F_z	$Z_{x, bf}$	σ	$Z_{x, tf}$	σ	$Z_{x, c}$	σ
	kN.m	kN	10^6mm^3	MPa	10^6mm^3	MPa	10^6mm^3	MPa
Stages 1, 2, 3	3382	3	-	129	-	-180	-	-4
HN72 traffic	3798	-217	36	106	286	-13	95	-40
Shrinkage	(not adverse)							
	7180	-214		235		-193		-44

Load combination 1A is assumed to be critical

F8.2.1 Loading for maximum shear

The value of maximum shear is needed to verify the shear resistance of the web and to determine the longitudinal shear on the stud connectors.

Table F.27 Maximum shear at pier position

	M_y	F_z
	kN.m	**kN**
Stages 1, 2, 3	-6057	956
HN72 traffic	-2805	877
Shrinkage	-1978	70
	-10,840	1903

Note: Load combination 1A is assumed to be critical

The value of the maximum shear is needed to determine the maximum longitudinal shear on the stud connector.

Table F.28 Maximum shear at splice position

	M_y	F_z
	kN.m	kN
Stages 1, 2, 3	-919	785
HN72 traffic	765	583
Shrinkage	-1532	70
	-1686	1438

Note: Load combination 1A is assumed to be critical

The value of the maximum shear is needed to determine the maximum longitudinal shear on the stud connector.

Table F.29 Maximum shear at span girder

	M_y	F_z
	kN.m	kN
Stages 1, 2, 3	3382	3
HN72 traffic	3271	-321
Shrinkage	(not adverse)	
	6653	-318

Note; Load combination 1A is assumed to be critical

Table F.30 Maximum shear at abutment

	M_y	F_z
	kN.m	kN
Stages 1, 2, 3	0	-559
HN72 traffic	0	-670
Shrinkage	(not adverse)	
	0	-1229

Note: Load combination 1A is assumed to be critical

F8.3 Effects of traffic load plus construction stages loads (SLS)

The values of effects at SLS are needed to verify crack control in the slab at the pier and to verify the slip resistance of the splice.

Table F.31 Effects at pier position (worst shear)

			Bottom flange		Top flange		Top rebars	
	M_y	F_z	$Z_{x, bf}$	σ	$Z_{x, tf}$	σ	$Z_{x, c}$	σ
	kN.m	kN	10^6mm^3	MPa	10^6mm^3	MPa	10^6mm^3	MPa
Stage 1	-1960	523	43	-46	34	58		
Stage 2	-1714	-12	48	-36	60	29	42	41
Stage 3	-1256	212	48	-26	60	21	42	30
Shrinkage -PE	-	-	-	-5	-	27	-	33
Shrinkage - SE	-1465	52	48	-31	60	24	42	35
HN72 traffic	-1844	525	48	-38	60	31	42	44
	-8239	1300		-182		190		183

Note; Load combination 1A is assumed to be critical

Table F.32 Effects at splice position (worst shear)

			Bottom flange		Top flange		Top rebars	
	M_y	F_z	$Z_{x, bf}$	σ	$Z_{x, tf}$	σ	$Z_{x, c}$	σ
	kN.m	kN	10^6mm^3	MPa	10^6mm^3	MPa	10^6mm^3	MPa
Stage 1	834	365	43	19	34	-25	-	-
Stage 2	-1480	67	48	-31	60	25	42	35
Stage 3	-56	163	48	-1	60	1	42	1
Shrinkage -PE	-	-	-	-5	-	27	-	33
Shrinkage - SE	-1135	52	48	-24	60	19	42	27
HN72 traffic	458	348	48	10	60	-8	42	-11
	-1379	995		-32		39		85

Note: Load combination 1A is assumed to be critical

F9 Verification of bare steel girder during construction

See figure 7.4 in the main part of the guide

The paired beams are susceptible to lateral torsional buckling under the weight of the wet concrete (ie before it hardens and provides restraint to the top flanges).

The beams are partially restrained against buckling by the bracing frames between each pairs at three points in the span.

Use gross section properties for verification of buckling resistance.

F9.1 Evaluation of lateral restraint

Refer to section 7.1.2 in the main part of the guide.

If the length of the segment between braces satisfies the requirements in *AS 5100.6*, clause 5.3.2.4, the nominal member moment capacity of a segment is taken as the nominal section moment capacity otherwise the member moment capacity is calculated using the expression in *AS 5100.6*, clause 5.6.1

$\frac{L}{r_y} \le (80 + 50\beta_m)\sqrt{\frac{2pd_f A}{2.5Z_{ex}}}\sqrt{\frac{250}{f_y}}$ if the segment is of unequal flanged I section

Refer to AS 5100.6, clause 5.3.2.4

$\beta_m = +0.2$ this value can be conservatively taken as -1.0

Refer to AS 5100.6, figure 4.2.2.2

$L_{s,max} = 7367mm$

$f_y = 280MPa$

$r_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{9.81E+08}{57200}} = 131mm$ $\quad \frac{L_{s,max}}{r_y} = \frac{7367}{131} = 56$ $\quad \rho = \frac{\frac{25 \times 500^3}{12}}{9.81 \times 10^8} = 0.26$

$d_f = 1067.5mm$

$$(80 + 50\beta_m)\sqrt{\frac{2pd_f A}{2.5Z_{ex}}}\sqrt{\frac{250}{f_y}} = 90\sqrt{\frac{2 \times 0.66 \times 1067.5 \times 57{,}200}{2.5 \times 18 \times 10^6}}\sqrt{\frac{250}{280}} = 60$$

$\frac{L}{r_y} \le (80 + 50\beta_m)\sqrt{\frac{250}{f_y}}$ $\quad \therefore M_b = M_s$

F9.2 Verification

$Z_{e,x} = \min\left(Z_{x,tf}; Z_{x,bf}\right)$

$Z_{x,tf} = 18 \times 10^6 mm^3$

$Z_{x,bf} = 26 \times 10^6 mm^3$

$Z_{e,x} = 18 \times 10^6 mm^3$

$\phi M_s = \phi Z_{e,x} f_y = 5040kN.m$

$I_y = 9.81 \times 10^8 mm^4$

$I_w = 0$

$J = 12240 \times 10^3 mm^4$

$\phi M_b = \phi M_s = 5040kN.m$

E = 200,000MPa

G = 80,000MPa

$M^* = 3211kN.m < \phi M_b = 5040kN.m - OK$

F10 Verification of composite girder

F10.1 In hogging bending

See figure 7.5 in the main part of the guide.

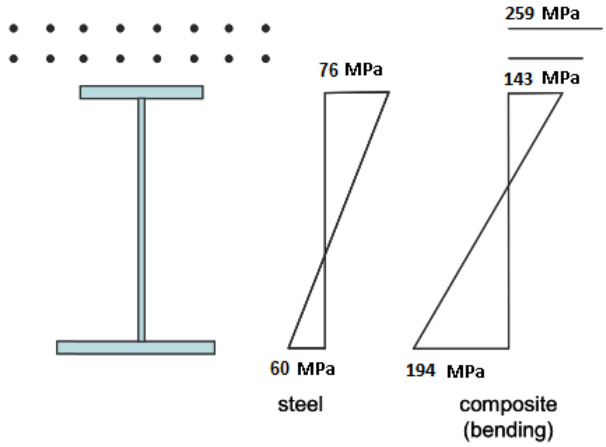
The composite section in hogging is not compact.

The elastic design bending resistance for a beam constructed in stages depends on the design effects at the stages.

From sections F8.1 and F8.2, the design moment on the steel section only is 2580kN.m and the total hogging moment is 11,574kN.m, which means that the moment on the composite (cracked) section is 8994kN.m. The stresses are shown in figure F.15:

Figure F. 15 Bare steel and composite stresses in hogging

Refer to AS 5100.6, clause 6.2.2.



Stresses:
Bottom: – ve = compression
Top: + ve = tension

As the composite section in hogging region is not compact, for verification of cross-section resistance, the stresses should not exceed the limiting stress.

$f_s^* \le \phi f_y$

$f_s^* \le \phi f_b$

$$f_b = \frac{M_b}{Z_{enc}} \le f_y$$

$$M_b = M_{ds}\left(\frac{Z_{ec}}{Z_{es}}\right) \le M_s$$

Refer to AS 5100.6, clause 6.4.2.2.

$Z_{es} = \min(1.5Z_{x,tf}; 1.5Z_{x,bf}; S_x)$

$Z_{es} = \min(1.5 \times 34E+06; 1.5 \times 43E+06; 41E+06) = 41E+06$

$Z_{ec} = 48E+06$

$M_{ds} = \alpha_d M_{ss} \le M_{ss}$

$$M_{ss} = \frac{41 \times 10^6 \times 280}{10^6} = 11,480\text{kN.m}$$

$$\lambda_d = 0.018\left(\frac{L_b}{r_y}\right)^{\frac{1}{2}}\left(\frac{d_w}{t_w}\right)^{\frac{1}{3}} - 0.4$$

$$\lambda_d = 0.018\left(\frac{28,000}{202}\right)^{\frac{1}{2}}\left(\frac{990}{20}\right)^{\frac{1}{3}} - 0.4 = 0.38$$

$$r_y = \sqrt{\frac{\frac{60 \times 700^3}{12}}{50 \times 700}} = 202\text{mm}$$

$$\alpha_d = 0.8\left(\sqrt{\lambda_d^4 + 3} - \lambda_d^2\right)$$

$$\alpha_d = 0.8\left(\sqrt{0.38^4 + 3} - 0.38^2\right) = 1.27$$

$\alpha_d > 1.00 \rightarrow \alpha_d = 1.00$

$M_{ds} = M_{ss} = 11,480\text{kN.m}$

$M_s = f_y Z_{enc}$

$Z_{enc} = 48E+06$

$$M_s = \frac{280 \times 48 \times 10^6}{10^6} = 13,440\text{kN.m}$$

Refer to AS 5100.6, clause 6.3.4

$$M_b = 11,480\left(\frac{48}{41}\right) = 13,440\text{kN.m} \le M_s = 13,440\text{kN.m} - \text{OK}$$

$$f_b = \frac{13,440 \times 10^6}{48 \times 10^6} = 280\text{MPa} \le f_y$$

Therefore, in the steel section:

$f^*_{s,bf} = 254\text{MPa} \approx \phi f_b = 0.9 \times 280 = 252\text{MPa} - \text{OK}$

$f^*_{s,tf} = 219\text{MPa} < \phi f_b = 0.9 \times 280 = 252\text{MPa} - \text{OK}$

And in the reinforcement

$f^*_{s,reo} = 259\text{MPa} \leq \phi f_y = 0.9 \times 500 = 450\text{MPa} - \text{OK}$

Refer to section 7.4.1 in the main part of the guide.

F10.2 Maximum shear at support

The maximum shear in the girder at the intermediate support is 1903kN.

The thickness of the unstiffened web shall satisfy the requirement in *AS 5100.6*, clause 5.9.1.

There are no longitudinal stiffeners intended to be used.

Refer to AS 5100.6, clause 5.9.1.

$$t_w \geq \left(\frac{d_1}{180}\right)\sqrt{\frac{f_y}{250}}$$

$$t_w = 20\text{mm}$$

$$\left(\frac{d_p}{180}\right)\sqrt{\frac{f_y}{250}} = \left(\frac{990}{180}\right)\sqrt{\frac{300}{250}} = 6.0\text{mm}$$

$$t_w = 20\text{mm} \geq \left(\frac{d_p}{180}\right)\sqrt{\frac{f_y}{250}} = 6.0\text{mm} - \text{OK}$$

$$V_v = V_u = \begin{cases} V_w & \text{if} \quad \dfrac{d_p}{t_w} \leq \dfrac{82}{\sqrt{\dfrac{f_y}{250}}} \\ V_b & \text{if} \quad \dfrac{d_p}{t_w} > \dfrac{82}{\sqrt{\dfrac{f_y}{250}}} \end{cases}$$

$$\frac{d_p}{t_w} = \frac{990}{20} = 49.50$$

$$\frac{82}{\sqrt{\dfrac{f_y}{250}}} = \frac{82}{\sqrt{\dfrac{300}{250}}} = 74.86$$

$$\therefore V_u = V_w$$

$$V_w = 0.6 f_y A_w$$

$$V_u = V_w = \frac{0.6 \times 990 \times 20 \times 300}{1000} = 3564\text{kN}$$

$V^* = 1903\text{kN} \leq \phi V_u = 0.9 \times 3564 = 3208\text{kN} - \text{OK}$

F10.3 Combined bending moment and shear

Refer to section 7.4.2 in the main part of the guide.

When the maximum moment is assumed to be resisted by the whole of the cross section, the member will be designed for combined bending and shear, and satisfy the requirement in *AS 5100.6*, clause 5.11.3.

F10.3.1 Maximum shear with coexisting moment

$V^* = 1903\text{kN}$ $\qquad M^* = 10,840\text{kN.m}$

$\phi M_s = 0.9 \times 13,440 = 12,096\text{kN.m}$ $\qquad 0.75\phi M_s = 9072\text{kN.m}$

$M^* = 11,123\text{kN.m} \gg 0.75\phi M_s = 9072\text{kN.m}$

Refer to AS 5100.6, clause 5.11.3.

Therefore,

$V^* \leq \phi V_{vm}$

$$V_{vm} = V_v\left[2.2 - \frac{1.6M^*}{\phi M_s}\right] \qquad V_v = 3564\text{kN}$$

$$V_{vm} = 3564\left[2.2 - \frac{1.6 \times 10,840}{12,096}\right] = 2731\text{kN}$$

$\phi V_{vm} = 2458\text{kN}$

$V^* \leq \phi V_{vm} - \text{OK}$

Therefore, the shear capacity of the web is sufficient.

In order to increase the shear capacity, a transverse web stiffener is provided at 1967mm from the support (ie divide the length to the first bracing into three panels).

An intermediate web stiffener has an area that satisfies the requirement in *AS 5100.6*, clause 5.14.3.

$$A_s \geq 0.5\gamma A_w(1-\alpha_v)\left(\frac{V^*}{\phi V_u}\right)\left(\left(\frac{s}{d_p}\right) - \frac{\left(\frac{s}{d_p}\right)^2}{\sqrt{1+\left(\frac{s}{d_p}\right)^2}}\right)$$

$s = 1967\text{mm}$ $\qquad d_p = 990\text{mm}$ $\qquad \frac{s}{d_p} \approx 1.98$

$$\alpha_v = \left[\frac{82}{\left(\frac{d_p}{t_w}\right)\sqrt{\frac{f_y}{250}}}\right]^2\left[\frac{0.75}{\left(\frac{s}{d_p}\right)^2} + 1.0\right] \leq 1.0 \qquad \alpha_v = \left[\frac{82}{\left(\frac{990}{20}\right)\sqrt{\frac{300}{250}}}\right]^2\left[\frac{0.75}{\left(\frac{1967}{990}\right)^2} + 1.0\right] \leq 1.0$$

$\alpha_v = 1.0$

$A_s \geq 0.0\text{mm}^2$

Also, an intermediate web stiffener should satisfy the requirement in *AS 5100.6*, clause 5.14.5 for the minimum second moment of area about the centreline of the web.

$$\frac{s}{d_1} \approx 1.98 \geq \sqrt{2} = 1.41$$

$$I_s \geq \frac{1.5 d_1^3 t_w^3}{s^2}$$

$$I_s \geq \frac{1.5 \times 990^3 \times 20^3}{1967^2} = 3 \times 10^6 \text{mm}^4$$

Try a 250mm x 16mm stiffener plate at each side of the web.

$$I_s = 2 \times \left[\left(\frac{16 \times 250^3}{12} \right) + \left(16 \times 250 \times \left(\frac{250+16}{2} \right)^2 \right) \right] = 183.20 \times 10^6 \text{mm}^4 \geq 3 \times 10^6 \text{mm}^4 - \text{OK}$$

Refer to AS 5100.6, clause 5.14.5

Therefore, the shear capacity of the stiffened web is:

$$\phi V_v = \phi \left(R_{sb} + V_w \right)$$

The effective length of the web is taken as:

$$l_e = \min \left[\frac{17.5 t_w}{\sqrt{\frac{f_y}{250}}} ; \frac{s}{2} \right] = \min \left[\frac{17.5 \times 20}{\sqrt{\frac{300}{250}}} ; \frac{1967}{2} \right] = 320\text{mm}$$

The strut effective cross section, A_s, is taken as the stiffener area plus the effective length of the web.

$$A_s = 2\left(250 \times 16\right) + \left(320 \times 20\right) = 14400 \text{mm}^2$$

Therefore,

$$\phi R_{sb} = \phi \alpha_c A_s f_y \qquad \alpha_b = 0.5 \qquad k_f = 1.0$$

$$I_s = 183.20 \times 10^6 \text{mm}^4$$

$$r_s = \sqrt{\frac{I_s}{A_s}} = \sqrt{\frac{183.20 \times 10^6}{14400}} = 113\text{mm} \qquad \lambda_\eta = \left(\frac{L_e}{r_s} \right) \sqrt{k_f \left(\frac{f_y}{250} \right)} = \left(\frac{320}{113} \right) \sqrt{1.0 \times \left(\frac{300}{250} \right)} = 3.10$$

Refer to AS 5100.6, clause 5.13.2.

$$\alpha_a = \frac{2100\left(\lambda_\eta - 13.5\right)}{\lambda_\eta^2 - 15.3\lambda_\eta + 2050} = \frac{2100(3.10 - 13.5)}{3.10^2 - 15.3 \times 3.10 + 2050} = 10.85$$

$$\lambda = \lambda_\eta + \alpha_a \alpha_b = 3.10 - 0.5 \times 10.85 = -2.33$$

$$\eta = 0.00326\left(\lambda - 13.5\right) \geq 0$$

$$\eta = 0$$

$$\xi = \frac{\left(\frac{\lambda}{90}\right)^2 + 1 + \eta}{2\left(\frac{\lambda}{90}\right)^2} = \frac{\left(\frac{-2.33}{90}\right)^2 + 1}{2\left(\frac{-2.33}{90}\right)^2} = 746.5 \qquad \alpha_c = \xi \left[1 - \sqrt{1 - \left(\frac{90}{\xi \lambda} \right)^2} \right] = 746.5 \left[1 - \sqrt{1 - \left(\frac{90}{746.5 \times (-2.33)} \right)^2} \right] = 1.0$$

$$\phi R_{sb} = \frac{0.9 \times 14,400 \times 300}{1000} = 3888\text{kN}$$

$$\phi V_v = 3888 + 3208 = 7096\text{kN}$$

Please note that the web connection of intermediate transverse stiffeners should be designed to resist a design shear force per unit length (kN/mm) of not less than:

$$\upsilon_w \geq \frac{0.0008 t_w^2 f_y}{b_{es}}$$

$$b_{es} = 250 - \frac{16}{2} = 242\text{mm}$$

$$\upsilon_w \geq \frac{0.0008 \times 20^2 \times 300}{242} = 0.396\text{kN/mm}$$

F10.3.2 Maximum moment with coexisting shear

$V^* = 1780\text{kN}$ $\quad M^* = 11,574\text{kN.m}$

$\phi M_s = 0.9 \times 13,440 = 12,096\text{kN.m}$ $\quad 0.75\phi M_s = 9072\text{kN.m}$

Refer to AS 5100.6, clause 5.14.8

$M^* = 11407\text{kN.m} \gg 0.75\phi M_s = 9072\text{kN.m}$

Therefore,

$V^* \leq \phi V_{vm}$

$$V_{vm} = V_v \left[2.2 - \frac{1.6M^*}{\phi Ms} \right]$$

$$V_{vm} = 3564 \left[2.2 - \frac{1.6 \times 11574}{12096} \right] = 2384\text{kN}$$

$\phi V_{vm} = 2146\text{kN}$

$V^* \leq \phi V_{vm} - \text{OK}$

F10.4 Sagging bending moment

Refer to AS 5100.6, clause 5.11.3.

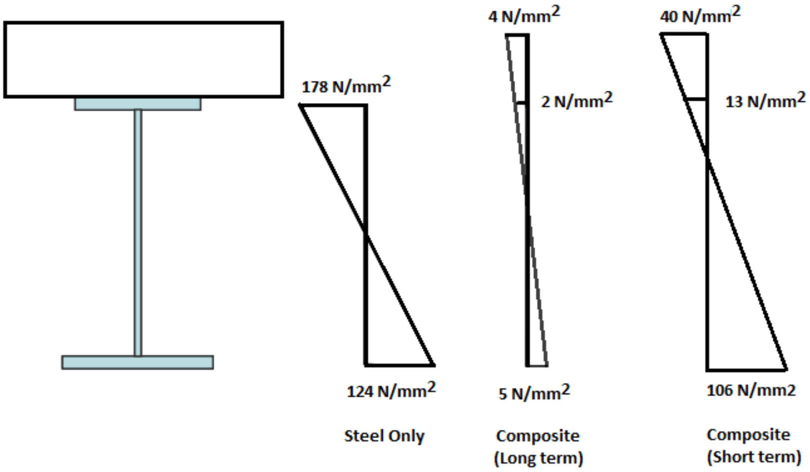
The PNA of the composite cross section in sagging is entirely within the concrete slab so, the plastic resistance can be utilised.

The short term plastic resistance of the composite section is 0.9 x 13,220=11,898kN.m and the total design value of bending effects is 7180kN.m, with a very small shear force, so the section is satisfactory on inspection.

Refer to section 7.4.2 of the main part of the guide.

It can be seen that the stresses calculated elastically, taking account of construction in stages, are also satisfactory as shown in figure F.16.

Figure F.16 Sagging bare steel and composite stresses



F10.5 Verification of crack control at SLS

F10.5.1 Minimum reinforcement spacing

The spacing of reinforcement crossing a potential crack and located next to the tension face should be smaller than the values given in *NZS 3101*, clause 2.4.4.4.

$f_s = 183\text{MPa}$ (max SLS stress at top rebars) $\quad c_c = 55 - \frac{25}{2} = 42.5\text{mm} \quad c_m = 55\text{mm}$

$$s = \min\left[\frac{90{,}000}{f_s} - 2.5c_c ; \frac{70{,}000}{f_s}\right] \qquad s = \min\left[\frac{90{,}000}{183} - 2.5 \times 42.5 ; \frac{70{,}000}{183}\right] = 382\text{mm}$$

$s_{provided} = 150\text{mm} \ll s_{required} = 382\text{mm} - \text{OK}$

F10.5.2 Crack control

Refer to section 7.5.4 in the main part of the guide.

$$\beta' = \frac{y - kd}{d - kd} = \frac{1350 - (1350 - 696)}{1225 - (1350 - 696)} = 1.22$$

$kd =$ depth of NA for long term composite section

$$g_s = \sqrt{\left(\frac{s}{2}\right)^2 + c_m^2} = \sqrt{\left(\frac{150}{2}\right)^2 + 55^2} = 93 \qquad w = 2.0\beta' \frac{f_s}{E_s} g_s = 2.0 \times 1.22 \times \frac{183}{200{,}000} \times 93 = 0.21\text{mm}$$

$w = 0.21\text{mm} < 0.3\text{mm} - \text{OK}$

F11 Longitudinal shear

Refer to sections 6.1.7 and 7.7 in the main part of the guide.

The resistance to longitudinal shear is verified for the web/flange weld, the shear connectors and the transverse reinforcement at the pier, at the splice and at mid-span.

F11.1 Shear forces

Table F.33 ULS values

	Pier	Splice	Span	Abutment
Shear on steel section (stage 1)	685	478	-73	-488
Shear on long term composite section	341	377	76	-71
Shear on short term composite section (worst effects)	877	583	-321	-670
	1903	1438	318	-1229

Table F.34 SLS values

	Pier	Splice	Span	Abutment
Shear on steel section (stage 1)	523	365	-57	-373
Shear on long term composite section	252	282	59	-50
Shear on short term composite section (worst effects)	525	348	-190	-401
	1300	995	-188	-824

F11.2 Section properties

To determine shear flows the parameter $A_t y_c / I_t$ is needed for each section and stage.

For composite sections, uncracked unreinforced composite section properties can be used to determine shear flow.

Table F.35 $A_t y_c / I_t$ for each section

	Pier girder		Span girder/abutment girder	
	Web/top fl	Top fl/slab	Web/top fl	Top fl/slab
	$A_t y_c / I_t$	$A_t y_c / I_t$	$A_t y_c / I_t$	$A_t y_c / I_t$
	m^{-1}	m^{-1}	m^{-1}	m^{-1}
Steel section	0.845	-	0.689	-
Long-term section	0.882	0.557	0.854	0.698
Short-term section	0.873	0.761	0.876	0.837

F11.3 Shear flow at ULS

Table F.36 Force at web/top flange junction

At pier	685 x 0.845 + 341 x 0.882 + 877 x 0.873 =	1645	kN/m
At splice	478 x 0.689 + 377 x 0.854 + 583 x 0.876 =	1162	kN/m
At mid span	-73 x 0.689 + 76 x 0.854 – 321 x 0.876 =	-267	kN/m
At abutment	-488 x 0.689 -71 x 0.854 – 670 x 0.876 =	-984	kN/m

Table F.37 Force at flange/slab junction

At pier	341 x 0.557 + 877 x 0.761 =	857	kN/m
At splice	377 x 0.698 + 583 x 0.837 =	751	kN/m
At mid span	76 x 0.698 – 321 x 0.837 =	-216	kN/m
At abutment	-71 x 0.698 – 670 x 0.837 =	-610	kN/m

F11.4 Shear flow at SLS

Table F.38 Force at flange/slab junction

At pier	252 x 0.557 + 525 x 0.761 =	540	kN/m
At splice	282 x 0.698 + 348 x 0.837 =	488	kN/m
At mid span	59 x 0.698 – 192 x 0.837 =	-120	kN/m
At abutment	-50 x 0.698 – 401 x 0.837 =	-371	kN/m

The shear flow at SLS is required for verification of the shear connectors.

F11.5 Web/flange weld

Design weld resistance is given in tables 9.8 and 9.9 of ASI (1999) design capacity tables.

For 6mm throat SP fillet weld t_w=8mm

$$v_{w_{6mm}} = 1.11\text{kN/mm}$$

Resistance of two welds = 2220kN/m >1645kN/m, shear flow in pier girder at top flange – OK

By inspection, 5mm throat SP fillet weld would be satisfactory at the splices and in the span regions.

Shear flows at bottom flange are slightly less and are OK by inspection.

F11.6 Shear connectors

Refer to section 7.7.3 in the main part of the guide

Shear stud connectors 19mm diameter, 150mm long are assumed, with f_u=410MPa.

The resistance of a single stud is given by AS 5100.6, clause 6.6.4.4 as the lesser of:

$$f_{ks} = 0.63 d_{bs}^2 f_{uc} \qquad \text{or} \qquad f_{ks} = 0.63 d_{bs}^2 \sqrt{f'_{cy} E_c}$$

$$f'_{cy} = 40\text{MPa} \qquad f_{uc} = 410\text{MPa} \qquad d_{bs} = 16\text{mm}$$

$$E_c = 5050\sqrt{f'_{cy}} = 5050\sqrt{40} = 32,000\text{MPa}$$

Note: for the purpose of evaluating the resistance of headed stud connectors embedded in solid slabs, the modulus of elasticity for concrete may be taken to be $E_c = 5050\sqrt{f'_{cy}}$. From structural reliability analyses conducted by HERA, it has been shown that the use of this value produces predictions that better reflect the resistance of studs in physical tests.

Therefore the design resistance of a single headed shear connector is:

Refer to AS 5100.6, clause 6.6.3.2.

$$f_{ks} = \min\left[0.63 \times 19^2 \times 410; 0.63 \times 19^2 \sqrt{40 \times 32,000}\right] = 93.25\text{kN}$$

Refer to AS 5100.6, clause 4.9.2.2

Longitudinal shear forces due to the primary effects of shrinkage or differential temperature are assumed to be transmitted across the interface between the steel beam and the concrete slab by shear connectors at each end of the beam, ignoring the effects of bond.

In the absence of a more accurate analysis, the forces on the connectors may be calculated by assuming that the rate of transfer of longitudinal force varies linearly from a maximum at the end of the beam to zero at a distance from the end equal to the total effective width of the slab. Alternatively, where stud shear connectors are used, the rate of transfer of force may be assumed to be constant over a distance from each end of the beam equal to one fifth of the span of the beam.

In this exercise, it is assumed that longitudinal shear flow is constant over the span of the beam.

$$\upsilon_L^* \le \phi \upsilon_{Ls}$$

$$\upsilon_{Ls} = 0.55 n f_{ks}$$

$$\phi = 1.0$$

$$u_L^* = 540\text{kN/m} + \left(\frac{1943 \times 5}{28 \times 2}\text{kN/m}\right)_{\text{Shrinkage PE}} = 713\text{kN/m}$$ (load combination 1A is assumed to be critical)

The shear connectors between the point of maximum negative bending moment and the adjacent point of zero moment, at the SLS, shall satisfy the requirement in *AS 5100.6*, clause 6.6.3.3:

For simplicity, it has been conservatively assumed that the negative moment envelop is within the cracked section regions.

$$F_h^* = 0.55 A_{rs} f_{sy}$$

$$A_{rs} = 2 \times 3272 = 6544\text{mm}^2/\text{m} \qquad f_{sy} = 500\text{MPa}$$

$$F_h^* = 0.55 \times 6544 \times 2.8 \times 500 \times 10^{-3} = 5039\text{kN} \qquad f_h^* = 5039/(2 \times 0.15 \times 28) = 600\text{kN/m}$$

If studs are grouped and spaced at 150mm spacing along the beam (to suit transverse reinforcement), then a row of three studs has a design resistance of:

$$\phi u_{Ls} = \frac{0.55 \times 93.25 \times 3}{0.15} = 1026\text{kN/m}$$

$$u_L^* = 713\text{kN/m} \le \phi u_L = 1026\text{kN/m - OK}$$

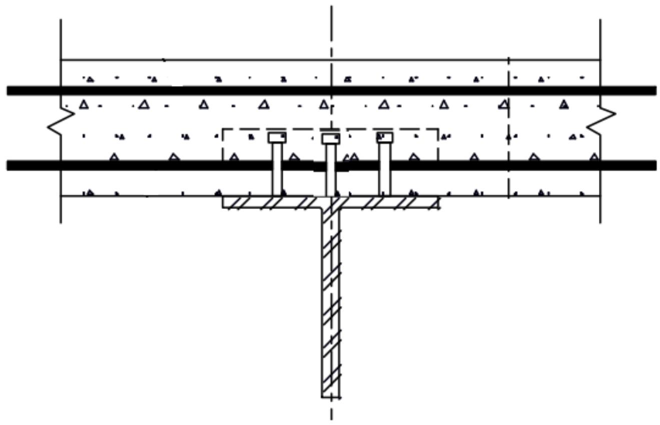
Shear connectors have been designed for critical location; however, further economies can be realised through adjusting the spacing to suit demand along the length of the girder.

F11.7 Transverse reinforcement

Refer to section 7.7.7 in the main part of the guide.

Consider the transverse reinforcement required to transfer the full shear resistance of the studs, ie 1026kN/m as well as the maximum ULS shear flow, ie 1028kN/m.

Figure F.17 Transverse reinforcement critical shear planes



For a critical shear plane around the studs, shown dotted above, the shear resistance is provided by twice the area of the bottom bars. The design shear resistance of the transverse reinforcement is given by *AS 5100.6*, clause 6.6.5.2 as follows:

$$u^*_{Lp} \le \phi\left(0.9us + 0.7A_{ts}f_{ry}\right) \quad \text{and} \quad u^*_{Lp} \le \phi\left(0.15uf'_c\right)$$

Refer to AS 5100.6, clause 6.6.5.2

Assumed HD20-150mm:

$$s = 1\text{MPa}$$
$$u = 2\times150 + 400 = 700\text{mm}$$
$$A_{st} = \left(2\times314\right)/150 = 4188\text{mm}^2/\text{m}$$
$$f_{ry} \le 450\text{MPa}$$
$$\phi u_L = \min\left[1.0\left(0.9\times700\times1.0 + \frac{0.7\times4188\times450}{10^3}\right);1.0\left(0.15\times700\times40\right)\right] = 1949\text{kN/m}$$
$$u^*_{Lp} = 857\text{kN/m} + \left(\frac{1.35\times1943\times5}{28\times2}\right)_{\text{Shrinkage PE}} = 1091\text{kN/m}$$
$$u^*_{Lp} = 1091\text{kN/m} \le \phi u_L = 1949\text{kN/m - OK}$$

The transverse bars are adequate.

F12 Main girder splices

Refer to section 7.8.4 in the main part of the guide.

F12.1 Forces and moments at splice position

Worst hogging moment at splice, at ULS and SLS

Worst shear at splice, at ULS and SLS

The worst sagging moment is much less than maximum hogging moment.

The in-service design combination of actions considered is Group 1A loading:

$$U_{1A} = 1.35\left(DL + 1.67LL\times I + SG + 1.30FP\right)$$
$$S_{1A} = DL + 1.35LL\times I + SG + FP$$

Table F.39 Main girder stresses and shears at splice location

	ULS hog	SLS hog	ULS shear	SLS shear	
Top flange stress	88	62	76	55	MPa
Bottom flange stress	-86	-59	-40	-32	MPa
Shear force	943	700	1439	995	kN

From the above stresses, the forces in each flange and moment in the web are as follows:

Table F.40 Main girder forces and moments at splice location

	ULS hog	SLS hog	ULS shear	SLS shear	
Top flange force	2640	1860	2280	1650	kN
Bottom flange force	-3612	-2478	-1680	-1344	kN
Web force	-36	-41	-367	-236	kN
Web moment	256	179	235	163	kN.m

It is noted that the compressive stress in the top flange is higher at construction stage 1, under wet concrete load in span 1. At this stage, the splice must provide continuity of stiffness, without slipping, and because the beams are slender at that stage it is appropriate to amplify the design force to ensure adequate continuity of resistance.

The maximum ULS stress in the top flange during construction is 32MPa. Thus the ULS design force for the top flange is 960kN.

The maximum ULS stress in the bottom flange during construction is 25MPa. Thus the ULS design force for the bottom flange is 1050kN.

The maximum SLS stress in the top flange during construction is 25MPa. Thus the SLS design force for the top flange is 750kN.

The maximum SLS stress in the bottom flange during construction is 19MPa. Thus the SLS design force for the bottom flange is 798kN.

F12.2 Slip resistance of bolts

Use M24 grade 8.8 tension-friction bolts in double shear in normal clearance holes.

$d = 24mm$ $\qquad d_h = 26mm$

$f_{uf} = 830MPa$

$A_s = 353mm^2$

$A_c = 324mm^2$

$\mu = 0.35$ assumed clean, as rolled surface

$k_h = 1.0$

$N_{ti} = 210kN$

SLS slip resistance in double shear:

Refer to AS 5100.6, clause 12.5.4.1

$V_{sf}^* \leq \phi V_{sf}$ $\qquad \phi = 0.7$

$V_{sf} = \mu n_{ei} N_{ti} K_h$ $\quad V_{sf} = 0.35 \times 2 \times 210 \times 1.0 = 147\text{kN}$

$\phi V_{sf} = 103\text{kN}$

F12.3 Shear resistance of bolts

Refer to AS 5100.6, clause 12.5.3.1

ULS shear resistance of bolts (assuming shear through threads)

$V_f^* \leq \phi V_f$ $\quad \phi = 0.8$

$V_f = 0.62 f_{uf} k_r (n_n A_c + n_x A_o)$

$k_r = 1.0$

$n_n = 2.0$

$V_f = 0.62 \times 830 \times 1.0 (2 \times 324 + 0) = 333\text{kN}$

$\phi V_f = 266\text{kN}$

F12.4 Bolt spacing and edge distances

Refer to AS 5100.6, clause 12.5.2.2 and AS 5100.6, table 12.5.2.

End and edge distance: $1.75 d_f = 42\text{mm}$ for sheared or hand flame cut edge

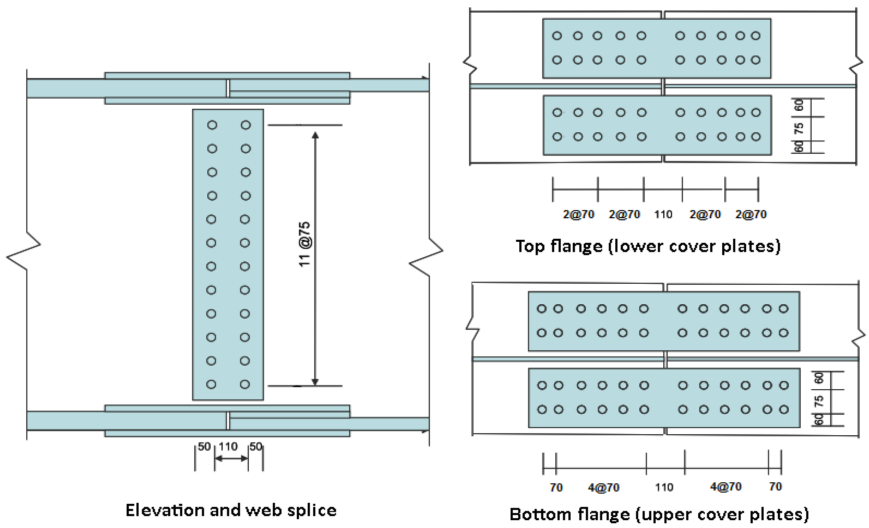
Spacing: $2.5 d_f = 60\text{mm}$

Note: spacing of 70mm is utilised to reflect New Zealand industry standard gauge lines.

F12.5 Splice configuration

Consider the following splice configuration

Figure F.19 Splice configuration



F12.5.1 Top flange splice

Dimension for lower covers

Bolt spacing:

Refer to AS 5100.6, clause 6.6.2.

$e_1 = 50mm \qquad e_2 = 60mm$

$p_1 = 70mm \qquad p_2 = 75mm$

Overall dimension = 600 x 195 mm

Thickness = 10mm

The length of the cover plate is sufficiently short such that shear connectors do not need to be welded to the cover plate. The maximum permitted longitudinal spacing is 600mm

Note: The tensile capacity of the cover plate also needs to be checked.

F12.5.2 Bottom flange splice

Dimension for upper covers

Bolt spacing:

$e_1 = 50mm \qquad e_2 = 60mm$

$p_1 = 70mm \qquad p_2 = 75mm$

Overall dimension = 860 x 195 mm

Thickness = 20mm

F12.5.3 Web splice

Refer to AS 5100.6, clause 6.6.2(g)

Bolt spacing:

$e_1 = 50mm \qquad e_2 = 50mm$

$p_2 = 75mm$

Overall dimension = 210 x 925mm

Thickness = 10mm

F12.6 Verification of connection resistances

F12.6.1 Top flange splice

There are four rows of bolts, with four bolts per row across the flange.

Slip resistance at SLS = 20 x 103 = 2060kN >1860kN – OK

Resistance at ULS = 20 x 266 = 5320kN >2640kN – OK

F12.6.2 Bottom flange splice

There are six rows of bolts, with four bolts per row across the flange

Slip resistance at SLS = 24 x 103 = 2472kN ≅ 2478kN – OK

Resistance at ULS = 24 x 266 = 6384kN >3612kN – OK

F12.6.3 Web splice

The splice has a single column of 12 bolts at 75mm spacing.

For this group the modulus for the outer bolts = $\sum r_i^2 / r_{max}$

Where r_i is the distance of each bolt from the centre of the group and r_{max} is the distance of the furthest bolt from the centre of the group.

Here, the modulus = 1950mm.

The extra moment due to the shear = shear force x eccentricity of group from the centreline of the splice.

Hence the force on the outer bolts at ULS and SLS are as shown in table F.41.

Table F.41 Web splice design actions

	ULS hog	SLS hog	ULS shear	SLS shear		
Shear V	943	700	1439	995	kN	
Longitudinal force F_L	-36	-41	-367	-236	kN	
Moment	256	179	235	163	kN.m	
Moment due to e = 55mm	52	39	79	55	kN.m	
Total moment M	308	218	314	218	kN.m	
Force per bolt due to M	158	112	161	112	kN	(= M / 1950)
Force per bolt due to F_L	3	4	31	20	kN	(= F_L / 12)
Total horizontal force H	161	116	192	132		
Vertical force due to V	79	58	120	83	kN	(= V / 12)
Resultant force	179	130	226	156	kN	(Vector sum)

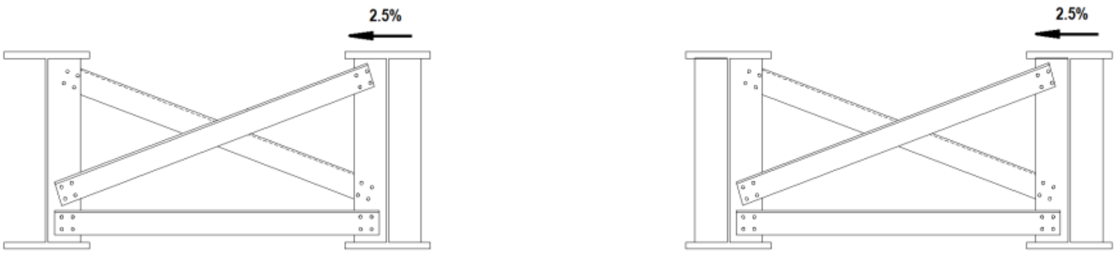
By inspection the web splice is satisfactory.

F13 Bracing

Refer to sections 7.3.5, 7.3.6 and 7.3.7 in the main part of the guide.

The configuration of the intermediate bracing system between girder pairs is as shown in figure F.20.

Figure F.20 Bracing system between girder pairs



Assume the use of 125 x 125 x 10 equal angle sections.

Here the bracing is only used to provide stability under the construction load.

$$M^*_{stage1} = 3211 kN.m$$

$$Z_{top\_flange} = 18 \times 10^6 mm^3$$

$$Z_{bot\_flange} = 26 \times 10^6 mm^3$$

Stress at the top/bottom flange is:

$$\sigma_{top\_flange} = 178 MPa$$

$$\sigma_{bot\_flange} = 124 MPa$$

Therefore the maximum flange force is:

Refer to AS 5100.6, clause 8.4.2

$$F_f = \frac{178 \times 500 \times 25}{1000} = 2225\text{kN}$$

ASI design capacity tables

$$2.5\%F_f \approx 56\text{kN}$$

$$fN_c = 129\text{kN}$$

The buckling resistance of the 3900mm diagonal is easily adequate for this force and two-bolt end connection will also be adequate.

F14 Fatigue assessment

Refer to sections 6.4.2 and 7.11.1 in the main part of the guide.

F14.1 Basic loadings for fatigue design

Clause 6.9 of *AS 5100.2* gives two basic loads as shown in figures F.21 and F.22.

Figure F.21 Modified individual A160 heavy axle load

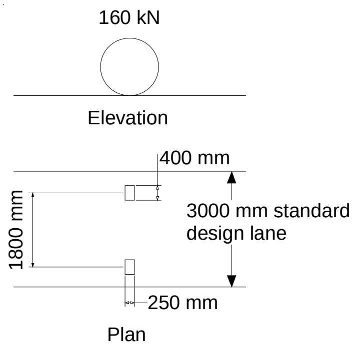
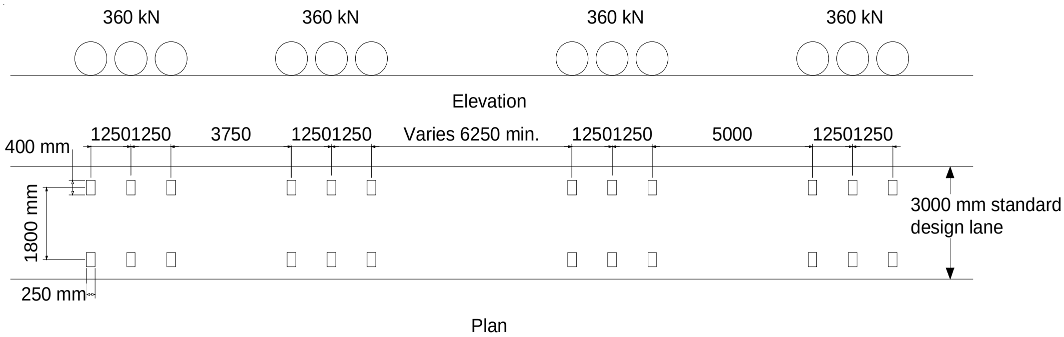


Figure F.22 Modified individual M1600 moving traffic load without the UDL component



The number of fatigue stress cycles to be used for the calculation of the fatigue capacity of the structural element under consideration should be as follows:

- For the fatigue design load of 0.70 x (A160 axle load) x (1 + α):
  - (current number of heavy vehicles per lane per day) x 4 x 10^4 x (route factor).
- For the fatigue design load of 0.70 x (M1600 moving traffic load without UDL) x (1 + α):
  - (current number of heavy vehicles per lane per day) x 2 x 10^4 ($L^{-0.5}$) x (route factor).

Dynamic load allowance α is given as following:

- $\alpha = 0.4$ for the A160 axle load

- $\alpha = 0.3$ for the M1600 axle load.

The route factor for the urban road specified as 0.3.

L is the effective span in metres and is defined as:

- For positive bending moments, L is the actual span in which the bending moment is being considered.
- For negative moment over interior supports, L is the average of the adjacent spans.
- For end shear, L is the actual span.
- For reactions, L is the sum of the adjacent spans.
- For cross-girders, L is twice the longitudinal spacing of the cross-girders.

F14.2 Range of effects due to passage of fatigue vehicle

By inspection it is clear that the fatigue loading of M1600 is more critical, therefore the fatigue loading is:

$0.70 \times (\text{M1600 without UDL}) \times (1 + \alpha)$

The corresponding total number of cycles is:

$$n = NHVD_{per\ lanex} \times 2 \times 10^4 \left(L^{-0.5}\right)\left(\text{route factor}\right)$$

$NHVD = 1500$ route factor $= 0.3$

Table F.42 Worst bending effects

		Pier	Splice	Span
		M_x (kN.m)	M_x (kN.m)	M_x (kN.m)
Lane 1	Positive	0	403	821
	Negative	-1035	-442	-190
	Range	1035	845	1011
Lane 2	Positive	0	361	822
	Negative	-905	-392	-162
	Range	905	753	984

Table F.43 Worst shear effects

		Pier	Splice	Span
		F_y (kN)	F_y (kN)	F_y (kN)
Lane 1	Positive	234	144	63
	Negative	0	26	-75
	Range	234	118	138
Lane 2	Positive	194	135	58
	Negative	0.8	24	-53
	Range	193	111	111

F14.3 Assessment of structural steel details

Design stress range at pier:

For the intermediate support the length of the effective span is $L = 28.0\text{m}$.

Therefore, the total number of cycles is:

$n = 1500 \times 2 \times 10^4 \times 28^{-0.5} \times 0.3 = 1.7 \times 10^6$

At the pier, the stress range f* in top and bottom flanges (at their mid thickness) is:

Top flange: $f^* = \dfrac{1035}{60} = 17\text{MPa}$

Bottom flange: $f^* = \dfrac{1035}{48} = 22\text{MPa}$

As $f^* = 22\text{MPa} \leq 27\text{MPa}$, fatigue assessment is not required; however, the detailed calculation is included for reference.

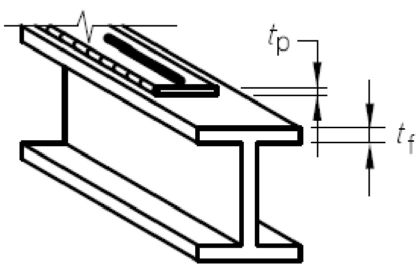
The worst detail category that might apply is for a bearing plate welded to the underside of the bottom flange, which, for a flange plate over 25mm thick, is category 36 (*AS 5100.6*, table 13.5.1-B).

Design value of fatigue strength ϕf_{fc} is:

Refer to AS 5100.6, clause 13.1.7.

$f_{rn} = 36\text{MPa}$

$$B_{tf} = \left(\frac{25}{t_f}\right)^{0.25} = \left(\frac{25}{60}\right)^{0.25} = 0.80$$



$f_{rnc} = B_{tf} f_{rn} = 0.80 \times 36 = 29\text{MPa}$

$n = 1.7 \times 10^6 \pounds 5 \times 10^6$

$\phi = 1.0$

$$\phi f_c = \phi f_{rnc} \left(\frac{2 \times 10^6}{n}\right)^{1/3} = 29 \left(\frac{2}{1.7}\right)^{1/3} = 31\text{MPa}$$

$f_{rn} \leq \phi f_{fc}$ – OK

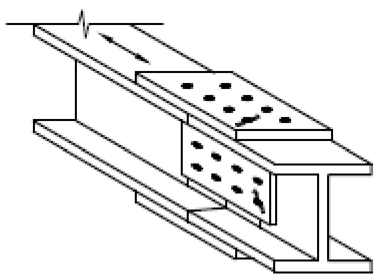
Refer to AS 5100.6, clause 13.7.2

F14.4 Design stress ranges at splice position

At the splice position, there is negligible stress range in the top flange. The stress range in the bottom flange is $f^* = \dfrac{845}{36} = 24\text{MPa}$. Again as the design stress range is less than 27MPa, the fatigue assessment is not required. However, for information, the detailed calculation is included as follows:

The most onerous detail at a bolted splice would be category 140. Thus the fatigue strength is:

$$\phi f_{fc} = \phi f_{rnc} \left(\frac{2 \times 10^6}{n}\right)^{1/3}$$



$n = 1500 \times 2 \times 10^4 \times 28^{-0.5} \times 0.3 = 1.7 \times 10^6$

$n = 1.7 \times 10^6 \leq 5 \times 10^6$

$f_{rn} = 140\text{MPa}$

$$\beta_{tf} = \left(\frac{25}{t_f}\right)^{0.25} = \left(\frac{25}{50}\right)^{0.25} = 0.84$$

$$f_{rnc} = \beta_{tf} f_{rn} = 0.84 \times 140 = 117.6\text{MPa}$$

$$\phi = 1.0$$

$$\phi f_{fc} = \phi f_{rnc}\left(\frac{2\times10^6}{n}\right)^{1/3} = 117.6\left(\frac{2}{1.7}\right)^{1/3} = 177\text{MPa} \qquad f^*_n \le \phi f_{fc}\text{ - OK}$$

Refer to section 7.7.6 in the main part of the guide.

F14.5 Design stress ranges at mid span

At mid-span there is negligible stress range in the top flange. The stress range in the bottom flange is $f^* = \frac{1011}{36} = 28\text{MPa}$.

The most onerous detail would be a transverse web stiffener, for which the fatigue category is 71. Thus the fatigue strength is:

$$\phi f_{fc} = \phi f_{rnc}\left(\frac{2\times10^6}{n}\right)^{1/3}$$

$$n = 1500\times2\times10^4\times28^{-0.5}\times0.3 = 1.7\times10^6$$

$$n = 1.7\times10^6 \le 5\times10^6$$

$$f_{rn} = 71\text{MPa}$$

$$\beta_{tf} = \left(\frac{25}{t_f}\right)^{0.25} = \left(\frac{25}{50}\right)^{0.25} = 0.84$$

$$f_{rnc} = \beta_{tf} f_{rn} = 0.84 \times 71 = 60\text{MPa}$$

$$\phi = 1.0$$

$$\phi f_{fc} = \phi f_{rnc}\left(\frac{2\times10^6}{n}\right)^{1/3} = 60\left(\frac{2}{1.7}\right)^{1/3} = 63\text{MPa}$$

$$f_{rn} \le \phi f_{fc} - \text{OK}$$

F14.6 Assessment of shear connection

The design value of the stress range in shear studs is 425MPa times the ratio of the longitudinal shear load on the stud to the nominal static strength specified in AS 5100.6, clause 6.6.3.

F14.6.1 Shear at pier

The range of vertical shear force at the pier is 234kN.

At the pier, the studs are 19mm diameter, in rows of three at 150mm spacing.

$$\upsilon_L^* = 234\times0.761 = 178\text{kN/m}$$

$$\left(\frac{A_t y_c}{I_t}\right)_{\text{pier girder, short-term}} = 0.761\text{m}^{-1}$$

$$\phi \upsilon_{Ls} = 1026 kN/m$$

$$\phi = 1.0$$

Therefore,

$$f_s^* = \frac{\upsilon_L^*}{\upsilon_{Ls}} \times 425 = \frac{\frac{178 \times 0.15}{3}}{1026} \times 425 = 4MPa$$

The fatigue strength of the shear stud is given in *AS 5100.6*, clause 13.6.3 as follows:

$$\phi f_f = \phi \left(\frac{2.08 \times 10^{22}}{n} \right)^{1/8} = \left(\frac{2.08 \times 10^{22}}{1.7 \times 10^6} \right)^{1/8} = 103MPa$$

$f_s^* \leq \phi f_f$ – OK

F14.6.2 Shear at splice

As the shear force at splice position is lower than that calculated above, it is apparent that the fatigue is not a matter of concern in this position.

F14.6.3 Shear at mid-span

As the shear force at mid-span is lower than that calculated above, it is apparent that the fatigue is not a matter of concern in this position.

Appendix G: Single span multi-girder bridge

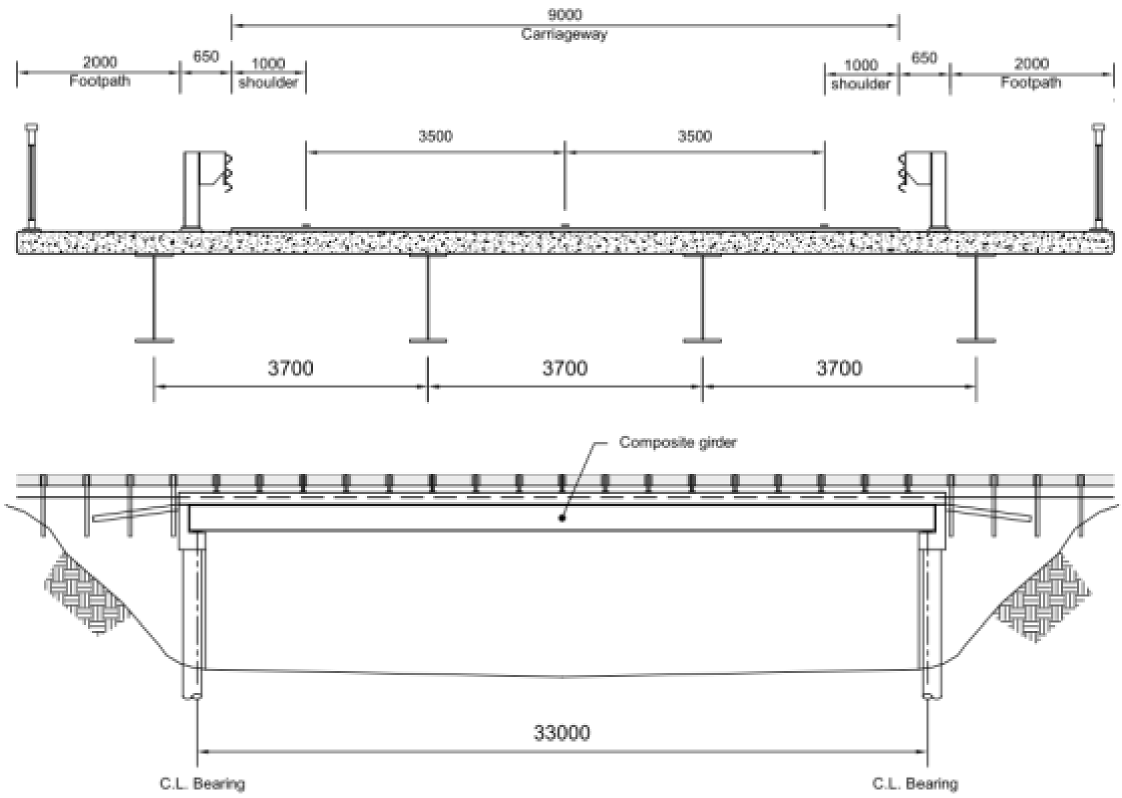
G1 Design statement

- The bridge carries a two-lane (each 3.5m wide) single carriageway road over a flood plain.
- The carriageway has two 1m wide shoulders and has 2m wide footways on either side.
- Speed environment is signed as a maximum of 50km/h but a design threshold of 60km/h is used.
- For barrier design the AADT is 15,000 vehicles per day with 20% commercial vehicles. An offset of 1.2m is assumed. Thus, a G9 modified TL4 Thrie beam is considered.
- The bridge clearance is minimum 1.5m from the soffit to design flood level, therefore flood load does not need to be considered.
- The bridge is being designed in accordance with following standards:
  - NZTA (2012) *Bridge manual*
  - *AS 5100.6* (SA 2004)
  - *NZS 3101* (SNZ 2006).
- Structural material properties:
  - structural steel: grade 300
  - concrete: C40
  - steel reinforcement: grade 500.
- Environmental exposure classification for superstructure = A2 – inland, exterior.
- Density of steel = 78.5kN/m^3.
- Density of reinforced concrete = 25kN/m^3.
- Nominal thickness of surfacing = 130mm.
- Density of surfacing = 23kN/m^3.
- Steel parapet selfweight = 2kN/m for both TL4 and pedestrian barriers.
- Construction load of 0.75kN/m^2 and the weight of the temporary formwork is assumed to be 0.5kN/m^2. Additionally, wet concrete is assumed to have a density of 1kN/m^3 greater than that of hardened concrete.
- Pedestrian live load = 5kPa
- This value can be reduced when considered in conjunction with road traffic loading.
- *Draft fatigue design criterion for bridges* (Clifton 2007a) is recommended for fatigue assessment.
- Coefficient of linear thermal expansion = 11.7×10^{-6} per °C.
- Characteristic value of shrinkage is $\varepsilon_{sh} = 0.0002$

G2 Structural arrangement

The bridge carries a two-lane single carriageway road over a flood plain. The carriageway has 1m wide shoulders and has a 2m wide footway on either side. A four-girder arrangement has been chosen and a deck slab of 250mm has been assumed. The deck cantilevers 1.6m outside the centrelines of the outer girders. The bridge is assumed to have 33m span with pinned end supports.

Figure G.1 Single span multi-girder bridge structural arrangement



G3 Design basis

The bridge is to be designed in accordance with the *Bridge manual*, *AS 5100.6* and *NZS 3101*.

The fatigue limit state is verified for the reference stress range due to the load application based on Clifton (2007a).

Crack widths in the deck slab are verified at the serviceability limit state (SLS) based on *NZS 3101*.

G3.1 Load combinations

Factors and load combinations of actions are given in the *Bridge manual,* tables 3.1 and 3.2 for SLS and ultimate limit state (ULS) respectively.

G3.2 Factors on strength

The values of various capacity reduction factors (ϕ) for strength limit states are given by *AS 5100.6*, table 3.2 for steel, and *NZS 3101*, clause 2.3.2.2 for concrete.

G3.3 Structural material properties

It is assumed that the following structural material grades will be used:

Structural steel:	grade 300
Concrete:	C40
Steel reinforcement:	grade 500

For structural steel, the value of f_y depends on the product material.

For rolled sections use:

ASI design capacity tables

320MPa for t ≤11mm; 300MPa for 11mm < t ≤17mm; and 280MPa for t >17mm.

For plates use:

320MPa for t ≤8mm; 310MPa for 8mm <t ≤12mm; 300MPa for 12mm <t ≤20mm; and 280MPa for t >20mm.

Note that designers may wish to use grade 350 steel for better economy.

For concrete, $f_c^{'} = 40\text{MPa}$

For steel reinforcement, $f_y = 500\text{MPa}$

The modulus of elasticity for both structural steel and steel reinforcement is taken as $E_S = 200\text{GPa}$

The modulus of elasticity of the concrete is given by *NZS 3101* as:

Refer to NZS 3101 clause 5.2.3

$$E_C = \left(3320\sqrt{f_c^{'}} + 6900\right)\left(\frac{r}{2300}\right)^{1.5}$$
$$E_C = 32\text{GPa}$$

This 28-day value will be used for determination of all short-term effects and resistance and the modular ratio is thus:

Refer to AS 5100.6, clause 4.4.2

$$n_S = 200/32 = 6.25$$

For long-term effects, the modular ratio is:

$$E_{c,L} = \frac{E_C}{1+\phi_{cc}} = \frac{32}{1+2} = 10.50$$
$$n_L = 200/10.50 = 19$$

The conservative value of ϕ_{cc} is given in RRU 70 as 2.00

National Roads Board (1984, section 2)

G3.4 Durability requirements

G3.4.1 Concrete

Environmental exposure classification

Superstructure: A2 – inland, exterior

Minimum cover to reinforcement

Surfaces in contact with the ground = 75mm

Surfaces with a damp proof membrane between the ground and the concrete = 50mm

Elsewhere = 40mm

G3.4.2 Steel

For appropriate and cost-effective coating system for structural steelwork, the *New Zealand steelwork corrosion and coatings guide* (El Sarraf and Clifton 2011), is used in conjunction with *AS/NZS 2312* (SNZ 2002) and *NZS 3404.1* (SNZ 2009).

G4 Action on the bridge

G4.1 Permanent actions

G4.1.1 Self-weight of structural elements

The density of steel is taken as 78.5kN/m^3 and the density of reinforced concrete as 25kN/m^3. Self-weights are based on nominal dimensions.

G4.1.2 Self-weight of surfacing

The nominal thickness of the surfacing is 130mm. Assume that the density is 23kN/m^3. The self-weight generally produces adverse effects and so it is based on nominal thickness + 55%. This follows international practice; however, in New Zealand a minimum of 1.5kPa for surfacing plus 0.25kPa allowance for services is used. Reference should be made to the *Bridge manual,* section 3.4.2.

$$\mathrm{DL} = 1.55 \times 0.13 \times 23 = 4.63\mathrm{kPa}$$

G4.1.3 Self-weight of footway construction

The nominal thickness of the footway comprising a concrete slab is 250mm and a uniform density of 25kN/m3 is assumed. The self-weight is based on the nominal dimensions.

G4.1.4 Self-weight of parapets

A nominal value of 2kN/m is assumed for each steel parapet.

Note that this value will be increased if a solid concrete crash barrier is used.

G4.2 Construction loads

For global analysis, a uniform construction load of 0.75kN/m^2 is assumed during casting and the weight of the temporary formwork is assumed to be 0.5kN/m^2. Additionally, wet concrete is assumed to have a density of 1kN/m^3 greater than that of hardened concrete; for a slab thickness of 250mm this adds 0.25kN/m^2.

Refer to section 6.1.5 in the main part of the guide

The total construction load is thus: $\mathrm{CN} = 0.75 + 0.50 + 0.25 = 1.50\mathrm{kPa}$

G4.3 Traffic loads

G4.3.1 Road traffic

For the road carried by this bridge, the *Bridge manual* specifies the HN-HO-72 traffic loading but only HN72 has been considered here for simplicity.

Refer to section 6.4.1 in the main part of the guide

G4.3.2 Pedestrian traffic

Pedestrian traffic is represented by the value in the *Bridge manual*.

$$\mathrm{FP} = 5\mathrm{kPa}$$

This value can be reduced ti 4.0kPa when considered in conjunction with road traffic loading.

G4.3.3 Fatigue load

Clifton (2007a) is recommended for fatigue assessment.

Refer to section 6.4.2 in the main part of the guide

G4.4 Thermal actions

G4.4.1 Overall temperature change

For a change of length in composite sections, the coefficient of linear thermal expansion is 11.7×10^{-6} per °C. This is then used for determining soil pressure on integral bridges or determining the expansion length in the case of simply supported abutments.

G4.4.2 Differential temperature change

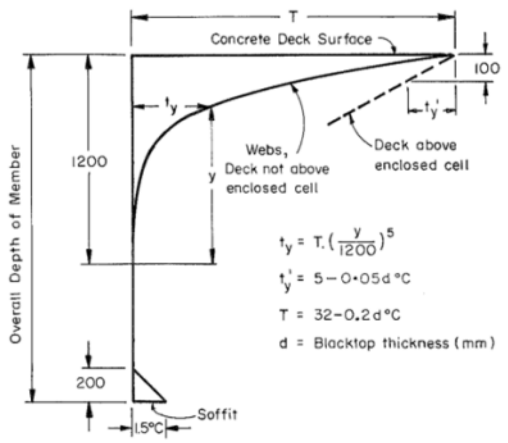
The vertical temperature difference given in the *Bridge manual* is used and the temperature difference will be considered to act simultaneously with the overall temperature change.

Refer to *Bridge manual*, section 3.4.6

The effects of vertical temperature gradients shall be derived for both positive differential temperature conditions (where the top surface is hotter than the average temperature of the superstructure) and negative temperature differential conditions (where the top surface is colder than the average temperature of the superstructure).

Note: The negative temperature variation is the same as that for bridge type 1 in figure 17.3 of *AS 5100.2*.

Figure G.2 Temperature variation with depth

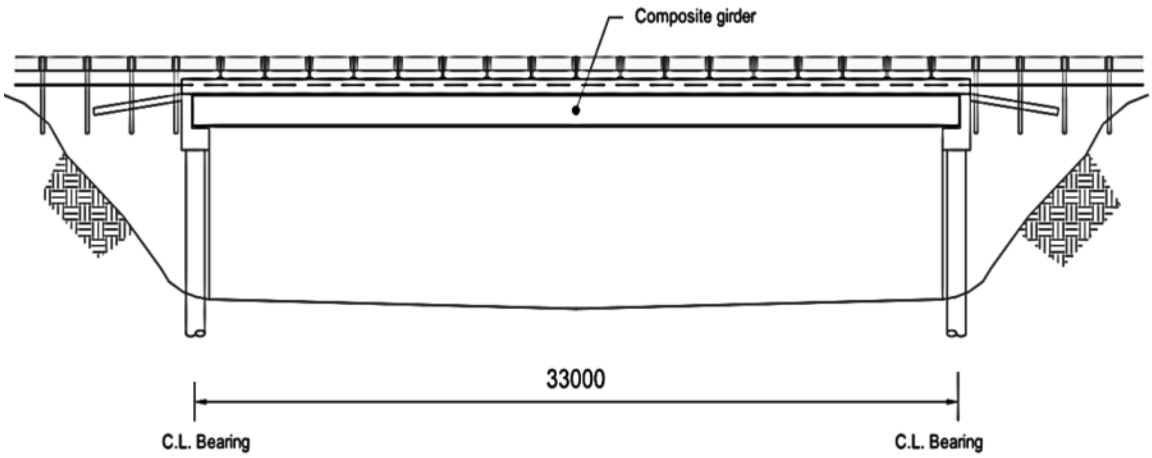


G4.5 Seismic actions

Seismic actions have not been considered in this study. Refer to section 5 of the *Bridge manual* for guidance on determining the horizontal force generated by seismic action on the bridge.

G5 Girder makeup and slab reinforcement

Figure G.3 Girder makeup and slab reinforcement



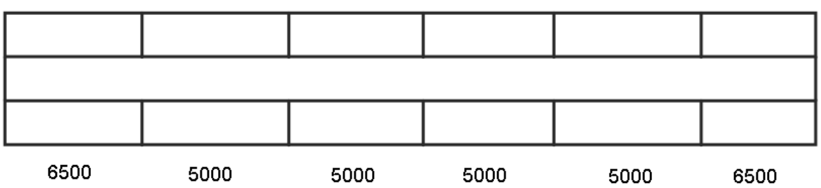
	33.0m span girder
Top flange	600 x 25
Web	20
Bottom flange	800x 50
Top rebar	HD16-150
Bottom rebar	HD16-150

The overall girder depth is 1450mm. the cover to the centroid of the top and bottom reinforcement is 55mm. This is appropriate with reference to *NZS 3101*.

One alternative configuration to in-situ concrete slab is to include precast planks, which typically have pre-stressing in the bottom, and an in-situ slab with lighter reinforcement.

Note: The above section sizes are based on an initial few iterations to determine the section properties for this worked example. For an actual bridge design the designer may wish to undertake further iterations in order to optimise the cross-sections used and gain economy.

Figure G.4 Bracing arrangements



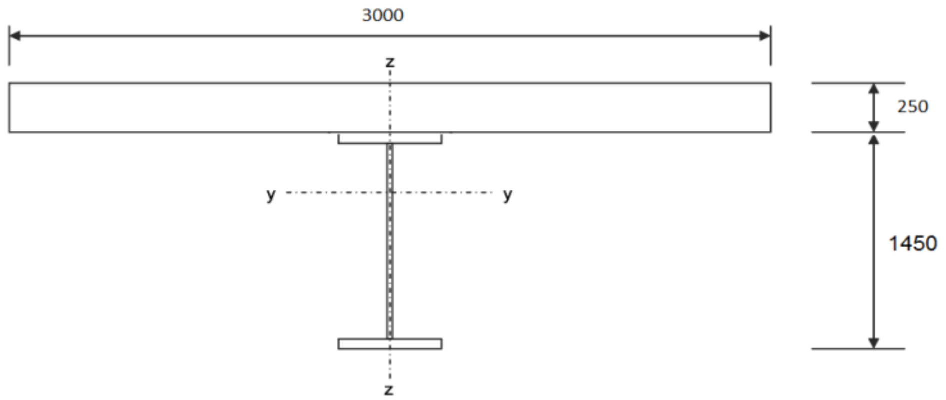
The above bracing arrangements are assumed for this example.

G6 Beam cross section

Refer to section 7.1.1 in the main part of the guide

G6.1 Section properties – internal main girders

Figure G.5 Internal main girders effective width



For determination of stresses in the cross section and resistances of the cross section, the effective width of the slab, allowing for shear lag is needed. The following calculations summarise the effective section properties for the section considered.

The equivalent span for effective width is:

Refer to AS 5100.6, clause 6.1.7

$L_c = 33{,}000\text{mm}$

$$b_{eff} = \min\left(\frac{L_c}{5}; S; 12t_s\right) = \min\left(\frac{33{,}000}{5}; 3700; 12\times 250\right) = 3000\text{mm}$$

Refer to AS 5100.6, clause 4.4.1

Properties for gross section in the mid-span are tabulated below.

Table G.1 Bare steel cross sections

		Span girder	Unit
Area	A	82,500	mm^2
ENA height	ENA	519	mm
Second moment of area	I_y	2.81E+10	mm^4
Section modulus, top flange	$Z_{y,\,tf}$	30E+06	mm^3
Section modulus, bottom flange	$Z_{y,\,bf}$	54E+06	mm^3
Section class		Not compact	
Bending moment capacity(for +ve moment)	M_s	8400	kN.m

Slenderness check:

$$\lambda_{e,t\_flange} = \frac{b_f - t_w}{2t_f}\sqrt{\frac{f_{y,t\_flange}}{250}}$$

$$\lambda_{e,t\_flange} = \frac{600-20}{2\times 25}\sqrt{\frac{280}{250}} = 12.28$$

Refer to AS 5100.6, clause 5.1.2

$$\lambda_{e,web} = \frac{d_1}{t_w}\sqrt{\frac{f_{y,web}}{250}}$$

$$\lambda_{e,web} = \frac{1375}{20}\sqrt{\frac{300}{250}} = 75.3$$

$$\lambda_{e_{y,flange}} = 14$$

$$r_e = \frac{(1450-519-25)}{1375} = 0.66 \qquad \lambda_{e_{y,web}} = \frac{60}{r_e} = \frac{60}{0.66} = 91$$

Refer to AS 5100.6, table 5.1

$$\text{Plastic neutral axis} = 154\text{mm}$$

$$r_p = \frac{(1450-154-25)}{1375} = 0.92$$

$$\lambda_{e_{p,web}} = \frac{111}{4.7r_p - 1} = \frac{111}{4.7\times 0.92 - 1} = 33.4$$

$$\lambda_{e_{p,flange}} = 8$$

$$\frac{\lambda_{e,t\_flange}}{\lambda_{e_{y,flange}}} = \frac{12.28}{14} = 0.88$$

$$\frac{\lambda_{e,web}}{\lambda_{e_{y,web}}} = \frac{75.3}{91} = 0.83$$

$$\frac{\lambda_{e,flange}}{\lambda_{e_{y,flange}}} > \frac{\lambda_{e,web}}{\lambda_{e_{y,web}}} \Rightarrow \text{flange governs}$$

$$\lambda_{e_S} = \lambda_{e_{flange}} = 12.28$$

$$\lambda_{e_S} \geq \lambda_{e_p}$$

Therefore, the bare steel section is **not compact**.

Table G.2 Composite cross section (short term) – sagging (n_s=6.25)

		Span girder	Unit
Area	A	202,500	mm^2
ENA height	ENA	1145	mm
Second moment of area	I_y	8.3E+10	mm^4
Section modulus, top of slab	$Z_{y,c}$	150E+06	mm^3
Section modulus, top flange	$Z_{y,tf}$	272E+06	mm^3
Section modulus, bottom flange	$Z_{y,bf}$	73E+06	mm^3
Section class		Not compact	
Bending moment capacity	M_p	24,671	kN.m

The cross section of the span girder is compact provided that the top flange is restrained by shear connectors within the spacing limits given in *AS 5100.6*, clause 6.1.8. (the spacing of the shear stud is more likely to be 150mm)

Plastic moment capacity for a compact composite section can be calculated using the formula given in *AS 5100.6*, appendix E.

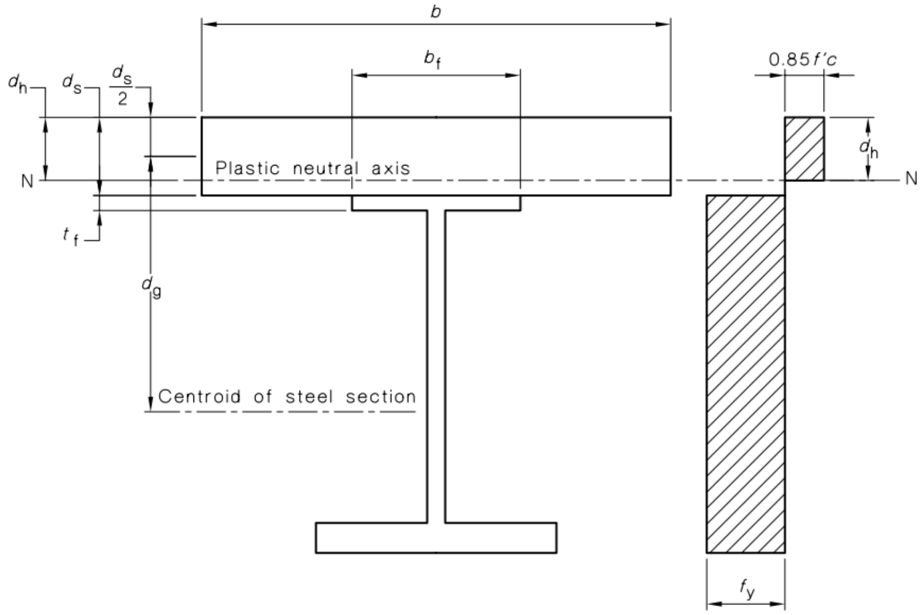
$$d_h = \frac{f_y A}{0.85 f'_c b} = \frac{280 \times 82{,}500}{0.85 \times 40 \times 3000} = 226\text{mm}$$

$$d_h \le d_s$$

$$M_p = Af_y\left(d_g + \frac{d_s - d_h}{2}\right) = 82{,}500 \times 280\left(1056 + \frac{250 - 226}{2}\right) \times 10^{-6} = 24{,}671\text{kN.m}$$

$$d_g = 1450 + \frac{250}{2} - 519 = 1056\text{mm}$$

Figure G.6 Section moment capacity compression zone entirely within the concrete slab



Note: During the construction of a composite bridge, it is quite likely that a beam will change its section class, because the addition of the deck slab both prevents local buckling of the top flange and significantly shifts the neutral axis of the section. Typically, a mid-span section could be compact after casting the slab but not compact prior to this. As a consequence of this, checks at intermediate stages of construction should be based on the relevant classification at the stage being checked.

Table G.3 Composite cross sections (long term) – sagging (n_L=19)

Refer to section 7.3.1 in the main part of the guide

		Span girder	Unit
Area	A	121,875	mm2
ENA height	ENA	860	mm
Second moment of area	Iy	5.8E+10	mm4
Section modulus, top slab	Zy, c	69E+06	mm3
Section modulus, top flange	Zy, tf	98E+06	mm3
Section modulus, bottom flange	Zy, bf	67E+06	mm3

G6.2 Effects of temperature difference and shrinkage

The primary effects of differential temperature through the depth of the cross section of a member are considered. In addition, the secondary effects in continuous members, due to redistribution of the moments and support reactions caused by the primary effects are also considered.

Longitudinal stresses and shear forces due to differential temperature effects are calculated by elastic theory assuming full interaction between the concrete slab and the steel beam. The stiffness is based on the transformed composite cross-section using a modular ratio appropriate to short-term loading and assuming the concrete slab to have an effective width calculated in accordance with *AS 5100.6*, clause 4.4.1.

When the effects of shrinkage modified by creep adversely affect the structure, they are calculated in the manner described for differential temperature effects, but using a modular ratio appropriate to long-term loading. The beneficial effects of the creep of concrete are taken into account.

The effects of shrinkage in flexural design of girders are neglected as these effects are not adverse in the case of simply supported bridges. However, the longitudinal forces due to the primary effects of shrinkage and differential temperature are both considered in the design of the stud shear connector and transverse reinforcement.

G6.2.1 Temperature difference

For calculation of primary effects, the short-term modulus of concrete is used.

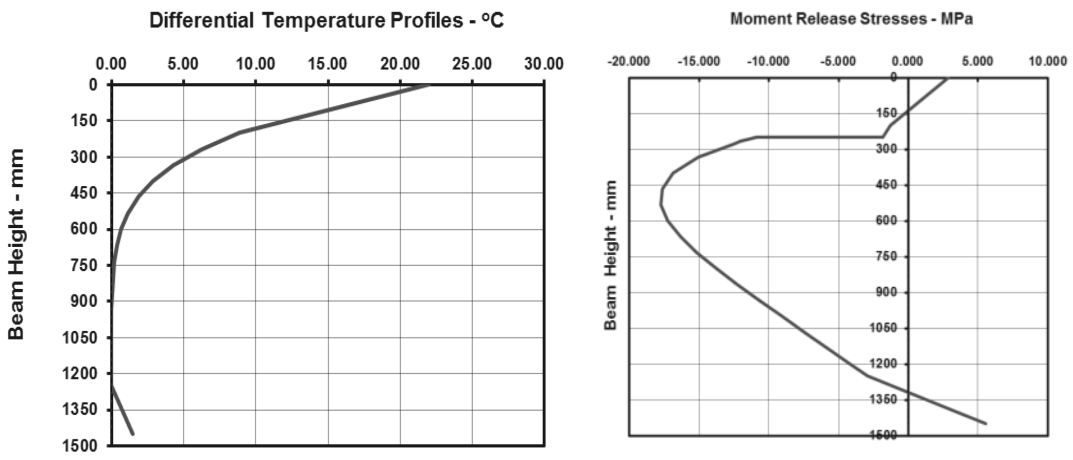
$E_C = 32\text{GPa}$ $\quad\quad$ $E_S = 200\text{GPa}$

For each element of the section, calculate stress as strain time's modulus of elasticity, and then determine force and centre of force for that area. The restraint moment in the inner beam, due to the characteristic values of temperature difference is noted in the *Bridge manual* as:

$T = (32 - 0.2d)^{O}C$ $\quad\quad$ $d = 50\text{mm}$ $\quad\quad$ $T = 22^{O}C$

The moment release stress is shown diagrammatically in figure G.7.

Figure G.7 Moment release stress over the depth of the span-girder section



The effects of the negative vertical temperature variation are insignificant and therefore have been ignored.

G6.2.2 Shrinkage

The effects of shrinkage are calculated for the long-term situation.

The characteristic value of shrinkage is ε_{sh} = 0.0002 and the long-term modular ratio is used.

Table G.4 Temperature and shrinkage induced restraint force

	Force	
Differential temperature axial fixity force	4313	kN
Shrinkage axial fixity force	1943	kN

G7 Global analysis

Refer to AS 5100.6, clauses 4.2 and 4.3

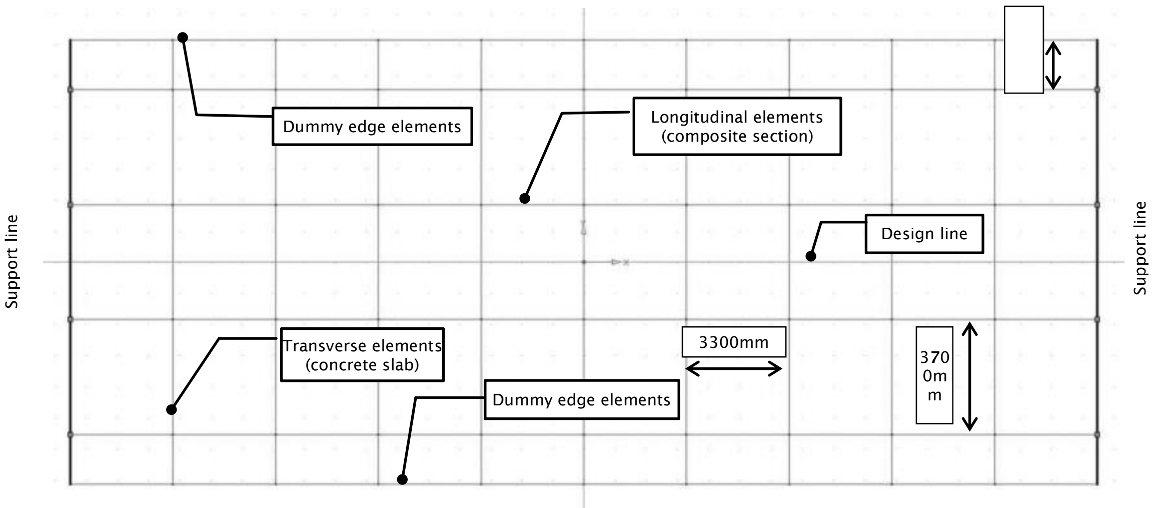
G7.1 Grillage model

Refer to chapter 6 in the main part of the guide

A grillage model of the superstructure was created, with longitudinal elements for composite girder and transverse elements for concrete slab.

As the bridge is simply supported at end supports, no concrete diaphragms were provided at both abutments.

Figure G.8 Grillage model of the superstructure



Note: the grillage model utilised a transverse elements comprising a rectangular slab 3300mm by 250mm. Modern computer models automatically generate section properties; however, these should be checked manually at least for torsional properties to suit the designer's design assumptions.

G7.2 Construction stages

The whole of the deck span is concreted in one stage. The edge beams are concreted after this.

Separate analytical models are therefore provided for:

Stage 1: All steelwork, wet concrete (non-composite section properties)

Stage 2: Composite structure (long-term section properties), the weight of the edge beams is applied

Stage 3: Composite structure (short section term properties)

G7.3 Analysis results

The following results are for design values of actions for an internal beam, ie after application of appropriate factors on characteristic values of actions.

For construction loading, results are given for the total effects at each of the construction stages. For traffic loading the results are given for the combination of traffic and pedestrian loading for worst bending effects at the location of the central span.

Note: The following results may not correlate with a slightly different model and hence should not be viewed as absolute. They do, however, provide an indication of the order of magnitude expected.

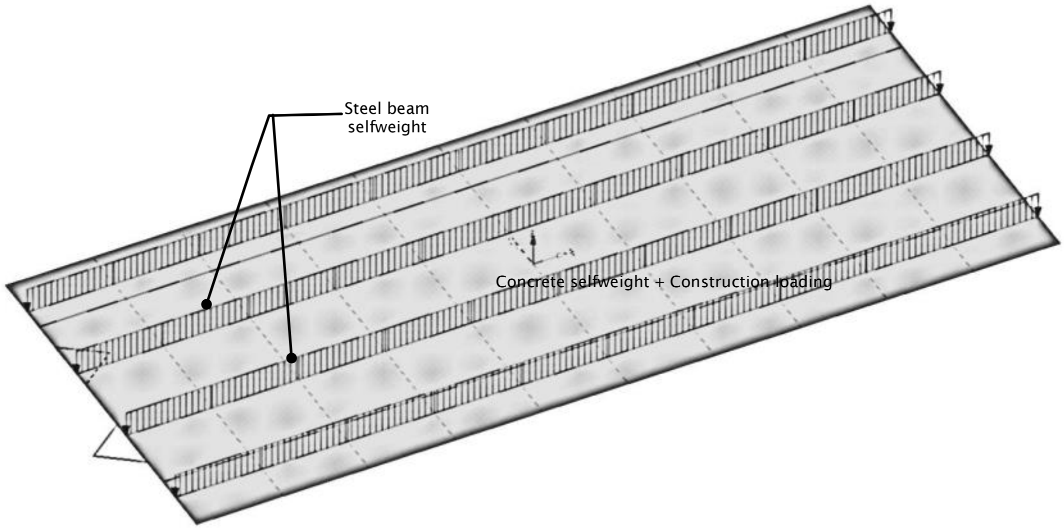
G7.3.1 Stage 1

Selfweight of steelwork

Selfweight of concrete

Construction load

Figure G.9 Stage 1 loading



Distance from support (m)	ULS		SLS	
	M_y (kN.m)	F_z (kN)	M_y (kN.m)	F_z (kN)
0	0	623	0	476
16.5	5867	0	4278	0

Load combination 5B is assumed to be critical

G7.3.2 Stage 2

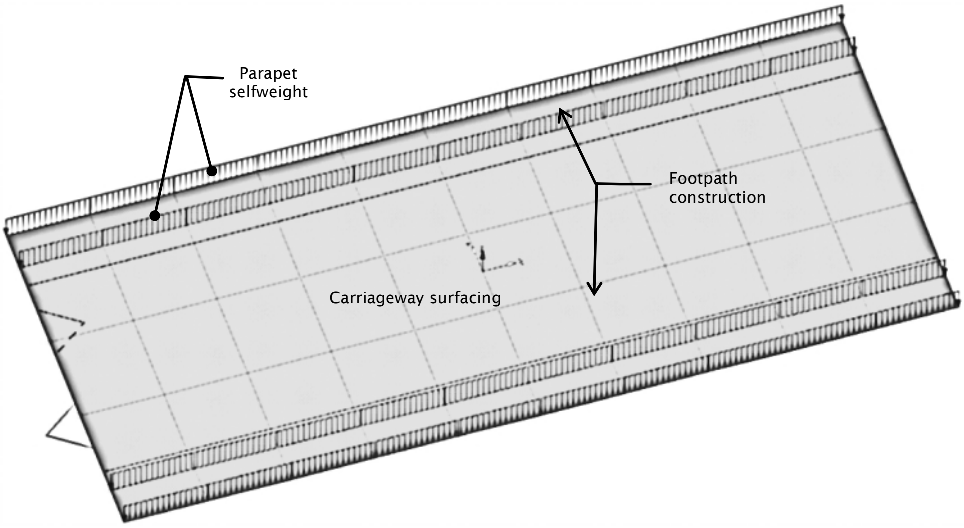
Selfweight of parapets

Selfweight of carriageway surfacing

Selfweight of footpath construction

Removal of construction loading

Figure G.10 Stage 2 loading



Distance from support (m)	ULS		SLS	
	M_y (kN.m)	Fz (kN)	M_y (kN.m)	Fz (kN)
0	0	278	0	206
16.5	2918	0	2162	0

Note: Load combination 5B is assumed to be critical

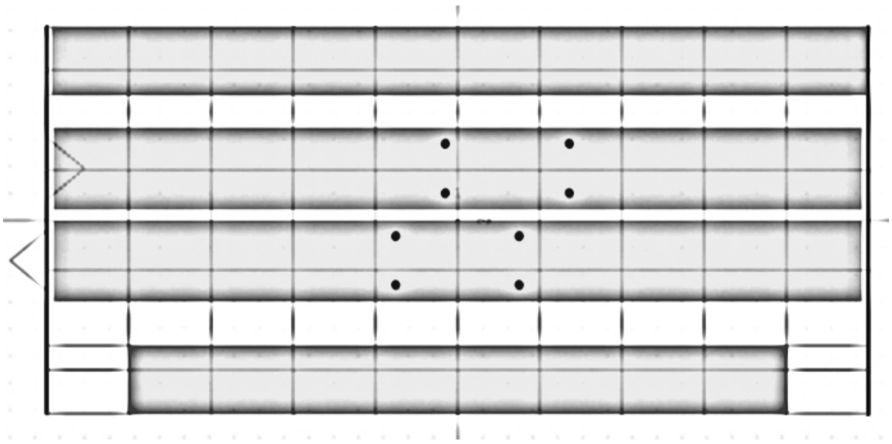
G7.3.3 Stage 3

Table G.5 HN72 traffic loads for worst sagging at mid-span and worst shear at supports

Distance from support (m)	ULS		SLS	
	M_y (kN.m)	Fz (kN)	M_y (kN.m)	Fz (kN)
0	0	921	0	550
16.5	6052	0	3603	0

Note: HN-72 loading is assumed to be critical

Figure G.11 Stage 3 loading



Position of vehicle axle loads relative to the bridge abutments for the worst sagging moment at mid-span.

G8 Design values of the effects of combined actions

Design values of effects are given below for certain design situations, for the design of the main girder and cross girder beams. In practice, further situations for other parts of the structure would also need to be considered.

G8.1 Effects of construction loads (ULS)

Generally, the effects of construction loads apply to different cross section properties, the cross sections for the main girder beams are different at stages 1 and 2. The following tabulations summarise the forces and moments at each stage and the stresses due to those effects, for selected cross sections. According to the *Bridge manual* the load combination 5B is the most critical load case for construction stages.

Table G.6 Stress at mid-span

			Bottom flange		Top flange		Top of slab	
	M_y	F_z	$Z_{x, bf}$	σ	$Z_{x, tf}$	σ	$Z_{x, c}$	σ
	kN.m	kN	10^6mm^3	MPa	10^6mm^3	MPa	10^6mm^3	MPa
Stage 1	5867	623	54	109	30	-196		-
Stage 2	2918	278	67	44	98	-30	69	-42
Diff Temp - PE	-	-	-	2	-	-6	-	2
	8785	901		155		-232		-40

Note: Load combination 5B is assumed to be critical

G8.2 Effects of traffic load plus construction loads (ULS)

Effects due to traffic actions are determined from the short-term composite section in mid-span.

Table G.7 Traffic load stress at mid-span

			Bottom flange		Top flange		Top of slab	
	M_y	F_z	$Z_{x, bf}$	σ	$Z_{x, tf}$	σ	$Z_{x, c}$	σ
	kN.m	k	10^6mm^3	MPa	10^6mm^3	MPa	10^6mm^3	MPa
Construction	8785	901		155		-232		-40
HN72 traffic	6052	921	73	83	272	-22	150	-40
	14,837	1822		238		-254		-80

Note: Load combination 1A is assumed to be critical

The resulting stresses in the tables above indicate that the steel section remains elastic under construction loading and full service load.

G9 Verification of bare steel girder during construction

Refer to figure 7.4 in the main part of the guide

The paired beams are susceptible to lateral torsional buckling under the weight of the wet concrete (ie before it hardens and provides restraint to the top flanges).

The beams are partially restrained against buckling by the bracing frames between each pairs at three points in the span.

Use gross section properties for verification of buckling resistance.

Refer to section 7.1.2 in the main part of the guide

G9.1 Evaluation of lateral restraint

If the length of the segment between braces satisfies the requirements in AS 5100.6, clause 5.3.2.4 the nominal member moment capacity of a segment is taken as the nominal section moment capacity, otherwise the member moment capacity is calculated using the expression in AS 5100.6, clause 5.6.1.

$$\frac{L}{r_z} \le \left(80 + 50\beta_m\right)\sqrt{\frac{2rd_f A}{2.5Z_{ey}}}\sqrt{\frac{250}{f_y}}$$ if the segment is of unequal flanged I section

Refer to AS 5100.6, clause 5.3.2.4

$\beta_m = \text{-}1.0$

$L_{s,max} = 6500\text{mm}$

$f_y = 280\text{MPa}$

$$r_z = \sqrt{\frac{I_z}{A}} = \sqrt{\frac{2.58E+09}{82,500}} = 177\text{mm}$$

$$\frac{L_{s,max}}{r_z} = \frac{6500}{177} = 37$$

$$\left(80 + 50\beta_m\right)\sqrt{\frac{2\rho d_f A}{2.5Z_{ey}}}\sqrt{\frac{250}{f_y}} = 30\sqrt{\frac{2\times0.17\times1412.5\times82,500}{2.5\times30\times10^6}}\sqrt{\frac{250}{280}} = 20.6$$

$$r = \frac{I_{cz}}{I_z} = \frac{\left(\dfrac{25\times600^3}{12}\right)}{2.58\times10^9} = 0.17$$

$d_f = 1412.5\text{mm}$

$A = 82,500\text{mm}^2$

$Z_{ey} = \min\left(Z_{y,bf};Z_{y,tf}\right) = 30\times10^6\text{mm}^3$

$$\frac{L}{r_z} = 37 \ne \left(80 + 50\beta_m\right)\sqrt{\frac{2\rho d_f A}{2.5Z_{ex}}}\sqrt{\frac{250}{f_y}} = 20.6$$

$\therefore M_b \ne M_s$

Refer to AS 5100.6, clause 5.6.1

G9.2 Verification

Maximum sagging bending moment at the mid-span under construction stage 1 load is 5867kN.m.

The flexural capacity of the main girder is given in AS 5100.6, clause 5.6 as follows:

$M_b = \alpha_m\alpha_s M_s \le M_s$

At the mid-span the bending moment assumed to be constant, therefore, $\alpha_m = 1.00$

$L_e = k_t k_L k_r L$

$k_t = 1.0$ (LL)

$k_L = 1.0$

$k_r = 1.0$

$L_e = 5.00\text{m}$

$E = 200{,}000\text{MPa}$

$G = 80{,}000\text{MPa}$

$I_y = 2580 \times 10^6 \text{mm}^4$

$I_w = 0$

$J = 31{,}952 \times 10^3 \text{mm}^4$

$M_s = 8400\text{kN.m}$

$$M_o = \sqrt{\left(\frac{\pi^2 EI_y}{L_e^2}\right)\left(GJ + \frac{\pi^2 EI_w}{L_e^2}\right)}$$

$$M_o = \sqrt{\left(\frac{\pi^2 \times 200{,}000 \times 2580 \times 10^6}{5000^2}\right)\left(80{,}000 \times 31{,}952 \times 10^3\right) \times 10^{-6}}$$

$M_o = 22{,}819\text{kN.m}$

$$\alpha_s = 0.6\left[\sqrt{\left[\left(\frac{M_s}{M_o}\right)^2 + 3\right]} - \left(\frac{M_s}{M_o}\right)\right]$$

$$\alpha_s = 0.6\left[\sqrt{\left[\left(\frac{8400}{22{,}819}\right)^2 + 3\right]} - \left(\frac{8400}{22{,}819}\right)\right] = 0.84$$

$M_b = 1.00 \times 0.84 \times 8400 = 7056\text{kN.m}$

$\phi M_b = 0.9 \times 7056 = 6350\text{kN.m}$

$M^* = 5867\text{kN.m} < \phi M_b = 6350\text{kN.m - OK}$

G10 Verification of composite girder

Refer to figure 7.5 in the main part of the guide

G10.1 Sagging bending in main girder

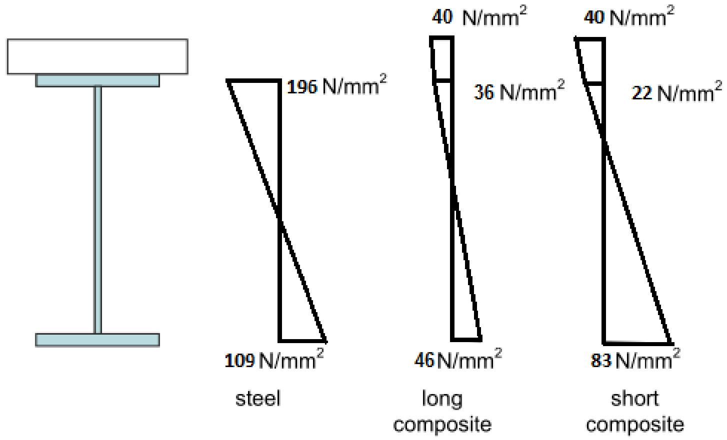
The elastic design bending resistance for a beam constructed in stages depends on the design effects at the stages.

The bare steel section is not compact in bending and the composite cross section is compact. The effects (stresses) in the cross section have been calculated on the basis of gross section properties for effects on the steel beam plus effective section properties on the composite section.

The composite cross section is compact (PNA in the concrete slab) so the plastic resistance can be utilised.

The plastic bending resistance of the short-term composite section is 0.9 x 24,671=22,204kNm and the total design value of bending effects is 14,837kNm. The cross section is satisfactory by inspection. It can also be seen that the stresses calculated elastically, taking account of construction in stages, are satisfactory, as shown in figure G.12.

Figure G.12 Bare steel and composite stresses in sagging



G11 Longitudinal shear

Refer to sections 6.1.7 and 7.7 in the main part of the guide

The resistance to longitudinal shear is verified for the web/flange weld, the shear connectors and the transverse reinforcement at the supports and at mid-span. (In practice, intermediate values would also be verified, to optimise the provision of shear connectors.)

G11.1 Effects for maximum shear

Table G.8 ULS values at supports

	Supports	
Shear on steel section (stage 1)	623	kN
Shear on long term composite section	278	kN
Shear on short term composite section (worst effects)	921	kN

Table G.9 SLS values

	Supports	
Shear on steel section (stage 1)	476	kN
Shear on long term composite section	206	kN
Shear on short term composite section (worst effects)	550	kN

G11.2 Section properties

To determine shear flows, the parameter $A_t y_c / I_t$ is needed for each section and stage.

Table G.10 Section properties

	Web/top fl	Top fl/slab
	$A_t y_c / I_t$	$A_t y_c / I_t$
	m^{-1}	m^{-1}
Steel section	0.490	-
Long term section	0.636	0.487
Short term section	0.675	0.626

G11.3 Shear flow at ULS

Force at web/top flange junction

At abutment 623 x 0.490 + 278 x 0.636 + 921 x 0.675 = 1104 kN/m

Force at flange/slab junction

At abutment 278 x 0.487 + 921 x 0.626 = 712 kN/m

G11.4 Shear flow at SLS

Force at flange/slab junction

At abutment 206 x 0.487 + 505 x 0.626 = 416 kN/m

The shear flow at SLS is required for verification of the shear connectors.

G11.5 Web/flange weld

Design weld resistance is given in tables 9.8 and 9.9 of ASI (1999) design capacity tables.

For 6mm throat SP fillet weld t_w=6mm

$$v_{w_{6mm}} = 0.835\text{kN/mm}$$

Resistance of two welds = 1670kN/m > 1104kN/m, shear flow at top flange – OK

Shear flows at bottom flange are slightly less and are OK by inspection.

G11.6 Shear connectors

Refer to section 7.7.3 in the main part of the guide

Shear stud connectors 19mm diameter, 150mm long are assumed, with f_u=410MPa.

The resistance of a single stud is given by *AS 5100.6*, clause 6.6.4.4 as the lesser of:

Refer to AS 5100.6, clause 6.6.4.4

$$f_{ks} = 0.63 d_{bs}^2 f_{uc}$$

or

$$f_{ks} = 0.63 d_{bs}^2 \sqrt{f'_{cy} E_c}$$

$f'_{cy} = 40\text{MPa}$ $\quad f_{cu} = 410\text{MPa}$ $\quad d_{bs} = 16\text{mm}$

$$E_c = 5050\sqrt{f'_{cy}} = 5050\sqrt{40} = 32{,}000\text{MPa}$$

Therefore the design resistance of a single headed shear connector is:

$$f_{ks} = \min\left[0.63 \times 19^2 \times 410; 0.63 \times 19^2 \sqrt{40 \times 32{,}000}\right] = 93.25\text{kN}$$

Longitudinal shear forces due to the primary effects of shrinkage or differential temperature are assumed to be transmitted across the interface between the steel beam and the concrete slab by shear connectors at each end of the beam, ignoring the effects of bond.

In the absence of a more accurate analysis, the forces on the connectors may be calculated by assuming that the rate of transfer of longitudinal force varies linearly from a maximum at the end of the beam to zero at a distance from the end equal to the total effective width of the slab. Alternatively, where stud shear connectors are used, the rate of transfer of force may be assumed to be constant over a distance from each end of the beam equal to one fifth of the span of the beam.

In this exercise, it is assumed that longitudinal shear flow is constant over the span of the beam.

$$\upsilon_L^* \le \phi\upsilon_{Ls} \qquad \upsilon_{Ls} = 0.55nf_{ks} \qquad \phi = 1.0$$

$$\upsilon_L^* = 416kN/m + \left(\frac{1943\times5}{33\times2}kN/m\right)_{\text{Shrinkage PE}} = 563kN/m$$

Refer to AS 5100.6, clause 6.6.3.2

If studs are grouped and spaced at 200mm spacing along the beam (to suit transverse reinforcement), then a row of 2 studs has a design resistance of:

$$\phi\upsilon_{Ls} = \frac{0.55\times93.25\times3}{0.20} = 769kN/m \ge \phi\upsilon_L = 563kN/m - OK$$

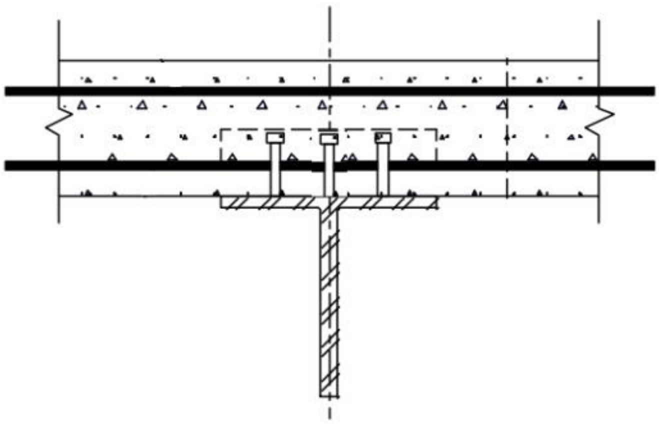
Refer to section 7.7.7 in the main part of the guide

G11.7 Transverse reinforcement

Consider the transverse reinforcement required to transfer the full shear resistance of the studs, ie 522kN/m as well as the maximum ULS shear flow, ie 1154kN/m.

Figure G.13 Transverse reinforcement critical shear planes

Refer to AS 5100.6, clause 6.6.5.2



For a critical shear plane around the studs, shown dotted above, the shear resistance is provided by twice the area of the bottom bars. The design shear resistance of the transverse reinforcement is given by *AS 5100.6*, clause 6.6.5.2 as follows:

$$\upsilon_{Lp}^* \le \phi\left(0.9us + 0.7A_{ts}f_{ry}\right)$$

and

$$\upsilon_{Lp}^* \le \phi\left(0.15uf_c'\right)$$

assumed HD16-150mm:

$$s = 1MPa$$

$$u = 2\times150 + 300 = 600mm$$

$$A_{st} = \left(2\times201\right)/150 = 2680mm^2/m \qquad f_{ry} \le 450MPa$$

$$\phi\upsilon_L = \min\left[1.0\left(0.9\times600\times1.0 + \frac{0.7\times2680\times450}{10^3}\right); 1.0\left(0.15\times600\times40\right)\right] = 1384kN/m$$

$$\upsilon_{Lp}^* = 1104kN/m + \left(\frac{1.35\times1943\times5}{33\times2}\right)_{\text{Shrinkage PE}} = 1303kN/m$$

$$\upsilon_{Lp}^* = 1303kN/m \le \phi\upsilon_L = 1384kN/m - OK$$

The transverse bars are adequate.

G12 Fatigue assessment

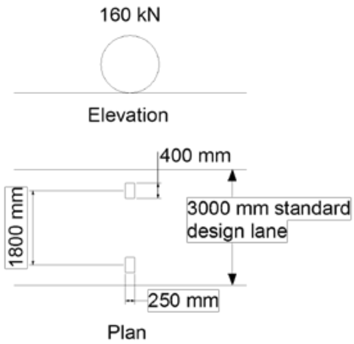
Refer to sections 6.4.2 and 7.11.1 in the main part of the guide

G12.1 Basic loadings for fatigue design

There are two basic loads given, as clause 6.9 of *AS 5100.2*, being:

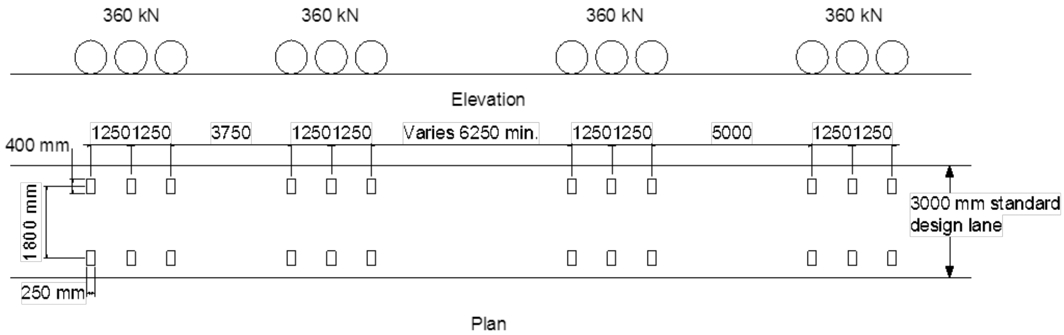
The modified individual A160 heavy axle load from *AS 5100.2*, as shown in below

Figure G.14 Modified individual A160 heavy axle loading



The modified individual M1600 moving traffic load from *AS 5100.2*, without the UDL component, is shown in figure G.15.

Figure G.15 Modified individual M1600 moving traffic loading



The number of fatigue stress cycles used for the calculation of the fatigue capacity of the structural element under consideration should be as follows:

1 For the fatigue design load of 0.70 x (A160 axle load) x (1 + α):

2 (Current number of heavy vehicles per lane per day) x 4 x 104 x (route factor).

3 For the fatigue design load of 0.70 x (M1600 moving traffic load without UDL) x (1 + α):

4 (Current number of heavy vehicles per lane per day) x 2 x 104 (L-0.5) x (route factor).

Dynamic load allowance α is given as following:

$\alpha = 0.4$ for the A160 axle load

$\alpha = 0.3$ for the M1600 axle load.

The route factor for the urban road specified as 0.3.

L is the effective span in metres and is defined as:

- For positive bending moments, L is the actual span in which the bending moment is being considered.
- For negative moment over interior supports, L is the average of the adjacent spans.
- For end shear, L is the actual span.
- For reactions, L is the sum of the adjacent spans.
- For cross-girders, L is twice the longitudinal spacing of the cross-girders.

G12.2 Range of effects due to passage of fatigue vehicle

By inspection it is clear that the fatigue loading of M1600 is more critical, therefore the fatigue loading is:

$$0.70 \times (\text{M1600 without UDL}) \times (1+\alpha)$$

And corresponding total number of cycles is:

$$n = \text{NHVD}_{\text{per lanex}} \times 2 \times 10^4 \left(L^{-0.5}\right)\left(\text{route factor}\right)$$

$\text{NHVD} = 1500$ route factor $= 0.3$

Table G.11 Worst bending effects

	Span
	M_y (kN.m)
Range	2236

Table G.12 Worst shear effects

	Span
	F_z (kN)
Range	336

G12.3 Assessment of structural steel details

G12.3.1 Design stress range mid-span

For the mid-span the length of the effective span is $L = 33.0\text{m}$.

Therefore, the total number of cycles is:

$$n = 1500 \times 2 \times 10^4 \times 33^{-0.5} \times 0.3 = 1.57 \times 10^6$$

At mid-span there is negligible stress range in the top flange. The stress range in the bottom flange is $f^* = \frac{2236}{73} = 31\text{MPa}$.

As $f^* = 31\text{MPa} \geq 27\text{MPa}$ fatigue assessment is required.

The worst detail category that might apply is for the weld between the bottom flange and the web, which, for manual continuous weld, is category 100 (*AS 5100.6*, table 13.5.1-B).

Design value of fatigue strength ϕf_{fc} is:

$$n = 1.57 \times 10^6 \leq 5 \times 10^6$$

$$\phi = 1.0$$

$\phi f_{rnc} = 100$

$$\phi f_{fc} = \phi f_{rnc} \left(\frac{2 \times 10^6}{n} \right)^{1/3} = 100 \left(\frac{2}{1.57} \right)^{1/3} = 108\text{MPa}$$

$f^* \leq \phi f_{fc} - \text{OK}$

G12.4 Assessment of shear connection

Refer to AS 5100.6, clause 13.7.2

The design value of the stress range in shear studs is 425MPa times the ratio of the longitudinal shear load on the stud to the nominal static strength specified in AS 5100.6, clause 6.6.3.

G12.4.1 Shear at supports

The range of vertical shear force at the support is 336kN.

At the pier, the studs are 19mm diameter, in rows of two at 200mm spacing.

$\upsilon_L^* = 336 \times 0.626 = 210\text{kN/m}$

$\frac{A_t y_c}{I_t} = 0.626\text{m}^{-1}$

$\phi \upsilon_{Ls} = 513\text{kN/m}$

$\phi = 1.0$

Therefore,

$$f_s^* = \frac{\upsilon_L^*}{\upsilon_{Ls}} \times 425 = \frac{\frac{210 \times 0.2}{2}}{513} \times 425 = 96\text{MPa}$$

The fatigue strength of the shear stud is given in AS 5100.6, clause 13.6.3 as following:

$$\phi f_f = \phi \left(\frac{2.08 \times 10^{22}}{n} \right)^{1/8} = \left(\frac{2.08 \times 10^{22}}{1.57 \times 10^6} \right)^{1/8} = 104\text{MPa}$$

$f_s^* \leq \phi f_f - \text{OK}$

Note: that designers may select the transverse reinforcement spacing to avoid clashed with shear studs.