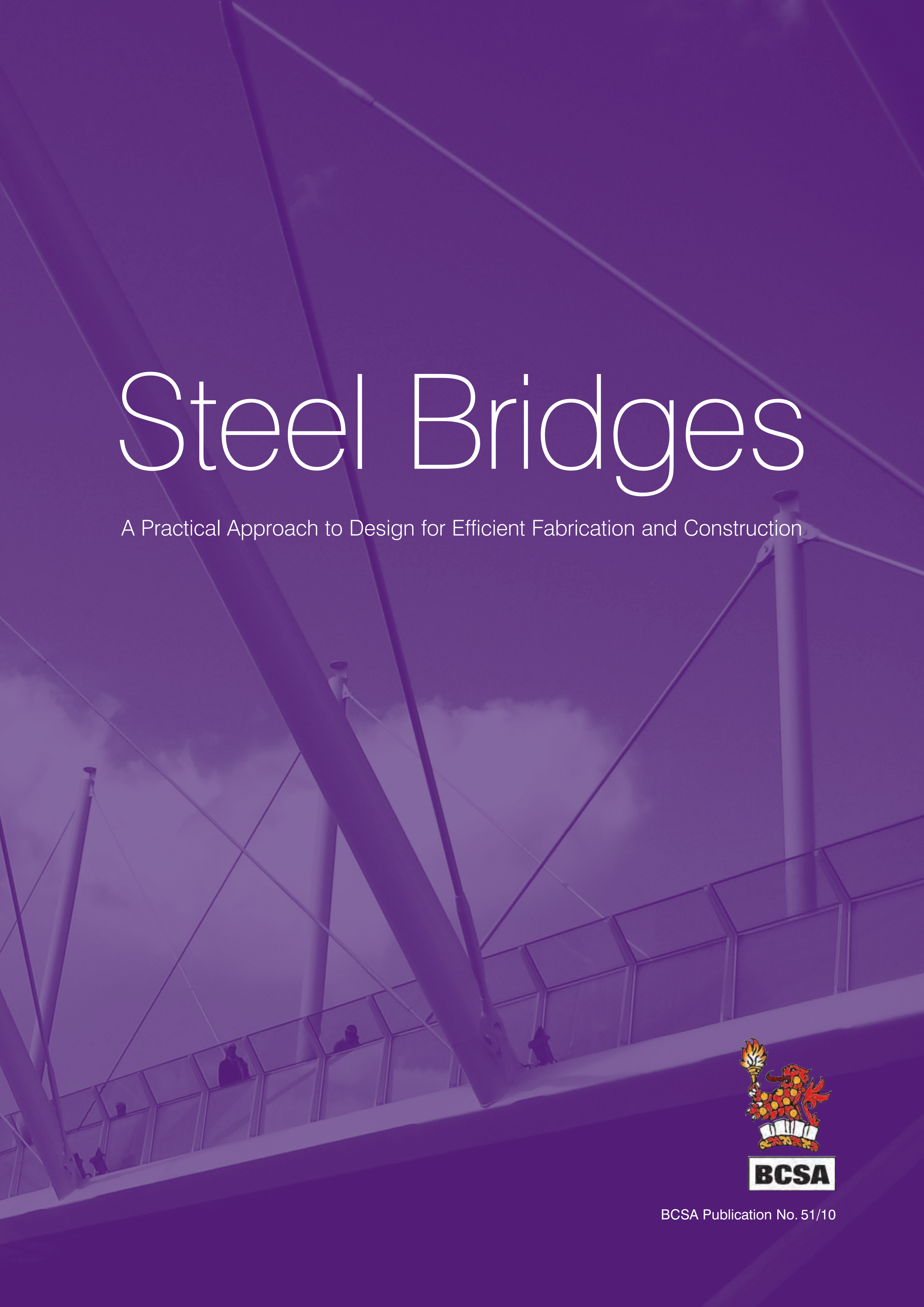
Steel Bridges

A Practical Approach to Design for Efficient Fabrication and Construction



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The principal objectives of the Association are to promote the use of structural steelwork; to assist specifiers and clients; to ensure that the capabilities and activities of the industry are widely understood and to provide members with professional services in technical, commercial, contractual and quality assurance matters. The Association's aim is to influence the trading environment in which member companies operate in order to improve their profitability.

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BCSA's website, www.SteelConstruction.org, can be used both to find information about steel construction companies and suppliers, and also to search for advice and information about steel construction related topics, view publications, etc.

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STEEL BRIDGES

A Practical Approach to Design for Efficient Fabrication and Construction

PREFACE TO THIRD EDITION

Design of steel bridges is achieved most effectively when it is based on a sound understanding of both the material and the methods adopted in processing steelwork through to the final bridge form. The aim of this publication is to provide a basis for this understanding by reference to the factors that influence safe, practical and economic fabrication and erection of bridge steelwork.

The first edition of this publication in 1985 was prompted by the need for the steel construction industry to give general guidance and information to designers of small and medium span bridges. It provided an insight to the practical aspects of fabrication and erection at that time and included some general design guidance that was not then available in other UK publications.

Since 1985, there have been significant developments in technology and capability and the standards for design, materials and products, workmanship, and inspection and testing have changed considerably. At the end of March 2010, the British Standard for the design of steel and composite bridges, BS 5400, was withdrawn and replaced by the CEN Eurocodes, a set of standards published in the UK by BSI; for steel structures, BS EN 1993: Eurocode 3 Design of Steel Structures applies. The rules in Eurocode 3 relate to steel structures that have been constructed in accordance with BS EN 1090-2, another new CEN standard. That standard in turn refers to a range of CEN and ISO standards, published by BSI as BS EN and BS EN ISO standards, for materials, products, workmanship, equipment, qualifications etc. These new standards have largely replaced the former UK national standards.

In addition to the technical developments, regulations concerning the health and safety of construction activities have also been introduced. Currently, these regulations are the Construction (Design and Management) Regulations 2007. The Regulations set out the responsibilities for all parties involved in construction contracts and in so doing require a greater mutual understanding of the roles of the various parties.

This third edition is a comprehensive revision of the previous publication (which had been updated as a second edition in 2002) that fully reflects the new standards and regulations. The original structure of the book has generally been retained: Chapters 1, 2 and 3 provide guidance on conceptual design, steel quality and design of members; Chapters 4 and 5 give practical information on bolting and welding for connections; Chapter 6 discusses the accuracy of fabrication; Chapter 7 gives some guidance on costs. A new set of case studies is included in Chapter 8 and Chapter 9 discusses competence in construction and in particular the role that the Register of Qualified Steelwork Contractors Scheme for Bridgeworks can play.

This third edition has been prepared under the direction of Alex Smale (Mabey Bridge Ltd) with the assistance of the following:

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Finally, this third edition was reviewed by David Iles (The Steel Construction Institute).

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1.1 Introduction

Steel is usually the material best suited to meet the requirements for highway and railway bridges, footbridges and moveable bridges. Steel construction is, though, a somewhat unusual part of civil engineering construction, as all of the planning and preparation is done off site, and most steel elements critical to the bridge are made in a factory remote from the site. The work at site may last only a few days although, for larger bridges, work could last several months, if not longer. Production of the bridge steelwork in a factory has many advantages in terms of precision, quality, economy and safety but the differences from other civil engineering activities need to be understood if the potential of steel is to be exploited by the designer.

The objective of this publication is to facilitate an understanding of those aspects of fabrication and erection that influence the quality and economics of steel bridgeworks so that the designer is able to achieve an optimum solution for the client.

Fabrication of the steelwork represents a significant part of the overall cost of a steel or composite steel and concrete bridge. Each fabrication shop has a layout that has been developed with equipment appropriate to the types and scale of structures in which the company specialises. There is not always a single preferred method for fabricating a particular component or detail. Some details may suit automated fabrication processes; such details may be less appropriate for those companies which use more traditional practices. Good quality work can be produced in highly automated shops, almost without the use of fabrication drawings, or by skilled tradesmen in shops with modest equipment. However, there are limitations and impractical arrangements to be avoided which, if anticipated knowledgeably at the design stage, will greatly facilitate economical and reliable fabrication.

Most of the advice given in this publication relates to highway, railway and pedestrian bridges in the UK, of short and medium span and of conventional types. Steel is eminently suitable for long span bridges, such as suspension bridges and cable-stayed bridges, and for movable bridges: whilst these bridges are outside the scope of this book, much of the content is relevant to them.

The production of a new bridge involves a team of organisations and individuals working together. Traditionally, a client would separately employ an engineer (to undertake design and specification) and a contractor (who would construct the bridge in accordance with the specification and other contract documents). With new forms of procurement, these separate roles do not necessarily exist. For the purposes of this publication, it is convenient to refer to 'the designer' for the roles of design and technical specification and 'the steelwork contractor' simply for the role of carrying out the construction work (chiefly the procurement and fabrication). This avoids identifying who employs the designer and ignores any other role of the steelwork contractor.

The steelwork contractor and the designer for a steel project must understand each other's roles and responsibilities to obtain the best outcome economically and technically. The advice given in this publication will help the designer understand the steelwork contractor’s role.

Ideally, the designer should have close contact with the steelwork contractor and be able to discuss ideas with him where they have a mutual bearing. Equally, it is important that the steelwork contractor is free to raise points with the designer where the end-product and the project outcome for the client will be improved. This is made easier when a design and construct contract is used, as has been demonstrated on many successful bridge projects over the last 20 years.

1.1.1 Standards

At the end of March 2010, the UK national standard BS 5400 *Steel, concrete and composite bridges* was withdrawn and superseded by the Eurocodes, a set of standards encompassing most types of structure (including bridges) using the most common construction materials. For steel bridge design, the key Eurocode Part is BS EN 1993-2: Eurocode 3 - *Design of steel structures - Rules for bridges*. This Part refers to many other Eurocode Parts and for a listing of relevant Parts see the references in Chapter 10.

The Eurocodes are published by the national standards bodies – in the UK by the BSI – and are implemented by the issue of National Annexes. Some of the UK National Annexes refer to ‘Published Documents’ issued by BSI. In designing a steel or a composite bridge, designers need to refer to all the relevant Eurocodes, National Annexes and Published Documents.

Eurocode 3 states that it is intended to be used in conjunction with BS EN 1090-2 *Execution of steel structures and aluminium structures – Technical requirements for steel structures*. That standard sets out requirements for materials, products and workmanship to ensure that the reliability intended by design to the Eurocodes is achieved. Although comprehensive, BS EN 1090-2 contains many options and alternatives that need to be covered in a project specification.

The implementation of the Eurocodes does not change the essential characteristics of working with steel. Nor does the introduction of BS EN 1090 affect the essential quality of construction, though the documentation is more comprehensive than that previously used.

In addition to these standards, for highway bridges reference has to be made to Departmental Standards within the Design Manual for Roads and Bridges and for railway bridges reference has to be made to Railway Group Standards.

1.1.2 Sustainability

Sustainability is a crucial issue for today's construction industry: construction should be considered in terms of social and environmental responsibility as well as economic viability. Steel is a truly sustainable construction material, offering real benefits under each heading of the 'triple bottom line' approach, and good design underpins sustainable construction.

Structural steel contains a significant proportion of recycled material, and is itself multicyclable without loss of properties at the end of the life of the structure. It has an intrinsic value, and a sophisticated network that captures 99% of structural steel that becomes available, for either reuse or recycling into new steel products, already exists.

The majority of steel bridge construction work is carried out off-site in controlled factory conditions. Investment, innovation and technological advances mean that modern fabrication is highly automated, resulting in higher productivity, less waste, greater energy efficiency, and better working conditions for a highly trained workforce.

Steel bridge erection is typically a brief and highly organised activity, utilising mobile plant and equipment and a small workforce of skilled people. Delivery can be timed to minimise inconvenience to local residents, and the erection process is clean with little dust or noise. Crucially, the speed of steel bridge erection minimises disruption to existing traffic, reducing the duration of any environmental impact.

The sustainability of a bridge project is an issue for both the steelwork contractor and the designer, and is dependent on the quality of the care and skill with which they fulfil their respective roles.

For further information see www.SustainableSteel.co.uk and the Corus publication *Steel bridges - Material matters, Sustainability*.

1.1.3 Health and safety in construction

Since the publication of the first edition of this book, the regulatory environment for health and safety in construction has been much enhanced. Health and safety considerations have become an integral part of design as well as in planning and carrying out construction works – the Construction (Design and Management) Regulations (known as the CDM Regulations) have been a major factor in this development. Although this edition has not been extended specifically to cover such considerations, its underlying purpose is to improve the designer's understanding of steelwork and his competence in using it for bridge construction. The advice will help the designer to consider issues such as designing for the work to be done in the workshop rather than on site, stability of slender girders during erection and access for site tasks.

The CDM Regulations also require the client, with professional advice, to satisfy himself that the chosen steelwork contractor is competent to undertake the challenge of his design. The establishment of the independent Register of Qualified Steelwork Contractors Scheme for Bridgeworks (RQSC) addresses this by including only those steelwork contractors who can demonstrate their relevant set of commercial and technical qualifications to undertake steel bridgeworks.

The Register, which is described in Chapter 9, categorises the competent contractors by size (turnover) and capability for different types of bridge. The Highways Agency now requires that only steelwork contractors on The Register are engaged for UK bridgeworks.

1.2 Initial conception

For new build highways or railways, the requirements for bridges are determined by the alignment, local terrain and the obstacles that are to be crossed. Ideally, the designer should be involved at an early stage to optimise overall geometry for the bridge spans and the interfaces with earthworks, taking account of the cost influences of sighting distances, skew, curvature and construction depth. At the time of construction of the early UK motorways, 'greenfield' conditions allowed considerable freedom of choice in alignments and methods of construction. However, since the 1980s, with the growing urbanisation and increase in traffic density in the UK, considerable restrictions have arisen in the provision of new transport systems and the extension of existing systems. Thus, the obstacles to be overcome are increased and designers are often constrained to adopt curved, tapered or skewed structures together with severe limitations on construction depth. Moreover, public objection to traffic disruption means that the methods and speed of construction and subsequent maintenance heavily influence bridge design.

These trends have encouraged a move towards prefabrication of elements, favouring the selection of structural steel as the primary medium for bridge spans, whilst capitalising on the merits of reinforced concrete for substructures, for permanent formwork and in bridge deck slabs.

Whole life costing is often an essential consideration in the design of bridges. Motorways, trunk roads and railways in the UK are all heavily trafficked: closures to allow repainting of bridge steelwork carry significant costs in delay to users and maintenance requirements should be minimised. For this reason, weathering steel has become an increasingly popular choice for bridges over railways and highways. Guidance on the use of weathering steel is given in Chapter 2.

For replacement spans, or new bridges beneath existing highways or rail tracks, the form of construction will be dictated by the needs of the live highway or railway in keeping disruption of traffic to an absolute minimum. This favours prefabricated forms of construction which can be erected rapidly during possession, or which can be assembled adjacent to the highway or rail track for speedy installation by sliding, rolling-in or wheeled transportation methods. Steel is ideal as the main structural material in these situations. Steel is especially advantageous for replacement railway bridges, where construction depth is often severely constrained, or has to be reduced to improve clearances for the highway below.

Where there is some freedom in the choice of span lengths it should be borne in mind that the optimum spans for steel and concrete bridges are not always the same. For a single span, steel bridges are most economic over the range of about 25 m to 45 m; single spans up to about 60 m can be achieved but the girders become large. With multiple spans it is rarely economic to use a succession of simple spans; continuous multiple spans are easily achieved in steel and lead to savings in amounts of material, bracings, bearings and expansion joints. The optimum span for multi-span construction depends on the relative cost of the substructure for intermediate supports: typically the optimum span is in the range 30 m to 50 m. Continuous spans require much less maintenance of bearings and expansion joints, compared to a series of simple spans, and offer improved appearance of the piers. The end spans of a multiple span bridge should be about 80% of the penultimate span for economy, but may be constrained by other factors; there may be uplift problems at the abutment if the end span is less than about 60% of the penultimate span.

Generally, bridge span lengths can be categorised as:

Short	Up to 30 m
Medium	30 to 80 m
Long	Greater than 80 m

When choosing the span arrangement, ground conditions and access for construction have a significant effect. Poor ground conditions lead to large and expensive foundations; a high-level viaduct increases the cost of both the piers and the erection of the superstructure.

Steel is able to deal with skewed or curved alignments efficiently although, for single spans, it is often convenient to provide a straight bridge and to increase the width appropriately. This is likely to be economic where the width increase does not increase the gross plan area by more than about 5%. In other cases, and especially for multiple spans, for best value and improved appearance the bridge should be truly curved. Use of curved girders that follow the curvature of the deck edge gives a constant length for the concrete deck cantilevers, which is preferred when using a proprietary formwork system. Curved bridges require a greater amount of bracing – see further comment in Section 1.6 – but this does not usually add significantly to the overall cost.

Selection of steel allows a variety of pier shapes to be used to suit functional or aesthetic requirements. When determining the form of the piers, it should be borne in mind that it is preferable to locate the bearings directly under the main girders. For wide decks spanning highways, it is often preferable to avoid solid leaf type piers, which can create a 'tunnel effect' for users of the highway below. Piers can be formed from concrete prismatic or tapered columns, or portal frames. For multi-girder decks, the number of columns within a pier can be reduced by the use of integral steel crossheads; for ladder deck structures, individual columns under each girder are used, giving an open appearance.

For highway bridges, integral construction at the abutments must be considered. Bridge decks up to 60 m in length and with skews (at the abutments) not exceeding 30° are expected to be made integral with the abutments unless there are valid reasons, such as large differential settlements, for them not to be so. 60 m is not a limit on maximum length for integral bridges, the Highways Agency (HA) has accepted integral bridges with lengths greater than 100 m. Use of steel allows the abutment concrete to be cast in situ around the ends of the girders and their bracing, and this is usually done at a stage subsequent to deck slab concreting so that there is no locked-in moment from the dead load of the deck. Where intermediate piers are of concrete construction, building them in so that they are monolithic with the deck slab should be avoided as this makes the bridge construction more complex and so will incur extra cost.

Movable bridges fall outside the scope of this publication, but are nearly always constructed in steel so as to minimise dead weight and thereby the substantial costs of the operating machinery, bridge operation and maintenance. Modern examples include bascule, swing, vertical lift, retractable bridges and roll-on/roll-off ramps as used in port areas and on highways where it is impracticable or uneconomic to provide a fixed bridge with enough navigation headroom. The type depends upon the required navigation width, height, deck width, frequency of opening and aesthetic requirements.

Cable-stayed bridges are suitable for very long spans; recent international projects have used cable-stayed spans of over 800 m. Suspension bridges are used for the longest spans (up to 1990 m, to date).

1.3 Cross sections for highway bridges

For highway bridges, I-section girders with a concrete slab on top, acting compositely, are generally preferred. For long spans, box girders, also with a compositely acting deck slab, may sometimes be chosen – they have a 'cleaner' appearance, offer greater torsional stiffness and can avoid the use of very thick flanges. Occasionally, half-through construction is used where construction depth is critical, as in waterway crossings in flat terrain.

For deck type construction, an economic construction depth, excluding surfacing depth, is generally about 1/20 of the span, but this can be reduced to about 1/30 of the span where necessary.

It is usual to cantilever the deck slab beyond the outer girder, which has a number of advantages as it

- is visually attractive, giving a shadow line that reduces the apparent depth of the girder;
- protects the outer face of the steelwork from rain washing and subsequent staining;
- reduces the width of piers; and
- optimises the deck slab design.

However, the deck cantilever should normally be restricted to 1.5 m for minimising cost and be no longer than 2.5 m to avoid high falsework costs. The use of proprietary cantilever formwork systems has become common recently. The systems can be used for cantilevers up to 2.5 m (including the edge beam) and are designed for ease of attachment to and removal from the outer girders. The connection of such systems and the temporary loads imposed on the outer girders needs to be considered by the designer.

Figure 1 shows typical cross sections for deck type composite bridges for a two-lane highway. On the left the section is within the span showing typical bracing, and on the right the section is at a support. For continuous spans up to about 25 m, multiple rolled section beams, using Universal Beams UB/Corus Advance® UKB sections, may be suitable. The number and spacing of the beams will depend upon the width of the deck, the available construction depth and the provisions that are needed for services. UKB/UB sections have relatively thick webs; shear is not usually a problem, rendering intermediate web stiffeners unnecessary, except where required for the attachment of the bracing system. It is quite likely that plastic bending resistance can be utilised in the hogging moment regions. Generally, an even number of girders should be used to allow erection in braced pairs. The most cost effective multi-girder solution will have girders at 3.0 to 3.5 m spacing.

Plate girders are usually more economic for spans greater than about 20 m and are suitable for spans up to about 100 m. Again, an even number of girders should be used to allow erection in braced pairs. The girder spacing will typically be about 3.5 m. Medium spans may use girders that are profiled in elevation, either with a curved soffit or with tapered haunches at intermediate supports. An important aspect to be considered in the design and construction of plate girders is stability during erection. Bracing will be needed for this temporary condition – see further comment in Section 1.6.

Twin plate girders with cross girders ('ladder-deck' bridges) offer certain advantages for medium span bridges up to about 24 m in width. With a ladder deck, there are fewer main girders to fabricate and erect and there is economy of web material. Steel cantilevers are a feasible option with this cross section, although they require some expensive detailing and extra effort during erection. Ladder decks are suited to erection of the complete deck structure by launching. For spans in excess of about 80 m, very thick flanges will be needed.

For both multi-girder decks and ladder decks, girders curved in plan can be fabricated with the flanges cut from plate to the line of the curve and the web plate set up to follow the curve.

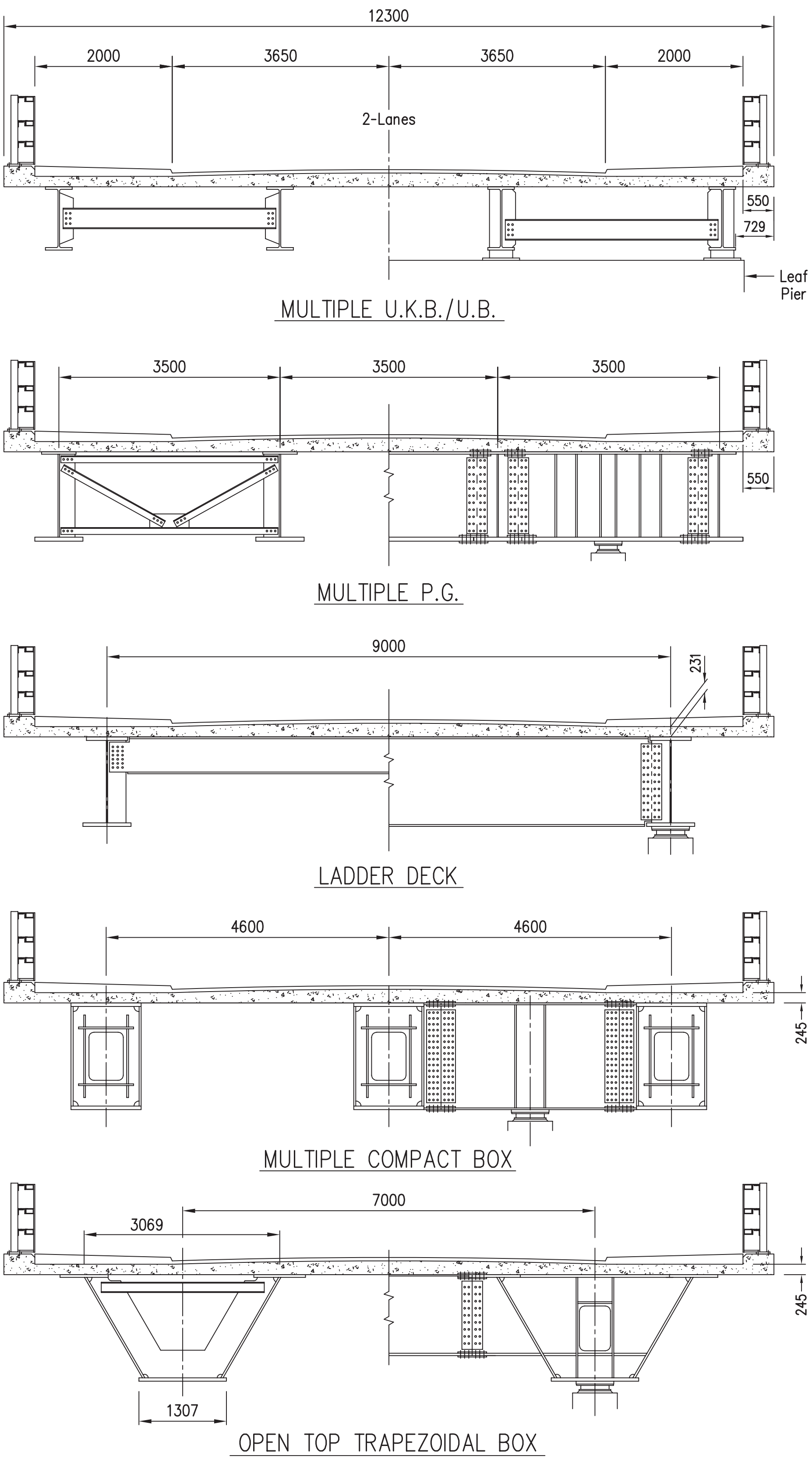
Multiple box girders are suitable for medium spans where appearance of the bridge soffit is very important and the presence of bracings is deemed unacceptable, as for example in a prominent urban area where pedestrians view the soffit. However, such boxes are rather small in terms of access facility: health and safety considerations will have significant consequences on detailing and lead to expensive fabrication.

Open-topped box girders avoid many of the health and safety issues with closed steel boxes; recent examples in the UK include the Olympic Park bridges in London. Lateral and plan bracing systems are, however, necessary to maintain stability of the top flanges during construction (the bracing should be designed to be left in place). Closed boxes, which are more stable during fabrication and erection, are also used, but health and safety regulations for confined space working can make work inside them very expensive.

For longer spans, twin box girders with cross girders and cantilevers become a viable option.

For very long spans, where the weight of the deck becomes dominant, it is more appropriate to use an all steel orthotropic stiffened deck plate instead of a concrete slab. For the primary members either single or twin box girders are used. Such boxes and deck are assembled from separate stiffened plate elements at a construction yard convenient for the site, and transported to site for erection as large units. This form of construction is beyond the scope of the present publication.

Two of the examples in Figure 1 show the use of integral crossheads at supports. These are used to reduce the number of bearings and to avoid the use of a leaf pier. Such crossheads are a very effective means to achieve those aims but they are expensive to fabricate and erect, particularly for curved and super-elevated decks, so the benefits must be balanced against the extra cost and time required.



SPAN RANGE (METRES)	REMARKS
Up to 25	May allow use of plastic bending resistance in hogging regions.
20 TO 100	Integral Crosshead shown to reduce No. of columns.
	May use Leaf Pier as above.
35 TO 90	Suitable curved soffit.
35 TO 100	No bracing up to 60m span.
	Less economical unless aesthetics govern.
	Steel Crossheads shown to reduce No. of columns.
35 TO 100	No bracing up to 60m span.
	Temporary Plan Bracing required.

Figure 1 - Cross sections for 2-lane highway bridges

1.4 Cross sections for railway bridges

The majority of railway bridges are replacements for existing spans and the cross sections that can be considered are usually strongly influenced by construction depth limitations because existing decks often have sub-standard ballasted track depth, or the rails are mounted directly via longitudinal timbers. Modern standards typically demand 300 mm ballast depth beneath sleepers giving a total track depth (including waterproofing and its protection) of approximately 690 mm below rail level. These considerations inevitably lead, in most cases, to half through construction. Then the positions of the upstanding girders must the be considered in relation to the structure gauge and lateral clearance for walkways, taking account of any curvature and cant, allowing for centre-throw and end-throw of rail vehicles. Where each track is separately supported, the girders should preferably not extend more than 110 mm above rail level (although in some cases they can fit within the gauge, up to a height of 915 mm). Additionally, current standards recommend the provision of a robust kerb extending at least 300 mm above rail level to contain derailed vehicles (most existing short span bridges do not possess this).

Three examples of half through construction are shown in Figure 2; these represent the most common cross sections for deck replacements and for new bridges beneath existing railways to avoid costly or impractical track lifting. Spans are generally simply supported, which greatly facilitates piecemeal construction under traffic conditions.

The Network Rail Standard Z-type bridge uses shallow plate girders of Z-shape cross section, usually with a taller girder at the outside to provide a nominal robust kerb, which also helps to retain the ballast on the bridge. This configuration provides a maintenance space between adjacent decks, with cross girders and enveloping deck slab. A variant is the U-deck (now a Network Rail standard, formerly known as the Cass Hayward U-deck), which integrates the main girders with a deck of either all steel or composite form to achieve a single piece for fabrication and erection.

For spans between about 20 m and 50 m, the girders of two-track plate girder bridges can be located outside the structure gauge, so increasing the span of the deck between the main girders. This configuration is modelled on the former type 'E' bridge, with rolled section cross girders spanning between rigid shear plate bolted connections in line with external stiffeners to give 'U' frame stability.

The Network Rail Standard Box Girder type bridge that covers a span range from 21 m to 39 m uses trapezoidal box girders with a transverse ribbed steel deck spanning between notionally pin-jointed shear plate connections: the box girders are stabilised by linear rocker bearings. This design is particularly suited to piecemeal crane erection during track possession. For some recent projects, plate girder alternatives have proved economic where the site has sufficient width to accommodate them.

With half-through construction, the deck can be either in situ concrete, partially encasing close centred cross girders, or a normal slab above more widely spaced cross girders. Stiffened steel plate construction can also be used, depending on the envisaged erection method and available construction depth.

For spans in excess of about 60 m, through or half-through trusses or bowstring girders become appropriate. Examples include the Newark Dyke bridge reconstruction with a braced arch span of 77 m, and the East London Line's Bridge GE19 with an 84 m span Warren truss, and New Cross Gate bridge with a Warren truss of 75 m span.

For new build railways, it is often possible to adopt a deck type solution with the benefit of an efficient cross section of a concrete slab acting compositely with plate or box girders. This configuration is similar to the multi-girder decks for highway bridges, except that the main girders are more closely spaced.

Where train speeds in excess of 125 mph are envisaged, special design criteria need to be applied concerning limits on vibration and deformation. These may influence the depth of construction and the form of the bridge deck. Recent examples of bridges designed for high speed trains are to be found on the Channel Tunnel Rail Link.

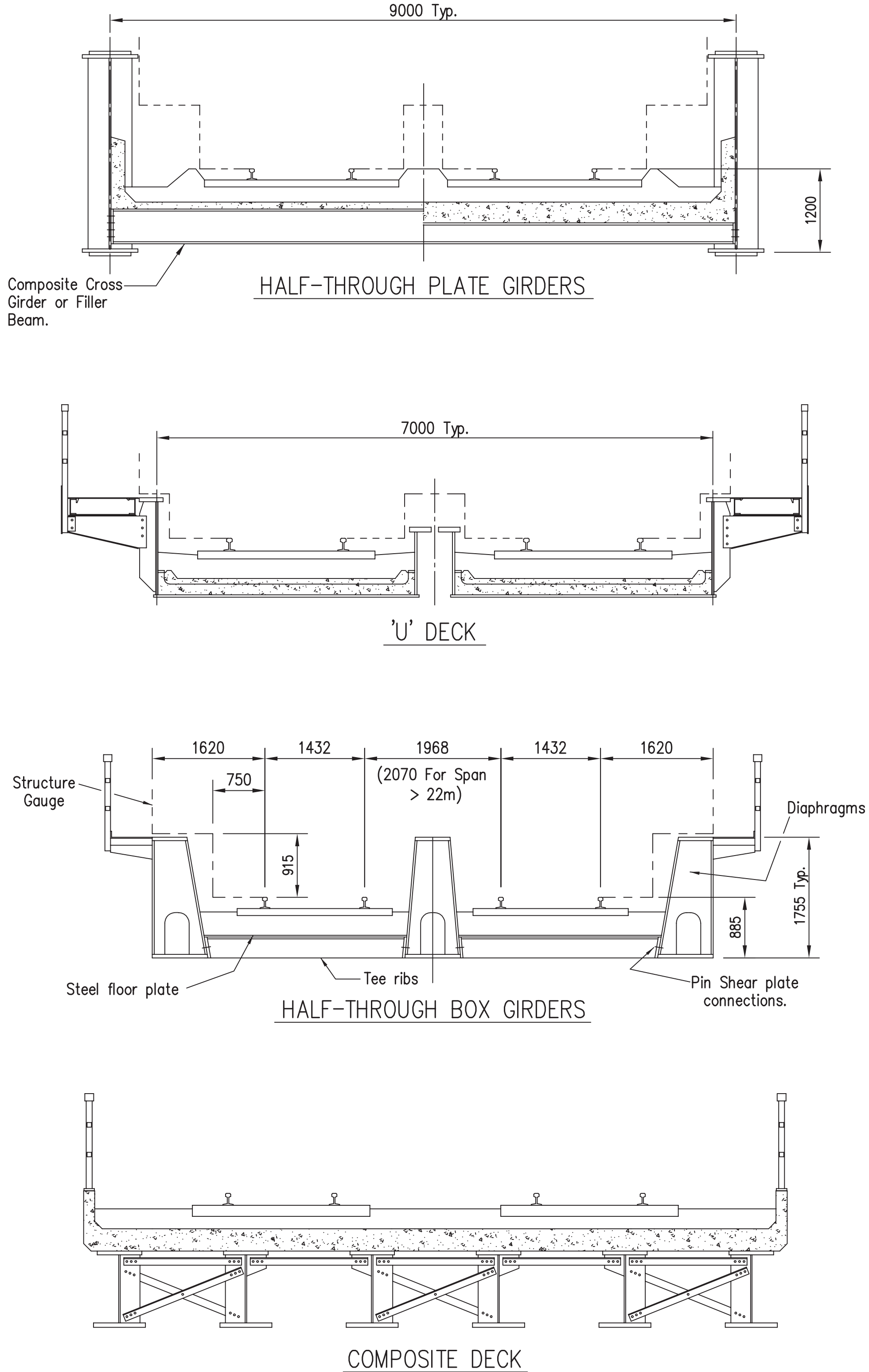
1.5 Cross sections for footbridges

The term 'footbridges' may generally be taken to include bridges which have been designed also for cyclists or equestrian users. Different requirements for parapets apply for these other users but otherwise similar cross sections are used. For most short and medium spans an all-steel configuration is usually selected. The advantage in using an all-steel configuration is that the whole cross section, including parapets, can be fabricated at the works for delivery and erection in complete spans; the weight of such spans is modest and easily handled by a mobile crane. For a more substantial deck, composite construction is sometimes used in a configuration somewhat similar to that for highway bridges

Half-through cross sections are strongly favoured because the very shallow construction depth minimises the length of staircase or ramp approaches. Steel plate or rolled section girders with a thin steel deck plate are used, but a very common solution is to use Warren trusses or Vierendeel girders fabricated from structural hollow sections. Some steelwork contractors specialise in design-and-build footbridges of this type. Full through construction may be used for longer spans with bracing above pedestrian level. Where such bridges form link access ways these may be fully clad, including the provision of a roof.

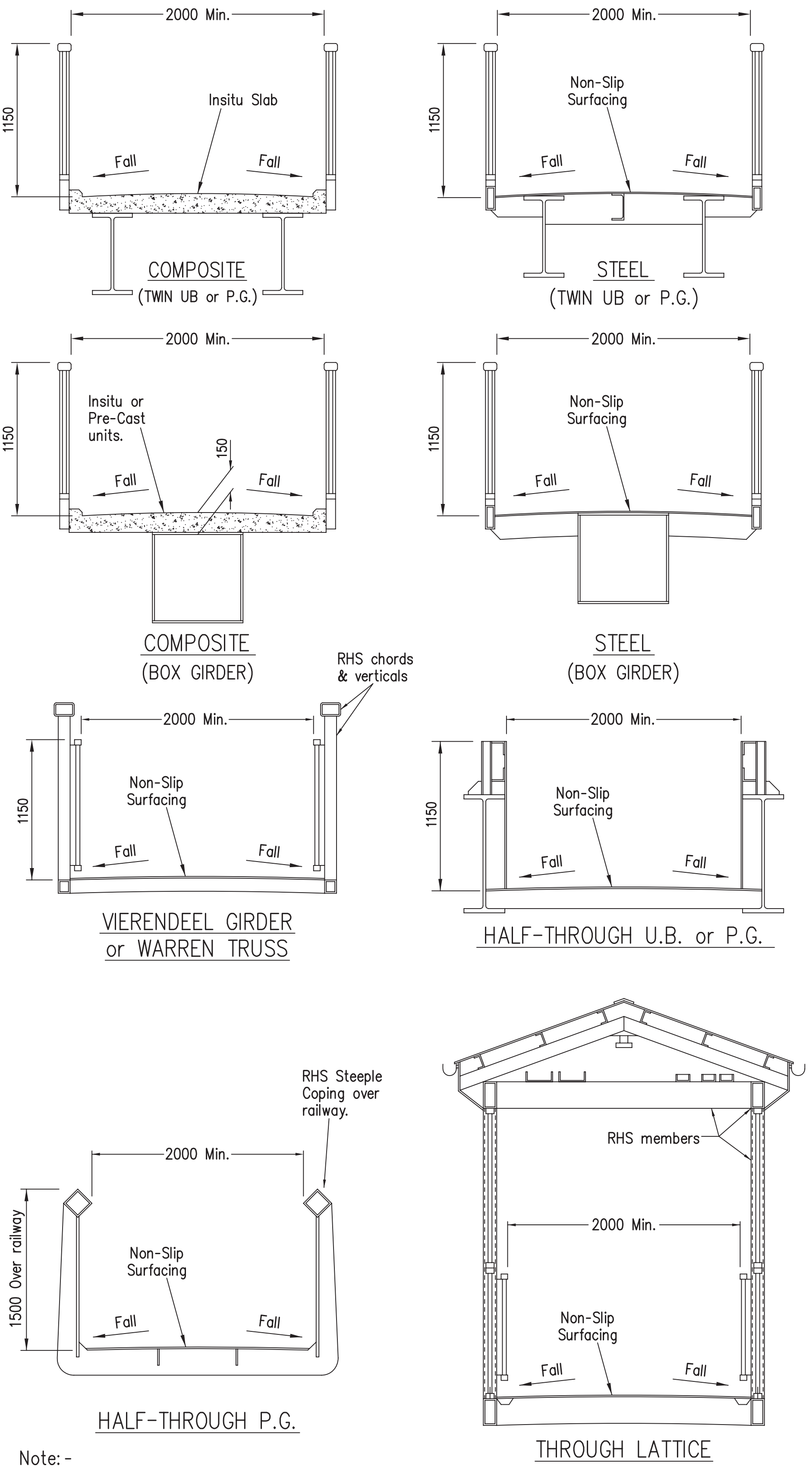
Arch-supported structures are a popular choice where a distinctive appearance is sought. For longer spans, cable-stayed solutions may be selected.

A range of footbridge cross sections is illustrated in Figure 3. General guidance on footbridge design is given in the Corus publication *The design of steel footbridges.*



SPAN RANGE (METRES)	REMARKS
20 TO 40	
8 TO 20	Use where consruction depth critical OR Minimum weight. E.G. RO-RO Link Span or Movable bridge.
21 TO 39	Use where construction depth critical. Network Rail standards for 2 boxes/track & 3 or 4 boxes per 2 tracks.
25 TO 50	Use where construction depth not critical.

Figure 2 - Railway bridge cross sections



Note:-
Parapets 1400 for cyclists
& 1800 for horses.

SPAN RANGE (METRES)	REMARKS
10 TO 50	Steel type may be erected complete.
20 TO 60	Steel type may be erected complete.
15 TO 60	Use where constructional depth critical.
15 TO 30	P.G.suited over rail
20 TO 60	Through suited to interconnect buildings etc...
	Half-Through P.G. used over railways where parapets must be impermeable with unclimable inside face,1500 Min. height.

Figure 3 - Footbridge cross sections

1.6 Bracing

1.6.1 Multi-girder decks

In service the only bracing that is usually needed for short and medium span deck-type multi-girder bridges is at supports and adjacent to intermediate supports (where the bottom flange is in compression). At the abutments the bracing is often combined with a trimmer beam supporting the end of the deck slab. With modern highway loadings, there is little benefit from utilising bracing to assist in transverse distribution of traffic loading in the design of wide decks.

During construction, bracing is usually necessary for stability of the girders during erection and during concreting of the deck slab. Mid-span regions are particularly susceptible to lateral torsional buckling before the concrete has hardened.

The designer should design the bracing for both construction and in-service requirements; usually these can be combined to provide an economical solution.

During construction the girders can often be erected (and sometimes delivered to site), as braced pairs to achieve mutual stability. Once erected braced pairs may need to be tied together to share wind loading in the construction condition. Plan bracing to stabilise girders during concreting should generally be avoided as its presence conflicts with other construction activities. Instead, the selection of top flange size and bracing should be such as to avoid the need for plan bracing (top flange width, for example, should be at least 400 mm). Bracing that is required only for the construction condition should nevertheless not be required to be removed on completion of the deck slab because that is a potentially hazardous, as well as costly, activity. However, for spans exceeding 60 m, use of full length plan bracing to the bottom flange may be needed for the in-service condition in some circumstances (to improve overall torsional stiffness) and advantage may be taken of its presence for the construction condition.

Figure 4 shows typical bracing layouts for multi-girder decks. and Figure 5 illustrates various types of bracing for use at supports and in the span. Where the headroom over a highway is substandard (less than 5.7 m) and vehicle impact loading must be considered, or where the bridge is curved in plan, additional bracing to restrain the bottom flange laterally will be needed. The spacing of such bracing depends on the width of the bottom flange and on the curvature.

1.6.2 Ladder decks

With a ladder deck configuration, the stiff connection of the cross girders to the main girders usually provides sufficient restraint (both during construction and in service) that additional bracing is not needed. In some cases, ‘knee bracing’ may be provided in regions adjacent to supports, though this is an extra complication and cost that is best avoided, if possible. Temporary plan bracing may be needed during launching bare steelwork for a ladder deck, as it has very little in-plan stiffness.

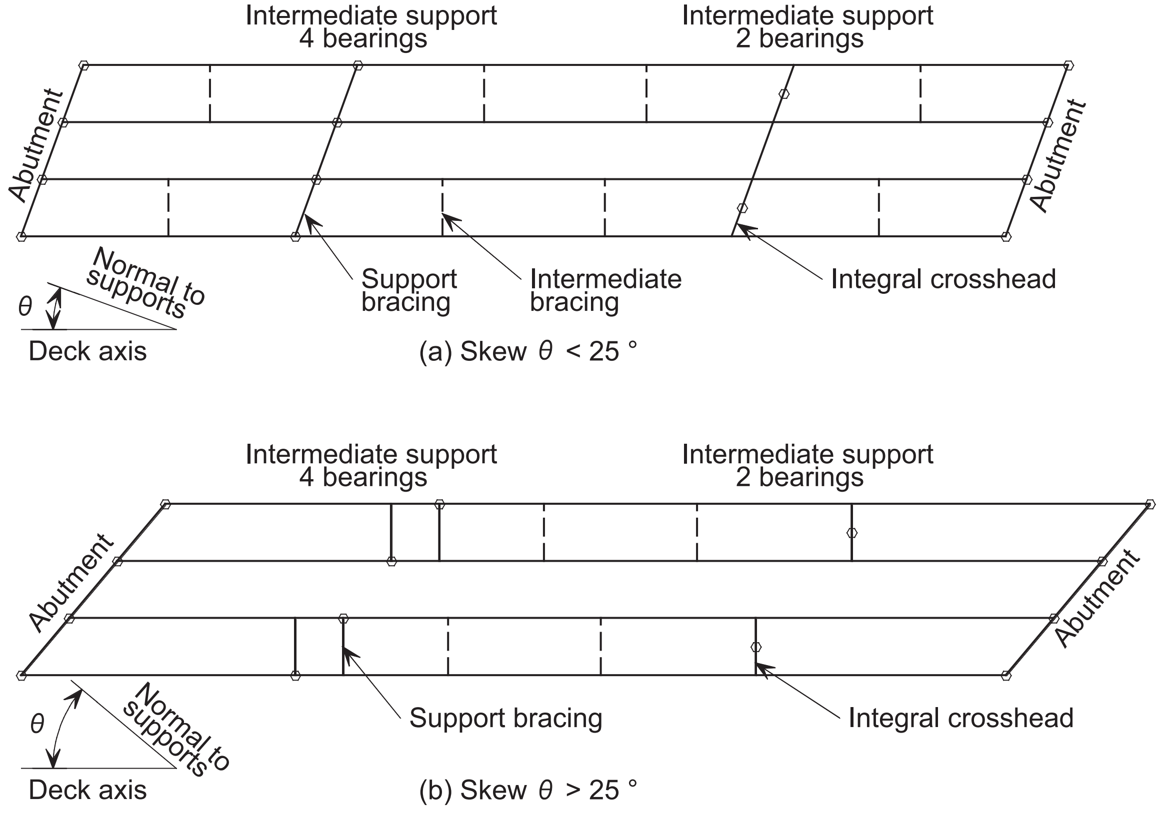
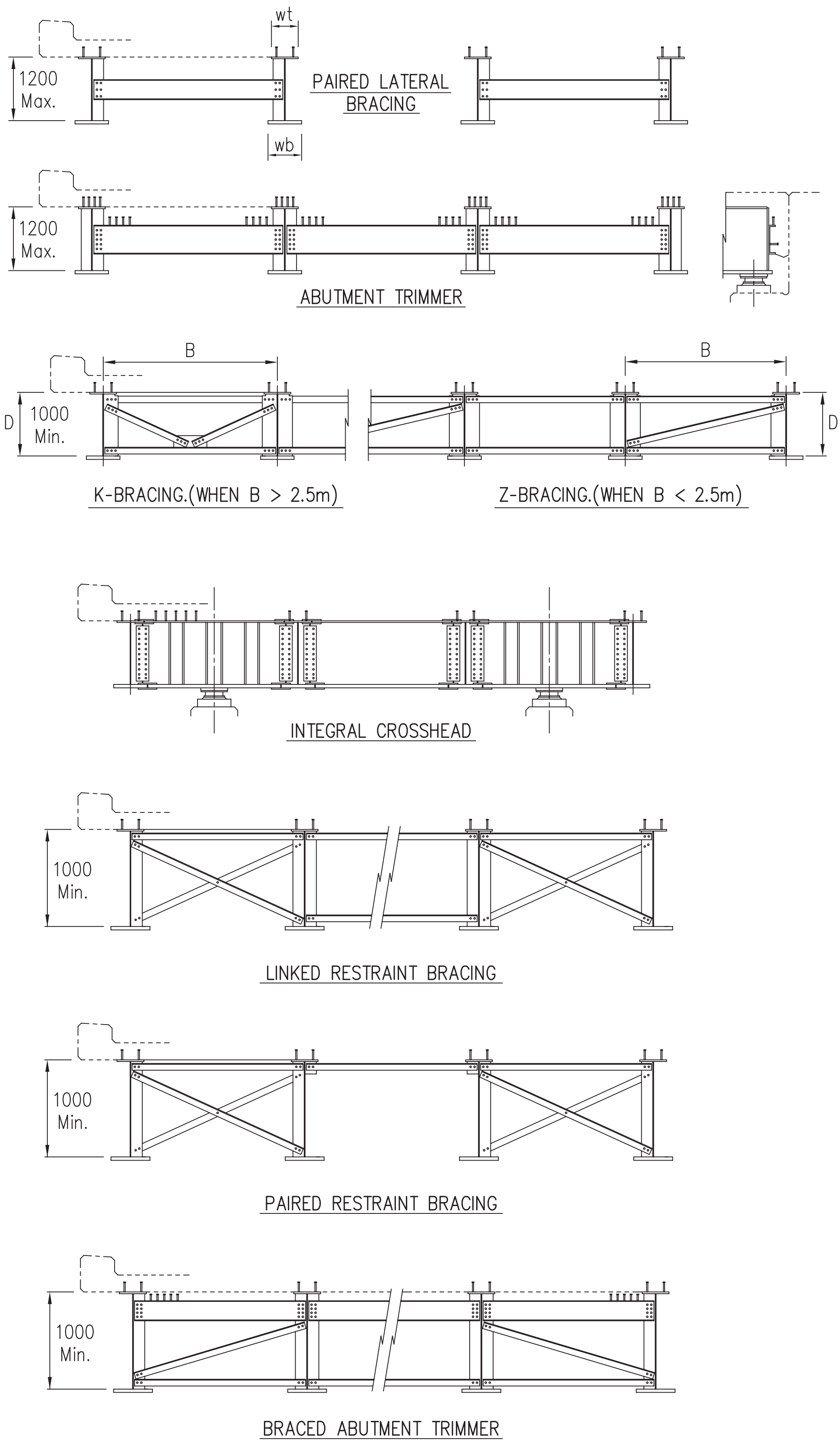


Figure 4 - Bracing layouts for multi-girder decks



TYPE	REMARKS
A	For Flange stability
	Ineffective for Load Distribution
B	For Girder stability & to stiffen Slab edge
C	For Flange stability
	Girders braced in pairs Ties between adjacent Girder pairs to maintain spacing during erection
D	Required where number of columns or width of leaf pier to be reduced —— Unjustified Otherwise
E	For Flange stability. Ensures all Girders act for Transverse Wind during erection.
	Ineffective for Load Distribution
F	For Flange stability
	Ineffective for Load Distribution
	Suits erection in pairs.
G	For Girder stability & to stiffen Slab edge

Figure 5 - Bracing types

1.7 Dimensional limitations

For road transport of fabricated items within the UK, the restrictions on size and weight given in Table 1 apply. These restrictions may have a considerable effect upon the member sizes and location of site splices.

Maximum height depends upon vehicle height and shape of the load; generally, items up to 3.0 m high can be transported. For rail transport, advice must be sought from Network Rail; items up to 3.0 m high, 2.9 m wide and 30.0 m long can generally be carried by arrangement, although transport of steelwork by rail is currently extremely rare.

In practice, girders up to about 50 m length have been fabricated and transported to site by road using a transporter with rear steerable bogies.

The steelwork contractor should be allowed flexibility in choosing to fabricate lengths above 30 m, provided the necessary permission to transport such loads can be obtained. Constraints at site or en route to site may restrict delivery dimensions or weight, so some flexibility to allow additional site splices may be required.

Except for very large projects, it is usually uneconomic for large members (such as box girders) to be assembled from individual elements on site. Generally, the longest possible members should be fabricated in order to achieve the minimum number of site joints. Erection costs are considerably influenced by unit weight, so crane sizes should be optimised wherever possible: the crane size is determined by piece weight and the radius for lifting, which is dictated by the site layout. A general guide is a maximum unit weight of 50 tonnes, although with modern cranes larger lifts can be achieved. In general, site splices and girder lengths should be chosen to facilitate erection and the appropriate erection method for the particular site.

1.8 Erection

With the facility of large capacity mobile cranes and the ability to deliver long components by road, the erection of most steel bridges can be carried out rapidly and economically without resort to temporary supports. However, the erection without temporary supports, coupled with imposition of wet concrete loads on a 'bare' steel structure, leads to significant stresses during construction: the stability of steel members at that stage is a significant issue. This, together with modern safety legislation, means the designer has to consider how the steelwork can be erected and the deck slab constructed safely. The designer is required to produce outline method statements, safety plans and risk assessments and the design and checking of steelwork strength and stability at all stages of construction is required to be rigorous and documented. The contractor is responsible for the detailed construction method statements and erection scheme as well as its implementation: the designer has to anticipate it properly and his assumptions have to be recorded and communicated.

Erection of short and medium span steel bridges is most commonly carried out using road mobile or tracked crawler cranes. Road mobile cranes require firm ground conditions to get on to site and, at the work positions, the crane platform needs to be properly designed to withstand both the overall crane weight and the pressure from its outriggers. The largest telescopic mobile cranes have the capacity to lift more than 50 tonnes at 50 m radius, or 100 tonnes at 35 m radius. Crawler cranes have flexibility in being able to travel with the load and, typically, can lift up to 100 tonnes at 50 m radius travelling. Hire costs and crane assembly periods for mobile cranes increase substantially for the larger cranes, which may be demanded for example where I-section girders are lifted in pairs, therefore it is advantageous if erection can be performed using the minimum number of lifts and crane positions. Crawler cranes are often not economic for short hire periods, but are usually more cost effective than road mobile cranes for long periods.

TABLE 1 - Road transport length/width restrictions

Method of Transport	Max Width (metres)	Max Rigid Length (metres)	Max Laden Weight (tonnes)	Max Axle Weight (tonnes)
Free movement	3.0	18.75	44	11.5
Police to be informed – escort required	5.0	30.0	150	16.5
VR1 Permit (at least 3 months notice)	6.1	30.0	150	16.5
Special Order (at least 3 months notice)	More than 6.1	More than 30.0	>150	–

Ground-supported temporary works should be avoided wherever possible, and personnel access assisted by use of mobile access platforms or cherry pickers. For heavy lifts, subject to more severe lifting conditions, cranes may be used in tandem. Mobile cranes are usually hired by the steelwork contractor specifically for the erection, so it is most economic if all the steelwork within a bridge can be erected in one continuous operation. Girders are generally lifted using double slings connected to temporary lifting lugs welded to the top flange. Sling lengths are usually selected commensurate with stability of the girder whilst being lifted, limiting the crane jib length needed and maximising the utilisation of the crane capacity.

For single spans, up to about 60 m in length, any splices, whether bolted or welded, will usually be made with the girders aligned on temporary stallages at ground level; each girder is then lifted complete. For continuous spans, a ‘span girder’ and a ‘pier girder’ will be spliced similarly at ground level, with erection proceeding span by span and over-sailing each pier to avoid the need for any ground-supported temporary works. For plate girders erected singly of length greater than about 40 m, stability considerations may demand use of bowstrings or other temporary props and restraints to stabilise the top flange against buckling once the crane is released. These erection methods are economic and of little hindrance to other site operations for they allow construction to advance rapidly without need for substantial ground-supported temporary works.

Erection by mobile cranes is generally the most economic method, provided that access and space is available for such cranes and for delivery of steelwork. Where temporary supports are required standard trestling is often used, with foundations using timber sleepers or concrete, depending on ground conditions. For long span bridges significant temporary works will usually be necessary. If welded splices need to be made in the final position, rather than at ground level, some form of temporary works and welding shelters with access for inspection, testing and protective treatment are necessary – this is considerably greater provision than for bolted connections.

Where erection has to be carried out during a limited period, such as in a railway possession or road closure, lifting of complete bridge spans is preferable, even though this increases the size or number of cranes. For railway bridge installation, rail-mounted cranes may be used but there are relatively few of these machines and they tend to have limited capacity. If they are being considered then it is essential that as much advance planning as possible is made as they still have to be brought to site via the railway itself. Where crane access is not feasible or overly costly, such as for a waterway crossing, then other erection methods including launching, sliding or rolling-in may be called for. Such schemes are less common today but, in the hands of competent steelwork contractors, are powerful ways of overcoming difficult obstacles or logistical constraints.

Launching is most suitable for new build highway or railway multiple continuous span bridges with constant height girders or trusses. Where possible, the steelwork is assembled full length behind one end of the bridge and launched forward on rollers or slide units mounted on tops of the permanent piers. A tapered launching nose is used to minimise stressing of the girders and remove the cantilever deflection of the leading end as it approaches each support. Propulsion may be by pulling with winches or strand jacks, or by incremental jacking. Once in position, the structure is jacked down onto the permanent bearings. Usually the steelwork alone is launched, followed by deck slab concreting in situ, but launching of concreted spans can be advantageous. Key considerations for the designer are: the stability of the girder web or truss bottom chord above the roller or slide units; the cambered shape of the girders; the interface of the girders with the temporary works (to suit the sliding or rolling action); and the space on top of the permanent supports (it must be large enough to accommodate the necessary jacks and restraints for jacking down as well as the permanent bearings).

Lateral sliding, rolling in or movement on multi-axle transporter units are used for bridge replacements. The whole structure is normally constructed and completed on temporary supports alongside the bridge before it is moved transversely and then jacked down onto its permanent bearings. The use of polytetrafluoroethylene (PTFE) and other low friction materials has made sliding-in increasingly favoured, because heavier loads can be carried. Propulsion is generally by strand jacks or incremental jacking, followed by jacking down onto permanent bearings. A combination of longitudinal launching and lateral sliding may be appropriate.

The erection of a steel bridge requires detailed consideration by the designer (to ensure that it is safe and practicable) and significant construction engineering on the part of the steelwork contractor (who will develop and implement the scheme that is actually used). It should be noted that fabrication and erection may be carried out by different steelwork contractors, but it is generally desirable that one steelwork contractor is responsible for both.

Site applied protective treatment is generally carried out after erection, and after deck concreting in the case of composite bridges; this work is usually sub-let by the steelwork contractor.

Further information about steel erection is given in the BCSA publication *Guide to the Erection of Steel Bridges*.

1.9 Repairs and upgrading

The enormous increase of highway traffic since the 1980s and the influence of heavier commercial vehicles has led to revised bridge loading standards. Many highway bridges have had to be strengthened and currently some still need to be upgraded to take full loading. Although railway loadings have not significantly changed, many rail bridges are well over 100 years old and are becoming life expired. Additionally, the planned introduction of higher train speeds (above 125 mph) has led to new design criteria to maintain passenger comfort levels and ballast stability; consequently some bridges require additional stiffness. A significant volume of current steel bridgework now involves repair and upgrading.

Steel is particularly suitable for strengthening by added material or duplicate members using bolted or site welding where appropriate. In the case of the M6 Thelwall Viaduct, a composite riveted plate girder structure strengthening was achieved by bolting on additional material and replacement of the existing deck using stronger concrete and additional shear connection. In other cases, strengthening has consisted merely of replacement of bearings and minor works. Some bridges originally designed non-compositely have been made composite by replacement of rivets by shear connectors.

Repairs to existing bridges have also been increasingly required due to accidental collision of vehicles with bridge soffits. Damage to steel members has often been found to be local, without fracture or risk of overall collapse, thanks to the ductile properties of steel. Several steel girder bridges have been repaired by heat straightening processes, as an alternative to costly girder replacement. The techniques were initially developed in the USA and have been further developed by UK steelwork contractors.

For further information refer to BCSA publication *The Use of Heat Straightening to Repair Impact Damaged Steel Bridges.*

Designers and steelwork contractors undertaking repair and strengthening works can face unusual challenges and hazards. It is often not clear exactly how an existing structure has been built and how it performs. Prior to starting any repair, technical issues such as procedures for welding to older materials must be resolved. There may also be safety hazards, i.e. working in confined space working or in the presence of potentially toxic lead-based paints and cadmium plating. Even the smallest repair project should be undertaken only by engineers and organisations with the relevant knowledge and expertise.

CHAPTER 2
STEEL QUALITIES

2.1 Introduction

The design rules in BS EN 1993-2 relate to design of bridge structures that are fabricated from materials (in the form of steel plate and rolled sections) to the standards listed in BS EN 1993-1-1. Those standards are BS EN 10025, BS EN 10210 and BS EN 10219; they cover a range of steel types, strength grades and toughness sub-grades. The execution standard, BS EN 1090-2, imposes some additional quality requirements but also leaves some options for the designer to make choices in the project specification. Different projects will almost certainly have different execution specifications, but the use of the Steel Bridge Group's *Model Project Specification* reflects best practice guidance and is recommended to ensure consistency across the industry.

The steel products are required to be supplied with a manufacturer's certificate and to have details of ladle analysis provided, so that the steelwork contractor can check the details for welding procedures.

2.2 Steel types

The majority of steel used on bridgeworks is non-alloy structural steel, sometimes called carbon steel or carbon-manganese steel. Such steel may be as-rolled, normalized, normalized rolled or thermomechanical rolled.

Steel cools as it is rolled, and the typical rolling finish temperature is 750°C, after which the steel cools naturally. Steel produced through this route is termed 'as-rolled'. Structural sections generally achieve the required mechanical properties through this efficient production route, but plates usually require further heat treatment. Normalizing is the process where an as-rolled plate is heated back up to approximately 900°C, and held at that temperature for a specific time, before being allowed to cool naturally. This process refines the grain size and improves the mechanical properties. Normalized-rolled is a process that achieves the same result by adopting a rolling finish temperature above 900°C, before the steel is allowed to cool naturally. Normalized and Normalized rolled steels are denoted 'N'. Thermomechanical rolled steel utilises a leaner chemistry, and has a lower rolling finish temperature before the steel cools naturally. Such steel is denoted 'M'.

Weldable fine grain steels are sometimes needed for thicker plates; they have improved toughness characteristics, resulting from the fine grain structure of the material. Such steels may be either Normalized/Normalized rolled or Thermomechanical rolled.

Other steels encountered on bridge projects include steel with improved atmospheric corrosion resistance, often called weathering steel (denoted 'W'), which is discussed in more detail below, and quenched and tempered steels (denoted 'Q'). However, these latter steels are rare in bridgeworks as they have yield strengths of 460N/mm^2 and above.

2.3 Strength grades

BS EN 1993-1-1 covers the use of grades up to S460. Higher grades are covered by additional rules in BS EN 1993-1-12.

In the UK strength grades S275 and S355 are readily available. S275 steel is often used on railway bridges, where stiffness rather than strength governs the design, or where fatigue is the critical design case. S355 steel is predominantly used in highway bridge applications, as it is readily available and generally gives the optimum balance between stiffness and strength and is more cost effective than S275.

S420 and S460 steels can offer advantages where self-weight is critical or the designer needs to minimise plate thicknesses. However, the use of such steels confers no benefits in applications where fatigue, stiffness or the instability of very slender members is the overriding design consideration. These steels are more expensive and are less readily available in the UK for plate thicknesses exceeding 50 mm. Grades above S460 are very rarely used.

Fabrication costs, and the cost of consumables, increase with use of higher strength grades, as the requisite welding procedures become increasingly more demanding.

2.4 Ductility

Ductility is an important property of steel that allows it to be fabricated and shaped by normal workshop practices. Requirements for steel bridges are given in BS EN 1993-2, and its associated UK National Annex: a minimum tensile strength not less than 1.1 times the nominal yield stress; a minimum elongation at failure of 15%. These requirements are automatically met by products complying with the referenced material standards

2.5 Brittle fracture and notch toughness requirements

The possibility of brittle fracture in steel structures is not confined to bridges for it has to be considered wherever stressed elements are used at low temperatures, especially in thick material where stresses are tensile, there are stress-raising details and the loading is applied rapidly. Steel having an appropriate notch toughness should be specified so that when stressed, and in the presence of a stress-raising detail, the steel will have a tendency to strain rather than fracture in a brittle manner. In fact, structural steelwork has a good history of very few brittle fracture failures. There are a number of contributory factors which help to prevent brittle fracture from occurring, e.g. the risk of brittle fracture is highest when the first high tensile stress coincides with a very low temperature; where the first high tensile stress occurs at a higher temperature, then some element of "proofing" takes place.

The standard Impact Test described in BS EN ISO 148-3 provides a measure of the notch toughness of steel. The test specimen is usually of square section 10 x 10 mm and 60 mm long with a notch across the centre of one side: it is supported horizontally at each end in the test machine and hit on the face behind the notch by a pendulum hammer with a long sharp striking edge. A pointer moved by the pendulum over a scale indicates the energy used in breaking the specimen, which is expressed as the Charpy energy in Joules at the temperature of testing. The resistance of steel reduces with temperature so the test is usually carried out at specific low temperatures, usually 0°C, -20°C, or -50°C.

The requirements for fracture toughness are described in BS EN 1993-1-10 and its associated UK National Annex. The procedure requires the calculation of a reference temperature (T_{Ed}), which is then used to determine a maximum permitted thickness for the steel part from a set of tabulated values. Published Document PD 6695-1-10 offers a Table that simplifies the process of choosing the appropriate sub-grade.

For a minimum bridge temperature of -20°C it gives limiting thicknesses according to the combination of:

- Steel material properties (yield strength and toughness)
- Member detail (type, stress concentrations etc.)
- Stress level (requirements are more severe in tension)

Additionally, adjustments are to be made where there is sudden loading or the element has been cold formed. An extract from Table 1 of PD 6695-1-10 is shown in Table 2 of this publication.

It should be noted that although cold formed hollow sections to BS EN 10219 can be supplied with a certified toughness (provided that the option in the standard is invoked), for rectangular hollow sections the toughness is only verified at the middle of the flat sides. The toughness is very significantly reduced in the corners by the cold forming process (the reduction in reference temperature given by BS EN 1993-1-10 can be as much as 99°C). Cold formed rectangular hollow sections are therefore unsuitable for bridges.

TABLE 2 - Maximum permitted thickness of steel grades for bridges at a minimum steel temperature of -20°C

Detail type		**Tensile stress level, $\sigma_{Ed}/f_y(t)$**									
Description	ΔT_{RD}	**Comb.1**	**Comb.2**	**Comb.3**	**Comb.4**	**Comb.5**	**Comb.6**	**Comb.7**	**Comb.8**	**Comb.9**	**Comb.10**
Plain material	+30°	≤ 0	0.15	0.3	≥ 0.5						
Bolted	+20°		≤ 0	0.15	0.3	≥ 0.5					
Welded - moderate	0°				≤ 0	0.15	0.3	≥ 0.5			
Welded - severe	-20°						≤ 0	0.15	0.3	≥ 0.5	
Welded - very severe	-30°							≤ 0	0.15	0.3	≥ 0.5
		Maximum thickness (mm) according to combination of stress level and detail type									
		Comb.1	**Comb.2**	**Comb.3**	**Comb.4**	**Comb.5**	**Comb.6**	**Comb.7**	**Comb.8**	**Comb.9**	**Comb.10**
Steel grade	**Subgrade**	External steelwork T_{md} = -20°C									
S275	JR	0	0	0	0	0	0	0	0	0	0
	J0	160	135	110	95	75	65	55	45	35	30
	J2	200	185	160	135	110	95	75	65	55	45
	M, N	200	200	185	160	135	110	95	75	65	55
	ML, NL	200	200	200	200	180	160	135	110	95	75
S355	JR	0	0	0	0	0	0	0	0	0	0
	J0	110	90	75	60	50	40	35	25	20	15
	J2	155	130	110	90	75	60	50	40	35	25
	K2, M, N	180	155	130	110	90	75	60	50	40	35
	ML, NL	200	200	185	155	130	110	90	75	60	50

Notes: Adjustments must be made where there is impact loading or where the steel is cold formed. See PD 6695-1-10.

2.6 Internal discontinuities in rolled steel products

Internal discontinuities are imperfections lying within the thickness of the steel product. These may be planar (laminar) imperfections or inclusion bands or clusters. Typically, laminar imperfections run parallel to the surface of a rolled steel product. They can originate from one of two main sources in the ingot or slab from which the plate or section is rolled but only very occasionally:

(a) entrapped non-metallic matter, such as steelmaking slag, refractories or other foreign bodies.

  Such material may not necessarily form a 'lamination', if the body fragments into smaller pieces on rolling, they may appear as discrete or clusters of 'inclusions'. The distinction between a lamination and an inclusion is purely arbitrary;

(b) a 'pipe'

  When an ingot solidifies, it may contain shrinkage cavities known as pipes. Providing they are not exposed to the atmosphere during reheating for subsequent rolling, pipe cavities generally weld up during the rolling process to give sound material. If a pipe is exposed, say by trimming off the head of the ingot, then its surface will oxidise during subsequent reheating and this will prevent the cavity from welding up. The resulting 'lamination', consisting largely of iron oxide, is then a plane of weakness. Ingots are very rarely used now and almost all steel produced in Europe today is by the continuous casting process, which eliminates this source of discontinuity.

Where a lamination is wholly within the body of a plate or section and is not excessively large, it will not impede the load-bearing capacity for stresses which are wholly parallel to the main axes of the member. This is the case for the majority of bridge steelwork details. However, laminations can cause problems if they are at or in close proximity to a complex welded joint where there are stresses through the thickness of the plate or section: thus a lamination could be a plane of weakness at a welded cruciform joint.

BS EN 10160 defines a number of acceptance classes for laminar imperfections: classes S0 to S3 refer to decreasing sizes and population densities of discontinuities anywhere in the main area or body of a plate; and classes E0 to E4 refer to decreasing sizes and numbers of imperfections near the edges of a plate.

BS EN 1090-2 requires internal discontinuity class S1 to be specified only for certain areas of plates (e.g. at welded cruciform joints) and gives the designer the facility to specify further areas requiring class S1. The Steel Bridge Group's *Guidance Note 3.06* provides guidance on appropriate areas where S1 is required (e.g. the webs at single-sided bearing stiffeners, and webs and flanges at welded bearing diaphragms). The *Model Project Specification* also advises that edge discontinuity class E1 should be specified for the edges of plates where corner welds will be made onto the surface of such plates.

Whilst BS EN 10160 refers to steel plate, it can also be applied to steel sections by special arrangement with the steel manufacturer. However, the ultrasonic testing of sections should really be specified to BS EN 10306.

A related phenomenon is lamellar tearing, which may occasionally occur at joints in thicker sections subject to through-the-thickness stresses under conditions of restraint during welding. This tearing results from a distribution of micro-inclusions that link up under high through-thickness stresses, resulting in a distinctive stepped internal crack. It is generally less common with modern steel-making practice, which produces lower average sulphur contents than were the case up to the 1970s. In practice it is unlikely that failure will occur in service, but full penetration welded cruciform joints should generally be avoided because the heat inputs from multi-run welding and back-gouging processes can cause a significant strain within the plate thickness, resulting in a tearing kind of failure. Cruciform welds should ideally have fillet welds or partial penetration welds, reinforced if necessary by fillet welds. If joint details are such that lamellar tearing may be a problem, then steel with improved through-thickness ductility (Z-grades to BS EN 10164) should be specified. Such steels are produced with ultra low sulphur levels.

2.7 Weldability

Procedures and precautions for welding have to be adopted to avoid cracking and to obtain adequate joint properties. The higher the carbon and alloy content of a steel, the harder and more brittle the heat affected zone near a weld becomes and the more susceptible it is to cracking. 'Carbon equivalent' formulae are widely used as empirical guides to 'weldability'.

The formula adopted in the standards is:

$$CE = C + \frac{Mn}{6} + \frac{Cr + Mo + V}{5} + \frac{Ni + Cu}{15}$$

To determine the carbon equivalent of a steel alloy, the chemical symbols in the formula are replaced by the percentages of the respective elements in the steel.

The CE value for most carbon and carbon-manganese steels is less than 0.47 and welding procedures for this level of weldability are relatively straightforward. For weathering steels, the level of alloy additions is generally higher, with a CE value sometimes ranging as high as 0.53, although developments in steel-making are generating significant improvements in weldability. Weathering steels up to 85 mm thick with CE typically of 0.44 (0.47 max) and thicker plates with a typical value of 0.5 (0.52 max) are readily available in the UK. Nevertheless long practical experience in the welding of weather resistant steels in North America and the UK has shown that they are readily weldable, provided that the normal precautions applicable to steel of their strength level and carbon equivalent are taken.

2.8 Weathering steels

Weathering steel is a term for high strength low alloy structural steels with improved atmospheric corrosion resistance, which were originally developed by American steel makers. The use of weathering steel in UK bridges has increased significantly since 2001 when a restriction on the use of the material over highways with less than 7.5 m headroom was removed. It is now the material of choice for a wide range of bridge decks.

The main advantage of a weathering steel bridge is that whilst the initial capital cost may be very similar to that of a conventional painted steel alternative (the extra capital costs may be offset by the elimination of the protective treatment), the structure requires virtually no maintenance, apart from occasional inspection. Consequently, the whole life costs of the structure may be greatly reduced because all the direct and indirect costs of periodic repainting are eliminated.

Examples of bridges constructed using weathering steels are shown in the Corus publication *Steel bridges – Material matters, Weathering steel* and general advice on selection and detailing is given in an ECCS publication *The Use of Weathering Steel in Bridges* and the Steel Bridge Group's *Guidance Note 1.07.*

2.8.1 Precautions to achieve good appearance

These steels owe their weathering resistance to the formation of a tightly adherent protective oxide film or 'patina', which seals the surface against further corrosion. The formation of the protective oxide film or patina is progressive with exposure to the atmosphere during and after construction; it gradually darkens with time and assumes a pleasing texture and colour, which ranges from dark brown to almost black, depending on conditions of exposure.

Plates are generally blast cleaned when they are taken in from the stockyard to remove millscale and clean them up prior to fabrication. However, for the best appearance, exposed surfaces should also be blast cleaned after fabrication, prior to delivery to site, to achieve a uniform appearance at the start of the weathering process. It is important that paint marks and contamination, especially by oil, are avoided because they cannot be entirely removed by blasting and would inhibit uniform patina formation. For most structures, only the very visible exposed faces may need the post-fabrication cleaning treatment, as it would serve little purpose to spend money cleaning the inner surface of a plate girder when that surface is not prominently seen. Hence, piece marks and erection marks, normally painted on members by the steelwork contractor, should be placed where they will not normally be visible.

Measures should be taken in design and construction to control run-off from weathering steel, especially during the initial weathering period, to avoid staining of adjacent materials. Drip flats should be provided on the steelwork, and supporting substructures should be protected with polythene sheeting during the construction period.

As with normal good design practice, crevices or ledges where water or debris can collect should be avoided, for these conditions may accelerate corrosion. In detailing, care should be taken to keep water run-off from expansion joints clear of the steelwork.

For most applications of weathering steels, the weathered appearance of the finished structure is one of its prime characteristics. It is important, therefore, that any welds that are exposed to public view should also, over a reasonably short period of time, weather in the same manner as the adjacent parent material. A wide range of electrodes is available with properties that are compatible with the parent steel.

It is not normally necessary to use electrodes with compatible weathering properties for small single pass welds, or for internal runs of multi-pass welds. In the former case, sufficient dilution of alloying elements from the base steel will give the weld metal a corrosion resistance and colouring similar to the base, whilst in the latter there is no need for the submerged runs to have such resistance. It is, of course, vital that electrodes with adequate mechanical properties are chosen.

2.8.2 Bolting weathering steel

To avoid any electrochemical reaction, it is important that the bolts, nuts and washers have a similar electrochemical potential to that of the steel structural member. Appropriate specifications for these would have chemical compositions complying with ASTM A325, Type 3, Grade A. Galvanized or electroplated bolts must not be used.

In a painted steel structure, bolted connections are protected from the ingress of water by the paint coatings. This is not the case, however, with a weathering steel structure and, as the connection is fully exposed, it is inevitable that at least some ingress of water to the interior joint surfaces can occur. This may be minimised with suitable bolt spacing, edge distances and sealing. Bolt spacing should not exceed 14t (maximum 175 mm) and edge distances should not be greater than 8t (maximum 125 mm). Such connections have performed successfully over many years of service.

The designer should limit the bolts on a weathering steel structure to size M24 for availability reasons. For a structure using 100 tonnes of weathering steel, typically only 1 to 1.5 tonnes of bolts are required. This is far too small to justify a special rolling of bar in the steel mill, and the production of nuts and bolts is dependent on the availability of the right sizes of bar kept in stock by the fastener manufacturers or steel stockholders dealing in weathering steels. It is advantageous in design using M24 bolts to adopt a bolt spacing appropriate to 1" (25.4 mm) diameter bolts (i.e. minimum spacing 65 mm) so as to permit procurement of bolts from the USA if necessary.

2.8.3 Availability of weathering steel

Weathering steel plates are readily available from the mill.

Rolled sections are not commercially available in weathering steel. This does not pose any problem for supply of main girders, as they can be fabricated from plate (even with ordinary structural steel, rolled sections are rarely used). However, angles, channels and hollow sections are often used for bracing elements on ordinary structural steel bridges, so for weathering steel bridges, alternatives must be considered.

Current industry advice on this issue is as follows:

Ladder decks
The nature of such bridges is that they only require bracing at intermediate supports. 'Knee bracing' using short lengths of angle sections is sometimes used, but the most economic solution is the use of a deep fabricated I-section cross girder.

Multi-girder decks
Avoid the use of 'X' or 'K' bracing and adopt fabricated I-section girders as stiff transverse beams in an 'H' configuration with the two main girders. However, for deep girders, angles or channels fabricated from plate may be required to form a triangulated bracing system.

Steelwork contractors have a wealth of knowledge and experience of the use of weathering steel and are pleased to assist in design development. Early discussions with steelwork contractors are recommended to achieve the most economic weathering steel bridge solution for a particular project.

2.8.4 Interfaces with other materials

Concrete
Interfaces between steel and concrete should be sealed with an appropriate sealant to prevent the ingress of moisture. Examples where this is needed include the edges of the top flange and the profile of a beam that is cast into the end wall of an integral bridge. Elements encased in concrete need not be painted, provided there is sufficient depth of concrete cover for the required durability.

Metals
Avoid connections to galvanically dissimilar metallic components, otherwise there is a risk of accelerated local corrosion. For the majority of weathering steel bridges, the only area to consider in terms of this effect is the connection between the steel girders and the structural bearings. (As mentioned above, zinc-plated bolts should be avoided, for the same reason.) For the case of weathering steel girders on ordinary structural steel bearings, there is no significant difference between the reactivity of the two metals and as the ordinary structural steel (of the bearings) is painted, there is no contact with an electrolyte. Hence, bimetallic corrosion is unlikely to occur.

2.9 Availability of plate material

The availability of plate lengths in different widths is a function of the rolling process: Table 3 shows readily available plate lengths in th UK. The generally available maximum sizes of plate dimensions should be adopted as a guide in design; however, it is often possible to obtain larger sizes, subject to consultation with the rolling mills, at a cost premium. Butt welding shorter plates can be more cost effective than specifying a very large single plate.

It should be noted that high premiums are paid on steel orders (of one size and one quality for delivery from one works at one time to one destination) of low quantities, typically less than 5 tonnes. Therefore, designers should try to standardise on material sizes to avoid cost penalties. Where small tonnages occur (say for gussets and packs) it is often more convenient to obtain material from a stockholder, even though costs are greater; normally only grades S275 and S355 are available, so other grades for such small items should be avoided.

TABLE 3 - Readily available plate lengths in the UK

Plate Gauge (mm)	Plate Width (mm)									
	>1250 ≤ 1500	>1500 ≤ 1750	>1750 ≤ 2000	>2000 ≤ 2250	>2250 ≤ 2500	>2500 ≤ 2750	>2750 ≤ 3000	>3000 ≤ 3250	>3250 ≤ 3500	>3500 ≤ 3750
10	18.3	18.3	18.3	18.3	18.3	18.3	18.3	18.3	15.5	
15	18.3	18.3	18.3	18.3	18.3	18.3	18.3	18.3	15.0	
20	18.3	18.3	18.3	18.3	18.3	18.3	18.3	18.3	15.0	
25	18.3	18.3	18.3	18.3	18.3	18.3	18.3	18.3	18.3	15.0
30	18.3	18.3	18.3	18.3	18.3	18.3	18.3	18.3	17.8	15.0
35	18.3	18.3	18.3	18.3	18.3	18.3	17.8	16.4	15.2	14.2
40	18.3	18.3	18.3	18.3	18.3	17.0	15.6	14.4	13.3	12.4
45	18.3	18.3	18.3	18.3	16.6	15.1	13.8	12.8	11.8	11.1
50	18.3	16.7	18.3	16.6	14.9	13.6	12.4	11.5	10.7	9.9
55	18.3	15.2	17.0	15.1	13.6	12.3	11.3	10.4	9.7	9.0
60	17.4	13.9	15.6	13.8	12.4	11.3	10.4	9.6	8.9	8.3
65	17.0	12.8	14.4	12.8	11.5	10.4	9.6	8.8	8.2	7.6
70	17.0	11.9	13.3	11.8	10.7	9.7	8.9	8.2	7.6	7.1
75	17.0	11.1	12.4	11.0	9.9	9.0	8.3	7.6	7.1	6.6
80	17.0	10.4	11.7	10.4	9.3	8.5	7.8	7.2	6.6	6.2
90	15.6	9.2	10.4	9.2	8.3	7.5	6.9	6.4	5.9	5.5
100	14.0	8.3	9.3	8.3	7.4	6.8	6.2	5.7	5.3	4.9

Notes:

1. Intermediate gauges are available.
2. Plates may be available in longer lengths by arrangement.
3. For plates > 100 mm thick and/or > 14.5 tonnes contact supplier to confirm availability.
4. Plates > 3750 mm wide may be available by arrangement.
5. For precise details of plate availability, contact supplier.

CHAPTER 3
DESIGN OF MEMBERS

3.1 Introduction

This chapter deals with the various practical aspects of the design and selection of the appropriate form of the principal structural members in a steel or composite construction bridge. For general guidance on the design of bridges, including guidance on the design of highway bridges in accordance with the Eurocodes, see the references in Chapter 10.

3.2 Rolled section girders

Where rolled UKB/UB sections are selected, the fabrication work normally involves cutting to length, welding of bearing stiffeners (discussed in more detail below for plate girders), and welding of shear connectors. As a characteristic of the rolling process, most deep rolled beams have web thicknesses greater than needed for structural purposes. Consequently, web stiffeners are rarely required. However, where they are needed, web stiffeners should not be butt welded to the webs of rolled sections because of the risk of lamellar tearing. If precamber is needed (for vertical profile or to offset permanent deformations) this is achieved by cold bending, which adds to fabrication cost.

Rolled sections are often selected for cross girders in railway bridges. Then the fabrication work often includes endplate details that require significant work and close dimensional control.

3.3 Plate girders

3.3.1 Piece sizes

With a plate girder, the designer is free to choose the dimensions of each flange and the web, Top flanges are usually chosen to be as wide as possible, to improve their lateral stability during construction (this minimises the need for bracing of the compression flange). Similarly, bottom flanges in regions adjacent to intermediate supports are usually chosen to be as wide as possible. Webs are usually chosen to be relatively thin because they are not efficient in contributing to bending resistance. Consequently, webs are often provided with transverse stiffeners in regions of high shear.

BS EN 1993-1-1 gives limits on outstand ratios for parts in compression, for both Class 2 and Class 3 cross sections (Class 2 are designed to plastic bending resistance, Class 3 to elastic bending resistance). Limiting sizes for a range of flange thickness in Class 3 sections are given in Table 4. The limiting ratio for Class 2 sections is approximately 70% of that for Class 3 sections. No limits are given in BS EN 1993-1-1 for tension flange outstands but it is good practice to limit outstands to 20 times the thickness to ensure suitably robust construction.

Flange thicknesses limited to 75 mm are generally advisable to avoid weld procedures needing excessive preheat. Flanges up to 100 mm thickness or more can be used, subject to selection of a suitably high notch toughness and availability.

Although thin webs can be stiffened to improve their shear resistance, welding such stiffeners is an added fabrication cost and there is an economic balance between the thickness of the web and the amount of stiffening. The temptation to use a thin web with many transverse stiffeners should be resisted, as it is likely to be more expensive. Apart from where stiffeners need to be provided for the attachment of bracing or cross girders, the usual practice is for intermediate stiffeners to be not closer than about three times the web depth in low shear regions and not closer than about 1.25 times the web depth in high shear regions.

TABLE 4 - Compression outstand limit to flanges (BS EN 1993-1-1 Clause 5.5.2)

Strength Grade		Flange Thickness (mm)												
		15	20	25	30	35	40	45	50	55	60	65	70	75
S275	Outstand limit (mm)	193	258	322	386	451	515	580	644	708	773	837	902	966
	Typical flange width (mm)	400	500	650	750	900	1050	1150	1300	1400	1550	1650	1800	1950
S355	Outstand limit (mm)	170	227	284	340	397	454	510	567	624	680	737	794	851
	Typical flange width (mm)	350	450	550	700	800	900	1000	1150	1250	1350	1450	1600	1700
S460	Outstand limit (mm)	149	199	249	298	348	398	447	497	547	596	646	696	746
	Typical flange width (mm)	300	400	500	600	700	800	900	1000	1100	1200	1300	1400	1500

Notes:

The above limits are based on 14ε times the thickness, where the value of ε is based on the yield strength of products up to 16 mm thick. Slightly higher limits would apply if the lower strength of thicker parts were taken into account.

Webs rarely need longitudinal web stiffeners although they may be appropriate in the support region of deep long-span girders. Such stiffeners add considerable cost and complexity to the fabrication.

For all but the shortest spans, designers will wish to vary flange size and web thickness along the length of the bridge for economy. Coupled with the practical limitations on plate lengths, this means that butt welds will be needed in the flange and web plates. However, the number of plate changes (and thus butt welds) should be the minimum commensurate with plate availability and economy, because of the high cost of butt welds. Figure 6 offers a guide as to where it is economic to change plate thicknesses within girder lengths. The Figure is typical for flanges or webs of medium size plate girders. The relative economy may vary in individual cases depending upon the welding processes used and general fabrication methods.

It is usual to cut flange and web plates in multiple widths from wide plates supplied from the rolling mill. The optimisation of 'nesting' multiple pieces within a single plate may influence the economy in selecting butt weld positions.

Changes in flange thickness should ideally occur at the inner faces, allowing a constant overall girder depth. This avoids conflicts with slab details, eases assembly using T and I machines, and provides a smooth bottom flange for girders that are to be launched. Changes in web thickness usually occur on both faces. Where a change of thickness occurs at a butt weld, the thicker plate is tapered in thickness at 1 in 4 adjacent to the weld, if the step is more than 2 mm.

A guide to choosing the make-up of I-section plate girders for a multi-girder highway bridge is shown in Figure 7 for span ranges up to 70 m. The guidance takes account of positioning site splices, available material lengths and the relative cost of making shop butt welds.

The Figure gives an indication of the minimum length (L) for which a selected thickness change will be economic for flanges and webs of girders. Below this length it will be more economic to continue the thicker plate (t1). Please note that the costs/metre of weld used to derive the figure below do not include the costs associated with the grinding of butts.

$$L\ (m) = \frac{r \times 10^3}{7.85t/m^3\,(t1 - t2)} \qquad \text{where } r = \frac{\text{Cost/m weld}}{\text{Cost/tonne of steel}}$$

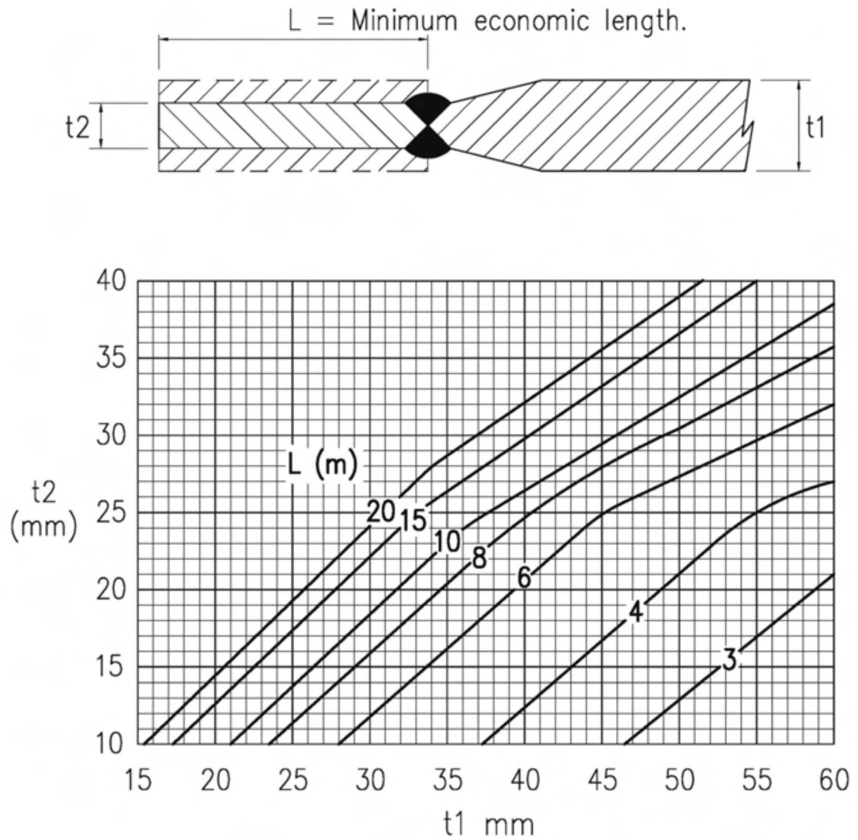
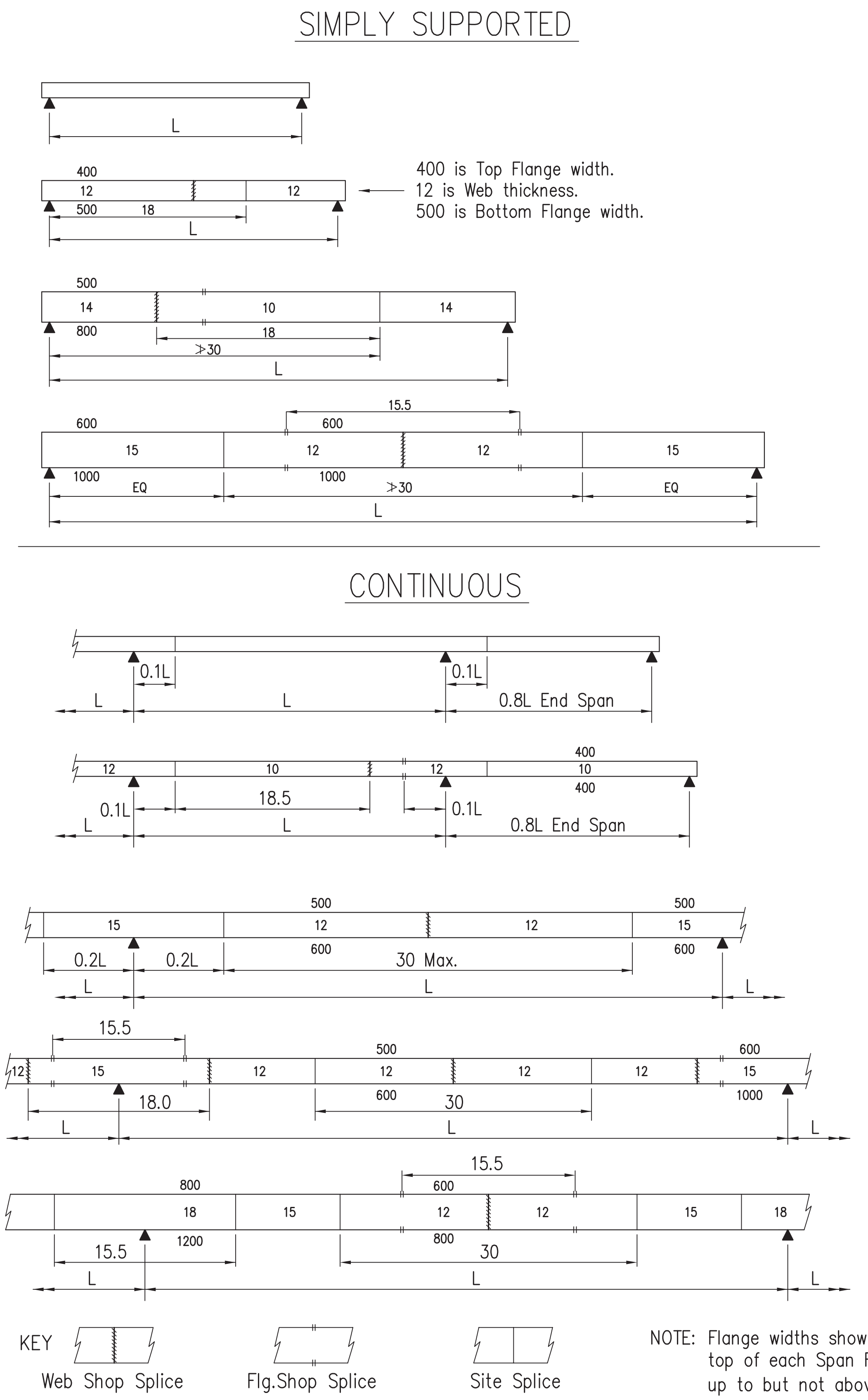


Figure 6 - Economy of flange/web thickness changes



SPAN RANGE L(m)	GIRDER DEPTH (m)
≯30	1.0
30 to 40	1.0 to 1.6
40 to 60	1.6 to 2.7
SPAN RANGE L(m)	**GIRDER DEPTH (m)**
≯25	to 1.0
25 to 30	1.0
30 to 45	1.8
45 to 52	1.8 to 2.2
52 to 70	2.2 to 3.3

NOTE: Flange widths shown are for top of each Span Range & girders up to but not above 3.5m c/c.

Figure 7 - Guide to girder make-up for a composite highway bridge

3.3.2 Detailing and fitting of stiffeners

In design and detailing of web stiffeners for plate girders, the welding should be carefully considered for both bearing stiffeners and intermediate stiffeners. Where stiffeners are closely spaced, access for welding must be considered. A reasonable rule is that the space between two elements should be at least equal to their depth. Thus, for example, two stiffeners 150 mm wide should be separated by at least 150 mm.

Bearing and jacking stiffeners should be attached to both flanges using fillet welds. These stiffeners should also be 'fitted' to the loaded flange (usually the bottom flange) to ensure sufficient bearing contact is made between the stiffener and the flange. The stiffeners must be forced against the bearing flange before they are tack welded to the girder web. The welding sequence is chosen such that the tendency is always to compress the fitted end of the stiffener against the loaded flange; the fillet welds to the unloaded flange are therefore left until last. Fitting stiffeners to both flanges should be avoided as it is very expensive and is rarely necessary in bridges. At the bottom flange, it may be necessary to use heavy fillet welds or partial penetration welds because direct bearing should not be assumed in fatigue checks. However, full penetration welds should be avoided because distortion of the flange may be significant giving problems in fit of bearings. For stiffeners up to 10 mm thick, say in a footbridge, the bearing load can be carried from the flange into the stiffener by the fillet welds; this avoids fitting such small stiffeners to the flange.

Intermediate web stiffeners do not act in bearing, so they need not be fitted. They are usually fillet welded to the top flange but do not need to be welded to the bottom flange unless they are transferring forces from bracing or cross girder flanges. All stiffeners should be detailed to enable the weld connections to be continuous around the edges of the stiffener and flange.

Stiffeners should be cut to clear web to flange welds. For painted bridges, this can be achieved using a close fitting snipe, which is then welded over the web to flange weld. This detail simplifies the welding, avoids the need to apply the protective treatment system to the backs of cope holes which are difficult to access properly, and makes for easier long-term maintenance. For weathering steel bridges, the issues concerning the protective treatment system do not apply, and it is better to provide cope holes to ensure adequate drainage along the girder length and to prevent the collection of water on the bottom flanges at stiffener locations. The size of cope hole will vary with stiffener thickness, with a minimum of 40 mm, increasing to 50 mm for stiffeners between 35 mm and 50 mm thick. The size of cope hole may need to be greater if stiffeners are skewed.

All details should be designed with a view to simplicity and minimum number of pieces to be welded or connected together. For example, where stiffeners need to be shaped so as to connect to bracings they should be cut from a single plate rather than being formed from separate rectangular pieces. All re-entrant corners should be radiused at 20 mm or 1.25 t minimum radius. Site connection details should provide angular and length tolerance, bearing in mind camber prediction (see Section 3.9) and variation between adjacent girders.

Bracings are normally of rolled angle section, connected by preloaded bolts via one leg to web stiffeners. In detailing clearance gaps, a 30 mm minimum clearance between the ends and the face of the web should be allowed for the stiffener welds and painting. It is good practice to dimension bolthole end distances 5 mm more than values assumed in design of the connection.

Figure 8 shows some detailing DOs and DON'Ts and the principles illustrated should be borne in mind throughout the design and detailing process. Some cusp distortion of flanges may occur due to the web to flange welds. However, this normally does not matter from a strength point of view, although care must be taken during fabrication at the location of the bearings to ensure proper fit-up and, for this reason, a separate bearing plate welded to the girder is desirable to ensure a flat interface with the bearing.

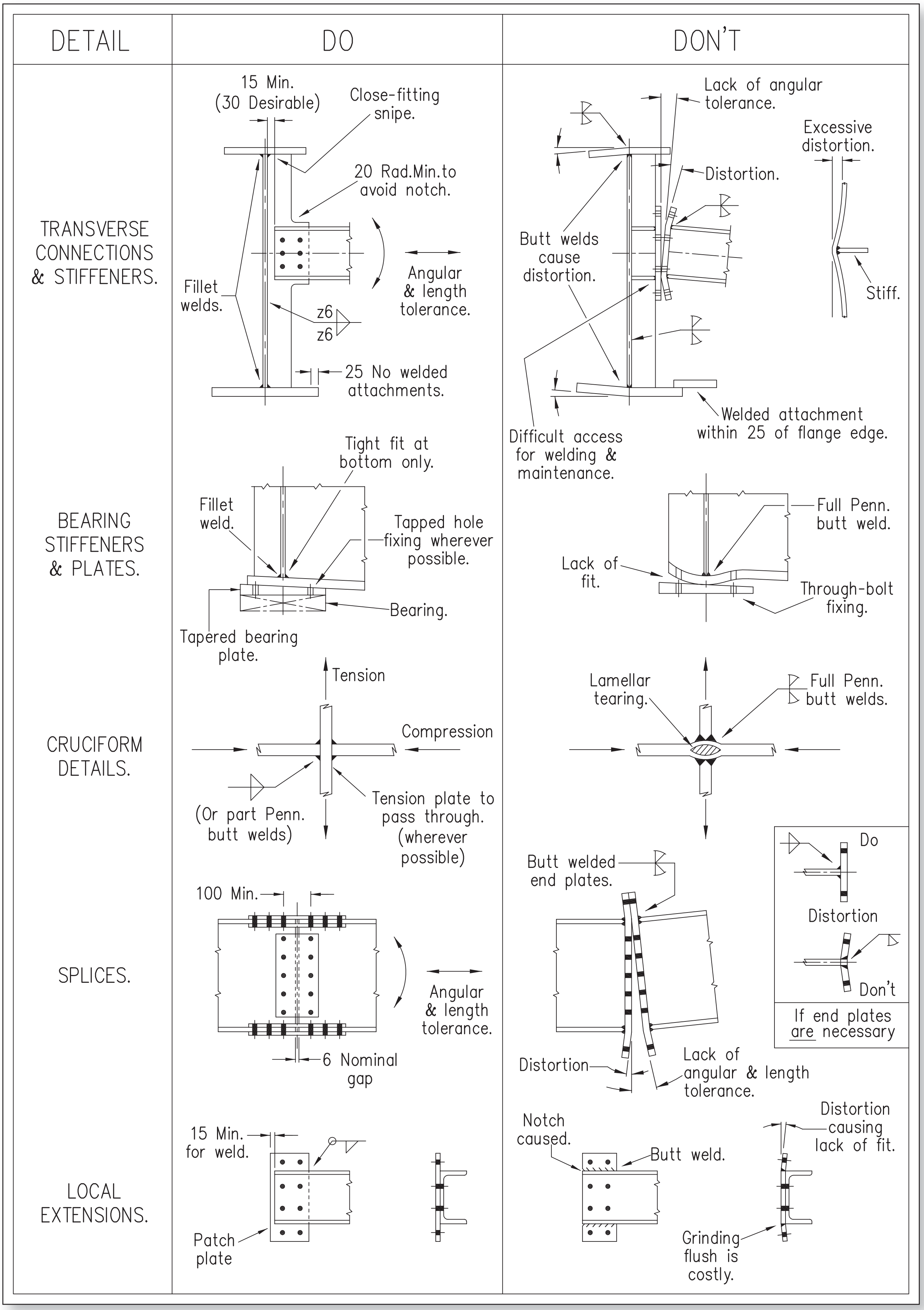


Figure 8 - Detailing DOs and DON'Ts

3.3.3 Fabrication sequence

The steelwork contractor usually butt welds the flanges and web plates to full length in the shop before assembly of the girder. This means that such shop joints can be in different positions in the two flanges and do not have to line up with any shop joint in the web.

The fabrication process for plate girders varies between steelwork contractors depending on how extensively their workshops are equipped, from basic facilities up to substantial automation. The alternatives are described in the following common sequence for plate girder preparation, assembly and finishing, and are illustrated in Figure 9.

(a) Butt weld plates for flanges and webs into longer lengths as required. If possible, the full width of plate supplied (from which several components will be cut) will be butt welded, as this will reduce the number of run-on/run-off plates and minimise butt weld testing requirements.

(b) Mark ends and machine flame cut or plasma cut flanges to width and length. If the girder is curved in plan then the flange will need to be profiled to the appropriate shape. If it is possible with the equipment available, drill boltholes for flange connections and mark stiffener positions. If not, these will need to be hand marked and drilled later.

(c) Machine flame cut or plasma cut webs to profile and camber, including any fabrication precamber. If it is possible with the equipment available, drill boltholes for web connections and mark any longitudinal stiffener positions. If not, these will need to be hand marked and drilled later.

(d) Machine flame cut or plasma cut stiffeners to shape. If it is possible with the equipment available, drill boltholes for bracing connections and mark any plan bracing cleat positions. If not, these will need to be hand marked and drilled later.

(e) Assemble the girder

Either…

Assemble the flanges to the web using tack welds, semi-automatically weld the flanges to the web, rotating the girder as necessary to minimise distortion and to obtain the necessary welding position. (This method is used for very short girders, such as diaphragms, and highly shaped girders.)

Or…

If T and I automatic girder assembly equipment is available, automatically weld the web to the first flange to form a T section as they pass through the machine. Turn the T section over, assemble on top of the second flange and weld the web to the flange, forming the I-section.

Or…

If automatic girder assembly equipment is available, place the web horizontally between the two flanges and clamp the flanges onto the web. Automatically weld the two uppermost web-flange welds before turning the girder to weld the other side.

(f) Tack in stiffeners (marking positions first, if necessary), then fillet weld in place using manual, semi-automatic or automatic (robotic) welding procedures as available and appropriate.

(g) Check compliance with specification tolerances for the fabricated girder.

(h) Trial erect steelwork, if required. Trial erection is a very expensive process and, with modern fabrication techniques, should only be necessary if the implications of a minor problem on a single connection would be so critical as to justify the expense. For example, any section of bridge being erected in a motorway or railway possession would normally be trial erected and this should be specified by the designer. Full trial erections should not normally be specified for 'green field' erection, but with some designs it may be advisable to fit some components before dispatch from the works. Seek advice from steelwork contractors if trial erection is to be considered. (See further comments in Sections 6.9 and 7.7.)

(i) Blast steelwork. Apply protective treatment to specification, if required.

(j) Assemble braced pairs or longer lengths prior to delivery to site, depending on erection method and site access. This can sometimes be combined with trial erection to minimise cost.

3.4 Girder splices

Site splices between girder sections are usually bolted, although on larger jobs, where the cost of the additional control and protection measures can be justified, welded splices may be used. Examples of bolted and welded splice details are shown in Figure 10.

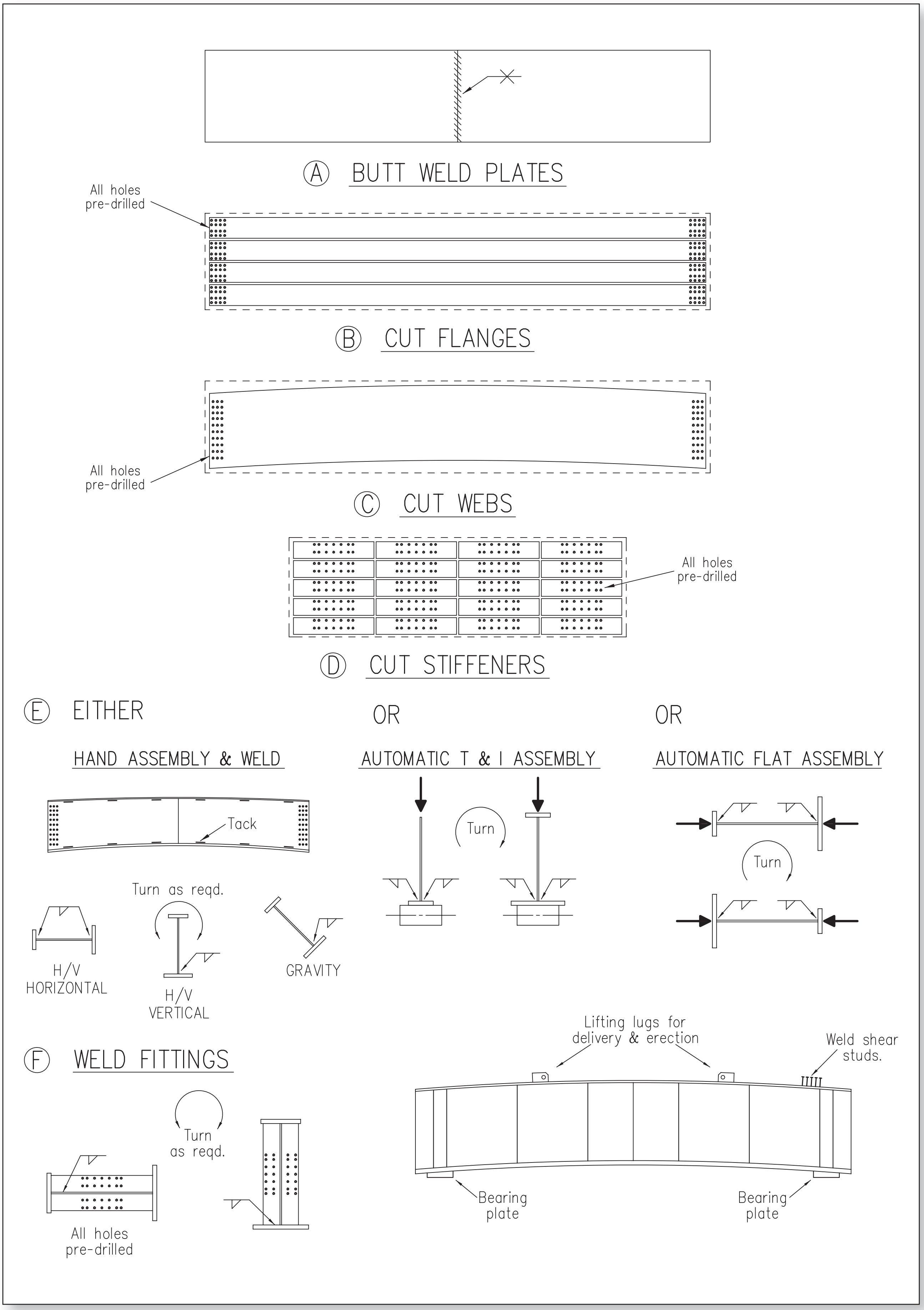


Figure 9 - Plate girder assembly

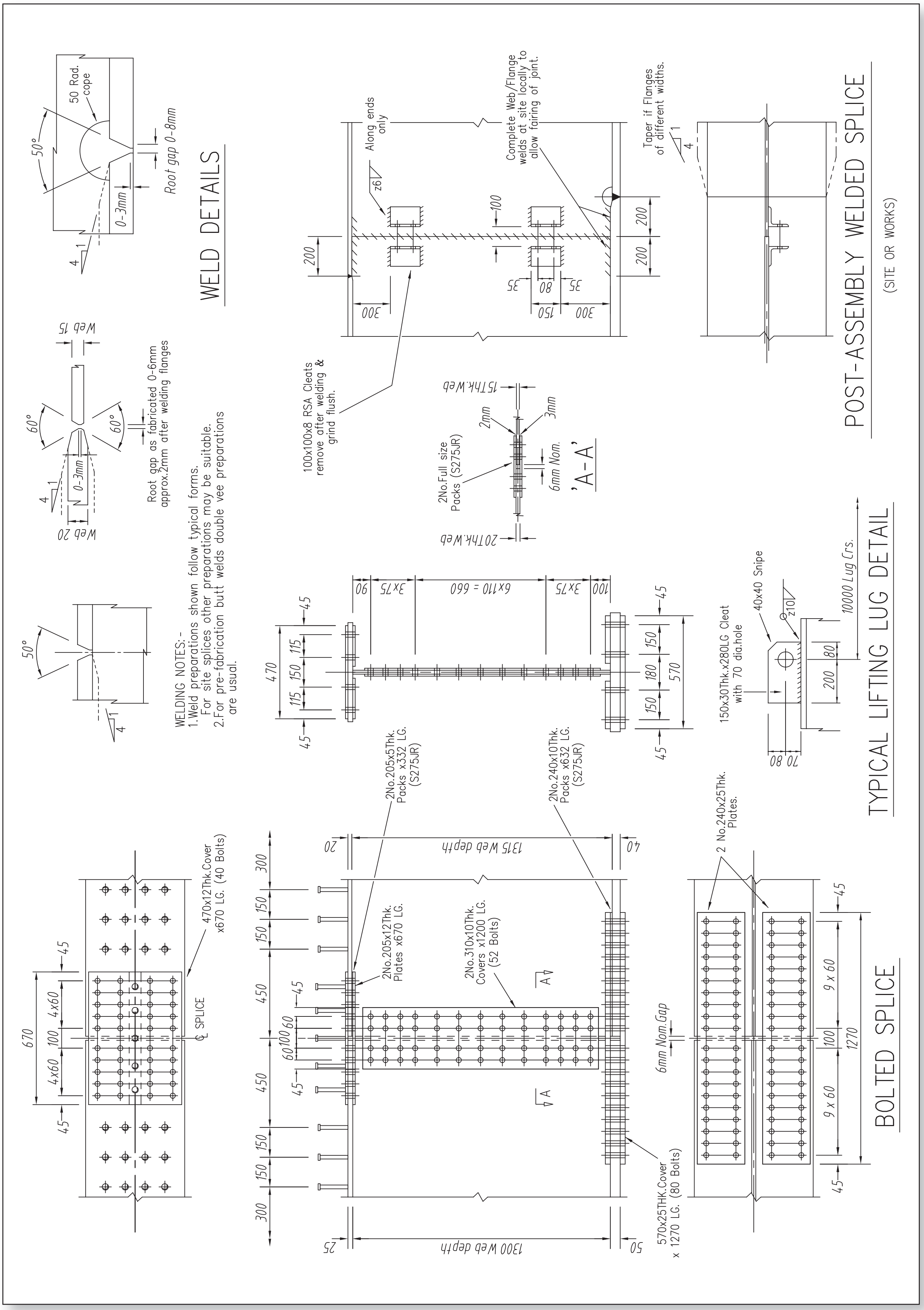


Figure 10 - Typical bolted and welded splices

3.5 Typical plate girder

A typical plate girder for a single span bridge is shown in Figure 11. Because the span exceeds 30 m, an optional splice is shown towards one end. Suitable bolted or welded splices are shown in Figure 10. In cases where the steelwork contractor can deliver the girders to site in one length, then the site splice would be unnecessary.

Figure 11 illustrates the make-up, the stiffeners, the bracing and the abutment details.

3.6 Trusses and Vierendeel girders

A through or half-through truss offers a relatively light solution where stiffness is particularly important and construction depth is limited. Although trusses are rarely used for highway bridges, they are used for longer railway spans and footbridges.

In railway bridges, the design requirements for the joints between web and chord members are onerous and beyond the scope of this publication. In footbridges, structural hollow sections are often used in trusses or Vierendeel girders. Guidance on the design of connections between hollow sections is given in BS EN 1993-1-8. Advice on detailing may also be obtained from steelwork contractors specialising in tubular structures.

3.7 Box girders

Box girders tend to be used for long spans where plate girder flange sizes become excessive, or where torsion, curvature or aerodynamic considerations demand greater torsional stiffness. Other than in these cases, plate girders will be a cheaper solution because assembly and welding of plate girders are largely automated and they take up less space and time in the workshop compared with box girders.

Box girders of width exceeding 1.5 m are usually made up of pre-welded stiffened plate panels. Web panels present situations very similar to webs in plate girders and, therefore, the stiffening tends to be similar. Longitudinal stiffeners are needed on wide thin plates in compression and on top flanges where there is direct loading, such as wheel loads. For highway bridges, the design of a stiffened steel deck is highly dependent on fatigue assessment and is outside the scope of this publication.

The shape of box girders needs to be maintained during fabrication by diaphragms at intervals acting as formers; normally these are placed on one flange and welded, followed by assembly of the webs and the other flange. Such diaphragms should form bearing diaphragms and intermediate frames as part of the permanent design. A rough guide for maximum spacing is three times the box depth, to ensure that distortion (or lozenging) of the box shape does not occur during fabrication or under action of deck traffic loading. If sufficient diaphragms are used then transverse bending of the longitudinal corner welds is negligible and partial penetration or fillet welds can be used. Full penetration welds in such situations are feasible, but are costly and can cause unpredictable distortion. A suitable detail is to use a 6 mm leg length fillet weld inside the box (often done as part of the assembly process) with external welds being partial penetration single V butt welds, typically size 8 mm throat thickness.

A further simplification is to over-sail the top flange and to use fillet welds both internally and externally.

Details should be devised to permit the maximum amount of fabrication and protective treatment prior to assembly of the box girders. Thus, internal diaphragms should allow for longitudinal stiffeners to slot through. Consideration should also be given to using weathering steel, to avoid having to apply protective treatment in enclosed spaces.

Diaphragms should incorporate holes of suitable dimensions to provide access for welding, painting and maintenance, with due regard for safety and emergency situations. In recognition of the designer’s responsibilities under the CDM Regulations, both during construction and in service, reference to health and safety guidelines is necessary to confirm acceptable sizes of openings, depending upon overall dimensions of the box and length of escape routes. It is suggested that a minimum size of access hole should be 600 mm wide and 600 mm high. Wherever possible, openings should be flush with the bottom flange to permit movement of a stretcher in emergency. Where a step is necessary, for example at a bearing diaphragm, the height of this should be minimised. It should be recognised that during construction further temporary access holes for welding and painting may be necessary, often through webs; these normally need to be reinstated with full penetration welded infill plates on completion.

For small boxes, less than about 1.2 m x 1.2 m, the inside should be permanently sealed by welding. Diaphragms are needed, at least for shape control during assembly, and would only be welded on three sides (no weld to the closing plate). For flanges less than 1.2 m wide, longitudinal stiffeners may be avoided, simplifying the fabrication. Bridges constructed of several such compact boxes can eliminate the need for any exposed lateral bracings.

The fit-up and fabrication of box girders requires considerable skill and experience. For pre-fabricated stiffened panels, the longitudinal stiffeners are usually welded first giving long runs where fully automatic welding can be used to advantage. The plate must be clamped down to avoid the inevitable weld shrinkage (on the side from which it is welded) from distorting the plate. The clamps are retained until the transverse stiffeners have been welded in. Some steelwork contractors use automated equipment to produce orthotropic stiffened plate panels in bulk economically to high quality.

For corner welds, the box may have to be turned over in the shop, after making the first two welds, to complete the second pair.

The hollow towers of cable-stayed and suspension bridges, as well as the ribs of some arch bridges, undergo stresses predominantly in compression. The site joints can be made by machining the ends of the components and the load is then transferred by direct bearing, the bolts or site welds being nominal for location and to carry any shear forces. In these cases, machining needs to be done after fabrication to overcome distortion due to welding. Drawings should clearly state that the required end plate thickness quoted is "after machining".

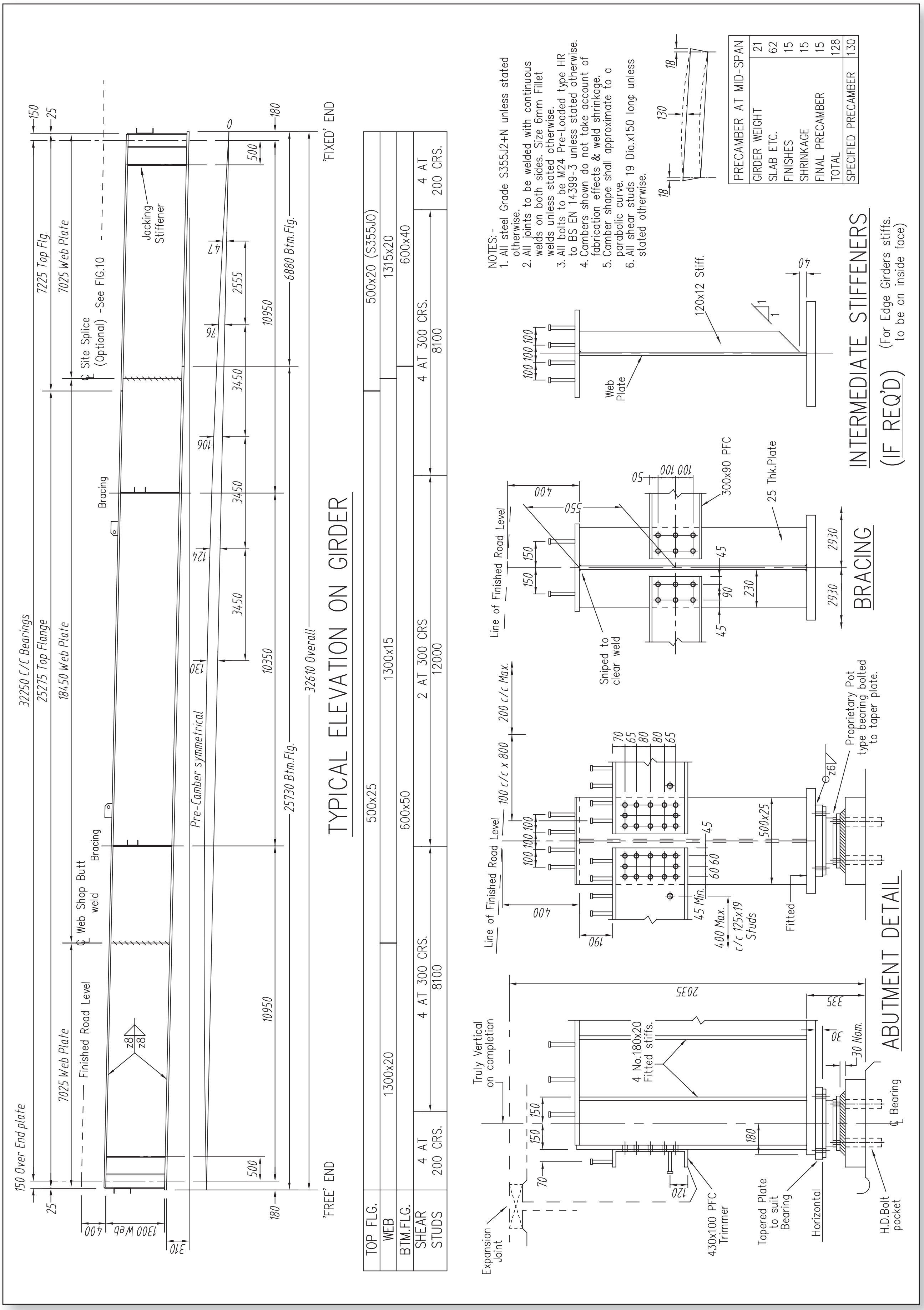


Figure 11 - Typical single span girder

3.8 Construction sequence

The designer will need to make assumptions in relation to the sequence of erection, concreting, use of temporary bracing and method of support of falsework. These assumptions should be clearly communicated to the steelwork contractor. Usually the design assumption made is that the steelwork is self-supporting and remains unpropped during concreting of the slab, so that composite behaviour is utilised for superimposed dead loads and traffic loading only. For steel composite bridges up to 150 m in length, the deck slab is often concreted in one continuous pour. However, for long viaducts the designer may specify a hit-and-miss concreting sequence where the deck slab is poured in the sagging moment regions either side of each pier before the hogging moment regions over the piers in order to militate against tension cracking in the slab. Sequential lengths of deck may also be poured. Whichever sequence is adopted, it needs to be defined at an early stage so that the allowances for permanent deformation of the steelwork can be properly determined.

3.9 Allowances for permanent deformation

Changes take place in the shape of a girder from the start of assembly of the prepared plate components in the fabrication shop until the time it is in service in the bridge. The pre-fabrication shape of the girder must anticipate these changes if the required final profile is to be achieved. Allowances need to be considered for:

(a) changes of shape during fabrication as a result of shrinkage and distortion due to flame cutting, welding and assembly sequence;

(b) deflection from the fabricated shape of the steelwork that takes place at site under its self-weight, the weight of the concrete slab, and the superimposed dead loads of surfacing and finishes. (For composite bridges the sequence of concrete pours will influence the deflection to some extent);

(c) long term effects such as concrete deck shrinkage and creep.

A camber is also often specified for appearance reasons or to achieve positive drainage fall, especially on footbridges. Fabrication allowances in (a) will be made by the steelwork contractor, often based on experience as well as theoretical calculations. The designer will need to define allowances to cover (b) and (c) and to supply the steelwork contractor with the information, usually in the form of a deflection diagram, along with the specified vertical geometry of the road or railway; the allowances will relate to the designer's assumed erection and concreting sequence.

It is good practice to define the girder shape after erection is complete and before concreting begins so that it may be checked and verified at this stage. The main contractor might wish to vary the concreting sequence; this would require with the designer's agreement (since it affects the permanent design) and would require recalculation of (b).

For significantly skewed multiple plate girder beam bridges it may be necessary to specify a pre-twist at end supports (see Sections 6.7).

In practice, the actual deformations may differ from those anticipated, for several reasons:

- allowances for thermal cutting and welding effects are difficult to estimate with precision due to the number of imponderables that are involved, including the residual stresses that exist in the material;
- "shake-out" of residual stresses may occur during transport and during erection, leading to changes in shape of components;
- for composite bridges, the design assumptions made for continuous spans relating to cracking of concrete in tension zones may not be fully realised, leading to deflections not being as predicted;
- for composite bridges, allowances for shrinkage and creep are difficult to predict accurately;
- uncertainty in temperature variations within the structure at the time of checking.

For these reasons, tolerance must be permitted in the steelwork dimensions and levels at completion. See further discussion in Chapter 6.

The *Model Project Specification* recommends a tolerance on level of the main girders at midspan of $\pm$ span/1000, up to a maximum of 35 mm in any one span, and on the level of one main girder relative to another adjacent main girder of 20 mm. Where the final bridge surface is intended to be level, it may be convenient to specify an upward camber (additional to any allowances for permanent deformation) of span/1000, but not less than 6 mm so that a downward sagging profile does not result if the allowances were inadequate.

Bracing connection details should, where possible, permit some degree of different permanent deformation between adjacent girders - bolted joints offer some degree of adjustment. For continuous spans, where fabricated lengths are joined at site, bolted splice details permit small adjustments in alignment; with welded joints some degree of trimming and fairing will often be needed at site.

A typical allowance made by a designer for effects (b) and (c) is shown in the example in Figure 11.

3.10 Bearings

3.10.1 Proprietary bearings

For most steel bridges, it is convenient to use proprietary bearings to transfer reactions and permit sliding and rotational displacements. The designer will choose an articulation arrangement for all the bearings; some bearings will be fixed against lateral displacement, some will be guided in one direction and some will be free.

For all proprietary bearings, it is important that full bearing details are supplied to the steelwork contractor at an early stage, before fabrication starts in the shop. If the choice of bearing and manufacturer is entrusted to the main contractor, rather than the steelwork contractor, the bearings might not be ordered until late in the construction programme and this could delay fabrication.

Proprietary pot bearings are suitable for most short and medium span bridges as fixed, guided and free bearings. They occupy minimal space and are normally bolted to the steelwork through a tapered steel bearing plate to take up longitudinal gradient of the girder, such that the bearing itself is nominally horizontal in the completed bridge. The bearing plate accommodates any departures from flatness of the flange: it is machined to form the taper and to achieve a flat surface for the bearing. It is usually welded to the girder, the bearing being attached by bolting into tapped holes in the bearing plate. The tapered plate should be of sufficient thickness to provide a thread engagement of at least 1.2 times the bolt diameter

Through bolts are an alternative solution, but the effect of holes through the flange needs to be taken into account and taper washers may be required. The holes might have to be oversize to allow for the slope of the flange, and the heads of the bolts have to be positioned so that they do not clash with the bearing stiffeners. Steelwork contractors prefer tapped holes because it allows them to mitigate the effects of the late supply of bearing fixing information (drilling of the main girder does not have to await confirmation of bearing details). Design of proprietary bearings is normally carried out by the manufacturer to BS EN 1337.

Lateral movements in guided and free pot bearings are accommodated on sliding surfaces of which one surface is PTFE, which has a low coefficient of friction (5% typical). Usually the sliding surface is above the pot bearing, so that debris cannot collect on it. The effect of eccentricity of the reaction, when the bearing slides, must be considered in the design of the girder. Bearings can be installed with the sliding surface below the pot bearing, provided that a protective skirt is provided to prevent debris collecting on the sliding surface, but this is an additional maintenance issue and is not generally preferred.

Elastomeric bearings permit lateral displacements by shearing deformation of the elastomer, except where an additional restraint system is provided. Elastomeric bearings are suitable for small spans and footbridges. The bearings should be retained in place by keep strips top and bottom and mounted on a lower steel plate secured to the substructure, or alternatively by the use of epoxy adhesives.

3.10.2 Fabricated bearings

In cases where uplift, excessive rotation or restraint about one axis only has to be accommodated, steel fabricated bearings such as a pin bearing may be more economic and suitable. In cases of uplift, a separate device or fabrication that resists upward forces may be preferable, for example a restraint against the effects of a vehicle impact on the bridge soffit.

For fixed bearings, and where rotation occurs longitudinally only, steel fabricated knuckle bearings are appropriate. They are also suitable for all the bearings in abutment-supported short spans up to about 20 m where higher friction can be accepted, as in many railway bridges. They consist simply of a steel block with radiused upper surface welded on to a steel spreader plate bolted to the substructure.

Where both longitudinal movement and uplift occurs, a swing link pinned bearing can be used. Fabricated bearings which can accommodate larger rotations are used in articulating Ro-Ro link spans and movable bridges.

Roller bearings of hardened steel are suitable where only longitudinal rotation and movement occurs; they are capable of achieving very low friction values (down to 1%) and are suitable on slender piers, to reduce bearing height. Fabricated roller bearings are used in the standard box girder rail bridges for spans greater than 20 m.

Design of fabricated bearings would normally be carried out by the bridge designer, possibly in collaboration with the steelwork contractor, or a bearing specialist.

3.10.3 Replacement Bearings

Generally, all bearings must be capable of replacement during the design life of the bridge and it is prudent to provide arrangements for jacking at supports. There should be sufficient space and support for jacking to be carried out and the girders should be provided with jacking stiffeners, if necessary.

3.11 Specification

The means to ensure that the assumptions in design about quality and accuracy of the steelwork are realised in construction is the ‘execution specification’. BS EN 1090-2 defines an execution specification as “the set of documents covering technical data and requirements for a particular steel structure including those to supplement and qualify the rules of this European Standard”. Thus, the execution specification comprises BS EN 1090-2 and all the necessary referenced standards, and a “project specification” that sets out the particular requirements and qualifications for the structure.

It is important when drawing up a project specification that only the necessary requirements and qualifications are included - specifiers should avoid introducing clauses simply as a matter of routine or asking for the highest quality simply to cover uncertainty in what is appropriate. The Steel Bridge Group’s *Model Project Specification* offers guidance on what clauses are needed to complement BS EN 1090-2 for typical bridge projects.

The following aspects have frequently been the subject of over-specification in the past and should generally be avoided:

Tightening method for preloaded bolts - It is generally accepted that the various tightening methods (part turn, torque control, direct tension indicator and tension control) all produce a satisfactory result but the individual steelwork contractors have their preferred methods. If a particular method is specified that is different from the contractor's preferred method, this is likely to add a risk allowance to the price.

Removal of drag lines - Drag lines do not need to be completely removed - a properly controlled as-cut edge is unlikely to be a critical detail in a fabricated girder, even for a railway bridge.

Exclusion of hard stamping - There is no reliable way other than hard stamping to mark girders, bracing members and cover plates. Using other methods leads to items going missing or being misidentified, causing difficulties with deliveries and delays to erection. Modern soft-nosed and dot matrix stamps do not create a sharp notch in the steel, so there is no technical reason why they should be banned, except possibly in areas subject to high fatigue stresses.

No butt welds in highly stressed areas - There is rarely any technical justification for restricting the location of butt welds. The fatigue class of the butt is rarely critical and the testing carried out on the weld will ensure that it is stronger than the parent material. Any restriction is likely to greatly increase the number of joints required.

Grinding of butt welds - Grinding of butt welds requires another operation on the critical path of fabrication that is hazardous, time-consuming and relatively costly for the perceived benefit. Grinding can produce a feature that is much wider and more visible than the cap of the weld, so it should only be specified where there are requirements for the sealing of grout or for drainage.

CHAPTER 4
BOLTING

4.1 Introduction

Bolting is generally preferred for the site connections in short and medium span steel bridges because it can be carried out more quickly than welding, and with less interruption to the flow of erection. Installation and tightening of bolts is a major site activity and the designer should consider the access for operatives and equipment. If insufficient attention is paid, the result can be components that cannot be fitted or bolts that cannot be tightened with standard equipment.

Permanent bolted connections of structural parts of bridge steelwork are almost always made with preloaded bolts, as recommended in BS EN 1993-2. Such connections are designed in shear as slip resistant connections, such that they do not slip at either the serviceability limit state or the ultimate limit state (the choice being a matter for design). Traditionally such connections have been known in the UK as high strength friction grip (HSFG) connections.

The purpose of this chapter is to present the options for the types of preloaded bolt, the means of tightening and the practical issues associated with those choices.

4.2 High-strength structural bolting assemblies for preloading

The design rules in BS EN 1993-1-8 for preloaded bolted connections relate to the use of bolts in accordance with BS EN 14399 *High-strength structural bolting assemblies for preloading*. That standard has 10 separate parts; of particular note are:

Part 3 : *System HR – Hexagon bolt and nut assemblies*

Part 4 : *System HV – Hexagon bolt and nut assemblies*

Part 9 : *System HR or HV – Direct tension indicators for bolt and nut assemblies*

Part 10 : *System HRC – Bolt and nut assemblies with calibrated preload*

Direct tension indicators are equivalent to 'load indicating washers' and system HRC is effectively the same as TCB bolts.

Currently only system HR and system HRC assemblies are used within the UK. System HV bolts use a slightly thinner nut and require greater control over manufacture, lubrication and installation; they are used in some continental European countries. If they were to be used instead of system HR bolts, the differences would have to be recognised by both designer and steelwork contractor; the use of both systems on the same job should always be avoided, because of the risks of confusion and mis-use.

System HR assemblies are available in thread sizes M12 to M36 inclusive, with property class 8.8 or 10.9. Generally, grade 8.8 sizes M24 and M30 are used in bridgework; these are readily available. HR bolts manufactured in the UK are usually only available in K0 class (see section 4.4 for discussion of K class).

System HRC assemblies are available in grade 10.9 and sizes M24 and M30 are readily available. HRC bolts manufactured in the UK are only available in K0 class.

BS EN 14399 does include a Part for countersunk preloaded bolts but these should not normally be used, except where a flush finish is essential for functional (not aesthetic) reasons.

4.3 Slip-resistant connections

Slip-resistant connections depend for their performance on the tightening of preloaded bolts to a specified minimum preload and a friction coefficient for the faying (interface) surfaces. The level of preload in a bolt is governed by three things:

- the tensile strength of the bolt material, the 'property class';
- the thread size of the bolt;
- the extent to which the bolt is strained (extended) during the installation and tightening process.

Frictional resistance is highly dependent on the surface conditions. Four classes of friction surface are defined in BS EN 1993-1-8 and slip factors given for each. The surface treatment that may be assumed for each of these classes is given in BS EN 1090-2 and is reproduced below in Table 5.

TABLE 5 - Classifications that may be assumed for friction surfaces

Surface Treatment	Class	Slip Factor (μ)
Surfaces blasted with shot or grit with loose rust removed, not pitted	A	0.50
Surfaces blasted with shot or grit; a) spray-metallised with aluminium or zinc based product b) with alkali-zinc silicate paint with a thickness of 50 μm to 80 μm	B	0.40
Surfaces cleaned by wire brush or flame cleaning, with loose rust removed	C	0.30
Surfaces as rolled	D	0.20

If the friction surface has any other surface treatment, the slip factors must be established by slip tests, as described in BS EN 1090-2- Annex G.

Weathering steel is not explicitly mentioned in the above surface treatments and this might give rise to some uncertainty as to whether surfaces of weathering steel might suffer any reduction in slip factor over time, before the joint is bolted up. Research into this by Lark (see references, Section 10.2) has demonstrated that there is no reduction in slip factor after 40 days and indeed the slip coefficient marginally improved through exposure. (The research also demonstrates that for carbon steel grades there is no appreciable reduction in slip factor at 40 days of exposure.)

Before assembly of the joint at site, any contamination of the slip resistant surfaces by oil or grease is best removed by suitable chemical means, as flame cleaning usually leaves harmful residues. If the joint cannot be assembled as soon as the surfaces have been treated, or as soon as any protective masking has been removed, it is sufficient to remove any thin film of rust or other loose material with a wire brush. During this process, the surfaces must not be damaged or made smooth.

Moisture on a face, in the form of clean water, rain or dew, should not be a problem unless it washes in dirt or is present for so long that serious rusting occurs. Light rusting is not detrimental to the performance of the joint.

4.4 Installation of preloaded bolts

For the installation of preloaded bolts, steelwork contractors should employ a certificated bolting co-ordinator and appropriately trained operatives, in accordance with the requirements of NHSS 20.

BS EN 1090-2 gives requirements for four methods of tightening preloaded bolts. Each method is intended to achieve at least the minimum preload. These four methods are;

- Torque method
- Combined method
- HRC method
- Direct tension indicator (DTI) method

In addition, the *Model Project Specification* gives requirements for tightening by the ‘part turn method’. Those requirements are similar to those for the HSFG bolts that have been used in the UK for very many years, before the introduction of BS EN 14399.

The suitability for preloading is determined by tests in accordance with BS EN 14399-2, which demonstrate the rotation/bolt force relationship for the assembly. Some of the tightening methods rely on the ‘K class’ of the assembly, determined according to the relevant Part of BS EN 14399; the class depends on the characteristics of the rotation/bolt force curves. Three classes are defined, K0, K1 and K2. The classes required for the various methods of tightening are shown in Table 6. The as-delivered calibration (K class) is valid for tightening by rotation of the nut. If tightening is done by rotation of the bolt head, calibration in accordance with BS EN 1090-2 Annex H or by supplementary testing in accordance with BS EN 14399-2 is required.

Currently, only class K0 assemblies are manufactured in the UK. There may be supply issues if a method that requires class K1 or K2 is selected.

4.4.1 General

All tightening methods need the components to be brought together to a snug-tight condition before commencement of preloading. Whichever method of bolt tightening is chosen bolts and nuts should always be tightened in a staggered pattern. When a bolt group comprises more than four bolts, tightening should be from the middle of the joint outwards and ensuring that all the plies are properly pulled together in full contact. Any residual gaps at the edges should not exceed 2 mm at this stage.

Tightening is performed by rotation of the nut, except where the access to the nut side of the assembly is inadequate. Special precautions, depending on the tightening method adopted, may have to be taken when bolts are tightened by rotation of the bolt head.

Pre-tightening and subsequent tightening are carried out progressively from the most rigid part of the joint to the least rigid part. To achieve uniform preloading, more than one cycle of tightening may be necessary. Pre-tightening should be completed for all bolts in a connection prior to commencement of subsequent steps.

TABLE 6 - K-Classes required for different tightening methods

Tightening Method	K-Classes
Torque Method	K2
Combined Method	K2 or K1
HRC Method	K0 with HRD nut only or K2
Direct tension indicator (DTI) Method	K2, K1, or K0
Part turn	K0

Note:

This table is based on Table 20 of BS EN 1090-2, with the addition of the class requirement for the part-turn method

If due to any cause a bolt or nut is slackened off after final tightening, the bolt, nut and washer must be discarded and not reused.

4.4.2 Torque method

The torque method is a three-stage operation (snug-fit, initial torque, final torque) using a calibrated torque wrench. This can be either a manual wrench or power tool fitted with a torque cut-out that must first be calibrated on a bolt from the job batch using a bolt load meter or similar device for determining bolt tension.

The procedure in BS EN 1090-2 requires the test bolt to be tightened to a calculated load related to the minimum shank tension, and the torque setting to obtain this tension is then used for tightening the bolts in the structure. It must be appreciated that torque can vary considerably (as much as ± 30% and even more if the bolts are coated) from bolt to bolt, even within the same batch, depending on a number of factors including the condition of the threads and nut/washer interface, and the amount of lubricant present on the threads. The result is an erratic torque-tension relationship and an unreliable preload.

The torque wrench is required by BS EN 1090-2 to have an accuracy of ± 4 % according to BS EN ISO 6789. Each wrench must be checked for accuracy at least weekly, and in the case of pneumatic wrenches, every time the hose length is changed. However, it is deemed good practice that the wrench be calibrated at least once per shift, or more often if required by the contract supervisor, for each change of diameter or batch and as specified for changes in bolt lengths.

The reference torque for tightening depends on the K class.

4.4.3 Combined method

The combined method uses the first two stages of the torque method, with the difference that the torque wrench needs to be capable of an accuracy of only ± 10 %, and to be calibrated annually. A second tightening step then follows, in which a specified part turn is applied to the rotated part of the assembly. The position of the nut relative to the bolt threads is marked after the first step, using a marking crayon or marking paint, so that the final rotation of the nut relative to the thread in this second step can be easily determined. Values for the required amount of part turn are given in BS EN 1090-2.

4.4.4 HRC method

HRC assemblies are tightened using a special wrench equipped with two co-axial sockets that react one against the other. The outer socket, which engages the nut, rotates clockwise. The inner socket, which engages the spline end of the bolt, rotates anticlockwise. The detailing of a ‘break-neck’ below the spline is such that it will shear off at a predetermined torque.

It is not necessary to calibrate the wrench in this case because the strength of the break-neck determines the maximum torque. However, the actual preload induced depends on the torque/preload relationship and this is partly related to the thread friction. The variability of the friction is controlled by supplying the bolts in drums to protect them from weather and contamination, but the lubrication can be affected by rain once the bolts are in place and ready to be tightened.

HRC bolts are currently not available in steel with improved atmospheric corrosion resistance in the small quantities required for most bridge projects.

4.4.5 Direct tension indicator (DTI) method

Direct tension indicators are specially hardened washers with nibs on one face. The nibs bear against the underside of the bolt head leaving a gap between the head and the load indicating face. Provision can be made for the DTI to be fitted under the nut when this is more convenient using a nut faced washer and tightening the bolt by rotating the head.

The compressible nibs deform to indicate when the minimum preload has been achieved. At the first step of tightening, to reach a uniform "snug-tight" condition, there is no significant deformation of the DTI protrusions. The second step involves progressive tightening and compression of the nibs, according to the procedure in BS EN 14399-9. The gaps measured on the indicating washer may be averaged to establish the acceptability of the tightening.

DTIs are available in steel with improved atmospheric corrosion resistance but their use in the UK has been prohibited by the Highways Agency in the Specification for Highway Works because crevices are created that would become corrosion traps. When used on painted bridges, the gaps in the DTIs have to be sealed by hand prior to painting.

4.4.6 Part turn method

The part turn method is similar to the combined method except that much greater reliance is placed on the rotation and less on the torque. Because it relies mainly on the rotation to achieve a particular strain and thus preload, the method does not require the use of K1 or K2 class assemblies. The method set out in the *Model Project Specification* has been validated for grade 8.8 bolts of sizes M24 and M30.

Tightening by the part-turn method comprises two steps:

- A first tightening step, using a torque wrench. The wrench is set to a torque value in accordance with Table 7. This first step must be completed for all bolts in one connection prior to commencement of the second step.
- A second final tightening step in which a specified part turn is applied to the turned part of the assembly. The position of the nut relative to the bolt threads is marked permanently after the first step, so that the final rotation of the nut relative to the thread in this second step can be easily determined. The second step is in accordance with the values given in Table 8.

TABLE 7 - Bedding torque for part turn method

Nominal Diameter (mm)	Bedding torque of bolt ± 10 % (Nm)
24	270
30	460

TABLE 8 - Final tightening of nuts

Total nominal thickness "t" of parts to be connected (inclusive all packers and washers)	Further angle of rotation to be applied, during the second stage of tightening. Degrees	Part turn
$t \leq 160$ mm	180	½

Advantages of the part-turn method

The main advantage of part-turn tightening is that it is a strain control method and is therefore almost totally independent of the friction and torque characteristics of the nut and bolt assembly. The part-turn method induces a specific strain (related to the part turn and thread pitch) that is well in excess of the elastic limit and which takes the bolt into a region where the load-elongation curve is relatively flat and thus the variations in (the relatively modest) bolt load applied during bedding result in only minor variations in the preload of the installed bolt.

This consistency provides the following benefits to the steelwork contractor and the client:

- Predictability : Preload always exceeds the minimum specified
- Reliability : Simple to control and supervise on site
- Economy : No calibration on site and less risk of re-work, so lower costs
- Versatility : Suitable for both non-alloy steel and steel with improved atmospheric corrosion resistant bridges

4.5 Inspection of preloaded bolts

A significant part of the cost of using preloaded bolts is related to the inspection. This is the key to the effective use of preloaded bolts, so should be carried out strictly in accordance with the requirements of BS EN 1090-2, or with the clauses from the *Model Project Specification* for the part turn method.

Preloaded bolting should be considered as a connection method that requires formal procedures in much the same way as welding does. While it is not specified in BS EN 1090-2, it is suggested that:

- Only personnel qualified by experience and training are used for preloaded bolt inspection.
- Only equipment that is tested and calibrated is used.
- Written procedures are in place and agreed by all parties for the control of the work to be carried out.
- Inspection is performed and records are kept to a formal procedure identifying the operator, inspector, method of installation along with the date and joint location.

CHAPTER 5
WELDING

5.1 Introduction

Today, welding is the primary joining process in steel bridge construction: virtually all shop joints, and frequently site joints, are welded. The output of the welding process is dependent on many factors and variables, so correct application and control are essential to assure weld integrity and achieve economic production.

The design and specification of a weld through to its final acceptance is a process that involves dialogue between designers, non-specialist engineers, and those directly executing welding operations. Welding technology uses its own special terminology and its application to steel bridge construction is subject to an extensive range of British, European and International Standards. The profusion of standards is potentially confusing, particularly in the progressive change to European Standards. It must also be recognised that weld integrity cannot always be verified simply by inspection and testing: good supervision, effective procedures and work instructions are required to assure the product quality.

The aim of this chapter is to give some insight into the process of welding, the terminology, the applicable standards, the common ways in which welds are made and the techniques for quality control. Further useful advice on many of the topics in this Chapter is available in the Steel Bridge Group Guidance Notes.

5.2 Principal welding standards

As mentioned in Section 1.1.1, BS EN 1090-2 sets out the requirements for the execution of steel structures. For bridge steelwork, it will be complemented by a project specification that identifies, where necessary, the options and alternatives for the particular bridge project.

BS EN 1090-2 introduces the concept of execution class, which is a means of specifying weld quality. Four classes are defined, denoted EXC1 to EXC4, EXC4 being the highest. The execution class may be defined for the whole structure or separately for different parts of the structure. The National Foreword suggests that EXC3 should be the default class for bridges and this is also chosen as the default in the Model Project Specification. Details where fatigue loading is onerous may need to be class EXC4.

To help identify the key standards for welding, Figure 12 presents a flowchart illustrating the relationships between BS EN 1090-2 and the principal welding standards. The standards are discussed below.

BS EN 1090-2 states that arc welding of ferritic and stainless steels should follow the requirements of BS EN 1011-1, -2 and -3. These standards deal with the production and control of arc welding of metallic materials and provide general guidance and advice on good welding practice. In addition, BS EN 1090-2 states that welding shall be undertaken in accordance with the requirements of the relevant part of BS EN ISO 3834. This latter standard is published in several parts; it defines quality system requirements necessary to ensure that welding is carried out in controlled conditions, both in the workshop and at site. It is designed to supplement existing quality management systems based upon BS EN ISO 9001. BS EN ISO 3834-2 defines comprehensive quality requirements and is appropriate for bridgeworks executed to EXC3.

BS EN 1011 assumes that execution of the provisions is entrusted to appropriately qualified, trained and experienced personnel and thus establishes another key principal of the BS EN standards – welding coordination. The welding coordinator is required to be suitably qualified and experienced and be able to supervise welding operations, thus establishing confidence in the associated processes and achieving reliable performance in production. Tasks and responsibilities in this pivotal role are defined in BS EN ISO 14731.

BS EN ISO 14731 confirms that welding coordination is the sole responsibility of the manufacturer and requires that the manufacturing company appoints at least one “responsible” welding coordinator (RWC). BS EN 1090-2 specifies the level of technical knowledge that welding coordination personnel need to have with respect to the welding operations that they supervise. The BCSA publication *Guide to the CE Marking of Structural Steelwork* explains in detail how certification of a company’s factory production control system in accordance with BS EN 1090-1 requires the identification and approval of the RWC during the certification process.

Further standards implemented by the execution standard relate to the qualification of welders, welding operators and inspection and testing personnel.

The formulation and testing of welding procedures is necessary to establish methods and to anticipate and overcome any difficulties likely to be encountered in the fabrication process. Normally, qualification testing of welding procedures is carried out in accordance with BS EN ISO 15614-1, although BS EN 1090-2 does permit testing to BS EN ISO 15613 as an alternative for EXC3 welding. BS EN ISO 15613 specifies the qualification of procedures based on a pre-production welding test. This method is useful where unusual joint configurations are welded in an application trial. Not only can welding data be recorded but inspection and testing techniques can be developed.

Welding procedure qualification records (WPQR) provide a basis from which detailed welding procedure specifications (WPS) or work instructions are developed within the ranges of approval of the test. WPS documents must be prepared in accordance with BS EN ISO 15609-1.

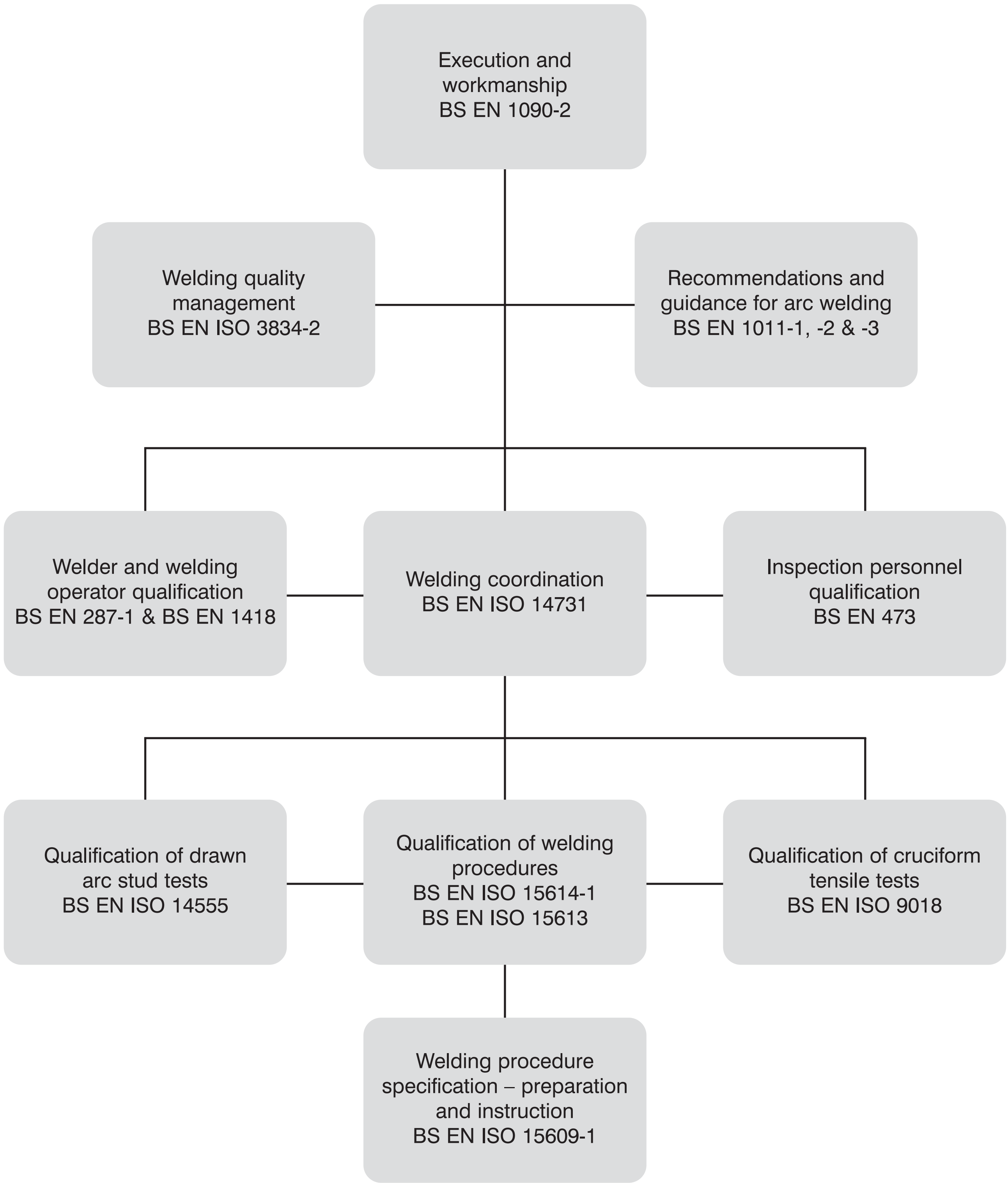


Figure 12 - Flowchart illustrating the interfaces between the standards and specifications for arc welding on bridge steelwork manufactured to class EXC3

Note:

Other standards are referred to in the text of BS EN 1090-2 but they relate to resistance and friction welding processes which are not used for welding bridge components.

BS EN ISO 3834 specifies a number of ISO standards with which it is necessary to demonstrate compliance. Several are not published to date or are superseded by requirements stated in BS EN 1090-2.

Inspection and testing requirements are described in BS EN 1090-2, with acceptance criteria specified in relation to criteria in BS EN ISO 5817. These criteria have not been specifically designed for bridge steelwork and may not be appropriate from a reliability or production point of view. Indeed, for EXC4, the acceptance criteria are more onerous than the basic requirements of the default welding procedure tests specified in the standard.

There are many cross references in BS EN 1090-2 to other welding standards including those relating to constituent products, welding consumables and inspection and testing techniques. Further additions and amendments to standards are often invoked through project execution specifications.

5.3 Types of welded connections

Most structural welded connections are described as either butt welds or fillet welds. Butt welds for bridgework are normally in-line plate joints in webs and flanges, either to accommodate a change of thickness or to make up available material to length. The positions of these butt welds are allowed for in the design, although material availability constraints or the erection scheme may require agreement of different or additional welds. Butt-welded Tee joints may be required where there are substantial loading or fatigue considerations in transverse connections.

Butt welds for bridges are full or partial penetration welds made between bevelled or chamfered materials. Full penetration butt welds are designed to transmit the full strength of the section. It is possible to weld these joints from one side but material thicknesses in bridges are such that they are usually welded from both sides, to balance distortion effects, with a back-gouging and/or back-grinding operation to ensure the integrity of the weld root. Single-sided butt welds with backing strips, ceramic or permanent steel, are common for joining steel deck plates and where there are closed box sections or stiffeners, which can only be accessed for welding from one side. Fatigue considerations limit the use of partial penetration welds.

Every effort should be made to avoid butt welding of attachments because of the costs associated with preparation, welding time, higher welder skill levels and more stringent and time-consuming testing requirements. In addition, butt welds tend to have larger volumes of deposited weld metal; this increases weld shrinkage effects and results in higher residual stress levels in the joint. Careful sequencing of welding operations is essential to balance shrinkage and to distribute residual stress, thus minimising distortion.

It is occasionally necessary to dress butt welds to a flush finish for fatigue reasons or to improve the testing regime. Dressing flush for aesthetic reasons alone should be avoided because it is difficult to dress the surface to match the adjacent as-rolled surface, and the result is often more visually noticeable than the original weld. Also, grinding is an additional hazard that is best avoided as far as possible.

Most other welded connections in bridgework use fillet welds, usually in a Tee configuration. They typically include the web to flange connections and stiffener, bearing and bracing connections. These are relatively simple to prepare, weld and test in normal bridge configurations, joint fit-up being the principal consideration.

Weld sizes must be detailed on the project design drawings together with any special fatigue classification requirements.

Attention is drawn to the fact that traditional UK practice has tended to use leg length to define fillet weld size, but this is not universal: throat thickness is used in European practice and BS EN 1993-1-8 gives requirements in relation to throat size, not leg length. The designer must be careful to ensure that it is clear which dimension is specified and all parties need to be aware of what has been specified.

5.4 Processes

The important factors for the steelwork contractor to consider when selecting a welding process are the ability to fulfil the design requirements and, from a productivity point of view, the deposition rate that can be achieved and the duty cycle or efficiency of the process. (The efficiency is a ratio of actual welding or arcing time to the overall time a welder or operator is engaged in performing the welding task. The overall time includes setting up equipment, cleaning and checking of the completed weld.)

The four main welding processes in regular use in UK bridge manufacturing are described below. The process numbers are defined in BS EN ISO 4063.

Variations of these processes have been developed to suit individual manufacturers' practices and facilities. Other processes also have a place for specific applications but are beyond the scope of this chapter.

Submerged arc welding (SAW), process 12

This is probably the most widely used process for welding bridge web-to-flange fillet welds and in-line butt welds in thick plate to make up flange and web lengths. The process feeds a continuous wire via a contact tip, where it makes electrical contact with the power from the rectifier, into the weld area, where it arcs and forms a molten pool. The weld pool is submerged by flux fed from a hopper. The flux immediately covering the molten weld pool melts, forming a slag and protecting the weld during solidification; surplus flux is collected and re-cycled. As the weld cools, the slag freezes and peels away, leaving high quality, good profile welds.

Solid wires from 1.6 to 4.0 mm diameter are commonly used with granular fluxes. Mechanical properties of the joint and the chemistry of the weld are influenced by careful selection of the wire/flux combination.

The process is inherently safer than other processes, as the arc is completely covered during welding, hence the term submerged arc. This also means that personal protection requirements are less. High deposition rates are a feature of the process because it is normally mechanised on gantries, tractors or other purpose-built equipment. This maintains control of parameters and provides guidance for accurate placement of welds.

The ability to exercise precise procedure control enables steelwork contractors to take advantage of the deep penetration characteristic of the process. By suitable control of welding parameters, the fusion zone extends into the contact area between the two components and the effective throat area achieved is greater than that defined by the leg length. BS EN 1993-1-8 allows account to be taken of this, provided that tests have demonstrated that the penetration can reliably be achieved.

Process variants include twin and tandem wire feeds and metal powder additions. These all increase deposition potential but the equipment requirements become more complex. The process is better suited to shop production but site use can be justified where applications include long runs and/or thick plate joints and the area can be weather-proofed.

Gas shielded metal arc welding. process 13
MAG welding with solid wire electrode, process 135

This is the most widely used manually controlled process for shop fabrication work; it is sometimes known as semi-automatic or CO_2 welding. A continuous solid wire electrode is passed through a wire feed unit to a 'gun', usually held and manipulated by the operator. Power is supplied from a rectifier or inverter source along interconnecting cables to the wire feed unit and gun cable; electrical connection to the wire is made in a contact tip at the end of the gun. The arc is protected by a shielding gas, which is directed to the weld area by a shroud or nozzle surrounding the contact tip. Shielding gases are normally a mixture of argon, carbon dioxide and possibly oxygen or helium.

Good deposition rates and duty cycles can be expected with the process, which can also be mechanised with simple motorised carriages. The gas shield is susceptible to being blown away by draughts, which can cause porosity and possible detrimental metallurgical changes in the weld metal. The process is therefore better suited to shop manufacture, although it is used on site where effective shelters can be provided. It is also more efficient in the flat and horizontal positions; welds in other positions are deposited with lower voltage and amperage parameters and are more prone to fusion defects.

MAG welding with flux cored electrode, process 136

This process utilises the same equipment as MAG welding, except that the consumable wire electrode is in the form of a small diameter tube filled with a flux. The advantage of using these wires is that higher deposition rates can be used, particularly when welding in the vertical position (between two vertical faces) or the overhead position. The presence of thin slag assists in overcoming gravity and enables welds to be deposited in position with relatively high current and voltage, thus reducing the possibility of fusion-type defects. Flux additions also influence the weld chemistry and thus enhance the mechanical properties of the joint.

MAG welding with metal cored electrode, process 138

Another variant of process 13 is to use a wire with a metallic powder core. This again enhances productivity by increasing the deposition rate, particularly for welds in the flat position or horizontal fillets (between horizontal and vertical faces).

Manual metal arc welding (MMA or stick welding), process 111

This process remains the most versatile of all welding processes but its use in the modern workshop is limited. Alternating current transformers, DC rectifiers or inverters supply electrical power along a cable to an electrode holder or tongs. A flux coated wire electrode (or "stick") is inserted in the holder and a welding arc is established at the tip of the electrode when it is struck against the work piece. The electrode melts at the tip into a molten pool, which fuses with the parent material forming the weld. The flux also melts, forming a protective slag and generating a gas shield to prevent contamination of the weld pool as it solidifies. Flux additions and the electrode core are used to influence the chemistry and the mechanical properties of the weld.

Hydrogen controlled basic coated electrodes are generally used for welding bridge steelwork. It is essential to store and handle these electrodes in accordance with the consumable manufacturer's recommendations in order to preserve their low hydrogen characteristics. This is achieved either by using drying ovens and heated quivers to store and handle the product, or by purchasing electrodes in sealed packages specifically designed to maintain low hydrogen levels.

The disadvantages of the process are the relatively low deposition rate and the high levels of waste associated with the unusable end stubs of electrodes. Nevertheless, it remains the main process for site welding and for difficult access areas where bulky equipment is unsuitable.

Drawn arc stud welding with ceramic ferrule, process 783

Composite bridges require the welding of shear stud connectors to the top flange of plate or box girders and other locations where steel to concrete composite action is required. The method of welding is known as the drawn-arc process and specialist equipment is required in the form of a heavy-duty rectifier and a purpose-made gun. Studs are loaded into the gun and on making electrical contact with the work, the tipped end arcs and melts. The duration of the arc is timed to establish a molten state between the end of the stud and the parent material. At the appropriate moment, the gun plunges the stud into the weld pool. A ceramic ferrule surrounds the stud to protect and support the weld pool, stabilise the arc and mould the displaced weld pool to form a weld collar. The ferrule is chipped off when the weld solidifies. Satisfactory welds typically have a regular, bright and clean collar completely surrounding the stud.

The equipment for stud welding is not particularly portable, so if only a few studs are to be installed or replaced at site, it is more economic to use a manual process.

5.5 Preparation of welding procedure specifications

The drawings detail the structural form, material selection and indicate welded joint connections. The steelwork contractor selects methods of welding each joint configuration that will achieve the performance required. Strength, fracture toughness, ductility and fatigue are the significant metallurgical and mechanical properties that must be considered. The type of joint, the welding position and productivity and resource demands influence the selection of a suitable welding process.

The chosen method is presented on a welding procedure specification (WPS), which details the information necessary to instruct and guide welders to assure repeatable performance for each joint configuration. An example format for a WPS is shown in Annex A of BS EN ISO 15609-1. Steelwork contractors may have their own corporate template but all include the essential information to enable the proper instruction to be communicated to the welder.

It is necessary to support the WPS with evidence of satisfactory procedure tests in the form of a welding procedure qualification record (WPQR) prepared in accordance with BS EN ISO 15614-1. The introduction of this standard states that welding procedure tests made to former national standards and specifications are not invalidated, provided that there is technical equivalence; additional tests may be necessary to achieve this. The major UK steelwork contractors have pre-qualified welding procedures capable of producing satisfactory welds in most joint configurations likely to be encountered in conventional bridgework.

For circumstances where previous test data is not relevant, it is necessary to conduct a welding procedure test to establish and to confirm the suitability of the proposed WPS.

5.6 Welding procedure tests

BS EN 15614-1 describes the conditions for the execution of welding procedure tests and the limits of validity within the ranges of qualification stated in the standard. The welding coordinator prepares a preliminary welding procedure specification (pWPS), which is an initial proposal for carrying out the procedure test. For each joint configuration, either butt or fillet weld, consideration is given to the material grade and thickness and anticipated fit-up tolerances likely to be achieved in practice. Process selection is determined by the method of assembly, the welding position and whether mechanisation is a viable proposition to improve productivity and to provide consistent weld quality. Joint preparation dimensions are dependent upon the choice of process, any access restrictions and the material thickness.

Consumables are selected for material grade compatibility and to achieve the mechanical properties specified, primarily in terms of strength and toughness. For S355 and higher grades of steel, hydrogen-controlled products are used.

The risk of hydrogen cracking, lamellar tearing, solidification cracking or any other potential problem is assessed not only for the purpose of conducting the test but also for the intended application of the welding procedure on the project. Appropriate measures, such as the introduction of preheat or post-heat, are included in the pWPS.

Distortion control is maintained by correct sequencing of welding. Back-gouging and/or back-grinding to achieve root weld integrity are introduced as necessary.

Welding voltage, current and speed ranges are noted, to provide a guide to the optimum welding conditions.

The ranges of approval for material groups, thickness and type of joint within the specification are carefully considered to maximise the application of the pWPS. Test plates are prepared of sufficient size to extract the mechanical test specimens, including specimens for any additional tests specified or necessary to enhance the applicability of the procedure.

The plates and the pWPS are presented to the welder; the test is conducted in the presence of an examiner (usually from an independent examining body) and a record maintained of the actual welding parameters along with any modifications to the procedure needed.

Completed tests are submitted to the independent examiner for visual examination and non-destructive testing in accordance with Table 1 of the Standard. Satisfactory test plates are then submitted for destructive testing, again in accordance with Table 1. Non-destructive testing techniques are normally ultrasonic testing for volumetric examination and magnetic particle inspection for surface breaking imperfections.

There is a series of further standards detailing with the preparation, machining and testing of all types of destructive test specimen. Normally, specialist laboratories arrange for the preparation of test specimens and undertake the actual mechanical testing and reporting. Typical specimens for an in-line plate butt weld include transverse tensile tests, transverse bend tests, impact tests and a macro-examination piece on which hardness testing is performed. For impact tests, the minimum energy absorption requirements and the testing temperature are normally the same as those required for the parent material in the joint. It is wise to test all welding procedures to the limit of potential application, to avoid repeating similar tests in the future.

The completed test results are compiled into a weld procedure qualification record (WPQR) endorsed by the examiner. A typical format is shown in Annex A of BS EN ISO 15614-1.

As previously mentioned, BS EN ISO 15613 describes the method of simulating actual joint configurations in a pre-production welding test. The simulation can be used to consider the effects of limited access, dimensions and configuration, restraint and other essential features. The welding procedure test is generally carried out in accordance with BS EN ISO 15614-1 as far as practically possible. It may be necessary to conduct more than one welding procedure test to gain maximum confidence in the application and range of qualification.

There is an additional general requirement concerning welding procedure tests that where paint primers are to be applied to the work prior to fabrication, they are applied to the sample material used for the tests. In practice, careful control of paint thickness is required to avoid welding defects, and most bridge steelwork contractors do not apply any paint prior to welding, except on internal faces of box girder sections – even then, weld areas are masked off to minimise the risk of porosity and other defects associated with contamination.

BS EN ISO 14555 describes the method of procedure testing for drawn arc welded stud connectors. The standard includes the test requirements necessary to prove the integrity of stud welds and it also specifies production testing requirements to monitor in-process stud welding. Qualification based on previous experience is also permitted and most bridge steelwork contractors can provide evidence to support this.

A further qualification test requirement of BS EN 1090-2 is to do cruciform tensile tests where procedures are to apply to transversely stressed fillet welds. BS EN ISO 9018 describes the procedure for testing in order to determine the tensile strength and location of fracture of the joint. Evaluation of the results is carried out in accordance with BS EN 1090-2.

5.7 Avoidance of hydrogen cracking

Cracking can lead to brittle failure of the joint, with potentially catastrophic results. Hydrogen (or cold) cracking can occur in the region of the parent metal adjacent to the fusion boundary of the weld, known as the heat-affected zone (HAZ). Weld metal failure can also be triggered under certain conditions. The mechanisms that cause failure are complex and described in detail in specialist texts.

Recommended methods for avoiding hydrogen cracking are described in BS EN 1011-2, Annex C. These methods determine a level of preheating to modify cooling rates and therefore to reduce the risk of forming crack-susceptible microstructures in the HAZ. Preheating also lessens thermal shock and encourages the evolution of hydrogen from the weld, particularly if maintained as a post heat on completion of the joint.

One of the parameters required to calculate preheat is heat input. A notable change in the standard is to discontinue use of the term arc energy in favour of heat input to describe the energy introduced into the weld per unit run length. The calculation of heat input is based upon the welding voltage, current and travel speed and includes a thermal efficiency factor; the formula is detailed in Part 1 of the standard.

High restraint and increased carbon equivalent values associated with thicker plates and higher steel grades may demand more stringent procedure controls. Low heat inputs associated with small welds may also necessitate preheating. Experienced steelwork contractors can accommodate this extra operation and allow for it accordingly.

BS EN 1011 confirms that the most effective assurance of avoiding hydrogen cracking is to reduce the hydrogen input to the weld metal from the welding consumables. Processes with inherently low hydrogen potential are effective as part of the strategy, as well as the adoption of strict storage and handling procedures for hydrogen-controlled electrodes. Consumable suppliers' data and recommendations provide guidance to ensure the lowest possible hydrogen levels are achieved for the type of product selected in the procedure.

Further informative Annexes in BS EN 1011-2 describe the influence of welding conditions on HAZ toughness and hardness and give useful advice on avoiding solidification cracking and lamellar tearing.

5.8 Welder qualification

BS EN 1090-2 requires welders to be qualified in accordance with BS EN 287-1. That standard prescribes tests to qualify welders based upon process, consumable, type of joint, welding position and material. Welders undertaking successful procedure tests gain automatic approval within the ranges of qualification in the standard. Welding operators are required to be approved in accordance with BS EN 1418, when welding is fully mechanised or automatic. This standard places emphasis on testing the operator's ability to set up and adjust equipment before and during welding.

Welder qualifications are time limited and need confirmation of validity depending on continuity of employment, engagement on work of a relevant technical nature and satisfactory performance. Prolongation of a welder's qualification depends on recorded supporting evidence demonstrating continuing satisfactory performance within the original test range, and the evidence must include either volumetric destructive testing or destructive testing.

The success of all welding operations relies on the workforce having appropriate training and regular monitoring of competence by inspection and testing.

5.9 Inspection and testing

BS EN 1090-2 sets out the scope of inspection before, during and after welding and gives acceptance criteria related to execution class. Most testing is non-destructive; destructive testing is only carried out on run-off plates.

Non-destructive testing

Non-destructive testing is carried out in accordance with the principles in BS EN 12062. For bridgeworks, the principal methods are magnetic particle inspection (usually abbreviated to MPI or MT) for surface examinations and ultrasonic testing (UT) for sub-surface or volumetric examinations. Radiographic testing is also mentioned in BS EN 1090-2. Radiography demands stringent health and safety controls; it is relatively slow and needs specialist equipment. Use of the method has declined on bridgework compared with the safer and more portable equipment associated with UT. Safety exclusion zones are required, in works and on site, when radiography is in progress. However, radiography can be used to clarify the nature, sizes or extent of multiple internal flaws detected ultrasonically.

Specialist technicians with recognised training and qualifications in accordance with BS EN 473 are required for all non-destructive testing methods.

BS EN 1090-2 requires that all welds be visually inspected throughout their length. From a practical point of view, welds should be visually inspected immediately after welding, to ensure obvious surface defects are dealt with promptly.

Further non-destructive testing requirements are based upon performance techniques and require more stringent examination of the first five joints of new welding procedure specifications, to establish that the procedure is capable of producing conforming quality welds when implemented in production. Supplementary non-destructive testing based upon types of joint, rather than specific critical joints, is then specified. The intent is to sample a variety of welds based upon joint type, material grade, welding equipment and the work of welders and thereby maintain overall performance monitoring.

Where partial or percentage examination is specified, guidance on selections of test lengths is given in BS EN 12062; where unacceptable discontinuities are found, the examination area is increased accordingly.

BS EN 1090-2 also tabulates minimum hold times prior to supplementary non-destructive testing based upon weld size, heat input and material grade.

Recognising that where fatigue strength requirements are more onerous and a more stringent examination is required, BS EN 1090-2 does provide for the project execution specification to identify specific joints for a higher level of inspection together with the extent and method of testing.

For class EXC3, the acceptance criteria for weld imperfections is quality level B of BS EN ISO 5817. Where it is necessary to achieve an enhanced level of quality to meet fatigue strength requirements, BS EN 1090-2 Table 17 gives additional requirements for class EXC4 as quality level B+.

Generally, the requirements for quality level B+ are not practically achievable in routine production. Normal welding procedure testing and welder qualification tests are not assessed against the requirements of this level. Where it is necessary to achieve this level of quality, the requirements should be focussed on the relevant joint detail, so that the contractor has the opportunity to prepare welding procedure specifications, to qualify welders and to develop inspection and test techniques accordingly.

Destructive testing

There is no requirement in BS EN 1090-2 to carry out destructive testing for test transverse joints in tension flanges. However, the scope to identify specific joints for inspection would allow the project specification to test, for example, samples from "run-off" plates attached to in-line butt welds. Additionally, production tests may be specified on: steel grades higher than S460; fillet welds where the deep penetration characteristics of the welding process are utilised; for bridge orthotropic decks where a macro examination is required to check the weld penetration; and on stiffener-to-stiffener connections with splice plates.

Production testing of stud welding

Stud welds for shear connectors are examined and tested in accordance with BS EN ISO 14555. The standard emphasises the need to exercise process control before, during and after welding. Pre-production testing is used to prove the welding procedure and, depending on the application, includes bend tests, tensile tests, torque tests, macro examination and radiographic examination.

Production weld tests are also required for drawn-arc stud welds. These should be performed by the manufacturer before the beginning of welding operations on a construction or group of similar constructions, and/or after a specified number of welds. Each test should consist of at least 10 stud welds and be tested/assessed in accordance with the requirements of BS EN ISO 14555. The number of tests required should be specified in the contract specification.

5.10 Weld quality

The effect of imperfections on the performance of welded joints depends upon the loading applied and upon material properties. The effect may also depend on the precise location and orientation of the imperfection, and upon such factors as service environment and temperature. The major effect of weld imperfections on the service performance of steel structures is to increase the risk of failure by fatigue or by brittle fracture.

Types of welding imperfection can be classified under one of several general headings:

(a) Cracks.

(b) Planar imperfections other than cracks, e.g. lack of penetration, lack of fusion.

(c) Slag inclusions.

(d) Porosity, pores.

(e) Undercut or profile imperfections.

Cracks or planar imperfections penetrating the surface are potentially the most serious. Embedded slag inclusions and porosity are unlikely to initiate failure unless very excessive. Undercut is not normally a serious problem unless significant tensile stresses exist transverse to the joint.

By selecting an execution class in BS EN 1090-2, acceptance criteria are established, beyond which the imperfection is considered a defect.

Where defects are detected as a result of inspection and testing during production, remedial measures are likely to be necessary, although in many cases the particular defect may be assessed on the concept of ‘fitness for purpose’. Such acceptance is dependent upon the actual stress levels and the significance of fatigue at the location. This is a matter for speedy consultation between the steelwork contractor and the designer for, if acceptable, costly repairs (and the potential for introducing further defects or distortion) can be avoided.

CHAPTER 6
ACCURACY

6.1 Introduction

The object of this chapter is to discuss the accuracy of steel fabrication and its significance for the designer as well as for the steelwork contractor.

The dimensions of any item may vary from those defined by the designer: such variations stem from the nature and behaviour of the material as much as from the process of making it. Modern steel fabrication involves the manufacture of large and often complex welded assemblies of rolled steel products: high temperature processes are used to make the steel products, to form the components and to join them together, so dimensional variation is inherent and unavoidable. This behaviour has implications for the designer, for the steelwork contractor, and for the builder of supporting and adjoining structures. In carrying out their roles, each has to anticipate the variations. The important questions are: which dimensional variations are significant; what limits must be put on those variations which are significant; and how should variations be managed, to ensure that the design is implemented to meet its performance requirements without delay?

In steel bridge construction, dimensional variation is significant in a number of ways, for it involves precise mechanical components, structural steelwork manufactured remote from the site, and civil engineering works. These interface with each other and yet their precision varies from the high accuracy of mechanical components to the inaccuracies inherent in placing concrete. It is convenient to distinguish between the following:

- Mechanical fit, which is vital, for example, for functioning between nut and bolt, between bearing and girder, and between machined abutting faces of compression members.
- Fit-up of fabricated members, which is essential for efficient assembly. In a bolted site splice, for example, relative position of holes is crucial for inserting the bolts but the positional accuracy of individual bolts has very little effect on the strength of the connection.
- Deviation from flatness or straightness, which affects the stiffness and strength of components. For example, buckling resistance is less for an out-of straight slender strut.
- Accuracy of assembly at site, where the steelwork must be assembled without having to apply unintended forces to connections and without deforming the structure from its intended geometry (thus inhibiting, for example, construction of the correct deck slab thickness).
- Interface with substructures, where adjustment has to be provided to accommodate the different accuracies of steelwork and substructure – for example, the provision of large pockets for holding down bolts and variable grout layers beneath bearings.

The control of dimensions is fundamental to the mechanical engineering discipline, and without which no mechanism could work, no parts would be interchangeable. It is achieved by specifying tolerances – limits to the deviation from nominal dimension. No mechanical drawing is complete without tolerances on all dimensions, limits and fits on mating parts, and flatness tolerances on surfaces. In contrast, civil engineering construction has largely ignored the concept of tolerances, depending on the calibration of its metrology to build the product satisfactorily in situ. Historically, steel fabrication found a workable compromise, making large manufactured products using workshop techniques that assured their efficient assembly at a remote site – tolerancing was not part of that process as a rule; it was implicit in much of the work and explicit only for mechanical bridge parts.

The level of accuracy common to a mechanical engineering workshop is generally unnecessary for steel bridgework – for which it would have to be justified because such accuracy comes at a substantial cost and needs special facilities, including machining. For example, the variation of flatness and thickness of a steel plate from the rolling mill is perfectly satisfactory for a girder, but it would be unacceptable for a machine part. With the widespread use of automated processes from the 1980s for plate preparation, hole drilling, girder assembly and welding, the geometrical accuracy to which steel fabrication can be made has much improved: this has been driven by the economics of practicable manufacture and the replacement of labour intensive traditional practice.

In the UK, formal tolerances on bridge steelwork were first introduced in the 1968 revision of BS 153. The need for appropriate tolerances was highlighted in the investigations following the box girder collapses in the 1970s and culminated in tolerances that were set in BS 5400-6 to match the assumptions inherent in BS 5400-3. With the introduction of the Eurocodes similar, but more comprehensive, tolerances on steelwork were introduced in EN 1090-2, in consideration of both fit-up and of achieving the intended structural performance intended by design in accordance with Eurocode 3.

For each new project, the steelwork contractor will assess the design to determine how best to undertake the fabrication and how to control dimensions to ensure proper fit-up and assembly at site. For box girders in particular, and for large bridges with steel decks, this may well include a project-specific regime of dimensional tolerances on sub-assemblies such as deck panels; these would be compatible with the tolerances set by the designer for the finished bridge.

The following sections concentrate on the fabrication process, the behaviour of steel in fabrication, and those aspects of accuracy which bear particularly on the strength of members and the shape of components, which are of primary concern to the designer.

6.2 Fabrication tolerances

Tolerances are specified in Annex D of BS EN 1090-2. Tolerances are grouped three distinct categories:

- Essential tolerances. These are the limits of permissible deviation for the mechanical resistance and stability of the structure and are used to support conformity assessment to BS EN 1090-1.
- Functional tolerances. These are the limits of permissible deviation for fit-up and appearance. Two classes of deviation are given, class 1 being the less onerous and is the default for routine fabrication. Class 2 requires more expensive and special measures through fabrication and erection.
- Special tolerances. Individual projects may specify special tolerances, either as a modification of the essential or functional tolerances or for aspects not already covered. There is a need for certain additional tolerances on most bridge structures and suggested limits are given in the Model Project Specification.

The values of the permitted deviations for essential manufacturing tolerances are given in tables BS EN 1090-2 D.1.1 through to D.1.10. Essential erection tolerances are given in Tables D.11 to D.15 but these relate to building steelwork, not bridge steelwork. Functional tolerances are given in Tables D.2.1 through to D.2.28. As an alternative to the functional tolerances in Annex D, BS EN 1090-2 does allow the use of BS EN ISO 13920; this is more likely to be used in cases of heavily welded structures where distortion from welding is the dominant factor in determining the dimensions and shape. That standard specifies general tolerances for linear and angular dimensions and for shape and position of welded structures.

6.3 Causes of fabrication distortion

Distortion is a general term used in steelwork to describe the various movements and shrinkages that take place when heat is applied in cutting or welding processes. All welding causes a certain amount of shrinkage and in some situations will also cause deformation from the original shape. Longitudinal and transverse shrinkage in many circumstances are only a minor problem but angular distortion, bowing and twisting can present considerable difficulties if the fabrication is not in experienced hands.

A full awareness of distortion is vital to all concerned with welding including the designer, detailer, shop foreman and the welders, as each in their actions could cause difficulties through lack of understanding and care. Weld sizes should be kept to the minimum required for the design in order to reduce distortional effects; in many cases, partial penetration welds can be used in preference to full penetration welds, deep penetration welds in preference to ordinary fillet welds.

Some distortional effects can be corrected, but it is much more satisfactory to plan to avoid distortion and thereby avoid the difficulties and costs of straightening to achieve final acceptability. Consider a single fillet weld making a 'T' joint, as shown in Figure 13. On cooling, the weld metal will induce a longitudinal contraction, a transverse contraction and an angular distortion of the up-standing leg. A similar section with double fillet weld will induce greater longitudinal and transverse contraction and the combined forces will produce an angular distortion or bowing of the table of the Tee. The longitudinal shrinkage is likely to be about 1 mm per 3 m of weld and transverse contraction about 1 mm, provided the leg length of the weld does not exceed three quarters of the plate thickness.

The contractions produced by a single V butt weld (see Figure 13) induce longitudinal and transverse shrinkage, producing angular distortion and possibly some bowing. The transverse contraction will be between 1.5 mm and 3 mm and the longitudinal contraction about 1 mm in 3 m. Angular distortion occurs after the first run of weld cools, contracts and draws the plates together. The second run has the same shrinkage effect but its contraction is restricted by the solidified first run, which acts as a fulcrum for angular distortion. Subsequent runs increase the effect. The angular distortion is a direct function of the number of filler runs and not the plate thickness, although of course the two are related.

The use of a double V preparation to balance the volume of weld about the centre of gravity of the section will significantly reduce any angular distortion. To allow for the effect of back gouging, asymmetric preparations are often used to advantage, but it must be remembered that longitudinal and transverse contractions will still be present. The contractions in a structure can be assessed, but a number of factors will affect the result. The fit-up is most important, as any excess gap will affect the weld volume and increase shrinkage. The largest size of electrodes should be used and where possible semi-automatic and automatic processes should be employed, to reduce the total heat input and the shrinkage to a minimum.

It is always worth considering the effects of weld preparation on quality of the weld, Single V butt welds give good access to the root of the weld and can limit the amount of positional work required for the welder. Double V butt welds will reduce distortion but increase the amount of positional welding, which may lead to expensive repairs. In both cases, the material thickness and process need to be assessed with an eye to practicality.

In certain circumstances, residual rolling stresses in the parent metal can have considerable effect and may cause otherwise similar sections to react differently. The extent of final distortion will be a combination of the effects of inherent stresses and those introduced by welding.

6.4 Methods of control of distortion

All members that are welded will shrink in their length, so each member will either be fabricated over-length and cut to length after welding, or an estimate of shrinkage will be added to anticipate the effect during the fabrication of the member. For the control of angular distortion and bowing, there are two methods of control that can be considered if the distortion is likely to be of significance (see Figure 13):

- Pre-setting. The section is bent in the opposite direction to that in which it is expected to distort and welding is then carried out under restraint. When cool, the clamps are removed and the section should spring straight. Trials and experience can determine the extent of pre-bend for any particular member.
- Clamping. The units are held straight by clamps whilst the welding is carried out, which reduces the distortion to tolerable amounts.

6.5 Effect of design on distortion

A good design will use the minimum amount of weld metal consistent with the required strength. Where a flange changes direction (for example at the end of a haunch) the plate should be bent, rather than butt welded. Where a cross section is asymmetric, the shrinkage of the two web-to-flange welds will lead to curvature, because the shrinkage is not balanced. The steelwork contractor can allow for this but highly asymmetric arrangements, particularly with a much heavier weld to one flange, or with a flange set to one side of a web (as in a channel or J section), should be avoided where possible. Where the shrinkage is balanced, only an allowance for the overall contraction needs to be made (see Figure 13).

6.6 Distortion effects and control

6.6.1 Fabrication of girders

Butt joints in flanges or webs of girders are completed before the girders are assembled wherever practical. Run-on/run-off pieces are clamped at each end of these joints; alternatively, they can be tack welded on the internal face of the weld: they should be of the same thickness as the plate material and have the same weld preparation. Extension pieces are removed after the completion of the welding and the flange edges carefully dressed by grinding; any visible imperfections are normally removed at this stage and the weld repaired.

The direction of weld runs is usually alternated to avoid the tendency for the joint to distort in plan. It may be necessary to balance the welding of the butt joints by making a number of runs in one side of the V preparation and then turning the flange over to make runs in the second side and so on. Back-chipping or gouging must be carried out before commencing welding on the second side. The use of suitable rotating fixtures will enable long flanges to be turned over without risk of cracking the weld.

On completion of all web and flange butt joints, the girder is assembled and welded. If automatic welding is to be used for the web to flange welds, the stiffeners are added after these welds are complete. If the weld on one face is made before the weld on the other face (unlike welding in a T and I machine), the flange will be set slightly out of square to allow for the greater effect of the welding of the first side fillet welds (see Figure 13). Where manual welding is used, it is normal practice to fit the transverse web stiffeners before the welding; these help to maintain the squareness of the flanges.

Distortion can come to the steelwork contractor's aid where bearing stiffeners need to be fitted. Local flange heating can be used to bow the flanges locally (away from the stiffener end), allowing insertion of the stiffener; the subsequent cooling causes the flanges to come into tight contact with the stiffener end. Such controlled heat input operations are part of the fabrication art and are generally not detrimental.

6.6.2 Site welded girder splices

Where a girder is spliced by welding (most often at site), it is usual to weld the flanges before the web; the flange, being thicker and requiring a greater number of runs of weld, will shrink more than the thinner web. (If a thin web were welded first, it would be likely to buckle as a result of flange shrinkage.) It is then necessary to anticipate this procedure by fabricating the web joint with a root gap larger than that specified by the weld procedure, by an amount equal to the expected weld shrinkage of the flange joint. In heavy girder joints, a variation of the procedure is often adopted: both flanges are completed to about two thirds of their weld volumes and then the web welds and finally the rest of the flange welds. This method helps to minimise tensile stresses remaining in the web.

6.7 Checking of deviations

The potential difficulty associated with working to specified tolerances is the amount of checking required in the fabrication shop. The specification of reasonable tolerances should not increase fabrication costs, as a good steelwork contractor should be able to comply with the values without special procedures or rectification measures. However, as well as the direct cost of checking, costs can be incurred when checking activities delay the work-piece from entering the next phase of production: checking adds time and cost to the overall fabrication process.

It should be noted that BS EN 1090-2 does not set frequencies for checking component parts: the reliance is on the steelwork contractor to have an approved Factory Production Control system (FPC) in place to maintain the quality of the finished work and set the frequencies of testing in an inspection plan for the project.

Normally, all member components are visually examined by the steelwork contractor; not all dimensions will be checked for compliance with the specified tolerances but critical dimensions and those which appear likely to be out of tolerance will be checked.

In making any checks for flatness, the scanning device is placed such that local surface irregularities do not influence the results. The checks are performed on completion of fabrication, and then at site on completion of each site joint. Local checks will be made on flatness of plate panels and straightness of stiffeners.

The verticality of webs at supports can only be checked during any trial erection of the steelwork or after erection at site. For the end supports of bridges with a skew of more than about 30°, significant twist of the girders is likely to occur when weight of the slab is applied. This arises from the displacement of the bracing. In such cases, the designer should either specify that the webs are vertical before

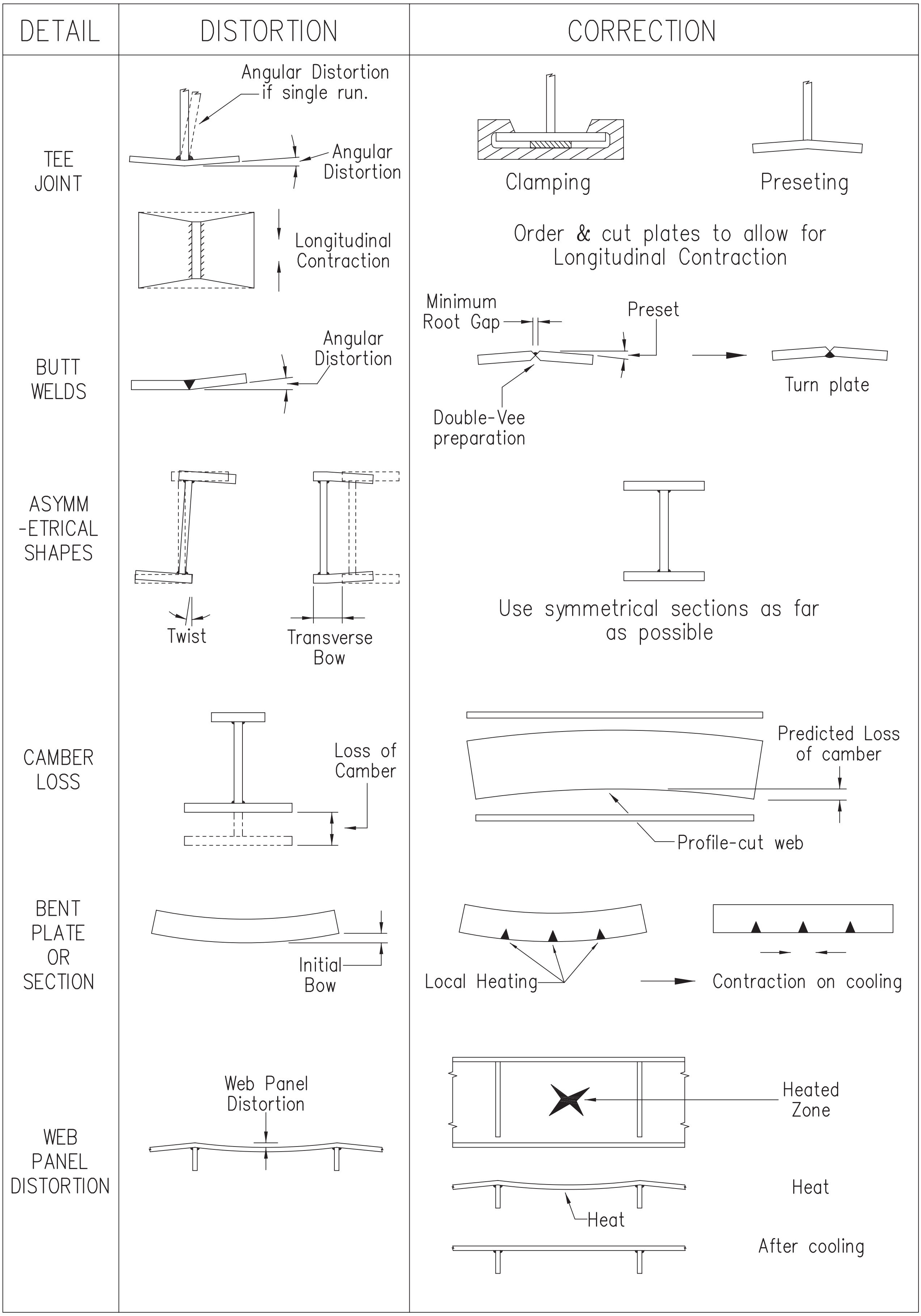


Figure 13 - Weld distortion and correction

concreting or supply values for the predicted twist so that the steelwork contractor can preset the girders to counteract it (though precise prediction is always difficult and it cannot be guaranteed that the girder will twist back to within required verticality). See discussion in Section 3.9.

Where the tolerances specified in BS EN 1090-2 are exceeded, remedial actions can be taken to remove the distortion, such as by heat straightening measures. Unless exceptionally severe, such straightening does not need to be referred to the designer but it should only be carried out according to an established procedure. In some cases, where remedial work is difficult or impracticable, the designer may be asked to consider whether the actual deviation, in the particular location, is acceptable.

6.8 Correction of distortion

Sections can be straightened with the aid of hydraulic presses or using special bar bending or straightening machines. Some sections are too large for this type of straightening and it is necessary to adopt techniques involving the application of heat – so-called heat straightening. In this procedure, heat is applied to the face opposite to that with the welds which caused the distortion. The technique is based on the fact that if heat is applied locally to a member, the heated area will try to expand but will be constricted by the surrounding area of cold metal, which is stronger than the heated area. Upon cooling, the metal in the heated area gains strength as it shrinks, causing the member to curve, with the cooling face on the inside of the curve.

The application of heat has to be carefully controlled to prescribed temperatures and considerable experience is required before it can be successfully applied – overheating will cause metallurgical problems. The method of heat application is also used to straighten long strips of plate that have been flame cut along one edge, where release of the internal residual rolling stresses and the effect of the heat of the cut have caused curving during cutting. In this case, heat is applied in triangular areas on the edge opposite to the flame cut edge (see Figure 13). Out-of-tolerance distortions in plate panels can also be reduced by suitable local heating of the panel, sometimes combined with jacking to provide restraint.

6.9 Trial erection

Trial erection of bridge steelwork at the fabrication works has been a traditional way of ensuring that fit-up and geometry can be achieved at site, so reducing the risk of delays in erection or damage to protective treatment. With the much-improved accuracy achieved by automated fabrication procedures, the need for trial erection has been much reduced. Today, trial erection of most bridges is unnecessary and indeed complete trial erection of a large structure may be impracticable. However, where a delay in assembly at site due to mis-fitting components would cause unacceptable delays or consequences, or remedial measures would be extremely difficult, trial erection is of considerable benefit. This is particularly true for a railway or highway bridge that is to be erected during a limited possession.

Partial trial erection involving complex or close fitting connections, such as in skewed integral crossheads or the shear plate connections of the standard rail underbridge box girders, is also justifiable. This may also enable the steelwork contractor to position or adjust and weld some components of the connection, such as end plates, during the trial erection, as a practical way of achieving fit-up.

The extent of any trial erection should be considered carefully, bearing in mind that simultaneous trial erection of a large bridge off site may be totally impracticable. Full trial erection is inevitably on the critical path so, apart from the substantial costs, it adds considerable time to the fabrication programme. If a multiple span bridge is to be trial erected, partial or staged trial erections may be appropriate and will depend upon the amount of space which the steelwork contractor has available. Depending upon the degree of repetition and the fabrication methodology, trial erection of a particular span only may be sufficient. Often the need for full trial erection can be reduced or dispensed with once the early stages of erection have successfully been proved.

The designer, in considering the need for trial erection needs to evaluate the risks and consequences of delay at site – who would be most at risk, is it worth the client in effect paying a large premium for assurance for the risk involved? For his part, the experienced steelwork contractor will plan his fabrication, fit-up and checking procedures to minimise the risk to himself and the project.

See further comment in Section 7.7.

CHAPTER 7
COSTS

7.1 Introduction

Cost is an important consideration at all stages of bridge construction. The client's aim is to secure the new bridge to the desired standard as economically and efficiently as possible. However, the cost of the steelwork in a typical new highway or widening project is only 4-5% of the total cost; even for a project consisting mainly of the construction of a large bridge, the steelwork cost is only around 20%. The influence of the steelwork construction on the rest of the project is therefore of significant interest to the client.

Very little guidance has been published on the cost of bridge steelwork in the UK because it is difficult to generalise for the wide range of bridge types and configurations, and, of course, at any particular time prices reflect the state of the market. The price a client pays for the steelwork in a new bridge covers the cost of many activities and services as well as the basic cost of materials used and the direct workmanship in fabrication and erection.

The objective of this chapter is to provide guidance on relative costs and, more importantly, to show how the decisions and actions of clients and designers can increase or reduce the cost of bridge steelwork irrespective of the value and quality of the finished product.

7.2 Scope of work

The project requirements will include a range of technical and commercial documents – drawings, specifications, bills of quantities, programme and conditions of contract – all of which bear on the cost of the work to be done and the price to be paid. Understanding and anticipating the whole process of production from the outset of the project is essential for success. The main thing to remember is that 'everything has to be paid for'.

Bridge steelwork is usually constructed under a sub-contract. The scope of works for a typical sub-contract for steel bridgework covers supply, fabrication, protective treatment and erection. In some cases, design may be part of the sub-contract but the additional responsibilities that arise in such circumstances are not addressed in this publication. Each of the UK steel bridge contractors, in their own particular way, is able to undertake additional activities beyond this typical scope. Doing so can offer benefits for cost and risk management; for example if the provision of an in situ concrete deck on a small rail bridge is taken on by the steelwork contractor at the factory, or if the steelwork contractor were to supply and install the bearings.

The opportunity for synergy between main contractor and steelwork contractor depends on their respective skills and experience and requires thoughtful definition of contractual interfaces, for example in the provision of groundworks and access for delivery and erection. If the steelwork contractor is required to undertake work outside his normal experience, risks for the project could increase, leading to higher costs – in time and money.

As a general rule, in defining the scope of work for each party, it is best to place control of costs and risk with the party which has most relevant experience and opportunity to deal with them.

The scope of work will be defined in detail by the drawings, the specifications, the bill of quantities and the method of measurement for the project.

7.3 Efficiency

7.3.1 Efficiency of the design

Efficient design of steelwork will use the steel components to their safe design capacity while at the same time making economic use of resources for fabrication and erection. Short-cuts taken to reduce design costs, by calling for a fillet weld size that has a strength equal to that of the plate, rather than the strength to carry the actual design forces, for instance, can have a significant effect on fabrication costs.

Traditional methods of measurement and bills of quantities encourage the thought that simple measures such as 'rate per tonne' can be used to evaluate schemes. Only a small percentage of the overall cost of bridge steelwork can be related simply to tonnage, as the raw materials are typically less than 30% of the overall cost. Even for the basic steel material cost, there is not a simple rate per tonne. Many factors such as grade, source, section size, plate length, plate width, test requirements and quantity all have a significant effect on the price of the material from the mills – and before such factors as wastage are taken into account.

The cost of bridge steelwork is influenced directly by the details of the design and the configuration of the structure on the site. These factors vary greatly, so a designer needing realistic estimates would be well advised to discuss his options with appropriately experienced steelwork contractors, even if only at an early stage of design, to assess budget figures. Budget rates per square metre of finished bridge deck will give a far more accurate estimate of cost than any guessed rate per tonne applied to a budget estimate of weight.

The cost per square metre of finished bridge deck will depend on various factors including structure type, span configuration, design loading, protective treatment system and erection method. A steelwork contractor with a comprehensive track record of carrying out the type of work involved should be able to provide a designer with an appropriate range of budget rates per square metre of deck.

Advice may vary between steelwork contractors due to differing expertise and fabrication facilities; so choosing a steelwork contractor with relevant experience and capability for the type and size of bridge envisaged is important. nevertheless, any guidance on cost for fabrication or erection must be treated with care.

For designers contemplating alternatives to plate girders, Table 9 gives is a typical breakdown of costs per tonne relative to plate girders at a datum of 100. These figures are only indicative and considerable variations are to be expected for any particular bridge.

Steel bridges with rolled section main girders are uncommon now, as they tend to be heavier than plate girder structures and offer no benefit on a rate per tonne basis. They are uncompetitive, except possibly for very small span highway bridges and footbridges.

Box girders are inherently more expensive to fabricate, especially with need to recognise the hazards of working in confined spaces and the measures necessary to deal with them. Protective treatment and erection costs for box girders can be reduced through the use of weather resistant steels and careful choice of site connections.

The minimum cost design is very unlikely to be the lightest design; certainly, it will not be the heaviest. Weight-saving is not an end in itself: for example, flange weights can be reduced at the expense of additional butt welds, or webs can be thinner if more stiffeners are added but this may cost more, as discussed in Section 3.3.1. Achieving the most efficient design requires careful selection of concept and layout, and judgement in drawing the balance between weight and workmanship in detail.

It is important to ensure that welds are properly sized to provide the design resistance and no more (although it is uneconomic to make multiple changes in weld size between flanges and webs). The savings in design effort by over-specifying weld sizes will be outweighed many times over, by the extra welding costs and the cost of correcting distortion. Cost factors for different sizes and types of weld are shown in Table 10 and Table 11.

Avoid the unnecessary specification of full penetration butt welds. For example, in the corners of box girders, a partial penetration weld plus a fillet weld is normally adequate. If the weld size is disproportionately large relative to the thickness of the plates being joined, the resulting rotations and shrinkages will also be disproportionate.

7.3.2 Production efficiency

Within the contract programme, the steelwork contractor needs sufficient time to do all that he has to do. Many activities are in his direct control but others such as the supply of steel are not. Typical fabrication periods vary from four weeks from receipt of steel for small simple structures to many months for large complex structures. The necessary period depends on the capacity of the factory and on its workload.

For most bridge steelwork, the time required to complete the work can be reduced by special measures or by taking risks, but such measures come at a price. It is important therefore not to compress the steelwork programme below the allowance of sufficient time for economy without recognising the enhanced cost and risk to the project.

In any project using fabricated steelwork, it is surprising how few people outside the factory doors understand what goes on within them, even when their roles give them an active interest or part in the process. The fundamental differences between production in a factory and production of civil engineering works on site, and in particular in their cost structures, lead to misunderstandings and commonly to unintended commercial surprises. Mutual recognition and understanding of the differences help relationships, control costs and ensure better value for the client.

The factory, which represents a large fixed overhead, is laid out with covered space, cranes, machinery and equipment to suit the steelwork contractor's product range. Steel bridge components are heavy and bulky, so the factory is designed to lift, move and manipulate the steelwork economically and safely between production stages. This non-productive work and the occupation of floor space represent a substantial proportion of bridge fabrication costs. Unlike a site, the factory works on a number of projects in parallel to achieve profitable utilisation of the factory and its permanent workforce. It is by doing this that it is able to offer steelwork at competitive prices.

TABLE 9 - Relative cost rates for different girder types

	Bridge Type		
	Universal beam	**Plate girder**	**Box girder**
Steel materials	45	30	30
Fabrication	35	40	60
Protective treatment	15	15	25
Erection	15	15	25
Total	**110**	**100**	**140**

Note:

All costs relative to a total cost per tonne of 100 for a plate girder bridge

TABLE 10 - Relative cost of fillet welds, according to size

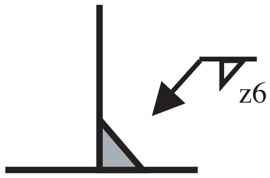
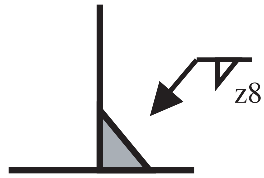
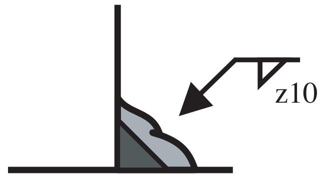
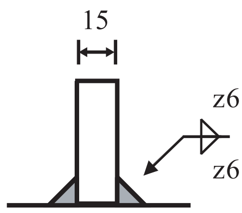
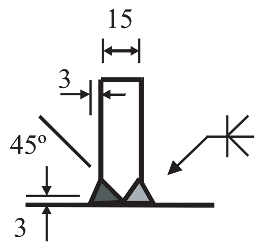
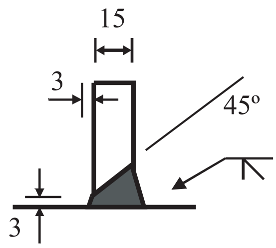
Size			
Cost factor	1	2	3

TABLE 11 - Relative cost of welds in Tee connections

Size			
Cost factor	1	2	3

Materials and fabricated steelwork have to be stored at the factory and financed until they are paid for, so costs will rise if slippage in delivery dates requires finished items to be stored for long periods. Most factories have a limited amount of storage space under overhead cranes, so it might be necessary to hire mobile cranes to move girders into and out of storage.

Full design information is not necessary to enable orders for steel material to be placed but sufficient detail must be available to define all components in advance of rolling dates, to minimise waste and costs. The information required at this stage is:

Geometry	Plan layout
	Levels at bearings
	Final profile of top flange
	Haunch definition (if any)
	Allowances for permanent deformation
	Tolerance on vertical profile (if any)
	Requirements for web to flange welds (for estimating effects of weld shrinkage)
	Orientation of splices and girder ends
Make-up	Plate and section dimensions
	Bolts – quantity, type, diameter, coating
	Shear studs – quantity, length, diameter
Specification	Steel grade for different thicknesses of plate and section
	Restrictions on butt weld positions (if any)
	Options on tolerances, surface conditions, testing, carbon equivalent value etc (if any)

Typical lead times (from order to delivery of materials) vary between six and 12 weeks for plate and between six and 16 weeks for larger sections and tubes. To implement the design of the steelwork efficiently and in a timely manner, it is vital that the design drawings express the designer's intent clearly and completely at the outset and certainly before the commencement of CAD modelling. Typically, this information is required between four and six weeks before fabrication starts.

Before fabrication can begin, the work has to be planned and programmed, drawings and production data produced and materials ordered. The target is to provide precise data to the shop floor by the planned date, and then to maintain the flow of data and material to meet the programme without delay or disruption.

Few projects reach their conclusion without the need for change. The cost effects become more severe the later they occur in the fabrication programme. The relationship between cost of change and the stage at which it takes place is shown in Figure 14.

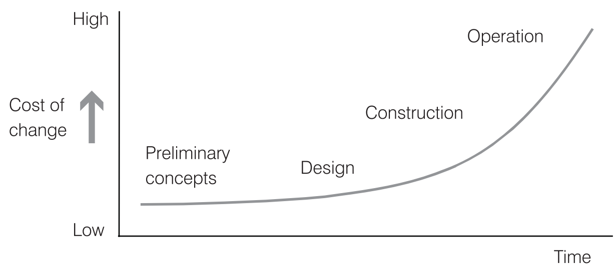


Figure 14 - Cost of changes

Even quite modest changes can have substantial time and cost consequences, for they not only put the project fabrication programme at risk, but they can disrupt work on other projects in the factory at the same time. For example, the installation of a few extra stiffeners in a girder may have to be done outside the normal fabrication area to avoid disrupting main production work. Changes often attract additional costs from the following:

- Piecemeal 3D modelling and CNC programming;
- Handling of the girder from storage to a suitable working area;
- A less efficient manual welding process;
- More difficult positional welding if unable to turn the piece;
- Delays from bad weather;
- Sheeting for protection from weather;
- Removal and repair of corrosion protection.

7.4 Complexity

In general, fabrication is more economic if connections are simple, geometry is straightforward, and the amount of welding is minimised. Modern CAD systems and CNC fabrication equipment can well accommodate complex geometry, easing the drawing and production processes, but they cannot eliminate all the extra costs in workmanship, organisation and management. In detailing, every additional component, cutting operation, assembly operation and weld adds to the cost, so it pays for the designer to consider the options and anticipate relative costs in making his choices.

To achieve good quality design that satisfies all criteria is not easy; it requires skill in the use of steel at the concept and detailed stages of design. Some designers are more experienced in steel bridge design than others; greater experience should lead to simpler and more economic design, which will reduce total costs.

Added complexity may increase fabrication costs. For example, creating too many types of stiffener prevents multi-headed profiling and unnecessary shaping of stiffeners prevents multi-headed stripping. Other components such as bracing members should be detailed for simplicity, with the aim of minimising the number of pieces to be cut. The ends of bracing should be cut square rather than mitred and single members should be used in preference to back-to-back members. The introduction of plate bending adds an activity which the steelwork contractor may have to sub-let; time and cost effects need to be considered.

High deposition automatic or semi-automatic welding processes are used wherever possible in shop assembly, normally using metal active gas (MAG) or submerged arc (SAW) processes. Manual metal arc (MMA) or stick welding may be the only choice for complex joints and details – a choice that is imposed by the complexity of design and where details have not taken account of how welds can be made. Generally, the route to minimum cost of welding in the UK is to maximise the use of automated equipment in the factory and minimise the use of welding on site.

Haunched girders will normally have to be assembled and welded by hand. Where possible, girders should be designed such that they can be fabricated in a T and I machine because it offers the following advantages:

- Speed of assembly;
- Better web-to-flange fit-up;
- Mechanical control of cross-section squareness;
- Double-sided welding, reducing flange rotation;
- Capacity for deep-penetration fillet welding.

Generally, the more complex the fabrication details are, the more expensive they will be. Different steelwork contractors will have different machines. Connections should be simple, avoiding the need for manual assembly and machined surfaces.

7.5 Buildability

The buildability of structural steelwork is a cost issue as well as a safety issue. The fulfilment of the designer's obligations under CDM to reduce exposure to hazards should also contribute to overall economy.

For construction work on site, the structural layout, member sizing and connections need to be consistent with a practicable economic erection method. Site constraints on time, say for railway or highway closures, and physical restrictions on access, crane position, temporary supports and the like, will determine what that method will be. Connections should be detailed to facilitate site fitting, with some rotational and dimensional tolerance, bearing in mind fabrication tolerances and the difficulty of precise prediction of deformations due to permanent loads. Adequate tolerances in the connections are important for safety and cost, and can be met readily with well thought-out bolted joints.

Good detailing is the key to ensuring buildability. It ensures that bolts can be readily entered and tightened, and that there is satisfactory access and visibility for welding and for applying effective protective treatment.

The cost of lifting and turning large assemblies, particularly box girders, to facilitate welding can be reduced by careful design of the connections of webs and flanges and the layout of the stiffening. Similarly, the avoidance of hazardous and high-cost work in the interior of box girders is very dependent on the thought given to internal details.

The earlier the steelwork contractor is involved in the development of the design, the cheaper the final product is likely to be.

The effects of late provision of bearing fixing information can be offset by uncoupling the manufacture of the bearing plate from that of the main girder. Welding on a plate with tapped holes is considerably cheaper than manually drilling holes in the bottom flange of the main girder for through bolts (which would be necessary if bearing details were not available until after the girder had been fabricated).

Detailing the steelwork to accommodate a particular erection method, for example, providing additional stiffening at lifting points, will be less costly if detailed at the outset, rather than being introduced as an afterthought.

Requirements for improved through-thickness properties to reduce the risk of lamellar tearing are usually very local in nature, so it is likely that only small quantities will be needed on any particular project. (In low and medium risk situations, the steelwork contractor can deal with the risk but in high-risk joints the designer will need to specify a Z grade.) Specifying a Z grade will add to costs, because the supplier may charge a premium for small quantities, the grade is more expensive and is less readily available: it is better to design details that do not require the use of steel with improved through thickness properties, if possible. Lack of availability can be dealt with by carrying out tensile strain tests on normal steel, but again this has to be paid for.

It is generally considered beneficial to the performance of the protective system to minimise site application and to eliminate it as far as practicable. There is a good case for leaving just the connections to be painted at site, and applying finish coat on the few surfaces that impact on the overall appearance of the bridge. Even when damage occurs during transport, erection and construction of the deck slab, the remedial measures should cost far less than application of the full top coat on site.

Application under site conditions is generally less favourable and more expensive than shop application because:

- Painting is normally suspended during winter months, as the required temperature and humidity criteria are rarely achieved;
- The exclusion of dust on a construction site during summer months is difficult;
- Safe access to erected steelwork and the encapsulation of areas being painted are expensive;
- Site-welded joints require blasting, which is a very aggressive process and requires containment and careful control.
- Painting over live highways or rail tracks in closures and possessions is generally slow and very expensive due to the limited closure periods and the increased risk of inclement weather at night when much of the work is done.

7.6 Specification

It is important that a project specification (and accompanying drawings) should express clearly the particular requirements for a structure and, where standards allow options and alternatives, which additional requirements apply. Failure in clarity will lead to extra provisions for risk and extra costs in resolving queries. The project specification should also avoid over-specification - requiring unnecessary quality and excessively tight tolerances will lead to higher costs. The project specification should generally follow recognised industry standards, such as that in the Steel Bridge Group's *Model Project Specification*, which offers a set of clauses that can be readily adapted and incorporated into individual project contracts

7.7 Trial erection

Trial erection has a major effect on the length of the fabrication programme. A member required for trial erection cannot be released for painting and delivery to site until the last member is fabricated and the trial erection has been checked. Generally, the considerable cost of man-hours, craneage and space in the works is non-productive, save for the assurance of minimising the risk of delay on site. Given the modelling techniques used now, the accuracy of modern fabrication machinery and the checks carried out during fabrication, trial erection and the associated cost can often be avoided by checking components during fabrication for gross errors.

7.8 Commercial issues

The price to be paid for steelwork will reflect not only the costs and risks inherent in the explicit requirements of the project documents, but also less tangible factors such as the relationships between the parties and market conditions.

As with any specialist work, the perceived degree of risk will depend upon the trust that each of the parties has with the others. Where the parties (client, designer, main contractor and steelwork contractor) have worked together successfully in the past by keeping to contractual agreements such as payment terms and by maintaining good working relationships, greater certainty of cost and fewer post-contract claims can be expected. The prices tendered will reflect that.

Market conditions will affect the steelwork contractor's costs, for example in the price and availability of material or labour and plant in remote locations. The contractor's margins will reflect the state of his forward workload and the overall demand and supply balance. However, no steelwork contractor can afford to be influenced by the immediate situation when considering projects that might become orders in the medium or long term.

The conditions of contract particular to a project and the related sub-contract conditions introduce elements of cost for the steelwork contractor that are reflected in his price. Comparatively aggressive terms and conditions will result in higher prices.

Factors which increase the cost of structural steelwork include the financing of materials and fabricated steelwork at the factory until it is paid for, and the risk provision for unreasonable damage clauses that are disproportionate to the size of the sub-contract. The provision of advance and interim payments for steel and steelwork should be included where it is required that work be done well in advance of erection dates. As with all other aspects of project requirements, the application of the contract conditions to a project needs to be thought through to ensure that they are in the best interests of achieving a successful project, even down to minor considerations such as times for agreement/approval. Price is related to risk, and offloading risk on to the contractor or sub-contractor increases the price to be paid for the work.

CHAPTER 8
CASE STUDIES

8.1 Introduction

This chapter describes nine recently constructed highway and railway bridges. A variety of forms of construction are covered – composite plate girders, open-top box girders, cable stayed bridges and truss bridges. Most were built within challenging constraints on site activities and illustrate the optimization of erection methods achieved by close cooperation between designer and steelwork contractor.

For examples of other projects visit www.SteelConstruction.org/resources/design-awards

8.2 M50 Upgrade, Dublin, Structure S13

Client:	**M50 Concession / National Roads Authority Ireland**
Designer (superstructure):	**Hewson Consulting Engineers Ltd**
Designer (substructure):	**Eptisa International**
Main contractor:	**M50 Construction**
Sub-contractor:	**Andrew Mannion Structural Engineers Ltd**
Completion date:	**2009**
Approximate steelwork weight:	**550 tonnes**

The M50 is the busiest road in Ireland, with an average daily traffic flow of approximately 100,000 vehicles, split equally in each direction. The M50 Upgrade project incorporates carriageway widening to three lanes and a hard shoulder, and free flow interchange layouts. Structure S13 is located at the N3 junction and carries two lanes of traffic from the northbound M50 slip road to the eastbound N3 to facilitate the free flow interchange. Due to the geometric constraints of the revised junction, the bridge alignment incorporates a transition curve and has a maximum skew of 60° to the M50. The curve and skew of the bridge required a torsionally stiff deck and therefore a pair of trapezoidal box girders was selected for the main beams. Using steel beams meant that motorway closures were kept to a minimum, as large sections could be lifted into place.

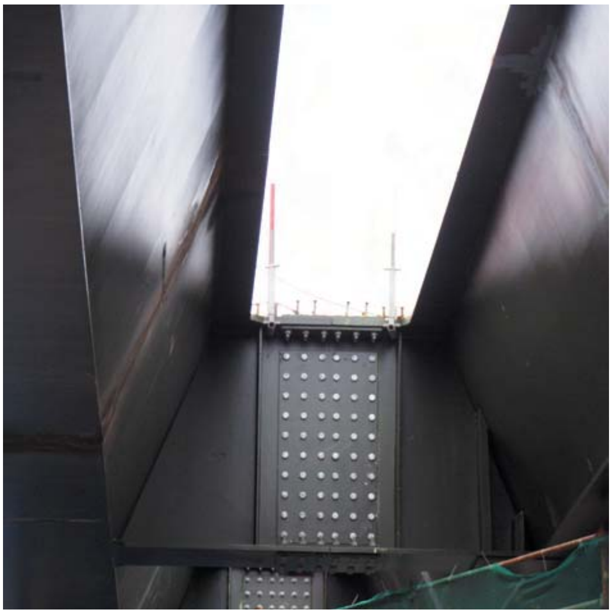
The bridge is a 115 m long continuous two-span structure with a maximum span length of 57 m. The deck is 13 m wide, with an 8 m two-lane carriageway and 1.25 m verges. The verge width on the southeast corner widens to over 2 m, to maintain sightlines. See Figure 15 and Figure 16. The bridge is supported on six Fressyinet bearings, two on each abutment, either side of the M50, and two on a pair of concrete piers in the median. Stiffened jacking locations have been provided adjacent to each bearing.

View of Lift 3 being placed

Soffit View of Lifts 3 and 4

The trapezoidal box girders are 2 m deep; the depth was limited by clearance to the motorway. The boxes are fully welded, with only one site splice per girder, following the lifting operations. The two open-top box girders are braced at top flange level by a substantial modified warren truss to provide stability around the curve in the temporary condition. The two boxes are connected by four full-depth cross girders: two square to the box over the pier locations and a skew girder at each abutment. Cross girder connections were made with preloaded bolts, while the main girders were completed with full penetration butt welds.

The deck slab of the bridge is a combination of full and part depth precast concrete slabs with in situ stitches. The full depth units also incorporated the parapet up-stands and fascia beams. This provided further timesavings on site and reduced working at height. Shop application of shear studs was carried out within tight tolerances to ensure a good fit with the pockets in the precast slabs.



Typical Bolted Splice

View from South West Corner

The geometry of the bridge was a key driver in the development of the design. The effect of the skew in combination with the transition curve meant that the skew of the supports varied from 42° to 60°. The skew supports also induced an additional torsion into the box girders. This was in part overcome by offsetting the bearings to the outside webs of the girders. The transition curve added further complexities, as the radius of the curve and cross fall of the deck varied throughout the west span. In order to ensure that the correct geometry was achieved following the installation of the beams, each web line was set out individually for line and level. The level values were then adjusted for the pre-camber of each web. Distortional effects are primarily controlled by cross frames at 6 m centres, with intermediate ring frames at 2 m centres.

Given the importance of the M50 to all traffic in Dublin, road closures were only available between 10 pm and 8 am on Friday and Saturday nights and between 12 pm and 6 am on all other days. As there was no available land at either end of the bridge to enable a launched solution to be used, it was necessary to design the structure such that the girders could be erected in just four lifts.

Each of the first two lifts was 63 m long and weighed 160 tonnes; both were erected in a single night-time possession of the M50. The girders were installed by tandem lifts using a 700 t and a 350 t cranes. Limited space at the site meant that only one of the lifts could be assembled adjacent to the bridge site. The second lift was assembled and welded together in a remote yard 2 km south of the bridge site. On the night of the lift the motorway was closed at 10 pm, at which time the cranes could move into place and complete rigging for the lifts. While the first lift was being placed, the second lift was transported along the M50 to the site in time for it to be lifted into place. The three cross girder beams linking the two box sections had to be spliced together with a total of 350 bolts before the second lift could be released and the cranes demobilised in order for the road to be open for 8 am.

Lifts 3 and 4 involved performing an aerial splice over the M50 using temporary support frames in order to enable a fully welded splice to be obtained. Given the complexity in achieving these splices accurately, these lifts were undertaken on two subsequent Fridays. Each of the lifts was 50 m long and in excess of 115 t; the two box sections had to be transported 8 km from the assembly yard to the site. Again, each lift was placed by tandem lift using the 700 t and 350 t cranes in a 10-hour possession of the M50. Final welding of the splices was completed at night within additional road possessions.

Fabrication of the deck steelwork began at the end of September 2009 with the first lifts on the weekend of 20 November and completion on 4 December 2009.

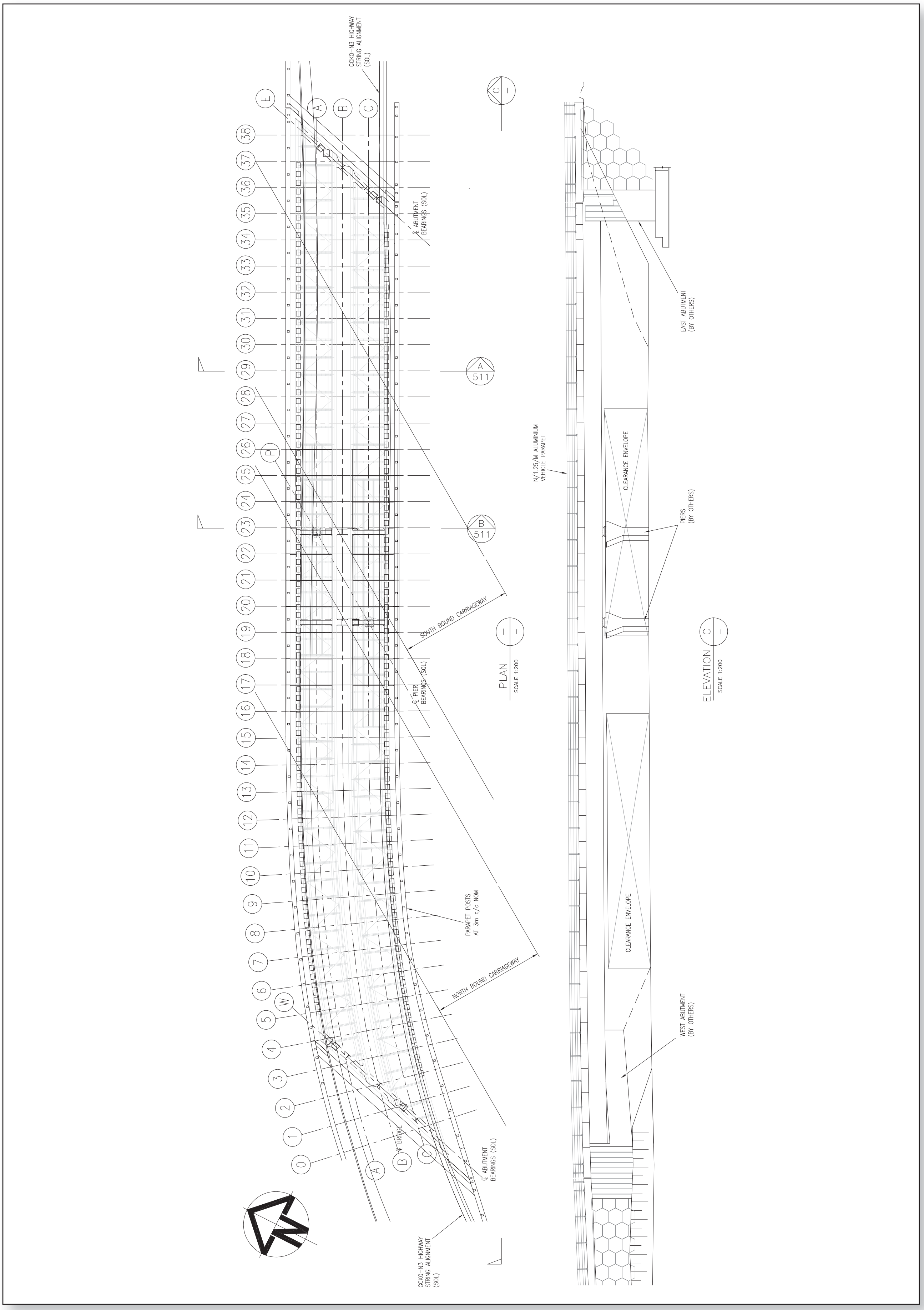


Figure 15 - Structure S13 - Plan and elevation

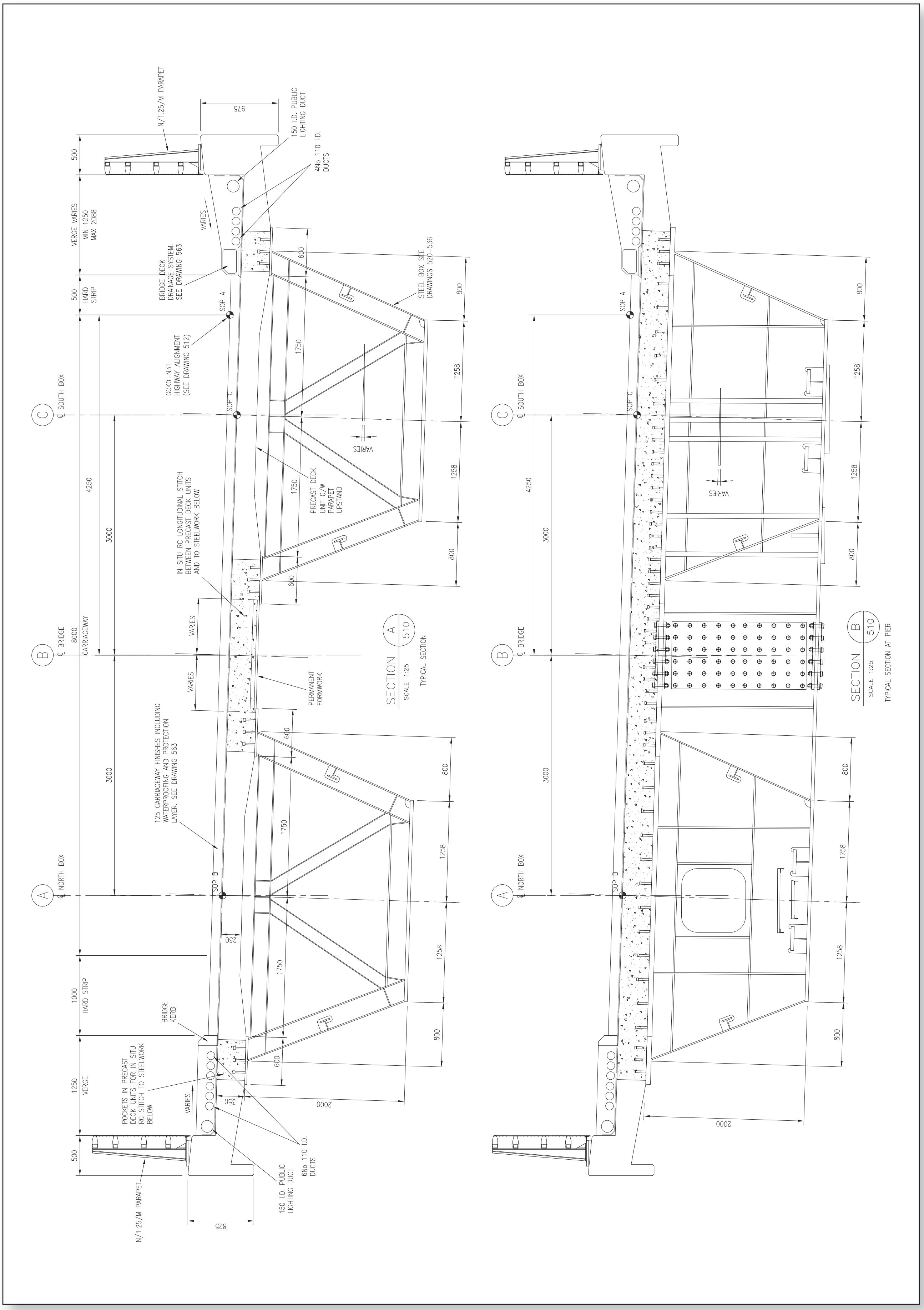


Figure 16 - Structure S13 - Cross sections

8.3 A2/A282 Intersection Improvement

Client:	**Highways Agency**
Designer:	**Jacobs**
Contractor:	**Costain**
Sub-contractor:	**Mabey Bridge Ltd**
Completion date:	**2007**
Approximate steelwork weight:	**4,000 tonnes**

The A2/A282 intersection in north Kent is used by nearly 200,000 vehicles every day. In 2005, the Highways Agency placed a contract for a £120M major improvement scheme at the junction, to reduce congestion and improve journey time reliability at Junction 2 of the M25 at Dartford. The project involved widening the A2 to four lanes in each direction over a distance of approximately 2km. It also added a lane in each direction to the A282, which is in effect a continuation of the M25 between the M20 and the northern entrance to the Dartford crossing in Essex. Work on site began in September 2006.

More than 4000 t of weathering steel was used for the construction of four new bridge structures, representing one of the largest single uses of this material in the UK. One of the main considerations for the project was that the bridges should need as little maintenance as possible because access constraints made such work difficult and expensive. Repainting the bridges in the future would be difficult logistically, particularly because one of the structures crosses both the A2 and A282.

One structure provides the main east-to-north link flyover; it is a 420 m long, nine-span viaduct with more than 1,800 t of steel. Two parallel five span structures of 1,000 t of steel each run on either side of the existing A2 and are 250 m long: one carries traffic from the A2 over the Darenth Valley, while the other feeds traffic onto the A2 from the A282 southbound. The fourth structure required 230 t of steel; it was the smallest of the structures and carries southbound A282 traffic over a small slip road before linking up with the A2.

Erection of the east-to-north link flyover began in February 2007. Four weeks were used to assemble some sections on site and the majority of steelwork was lifted using a 1000 t telescopic crane, the largest available in the UK.

The middle section of the east-to-north link was erected first, followed by the remaining spans toward the south abutment, completing six of the nine spans in one sequence. The remaining spans toward the north abutment were erected thereafter. On 19 May, the project reached a critical milestone with the final section of the structure lifted in overnight to complete the viaduct.

The nine spans carry the structure over eight piers. Five of the concrete-formed piers have four columns while the three middle piers only have two columns. The middle three spans traverse open ground and consequently there was enough room for temporary trestles to stabilise the steelwork during erection.

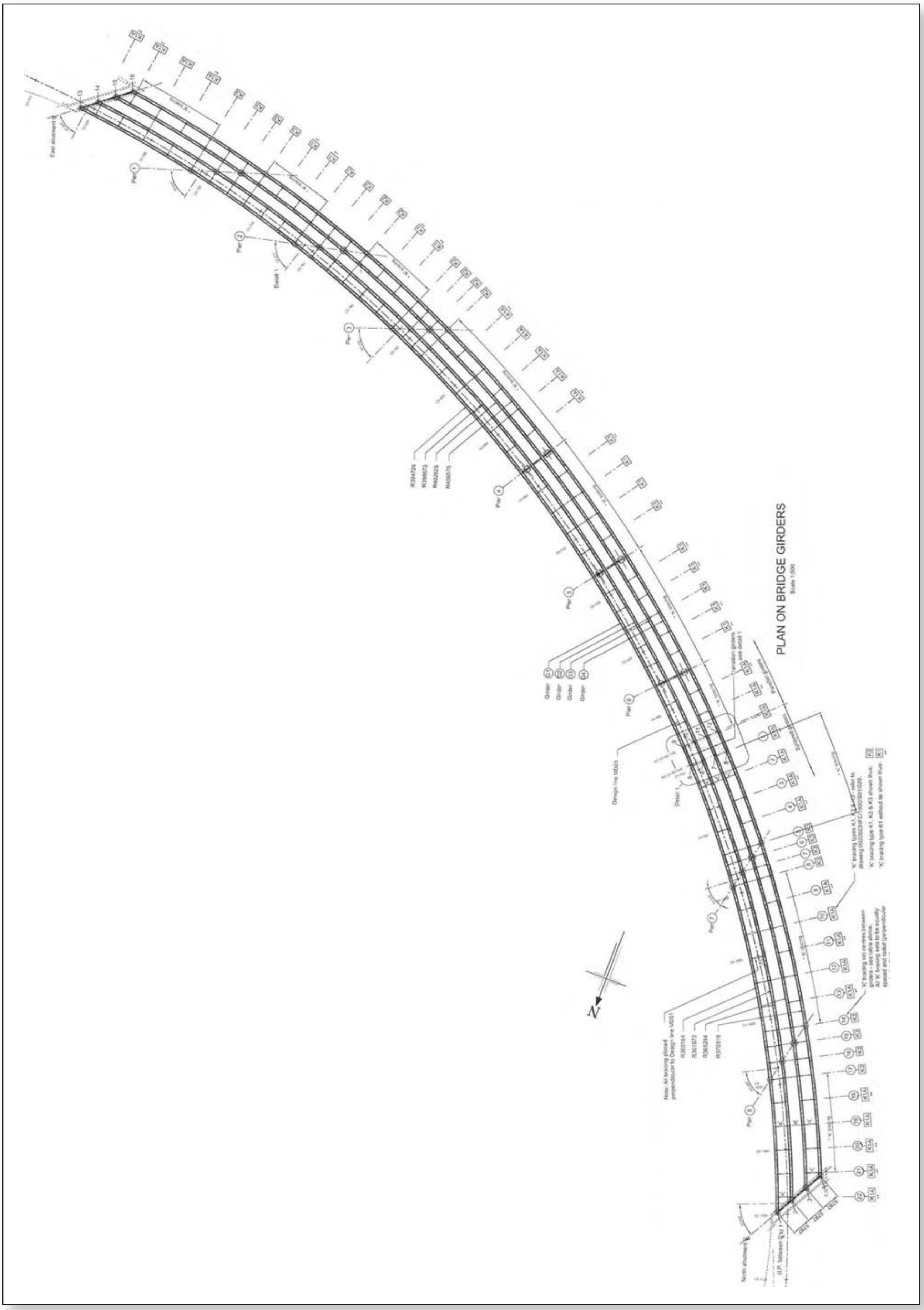


Figure 17 - A2/A282 Intersection, East to North Link - General arrangement

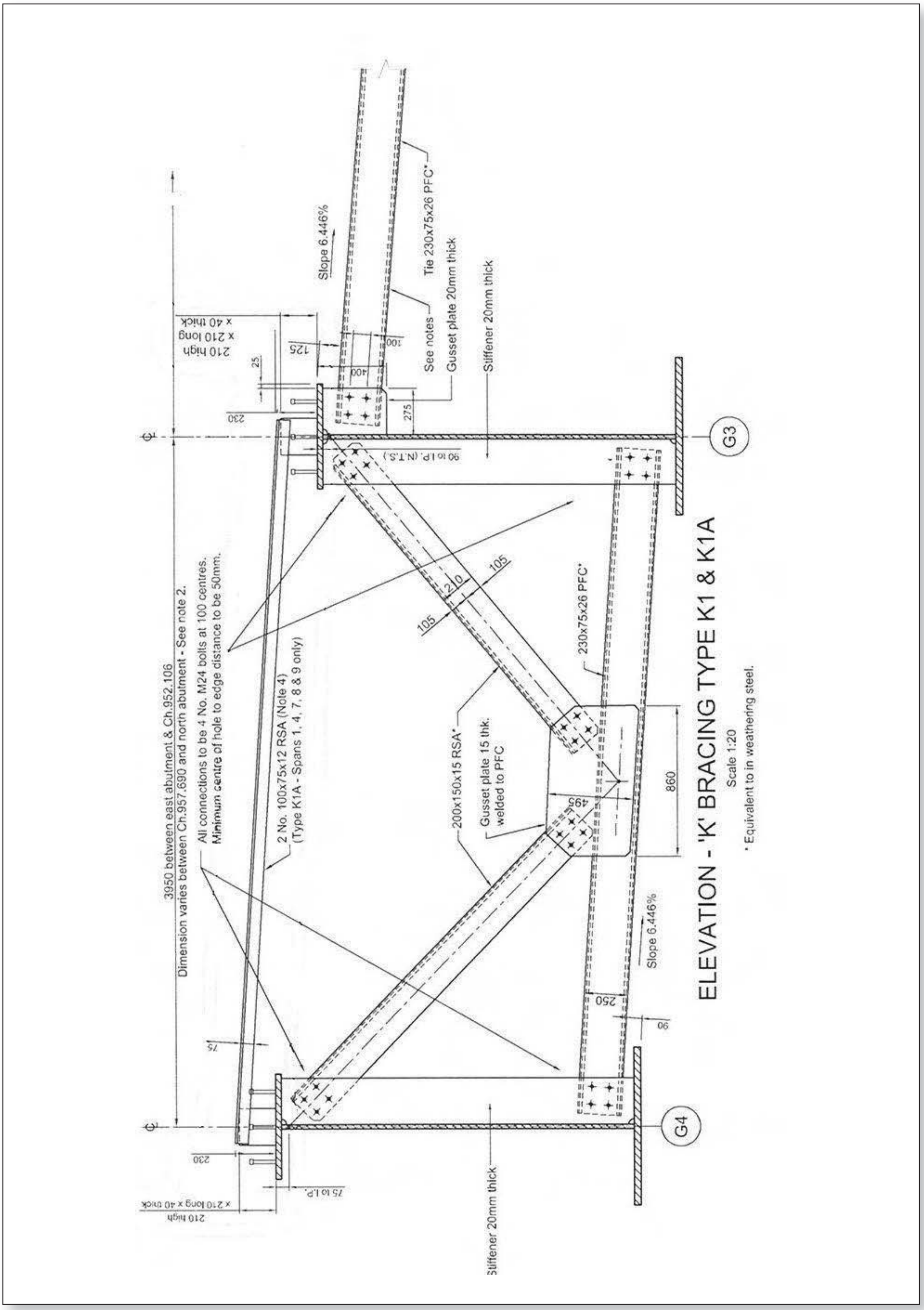


Figure 18 - A2/A282 Intersection, East to North Link - Typical bracing details

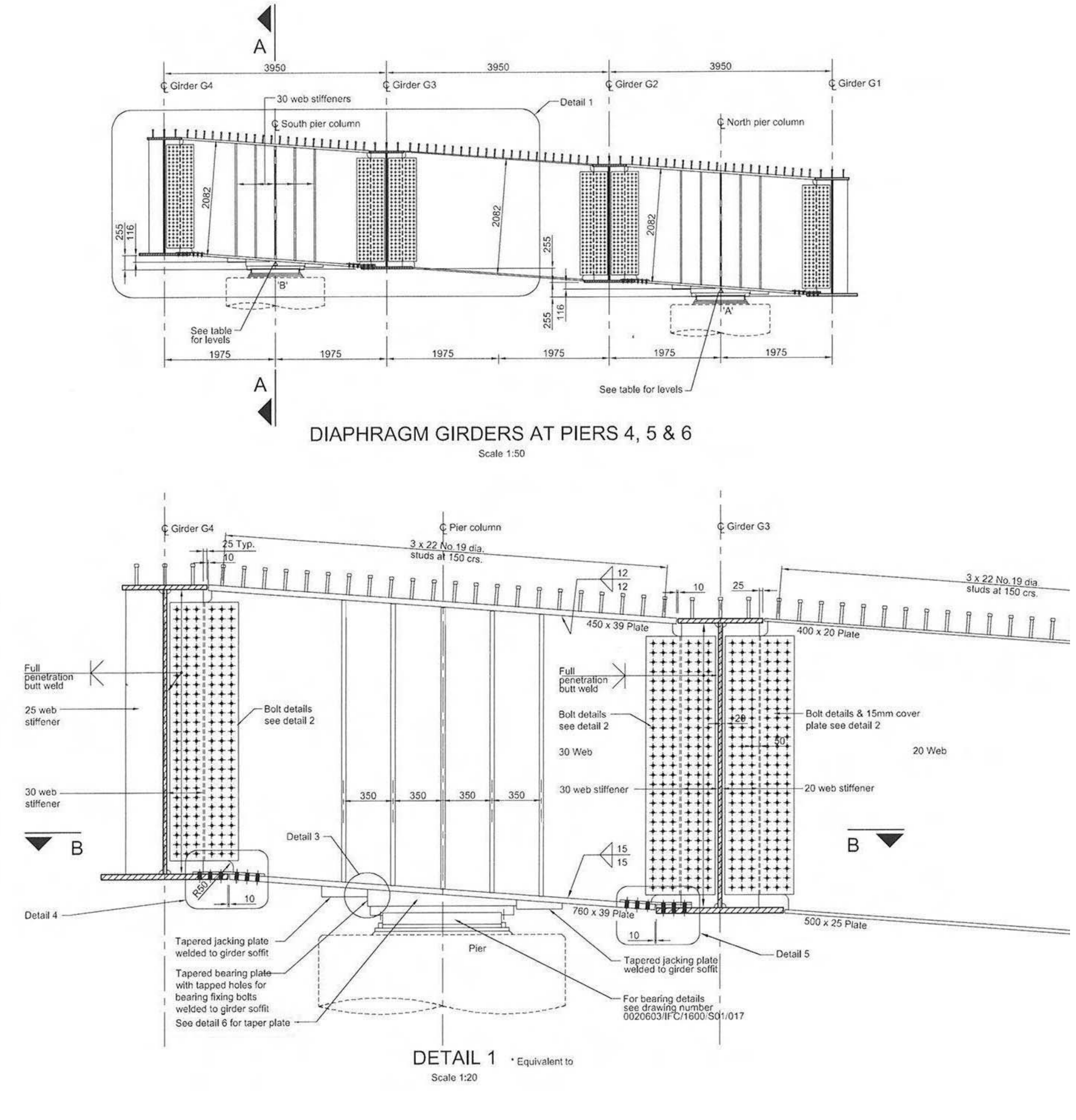


Figure 19 - A2/A282 Intersection, East to North Link - Pier diaphragm details

8.4 East London Line, Bridge GE19

Client:	**Transport for London (TfL)**
Designer:	**Scott Wilson (Benaim)**
Contractor:	**Balfour Beatty Carillion Joint Venture**
Sub-contractor:	**Mabey Bridge Ltd**
Completion date:	**2008**
Approximate steelwork weight:	**812 tonnes**

The East London Line extension is part of the new London Overground network. London Overground services, which will also include the North London Railway, will be shown on the Tube map and deliver an Underground-style service.

Bridge GE19 is an 84 m span, 8 m deep Warren truss steel bridge, carrying the twin tracks of the East London Line. The structure is over the Great Eastern main line in East London, crossing six tracks. Bridge GE19 was assembled behind the east abutment and launched during a 52 hour possession.

Steelwork forming the deck was made up of 10 longitudinal beams, between 16 m and 20 m in length, weighing between 20 t and 24 t each, and 31 cross girders, each 9 m long weighing 4 t. The side trusses were formed by the longitudinal beams, 20 diagonal members, 5.6 m long and weighing 3 t, and 10 top beams, 11 m to 20 m in length, weighing between 15 t and 30 t. The main beams and the side trusses were trial erected at Mabey Bridge's works.

The top plan bracing between the two side trusses, which was made from 19 girders, 9 m to 12 m long weighing 3 t, was also trial erected.

In addition, the nose used to install the bridge via launching was constructed from 30 girders and 22 bracing members, weighing a total of approximately 150 t.

The bridge was launched from the eastern abutment. Hydraulic strand jacks at the rear of the bridge climbed along strands anchored to the eastern abutment. The bridge moved along a rocker on the abutment with a multi-axle trailer to its rear. When the bridge's 40 m temporary steel nose reached the western abutment, 320 t of steel counterweight at the rear of the bridge was removed. At this point, the bridge had travelled 82 m at an average speed of 6 m per hour. The structure was then launched a final 41.1 m until it was in its permanent location. Temporary restraints were attached to secure the structure

The bridge was then jacked down onto its permanent bearings before handing over to the main contractor to complete the concreting of the deck.

First train crossing Bridge GE19

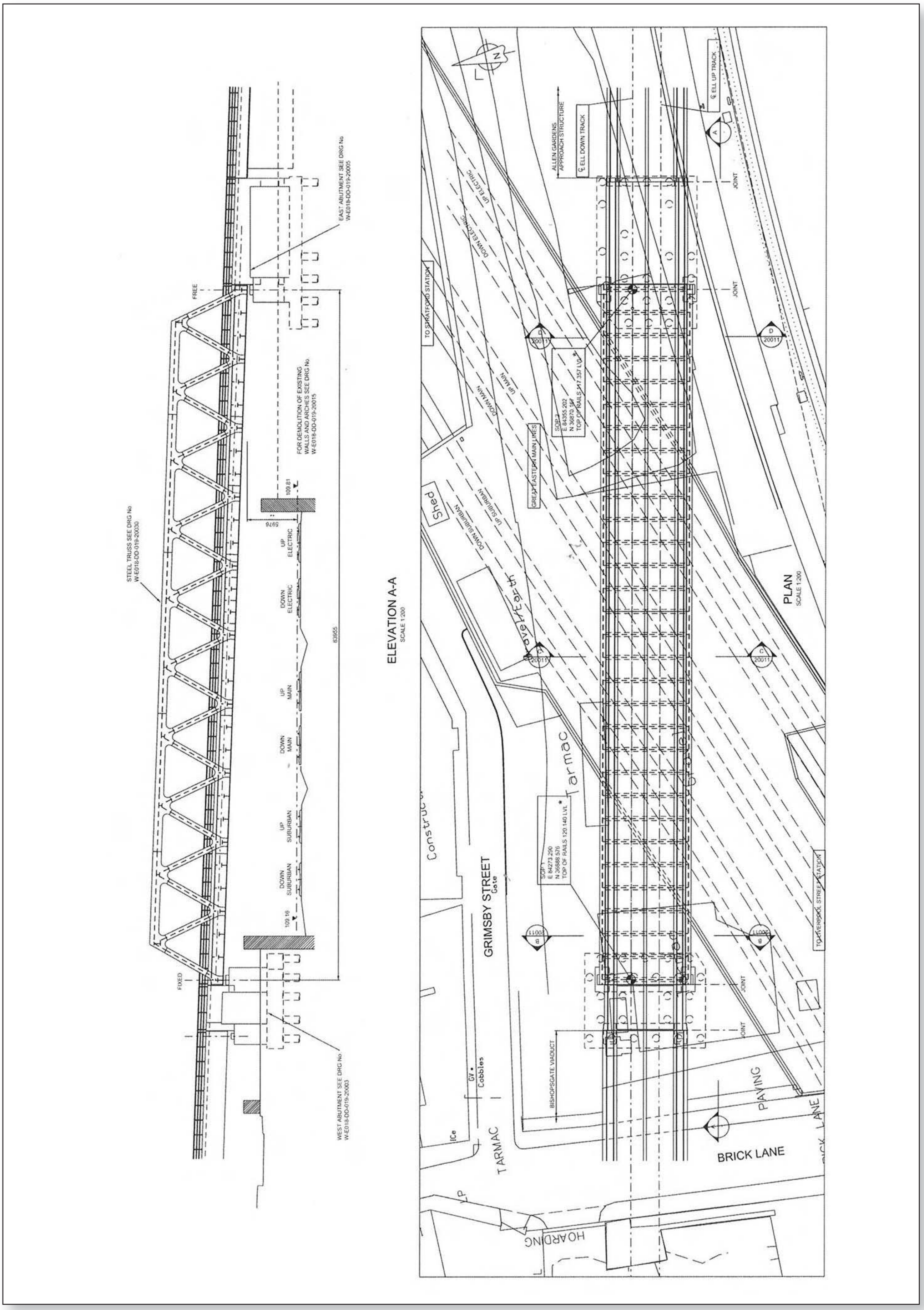


Figure 20 - Bridge GE19 - General arrangement

8.5 East London Line, New Cross Gate Bridge

Client:	Transport for London (TfL)
Designer:	Scott Wilson
Contractor:	Balfour Beatty Carillion Joint Venture
Sub-contractor:	Mabey Bridge Ltd
Completion date:	2008
Approximate steelwork weight:	602 tonnes

Like bridge GE19, New Cross Gate Bridge is part of The East London line extension of the new London Overground network. The bridge is a 75 m long 8 m deep steel Warren truss carrying the new East London Line link over the Network Rail London to Brighton Lines.

Steelwork forming the deck was made up of eight longitudinal main girder sections, each 20 m long and weighing between 20 t and 25 t, and 37 crossbeams, each 10 m long and weighing 3 t. The side trusses were formed by the main girder sections, 36 diagonal beams, each 6 m long and weighing between 2 t and 4 t, and 12 top members, varying in length and weight. The truss was braced in plan at the top chord by 25 bracing members.

The bridge steelwork was trial erected at Mabey Bridge's works. The deck crossbeams were machined to length to ensure the correct fit-up within the end plate connections.

The steelwork was delivered to site piece-small and assembled parallel to the existing rail infrastructure. The process began with the installation of temporary support stallages. Following on from the stallage placement, the steelwork forming the deck was fully assembled. The top chord of the truss was then assembled, and the whole of the structure fully bolted. Once the steelwork was fully assembled, the deck was cast and when it had achieved sufficient strength, the structure was jacked up to approximately 6 m above the ground and self propelled modular transporter units installed. During a full possession of the track, the structure was rolled across the protected tracks and lowered into place over the bridge abutments.

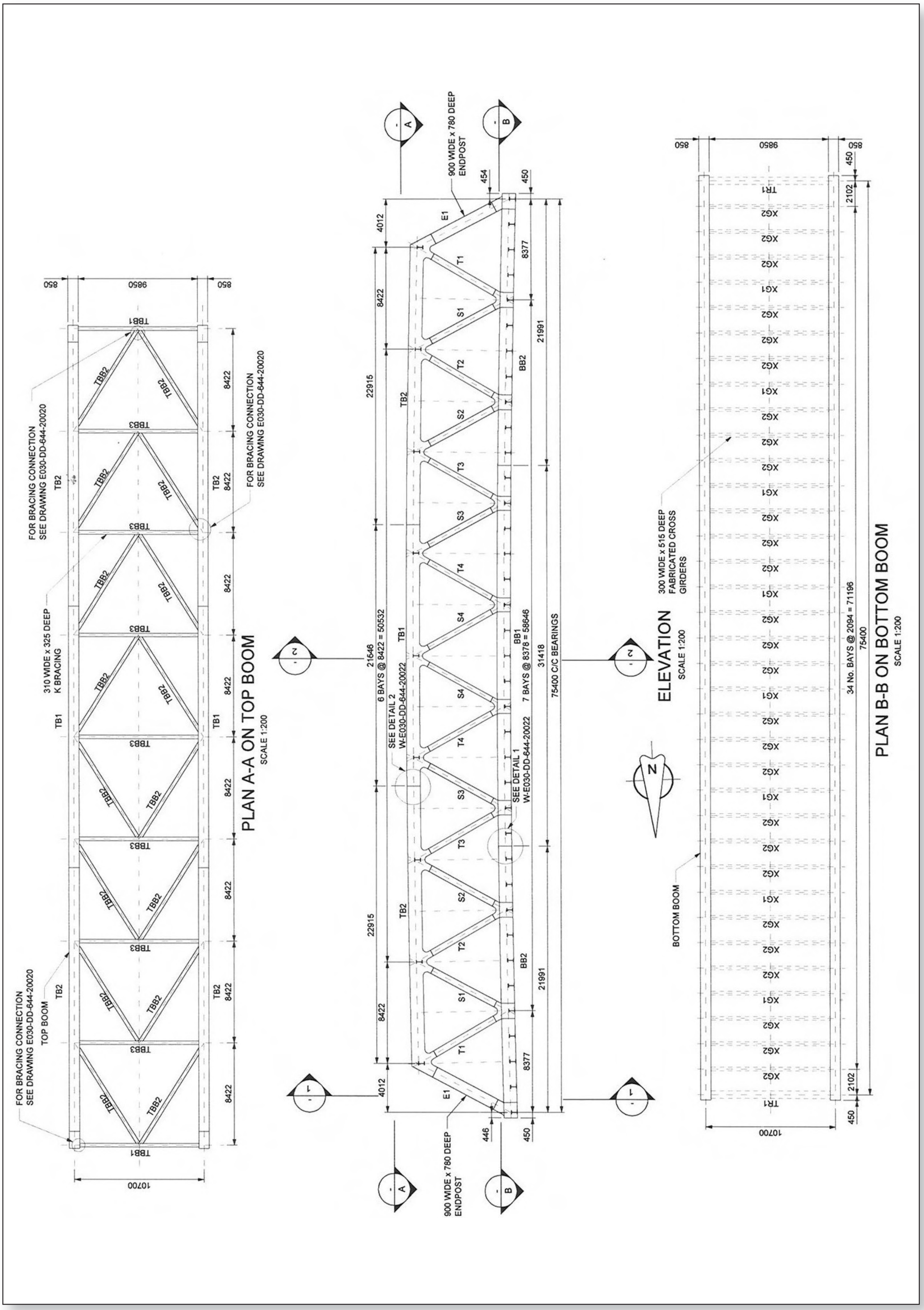


Figure 21 - New Cross Gate Bridge - General arrangement

8.6 The Sheppey Crossing

Client:	Highways Agency and Sheppey Route Ltd
Designer:	Cass Hayward LLP
Contractor:	Carillion
Sub-Contractor:	Mabey Bridge Ltd
Completion Date:	2006
Approximate Steelwork Weight:	10,000 tonnes

Until the completion of the new crossing, the Kingsferry Lift Bridge was the only means of crossing the Swale to the Isle of Sheppey. In order to allow ships to pass underneath the bridge the centre span of the Kingsferry Bridge had to be raised (approximately 2000 times each year). With vessels enjoying priority over the road traffic, severe delays occurred on the existing trunk road. The new crossing has eliminated that restriction; the new dual carriageway A249 now enables free flow of traffic.

The completion of the new 'Sheppey Crossing' in 2006 provided the first fixed link from mainland Kent to the Isle of Sheppey, a high-level viaduct. The bridge, which used 10,000 tonnes of fabricated steel plate girders and 60,000 t of structural concrete, is 1,270 m long with 19 spans, the longest being 92.5 m over the central navigation channel of the Swale. The spans grow in length gradually from the abutments towards the main central span and the bridge depth increases proportionately to a maximum of nearly 4 m at mid-crossing. This unusual arrangement produces a most elegant elevation that is enhanced by the sweeping curve of the highway, which rises to a crest of 30 m above the estuary. The new road was officially opened on 3 July 2006 and it now carries approximately 26,000 vehicles per day.

The decision to launch the majority of the structure (15 spans in three phases using 14 separate launches) heavily influenced the design and also reduced the work load done by large capacity mobile cranes during the construction phase.

The four main girders at 5.5 m centres varied continuously in depth from approximately 1.5 m at the abutments to 3.5 m over the central navigation channel. The cross girders, 940 mm deep at 3.5 m longitudinal centres, were fabricated in long lengths and then cut to the required length; this suited Mabey Bridge's automated processes. Plan bracing to the central seven spans located between the two inner main girders ensured temporary stability during the launch and acceptable aerodynamic behaviour of the completed deck. The plan bracing was located at an aesthetically pleasing 300 mm above the bottom flanges. Material grades used were S355J2+N (6 to 61 mm thick) supplemented with S355 K2 (62 to 77 mm thick) and S355 NL (78 to 98 mm thick) as necessary. Over 185,000 preloaded bolts were used in the structural connections, totalling approximately 100 t in weight.

Value engineering of the scheme was continuously performed through close teamwork so that all suggestions could be incorporated into the design. Through design development, the structure was developed from a box-type structure to a multi girder ladder deck structure using plate girders. Some of the value engineering affecting fabrication costs resulted in modifications of bracing types and alterations to the shape of stiffeners. Other value engineering suggestions concerned the practicalities of erection, such as use of slotted holes in the splices (of the closure spans) in order to reduce risks associated with erection. The use of a sliding system rather than conventional steel rollers meant that damage to paintwork (from the launch) was virtually eradicated, even on the record-setting 92.5 m cantilever.

Allowance for maintenance scaffold loading was made in the design of the main steel girders. Jacking stiffeners allow replacement of bearings without the need to close the viaduct to traffic.

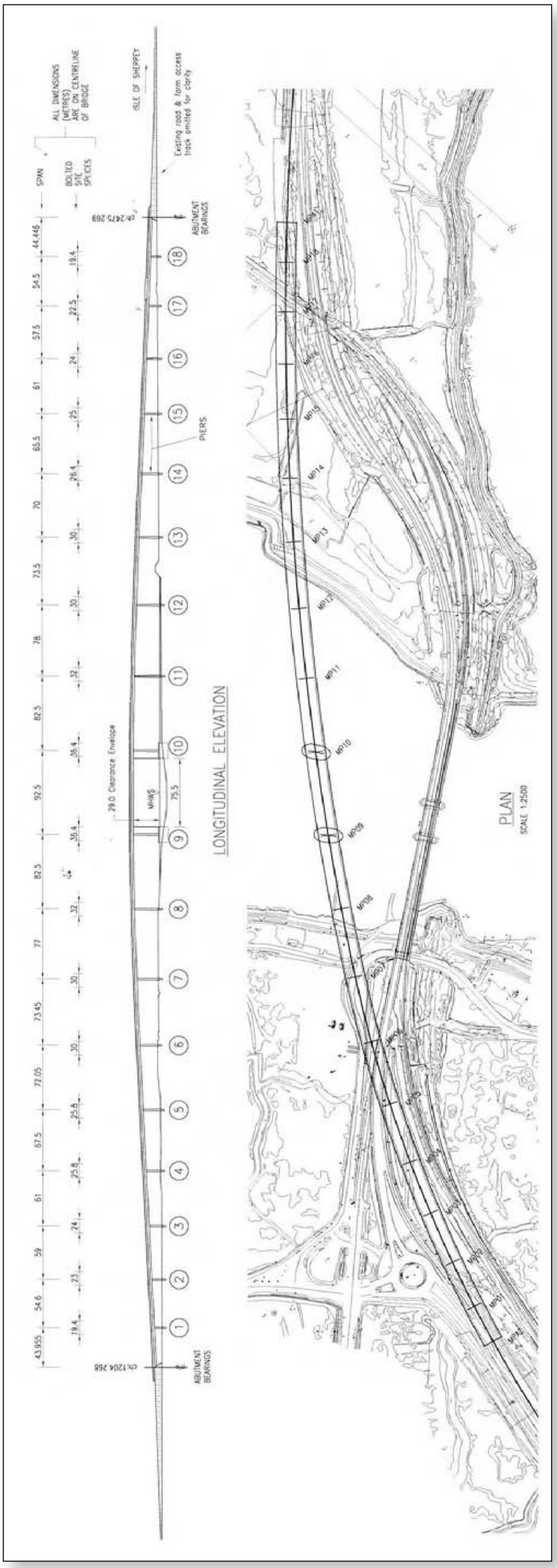


Figure 22 - Sheppey Crossing

8.7 N25 Waterford Bypass River Suir Bridge

Client:	**National Roads Authority (NRA)**
Designers:	**Arup, P H McCarthy, Eptisa and Carlos Fernandez Casado SL**
Contractor:	**Waterford Joint Venture (BAM/Dragados)**
Sub-contractor:	**Mabey Bridge Ltd**
Completion date:	**2009**
Approximate steelwork weight:	**2,800 tonnes**

The River Suir Bridge is the largest cable stayed bridge in Ireland and is part of the N25 Waterford Bypass. It is the landmark structure of the scheme. Due to the flat nature of the surrounding area and size of the structure, it can be seen from several miles in all directions.

The structure is a ladder deck design that carries a dual carriageway road. The deck is made up of 80 main girder sections, each approximately 18 t. The fabrication of the main girders for the cable supported section included some elements of complex geometry in the hanger locations, with large welds and complex angles, dimensional control in these areas became critical. Two box sections at the north abutment, each approximately 170 t, were manufactured at the workshop in segments and welded into full lengths at ground level on site. 102 cross girders, each approximately 10 t, were made in full lengths of approximately 20 m. Due to the width of the road deck, and to maintain as low a deck weight as possible, the tops of the cross girders were profiled to follow the road camber. The deck slab was formed using precast concrete units.

The structure was erected in two main stages. The back span, which is over land, was erected from ground level with mobile cranes onto trestles up to the central pylon. The deck was subsequently completed with a precast concrete slab up to the pylon. This enabled the front span to be erected in cantilever from the pylon using a modular technique (each module comprising two main girder sections and cross girders) with the steelwork delivered piece small up to the erection crane sited on the cantilevered end of the deck. After each module was erected, the cables were installed and prestressed and the precast concrete deck placed. Each module of the deck was erected, bolted, cables installed and concrete placed within a seven day cycle. There was also a supported section of the front span erected in conjunction with the modular build, of which the largest components were the box sections at the north abutment; these were lifted into position using a 1,000 t floating crane, the Mersey Mammoth which is currently the largest floating crane available in the UK. These sections were supported on an arrangement of trestles and header beams prior to the front span connection being made. These were subsequently jacked down into final position and the trestling removed.

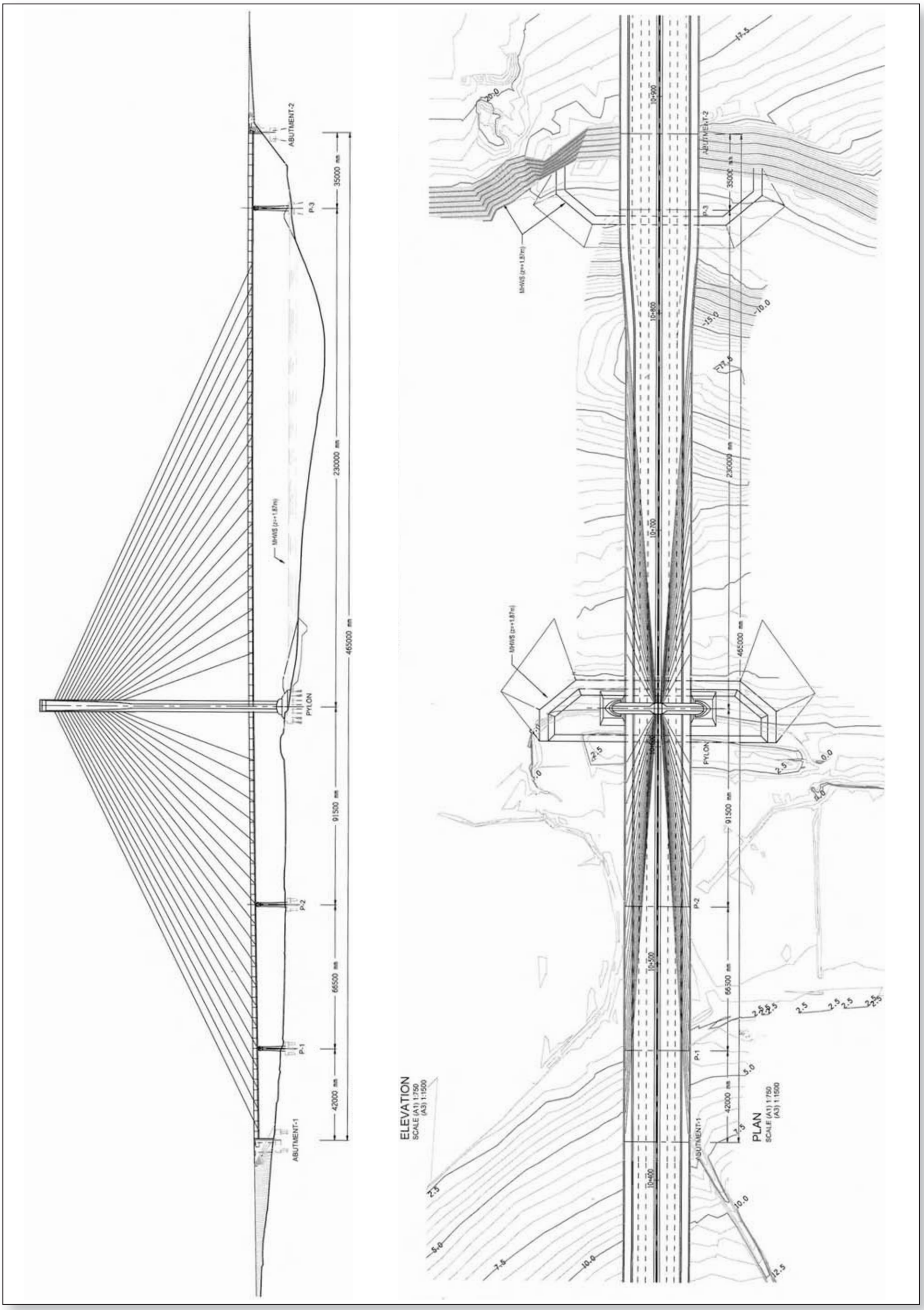


Figure 23 - River Suir Bridge

8.8 Forthside Bridge, Stirling

Client:	Stirling Council
Design:	Gifford and Wilkinson Eyre
Main Contractor:	Edmund Nuttall Ltd
Sub-contractor	Rowecord Engineering Ltd
Completion Date:	Spring 2009
Approximate steelwork weight:	448 tonnes

This new bridge greatly improves pedestrian connections between Stirling's town centre and its railway station. The bridge is aligned to suit pedestrian desire lines and to promote physical and visual connectivity with the town, to establish an enhanced sense of place.

The bridge is a 114 m long three span asymmetric inverted Fink truss with a main span of 88.2 m, spanning seven rail tracks, a service road and a car park adjacent to Stirling Station. It is a contemporary interpretation of the traditional Fink truss structure, which is inverted here to support the deck from above. The trusses are arranged asymmetrically and change size incrementally along the length of the bridge to create an organic twisting form.

The resulting structure is both dramatic and visually 'light', the steel masts and cables contrasting with laminated glass infill at parapet level. At night these appear to glow, creating a shifting 'glass ribbon' of colour along the length of the bridge.

The arrangement of the bridge is such that the approaches (stairs and lift assembly) are effectively landscape features that are offered up to a high-level pier. The bridge is a visually and structurally independent structure that connects the two approach assemblies. The defining feature of the bridge is the pair of up-stand trusses to either side of the deck. The trusses feature steel masts with cable cross-bracing but critically are devoid of a top chord.

Each truss is effectively a propped cantilever, with its visual and structural mass resting on one side of the railway and its diminishing form reaching across to the other. One truss reaches from Forthside to Stirling and the other from Stirling to Forthside, in an overlapping and interlocking arrangement akin to a structural handshake.

Each truss comprises eight masts, of which the end members are support pylons and the central six are stanchions connected to the deck structure. The trusses reduce in height, diameter and inclination over the length of the bridge, defining a warped plane of structure to either side of the deck.

With the bridge spanning seven rail lines and erection only permissible during weekend night time closures, the residual risks associated with ensuring closure of the two bridge sections during very limited working times over the rail were deemed too significant, and the chosen construction sequence was a conventional sequential erection method with a series of temporary supports.

The bridge structure was erected as five separate assemblies, with temporary trestle supports located between the live rail tracks.

The two end sections of the bridge were too large to transport as completed fabrications. Instead, these two sections were each split longitudinally into two halves. The resultant assemblies of 80 t on the west (Stirling) side and 52 t on the east (Forthside) side were both erected using 800 t mobile cranes from outside the rail boundaries.

The three central sections were transported to site by road using inclined bespoke transport frames in order to keep the overall dimensions within the limits of the highways authority. The three sections were then off-loaded onto temporary stools where the masts were welded onto the main jumbo hollow section booms. One of these sections was erected from the west car park area using an 800 t mobile crane and the other two sections were assembled and erected from the east side using 500 t mobile cranes.

The installation of some of the diagonal bars linking the top of the masts to the deck nodes could be carried out on the assemblies prior to lifting in, whereas the remaining interconnecting diagonal bars had to be erected on the temporarily supported in-situ bridge structure by means of mobile cranes from outside of the rail boundaries.

Erection of the stair and lift assemblies, undertaken in green zone working, followed the bridge installation.

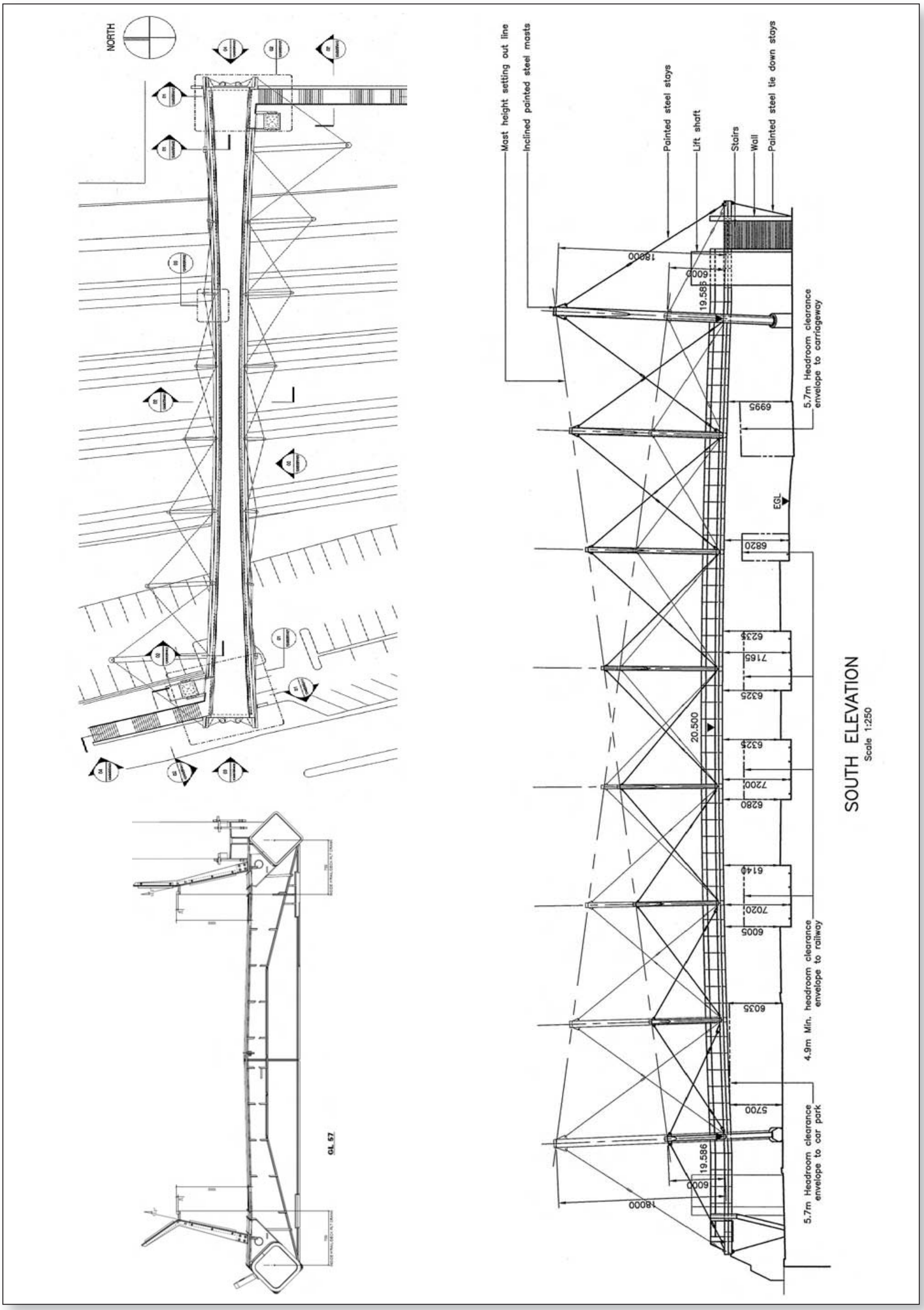


Figure 24 - Forthside Bridge

8.9 Rhymney River Bridge, Bargoed

Client:	Caerphilly County Borough Council
Design:	Capita Symonds
Main Contractor:	Hochtief Griffiths JV
Sub-contractor	Rowecord Engineering Ltd
Completion Date:	August 2008
Approximate steelwork weight:	271 tonnes

Featuring a graceful concrete arch design, this 130 m long bridge forms a landmark structure that sympathetically reflects its location in a country park. The bridge reaches a height of 24 m above the valley floor where it crosses the Rhymney River.

The bridge comprises a concrete arch a two-span continuous composite deck spans between each abutment and the arch crown, with intermediate north and south piers.

Each plate girder was threaded through the narrow valley roads to be delivered to the site in three sections, between 15 m and 25 m long. Two of the sections, those closest to the abutments, were bolted together on the ground to form a girder 40 m long and lifted into place by a 500 t crane nestled against the back of the concrete abutments. These sections of girder cantilevered 7.5m beyond the intermediate pier, and were supported by jacks at the abutment and the intermediate pier, and stabilised by special frames fitted to the top of the piers. The third section, 17.5 m long, was lifted into place using the same crane, and a bolted splice carried out in the air. The splice was made by labour in a man-basket supported by the tower crane.

When all the girders were in place, and checked for alignment and level, the cross-girders were lifted into place using the tower crane. Because the valley profile prevented the use of mobile platforms to access large parts of the structure; a special cradle was designed, which spanned between the plate girders. The cradle could be moved by the tower crane and provided a working platform for making the bolted joints of the cross-girders and applying surface protection to all bolted connections, without tying up the crane for other activities

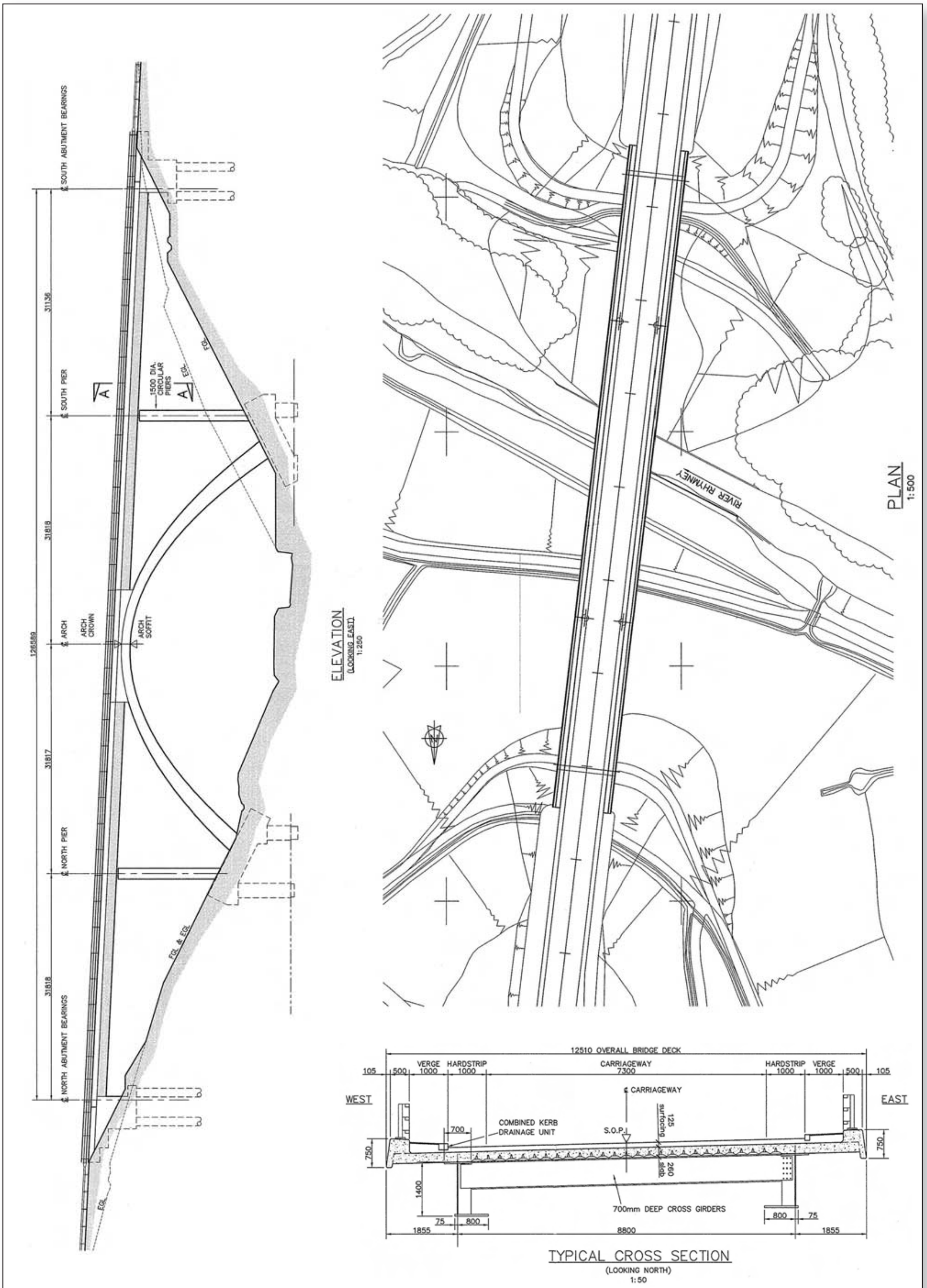


Figure 25 - Rhymney River Bridge

8.10 Petts Hill Underbridge

Client:	Harrow Council
Design:	Atkins
Main Contractor:	Birse Rail Ltd
Sub-contractor	Rowecord Engineering Ltd
Completion Date:	December 2008
Approximate steelwork weight:	289 tonnes

The Petts Hill railway bridge carries the Chiltern Line over the A312/A4090 road on the boundary between the Boroughs of Harrow and Ealing. The bridge was needed to remedy the following deficiencies:

- The restricted width of the carriageway under the railway bridge often created congestion at the junction of the A312 (Northolt Road) and A4090 (Alexandra Avenue).
- The footpaths to Northolt Park Station and underneath the railway bridge were narrow.
- There was no provision for cyclists under the bridge.
- Heavy traffic congestion made bus journey times unreliable.

The extent of work comprised the supply, fabrication, protective treatment, delivery and erection of structural steelwork for the new bridge. To create minimal disruption to the railway the dismantling of the existing bridge and erection of the new bridge was undertaken during a Christmas possession.

The project comprised the demolition of the existing structural steel bridge, carrying two non-electrified tracks, supported by mass brickwork abutments with associated wing-walls and the subsequent construction of a half-through all-steel deck bridge with trapezoidal box sections, supported on new piled foundations and wing-walls.

The steel deck is supported longitudinally by trapezoidal box girders, with transverse ribbed stiffeners directly below the main deck plate. The bearings were bespoke and were fabricated in-house. The bridge was designed in accordance with Network Rail standard drawings.

The steel material generally used was high yield S355J2+N grade, although other sub-grades of S355 were used in less stressed areas. The two trapezoidal girders either side of the deck are 32 m long, 2.2 m high and 1 m deep; each weighed 55 t. They were transported to site in single sections on specialised abnormal load trailers.

The 11 deck sections were 2.9 m long and 9.1 m wide, spanning between the trapezoidal girders; each weighed 11 t. Each deck section was stiffened with five rolled section T stiffeners the full width of the deck and were bolted to the girders and to each other, using M22 TCB bolts (1700 bolts in total). The joints between deck sections were reinforced using a double flange plate spanning the joint on the underside of the deck.

The four main bearings were fabricated from 40 mm and 60 mm plate and were connected to the abutment walls and main deck section with M30 bolts, with a machined finish to the topside and underside of the bearing.

In order to ensure accuracy of fabrication and fit-up the entire bridge was trial assembled within the fabrication works. In order to reduce the on-site duration of work, the deck was concreted at this stage and waterproofing was applied, requiring only the joints to be made good at site.

Due to inherent uncertainties with demolition activities, installation of the bridge was planned with a floating period of 12 hours within a 16 hour window. Installation commenced at 16:30 on the 25th December with fitting of the bearing base plates onto the abutment wall. The main trapezoidal girders, with pre-fitted bearings, were then singly lifted into position and secured. Deck sections were fitted either end of the structure to achieve a 'square' set-up, ready to accept the internal deck sections. Following the fitting of the internal deck sections, waterproofing was completed and ballast retention plates were installed. After checking line and level, the lifting brackets were removed, leaving the structure available for the main contractor to complete the works.

During the 12-hour installation period, 1900 Tension Control Bolts were fitted and tightened. The duration of this operation was calculated in advance to ensure the correct number of bolting 'teams' were deployed to achieve the operation in the extremely tight available window.

The whole civil and mechanical programme was achieved in the allotted 60 hour possession and traffic started to flow along the A312/A4090 road under the structure immediately after completion.

CHAPTER 9
COMPETENCE IN STEEL CONSTRUCTION

9.1 Introduction

The client's interest and obligation, through the procurement process, is to appoint a project team that is competent to produce the bridge – competent in design, in management, and in the particular construction activities required. Each of the key roles in the project influences the erection processes directly or indirectly, particularly the management of health and safety; so it is important that the key personnel with responsibility for the project have the necessary experience and expertise in steel bridge construction.

9.2 Management of health and safety

Under the requirements of the CDM Regulations the principal contractor has overall responsibility for health and safety during construction, and this responsibility is effected through the Construction Health and Safety Plan as he develops the plan for the new bridge construction.

The principal safety objectives when erecting steel bridgeworks are

- Safe access and working positions
- Safe lifting and placing of steel components
- Stability and structural adequacy of the part-erected bridge

The most serious hazards during bridge erection are related to falls from height, either from working positions or while gaining access to them. Other serious hazards are related to structural instability or failure during erection and while transporting, handling and lifting heavy components. The steelwork contractor's health and safety management system addresses the particular hazards and risks in steel construction as well as the normal range of issues in working on construction sites. His planning for health and safety is systemic to all the preparation for erection through risk assessment, devising safe systems of work and working up the erection method statement.

Cooperation between the steelwork contractor and the principal contractor is essential in the planning and the implementation stages; it is also required by law. The steelwork contractor's safety plan prepared for the project will be complementary to the Construction Health and Safety Plan.

One useful tool to assist in the cooperation between the steelwork and principal contractors, and facilitate the safe erection of bridge steelwork is the bridge version of the BCSA *Safe Site Handover Certificate (SSHC)*. This was specifically developed for bridge construction to provide a consistent approach to safe site conditions and assist clients, principal contractors and steelwork contractors alike to meet their respective responsibilities under health and safety regulations.

Further information regarding the health and safety during construction of bridges and a template for the SSHC are available in the BCSA publication *Guide to the Erection of Steel Bridges*.

9.3 Selection of steelwork contractor

The current practice of requiring bidders to submit quality statements as well as financial bids gives clients a surer basis for satisfying themselves of the bidders' competence. The quality statement should demonstrate the approach to the specific project, its challenges and opportunities, as well as historical experience; it should nominate a competent team of key personnel. This process enables clients to assess more objectively which bidder can give the best value.

Selection of a competent steelwork contractor is a necessary precondition to ensure that competent personnel are mobilised to undertake fabrication, erection and protective treatment. The Highways Agency has developed National Highway Sector Schemes (NHSS) which define the specific competence requirements for companies looking to work for the Highways Agency. Companies gain registration to sector schemes following external audit by UKAS accredited audit bodies. NHSS 20 covers the execution of steelwork for infrastructure assets and NHSS 19a covers the application of corrosion protection to infrastructure assets. Both of these are relevant to the supply of steel bridges to the Highways Agency but can also be adopted by any other customer for a road, rail or footbridge. Further information on National Highway Sector Schemes can be found at the Highways Agency's web site and the UKAS web site (see Section 10.3).

For bridgework contracts where the ultimate client is the Highways Agency it is mandatory that only companies listed on the Register of Qualified Steelwork Contractors Scheme for Bridgeworks for the type and value of work to be undertaken will be employed in the fabrication and erection of bridgeworks.

For bridgeworks contracts where the ultimate client is Network Rail, companies should have the relevant link-up approval.

9.4 The Register of Qualified Steelwork Contractors Scheme for Bridgeworks

The Register of Qualified Steelwork Contractors Scheme for Bridgeworks (the Bridgeworks Register) provides procurement agencies with a reliable listing of steelwork contractors that identifies their capabilities for types of steel construction, and suggested maximum contract value. The Bridgeworks Register is administered by BCSA but is open to any steelwork contractor who has a fabrication facility within the European Union. For a current list of Registered Bridgeworks Contractors see www.SteelConstruction.org/directories/bridges.

In order to be registered for bridgeworks, a company must have a minimum turnover in steelwork for bridges (or contracts of similar complexity) of £1 million in the most recent year or alternatively per annum if averaged over the last three years.

The company must present references for completed supply and erect contracts that include at least three bridgework contracts (or contracts of similar complexity), of which two must each exceed £100,000 contract value and have been completed within the last three years.

Companies are registered under one or more of the sub-categories listed below.

Bridgeworks Register Sub-categories:

FG Footbridges and sign gantries

PG Bridges made principally from plate girders

TW Bridges made principally from trusses

BA Bridges with stiffened complex platework (in decks, box girders or arch box members)

CM Cable-supported bridges (cable-stayed or suspension) and other major structures

MB Moving bridges

RF Bridge refurbishment

AS Ancillary Structures (eg grillages)

X Unclassified

An experienced professional auditor visits each registered company at its premises both before admission to the Bridgeworks Register and at intervals thereafter. The auditor assesses the company's capabilities in the seven subcategories of bridge construction. The auditor also takes up relevant project references before registration is accepted. Registered companies make annual returns and the auditor re-assesses each company triennially, and when there are significant changes.

All companies on the Bridgeworks Register also have to satisfy the auditor of their financial standing and resources. The company's track record and the company's systems, existing facilities and employed personnel are all used to establish its capability.

9.5 Benefits of using the Bridgeworks Register

The Bridgeworks Register provides a straightforward, effective prequalification mechanism to match steelwork contractors to the needs of particular bridge tenders. Although the Bridgeworks Register itself is not a quality assurance scheme, listing in the Bridgeworks Register verifies that the company's quality management systems are third party certified by an accredited body.

The use of a registered company matched to the demands of the project assists clients, designers and principal contractors in ensuring that sub-contracted work is done safely, and is a prima facie defence to any allegation that insufficient care was taken in selecting a competent steelwork contractor.

By using the Bridgeworks Register to match companies to suitable tender opportunities, the danger of a company submitting an unsustainably low price is removed. This assists in ensuring fair competition, and is the best way of ensuring that the successful tenderer is able to finish the job. It also contributes to the stability of the industry, such that appropriate, competent companies remain in business for the next project.

9.6 CE Marking

The introduction of the Eurocodes for structural design is part of an initiative to remove technical barriers to trade in Europe by implementing pan-European requirements for construction products. This move will initially permit and ultimately mandate the CE Marking of construction products, including steel plates, fasteners and fabricated steel components. BS EN 1090-1 *Execution of steel structures and aluminium structures – Requirements for conformity assessment of structural components* provides the basis for CE Marking of the components of steel bridges. The BCSA has published a *Guide to the CE Marking of Structural Steelwork* that includes an appendix that deals specially with the issues associated with bridges.

CHAPTER 10
REFERENCES

10.1 Codes and standards referred to in this edition

- BS 5400 Steel, concrete and composite bridges.

  BS 5400-3:2000 Code of practice for design of steel bridges

  BS 5400-6:1999 Specification for materials and workmanship, steels

- BS EN 287-1:2004 Qualification test of welders - Fusion welding - Steels.
- BS EN 473:2008 Non-destructive testing – Qualification and certification of NDT personnel – General principles.
- BS EN 1011 Welding - Recommendations for welding of metallic materials.

  BS EN 1011-1:2009 General guidance for arc welding

  BS EN 1011-2:2001 Arc welding of ferritic steels

  BS EN 1011-3:2000 Arc welding of stainless steels

- BS EN 1090 Execution of steel structures and aluminium structures.

  BS EN 1090-1:2009 Requirements for conformity assessment of structural components.

  BS EN 1090-2:2008 Technical requirements for the execution of steel structures

- BS EN 1337 Structural bearings.
- BS EN 1418:1998 Welding personnel – Approval testing of welding operators for fusion welding and resistance weld setters for fully mechanized and automatic welding of metallic materials.
- BS EN 1993 Eurocode 3: Design of steel structures.

  BS EN 1993-1-1:2005 General rules and rules for buildings

  BS EN 1993-1-5:2006 Plated structural elements

  BS EN 1993-1-8:2005 Design of joints

  BS EN 1993-1-10:2005 Material toughness and through-thickness properties

  BS EN 1993-1-12:2007 Additional rules for the extension of EN 1993 up to steel grades S700

  BS EN 1993-2:2006 Steel bridges

- BS EN 10025 Hot rolled products of structural steels.

  BS EN 10025-1:2004 General technical delivery conditions

  BS EN 10025-2:2004 Technical delivery conditions for non-alloy structural steels

  BS EN 10025-3:2004 Technical delivery conditions for normalized/normalized rolled weldable fine grain structural steels

  BS EN 10025-4:2004 Technical delivery conditions for thermomechanical rolled weldable fine grain structural steels

  BS EN 10025-5:2004 Technical delivery conditions for structural steels with improved atmospheric corrosion resistance

  BS EN 10025-6:2004 Technical delivery conditions for flat products of high yield strength structural steels in the quenched and tempered condition

- BS EN 10160:1999 Ultrasonic testing of steel flat product of thickness equal or greater than 6 mm (reflection method).
- BS EN 10164:2004 Steel products with improved deformation properties perpendicular to the surface of the product. Technical delivery conditions.
- BS EN 10210-2:2006 Hot finished structural hollow sections of non-alloy and fine grain steels. Tolerances, dimensions and sectional properties.
- BS EN 10219-1:2006 Cold formed welded structural hollow sections of non-alloy and fine grain steels. Technical delivery requirements.
- BS EN 10306:2002 Iron and steel. Ultrasonic testing of H beams with parallel flanges and IPE beams.
- BS EN 12062:1998 Non-destructive examination of welds – General rules for metallic materials.
- BS EN 14399 High-strength structural bolting assemblies for preloading.

  BS EN 14399-1:2005 Part 1. General requirements

  BS EN 14399-2:2005 Part 2. Suitability test for preloading

  BS EN 14399-3:2005 Part 3. System HR. Hexagon bolt and nut assemblies

  BS EN 14399-4:2005 Part 4. System HV. Hexagon bolt and nut assemblies

  BS EN 14399-5:2005 Part 5. Plain washers

  BS EN 14399-6:2005 Part 6. Plain chamfered washers

  BS EN 14399-7:2007 Part 7. System HR. Countersunk head bolt and nut assemblies

  BS EN 14399-8:2007 Part 8. System HV. Hexagon fit bolt and nut assemblies

  BS EN 14399-9:2009 Part 9. System HR or HV. Bolt and nut assemblies with direct tension indicators

BS EN 14399-10:2009 Part 10. System HRC. Bolt and nut assemblies with calibrated preload

- BS EN ISO 148-3:2008 Metallic materials. Charpy pendulum impact test. Preparation and characterization of Charpy V-notch test pieces for indirect verification of pendulum impact machines.
- BS EN ISO 3834-2:2005 Quality requirements for fusion welding of metallic materials – Comprehensive quality requirements.
- BS EN ISO 4063:2009 Welding and allied processes. Nomenclature of processes and reference numbers.
- BS EN ISO 5817:2007 Welding. Fusion-welded joints in steel. nickel, titanium and their alloys (beam welding excluded) - Quality levels for imperfections.
- BS EN ISO 9001:2008 Quality management systems – Requirements.
- BS EN ISO 9018:2003 Destructive tests on welds in metallic materials. Tensile test on cruciform and lapped joints.
- BS EN ISO 13920:1997 Welding - General. Tolerances for welded constructions – Dimensions for lengths and angles – shape and position.
- BS EN ISO 14555:2006 Welding – Arc stud welding of metallic materials.
- BS EN ISO 14731:2006 Welding coordination – Tasks and responsibilities.
- BS EN ISO 15614-1:2004+A1:2008, Specification and qualification of welding procedures for metallic materials - Welding procedure test. Arc and gas welding of steels and arc welding of nickel and nickel alloys.
- BS EN ISO 15609-1:2004 Specification and qualification of welding procedures for metallic materials. Welding procedure specification. Arc welding.
- BS EN ISO 15613:2004 Specification and qualification of welding procedures for metallic materials. Qualification based on pre-production welding test.
- BS EN ISO 15614-3:2008 Specification and qualification of welding procedures for metallic materials. Welding procedure test. Fusion welding of non-alloyed and low-alloyed cast irons BS EN ISO 4063:2009 Welding and allied processes – Nomenclature of processes and reference numbers.
- BS EN ISO 9018:2003 Destructive tests on welds in metallic materials – Tensile test on cruciform and lapped joints.
- PD 6695-1-10:2009 Recommendations for the design of structures to BS EN 1993-1-10.
- ASTM A325 - 07a Standard Specification for Structural Bolts, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength, ASTM, 2007.
- Specification for Highway Works, Series 1800, Structural steelwork.

10.2 Other documents, references, standards referred to in this edition

- Lark, R J, HSFG bolted connections using weathering steel materials, Proceedings of the Institution of Civil Engineers, Bridge Engineers , 157 (BE2), 2004.
- Managing health and safety in construction. Construction (Design and Management) Regulations 2007. (CDM) Approved Code of Practice, HSE, 2007.
- The use of weathering steel in bridges, Publication No. 81, ECCS, 2001.
- BCSA publications

  Guide to the CE Marking of Structural Steelwork (Publication No 46/08).

  Guide to the Erection of Steel Bridges (Publication No 38/05).

  The Use of Heat Straightening to Repair Impact Damaged Steel Bridges.
- Corus publications

  Steel bridges – Material matters, Sustainability. (Available on www.corusconstruction.com).

  Steel bridges – Material matters, Weathering steel. (Available on www.corusconstruction.com).

  The design of steel footbridges. (Available on www.corusconstruction.com).
- SCI Publications

  Steel Bridge Group: Model Project Specification for the Execution of Steelwork in Bridge Structures (P382), 2009.

  Steel Bridge Group: Guidance Notes on Best Practice in Steel Bridge Construction (P185), 5th Edition, 2010.
- UKAS publications

  National Highways Sector Schemes for Quality Management in Highway Works:

  19A, For corrosion protection of ferrous materials by industrial coatings, 2006.

  20, The execution of steelwork in transportation infrastructure assets, 2008.

10.3 Sources of information and suggested further reading

Web sites

BCSA www.steelconstruction.org

Corus www.corusconstruction.com

SCI www.steelbiz.org

Highways Agency
www.highways.gov.uk/business/10379.aspx
(information on sector schemes)
www.standardsforhighways.co.uk/
(for DMRB and MCHW)

RSSB www.rgsonline.co.uk (for Railway Group Standards)

UKAS www.ukas.com

BCSA publications

BCSA Guide to Steel Erection in Windy Conditions (Publication No 39/05).

BCSA Guide to the Management of Site Lifting Operations (Publication No 47/09).

BCSA Guide to Work at Height during the Loading and Unloading of Steelwork (Publication No 43/07).

Galvanizing Structural Steelwork (Publication No 40/05, published jointly with GA).

Health and Safety on Site – A Guide for Steelwork Contractors (Publication No 27/01).

Historical Structural Steelwork Handbook (Publication No 11/84).

Typical Welding Procedure Specifications for Structural Steelwork (Publication No 50/09).

SCI publications
Composite highway bridge design (P356), 2010.
Composite highway bridge design:
Worked examples (P357), 2010.
Design guide for steel railway bridges (P318), 2004.

Steel Bridges

A Practical Approach to Design for Efficient Fabrication and Construction

BCSA Publication No. 51/10