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Bridge detailing guide

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Bridge detailing guide

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Construction Industry Research and Information Association

Publication C543

© CIRIA 2001

ISBN 0 86017 543 X

Keywords

Bridges – detailing, design, construction, durability, maintenance, inspection, concrete superstructures, steel superstructures, subways, culverts, support structures, bearings, joints, fixings, retaining walls, integral construction

Reader interest	Classification	
Bridge-owners and operators; detailers, designers and specifiers; constructors and specialist	AVAILABILITY CONTENTS	Unrestricted Recommendations based on best current practice
suppliers	STATUS	Committee-guided
	USER	Bridge design engineers, technicians and constructors

Published by CIRIA, 6 Storey's Gate, Westminster, London SW1P 3AU.

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Summary

This guide was commissioned by the Quality Services Civil Engineering Division of the Highways Agency (HA) and has been prepared for those active in the bridge engineering industry. The need for a bridge detailing guide was identified by an earlier HA/CIRIA research project on bridge buildability. In particular, guidance is provided for engineers and technicians engaged in the preparation and development of details for highway and accommodation bridges, subways, culverts and retaining walls. Thus the guide concentrates on the detailing issues that follow conceptual and analytical design. The scope is further limited to spans up to 60 m and, in the case of steelwork details, to steel girder/concrete slab composite construction.

Details selected for the guide represent basic principles that have proved to be reliable in everyday use in terms of durability and ease of construction, inspection, maintenance and repair. Explanatory notes emphasising the principles and issues involved are provided. However, the guide is intended as a live document and will be revised and extended as a result of feedback by the industry. A formal feedback procedure is included.

The guide is based on research in the UK and internationally, and the selected details have been subject to wide review by practitioners within the industry. In cases where there were differences of opinion the preferred details represent a majority view.

Health and safety

Construction activities, particularly on bridge construction sites, have significant health and safety implications. These can be the result of the activities themselves, or can arise from the nature of the materials and chemicals used in construction. The report does not endeavour to give comprehensive coverage of the health and safety issues relevant to the subject it covers, although its importance is emphasised by particular sections dealing with health and safety. Readers should consult other specific published guidance relating to health and safety in construction.

Government reorganisation

Recent Government reorganisation has meant that the responsibilities of the Department of the Environment, Transport and the Regions (DETR) have been moved variously to the Department of Trade and Industry (DTI), the Department for the Environment, Food and Rural Affairs (DEFRA), and the Department for Transport, Local Government and the Regions (DTLR). In particular, the DTLR now has responsibility for the Highways Agency. References made to the DETR in this publication should be read in this context.

For clarification, readers should contact the Department of Trade and Industry.

Acknowledgements

The project leading to this guide was commissioned by the Quality Services Civil Engineering Division of the Highway Agency, and was carried out under contract to CIRIA by Hyder Consulting Limited.

Research team (Hyder Consulting Limited)

Mr M A Soubry

with valuable contributions, assistance and research from many other members of staff, both in the UK and overseas and, in particular, by Mr J B Harris.

Steering group

The guide was prepared with guidance from a steering group and two technical committees, which included the following:

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Mr J M Barr	High Point Rendel
Mr C V Castledine (Steel T C chairman)	Butterley Engineering Ltd
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Dr B W Staynes (research manager)	CIRIA

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Acer Engineers and Consultants Inc (USA) Amec Civil Engineering Ltd **Babtie Group British Steel** Carl Bro Group Concrete Bridge Development Group Mr R Craig (WS Atkins Transportation Engineering) DOE Northern Ireland Roads Service Durham County Council Mr J E Evans (Flint and Neil Partnership) Fairfield Mabey Ltd W A Fairhurst & Partners Freeman Fox International (Dubai) Grace Construction Products Ltd Hyder Consulting (Australia) Pty Ltd Hyder Consulting (Thailand) Limited Mr D C Iles (Steel Construction Institute) Ove Arup & Partners Priority Metals and Fasteners Ltd PSC Freyssinet Ltd Mr J Robb (Mott MacDonald Ltd) Schlaich Bergermann and Partner (Germany) Sir William Halcrow & Partners Ltd Tarmac Construction Ltd Thorburn Colquhoun Ltd Tony Gee & Partners Universal Sealants (UK) Ltd Warwickshire CC Watson Steel Ltd Mr P J Williams (the British Constructional Steelwork Association Ltd).

Due to the wide coverage of the contents of this document many others, not listed, were involved and their contributions are, nevertheless, greatly appreciated.

Note

CIRIA and the author gratefully acknowledge the support of the funding organisations and the technical help and advice provided by the members of the steering group. Contributions do not imply that individual funders necessarily endorse all views expressed in published outputs.

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Introduction

As has happened in many other countries, the UK highways bridge stock has exhibited durability problems in recent years. While there are several causes of deterioration, many of the problems are attributed to poor detailing and a lack of appreciation of buildability by designers^(1, 2). CIRIA Report 155⁽³⁾, published in 1996, addresses bridge buildability issues.

This guide has been prepared for use by active members of the bridge engineering profession. The target readership comprises consultants, contractors, bridge owners and their maintaining agents. It will be of direct use to trainee engineers (including graduates), technicians and incorporated engineers involved in detailing highway bridge designs. It should also be of value to chartered engineers as they develop designs, and to site staff, as it provides advice on the function and relative merits of various details.

The design process develops from feasibility and conceptual design through to detailed design. The details in this guide are intended for use in the detailing stage that follows. While designers will be aware of and may anticipate use of details in this guide, in general, advice on design is specifically excluded. Some design issues, such as access during and after construction and CDM requirements, are directly relevant to the detailing stage, however, and are identified in Chapter 2.

Although there are many sources of advice on good detailing, rarely is such advice collected together. Organisations engaged in bridge design and/or construction usually have their own preferences. Most available professional information comes as advice incidental to treatises on particular projects or subjects.

As modern mass production methods are increasingly applied in the construction industry, details will tend to be repeated within projects and from project to project. It is therefore important to identify best practice and eliminate deficiencies.

When selecting details for this guide, emphasis has been placed on those representing basic principles that have proved to be reliable in everyday use in terms of durability and ease of construction, inspection, maintenance, repair and demolition. Only details that can be clearly defined have been included, together with supportive text explaining the rationale and dealing with durability and buildability issues. The principles formulated should enable sound special case details to be developed. The details have been prepared in a way that allows them to be readily adopted by designers, but care will still be needed to ensure that the details, and developments from them, are correctly interpreted and applied.

The details are also supplied on the CD-ROM provided with this book. The software used is AutoCAD R14.

1

1.1 OBJECTIVES

- 1. Provide examples of bridge details that represent current good practice, enhanced by explanatory notes and advice to emphasise the principles and issues involved in each. The guide contains a set of details that:
 - are technically sound
 - are inherently durable
 - satisfy requirements for ease of construction
 - result in low maintenance costs
 - have a good appearance.
- 2. Recommend details through which a degree of standardisation is encouraged. This should enable the design, detailing and construction of bridge structures to become more efficient, and lead to both short- and long-term benefits for bridge owners and for those charged with their maintenance.
- 3. Provide a sound basis upon which alternative approaches may be developed by identifying the function of the details in the guide and relevant prompts and pitfalls. There is no intention to inhibit the development of alternative approaches.
- 4. Encourage feedback, through which details in the guide can be improved in the light of constructive comment and be supplemented by additional details (see Section 1.6).

1.2 SCOPE

The details are intended for use in the following highways structures:

- highway bridges
- accommodation bridges
- subways and culverts
- retaining walls.

A significant proportion of bridge work in the UK includes renovation. While the book concentrates on details for new structures, they may also be considered for strengthening or renovating existing structures.

The scope of the guide has been limited in four ways.

- 1. It concentrates entirely on details and detailing issues to be implemented after the conceptual and analytical design process has been completed.
- 2. It provides details for spans up to 60 m, although most of the details are based on the span lengths of the majority of bridges, ie up to 20 m.
- 3. Steelwork details are limited to steel girder/concrete slab composite bridges, the most common form of steel highway bridge constructed in the UK.
- 4. Details for foundations are not included.

This guide has been prepared primarily for application to UK highway bridges, but will be useful for other applications.

METHODOLOGY

1.3

1.4

The work included a desk study to review:

- the research contractor's in-house information
- details and advice made available by industry sources
- feedback on health and safety issues
- feedback on durability from bridge management programmes
- relevant overseas practice.

The bodies approached for details were chosen as representing sources with high concentrations of state-of-the-art bridge engineering. They included various industry institutions, bridge maintaining authorities and county councils. Approaches were also made internationally.

A CIRIA steering group and specialist technical panels provided advice and guidance on the content and development of the project.

ASSOCIATED DOCUMENTS

CIRIA Report 155, *Bridges – design for improved buildability* ⁽³⁾, should be read in association with this *Bridge detailing guide*, as it addresses problems of buildability and sets down guidelines for improvements in this area.

Other important publications include:

- The Institution of Structural Engineers' manual on reinforced concrete detailing ⁽⁴⁾
- The Steel Detailer's Manual, by Hayward and Weare⁽⁵⁾
- SCI-P-185 Steel Bridge Group: Guidance notes on Best Practice in Steel Bridge Construction ⁽⁶⁾
- SCI-P-163 Integral Steel Bridges Design Guidance⁽⁷⁾.

Advice on durability issues can be obtained from:

- DMRB Standard BD 57/95⁽¹⁾
- DMRB Advice Note BA 57/95⁽⁸⁾
- DMRB Advice Note BA 42/96⁽⁹⁾
- DMRB Standard BD 47/99 ⁽¹⁰⁾.

Relevant CIRIA publications include:

- Report 146 Design and construction of joints in concrete structures ⁽¹¹⁾
- Report 166 CDM Regulations work sector guidance for designers ⁽¹²⁾
- Report 174 New paint systems for the protection of construction steelwork ⁽¹³⁾
- C558 Permanent formwork in construction ⁽¹⁴⁾
- C559 Improving freeze-thaw resisting concrete in the UK⁽¹⁵⁾
- C568 Specifying, detailing and achieving cover to reinforcement⁽¹⁶⁾.

1.5 HOW TO USE THIS GUIDE

Figure 1.1 shows the principal parts of a typical highway structure and indicates the chapter containing relevant details.

Chapters are divided into sections covering particular bridge types or structural forms. Many details are common to some or all the structure types. These are found at the beginning of the chapters concerned. Cross-reference back to these details is made where appropriate.

In Chapters 3 and 4 the general forms of the particular bridge types are illustrated. The details available in the guide are identified by reference numbers on these illustrations.

The numbers of the details relate to their location in the guide. The first three digits indicate the section number in which the detail is found. The fourth digit is a sequence number of the details within the section.



Typical cross-section of bridge and pier

- Note
 Retaining walls (Chapter 7)
 - Integral bridges (Chapter 8)

Figure 1.1 Location of details in the guide



Principles behind good detailing

The sections below introduce the principles behind the details in this guide. The primary requirements of the structural design are strength, safety, durability and buildability of the principal members. It is, however, the competence of details that determine whether the performance as a whole is satisfactory.

Bridges that satisfy requirements for safety (Section 2.2) and buildability (Section 2.3), and have details that are durable and easy to maintain (Sections 2.4 and 2.5), are likely to be economical to construct and maintain.

2.1 AESTHETICS

2

Most bridges have a pleasant appearance when newly built, but after some years they develop their real character. Rainwater channelled by intricate features begins to stain the surfaces. Such features become less prominent relative to these natural markings. It follows that bridges should be kept as simple as possible. Rainwater flows should be anticipated and details incorporated to suit. For example, grooves and similar features in the exposed surfaces can channel water and create beneficial shadow patterns if carefully located, although the influence of the prevailing wind makes prediction very difficult.

Structural details should be rationalised and, so far as possible, made common to provide visual cohesion throughout a bridge, and with the other bridges in the same project. Details should not be over-complicated or dominate the overall structure.

Construction joints should be located to ensure that they do not detract from the bridge's appearance. They are often defined with a groove or recessed feature, which avoids the concrete surface from showing a ragged joint and allows two different concrete pours to show slightly different colours without unpleasant effect.

Movement joints often concentrate moisture flow, usually accompanied by leaks. Deck joints and other discontinuities should be kept to a minimum, and so located as to minimise any deleterious effect. Integral bridges are now being specified for appropriate new works. The minimal joints they incorporate are expected to improve appearance.

Disguising poor details is rarely satisfactory. Fair-faced high-quality concrete provides a long-lasting satisfactory appearance. The designer should seek to use the structure's natural facets to best advantage rather than impose additional features or components. If features are added that result in congested reinforcement, poor-quality concrete or cracking, these attempts to improve appearance will be negated. Unless required for special aesthetic reasons, cladding, which requires extra maintenance, should be avoided.

A durable bridge should sustain its good appearance if the design takes into account:

- good detailing of drainage
- shapes that control staining
- a minimum of crevices or discontinuities for the build-up of dirt
- ease of inspection and maintenance.

The Design Manual for Roads and Bridges ⁽¹⁷⁾ provides further guidance on this topic.

2.2 HEALTH AND SAFETY

2.2.1 General

Construction activities, particularly on site, have significant health and safety implications. These can arise from the nature of the processes, materials and chemicals used in construction. This section does not endeavour to give comprehensive coverage of the health and safety legislation, but it raises relevant detailing issues and, in particular, those related to access during construction, operation and maintenance.

The choice of a particular form of construction should be made with an appreciation of the construction process and the need for maintenance. Where maintenance will be carried out in high-risk areas, such as adjacent to high-speed traffic, the requirement for such activity should be minimised.

When working close to high-speed traffic, there should be a safety zone for protection of the workforce in addition to the necessary working space. For motorways, this safety zone is a minimum of 1.2 m wide, so for bridge structures with narrow verges the near-side lane will need to be closed when maintenance activity takes place within the verge.

Other published guidance on health and safety issues in construction should be consulted as necessary. Further information can be obtained from CIRIA Report 166, *CDM Regulations – work sector guidance for designers* ⁽¹²⁾, which has sections on bridge construction and bridge maintenance.

2.2.2 CDM

The Construction (Design and Management) Regulations 1994 (CDM) ⁽¹⁸⁾ require a health and safety plan and a health and safety file to be properly prepared. Designers need to consider hazards and risks that will arise at the following stages:

- construction
- operation (use)
- inspection and maintenance
- modification (eg widening) or demolition.

The regulations require hazards to be eliminated, or reduced so far as is reasonably practical, with residual risks identified so that they can be managed by the relevant organisation (eg main contractor).

Hazards and risks that may occur during inspection and maintenance should be included and/or referenced in the maintenance manual for a bridge or suite of bridges (Section 2.5).

Bridge details should conform to the above requirements. The general issues related to access are equally applicable to the construction, inspection and maintenance stages.

2.2.3 Construction operations

Hazardous situations can be created where insufficient space is available to undertake the work safely, eg where rectangular voids with restricted headroom have been detailed in a deck and the soffit formwork has to be stripped out through a narrow gap. In such cases, the use of permanent formwork or void formers should be considered.

Badly detailed and congested reinforcement can also create construction difficulties.

Where ground conditions are unsuitable to support necessary falsework, consideration should be given to supporting the falsework off the permanent works foundations.

Permanent formwork offers the advantage of protecting the areas beneath the bridge deck against falling items, and avoids the need to send operatives below the deck to remove temporary works. CIRIA C558, *Permanent formwork in construction* ⁽¹⁴⁾, provides detailed advice on its use.

2.2.4 Access – general

Design/detailing considerations regarding general access to bridges may be affected by:

- nature of the crossing (road, railway, river etc)
- adjacent landscaping (steep embankment slopes, large trees etc)
- location of buried services
- height of parapets and pilasters
- verge or pavement widths and surfacing
- street furniture including lighting columns.

It is no longer normal practice to provide access manholes in road surfaces, chiefly for safety reasons. Closing traffic lanes on busy highways creates risks for both drivers and operatives. Traffic congestion resulting from lane closures creates additional risks.

Access into box girders should be arranged from the abutments or, where the boxes are discontinuous, through the soffit. Care must be taken to provide safe access to locations in the soffit. Size of openings, ease of entry and rescue requirements including anchor points also need to be considered. Heavy skews may create particular difficulties, and special measures are needed for arch, cable-stayed and suspension bridges.

2.2.5 Internal access

The size of openings at entry and between the cells of a structure should be decided as part of the designer's consideration of hazards and risks. Any minimum required by statute or other applicable authority should be taken into account. Based on Section 30 of the Factories Act 1961 (now withdrawn) it is recommended that absolute minima of 460 mm \times 410 mm or, if circular, 460 mm diameter, should be provided unless there are other adequate means of egress. Access size should allow necessary equipment (eg ventilation or stressing equipment and/or a loaded stretcher) to be handled safely. The spacing of the access points influences this assessment. Platforms should be provided at access and egress points along with appropriate lifting points.

Designers should avoid details that present hazards or create access problems. Box girder structures present particular difficulties, as internal inspection is required. Under the Confined Spaces Regulations 1997⁽¹⁹⁾, the interior of a box girder must be recognised as a confined space. Associated requirements include:

- trained personnel
- risk assessments
- emergency procedures
- controlled entry
- approved methods of working
- air monitoring.

The designer/detailer should therefore consider:

- the means and ease of access
- spacing of manholes
- spacing of ventilation openings
- frequency of inspections
- methods of internal protection
- frequency of subsequent maintenance.

2.2.6 Lighting and walkways

The frequency of inspections and maintenance visits makes installation of permanent lighting essential in large box girder bridges. They improve both safety and efficiency, thereby justifying the investment. The infrequency of visits to the interiors of small bridges makes a permanent lighting installation unnecessary, although the provision of intrinsically safe power-points protected from misuse is appropriate. Incorporation of permanent walkways and materials-handling routes can be considered, but these in turn need to be maintained and require handrails if there is likely to be a fall greater than 2 m (eg tops of piers).

2.2.7 Seepage of water

Water may enter structures through faulty weatherproof seals, leaking road drainage pipes or condensation. As part of their risk assessment, designers should minimise the hazards of slipping on wet surfaces and of infection from the build-up of fungi in box girders by making allowance for water to be dispersed.

Water ingress into smaller hollow sections should be considered even when no entry is envisaged. Problems from deadweight effects and bursting due to ice formation have been known to occur.

2.2.8 Security

Improved access to all parts of bridges makes security more difficult. The security risks at each location of a new bridge should be assessed and appropriate measures taken. Secure doors to the access routes may be necessary in some locations and surveillance systems may need to be installed for full security. Public access to girders over roads and railways etc should be prevented. For example, permanent access ladders should stop out of reach from the ground, or locked fold-down ladders should be provided.

2.3 BUILDABILITY

2.3.1 General

Reference should be made to CIRIA Report 155, *Bridges – design for improved buildability* ⁽³⁾.

Designers need to recognise the significance of labour and plant costs. Minimising the material in each element does not necessarily result in overall economy. Within a project the geometry of details should be rationalised and dimensional standardisation sought to maximise the reuse of items such as formwork.

Designers need to consider the erection process, and any requirement for temporary stability measures should be included within the information provided to the contractor. The contractor may be responsible for the design of temporary bracing and will be responsible for the erection process, but he must be made aware of any features within the permanent works design that may affect stability during the construction phase.

Lifting operations must be considered and lifting points provided in any element where normal slinging techniques would be unsafe or inappropriate.

The need to modify elements during construction on site should be avoided. It is therefore important to be realistic about tolerances and clearances appropriate to the construction industry. In particular, allowance needs to be made for potential errors in alignment and/or position of previously constructed components.

Most bridge structures need joints. Examples include:

- construction joints in concrete
- site assembly joints in steel
- movement joints to permit flexural and temperature movements etc.

Joints should be kept to a minimum. Integral bridges (Chapter 8) reduce the need for joints, hence the trend to this type of bridge.

The designer/detailer should allow for necessary clearances. In addition to health and safety requirements (Section 2.2), all construction operations need space for access and/or the use of construction equipment. Simple examples include clearance for the body and movement of hydraulic jacks to tension pre-stressing tendons, and the space needed for swaged couplers and their equipment. When preparing details, due allowance should be made for construction clearances and for future inspection and maintenance.

Simplicity is desirable. Complex details, although sometimes unavoidable, can create difficulties and attract extra cost. Standard details should, in general, be simple details.

National and international standards and specifications list many materials and grades of construction materials and products. Not all material grades are readily available and there will be regional variations. It is prudent for those planning construction to investigate the sourcing and availability of materials.

2.3.2 Concrete

The principle of maximising the repetition of details is particularly applicable to formed concrete shapes. In addition, the designer should consider the difficulties that can occur when striking formwork, the use of permanent formwork (see Section 2.2), the potential for the use of travelling formwork and the ease of concrete compaction.

The use of travelling formwork is possible if the lines and angles of the structural shape are generally uniform. For example, in the case of constant-depth box girders, designers should consider keeping the internal cross-section uniform throughout the length of the structure. Changes of cross-section may minimise the material quantities but result in extra formwork costs.

Compaction is easier in shapes that allow direct placing of the concrete. Re-entrant corners, nibs and unnecessary fillets should be avoided, as the formwork is difficult to construct and may be difficult to concrete.

Where a "fair face" finish avoiding shutter joints is specified, options are limited. This is illustrated in Figure 7.3 of CIRIA Report 155⁽³⁾. Unless it is an essential part of the design, an expensive finish such as F3 (see Clause 1708.4 (i) of *Specification for Highway Works*⁽⁴⁶⁾), which does not permit conventional shutter ties, should be avoided.

The correct choice of construction joint locations should be planned, and a practical construction sequence determined during the design process. The location of construction joints is part of this process and, where possible, early discussion with the contractor can be valuable. Despite the widespread use of "kickerless" construction in building works, the use of "kickers" cast with the previous pour is still recommended for bridges to allow positive location and sealing of shutters for interconnecting elements. Further information maybe obtained from CIRIA Report 146, *Design and construction of joints in concrete structures* ⁽¹¹⁾.

Satisfactory reinforcement detailing is a critical part of achieving buildable and durable structures. Simplicity of arrangement should be sought. Bars should be lapped or curtailed at locations appropriate to the envisaged construction joints. Congestion can be reduced by reversing alternate bars and/or staggering laps. Also, a careful choice of reinforcing bar shape (for example, see Detail 7.2.2-2) can help avoid the risk of inadequately fixed reinforcement becoming displaced during concreting.

Reinforcement protruding from one pour should not have a critical cover dimension on a subsequent pour. Tolerances need to allow for the total of cutting, bending and shutter tolerances. The size of reinforcement bends may also be significant. In such cases, large-scale details, taking into account cutting and bending tolerances, may be needed to ensure fit. CIRIA publication C568, *Specifying, detailing and achieving cover to reinforcement* ⁽¹⁶⁾, provides additional guidance.

2.3.3 Steel

The designer needs to take into account the relevant phases of steel construction. These may include fabrication of the transportable elements, site assembly and erection. Different techniques and processes may apply to each.

Some modern fabrication methods are highly automated. The designer should be aware of fabricators' equipment, plant and welding techniques. The most economic production and the most consistent quality will usually be achieved when automated factory fabrication accounts for a high proportion of the work.

Whether for works fabrication, site assembly, automated processes or handwork, the principle of maximising the repetition of details also applies to steelwork. It is important to avoid specifying oversize welds (see Section 4.1.3) and to provide access for tightening HSFG bolts (see Section 4.4).

Designers should try to avoid the use of butt welds unless they are essential for strength or fatigue performance. The work, including special edge preparation where required, is more labour-intensive than for fillet welds, and testing requirements are more expensive and demanding.

An erection procedure should be established at the planning stage and allowed for in the design. Items affecting site erection include access constraints, lifting limitations, stability of elements in the temporary condition, and the detailed relationship between steel and precast concrete elements.

The detailing of site joints will significantly affect the success of a project. Site-bolted connections are generally preferred by steelwork erectors because they can be made quickly and are not so weather-constrained. The choice between using bolted or welded connections on site is project-specific depending on, for example, aesthetic considerations and the number of connections to be made. The theoretical location of shear studs can be onerous. Consideration should be given to avoiding provision of shear studs on splice plates.

Some of the details requiring particular care in composite steel/concrete bridge construction relate to permanent formwork (BA 36/90⁽²⁰⁾). These details are usually repetitious and the benefits of any improvements are multiplied many times. CIRIA Publication C558, *Permanent formwork in construction*⁽¹⁴⁾, provides detailed advice.

2.4 DURABILITY

2.4.1 Concrete

Exposure to the cumulative effect of humidity, runoff, rain, spray, freeze-thaw, de-icing salts and atmospheric contaminants causes deterioration of bridge structures. Penetration of de-icing salt (a chloride) is the main cause of rapid corrosion of steel reinforcement. It is therefore essential to provide resistance to penetration of de-icing salts. CIRIA C559, *Improving freeze-thaw resisting concrete in the UK*⁽¹⁵⁾, provides guidance.

Durable bridge structures usually have simple details that allow speedy natural shedding of rainwater and spray. Bridge decks should have uninterrupted top surfaces because it is almost impossible to ensure the water-tightness of discontinuities. In this context:

- manholes and prestressed anchorages in the upper surface of decks should be avoided
- movement joints should be kept to a minimum because they are particularly susceptible to penetration by water
- reinforcement should be designed to control crack widths.

Generally, there should be more than one line of defence against the penetration of moisture. Although in dryer climates waterproofing is not always necessary, in the UK the practice is for the top of structural bridge decks, ie beneath the surfacing, to be waterproofed with an impermeable membrane. Specific bridge details related to waterproofing are included in Section 3.1.4. A system of approval for the membranes themselves has been established (see BD $47/99^{(10)}$).

Vertical surfaces in contact with soil are also generally waterproofed (Chapter 7). Other concrete faces are capable of shedding water naturally and so are left exposed to the elements. Where these surfaces are subject to spray from passing traffic, it is normal practice to reduce the permeability of the concrete surface by applying a silane treatment to protect against contamination by de-icing salt. The remaining exposed concrete needs to be adequately resistant to water penetration.

Weaknesses in waterproofing membranes or treatments may lead to deterioration of reinforced concrete. Detailing should be such that failures of a waterproofing system become evident during routine maintenance inspection. For example, the discharge of sub-surface drainage to a visible open channel should be considered.

Only very dense concrete has long-term durability. While the quality of the concrete mix is significant, the prime cause of reduced concrete density is inadequate compaction. Poor reinforcement detailing and formwork shapes that impede the flow of concrete and restrict access to vibrators make it more difficult to compact concrete fully under site conditions. The use of concrete details that facilitate compaction, coupled with well-designed concrete mixes, is essential. Awkward ledges and corners where water can collect, which in winter probably contains salt in solution, should be avoided.

A principal cause of poor durability of reinforced concrete is inadequate concrete cover to reinforcement. Adequate cover thickness must be specified to suit the conditions, but inadequate cover can still occur during positioning of the reinforcement. While lack of attention to good workmanship is one of the causes, related problems commonly occur. For example, the constrained leg of a bar (eg the middle run of a U-bar running between two faces of a wall) should have adequate tolerance and not be the run-out dimension. Unless due allowance is made for the presence of drip grooves and similar re-entrant details, cover may be reduced below the minimum required. Detailed advice is provided in CIRIA C568, *Specifying, detailing and achieving cover to reinforcement* ⁽¹⁶⁾.

Further information can be obtained from the HA/CSS/TRL publication *Water* management for durable bridges ⁽²¹⁾.

2.4.2 Steel

As with concrete bridges, it is important to avoid accumulation of water on or within the bridge structure. Details that avoid water entrapment should be selected. The adoption of such details should not have a significant effect on steel bridge economics.

The failure of any imperfect welds can also lead to durability problems, eg breakdown of protective coatings. Full attention should therefore be paid to the detailing and quality control of all connections, ie not only main structural joints but also the attachment of subsidiary members such as bracing, access platforms and handrails.

Weathering steel

Weathering steels achieve their resistance to corrosion by their capacity to develop a protective patina through oxidation. Other steels rely upon protective coatings to achieve satisfactory durability. Some aspects of detailing particular to the use of weathering steel can be found in Section 4.8.

Protective coatings

When protective coatings are used it is essential that the steelwork details enable painting or spraying of all exposed surfaces to be readily achieved, as corrosion will start where there is incomplete coverage. It should be noted that too thick a layer of paint under HSFG bolts may result in a loss of tension.

Protective coating technology is under continual development. In recent years, the emphasis has been on improving ease and speed of application, environmental friendliness and toxicological characteristics. CIRIA Report 174, *New paint systems for the protection of construction steelwork* ⁽¹³⁾, gives guidance on the selection, application and specification of coatings for use in the general fabrications industry. It also takes into account statutory regulations, including: Environmental Protection Act (1990) ⁽²²⁾, the Construction (Design and Management) Regulations 1994 (CDM) ⁽¹⁸⁾ and Control of Substances Hazards to Health Regulations (COSHH) ⁽²³⁾.

Fatigue

Fatigue cracks cause durability problems in parts of steel bridge structures directly affected by cyclic traffic loading, ie steel decks or top flanges of composite steel/ concrete superstructures. Designers should ensure that stress concentrations are minimised. For example, notches should be avoided. This is particularly important in tension members.

BS 5400: Part 10⁽²⁴⁾ sets out classifications for welds of different types. While detailed analysis can be used to evaluate fatigue performance, the most effective way to achieve durability is to select details with a non-critical fatigue life.

2.5 MAINTAINABILITY

2.5.1 General

Two issues have prompted major bridge owners to assess the health of their assets:

- accelerated deterioration resulting from the increased use of de-icing salt became more and more evident
- the unification of European traffic loading, which led to the UK road bridge stock having to be formally certified as being capable of carrying higher loads than those for which they were originally designed.

Also, as the demands on the network increase, computerised bridge management systems including comprehensive databases of bridge stock have been, and are being, developed. These aid engineers in deciding where best to invest maintenance funding.

2.5.2 Inspection

The process of maintenance involves a programme of inspections. The design engineer should make adequate provision for access. Although sometimes compromises have to be made, the aim should always be for all parts of a structure to be accessible and visible for direct inspection.

The chosen means of access should affect the use of the bridge as little as possible, ie the need to reduce the road capacity while inspection or routine maintenance is being carried out should be avoided.

Additional space should be provided within abutments (see Detail 6.5.0-2) etc, to enable the bearings and the ends of superstructure members under movement joints to be inspected. Early consideration during design/detailing stages usually enables simple solutions to be_incorporated. Allowance should be made for access for equipment. Permanent provision of some equipment may be appropriate on major structures.

Section 2.2 (Health and safety) provides further advice on this subject.

2.5.3 Maintenance activity

A bridge should last for its design life (120 years) provided that due attention is paid to the maintenance of the less durable parts. Unfortunately, the life of some manufactured components such as bearings and movement joints cannot be given a life rating of more than about 20 years. The trend, therefore, is to use integral bridges that avoid bearings and joints. Where the use of bearings and joints is unavoidable, ease of replacement is an important consideration.

Designers should endeavour to avoid all short-life components and make all other aspects of bridge structures as free from maintenance as possible. Simple examples include the natural shedding of dirt and debris (by avoiding nooks and crannies) and the use of self-draining slopes (eg 1 in 20). Accumulations of dirt tend to hold water that may contain de-icing salt.

Design/detailing issues should include provision of information relating to replacement of manufactured components, taking into account:

- health and safety requirements (Section 2.2)
- the effect on the structure and waterproofing when joints are cut out for replacement
- the load to be taken while a bearing is removed for replacement
- the jacking-point positions, and sufficient room for jacks with the capacity required
- lifting synchronisation
- the ability of jacked members to carry the temporarily redistributed loads.

Restrictions on lane loading should be minimised. Early consideration at the design/ detailing stage will usually allow simple solutions to be incorporated.

The designer will prepare a maintenance manual for a structure or suite of structures taking into account matters arising during construction. This will identify particular characteristics of the structure, and the recommended inspection and maintenance programme. Replaceable items and the planned sequence for the construction will be listed, and advice on access provided.

Health and safety risks will be identified in the health and safety file, which should include appropriate cross-referencing to the maintenance manual to avoid duplication. Sometimes the two documents are amalgamated. The maintenance manual and the health and safety file need to be easy to use, store and update.

Concrete superstructures

3.1 GENERAL

3

3.1.1 Preamble

In addition to this General section, this chapter is divided as follows:

- slab bridges
- beam and slab bridges
- box girder bridges
- subways and culverts.

This first section includes details that are considered to be applicable to most concrete superstructures. Reference should be made to the relevant sections for the individual bridge types. Appropriate cross-references are made to other parts of the guide.

The details to be found in this chapter are as follows:

3.1.2-1	Chamfer	3.3
3.1.3-1	Drip inducer (water-shedding)	3.4
3.1.4-1	Waterproofing tuck	3.7
3.1.4-2	Waterproofing fillet	
3.1.4-3	Waterproofing – chamfer	
3.1.5-1	Surface water drainage – kerb channel	
3.1.5-2	Surface water drainage – positive system (kerb inlet)	
3.1.5-3	Surface water drainage – through-deck connection	
3.1.6-1	Subsurface drainage – dispersal through kerb inlets	3.14
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3.1.6-3	Subsurface drainage - soffit outlet detail	
3.1.7-1	Verge – waterproofing and falls	
3.1.7-2	Verge	
3.1.7-3	Verge service trough – in situ concrete	
3.1.7-4	Verge service trough - precast concrete U-beams	
3.1.7-5	Verge service trough - precast concrete Y-beams	
3.1.8-1	Parapet beam – typical features	
3.1.8-2	Parapet beam – reinforcement	
3.1.8-3	Parapet beam – discontinuity joint	
3.1.9-1	Movement joint (contraction) - range 0 mm to +3 mm	3.29
3.1.9-2	Movement joint - range 0 mm to 10 mm total	3.30
3.1.9-3	Movement joint - range 10 mm to 20 mm total	3.30
3.1.9-4	Movement joint - range 20 mm to 40 mm total	3.31
3.1.9-5	Movement joint – range >40 mm	3.32
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3.2.2-1	Transverse reinforcement – arrangement	3.39
3.2.2-2	Transverse reinforcement - voided slabs	3.40
3.2.2-3	Transverse reinforcement – narrow decks	
3.2.3-1	Cantilever reinforcement	
3.2.4-1	Void drainage	
3.3.2-1	Diaphragms – ending at side face	
3.3.2-2	Diaphragms at abutment – soffit	

3.3.2-3	Diaphragms at skew abutment	
3.4.2-1	Post-tensioning (external) – deviator block arrangement	
3.4.2-2	Post-tensioning (external) – deviator ducting	
3.4.2-3	Post-tensioning (external) – anchorage blister	
3.4.3-1	Ventilation – box superstructure	
3.4.4-1	Drainage through deviators	
3.5.3-1	Subways – typical cross-section	
3.5.3-2	Subways – waterproofing	
3.5.3-3	Subways - waterproofing at end	
3.5.3-4	Subways – cut-off drains	
3.5.4-1	Subways – joints	
3.5.4-2	Subways - construction joint	
3.5.4-3	Subways – movement joint	
3.5.4-4	Culverts – joints, in situ construction	
3.5.4-5	Culverts - joints, precast construction	
3.5.5-1	Subways – lighting	

The text discusses the principles behind the choice of detail, but each detail is to be read and used in conjunction with its own notes. Some of the details in this section will apply also to steel/concrete composite construction (see Chapter 4).

3.1.2 Chamfer

A chamfer is a feature whereby an external corner in concrete is formed using angles of less than 90°, generally 45°.

Sharp corners (90° or tighter) are difficult to form to a clean edge in concrete because the aggregate cannot get into the corner. A weak laitance finish is left, which is easily damaged, often upon removal of shutters. To avoid this untidy edge and so enhance the appearance of the finished structure and improve durability, it is generally accepted that corners should be chamfered.

Detail 3.1.2-1 Chamfer



PREFERRED	•	Size and shape as shown.
AVOID	•	Square or sharper corners in formed concrete.
REMARKS	•	Circular or more complex decorative details should be avoided, as they tend to have the same inherent weakness as square corners.
	•	The 25 \times 25 chamfer is the easiest to form from standard timber and so is the most economical.
	•	Larger chamfers may be more suitable on heavy civil engineering pours where larger aggregate (eg 37.5 mm nominal) is used. In these cases, larger-radius corners on the reinforcement might be required.
	•	Chamfers are normally detailed by use of a general note. This would appear on a general notes drawing or be repeated on all the concrete detail drawings.
	•	A set of standard details prepared for a specific project should include a scale illustration of the standard chamfer.
	•	The standard chamfer would not normally be shown on drawings to scales of 1:20 or smaller but should be shown on larger-scale drawings.
	•	Larger chamfers are required on external corners that are to be covered by a waterproofing membrane (see Detail 3.1.4-3).

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3.1.3 Drip inducer (or drip)

A drip inducer is a recess or groove within, or a protrusion below, an underside surface. Its purpose is to interrupt the flow of any water along that surface and cause it to drip off, a watershed.

Water running down or blown against the side face of the parapet beam of a bridge will then run or be blown along the soffit of that beam and any adjacent cantilever and down the side face of the deck itself, causing unsightly staining and reducing durability. By preventing most of the water from following this path, the presence of a drip in the underside of the parapet beam helps avoid this defacement.

Detail 3.1.3-1 Drip inducer (water-shedding)





- Option A tapered recess. While both A and B have been used extensively, Option A is the more recent and is considered to be the most effective.
 - Option C is also favoured where a downstand beam is required to mask irregularities in the deck pour and achieve a good alignment to the parapet beam. In this case, the downstand itself forms the drip.
- AVOID

 Right-angled recesses.
 - Any reduction in cover to reinforcement in the vicinity of the drip.
- A right angle offers the most effective drip, but the edges are tapered slightly to allow easy removal of the timber former. The lead drip edge could be a right angle.
 - The dimensions shown in Detail A are considered to be the minimum to achieve the desired effect. A wider, deeper recess detail will achieve some improvement but not beyond 75 x 25 mm.
 - The need to provide a recessed drip in conjunction with Detail C should be considered if the width of the downstand increases beyond 200 mm.
 - Because of the continuity of the drip groove and its location in a very vulnerable area, open to the elements, it is important that cover to the reinforcement in the vicinity of the drip does not reduce below that required by the specification.
 - Consideration should be given to where the drips of water will land, so that the problem is not transferred to another critical point, eg the top of the bottom flange of an adjacent main girder.
 - Another way of forming a drip is to use a flat aluminium or plastic strip glued or riveted to the concrete edge face and projecting 20–25 mm below the soffit. This is more likely to be suitable where aesthetic considerations are not important and the main aim is to protect the concrete soffit and deck face, such as within abutment galleries.
 - Dimension D, downstand of Option C, value to be 50 mm minimum for drip inducement, but may be greater for other purposes, eg see Detail 3.1.8-1 for use with edge beams.

3.1.4 Waterproofing

A waterproofing layer is a continuous impermeable layer designed to protect the bridge deck against the deleterious effects of surface water, especially waterborne de-icing salts.

Asphalt surfacing is not waterproof and rainwater will pass through it. At times and in certain places the rainwater will contain high concentrations of chlorides arising from winter salting operations. Waterproofing of bridge decks in the UK is therefore essential to safeguard their long-term integrity.

The approval, manufacture and use of waterproofing systems on highway bridges in the UK is governed by BD 47/99⁽¹⁰⁾, which stipulates bond performance for the adherence of the waterproofing system to the deck. This is to allow for the effects of vehicle braking and also to contain the effects of any failure that might occur by preventing the flow of water underneath the layer. The achievement of this is heavily dependent on the skill and application of the workforce.

BD 47 ⁽¹⁰⁾ also requires all systems to have a protective layer of 20 mm of sand asphalt of a different colour, generally red, to the other surfacing materials. This is intended as an aid during highway resurfacing operations when machinery is used to plane off the worn surface. The distinctive colour of the protective layer allows it to be identified when the thickness of the road surfacing varies from that expected and the depth of planing can be adjusted to suit so as to avoid damaging the waterproofing.

The efficiency of the materials themselves is usually without question. Failure of a waterproofing system usually occurs at its joints and its ends. The extent of the effect of a failure depends upon the bond. The success of a system therefore relies heavily upon the skills of the applicator, but good detailing can also contribute greatly to that success. Features instrumental in securing the effectiveness of a waterproofing system are tucks, fillets and chamfers – see Details 3.1.4-1 to 3.1.4-3.

Integrity testing during application is an important activity.

The choice of system is highly dependent on the finished condition of the deck. The manufacturer should be involved in the deck preparation process.

A protective layer must be provided from coping to coping. The footway/verge area is just as vulnerable to damage as the carriageway area because of the activities necessary in relation to services.

Waterproofing tuck

A tuck is a recess in the concrete in which to seal (tuck) the end of a waterproofing layer to guard against the ingress of water underneath or behind it.

Detail 3.1.4-1 Waterproofing tuck



- Option A, because it allows easy removal of the former and offers a gentle angle around which to return the waterproofing layer, thereby avoiding damage.
- AVOID Option B.
- The right-angle top edge is preferred to enhance the drip effect. Aesthetics are not important here.
 - Irrespective of the waterproofing system, tucks should always be detailed and provided to facilitate maximum choice, both in the original construction and in the future for maintenance work.

Waterproofing fillet

A fillet ensures that the angle of a re-entrant corner is reduced generally to a change in direction of 45° at any single point.

There is a danger when laying waterproofing material into a sharp corner that a void will be left behind it, which may allow the layer to be punctured during subsequent overlaying operations. The risk may be minimised by reducing the degree of angle by means of a fillet.

Detail 3.1.4-2 Waterproofing fillet



- PREFERRED Either of these options is acceptable, although Option B will generally be used if structural design requires the thicker section.
 - Option A is easy to form.
- AVOID Timber fillets are subject to decay in the long term and should be avoided.
- REMARKS Both options require some care to secure a good finish for the waterproofing.

•

- Option B is likely to be reinforced across the splay.
- Fillets should be used whichever waterproofing system is planned.
- Care is needed to dress the waterproofing layer properly around the fillet.
- Proprietary pre-formed bitumen fillets can also be used so long as they are compatible with the waterproofing system.

Waterproofing external corners

External corners will need to be chamfered if waterproofed to avoid thinning (stretching) or splitting the material on sharper corners.

Detail 3.1.4-3 *Waterproofing – chamfer*



REMARKS • Dimension A: size of external corner chamfer where the waterproofing membrane is applied; value to be 50 mm minimum

• On abutment back walls dimension A is often 100 mm. Using this larger size is expected to affect the reinforcement of the corner, requiring a larger bend radius on the bar.

3.1.5 Surface water drainage

Rainwater falling onto a bridge deck affects the structure in three ways.

- 1. As surface water on the top of the paved carriageway, footway and verge.
- 2. As subsurface water in the matrix of the permeable highway construction.
- 3. As leakage through any points of weakness in the waterproofing system and expansion joints.

To protect the structure from the adverse effects of this water and the contaminants it carries it must be collected and removed from the bridge quickly and simply.

Surface water

The crossfalls and longitudinal alignment of the highway on the bridge should be designed to allow surface water to run off the deck and into gullies beyond the bridge, thus avoiding the need for a positive drainage system on the bridge deck. See Detail 3.1.5-1.

If the bridge is too long or the alignment unsuitable for this to occur, then the system for collecting and transporting the water must be as simple as possible. See Detail 3.1.5-2.
Detail 3.1.5-1 Surface water drainage – kerb channel



- This detail is preferred for its simplicity. Water runs from footway and carriageway and then off the decks to the drainage system beyond.
- REMARKS This will be the normal detail where the bridge span is less than the spacing between gullies for the road carried. It will also be the detail where the longitudinal and crossfalls on the bridge are deemed adequate to keep the build-up of water in the channel to an acceptable level.
 - The adjacent highway drainage system should be designed to provide a gully immediately uphill of the bridge to minimise the volume of surface water runoff onto the deck.
 - Ponding on the deck must be avoided.
 - Seals between the kerb and carriageway surfacing were often detailed in the past but, with the greater effectiveness of waterproofing systems, are now considered to be unnecessary.
 - Avoid low spots (sag profiles) on bridge decks if at all possible.
 - See Detail 3.1.7-2 in Section 3.1.7 for additional information.



Gullies and carrier drain

If the amount of water to be collected is such that kerb inlets are not adequate, either gullies or a more positive carrier from the kerb inlet system will be needed.

Detail 3.1.5-3 Surface water drainage – through-deck connection



PREFERRED

The drain

- The drainage system to be above the waterproofing.
- If it is not possible for the drainage system to be above the waterproofing, breaks through the waterproofing must be limited to accommodate the adequate minimum number of outlets and the edges of the waterproofing system sealed around them.
- Any pipe runs through the deck should be short, straight and as near vertical as possible.
- Longitudinal carrier drains that cannot be located above the waterproofing should be below the soffit of the deck.
- AVOID
 Longitudinal pipe runs cast into the deck.
 - Bends in down pipes through the deck.
- REMARKS There must be no joints in pipes running through the deck.
 - Waterproofing must be dressed around the outlet pipe in a way that prevents subsurface water leakage between the pipe and the deck concrete.
 - For deep deck slabs (more than 700 mm) use spigot and socket pipe with the socket at the top.
 - Dimension A, defining location of corner drain: value to be appropriate for pipe connections and bends.
 - Dimension B, distance of outlet from kerb line: value to suit maintainable connections.
 - Dimension F, length of pipe top from puddle flange: value to ensure flange is near centre of deck slab.
 - Drains must permit rodding to clear obstructions.

3.1.6 Subsurface drainage

This is water within the matrix of the carriageway, footway and verge construction materials. Asphaltic surfacing is not waterproof, so surface water will permeate into and through the pores. The degree to which this occurs depends on the speed of surface water runoff and the material specification.

This may not be a problem if there is a longitudinal fall in the horizontal alignment over the bridge and the joint at the low end is of the "buried" type (see Details 3.1.9-1, 3.1.9-2 and 3.1.9-4), which will allow the water in the matrix to flow over the joint and away.

Where any other type of joint forms a barrier or the falls are insufficient to facilitate effective dispersal, then water will be trapped inside. If not dispersed, this may freeze in cold weather and cause damage to the surfacing. Also the pumping action caused by the passage of wheels over the surface of a saturated matrix can result in very high subsurface pressures, which also damage the surfacing and will exploit any weakness within the waterproofing system.

Subsurface drainage is essential, therefore, and should be provided at all low areas and critical locations, ie kerb inlets (Detail 3.1.6-1) and through-deck outlets (Detail 3.1.6-2).

Care needs to be taken to locate the discharge pipes clear of structural members below. See also Section 4.8, Weathering steel.

Detail 3.1.6-1 Subsurface drainage – dispersal through kerb inlets



Typical kerb unit

• This unit has inlet pores below surface level that drain the water from within the blacktop material. In this case, the outlet pipe will be located lower in the unit to avoid trapping the water.



Typical subsurface drainage unit

REMARKS

Outlets sited at all low points.

• Special attention is needed to accommodate outlets through the reinforcement and to avoid loss of cover at the external surface.

Location of outlets at abutments

- The build-up of subsurface water can be alleviated by the use of buried joints at the low end of the deck that allow water to flow over them.
- Downpipes must permit drilling or rodding to clear obstructions.
- Some expansion joints have slotted drains that run the length of the joint and discharge through the expansion gap. This may offer additional capacity but should not replace the through-deck outlet.
- All drainage discharge pipes should be led to drain channels, or channels located to suit pipes, to avoid water running over the surface of the concrete and potentially creating a slippery surface.



PREFERRED • Outlet Option A.

- REMARKS
- Outlet Option C involves cutting holes in soffit formwork and thus increases costs.
 - Outlet Option A is the simplest to form, but it is important that the threaded or clipped connection is sealed. Proprietary products exist for these.

3.1.7 Verges and troughs

This section deals with the area between the parapet beam and kerb. A trough for services cables and pipes is often required here.

Verges

The width of the verge is determined by:

- the need for a footway
- curvature of the carriageway on a straight bridge or its angle to it
- the need to provide a visibility splay due to alignment or an adjacent junction.

In new works it will generally be a minimum of 600 mm wide, but it may be narrower on existing structures.

The area is vulnerable to the ingress of surface water through permeation. This could result in full saturation occurring, depending upon the nature of the fill used and the existence or absence of an outlet. This permeation occurs through the surface of the verge or footway and, at the low side of the deck, from water in the road channel. Permeation also occurs in the matrix of the surfacing seeping through the joints in the kerbs and any cracks or interstices in the backing or bedding concrete.

The verge is often used for the location of services. Activities associated with services increase the possibility of damage being caused to the deck and the waterproofing membrane. This, and the other issues mentioned above, should be considered when detailing a verge or trough.

Detail 3.1.7-1 Verge – waterproofing and falls



PREFERRED • Option A or B.

- AVOID
- Backfall on the surface towards the parapet beam must be avoided.
- The surface over a verge or trough will normally be paved and laid to a crossfall to minimise the possibility of permeation through the surface. The water will then drain off to the road channel and thence into whatever drainage system there is.
 - The waterproofing, together with its protecting layer, must be carried across the full width of the deck, including verges and troughs. The protecting layer should preferably be of the same type as within the carriageway, but it does not have to be. However, it must be continuous from parapet beam to parapet beam including under the kerb and its bedding.

Detail 3.1.7-2 Verge



- REMARKS Infill matrix to be structural concrete to avoid area becoming saturated.
 - If services are to be accommodated, place continuous ducts within the infill concrete.
 - If access to the location of services is required through the verge surface (for activities where drawing or withdrawing cables through existing ducts is not possible) measures to ease the excavation should be taken. An infill of either foamed concrete or a freedraining material such as a granular fill or a weak no fines concrete, both with positive drainage outlets, can be used.
 - Dimension A to provide positive upstand value to be not less than 50 mm but not greater than 100 mm. Allowance will need to be made for width and crossfall variations to maintain minimum height.
 - If any space remains in the verge area after utilities ducts have been provided, additional empty ducts should be laid down at the time of construction. This will avoid the damage that is likely to arise if ducts are added later.
 - Precast concrete kerb should be chosen to suit the requirements of the authority for type of highway carried, ie:
 - (a) for motorways, full batter, 45° splay
 - (b) for roads other than motorways, either:
 - (i) as for motorways
 - (ii) half batter.
 - Dimension B to suit requirements of the authority for the highway carried.
 - Fall on verge to be towards carriageway. Value to be:
 - (a) preferably, at least 3.0 per cent
 - (b) not less than 2.5 per cent.
 - For further issues to be considered when accommodating services see Section 5.8.

Troughs

Sometimes a larger utility main than can be fitted into the verge space needs to be accommodated. This can be done in several ways; see Section 5.8. One solution is to provide a service trough, generally in the footway/verge area of the deck. It is unsatisfactory to position a service trough in the carriageway because disruptive traffic management may be necessary when work is required.

A typical example is given in Detail 3.1.7-3. Other options when using precast concrete beams are illustrated in Details 3.1.7-4 and 3.1.7-5. Reference should also be made to Section 4.9 and Detail 5.8.0-1.

A design and detailing difficulty with troughs is the need to ensure that the arrangement is sufficiently strong to transfer forces from parapet impact into the main body of the superstructure.

Detail 3.1.7-3 Verge service trough – in situ concrete



Service trough at edge of deck

- AVOID
- Service troughs to be avoided if at all possible.
- REMARKS A trough must be waterproofed throughout and provided with a good protective layer, especially to the invert, eg 50 mm-thick fine concrete.
 - Consideration should also be given to protecting the waterproofing on the side walls of the trough, possibly with an additional layer of membrane.
 - An additional tuck to the deck side of the trough should be provided to seal the main deck waterproofing and so limit seepage in the event of any damage occurring.
 - The fill material must be free-draining with positive outlets.
 - An option to limit the ingress of water and so reduce the likelihood of saturation is to provide precast concrete overslabs, as shown in Detail 5.8.0-1 (Section 5.8), and to waterproof over the top.
 - Reinforcement to the service trough to be detailed to permit the service trough and parapet beam to be cast separately to the main deck. Special care is needed in detailing to control restraint cracking.
 - Reference S is to refer to the relevant project specification clause for granular fill material.

Detail 3.1.7-4 Verge service trough – precast concrete U-beams



- The detail using U-beams will require the deck slab to continue over the top of the trough but with discrete access being provided within. Deck and hatch covers must be waterproofed over. Covers could be inset into deck slab.
 - No water or gas mains must be allowed in U-beam troughs due to the inaccessibility and the danger of escaping gas or liquid filling the gap.
 - There must be provision for longitudinal drainage and outlet.

Detail 3.1.7-5 Verge service trough – precast concrete Y-beams



REMARKS

Free-draining infill must be provided for use with this detail.

3.1.8 Parapet beams (edge beams)

The beams along the edge of a bridge have several important functions. In particular, and since they are usually in full daylight compared with other bridge elements, they provide the architectural feature that plays one of the most important parts in defining the bridge's character. They also form the surface upon which the parapets are mounted. The beam can either be cast *in situ* or made in precast units.

In situ parapet beams are normally cast after completion of the deck to achieve a better vertical alignment. While *in situ* parapet beams can be designed as continuous members to provide stiffening to the deck edges, they more often incorporate discontinuities along their length at regular intervals. The form a discontinuity can take is shown in Detail 3.1.8-3. The designer will choose whether or not to include discontinuities and, if so, at what spacing. The choice will relate to the control of cracking, the effective location of the neutral axis of the various members making up the superstructure and the reduction of design loading effects attracted to the edge beams.

Parapet beams, due to their location, are exposed to significant attack from chemicals, abrasion and impact effects. Careful attention must be given to protecting and prolonging the durability of parapet beams by accurate reinforcement detailing and use of air-entrained concrete as well as silane impregnation.

Detail 3.1.8-1 *Parapet beam – typical features*



(A) Downstand



(B) For use with composite decks



PREFERRED	•	Face of	parap	et beam	is
			F F		

Option A is preferred because placing of reinforcement may be easier.

cast in one pour.

- The depth of the parapet edge beam should be constant along the deck. It may differ from one side of the bridge to the other depending on crossfall, however.
 - The width of the parapet edge beam is determined by parapet post fixings and the position of the reinforcement. A width of 500 mm is regularly adopted.
 - 1 in 20 minimum crossfall to parapet beam is required to ensure surface water falls into the deck rather than down the face of the parapet beam, which would cause staining.
 - Where a parapet edge beam carries a lighting column in addition to a parapet system, local widening on the outside of the beam may be necessary (see Detail 5.7.0-4).
 - 75 mm downstand allows for fluctuations in alignment of main deck to achieve good alignment to the parapet beam.
 - Dimension A, width of downstand in Option A, needs to be chosen with care in relation to requirements for reinforcement (see Detail 3.1.8.2) and concrete cover. A value of 200 mm is suggested. See also Section 3.1.3 concerning the downstand.
 - Dimension C, thickness of parapet beam at end of cantilever on Option C, needs to be chosen with similar considerations, but excluding downstand, as for Dimension A.
 - If the main deck is to be left for any length of time before the parapet beam is cast, exposed steel should be grout-washed and a temporary stick-on drip applied to the exposed vertical face to avoid staining of the soffit.
 - Option B is more likely to be used on steel/concrete composite decks where fluctuations in level are greater.

Detail 3.1.8-2 Parapet beam – reinforcement



- PREFERRED Reinforcement Option A is preferred because it is easier to construct. There is no requirement for reinforcement bars to pass through the vertical stop-end at the edge of the deck slab.
- REMARKS Illustration of reinforcement is diagrammatic. The size and strength of the reinforcement needs to be designed to suit the particular circumstances.
 - Cover must be maintained at drip grooves and tucks.
 - The bob (right-angled bend) in the downstand is provided to ensure that longitudinal bars do not fall to the bottom of the shutter during concreting.
 - Option C provides greater flexibility for adjustment of level, which may be needed in steel/concrete composite decks but has the disadvantage of bars projecting through end construction joint of deck cantilever.
 - Care must be taken in detailing the reinforcement to accommodate the anchorage cradle for the parapet.



Sectional plan A-A

REMARKS

- If discontinuities are required for any reason, sealing prevents unsightly stains/leaching.
- Joints are best located mid-way between parapet posts for aesthetic reasons.
- Reinforcement must not pass through the discontinuity joint.
- If the parapet edge beam is cast continuously then the width of the joint is determined by the thickness of a board or separator able to support wet concrete. If lengths of beams are cast in alternate lengths then building paper would be a suitable separator.
- When determining sealant gap width refer to CIRIA Special Publication 80⁽²⁵⁾.

3.1.9 Movement joints

Almost all bridge superstructures contain joints of some kind. Construction joints in concrete permit initial shrinkage. Except where a bridge is particularly designed as an integral bridge (see Chapter 8), joints will also be required to permit expansion and contraction movements caused by temperature fluctuations. The amount of movement will depend upon the length of continuous deck able to move from a fixed point. Even fixed ends of the bridge must allow for some flexural movements. Various joint seals to suit different amounts of movement are shown in Details 3.1.9-1 to 3.1.9-5.

When the longitudinal deck movements become significant it may be necessary to cover the gaps with sliding plates, particularly where pedestrian access is involved. Detail 3.1.9-6 is an example of the use of such plates.

There are three basic functional requirements for movement joints.

- 1. Accommodate movement of the bridge.
- 2. Protect the edges of the surfacing.
- 3. Protect against the ingress or entrapment of water.

These requirements are needed for the following reasons.

- 1. The movements of a bridge cause varying gaps to form between the abutment and the deck and these can get quite large. An expansion joint should afford a smooth transition for the safe passage of vehicles, cyclists and pedestrians onto and off the bridge deck. Open joints collect debris and prevent movement joints functioning and are no longer acceptable, so seals are required that facilitate the movement.
- 2. If unsupported, the exposed edges of the surfacing would soon break away under the action of the traffic, causing potholes to form and presenting a danger to the public. Expansion joints must provide that support.
- 3. Failure to seal the expansion gap against the ingress of surface water opens the inaccessible but exposed faces of deck and abutment to attack from waterborne deicing salts, thereby threatening reinforcement and stressing anchorages and cables in this area. If water is trapped in the surfacing behind the expansion joint, the pumping action of the wheels of passing vehicles can cause the material to break up.

Additional functional requirements are that the joint must, in operation:

- withstand traffic loads and accommodate movements. In so doing, it must not give rise to unacceptable stresses in the joint or in other parts of the structure
- have good riding quality and not cause inconvenience to any class of road user
- not present a skidding hazard
- not generate excessive noise or vibration under traffic
- be able to be easily inspected and maintained. Any parts liable to wear should be easily replaceable
- form a continuous waterproof layer with the deck waterproofing system.

The current UK Highways Agency standard for expansion joints is BD 33/94 ⁽²⁶⁾ and its accompanying Advice Note is BA 26/94 ⁽²⁷⁾. These lay down qualitative and quantitative requirements for proprietary expansion joints both in manufacture and installation.

It is not the purpose of this guide to proceed through the design/specification process, but it may be useful to highlight the importance of the following practical requirements:

- determining the effective bridge temperature at time of installation to get the correct gap width within the range required
- distributing the total movement between the number of expansion joints on the structure (although the current preference is to use continuous construction with a minimal number of joints)
- having due regard to the different users of the bridge, eg use of cover plates for pedestrians
- preparing notes of any special maintenance requirements for inclusion in the maintenance manual.

The generic descriptions of the different types of joints are:

- buried
- asphaltic plug
- nosings
- elastomeric/reinforced elastomeric
- elastomeric elements in metal runners
- cantilever, comb or tooth.

Integral bridges (see Chapter 8) are designed to accommodate movement without expansion joints, but they still require joint seals at the extreme ends.

Detail 3.1.9-1 Movement joint (contraction) – range 0 mm to +3 mm



- AVOID This detail has little capacity for repetitious movements and should not be used for joints at, for example, the fixed end of bridges.
- This joint should be used to maintain integrity of waterproofing at any construction joint where slight movement is anticipated.
 - Use only at construction joints or cracks.

Detail 3.1.9-2 Movement joint – range 0 mm to 10 mm total







Detail 3.1.9-4



- PREFERRED Asphaltic plug type of joint, Option A, is generally proving to be the preferred type.
- REMARKS
- The designer should refer to BA 26/94 ⁽²⁷⁾ and, in particular, to Table 1 of BD 33/94 ⁽²⁶⁾, for further information on joint type options and associated movements.
- At the low end of a bridge a buried joint (Details 3.1.9-2 to 3.1.9-4) should be used whenever possible to avoid entrapment of water.
- For sizes of the joint materials, refer to the manufacturer's details. The sizes shown on the details are typical.

Detail 3.1.9-5 Movement joint – range > 40 mm



Typical proprietary movement joint cast into deck



Typical proprietary movement joint bolted to deck

- REMARKS
 - Where a joint has to permit longitudinal movements in excess of 40 mm, proprietary units will be required.
 - Proprietary joints are usually assembled at the factory ready for use and delivered to site, pre-set for a specified gap dimension, in lengths up to 12 m for casting in or bolting to the structure.
 - Lengths are joined at site by welding or vulcanising to the desired profile.
 - Normally, any neoprene elements in the joint are provided as a single continuous strip, but this can be cut and vulcanised to suit shapes of joints or intersections.
 - Where joints are cast into a structure it is important to allow for connecting the joint to the deck reinforcement in the manner described in the manufacturer's instructions.
 - Where joints are to be bolted to the structure it is important to consult manufacturer's instructions and details when detailing the concrete outlines at the joint. Bolts should come within the deck reinforcement.





Typical plan on parapet beam cover plate

- REMARKS
- This detail need only be considered for joint gaps in excess of 75 mm.
 - Ensure fixing holes are clear of parapet rails and post fixings.
- As an alternative sliding surface to the polyurethane pads shown, neoprene pads or a skim of epoxy mortar would be suitable.
- An allowance of 5 mm should be made for transverse deck movements.
- An alternative arrangement to bending the stainless-steel plate round the angles of the chamfer is to fabricate them round the 90° corner without a chamfer.
- The designer may wish to extend the deck movement joint into the parapet edge beam, in which case a parapet beam cover plate detail would not be required.
- The recess to take the plate is optional. The plate can be fitted without a recess.
- Reinforcement cover must always be considered and maintained.

3.2 SLAB BRIDGES

3.2.1 Preamble

For the shortest spans, simple reinforced construction is the usual choice. It is costeffective since the flat, level soffit results in uncomplicated falsework, formwork and reinforcement. As the span length increases, the slab has to be thicker to carry the load. This extra weight of the slab itself then becomes a problem, which can be solved in one of three ways. The first is to add tendons and pre-stress the *in situ* solid slab; the second is to reduce the dead weight of the slab by incorporating "voids", often polystyrene blocks; thirdly, precast pre-stressed inverted tee beams can be used with *in situ* concrete infill (see Figure 3.1). The precast beams are efficient up to about 16 m span. Voided slab bridges can be used successfully up to about 25 m span and are generally more economical than pre-stressed *in situ* concrete slabs.

Figure 3.1 shows, diagrammatically, cross-sections of typical slab bridges in the various forms of construction, ie the solid slab, inverted tee beam with *in situ* concrete infill and voided slab

3.2.2 Reinforcement detailing

When reinforcement is finally positioned within the confines of a shutter it will be because the designer has considered factors other than those pertaining just to the reinforcement itself. These will include specification, reinforcement type and scheduling, wastage, storage on site, laps, protection and the placing and vibrating of concrete around it. The following text expands on these issues, although fuller guidance should be sought from other industry publications.

Specification

This will normally be to a British Standard or a Eurocode or both, and is usually determined before the contract award.

Type and scheduling

The designer will determine the type – either high-yield or mild steel. Thereafter reference should be made to Clause 5.8.3.2 of BS5400 : Part 4 : 1990 ⁽²⁸⁾ for instruction on scheduling



CIRIA C543

Wastage - supplier

The reinforcement supplier will usually charge its clients for the total steel tonnage used to manufacture the required shape. This tonnage will include those offcuts that remain after cutting and cannot be further used. Prior to cutting and bending, reinforcement is initially provided to the supplier in 12 m straight lengths. The designer should therefore consider either detailing bars as 12 m lengths wherever possible, to minimise handling time, or restricting cut lengths, for whatever final shape, to 2.4, 3, 4 or 6 m. Detailing the reinforcement can then be based on these lengths albeit with a slightly higher frequency of lapping as a result. This practice will also make for easier recognition of the correct bar from a stack on site. Overall, this should lead to cost/time benefits for the client.

Laps/lapping

Lapping should, first, be avoided at positions of maximum stress and, second, staggered as shown on Detail 3.2.2-1, so that no two laps are adjacent to each other.

Placing and vibrating concrete around reinforcement

Clause 5.8 of BS 5400 : Part 4 : 1990 ⁽²⁸⁾ gives guidance on matters affecting design details, particularly with respect to concrete cover to reinforcement. It is important for the designer to consider further whether the aggregates can pass between layers of reinforcement either vertically or horizontally and whether sufficient space is available for vibration and consolidation of the concrete. Space should be left for the normal 75 mm-diameter poker vibrator used on site, although a 50 mm version can be obtained.

Protection

The ingress of chloride ions from winter salting is the main cause of corrosion of embedded reinforcement in the UK so protective measures must be carefully considered at the design stage. Such measures may include corrosion-inhibiting concrete admixtures, increased concrete cover and a concrete surface sealant or impregnant such as silane. Alternatively, reinforcement can be coated with fusion-bonded epoxy, but the use of such reinforcement is not covered by standards and its use in the UK has not been frequent.

It is also important to provide measures for control of early age cracking in concrete resulting from thermal restraints, plastic settlement, plastic shrinkage and loss of moisture. Such measures should include the provision of adequate distribution steel, and good detailing and mix design. Where practical, the use of precast concrete made in factory conditions should be considered to reduce the effect of the above difficulties. Good details and specifications allied to effective site supervision will further assist preventative measures.

The complexity and size of the structural elements influence the amount of concrete cover to reinforcement and placing tolerance. Therefore, it is important when scheduling reinforcement to ensure that the specified cover can be achieved. Particular care is needed at grooves, tucks etc. Detailing of reinforcement also needs to take care of all stages of reinforcement fixing, particularly the need for subsequent layers to be passed between layers already placed and achieve the required cover at all points. Further requirements and advice are to be found in DMRB BD 57⁽¹⁾ and BA 57⁽⁸⁾.

When dealing with voided construction, two approaches to casting the deck are possible. With circular voided construction it is usual to cast the whole deck depth in one operation, and the polystyrene void formers must be secured against flotation as shown in Figure 3.2 in Section 3.2.4. With polygonal voided construction the bottom slab must be constructed first and then the void formers fixed in position as shown in Figure 3.3. Construction of the deck then continues with the webs, followed by the top slab.

Detail 3.2.2-1 Transverse reinforcement – arrangement



Note Detail 3.1.8–2 for reinforcement arrangement. **Cross-section**



Plan

REMARKS

Reverse alternate reinforcement in top and bottom of bridge decks has four advantages.

- (1) Bars are easily adjusted to take out tolerances in the width of deck shuttering, ensuring that the required concrete cover to reinforcement at the bridge deck fascia can be provided.
- (2) Laps are staggered as an inherent part of the method.
- (3) The number of different bar marks is minimised.
- (4) Reinforcement congestion is minimised, thus facilitating placing and compaction of the concrete.
- Last reinforcement to be placed are the top transverse bars (supported by U-bars from bottom).



- Reinforcement shown is indicative only; reinforcement other than main transverse
- reinforcement has been omitted for clarity.

Detail 3.2.2-3 Transverse reinforcement – narrow decks



REMARKS

- Reinforcement shown is indicative only, with reinforcement other than main transverse reinforcement omitted for clarity.
- To minimise cover variations bar scheduling should ensure that leg of reinforcement across the bottom is not a run-out dimension.

Edge cantilevers

3.2.3

When cantilevers are adopted as an edge of deck detail their arrangement is usually dictated by other features of the bridge such as kerb lines, optimum spacing for the supporting members, need for service troughs, appearance (shadow cast), etc.

There is a movement in the industry that favours standardising the overall cantilever length, ie the structural width and cantilever length should be matched so that a standardised cantilever length is achieved. A value of 1400 mm is put forward for current consideration and feedback (see Detail 3.2.3-1). This relates to the standard shutter board width (1219 mm or 4 ft), assuming a 200 mm-wide edge beam. This reasoning may not hold when the cantilever soffit is significantly sloped. Other influences on length of cantilever, W, are considered to be the relationship with depth, D, of the main superstructure. For aesthetic reasons W should be greater than D, preferably 1.5D. (See also Detail 4.8.0-1.)

A parapet beam (sometimes referred to as the string course or edge beam), will usually be required. Reference should be made to Details 3.1.8-1 and 3.1.8-2 for the principles to follow.

Detail 3.2.3-1 Cantilever reinforcement



(B) For horizontal cantilever

REMARKS • Laps in reinforcement are to be provided if cantilevers are to be shuttered after the main deck pour.

3.2.4 Voided slabs

Construction

A reference is made to the placing of the bottom layer of reinforcement in circular voided slabs in Section 3.2.2. Advice on the installation of void formers in bridge decks can be found in BA $36/90^{(20)}$.

Principal features in the construction process that influence the reinforcement detailing of voided slabs are:

- sequence of placing the void formers and fixing the reinforcement
- buoyancy of the void formers in wet concrete
- shape of void formers
- sequence of casting the concrete.

Because void formers are invariably subject to buoyant forces that exceed the weight of the mat of reinforcement it is necessary to tie the voids down positively, to the deck soffit formwork. Circular voids will be tied down with fixings through the deck formwork, see Figure 3.2. The choice of soffit formwork finish must therefore allow for the tie anchors. On polygonal voided slabs the bottom layer should be cast first in order to assist placing and compacting of the concrete and to provide an anchorage for the void former during the subsequent pours (see Figure 3.3). It is usual for the bottom layer of concrete to be curtailed before the edge face is reached so that the construction joint does not affect the fascia concrete (see Figure 3.3). Reinforcement should be detailed with this in mind.

For the polygonal voids the bottom mat of reinforcement is placed on the soffit shutter first together with starter bars for the "webs". Once the top mat of reinforcement is in position it is impossible to place the void formers. Detailing of the reinforcement must take account of this sequence, and starter bars and laps must be placed accordingly.

Drainage

It is inevitable that any leakage through the waterproofing will find its way into the voids and an appropriate drainage route should be provided (Detail 3.2.4-1). The outlet detail follows the recommendation of BA $36/90^{(20)}$. Alternative outlets can be adopted from Detail 3.1.6-3, however.



Figure 3.3 Construction of polygonal voided decks

Void drainage





Section A–A through void (circular void shown)

Enlarged view of downpipe

REMARKS

Outlet pipes are normally positioned at the low ends of voids where a significant longitudinal fall is present. In the case of circular voids, one outlet pipe is sufficient, while in the case of rectangular or polygonal voids two pipes at the same end may be necessary.

- If the voids do not have a significant fall it is recommended that outlet pipes are used at both ends of the same void.
- The detail shown where the outlet pipe leaves the deck slab is as shown in BA 36/90 ⁽²⁰⁾. Other alternatives at the exit point are shown on Detail 3.1.6-3.
3.3 BEAM AND SLAB BRIDGES

3.3.1 Preamble

Beam and slab bridges are probably the commonest form of concrete bridge with spans of up to 40 m in the UK today. This is largely due to the introduction of standard precast pre-stressed concrete beams specifically for use in highway bridge construction. Reference should be made to Precast Concrete Association publications for details of available cross-sections.

They have the virtue of simplicity, economy, wide availability of the standard sections, and speed of erection. The precast beams are placed on the supporting piers or abutments, usually on rubber bearings. An *in situ* reinforced concrete deck slab is then cast on permanent shuttering that spans between the beams.

The precast beams can be joined together at the supports to form continuous beams, which are structurally more efficient but may entail higher costs. However, this approach does allow the number of bearings to be reduced, thereby affording scope for improving access. Further information about continuous construction can be found in Chapter 8.

Some of the different cross-sections can be mixed on the same deck so that the bridge edge appearance and/or function is different from the interior. This is normal practice.

Figure 3.4 shows, diagrammatically, a typical concrete beam and slab bridge.



Figure 3.4 Concrete beam and slab bridge superstructure

3.3.2 Diaphragms

Diaphragms are members constructed transversely to the main superstructure members that provide stiffening to the overall action of the superstructure thereby increasing its load-sharing characteristics. They are constructed over the full depth or part depth of the main beams. Diaphragms mainly occur at support positions where they can also be designed to transmit jacking loads for bearing replacement. They can reduce the number of bearings needed and so provide better access.

In some cases, diaphragms act as little more than trimmers, stiffening what would otherwise be an unsupported edge of the deck slab, but can still be part of a bearing replacement scheme.

The depth and thickness adopted for diaphragms is a matter for careful consideration because the derivation of appropriate details will influence this significantly. Various details follow for one particular type of bridge, but the principles illustrated can be extended to other types.

In this section, diaphragms only at end supports (abutments) are shown. For diaphragms at intermediate supports see Chapter 8, where their continuity is considered in connection with integral bridges.





REMARKS

Either of the two positions shown is an acceptable position for the end of a diaphragm if such a diaphragm is deemed necessary by design.





Detail 3.3.2-3 Diaphragms at skew abutment



- PREFERRED Squared ends with appropriate clearances detailed are preferred.
- REMARKS Width of diaphragm needs to be increased to make allowance for skew dimension of beams.

3.4 BOX GIRDER BRIDGES

3.4.1 Preamble

For a span range of 45 m to 250 m, prestressed concrete box girders are an effective and economic choice of bridge construction. The main spans are hollow and the shape of the box may vary from span to span and along the bridge, ie deeper in cross-section at the abutments and piers and shallower at midspan. Techniques for construction will vary according to the actual design and situation of the bridge.

Figure 3.5 shows, diagrammatically, a cross-section of a typical concrete box bridge. The superstructure cross-sections can have vertical or sloping webs (for example, see Detail 3.4.4-1 or Detail 3.4.3-1) and multiple cells. While concrete box girders are often post-tensioned (using either an internal or external pre-stressing system) they are also successful in reinforced concrete.



vertical walls

Figure 3.5

Concrete box girder bridge superstructure (single-cell type)

3.4.2 Post-tensioning

Post-tensioning the concrete in a box girder bridge is chosen from two fundamental methods. Either "external" cables are used (cables inside the box, usually, but clear of, running alongside, the main concrete sections), or "internal" cables (cables that are threaded through ducts that have been cast into the main concrete sections, then surrounded with grout making them effectively monolithic with the concrete section). The method pursued will follow consideration at the design stage of issues such as costs, ease of inspection, maintenance and cable replacement. It is clear from an inspection, maintenance and replacement standpoint, that external cables are an advantage. Other aspects, however, may lead the designer towards adopting internally grouted cables. The internal cable method has proved popular in the past but concerns have arisen about the long-term overall structure durability. The designer is advised to seek further guidance from specific publications such as the Concrete Society Technical Report No. 47, *Durable Bonded Post-Tensioned Concrete Bridges*⁽²⁹⁾, and BD 58 and BA 58, *The Design of Concrete Highway Bridges* and *Structures with External and Unbonded Prestressing* (DMRB 1.3.9 and 1.3.10).

The principal components of the external pre-stressing system are the anchorages and the deviators. The anchorages are where the large pre-stressing forces for the tendons are locked off and transferred into the structure. The deviators are the devices to change the slopes of the tendons and convert the longitudinal forces in the tendons into the vertical forces that support the bridge.

Exposed anchorages are usually securely protected behind a removable cap. If the bridge needs to be rehabilitated, the caps can be removed and the tendon ends bared for detensioning if sufficient length of cable has been left uncut.

In whatever system, if replacement of pre-stressing cables is intended, consideration must be given to the design of the gallery/back wall configuration and the allowance for sufficient strand to remain beyond the anchorage for de-stressing.



- (B)
- PREFERRED •
- "Vertical" deviators, Option A, are preferred as causing less obstruction to access and drainage and offering deviator capability at both top and bottom.
- REMARKS
- Option B is suitable where bottom tendons are used at the centre of the box.
 - Option B obstructs access and drainage.
 - The possibilities for standardisation of the form of deviators within a project, along with the need for anchorage blisters (see 3.4.2-3), need to be considered by the designer and detailer.







• Dimensions Y and H, setting out point for anchorage, value to suit pre-stressing jack clearances and tendon profile.

3.4.3 Ventilation and access

Where inspection and maintenance is required to take place inside a box girder the working space must be treated as a confined space. As such, it is subject to the Confined Spaces Regulations 1997 and, in particular, the Approved Code of Practice, Regulations and Guidance ⁽¹⁹⁾ issued by the Health and Safety Commission (HSC).

Aside from these issues, it is good practice to provide ventilation to what would otherwise be a closed box; Detail 3.4.3-1 shows one such measure. Another method is to keep the ends of the boxes open, in which case other forms of ventilation may not be necessary.

If remedial works need to be carried out inside a box, forced ventilation would most likely be required regardless of whatever permanent vents are present.

Detail 3.4.3-1 Ventilation – box superstructure



Detail of ventilation tube

REMARKS

Size and spacing of ventilation ducts are dependent upon box volume and whether any other means of ventilation are present.

- Extra ventilation ducts are only as required by design. Access manholes may double as ventilation points.
- Locate vents so that they are not obstructed by external tendons at high level.
- Ventilation ducts often serve other purposes during construction.

Box – drainage

Although in theory the application of waterproofing to the top surface of box structures will keep the interior dry, industry experience indicates that some seepage will still occur. Therefore the design should ensure that seepage water cannot collect inside a box. Unless a truly sealed environment is required for the box interior, drainage outlets should be incorporated through the box soffit. The drainage path must be continuous and trapped water, eg behind deviators, avoided. Outlets should be positioned clear of tendons and other interior items.

Detail 3.4.4-1 Drainage through deviators



- REMARKS
- Drainage holes through deviators should be positioned on the low side of the bottom slab, but they may be required on both sides if the bottom slab is level.
- Where the positioning of the post-tensioning ducts necessarily encroaches on the most suitable position for the drainage hole, the hole may be re-sited as close to the optimum position as possible.
- The hole shape and dimensions are considered to be the most efficient, but each aspect may be varied to suit individual conditions.
- Water collected inside the box at the lowest positions in each span can be discharged through outlet pipes as shown on Detail 3.1.6-3.
- Where pre-stressing anchorage blisters occur at low level and holes cannot be permitted to pass through, then the floor of the box should have falls to re-direct the water.

3.5 SUBWAYS AND CULVERTS

3.5.1 Preamble

Figure 3.6 shows a longitudinal section and end elevation on a typical rectangular subway or underpass.

The internal dimensions of the structure are determined by the intended usage and can accommodate pedestrians, equestrians, cyclists and vehicles either singly or in any combination. Refer to the DMRB $^{(17)}$ for guidance on these dimensions.

The box sections can be formed using *in situ* concrete or precast concrete, the latter being manufactured in a range of sizes to suit most applications mentioned above and also for culverts. Where precast concrete is selected, consideration must be given to the weight of the units and how they will be manoeuvered on site.

For details of precast subways and culverts reference should be made to the proprietary suppliers.

The majority of the remainder of this section concerns in situ concrete construction.

3.5.2 Reinforcement

Subways and culverts are often constructed *in situ* in three stages, ie the base, the walls and the top slab. Starter bars are needed at the construction joints, which are at the box corners, and bars must project sufficiently to develop the required bond strength.

Sometimes the height of wall and span of top slab are such that the ends of one set of laps encroach very near to the start of the next. Also, if conventional starter bars are adopted for the top slab, the bars will need to cantilever out from the inside face of the wall across the top of the box. This cantilevering of bars obstructs the movement of shutters during wall construction and, in extreme cases, the bars will need to be separately supported. In these cases, the designer should ensure where possible that "standard" lap lengths are replaced by designed lap lengths.

Several studies have been made into the relative strengths of different reinforcement arrangements including those reported by the Cement and Concrete Association ⁽³⁰⁾. Some arrangements avoid any cantilevering of bars across the box. If the design can tolerate the reduction in efficiency of this type of reinforcing, their use can greatly simplify the construction.



Typical longitudinal section



Half cross-section/half portal elevation

Figure 3.6

,.....

Subway/underpass structure

3.5.3 Waterproofing and drainage

To avoid unsightly penetration of damp into subways a waterproofing system must be applied to the outside of the structure. Waterproofing the roof is essential to remove the greatest risk of penetration. A good drainage system behind the walls will relieve water pressures and reduce the risk of penetration through the walls and, depending upon the finishes chosen, it may not be necessary to apply a full tanking system to the walls.

Detail 3.5.3-1 shows a typical subway cross-section. Details 3.5.3-2 and 3.5.3-3 show an arrangement of waterproofing on the cross-section and behind the headwall.

Detail 3.5.3-4 shows the collection of surface water at the subway portal. Where the discharge is too low for the normal highway drainage system, pumping will be needed. This requires careful detailing of the collection chamber and security measures to allow ease of maintenance, pump replacement and resistance to vandalism.

Detail 3.5.3-1 Subways – typical cross-section



REMARKS

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- Base extended to increase bearing area can act as a foundation to permeable backing.
- Surfacing shown is appropriate for accommodation (access) traffic. Where other wearing surfaces are required, appropriate detail must be provided.

Detail 3.5.3-2

Subways – waterproofing



Typical cross-section

Detail 3.5.3-3

-3 Subways – waterproofing at end



Waterproofing to wrap over side edge to extend below construction joint





- PREFERRED

 Option A
- Option B is cheapest to form but is not recommended for pedestrians, cyclists or equestrians.
 - Water collected in cut-off drains must be discharged into a main highway drainage system. Direct discharge may not always be possible due to difference in levels, in which case this would need to be through pumps.
 - Grating must be robust and fixed down.
 - Outlet pipe must be of bigger diameter than holes in inlet grating. Generally, a minimum diameter of 150 mm is recommended; experience has shown that smaller pipes are easily blocked.

3.5.4 Joints

Where possible, subways and culverts should be designed to be constructed without joints. Where joints cannot be eliminated the principles in Detail 3.5.4-1 should be followed.

Detail 3.5.4-1 Sul





REMARKS

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- Designer should establish the spacing of joints, both construction and movement, to suit reinforcement of wall and necessary resistance to cracking. Detail shows typical solution only for identification of details.
- For discussion of position of construction joint in wall above base slab (kicker height) refer to Section 7.2.



Base construction joint

REMARKS • Groove for sealant is cast into the first side of joint poured. Joint former is left in during casting concrete of subsequent pour.

Detail 3.5.4-3 Subways – movement joint



- REMARKS
- Roof movement joint: detail of top shown is appropriate where vertical clearances are critical and road surfacing needs to be laid directly on to the subway roof.
- Where vertical clearances are sufficient to allow a protection layer to be applied over roof waterproofing (see Detail 3.5.3-2), carriageway construction need not be saw-cut and sealed.
- Base movement joint: detail of top shown is appropriate for accommodation access (minimal) traffic. Where other wearing surfaces are required, appropriate detail must be provided.
- Proprietary products used at joints are to be selected and applied in accordance with the manufacturers' requirements.

Detail 3.5.4-4 Culverts – joints, in situ construction



REMARKS • Special precautions should be taken during construction of Option B to ensure good alignment of dowels.

Detail 3.5.4-5 Culverts – joints, precast construction



REMARKS • Joint sealant on either or both faces is optional.

3.5.5 Lighting

The need for the lighting of a subway will depend upon its usage and the policy of the owning authority. Civil engineering provisions for lighting should be established in discussion with the lighting and electrical engineers at an early stage of the design to allow incorporation of the necessary details.

Provisions will depend upon the nature and number of the lighting fittings required. The choice of fittings will be determined by the location of the subway and the need for resistance to potential vandalism.

Detail 3.5.5-1 shows one such installation for cross-connection. Actual requirements will be decided to suit local requirements.



- REMARKS •
- If cast in ducting is needed, duct sizes and bends radii must be sufficiently large to allow easy drawing of electric cables.
- Locations of junction boxes and draw pits to be decided in discussion with services engineers.
- Corrosion-resistant fittings should be specified.
- All units in pedestrian subways should be cornice-mounted and vandal-resistant.

Steel superstructures

GENERAL

4.1.1 Preamble

4

4.1

This chapter illustrates details for the most frequently used form of steel bridge, the steel girder and concrete slab composite bridge (see Figure 4.1). This is the commonest usage of steel in highway bridges in the UK. Its benefits are the simplicity and speed of erection of the main support members (steel). This is followed by casting of the concrete deck within formwork supported by the main members.

For bridges with spans up to about 25 m, universal beams are feasible; however, above this span fabricated plate girders are normally used. The details in this chapter primarily relate to plate girders, but many apply also to universal beam solutions.

Figure 4.2 shows a beam and slab superstructure in diagrammatic form. It provides a key to some of the details to be found in this chapter, which are as follows:

4.1.3-1	Welds – web to flange	4.7
4.1.5-1	Doubler plate	4.9
4.2.1-1	Shear connectors – studs	
4.2.1-2	Steel/concrete interface - top flange with in situ deck	
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4.2.3-1	Permanent formwork - precast concrete planks	
4.2.4-1	Permanent formwork - GRP - arrangement of reinforcement	
4.2.4-2	Permanent formwork – GRP – panel arrangement	
4.2.4-3	Permanent formwork - GRP - bearing on to flange	
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4.2.4-5	Permanent formwork - GRP - typical data tabulation	
4.3.2-1	Intermediate stiffener	
4.3.2-2	Layout of intermediate web stiffeners	
4.3.3-1	Bearing stiffener - intermediate support, single leg	
4.3.3-2	Bearing stiffener - intermediate support, twin leg	
4.3.3-3	Bearing stiffener – end support	
4.3.4-1	Welds - cope hole	4.30
4.4.0-1	Splice	
4.4.0-2	Splice – plate	
4.5.2-1	Cross-bracing – elevation	
4.5.2-2	Cross-bracing – connection	
4.5.2-3	Cross-bracing – intersection	
4.6.0-1	Plate girder cross-head	
4.6.0-2	Cross-head to main girder connection	
4.6.0-3	Typical stiffener/bottom flange connections at bends in flange	
4.7.0-1	Steel girder make-up	
4.7.0-2	Variable-depth girder – bent flange at pier	
4.8.0-1	Weathering steel – edge cantilever	
4.8.0-2	Weathering steel – runoff strip	

Each detail is to be read and used in conjunction with its own notes, but the text discusses the background to the choice of details.

The principal arrangements of beam and slab bridge superstructures are those where the main girders are supported:

- directly on the columns and abutments, leaf piers or by column cross-heads; or
- by a transverse spanning pier diaphragm beam that acts integrally as a cross-head and is itself supported by the substructure.

The latter arrangement is further discussed in Section 4.6.

When detailing steelwork, account should be taken of the maximum sizes of unit that can be transported to the site. The positions of site joints may depend upon this. Each site will have its own access problems and must be studied individually. The overall height limit (including vehicle bogies and bed) is generally 4.5 m with the permissions for special loads typically allowing up to 5.0 m. However, the vehicular transport regulations limit ordinary loads to 18.3 m in length. Overall widths are generally limited to 2.9 m where possible, but the abnormal load width limit is 4.3 m. Curvature will, of course, increase encroachment on these limits for the same cross-section. The standard limits can be exceeded when special transport arrangements are made and reference should be made to SBG Guidance Note 7.06 ⁽⁶⁾ and CIRIA Report 155 ⁽³⁾.

Available steel plate sizes vary from manufacturer to manufacturer and reference needs to be made to them. Usually flange and web plates are cut from plate, available in thicknesses 10, 12 and 15 to 75 mm in 5 mm increments, up to a maximum length of 18.3 m. Other thicknesses can be provided if ordered in sufficient quantity. Longitudinal camber in a plate girder is achieved by cutting the web to suit.

Two principal forms of connection are in use in bridge construction, namely:

- bolts
- welds.

Welding provides the more versatile connection and it results in a tidy finish. As a general rule, shop welding is cheaper than bolting and, therefore, unless bolts are required for a particular reason, workshop connections should normally be specified as welds. Reference should be made to SBG Guidance Note 1.09⁽⁶⁾ for a comparison of bolted and welded site connections.

The usage of bolts is outlined in Section 4.1.2. Welds are dealt with generally in Section 4.1.3, which includes a particular detail of a web to flange weld, typical for bridge I-beams.

Other forms of construction, such as orthotropic decks, are outside the scope of this guide and readers should obtain specialist advice about these.





Cross-section

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Figure 4.2 Steel/concrete composite bridge superstructure (diagrammatic)

Half cross-section showing typical cross-bracing

Bolted connections

4.1.2

High-strength friction-grip (HSFG) bolts make the connection by pressing together the two surfaces to be joined sufficiently tightly to transmit the design force in friction between the surfaces in contact (faying surfaces). This ensures no slippage occurs up to the designed serviceability capacity and, as a result, the stiffness of the joint is more predictable than that of one using black bolts. HSFG bolts can be tightened with controlled applied torque or using the "part turn" method. To ensure full design friction is achieved, detailing must be compatible with the design assumption for slip factor, eg the surfaces are coated with aluminium spray, zinc primer, or are bare (grit-blasted) etc. Full paint systems are never used on faying surfaces. Reference should also be made to SBG Guidance Note 2.06 ⁽⁶⁾.

Use of fitted bolts (into close-tolerance holes) is also permitted, but they are rarely used. They are not recommended because of the high cost of the accurate drilling required and because of the risk of thread damage.

"Black" bolts (bolts that are not pre-loaded in normal clearance holes) are permitted only for joining or attaching non-structural elements of items. It is a requirement of BS 5400 ⁽³¹⁾ that black bolts are not used in permanent main structural connections of highway and railway bridges. Where members are included for temporary purposes only, such as temporary wind bracing or stiffening for transportation, black bolts are normally satisfactory. They are tightened using simple conventional methods without torque control and are cheaper. Where black bolts are used in permanent (non-structural) connections consideration should be given to providing lock nuts or similar to prevent the bolts working loose.

4.1.3 Welded connections

Welding is a skilled process utilising trained and tested operatives working to an approved or certified welding procedure. To be performed successfully, the working conditions need to be of a minimum standard. Under factory (workshop) conditions the environment should be suitable for both automated and manual welding.

Welding of site joints is usually avoided unless appropriate precautions can be taken. Site welding is appropriate:

- when there are particularly demanding requirements with respect to visual appearance; or
- when set-up costs including testing and time-related costs can be spread over a significant number of joints.

The working environment must be considered and, where found to be unsatisfactory, must be controlled if welding is proposed.

There are two main types of weld, fillet and butt. Fillet welds are typically used in Igirder web/flange joints and for attachment of stiffeners and gusset plates and/or when joints can be made by lapping the members. Butt welds may be full or partial penetration. Full-penetration welds are typically used in shop splices in webs or flanges (including at changes in plate thickness).

Fillet welds are detailed according to the design strength required but are usually standardised so that not too many different weld sizes are specified. The 6 mm fillet weld is recognised as the industry minimum for bridges. This size and 8 mm can usually be completed with one pass using any welding process, while 10 mm and 12 mm are

possible with automatic equipment. Larger sizes are proportionately more expensive, often needing more than one pass of the welding equipment and additional preparation per pass. The designer will design for strength according to the throat thickness of the weld, but the size of the fillet weld on drawings in the UK has traditionally been given as the leg length (which has a nominal value $\sqrt{2}$ times the throat size). In other European countries the convention is not firmly established and either throat or leg length can be in use. The trend is towards using the throat thickness to specify fillet weld size (see Detail 4.1.3-1 and BS EN 22553⁽³²⁾, the current UK standard which covers both methods).

Certain processes, such as submerged arc, are often used for web/flange welds in plate girders and achieve significant penetration (and thus a larger effective throat) with a given leg length.

Where butt welds are specified it should be made clear (in most cases) that they are required to be full-penetration welds. Where the design strength of the weld is required to be equivalent to the full strength of the parent metal this should additionally be specified as a full-strength butt weld. Note that a "full-strength" weld can be made without full penetration but with external reinforcement.

Partial-penetration butt welds should generally be avoided.

Whichever weld type is chosen the access available for welding should be considered by the designer, to ensure that welds can be properly made and inspected.

Care should be taken to design the correct size of weld for its function. Distortion increases as weld sizes increase. Oversize welds should therefore be avoided.

Detailing principles are illustrated on a web to flange weld, Detail 4.1.3-1, which is to be read in conjunction with Figure 4.3.



- Option A shows a requirement for deep-penetration fillet weld. This results in a connection that, under workshop conditions, is cost-effective. Option B shows a standard fillet weld (which requires a greater weld metal deposit to achieve the same strength). (NOTE: Not all fabricators have equipment capable of achieving deep-penetration welds).
- Welds specified as full-penetration butt welds should be avoided because they require the additional work of back-gouging and testing.
 - Welds specified as partial-penetration butt welds should be avoided because of extra preparation costs. Imperfections are difficult to eliminate and much testing is required to ensure the required strength is achieved and that the root shape is not sub-standard.
- Detailing should permit automatic fabrication to be used wherever possible, best achieved by early pre-design discussion with contractors.
 - The size of weld should be kept as small as is adequate for the design, because shrinkage effects and cross-bending of the flanges will increase as the weld size increases. 6 mm fillet welds are commonly used.
 - Symbols specified in accordance with BS EN 22553 ⁽³²⁾ require dimensions W, P and/or F:
 - W, specified design throat thickness of the deep-penetration fillet weld, is to be stated using the "s" before the value
 - P, the apparent (surface) size of the deep-penetration fillet weld, is to be stated after the throat thickness using the "a" before the value. The difference between W and P is the depth of penetration
 - F, the designed fillet weld leg length, is to be stated using "z" before the value.





Deep-penetration fillet





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Welds – diagram showing dimensioning

4.1.4 Fatigue

Elements of steel bridge superstructures are subjected to cyclic stress fluctuations principally arising from the repeated passage of vehicles. Where the range of these stress cycles is high or where there are a great number of reversals of even a small range there is a potential for fatigue damage. Most elements have a reasonable resistance to fatigue. However, the basic live load stress in an element can be magnified by the presence of geometric stress concentrations and particular weld details can cause further, local, stress concentration. The choice of welded connection detail is a specialist activity.

BS 5400 : Part 10 ⁽²⁴⁾ classifies shape, weld and attachment details broadly in accordance with the nature of their local effects on stress and, hence, their susceptibility to fatigue damage. The level of stress concentration is usually taken into account in the detail class, but sometimes has to be calculated, and is a major consideration in the detailing of railway bridges.

Typical constraints resulting from requirements to avoid fatigue damage can be seen in Details 4.3.2-1 and 4.8.0-2.

4.1.5 Doubler plates

Limitations in construction depth occasionally dictate that thick flanges and webs are appropriate. If plate thicknesses required exceed those available or practicable the required section strength can be achieved by welding or bolting on additional plates (doubler plates) to increase the flange cross-sectional areas.

While doubler plates are sometimes used over the full length of the girder they are generally required to increase the strength only over portions of a girder and can be curtailed outside these portions. As the need for the additional plating ceases, doubler plates are often tapered in width (and sometimes in thickness although this is an expensive process) to reduce stress concentrations and transfer the forces more efficiently. The taper also provides a doubler plate detail with acceptable fatigue endurance (reduction of length of the end transverse weld permitting higher design stresses at the connection).

Care needs to be taken to ensure that the thickness of a doubler plate is not reduced below that at which problems of local buckling could arise. The b/t ratios for the plates attached by side welds need to be checked.

Detail 4.1.5-1 Doubler plate



Flange doubler plate end

REMARKS • Doubler plates are most commonly used for railway bridges to provide a minimum construction depth.

- Dimension t_d, doubler plate thickness, should not be more than flange plate thickness, t_f.
- Dimension R, radius at end to reduce stress concentrations. Recommended value of R not less than W_d/4.
- Dimension W_d, width of doubler plate, should generally be less than parent girder flange width, W_f, by at least 2 × 50 mm or 2 × t_f, whichever is greater.
- Dimension L is taper length designed to reduce stress concentrations. Rate of taper is usually 1 in 4 each side.
- Dimension F is the designed fillet weld leg length.

4.2 STEEL/CONCRETE INTERFACES

4.2.1 Shear connectors

For a steel/concrete girder to act compositely the steel and concrete must be connected together in a way that allows full transmission across the interface of the horizontal shear stresses. The most frequently used form of this connection is by using shear studs and the details herein refer only to these. Studs have the benefit of the results of extensive testing and successful use in service, which have led to the established requirements for their use.

Edge distance of shear stud to edge of flange is required by BS 5400 to be a minimum of 25 mm and should preferably be larger; 50 mm is suggested (Dimension G on Detail 4.2.1-1). This larger dimension is compatible with the usually specified "return" of the shop coats of protective coating. The final choice of edge distance must also take account of the width of seating required for any permanent formwork (see Section 4.2.3).

Detail 4.2.1-1 (and Details 4.2.1-2 and 4.2.1-3, which also refer) are suitable for rolled beams, fabricated plate girders or box girders.





REMARKS

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- Dimensions are to suit the requirements listed in Table 4.1 and the additional conditions below.
- Dimensional constraints in Table 4.1 are derived to satisfy the requirements of BS 5400 but are enhanced by the recommendations of good practice within the industry and Eurocode 4 Part 2 ⁽³³⁾.
- Dimension A, centres of studs across line of girder, value to suit requirements for resistance to separation.
- Dimension D, shear stud diameter, should be chosen with due regard to the available supply.
- Dimension E, giving position of bottom mat of transverse steel reinforcement above flange, value to comply with requirements for durability as the reinforcement extends into adjacent exposed concrete soffit.
- Dimension L, overall height of shear connector, value to suit requirements of dimensions C and F.
- Reinforcement shown is illustrative only but is typical of normal requirements of an *in* situ deck slab.
- Where permanent formwork is used (see Detail 4.2.3-1) other constraints apply to the dimensions.

Table 4.1

Dimension		Not greater than		Not less than		Additional constraints
		Relative	Definitive (mm)	Relative	Definitive (mm)	
'A'	Centres of studs across line of girder	40 t	600	2.5 'D' *		* except in haunches with sides >30° to plane of steel flange where not less than 4 'D' is required
'B'	Centres of studs along line of girder	4 'L' 3 h 25 t	600	5 'D'		
'C'	Centres of transverse reinforcement (across shear plane)	4 'J'	600			
'D'	Shear stud diameter	2t*				* except where the flange is subjected to tensile stress where 1.5 t is required
Έ'	giving position of bottom mat of transverse steel reinforcement above flange		50 *	a - 5 mm	25	* except where permanent formwork is used or there is a haunch not exceeding 50 mm: when 'E' =< 80 mm.
'F'	giving position of underside of head of shear stud above top of transverse steel reinforcement				40 *	* except in regions of sagging longitudinal moment where the underside of head to be not less than 40mm into the compression zone
'G'	Clear distance from shear stud to edge of flange	9 t	100		25 *	 * 50 preferred, * except where permanent formwork is used when adequate clearance to ends of precast planks is required
'H'	Diameter of head of stud			1.5 'D'		
Έ'	Overall height (length) of stud	h - r ** preferred		h - 25mm*	100*	**absolute maximum to be not greater than h - 25 mm *lesser of

Key

t Thickness of flange plate

h Thickness of concrete deck above flange

r Nominal cover to reinforcement

a Aggregate size

values in italics will reduce when fy exceeds 235 N/mm²

reduction factor = $\sqrt{235/fy}$

where fy = nominal yield strength of the steel in N/sq mm

Detail 4.2.1-2 Steel/concrete interface - top flange with in situ deck



AVOID
Detail 4.2.1-3 Steel/concrete interface – at in situ concrete downstand



- AVOID Need for downstand results from effect of crossfall or camber on wide bridges or box girders or other design constraints. Avoid use of a downstand where possible.
 - Where Dimension G needs to be less than 50 mm (because of the required arrangement of shear studs) Option B should not be used (because of compromise to concrete cover).
- REMARKS •

Width at top of downstand not to come within 45° line from base of edge shear stud (BS 5400)

- Where slopes are used and crossfall causes one slope to be critical, keep the other slope the same.
- Dimension G, clear distance from stud to edge of vertical concrete, value to suit:
 - (a) minimum distance (see Detail 4.2.1-2, Shear connectors studs)
 - (b) minimum recommended concrete cover to suit environmental conditions at the deck soffit.

Permanent formwork

4.2.2

For decks cast *in situ*, support has to be provided for the wet concrete. The provision and removal of conventional plywood or steel formwork, along with its temporary supports, can be an expensive operation for bridges. Formwork that is left in place for the life of the structure can offer substantial cost savings. Permanent formwork must be durable and guaranteed to have the same life expectancy as that of the structure itself. Concrete (reinforced) planks are commonly used and glass-fibre-reinforced plastic also satisfies durability requirements. Reference should be made to DMRB Advice Note BA 36/90⁽²⁰⁾.

Where permanent formwork is used (Details 4.2.3-1 and 4.2.4-2) the tolerance on the length of formwork planks or panels is critical. In the temporary state, the length should:

- provide adequate bearing on the girder flange, allowing for tolerances
- have sufficient seating length to allow for accidental lateral displacement (displacement transverse to the main bridge girders).

Since the lateral spacing between the steel girders interrelates with these requirements the placing of the girders is equally critical. To control the lateral alignment of the girders it may be necessary to detail spacer bracing in locations where it would not otherwise be required for structural reasons (see Section 4.5).

The seating length for the permanent formwork should be minimised (see Details 4.2.3-1 and 4.2.4-4), to maximise the extent of *in situ* concrete surrounding the shear studs on top of the steel girder. There is thus a need to consider the risk of accidental dislodgement. As soon as each permanent formwork element is landed it should be adequately secured to the main girders, or to adjacent elements that have themselves been well secured.

4.2.3 Precast concrete permanent formwork

The designed position of all reinforcement in precast concrete plank formwork (Detail 4.2.3-1) is critical especially where the plank meets the girder flange. The reinforcement is constrained in all directions to:

- provide correct effective depth for strength
- comply with specified concrete cover requirements for durability
- provide space between bars and the permanent formwork in accordance with design code of practice requirements to avoid impeding concrete placing
- avoid interference with the positions of shear studs
- be compatible with all top flange steelwork details such as cover plates etc.

The resulting arrangement of the reinforcement that runs parallel to the formwork planks needs to be consistent with the width of the planks.

Some concrete planks in common use for permanent formwork rely upon a welded truss of reinforcing bar for their strength to span in the temporary condition. The bottom of this truss is cast into the surface of the concrete (Detail 4.2.3-1). The manufacturer designs the truss to suit the plank span, main deck slab thickness and concrete placing loads. Should the bridge designer require any of the reinforcement within the permanent formwork plank to be participating in the permanent works design strength it is recommended that the detailer calls up such reinforcement and adds the instruction "do not tack weld" or "permanent works reinforcement not welded to lattice".

Control of the manufactured dimensions of the planks and of the detailing are critical to the success of the use of precast concrete plank permanent formwork.

Detail 4.2.3-1 Permanent formwork – precast concrete planks



Detail at end of permanent formwork plank

Section A-A through plank

REMARKS

• Refs S, T, and U are to refer to the relevant project specification clause numbers.

- Dimension D, clearance between main slab reinforcement and precast plank, value to be aggregate size + 5 mm.
- For critical dimensions of shear studs, see Detail 4.2.1-1.
- Refer to Table 13 of BS 5400 : Part 4 ⁽²⁸⁾ and BD 57/95 ⁽¹⁾ for concrete cover to reinforcement.

4.2.4 Glass-reinforced-plastic (GRP) permanent formwork

GRP formwork is generally thinner than precast concrete planks, so the positioning of reinforcement in the formed concrete slab is less critical. Transverse reinforcement may be spread more evenly along the length of the girders, care being required only to ensure adequate clearance between bars and the formwork ribs to avoid obstructing the flow of concrete during placing (see Detail 4.2.4-1). The GRP itself provides a durable protective layer to the underside of the deck slab and the provision of concrete cover to the soffit of the slab is then not as critical to durability.

The inherent flexibility of the thinner GRP formwork dictates that particular allowance must be made for its deflection under the load of the wet concrete. Either:

- the formwork must be manufactured with an accurate pre-camber that will avoid a permanent downward bulge (increase of thickness) in the soffit of the deck slab between the girders (see Detail 4.2.4-4); or
- an allowance would need to be made for the extra weight of the increase in average thickness of the concrete.

GRP formwork (in its conventional form) does not contribute to the final strength of the reinforced concrete deck slab. It must, however, be of a form that attaches to the soffit of the formed concrete deck sufficiently securely to ensure it will not become detached during the whole life of the bridge.

Detail 4.2.4-1 Permanent formwork – GRP – arrangement of reinforcement



Note: Bars B1 to be placed locally to maintain 'C' cover to formwork

Section through permanent formwork showing relationship to reinforcement

REMARKS

Dimension C, clearance between main slab reinforcement and GRP panel, value to be aggregate size + 5 mm.



Plan on permanent formwork units

REMARKS

- Dimension B, nominal width of a panel, to be slightly larger than the manufactured dimension of the individual panels, which allows a manufacturing and placing tolerance.
 - Dimension P, position of end ribs, is from manufacturer's data.
 - Panel details may vary from those illustrated, depending upon the manufacturer.
 - For sample of tabulated data, see Detail 4.2.4-5.
 - For typical value of Dimension G, overlap of permanent formwork on to beams, see Detail 4.2.4-3.

Detail 4.2.4-3 Permanent formwork – GRP – bearing on to flange



Section at edge of top flange

- REMARKS
 Dimension G, overlap of permanent formwork onto beams. Value depends upon formwork design and span and as otherwise dictated by interference of structural features, such as the presence of girder splice plates. Typical minimum value is 40 mm.
 - Dimension T, thickness of GRP, value to be from manufacturer's data.
 - Dimension K, extent of outer-edge of butyl mastic strip, to be approximately 2 mm less than G.

Detail 4.2.4-4 *Permanent formwork – GRP – pre-camber*



Section through deck beams

REMARKS • For Dimensions A, C and D, refer to Detail 4.2.4-5.

Detail 4.2.4-5 Permanent formwork – GRP – typical data tabulation

TABLE OF PRE-CAMBER VALUES (SAMPLE)					
LOCATION REF (eg STRUCTURE No)	GIRDER SPACING C/C (mm)	ACTUAL CLEAR SPAN (mm)	MODEL TYPE (manufacturer's reference)	PRE-CAMBER (mm)	
SN	A	С	E	D	

TABLE OF GRP	ORMWORK DATA	(SAMPLE)		
MODEL TYPE	OVERALL	CENTRES OF	No OF RIBS/	MAXIMUM
(manufacturer's	HEIGHT (h)	RIBS (†)	PANEL	CLEAR SPAN
reference)	(mm)	(mm)		(mm)
E	Н	R	(from	J
			manufacturers'	
			data)	

- REMARKS Tabulation of parameters and dimensions of formwork panels may be necessary for correct manufacture of the GRP panels.
 - Notation SN is to be appropriate identifier code for panel location.
 - Dimension A, girder spacing: keep identical as far as possible to help maintain Dimension C as constant.
 - Dimension C, clear spans between girders and thickness of concrete deck slab. Keep identical as far as possible to avoid small changes in the panel dimensions.
 - Non-standard details of permanent GRP formwork panels (ie different from the particular manufacturer's norm (E)) may need to be agreed with the manufacturer and developed for particular situations. In such cases, the non-standard dimensions and parameters should be clearly identified on the drawings.
 - Dimension D, required pre-camber, value to suit stiffness of particular formwork model type and actual span.
 - Dimension H, overall height of panel, from manufacturer's data.
 - Dimension R, centres of ribs of permanent formwork, from manufacturer's data, but may, with agreement of manufacturer, be non-standard to suit required reinforcement pattern.
 - Dimension J, maximum clear span, is the limiting clear span dictated by design of GRP formwork for the particular thickness of concrete slab.
 - See Detail 4.2.4-4 for illustration of dimensions

4.3 STIFFENERS

4.3.1 Preamble

The principal function of a stiffener is to prevent web buckling by reducing the size of the unstiffened panel dimensions. Web stiffeners are usually required on only one side of a web (see Detail 4.3.2-1). Stiffeners can also transfer forces from bearings or point loads up and down through the body of the girder, or as part of a bracing system as full-length members or shorter members (gussets) connecting with flanges.

The thickness of stiffener plates should be equal to a plate thickness already used elsewhere on the bridge, because small amounts of plate of different thicknesses are uneconomic. The thicknesses chosen should preferably be from plate in general stock (see Section 4.1.1).

Stiffeners may need extension beyond the widths derived from strength design to allow the connection of bracing members (see Detail 4.5.2-1) etc. Provided that the strength of the extended stiffener is adequate with reduced notional yield strengths their extension need not comply with the nominal limits.

4.3.2 Intermediate web stiffeners

Transverse (vertical) stiffeners

The intermediate stiffener in Detail 4.3.2-1A is attached to the top flange and stabilises/ stiffens the connection between top flange and web. This provides resistance to transverse flexure arising from local loading of the deck slab. Potential fatigue problems can arise in the flange/web weld if there is transverse participating bracing, and this may need to be taken into account. Local transverse plane frame analyses can indicate the order of magnitude of the stresses.

The unattached ends of stiffeners are shown shaped (a slope of 1:2 is suggested as being the compromise most preferred in the industry). This allows for making the weld and for access for application and maintenance of the protective coating over the main girder flange to web weld and to the end of the stiffener itself.

Sizes of stiffeners are governed by the outstand limits, which are related to the thickness of the stiffener plate (refer to BS 5400 : Part 3 $^{(31)}$, which specifies a maximum width-to-thickness ratio of 10:1 for S355 steel unless other design criteria are satisfied).

To improve long-term durability, ease of maintenance and aesthetics, intermediate stiffeners on edge girders of steel/concrete composite bridges should be located on the inside face of the web, thus reducing their exposure to weather. A typical layout of intermediate stiffeners is shown in Detail 4.3.2-2.

-1 Intermediate stiffener



REMARKS

- Dimension D9, gap between stiffener and flange, typically 25 mm to 35 mm, to:
 - (a) be sufficient to allow welding rod manipulation for sealing weld across ends of stiffener
 - (b) but not exceed maximum of five times thickness of the girder web
 - (c) preferably not exceed 50 mm.
 - Dimension D10, setback of stiffener where it joins flange, typically 20 mm to 25 mm, to be sufficient to ensure that edge of flange outstands at least 10 mm from toe of stiffener weld (fatigue constraint).
 - If intermediate bracing is connected to an intermediate stiffener, the stiffener should be welded to both top and bottom flanges.
 - Fabricators may wish to extend intermediate stiffeners to a flange to simplify assembly. In such cases consideration must be given to durability and fatigue.
 - Dimension F is the designed fillet weld leg length.
 - Dimension W is the designed width of stiffener.
 - Dimension T is the designed thickness of stiffener.



REMARKS

The positioning of stiffeners should take account of the need to attach cross-bracing and supporting services. There are benefits for aesthetics, maintenance and durability if stiffeners are not placed on the outside face of the bridge.

Longitudinal (horizontal) stiffeners

Horizontal or longitudinal stiffeners are rarely required other than for long-span bridges. For deep girders, they are sometimes necessary to reduce the web panel height to prevent buckling.

The detailer needs to give consideration to the intersection of the vertical and horizontal stiffeners. Where the design permits, horizontal stiffeners are best detailed in discontinuous lengths with unattached ends (see Figure 4.9). However, the design may require horizontal stiffeners to be continuous, being achieved by:

- welding to vertical stiffeners
- fitting and welding to vertical stiffeners, or
- passing through slots in vertical stiffeners.

For further information refer to Section 4.7.

4.3.3 Bearing stiffeners

Bearing stiffeners are required at supports and where applied loads or supporting reactions are concentrated at particular locations along the length of a girder. Adequate connection to the main girder web allows bearing stiffeners to be designed to act as struts compositely with a portion of the web. Connection to the flanges allows a bearing stiffener to transmit load through the flange to the point of application of a load above, or to a bearing below. The bearing connection to the flanges may either be through a weld fully designed for the purpose (often a butt weld if the loads are high) or, preferably, by preparing the ends of the stiffener to fit the surface of the flanges (with provision of a fatigue-resistant fillet weld).

Bearing stiffeners should use plates symmetrical on both sides of the main girder web. Unsymmetrical bearing stiffeners are contrary to the requirements of Standards and may only be used where the design has accounted for the eccentric stresses that result.

Bearing stiffeners can use a single leg (see Detail 4.3.3-1) or, if temperature and other movements and/or high loading dictates, utilise multiple legs. Details 4.3.3-2 and 4.3.3-3 show typical twin leg stiffeners for use at either intermediate support points or at end supports respectively. Reference should also be made to SBG Guidance Note 2.04 ⁽⁶⁾.

Bearing stiffeners should be detailed so as to be in a vertical plane under dead load conditions.

Detail 4.3.3-1 Bearing stiffener – intermediate support, single leg



- Dimensions D1 and T, stiffener outstand and thickness, to be sufficient to carry axial load and spread it sufficiently to the bearing.
 - Dimension D10, setback of stiffener where it joins flange, typically 20 mm to 25 mm, to be sufficient to ensure that edge of flange outstands at least 10 mm from toe of stiffener weld.
 - Dimension F is the designed fillet weld leg length.



(c) not greater than 25 × thickness of web to comply with BS 5400 : Part 3 ⁽³¹⁾, clause 9.14.2.2, requirements for pairs of bearing stiffeners acting together.

REMARKS

Detail 4.3.3-3 Bearing stiffener – end support



- REMARKS
- Dimension A, overrun of bottom flange. Typically 20–25 mm to provide clearance for adequate flange to web weld return (the extension of the weld continuously around the corner).
 - Dimension B, location or centres of shear studs, sufficient to allow appropriate clearance from edge of steelwork to stiffener for surrounding concrete.
 - Dimensions C × D are defined width and length of bearing plate to suit size of bearing.
 - Dimension E, thickness of bearing plate, to be sufficient to permit tapering as well as retain adequate depth for any tapped holes for bearing fixings, and for capability to spread the load.
 - Dimension G to be the size of fillet weld to secure bearing plate.
 - Dimensions W and T, stiffener outstand and thickness, to be sufficient to carry axial load and spread it sufficiently to the bearings.
 - The detail shown is suitable for use with a concrete end trimmer. The channel is not always needed but some form of bracing can be expected to be required for the construction condition. Effective bracing can also be provided without the use of concrete.

4.3.4 Cope holes

When three plates to be welded are brought together at right angles, eg web, flange and stiffener, the third plate needs to be trimmed to avoid the weld connecting the first two plates. There are two preferred solutions for this.

- 1. To shape the stiffener to suit the web/flange weld and to subsequently weld over to seal the corner completely, Detail 4.3.4-1A. This is particularly applicable where automated stiffener welding equipment is used.
- 2. To use a quadrant cope hole, Detail 4.3.4-1B. This should have a radius as large as practicable to allow the sealing welds and corrosion protection to be completed within the cope hole.

In the past, a larger snipe, as shown in Detail 4.3.1-C, has been used. This has the following disadvantages:

- difficulty of welding through the snipe to seal the stiffener at the 45° corner
- difficulty of applying the protective coating into the hole through the corner
- potential fatigue due to high stress concentrations at the corners of the triangular hole

Detail 4.3.4-1 Welds – cope hole



PREFERRED •	•	A small snipe (45° chamfer) which is welded over, Option A, or a properly formed
		quadrant cope hole, Option B, are preferred.

- A large snipe (45° chamfer), Option C, should be avoided because of the difficulty of satisfactorily welding and protecting in the corners.
- REMARKS Dimension C, radius of cope hole, value to be as large as practicable but constant throughout project.
 - Typical recommended value for C, 50 mm, or a minimum of 40 mm.

SPLICES

4.4

Bolted splices are often required to connect steelwork sections together on site. Splices are normally positioned near the point of contraflexure. Bolts may need to be arranged to minimise section loss resulting from the holes in tension splices.

On a typical I-beam there are two flange splices and a web splice. Each splice requires the plates of the two lengths of beam to be connected together using splice plates. Where the thickness of flanges or web of the two lengths are different, pack plates are required on the thinner of the two plates to make their surfaces flush at the splice.

Web splice plates should extend over as much of the web depth as practicable. Very long plates can be split into two or three sections for ease of handling but this adds to the difficulty of sealing. In positioning the bottom and top rows of bolts in a vertical splice plate, care must be taken that they do not clash with the flange bolts to the extent that a tightening tool cannot be fitted on either the web or the flange bolts. (Refer to Hayward and Weare⁽⁵⁾ for definitive advice on torque wrench clearances.) Account may be taken of the principle that bolts in flanges will normally be inserted with the head underneath and that, in webs of outer girders, bolt heads will be on the outside. Tightening tools (torque wrenches) will normally require clearance for tightening the nut, which will therefore be on top or inside.

Generally a single outer splice plate and two inner splice plates are used on flanges. The single plate could be thinner than the two inner plates so that its cross-sectional area matches them, but it is preferable for the inner and outer plates to be of the same thickness for simplicity of fabrication and of thicknesses available from general stock (see Section 4.1.1). However, the form of the plates will sometimes be varied to suit the requirements for construction, for example to provide an unobstructed path for launching rollers. Where the cross-sections of inner and outer plates are not detailed to match, the designer should take account of any moment developed due to the eccentricity between the centroid of the splice plates and the centroid of the flange.

The gap between spliced girders typically needs to be 5-10 mm, with the larger clearances used for larger girders.

Splice plates should normally be rectangular and of constant thickness. Except where stress and maximum bolt-spacing limitations dictate, there is generally no advantage in tapering the end edges of splice plates. (This is unlike the ends of flange doubler plates – see Section 4.1.5 – which provide increased cross-sectional areas over finite lengths of girder).

Nuts and washers and/or bolt heads on top flange splice plates restrict the space available for shear connectors. A general recommendation is that studs, at reduced numbers per row but still complying with allowable maximum spacing and probably of shorter length, should be fixed to the flange splice plate to provide some continuity of shear connection across the splice location (see Detail 4.4.0-1).

Flange splice plates should be set back from the edge of the parent flange by 5 mm or 10 mm. This allows for tolerances and can also improve its appearance. Where permanent formwork is seated on the flange, the flange splice plates will need to be further reduced in width to give space for seating the formwork to avoid special formwork details being needed.



Plan on bottom flange



REMARKS

- Dimensions D1, D2, D3 and D4 indicate varying flange thicknesses. Splice positions will generally be chosen to be appropriate for changes of flange section.
- Dimension n @ D7 indicates a designed number of uniform bolt spaces. Value of D7 to satisfy minimum and maximum bolt-spacing limits.
- Dimensions D8, edge distance from centre of bolt. Value to satisfy requirements for minimum distance between edge of hole and edge of plate.
- Values of D7 and D8 are recommended to be slightly larger than nominal minimum value to allow for tolerances on position.
- Dimensions D5, distance between centre-line of bolt and centre of flange. Value to be adequate to allow clearance for installation and tightening of bolts.
- Dimensions W, T and L to be designed width, thickness and length of particular splice plates.
- Dimensions D9, location of web splice plate, value to be appropriate for clearance for installation and tightening of bolts.
- Refer to Hayward and Weare ⁽⁵⁾ for advice on torque wrench tightening clearances.

4.5 BRACING

4.5.1 Requirements for bracing

Bracing is required to perform one or more of the functions listed below.

Design requirements

- to provide torsional restraint to the girders at supports
- to transfer horizontal loads to positions of lateral restraint
- to restrain flanges where they are in compression
- to ensure, where appropriate, adequate load distribution between girders when in service.

Temporary condition requirements

- to ensure the stability of newly landed girders until they are satisfactorily interconnected with the rest of the structure
- to provide stability to the main girder system until the deck is in place and at full strength, ie during the period of concreting and curing of the of the deck slab in composite construction
- to share wind loading between individual girders or groups of girders until all elements are connected by the finished deck
- to assist and control the lateral alignment of the girders, for example, to ensure the spacing between girders is sufficiently accurate for the placing of prefabricated transverse members (eg permanent formwork (see Detail 4.2.3-1))
- to assist maintenance of correct pre-cambering and/or pre-deflection of the girders during placing of the deck
- to be braced in pairs to facilitate erection.

Any bracing installed specifically, and only, for benefits during the temporary condition could be removed after the bridge structure is complete. However, to remove the bracing requires workers to go back beneath the bridge deck. In many cases, the risk of accident would be increased during the removal operation. A risk assessment must be carried out to balance the disadvantages against the advantages of removing the bracing, which are:

- recovery of steel for reuse or scrap value (though the scrap value will be very low)
- elimination of future maintenance of bracing members
- improvement of aesthetics by reducing the clutter on the underside of the bridge
- cost of bracing being reduced as it would be reduced in size and it does not need to be corrosion-protected.

While, in some situations, the removal of bracing also removes potential obstacles to accessing permanent structural elements, consideration should be given to the value of the bracing, if appropriately detailed, to assist the safety of inspection and maintenance access.

Bracing for the construction condition that is left in permanently will be subject to fluctuating stresses, which may cause fatigue problems, and is the most common reason for it being removed. If bracing is to be removed, HSFG bolts need not be used for the temporary fixing, although it may be economic to use a connection system consistent with the main works.

Reference should also be made to DMRB BA 53/94 ⁽³⁵⁾ and SBG Guidance Note 1.03 ⁽⁶⁾.

Types of bracing

Figure 4.4 shows, for comparison, possible bracing types that are successfully used in bridges following normal good practice. Some suggested limitations on the type of bracing in relation to bridge girder depth are shown. However, the principal constraint on the effectiveness of triangulated bracing is the height-to-width ratio of the resulting frame, which should preferably not be shallower than 1 in 5.

Figure 4.5 shows two types of frame bracing at the ends of a span. This bracing is often used in conjunction with a concrete trimmer beam or diaphragm and made composite with it using shear connectors.

The use of open steel section, eg angle and/or channel, for bracing members is preferred because of the simplicity of their connection, lapping and bolting (see Detail 4.5.2-1).

For detailing of cross-bracing see Section 4.5.2. The usage of other types of bracing is outlined in Section 4.5.3.







4.5.2 Cross-bracing

Cross-bracing (Figures 4.2 and 4.4 and Detail 4.5.2-1) is commonly and successfully used. This bracing may be required for a combination of the reasons listed in Section 4.5.1. The alternatives of the horizontal members at either top or bottom are shown in the illustrations.

In the most common range of bridge sizes, cross-bracing (properly triangulated) will generally be used for pairs of girders to provide stability during all phases of construction. It should be noted that:

- unless the bracing members and their connections have particular bending capacity, at least one horizontal member (to complete the triangulation) is always needed
- unless horizontal bracing members are provided both top and bottom, diagonal bracing members must be designed in compression where the disturbing forces are reversible.

Theoretically, placing the horizontal member at the top provides a bracing system better able to stabilise the compression flange under the action of sagging moments at mid-span (and placing it at the bottom is better for regions of hogging moment near supports). However, for practical reasons the horizontal will normally be placed either at the top or the bottom throughout.

Within spans, girders are normally braced in pairs to restrain them against buckling. Where there is an odd number of girders, link bracing to top and/or bottom flanges is provided to tie in the unbraced girder. Full bracing across the bridge width is generally avoided as this can attract unwanted transverse distribution loads. At piers and abutments, bracing (cross and double link, or link at bottom) is usually provided across the full width so as to efficiently transmit lateral loads to the bearing which provides lateral restraint.

Wind loading during construction can be shared between all girders by the addition of simple lacing members (horizontals) connected between the cross-braced pairs (see Figure 4.2). The designer, in consultation with those responsible for the construction sequence, will specify (horizontal) link bracing if required during construction to resist wind or other disturbing forces. The value of link bracing can also be important as girder spacers where pre-prepared deck units or permanent formwork span between the edges of flanges of the main girders and have limited bearing (landing) width.

The presence of horizontal members close to the soffit of the deck has the disadvantage of restricting slab construction and is more difficult to remove. Where not needed at the top, the preference is for horizontals to be at the bottom of the X (see Figure 4.4).

The designer will decide the required location and sizes of bracing members to suit the particular requirements of permanent and temporary states of the bridge. The usage of the bracing, ie either temporary or permanent, should be indicated on the drawings. Any requirement to remove temporary bracing should also be specified.

Detail 4.5.2-1 Cross-bracing – elevation



Elevation of typical cross-bracing (with horizontals at top)

- Dimension D, between intersection of member centroids, value used for design.
- Dimension C, value to allow sufficient clearance to avoid interference with concrete deck slab construction.
- Dimensions A, B and M to be designed identifying dimensions and weight per metre length of cross-bracing members.
- Dimensions E and t, stiffener outstand and thickness, to be sufficient to accommodate cross-bracing bolts and holes with sufficient edge clearance.



- Bracing to be positioned so that lines of centroids of bracing elements meet at common point with main element centroid line and as close as practicable to intersection of flange with web. Designer needs to check allowance for eccentricity and compare with tolerances in BS 5400 : Part 6 ⁽³⁶⁾.
 - Angle bracing members should, preferably, have their horizontal leg at the top to reduce accumulation of material on the ledge. However, where bracing members are necessary close to the underside of a slab there are maintenance advantages in keeping the horizontal leg low, ie away from the slab.
 - Dimension D1, value to suit minimum edge and end dimension limits.
 - Dimension nD2 (where n = no of bolt spaces), value to suit minimum and maximum fastener pitch. Note: critical dimensions where bolts are staggered (as in this detail) are measured directly between closest bolts at an angle. Orthogonal dimensions for the detail are to be derived from this.
 - Dimension D3, value to suit minimum space for bolt head and for washer, making allowance to clear root radius of angle.
 - Dimension D7, clearance between end of bracing member and web. Value to suit avoidance of stiffener to web weld plus tolerance to include for possible oversize weld and large enough to allow maintenance of the protective coating (minimum 15 mm).
 - Dimension D8, nominal clearance between ends of bracing members. When they are on the same side of the stiffener sufficient clearance is required to allow maintenance of protective coating. Value recommended to be a minimum of 15 mm.
 - Dimension D10, typically 20 mm to 25 mm, to be sufficient to ensure that edge of flange outstands at least 10 mm from toe of stiffener weld.
 - Where edge, end and spacing dimensions are involved the use of a value 5 mm to 10 mm greater than the absolute minimum value is recommended where possible.

Detail 4.5.2-3 Cross-bracing – intersection



- PREFERRED A single securing bolt is preferred as being the simplest positive solution.
- REMARKS The use of a minimum of two bolts, and of an oversize packing plate, are held in some quarters of the industry as being the norm. Smallness of bracing members sometimes renders the two-bolt solution impractical.
 - The disadvantages of a single bolt compared with a two-bolt solution are considered inconsequential.
 - Should the members be required to provide compression resistance (see 4.5.2) their connection at the intersection can be taken into account by the designer in reducing the member's effective length.
 - Often cross-bracing is not joined at all at their intersection. The packing plate and securing bolt can be omitted where bracing is not subject to loads requiring a connection, ie other than those arising from their function as bracing. However, this leaves a poor detail for maintenance and is not recommended.
 - Dimension D4 value to suit:
 - (a) minimum edge and end dimension limits
 - (b) space for bolt head, making allowance to clear root radius.
 - Thickness D5 value to match thickness of web stiffener plates to which cross-bracing is attached

4.5.3 Other types of bracing

Lateral (channel) bracing

Generally used on smaller, compact bridge structures, the lateral channel bracing (see Figure 4.4) may be used with universal beams or, perhaps, small plate girders to provide stability during the erection of pairs of girders. Extra bracing would be expected at the abutments to provide torsional restraint and the illustration (Figure 4.5) shows this with shear connections ready for composite action with a transverse concrete downstand edge stiffening.

K-bracing

The same general principles as for cross-bracing (see Section 4.5.2) apply to K-bracing except that two horizontal members are needed unless the back of the K (bottom member in Figure 4.4) has sufficient bending capacity.

For ease of construction, the connecting plate at the intersection of the diagonals will be welded to the horizontal before erection.

Z-bracing

The same general principles as for cross-bracing apply to Z-bracing (see Figure 4.5 for an example at an abutment trimmer), but the arrangement needs to ensure that both flanges of each girder are given restraint.

4.5.4 Skew

Apart from the normal implications of rearranging details to enable elements of a bridge to fit a skew there is a particular phenomenon with end deflections which needs to be taken into account in the detailing of braced plate girder skew bridges.

The deflection during construction of heavily skewed spans causes a significant twist about the longitudinal axes of the girders at the end supports ⁽³⁷⁾. The weight of wet concrete causes vertical deflection of a girder and this is normally allowed for by precambering it. At the end supports of skewed bridges there is a corresponding rotation of each girder about its transverse axis (passing through its centre-line of bearing). This end rotation is normal to the plane of the end diaphragm and bracings near the end, which, because they are weak in torsion but stiff in their planes, will force a lateral rotation of the adjacent girders about their longitudinal axis. The rotation will always occur unless forcibly restrained. With stress also arising in the members, this may result in the girders being out of vertical by an unacceptable amount, dependent on the stiffness and depth of the girders, if the skew significantly exceeds 20°. The girders must then be twisted out-of-vertical before concreting to allow for this.

However, the transverse bracing between girders at the end supports usually has to be fitted before concreting (for stability and strength under lateral loads) and, therefore, before this rotation takes place. Where the bracing is a triangulated system, that bracing can only be rotated about an axis in its plane, ie parallel to the abutment. The fabrication details should ideally be dimensioned so that the bracing is unstressed in the final dead loaded state. The bracing members so dimensioned to twist the girders initially can therefore be expected to be temporarily under stress as they are fitted between girders before concreting. This will require measures to distort the elements to fit them and this should be anticipated to avoid site modifications being made to rectify an apparent misfit. As the weight of concrete is added the ends will rotate and the girders will become vertical.

The same argument applies when un-triangulated bracing is used (eg the lateral and abutment trimmer bracing, and the central bay of the abutment trimmer shown in Figure 4.5). However, being more flexible in the plane of the abutment than if it were triangulated, it will accept misalignment mostly with less distress. The estimated amount of twist expected should be included in the design information.

Attention should be drawn to this effect of skew in the contract documents for construction so that the pre-set can be calculated carefully. An I-section has a low torsional stiffness, so the pre-set twist is usually not too difficult to achieve.

Bridges with a skew exceeding 30° need specialist advice. Particular attention must be given to:

- ensuring bearings are appropriate for the control and direction of movements expected
- reinforcing deck concrete to resist the unusual stresses arising
- tying down acute-angled corners to resist uplift.

PLATE GIRDER CROSS-HEADS

The use of plate girder cross-heads integral with the construction depth of the main longitudinal girders allows the number of support columns to be reduced. Where necessary it can, with an increased number of longitudinal girders, allow a reduction of the superstructure construction depth. This avoids the penalty of increasing the number of supports or introducing unsightly cross-heads across the tops of the columns.

The connection details of an integral cross-head to the main girders need careful attention, particularly the influence of longitudinal and transverse falls. There are various ways of dealing with the falls. Figures 4.6 and 4.7 show the industry's typical solutions.

With both these solutions, the main girders run through the support points with their flanges horizontal in the transverse direction. The adjacent girder is at a higher or lower level depending upon the direction of the crossfall.

In Figure 4.6 all splices and splice plates are kept parallel with the main girder flanges. The cross-head girder bottom flanges are extended level from the bearings of the adjacent lower main girder flange. Detail 4.6.0-1 shows this solution.

In Figure 4.7 the diaphragm element is fabricated with flanges that run straight between the splices at the edges of the main girder. This necessitates a kink at the splice point and the splice plates need to be bent to suit. The arrangement keeps the diaphragm girder elements simple, it reduces the number of bends in the flange plates and the resulting kinks are also the least angle compared with other arrangements. However, bearing plates will be tapered in two directions when there is an overall fall in the structure.

In Figure 4.6, stiffeners are shown at the flange kink to resist the vertical load that occurs due to the vertical change in direction of the flange. Such a force also occurs at the kinks in the arrangement shown in Figure 4.7, although the force is less because the angle of kink is reduced. However, because the kink occurs at a splice, a stiffener cannot be provided. The force has to be resisted by the flange cantilevering from the web. This is usually possible because the vertical load occurs within the splice where the stress in the flange is reduced where the cover plates are carrying the load.

Other options are that the top flanges of the main girders could be inclined and the flanges of the diaphragm follow through straight. The actual method chosen finally could depend on the preferences of the fabricator.

To deal with longitudinal falls it is usual to use the diaphragms vertical, with flanges angled to follow the longitudinal fall of the main girders. Using flanges at right angles together with tapered packer plates is possible but unusual. The usual method avoids machining of tapered packer plates and simplifies the site connection and is preferred, even though it is rather difficult to fabricate.

4.6









Half cross-section showing elevation of cross-head (diagrammatic)



4P743







REMARKS

4P743

 Dimensions A, B, C, D and E to specify correct location of stiffeners measured relative to centrelines of main girders.

- Notation ST1, ST2, SW1, SW2, SB1 and SB2 identify different types of splice detail to be designed and drawn to suit. For typical examples, see 4.6.0-2 and 4.4.0-1 (Section 4.4).
- Dimensions W × T to be designed width and thickness of flange and web starter plates.
- Dimensions T1 and T2 to be designed thickness of web plates.



REMARKS

• For general splice guidance refer to Section 4.4

- Dimensions shown are typical and for illustration only. Actual dimensions to be chosen to suit project design. Bolts are omitted for clarity.
- The welded cruciform joint resulting from the connection between the starter plates and the main girder web needs consideration of the risk of pulling the through-plate apart. The development of this detail is a specialist activity related to the specification of the steel and welding procedure. For more information refer to SBG Guidance Note 3.02⁽⁶⁾.

Detail 4.6.0-3

Typical stiffener/bottom flange connections at bends in flange



REMARKS

- Stiffeners or gussets are required at all significant changes of flange direction.
- Dimension F is the designed fillet weld leg length.
- Dimension R, radius of bend in flange plate. Value to be sufficiently large to ensure that the plate properties are not affected. Value to suit fabricator, but typically 10 times thickness of flange plate or 150 mm minimum.
4.7 VARIABLE-DEPTH GIRDERS

Variable-depth (haunched) girders are an economic option in large continuous bridges, due to high moment and shear capacity requirements at the piers^{*}. Haunched profiles provide reduced construction depth over the central portion of spans, where maximum clearances are usually required. The curved soffits of variable-depth girders also present a more pleasing appearance for larger-span bridges.

A part-elevation/section of a typical steel bridge with variable-depth girders is shown in Figure 4.8. As with parallel-flanged girder bridges, the stiffeners at piers are provided on both sides of the web, but intermediate stiffeners are necessary only on one side. For aesthetic reasons these are located on the inner face, at the same time improving durability by presenting the uncluttered surface to weathering. A typical layout of intermediate web stiffeners is shown in Detail 4.3.2-2.

While the analysis of girders with non-parallel flanges can be more complex, in most cases variable-depth girder details are the same as those used for parallel-flange girders. The details included in Section 4.3 (Stiffeners) and Section 4.5 (Bracing) are applicable.

It is usually possible to obtain steel plate of sufficient width for the webs of variabledepth girders direct from the rolling mills. The web plates need to be cut accurately to the required profile before attachment of the bottom flange. The curvature of the bottom flange is achieved by drawing (forcing) the flange plate to follow the profile of the web plate closely during the fabrication process. It is only necessary to pre-bend flange plates at abrupt changes of angle, for example adjacent to the bearings. Bends in a bottom flange plate at a pier with typical stiffeners are shown in Detail 4.7.0-2. Such changes of angle can also be achieved by full-strength butt welding of straight pieces of plate.

Theoretically, increasing the effective depth of girders enables the flange plate dimensions and web thicknesses to remain constant. In practice, however, designers often increase the thickness of flange and web plates as the load effects concentrate towards the piers, thus keeping the plate thicknesses generally in proportion to the overall section. The thicker webs and flanges are easily introduced at the normal splice points required (see Detail 4.4.0-1).

Buckling of the web plate is dependent upon the depth and thickness of the plate, and the length of the web panels between stiffeners. Web buckling can be avoided by one or more of the following:

- thickening the web plate
- reducing the spacing of the intermediate transverse (vertical) stiffeners
- introducing longitudinal (horizontal) stiffeners.

It is generally more economical to keep the web panels between stiffeners approximately square.

* The term "haunched" is sometimes reserved for girders with an abrupt change in the slope of the soffit.







Half-elevation of steelwork and girder make-up

Detail 4.7.0-1 Steel girder make-up (continued)

- - REMARKS
 Dimension A, centres of intermediate stiffeners. Value to be designed to suit resistance to web buckling and positions of bracing. Locations to be clearly indicated in relation to primary setting-out points (centres of bearings).
 - Dimension B, longitudinal centres of shear connectors. Value to suit design for number of connectors per unit length, and detailing constraints (see Table 4.1).
 - Dimension C, giving location of changes in flange or web plate thicknesses, to be clearly indicated in relation to overall dimensions of girder.
 - Dimensions D and H, minimum and maximum overall depths of girder. Value to suit design and to define extent of variable depth.
 - Dimension E, end of girder. Value to be sufficient for end support bearings, bearing stiffeners and any end bracing required.
 - Number of shear connectors across flange, N, to be compatible with designed number of connectors per unit length, and detailing constraints (see Detail 4.2.1-1).
 - Dimension R, radius of girder soffit, is a method of defining the shape of the variable depth required in relation to the minimum and maximum dimensions of depth.
 - Dimension S, giving location of splices, to be chosen to suit transportation size and weight and any changes in plate thicknesses.
 - Dimensions W and T, designed widths and thicknesses of plates, respectively, in flanges and webs.





Elevation on J-J

REMARKS

- Stiffeners or gussets are required at all changes of flange direction.
- Dimension R, radius of bend in flange plate. Value to be large enough to ensure that the plate properties are not affected. Value to suit fabricator, but typically ten times thickness of flange plate, or 150 mm minimum.
- Further information on plate bending is available in SBG Guidance Note 5.02 ⁽⁶⁾.
- Dimension A, spacing of stiffeners and start of radius to be sufficient to allow abutting cross-head plates or bearing plates (where applicable) meet a flat level plate over their full width. Bends need to be square to line of girder, so additional allowance needs to be made for skew of abutting plates where applicable.
- Dimension F is the designed fillet weld leg length.
- Dimension W, width of stiffener. Value to allow setback from edge of flange, typically 20 mm to 25 mm, to be sufficient to ensure that flange outstands at least 10 mm from the toe of stiffener weld.
- Dimension T, thickness of stiffener, value to satisfy minimum design and detailing requirements relative to W.
- For cross-head to main girder connection, see Detail 4.6.0-2.

If longitudinal stiffeners are used the following detailing issues need to be considered.

- 1. If they are used to improve the flow of stress towards the bearing position they need to be continuous, and will act as part of the girder cross section. Continuity is normally achieved by slotting the intermediate stiffeners and allowing longitudinal stiffeners to pass through. Alternatively, the continuity can be achieved by close fitting and welding on either side of intermediate stiffeners.
- 2. If they are designed solely for web stiffening, it is important that they do not transmit longitudinal forces. In this case they should be made discontinuous at intermediate stiffeners. A gap can be left at both ends, see Figure 4.9, or they can be welded to the face of the intermediate stiffener at one end (to ensure satisfactory location) with a paintable gap left at the other end.

Longitudinal stiffeners can collect debris and increase maintenance requirements. Shedding of moisture and debris can be improved by:

- sloping the stiffener away from the web (see Figure 4.10)
- providing a slot for drainage at the lower end of inclined stiffeners.

If longitudinal stiffeners are being considered the benefits need to be evaluated against the increases in fabrication costs and aesthetic and maintenance shortcomings.



Figure 4.9 Discontinuous longitudinal stiffeners



Figure 4.10

Sloped longitudinal stiffeners

WEATHERING STEEL

In certain environments "weathering steel" can be used successfully for bridge construction without any protective coatings. This steel relies upon the natural formation of a protective impervious patina during the first few years of weathering. In more severe environments, weathering steel still provides greater resistance to corrosion than do regular grades of steel, but it is unable to form an even protective patina. This leads to the developing colour being uneven, which may be unacceptable in certain situations. Weathering steel should therefore not be used without protective coatings in the following situations:

- in marine environments
- where subject to de-icing spray, ie chlorides
- in continuously wet or damp conditions
- when buried
- in corrosive industrial environments.

In general, the detailing requirements for structures in weathering steel are the same as for the regular grades. However, careful must be taken to avoid creating any areas on which water will collect. Also, a certain amount of rust stain runoff needs to be anticipated as the steel "weathers" and oxides of iron are formed. To preserve a uniform appearance of the steel, the bridge should be detailed to maintain an even exposure as far as possible. Detail 4.8.0-1 illustrates a method of maintaining the even exposure of a composite plate girder bridge. The cantilever is extended sufficiently to avoid rain regularly being blown onto the bottom half of the girders, which would occur if the cantilevers were much shorter.

Methods of avoiding concentrated runoffs are not so simple to identify. Prevailing conditions are likely to be different for each bridge. Detail 4.8.0-1 goes some way to achieving what is required by reducing the amount of runoff itself.

Where rainwater does reach the girders, water will tend to collect on the bottom flanges in sheltered corners, which may lead to localised corrosion of the bottom flange. The problem can be exacerbated by the slight upward distortion of the flanges that may occur during welding, thus concentrating the collection of water alongside the root. It is very important that stiffeners are not allowed to impede the free runoff of water from the flange. Details, such as good-sized drainage slots, should be provided to ensure that water can drain away freely. To avoid these problems, girders should be arranged to have a camber and/or fall so that water is less likely to collect or pool.

The collecting water will follow the slope and generally shed itself at the end, either at an abutment or pier. Suitably positioned drip plates or runoff strips (Detail 4.8.0-2) can be successful in keeping the water clear of concrete or other stainable surfaces.

Coating of localised areas of a weathering steel structure, or even the abutments and piers, can be carried out if it is difficult to ensure that the local environment will remain satisfactory throughout the life of the bridge.

4.8

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As indicated above, some corrosion is inevitable in the formation of the patina and prediction of the extent of further corrosion is difficult. To allow for this corrosion, additional thickness needs to be provided on all exposed faces. The designer should adopt a (standard) plate thickness and use a reduced thickness for strength calculations to suit the allowance. The DMRB allowances ⁽³⁸⁾ are:

- faces in mild environments: 1 mm per face
- faces in severe environments: 2 mm per face.

It should be noted, however, that localised corrosive effects, such as from leakage of chloride contaminated water through an expansion joint, can cause serious deterioration. Care must be taken in such details.

Reference should also be made to SBG Guidance Note 1.07⁽⁶⁾.

Detail 4.8.0-1 Weathering steel – edge cantilever



AVOID • Road drainage must not penetrate the slab (ie simple discharge pipes must not be used).

• Make W, width of cantilever, > D, depth of girder. However, cantilevers exceeding 1.5 m are more difficult to build.



REMARKS

On unpainted weathering steel bridges, runoff strips can be used on exterior flange legs

- (a) collection of rainwater at bearing stiffeners and subsequent corrosion
- (b) staining of abutments by oxides during weathering process.
- Dimension A, distance from support. Value to be sufficient that rainwater runoff falls clear of surfaces (such as concrete abutments) that could suffer staining. Allowance for wind-borne action should be made.
- Dimensions D and T, cross-section of runoff strip to be selected to suit size of girder, anticipated runoff, and one dimension to match plate thickness elsewhere on bridge.
- S is specification for grade of runoff strip steel to match that of main girder steel.
- Dimension N for no welds required only where BS 5400 ⁽²⁴⁾ class G welding detail is not satisfactory for fatigue, otherwise weld all round for better durability. Value of N to extend typically 25 mm from edge of flange (fatigue constraint).
- For weld detailing nomenclature, see Detail 4.1.3-1.

of outer beams to avoid:

4.9 SERVICES

Many bridges are required to carry services in addition to the primary transportation loads. Provision needs to be made for services at an early stage in the design process. It is usually necessary to decide upon the particular arrangement to suit the type and size of service to be carried.

It is the general and recommended practice for services on steel bridges to be carried in ducts cast into non-structural concrete forming the raised verges and footways at the edges, or the central reserves, of the bridge. The size and number of services to be carried and/or the configuration of the bridge structure may dictate the arrangement. Where space is available, the opportunity should be taken to include some spare service ducts. Reference should be made to Section 5.8 of this book.

Where services are too large to fit within the non-structural concrete, consideration must be given to alternative locations. The space between the girders of steel/concrete composite bridges is usually available. Suspension brackets using simple proprietary hangers from the underside of the concrete deck for the support of the ducts or pipework are recommended. Maintenance of services beneath bridge decks needs greater planning and preparation. Elaborate permanent arrangements should be avoided if the services can be reached by modern mobile access equipment.

Utility authorities often seek large service troughs cast into bridge structures, accessible from the top of the bridge deck. However, such troughs may adversely affect the transverse structural stiffness characteristics of the bridge deck.

Reference should also be made to Section 3.1.7 on verges and troughs.

Fixings for bridge furniture

GENERAL

5

5.1

This chapter deals with fixing to the bridge structure of various components, many non-structural. These components are known as bridge furniture and include:

- parapets
- safety fences
- lighting columns
- signs
- noise barriers
- services
- movement joints.

The bridge designer has several concerns when seeking to secure these components to the structure, which are, principally:

- that the bending moments and shear forces from the component are imparted to the structure in the manner expected
- that the fixing is not itself damaged by the failure of the component
- that the fixings on the deck do not interrupt the continuous waterproof membrane protecting it.

The proprietary nature of bridge furniture means that there may be several competing systems available for use with different fixing arrangements (the choice is generally made by the contractor and agreed by the design engineer). This may require the designer to show a designated outline on the drawing where the fixing will be accommodated. Details will have to be finalised when the selection is made.

Further information on fixings may be found in CIRIA Technical Note 137⁽³⁹⁾.

The details to be found in this chapter are as follows:

5.2.0-1	Anchorage – cast-in cradle system	5.3
5.2.0-2	Anchorage - individual socket and bolt	5.4
5.2.0-3	Anchorage – resin-grouted stud	
5.2.0-4	Anchorage – expanding socket bolt	
5.3.0-1	Bedding of base plate	5.7
5.3.0-2	Levelling of parapet beam for large furniture	
5.4.0-1	Fixings – bolts, nuts and washers	
5.4.0-2	Base plate fixings	5.10
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5.5.3-3	Precast concrete parapet – seating	5.15
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5.6.0-2	Safety fence fixing - over waterproofing	5.18
5.7.0-1	Lighting column position - at edge of bridge	5.19
5.7.0-2	Lighting column positions – in median	5.20
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5.7.0-4	Lighting column and draw pit – at deck edge	5.22
5.8.0-1	Services siting – location	5.24
5.8.0-2	Services ducting at movement joint - range 0 mm to 10 mm total	5.25
5.8.0-3	Services ducting at movement joint - range 10 mm to 20 mm total	5.26
5.8.0-4	Services ducting at movement joint - range 20 mm to 40 mm total	5.27
5.8.0-5	Services ducting at movement joint - range 40 mm to 50 mm total	5.28
5.8.0-6	Services – movement joint	5.29

5.2 ANCHORAGES

The anchorage is that part of the fixing embedded within the structure. Anchorages can be formed in several ways. The most satisfactory are:

- cast-in cradle system (comprising four interconnected sockets cast into the concrete)
- individual sockets and bolts (cast-in or resin-grouted sockets as required).

Other methods that may be required in special circumstances are:

- resin-grouted stud anchorages
- expanding socket bolts (not recommended for new construction)
- through bolting (exceptionally, provided that it is cement-grouted).

It is important to check what is acceptable to the client for the use intended. HAPAS (Roads and Bridges), the Highways Authorities Product Approval Scheme administered by the British Board of Agrément, lists approved anchorage types for bridge parapets. The approvals are reviewed from time to time with the benefit of continuing experience and feedback. Details 5.2.0-1 to 5.2.0-3 show typical examples of each type.

The choice of anchorage will depend upon the circumstances including number, location, what is being fixed and where, and whether it is new works or maintenance. Anchorages for certain uses are tightly specified, such as with parapets, but in other situations the designer has more freedom of choice. The following notes and the remarks connected with each detail should assist in making that choice.

In new works, anchorages on decks should be located in individual or continuous plinths raised above verge level and with a tuck in which to seal the waterproofing.

End anchorages for parapets should be at least 150 mm clear of end of parapet beam or expansion joints therein. The details should ensure that anchorages for parapet posts are well clear, longitudinally, of anchorages for lighting columns.

Anchorages should be independent of the furniture. The type of fixing where the furniture post is recessed into pre-formed holes in the concrete, once a popular fixing, should not be used. Experience has shown that it is impossible to keep them sealed and to maintain protection to the hidden portion.

For further information regarding the use of anchorages, see Section 5.4.



Typical proprietary cradle anchorage

- REMARKS See also Detail 5.4.0-2 in Section 5.4.
 - Cradle offers good mechanical connection to reinforcement.
 - Cradle fixing is more stable than individual cast-in sockets during concrete pour.
 - Easy to place and align.
 - Easy to remove and replace bolts after damage to the component (eg after accidental impact on a parapet.
 - An early decision is needed because of different bolt spacings for different components and cradle must be placed in position at the optimum time.
 - Upstand size is affected.
 - Needs careful detailing of surrounding reinforcement.
 - Used for fixing steel and aluminium bridge parapets, lighting columns, safety fences etc.

Detail 5.2.0-2 Anchorage – individual socket and bolt



REMARKS

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- Choice of cast-in or resin-grouted fixing (for remarks on resin grout see Detail 5.2.0-3.
- Choice of preformed or drilled hole.
- Sockets can be set at different levels if no projection is wanted (but care is needed with threaded connection).
- Easy to remove and replace bolts after damage to the component (eg after accidental impact on a parapet).
- May be used to replace damaged stud fixings if required.
- Cast-in socket alignments are difficult to match at one location. A template should be used to hold them in position during casting. (Preformed holes may offer some flexibility.)

Detail 5.2.0-3



REMARKS

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- Easy addition to existing structure.
- If being used in new works reinforcement needs to be fixed to suit.
- Possible loss of reinforcement section when drilling.
- Smooth face to hole if coring used. Rotary percussive drilling preferred but coring necessary if reinforcement hit. Roughen edges of hole.
- Needs care with alignment to achieve accuracy.
- Materials come under COSHH regulations.
- Exposed projecting thread needs to be corrosion-resistant or protected.
- Projecting thread is easily damaged. Replacement difficult. Generally needs overcoring

 more damage to reinforcement. Replacement with resin-grouted socket to avoid future
 problems is recommended.
- It is critical to get adequate length of resin-bonded anchorage.
- Should not be used for new construction.

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Detail 5.2.0-4 Anchorage – expanding socket bolt



REMARKS

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- Should not be used for new construction.
- Easy installation suitable for non-critical locations such as hand rail or lamp bracket.
- Ease of installation is especially useful on horizontal or upwards fixings.
- Not suitable to resist impacts.
- Concrete cover is critical to resist expansion forces.
- Can loosen under vibration.
- Critical to get adequate length of embedment.
- Prone to corrosion.
- Inferior fatigue and dynamic performance.

BEDDING OF BASE PLATE

The bedding is the layer of mortar between the base plate and the concrete surface. There is unlikely ever to be a smooth fit between these manufactured and formed or floated surfaces. The bedding is necessary to fill the gap and has three main purposes:

- ensuring an even spread of applied load to the concrete
- protecting the section of fixing bolts and/or studs between the underside of the base plate and the concrete
- making up to level any slope of the structure surface to allow for components fabricated square to their base plate.

An added advantage is that it provides a small additional height to the base plate of the furniture to keep it clear of surface moisture.

Detail 5.3.0-1 Bedding of base plate

5.3



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Detail 5.3.0-2 Levelling of parapet beam for large furniture



Elevation

Section A-A



REMARKS • Bedding to be sufficiently thick that underside of base plate is above adjacent surface of parapet beam.

BOLTS, NUTS AND STUDS FOR FIXINGS

Generally nuts, bolts, studs and washers for fixing bridge furniture will be of stainless steel. In any event, they, and any connecting sockets, will be of the same or similar metal to avoid galvanic action.

Nuts, bolts, studs and washers must be protected from touching any part of the base plate they are fixing if there is any danger of galvanic action occurring (see Detail 5.4.0-1).

Lock nuts may be necessary if vibration or flexing of the component is likely to be a problem. This can be particularly so with lighting columns.

Anti-theft nuts should also be considered (one per post fixing) especially in conjunction with aluminium parapets.

The minimum length of threaded connection needs to be determined. The actual location of a socket or fixing may necessitate a change in length of bolt during construction.

Fixings should be given a final tightening after the bedding has cured. Specifying a torque force may be useful, but inaccuracies may result from the overcoming of frictional resistance of existing old fixing sockets.

Any slots in the base plate that remain exposed after the bolt/nut and washer are in place should be sealed to protect the fixing and anchorage area.

Before being tightened, nuts and bolts should be lubricated with high-creep-resistant, anti-seize grease.

Detail 5.4.0-1 Fixings – bolts, nuts and washers



• The fixing system may be insulated from the base plate of the bridge furniture to avoid galvanic corrosion by inclusion of an isolating top-hat washer.

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5.4

Detail 5.4.0-2 Base plate fixings



Stainless steel anchorage unit for parapets

- REMARKS The principle of the cradle system (Detail 5.2.0-1) is used to secure lighting columns, safety fences, acoustic barriers, signs and parapets.
 - Bridge furniture elements can be of aluminium or galvanised steel, so the problem of galvanic corrosion should be considered at an early stage.
 - The designer must ensure that clear details are given on the drawings showing how dissimilar metals are to be isolated to prevent corrosion from the effects of electrolytic action.
 - It is preferable to specify stainless-steel holding-down sockets and bolts.
 - Site staff must be made aware of the minimum length of threaded connection necessary to provide the requisite fixing.
 - All bolts in the system must be checked to ensure they are in place and correctly tightened.
 - Refer also to notes dealing with fixing anchorages in general, Section 5.2.

PARAPETS

5.5.1 Preamble

5.5

A parapet is a restraint system generally located on the edge of the bridge to protect pedestrians and errant vehicles.

At the time of carrying out the design of a bridge deck, it is likely that most of the factors related to the chosen parapet system will have been established, eg the class of containment required, height and structural form. The designers should be aware of two documents that will assist in understanding how the factors were developed.

- 1. British Standard BS 6779, Highway parapets for bridges and other structures ⁽⁴⁰⁾.
- 2. DMRB Standard BD 52/93, The Design of Highway Bridge Parapets ⁽⁴¹⁾.

In the UK, parapets are known by their DMRB group designation, summarised as Table 5.1.

Group	Application	Typical location	Level of containment
P1	Vehicle	Motorway	Normal
P2	Vehicle/pedestrian	All-purpose road	Normal
P4	Pedestrian etc	Footpath and bridleway	Low
Р5	Vehicle	Railway overbridge	Normal
P6	Vehicle/pedestrian	High-risk	High

Table 5.1 Summary of parapet group designations

Three structural forms are currently used on UK motorways and trunk roads: galvanised mild steel, aluminium and concrete. Section 5.4 discusses other forms of parapet.

The two publications mentioned above identify how each type behaves at impact and how different structural forms can be used on the same structure.

Preserving the integrity of the parapet system is a major issue for the bridge owner or maintenance authority, so a method covering the future inspection, maintenance, repair or replacement of any damaged components that make up the system must be determined during the design stage.

Provision for thermal movement of the bridge parapet needs to be considered at an early stage of the design and detailing.

All parapet systems, except pedestrian parapet (P4), require special end treatment to protect the road user against impact or impalement. This will normally take the form of splayed ends to unprotected metal rails and safety fencing on approach and egress.

5.5.2 Metal parapets

Metal parapets systems normally used in the UK are constructed from either galvanised mild steel or aluminium.

Systems using either material are supplied in component form for assembly in their final position on the deck. Each should, preferably, be secured to the deck using the cast-in cradle anchorage system and bedding (see Sections 5.2 and 5.3).

Because of the many fastenings involved in a metallic system, the matter of galvanic corrosion, caused by the electrolytic action of dissimilar metals touching, must be considered early in the design and detailing stage.

Other concerns that the designer and detailer must consider with metallic parapets are listed below.

Galvanised mild steel

- One of the most serious problems associated with galvanised steel parapet systems is the entrapment of water inside a RHS (rectangular hollow section). This may result from the drain holes or galvanising breather holes at the base of the section having become clogged. Apart from corrosion, the winter period freeze/thaw cycle of the trapped water can cause swelling or distortion of the RHS resulting in possible splitting and loss of strength. This matter should be raised when the order is placed with a manufacturer.
- The paint protection system needs a particular specification appropriate to new galvanised surfaces.

Aluminium

- The lightness of the metal and the ease with which it can be cut makes theft of aluminium sections or components a real danger. One way of combating the theft is to camouflage the aluminium with paint. Fewer coats are needed than for steel because it is not for protection
- the long-term effects of oxidation of the metal can affect the mode of failure, particularly if hidden from the effects of regular washing by rain
- there is a limited number of aluminium fabrication specialists and this may affect the competitiveness of the material in local areas.

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Concrete parapets

5.5.3

Concrete parapets are mostly used where a high degree of vehicular containment is required, ie road over railway or road over valleys, and are therefore likely to be a P5 or P6 designation. They can be formed *in situ* as part of the bridge deck or can be precast and attached to the deck by bolting or stitching. The principles applicable to parapet beams (see Section 3.1.8) also apply.

Concrete parapets need to be able to provide the required vehicle containment without doubling as a main structural member for the bridge (BD 52/93⁽⁴¹⁾). To avoid the cross-section of *in situ* concrete parapets attracting longitudinal stress and affecting the stress distribution through the main structure, vertical joints through the parapet are often introduced, usually at centres not exceeding 4.5 m. These joints are normally designed to transmit shear between panels, thereby providing continuous shear resistance along the whole length of the parapet. Where joints are not provided, the bridge design needs to take account of the parapet's interaction with the main bridge cross-section, including the effects of differential shrinkage.

Precast concrete parapet units can be heavy and unwieldy items and will require a method of lifting and manoeuvring to be determined during the design stage. There will also be a need to provide a stable temporary support during the final adjustments to the vertical and horizontal alignments. Some typical fixing details are illustrated in CIRIA Report 155⁽³⁾, Chapter 5, Figure 5.3. If a precast anchored system is the chosen form then care must be taken to provide accessibility to the permanent fixings for inspection and maintenance.

Detail 5.5.3-1 Concrete parapet – in situ



REMARKS

- A parapet or safety fencing is required on the approaches to the ends of a concrete parapet.
- Where metal parapets are used on the approach, the type of parapet must be confirmed during the design and detailing stage, or provision must be made to accommodate the various parapet options.

Detail 5.5.3-2 Concrete parapet – precast



- Using an *in situ* concrete stitch (forming a lapped link between continuity reinforcement) avoids the problems of corrosion and maintenance of other forms of connection.
 - Where adjustable anchor bolts are not provided the levelling and alignment of the precast units, relying solely upon bedding to level, needs greater attention.
 - Lifting eyes need to be designed and detailed to suit the erection method.
 - Lifting eyes in the precast units should be retained and protected against damage and corrosion for future use when replacement of a parapet unit is needed.



Detail 5.5.3-4 Precast concrete parapet – temporary stabilisation



- PREFERRED Stability provided by anchor studs into the deck slab (typical detail shown) is preferred.
- Precast concrete parapet units are unstable, or close to unstable, when placed. Means must be provided to ensure they are stable during construction.
 - Stability can be provided by counterweighting, anchoring to the deck slab or use of horizontal/inclined ties.
 - Dimensions D, H S, L and P, diameters of stud and of hole, length of stud and of hole and projection of stud. Values to be chosen to be compatible with designed temporary forces, thickness of deck and centres of anchorage.
 - Dimensions of plastic sleeve, washer and pocket for nut are typical and may need to be changed to suit actual project requirements.

5.5.4 Other forms of parapet

There are situations where a non-standard form of parapet may prove acceptable. However, this has to be agreed with the Technical Approval Authority. Such a situation might be in an environmentally sensitive area and/or in a location where resistance to impact is not critical. The following forms of construction could then be pursued further:

- masonry or brickwork
- reinforced brickwork
- brick-faced reinforced concrete wall
- timber
- tubular hand railing.

Where vehicle access is possible it is likely that a safety fence, barrier or kerb would also be installed to give the necessary degree of protection to both pedestrians and parapet.

The list of non-standard parapet options mentioned above is not meant to be exhaustive; others may be devised to suit particular situations. Their fixing to the bridge deck can best be established at the time of design once the parapet form is known.

SAFETY FENCES

5.6

Safety fences on bridge works are normally found in three locations:

- in the central reserve separating the carriageways
- leading on to and away from the deck parapet system to avoid end-on impact and aid redirection of errant vehicles
- in a position to deflect vehicles away from piers and abutments.

The designer should refer to DMRB documents TD 19/85, *Safety Fences and Barriers*⁽⁴²⁾, and TD 32/93, *Wire Rope Safety Fences*⁽⁴³⁾, for details of the sections used for the fences and posts.

Detail 5.6.0-1 Safety fence fixing – on bridge deck plinth







PREFERRED .

Cradle anchorages for new works.

 Plinth as base for fixing to avoid holes in waterproofing. This also assists in providing adequate depth for anchorages if in a thin slab.

REMARKS • Mortar bedding optional assuming stainless-steel bolt used.

- Size of plinth may be affected by anchorage type.
- Continuous plinth allows easier adjustment to location of fixings.

Detail 5.6.0-2 Safety fence fixing – over waterproofing





- REMARKS
- Anchorage cradle to be positioned so that the top of socket is at top of surfacing.
- Waterproofing membrane to be dressed carefully around the sockets.

_

LIGHTING COLUMNS

Every effort should be made at the design stage to avoid the need to have lighting columns on a bridge. If this is not possible then they must be located behind a protective barrier. This may require the safety fence post spacing to be locally reduced.

Detail 5.7.0-1 Lighting column position – at edge of bridge



Lighting column behind metal parapet on corbel

REMARKS

5.7

- Protective barrier may be a safety fence, vertical concrete barrier (VCB) or parapet.
 - Whichever system is used there must be the full design clearance between the barrier and the lighting column to allow for the anticipated deflection under impact.
 - With the lighting column on the edge of the structure behind the parapet this will normally require the parapet beams to be corbelled, locally, to receive the lighting column, see Detail 5.7.0-4.
 - Any corbelling required should be of a size that assists access for maintenance.
 - Avoid having the metal parapet post and the lighting column in the same transverse position.
 - Ensure the lighting column is aligned such that its maintenance door can be opened. and in a direction appropriate for maintenance access.
 - Ensure the lighting column maintenance door has a safety chain attached to stop it falling and creating a hazard below.



- REMARKS •
- Where column is above a VCB or on a high foundation above the main impact zone, ensure it is set back sufficiently to avoid residual impact (from vehicle superstructures).

Detail 5.7.0-3 Lighting column and draw pit – in median



Drawpit at lighting column on bridge deck upstand



Section A-A

REMARKS

- A wide slab is shown. Where a discrete plinth is used, do not skimp on the plan size of plinth the bigger the better.
 - Duct should project above top of slab or plinth to avoid getting grout into it.
 - Designer should draw large-scale details to demonstrate path of duct through reinforcement and cover to reinforcement.
 - Duct bend must be a radius bend. Check that the radius is suitable for the cable.

Detail 5.7.0-4 Lighting column and draw pit – at deck edge



Detail of lighting column corbel

- REMARKS
 - Dimensions A and B give overall size of draw pit cover.
 - Dimensions C and D give internal size of draw pit/junction box.
 - Avoid positioning parapet post next to lighting column. See Detail 5.7.0-1 for other positioning requirements.
 - Corbel and deck cantilever need to be designed and detailed to resist ultimate failure load of column to ensure structure is not damaged if a large vehicle strikes the column.
 - Waterproofing to be dressed around duct where it enters the plinth.

5.8 SERVICES

It is important for the designer to identify at an early stage those services, if any, that will need to be carried by the bridge and how they are to be accommodated. These can be:

- in the verge
- in a service bay/trough
- cast into the deck or in a void in the deck
- attached externally to the bridge generally between the beams in a composite deck.

Services carried on bridge decks must always be accessible for inspection or maintenance. However, statutory undertakers and other service providers invariably request their apparatus be hidden from view as a precaution against vandalism.

Where more than one service is to be carried in a service trough or duct, then positions relative to each other must be discussed and agreed by the providers concerned, the biggest concern being the close proximity of a gas or water pipe and electric cable. Apart from these, most other services can sit safely in the same trench, although working space around each service is usually mandatory.

Where services cross a movement joint the designer needs to consider the necessary accommodation of a similar movement within the service itself or its duct. Alternatively, the service could pass beneath the expansion joint off the deck where the movement can be accommodated. Provision for road network communication cables sited in verges or medians should be anticipated by the designer on all bridge decks carrying major roads.

In conclusion:

- attempts should be made to persuade the utility authorities to install their services below the ground. The presence of services in the deck seriously interferes with future maintenance of waterproofing and increases costs
- services should, where required to cross the bridge, preferably be located in the deck or externally (generally only acceptable between beams).

Gas mains should not be cast in. Water mains should only be cast in if the pipe is carried in a continuous duct throughout the bridge. Therefore:

- neither gas nor water should be placed in voided concrete slabs
- if pressure mains are located in a trough or verge and need a bolted fixing this should only be allowed if plinths can be formed at each fixing into which the waterproofing can be dressed.

Utilities have a right to be in the highway and a bridge is part of the highway. There are five main problem areas arising from services in bridges:

- they inhibit/prevent access for maintenance and inspection
- they give problems at movement joints
- ducts carry water and any sumps should be located at the uphill end of the bridge deck to avoid the need for water to drain through the ducts on the bridge
- service bays, being often granular-filled and unprotected, fill up with water unless well drained
- damage is caused when services are worked upon.

The designer needs to cover all these issues.

Detail 5.8.0-1 Services siting – location





Services sited in service trough

REMARKS • Refer to Sections 3.1.7 and 4.9 for various treatments at verge.

Detail 5.8.0-2

Services ducting at movement joint – range 0 mm to 10 mm total


Detail 5.8.0-3 Services ducting at movement joint – range 10 mm to 20 mm total



Typical edge detail at expansion joint



• This detail is appropriate to small-diameter services, eg telecom and electric. See also notes on Details 3.1.9-2 to 3.1.9-4.

• This detail is limited to a movement of 20 mm total.

Detail 5.8.0-4 Services ducting at movement joint – range 20 mm to 40 mm total



Detail 5.8.0-5 Services ducting at movement joint – range 40 mm to 50 mm total



Ducts beneath footway



Ducts in verge





- PREFERRED Expansion unit for service to be off the bridge superstructure and behind the abutment in a purpose-made chamber.
- It is preferable to avoid incorporation of major services within bridge superstructure.
- REMARKS Both bridge owner and utility owner will have input into choice of detail. The utility owner's interest is mainly in the movement and the bridge owner's in the effective sealing.
 - Where services are carried on the superstructure, care must be taken to establish which services can co-exist in the same access chambers.
 - Sleeves and holes are to be of sufficient size such that they do not obstruct the passage of the pipe flange.

Support structures

6.1 GENERAL

6

This chapter deals with elements that support the bridge superstructure and is divided into:

- end supports
- intermediate supports
- bearing plinths and downstands
- access to bearing shelves
- drainage of bearing shelves.

Most of the details that follow are considered to be applicable to bridgeworks in both concrete and structural steelwork. Readers should refer to the relevant sections for the individual bridge types. Appropriate cross-references are made to other parts of the guide.

The details to be found in this chapter are as follows:

6.2.2-1	Abutment – bank seat	6.3
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6.2 END SUPPORTS

6.2.1 Preamble

Structural details for end supports need to provide for interaction between soil and structure. The strength of the design can be based upon the strength of the surrounding soil with the structure designed to move with it (flexible or sliding design). Otherwise the structure needs to be designed to resist all the forces, including those from the retained soil (rigid design). Many aspects of end supports are similar to those of retaining walls and this section should be read in conjunction with Chapter 7.

Important aspects of detailing for the various structural forms of end support (commonly called "abutment" on conventional bridges) are noted with the details that follow. Abutment galleries for access are dealt with in Section 6.5.

Further discussion of end supports where they are used for integral bridges is to be found in Chapter 8.

"Run-on", or "approach", slabs are sometimes considered necessary where settlement of the backfilling behind the abutment wall is anticipated or is a potential risk. Details of run on slabs may be found in Section 8.5.

6.2.2 Abutments

The details available for this section show a bank seat, a cantilever (full-height) wall, a skeletal and an embedded wall abutment. Embedded walls can be of diaphragm walling (such as shown in Detail 6.2.2-5) and also of contiguous or secant piling for lighter bridging. Steel sheet piling or strengthened or reinforced soil may also used in abutment construction, but these are most effective in conjunction with a small bank seat.

Some details in Chapter 7 show capping beams for steel sheet piling walls. The suitability of sheet piling for the particular environment needs careful consideration before they are adopted. Reference should also be made to the Steel Construction Institute publications.

Basic types of reinforced soil wall are illustrated in Section 7.5. The bank seats that they would support should be limited to those that incorporate bearings. The constant movement of the bank seats of integral bridges can cause problems for reinforced soil.

The detailing of all end supports needs to take account of tolerance and seasonal variations in the bridge superstructure length.



- Front and back faces of abutment should be kept parallel to ease reinforcement detailing, and reduce formwork and construction costs.
 - The dimensions at the top show a typical way of achieving the required gallery size shown in Detail 6.5.0-2.

Detail 6.2.2-2 Abutment – cantilever wall



REMARKS

- On smaller abutments the general preference is for the back wall of the abutment behind the gallery to follow straight down to the top of the foundation slab (see alternative outline shown).
 - Where abutment gallery projects out from back of abutment wall below, placing of back filling and drainage layers can be improved by providing a splay beneath the abutment gallery projection (see alternative outline shown).
 - Depending upon the bridge and embankment construction sequence, the abutment gallery can be constructed using the fill and blinding as the soffit shutter.
 - Water from bearing shelf to be piped to primary drainage system.

Detail 6.2.2-3 Abutment – skeletal/spill-through



- The purpose of this type of abutment is to reduce the horizontal longitudinal forces from the soil on the abutment or to provide an economical transfer to the foundation level of large anchorage forces.
 - The pier cross-section (for rigid design) is often rectangular with a sloping front face to provide increased width of pier at the base necessary for appropriate increased design strength and stability. The pier cross-section can also be circular (usually for flexible design).
 - Where top and bottom are connected by a sloping face, the slope is best kept steeper than 1:6 to reduce formwork uplift problems during casting.





REMARKS • Incorporate set-back dimensions (minimum 100 mm) to avoid clashing of reinforcement.



- A T-section diaphragm wall is shown. The detailing principles apply equally to plain diaphragm wall, contiguous piled or secant piled abutments.
 - The capping or abutment beam should only be cast after compaction of backfill is completed.
 - Exposed surfaces can be faced with sprayed concrete or screened with a separate system.



- Selected facing could be built off a beam at ground level. Facing would need to be protected by safety fence in verge (see example in Detail 7.3.0-1A).
 - Reference should also be made to Sections 7.2 and 7.3, particularly with respect to water collection and discharge.

6.2.3 Wing walls and slopes

Where abutments are not full-height vertical walls the embankment or cutting will slope up towards the bearing shelf. It is not possible for vegetation to grow satisfactorily on slopes that do not have the benefit of sun and rainfall. The surfaces will need to be paved (see Detail 6.2.3-1).

Walls extending alongside the abutment (wing walls) will retain the soil supporting the approach road and form the transition between structure and natural ground. Settlement of these slopes is common and it is important that the walls penetrate sufficiently deeply below nominal finished ground level to avoid their being undermined (see Detail 6.2.3-2).

Detail 6.2.3-1 Paving to slopes beneath superstructure



REMARKS

For dimensions of maintenance platform, see Detail 6.5.0-1.

- Where solid slabs are used the sub-base should be a lean-mix concrete.
- The edges of both the maintenance ledge and the embankment slabs should be finished with a side edging. Such edging can take the form of precast concrete blocks with a channel formed in the top face to guide discharged abutment shelf drainage towards and into the main highway drainage system.
- On slopes adjacent to the bridge the designer may wish to promote the use of a slab type that can be soiled and seeded, thus allowing grass to grow. In this case the subbase should be complementary to this course and allow adequate drainage to occur.



• Adjacent to the end of the wing wall, earthworks tend to settle and get rounded off. The wing wall should therefore extend at least 1 or 2 metres beyond the theoretical top of the slope, as shown, to ensure end of wing wall is well into the ground on approach.

Detail 6.2.3-3 Abutment – wing wall internal angles



- Fillets are incorporated to prevent design reinforcement being bent through an angle of greater than 90°.
 - A fillet allows compaction against the vertical face.

INTERMEDIATE SUPPORTS

6.3

Intermediate support is the term applied to all piers positioned between the end supports (abutments) that define the spans. They comprise one or more vertical load-carrying members. Such members may take the form of columns (plan dimension ratio less than 4:1) or walls (plan dimension ratio greater than 4:1) with plan shapes either circular, rectangular or other faceted design to create a pleasing aesthetic effect. Construction can be in steel or concrete or a combination of both.

Support can be provided to the superstructure either:

- through proprietary bearings, or
- by connecting support and superstructure monolithically.

Bearings permit the thermal and flexural movements to take place with a designed degree of restraint. If the connection is monolithic the support must be designed and detailed in a way that suits the articulation of the bridge integrating the rigidity or flexibility of the other supports. They then need to be sufficiently strong to support the superstructure but be sufficiently flexible to permit movements.

If bearings are to be used at intermediate support positions then a method of replacing them must be established and recorded in the structure maintenance manual or health and safety file (see Sections 2.2.2 and 2.5.3). When replacement is required the recorded method is available to the maintenance contractor. If the contractor wishes to devise a method of their own, they will need to supply proof to the bridge owner that the structure will not be detrimentally affected. Areas adjacent to the actual bearing on top of the column or pier can be designated as jacking areas and would need to be suitably reinforced to allow this to occur. Other ways of providing jacking areas are to incorporate corbels on the sides of the columns or piers or, alternatively, for the jacks to be supported on temporary trestling erected on existing foundations.

Cross-heads, connecting across the tops of the supports, are needed where the positions of the superstructure elements cannot match those of the supports. Cross-heads can be supported on bearings (an integral cross-head) or can, be part of the support itself and carry the bearings. Built in cross-heads (without bearings) are also very efficient – for example, see Detail 8.2.3-3 in Section 8.2.

6.4 BEARING PLINTHS AND DOWNSTANDS

Bridge design can involve large forces, both vertical and horizontal, being transferred between superstructure and substructure. Structural bearings provide the necessary load transfer while permitting the designed movements unless the superstructure is "built in" to the substructure.

The decision to incorporate bearings into the bridge articulation must, however, take account of the fact that bearings generally have a shorter life than the structure and that at some stage they may need replacing.

As discussed in Section 6.3, a method by which the superstructure can be supported during bearing resetting or replacement must be established so that intended areas of support during jacking can be adequately reinforced.

To assist in determining the condition of a bearing, noting its defects and then dealing with them, adequate room for inspection and maintenance must be provided all around.

Headroom between soffit and the bearing shelf should be commensurate with installing and replacing the bearing, taking into account bolt lengths and direction of access for maintenance and inspection and for clearing drainage channels. To facilitate access, there must be minimum headroom of 300 mm between the superstructure soffit and the abutment shelf or the top of column. The clearance requirement will vary as the width of shelf varies (see Detail 6.4.0-1). Smaller headroom is sometimes acceptable where front and rear access is available, for example where discrete beams are used.

Other factors that can affect the headroom are the depth of the chosen bearing and the positioning of jacks for future bearing replacement. Bearing plinths and deck downstands are ways of securing this adequate headroom. They also, together, facilitate the mid-height positioning of the bearing, which is beneficial for visual examination.

Downstands are difficult and expensive to form and are rarely necessary solely for seating the upper bearing plate. However, downstands do offer easier inspection of the bearings and reduce the required height of bearing plinth. Where downstands are present, the same design considerations as for the lower plinth will apply. For columns there are also aesthetic reasons that lead to plan dimensions of the soffit downstand preferably mirroring those of the column.

Pockets for dowels or holding-down bolts are, generally, only required in the lower plinth, as top plate fixings are cast in position with the deck or connected to a girder.

The way the bearing is orientated with respect to the plinth and downstand, as well as the dimensions of the top and bottom plates (if present) of the bearing, will influence the plan dimensions of the plinth/downstand.

Detail 6.4.0-1 Bearing plinth at abutment – dimensions



Table A

Dim 'Y'	Dim 'Z'
< 450	300
450 - 750	450
> 750	700

Dimensions designed to allow access for maintenance of bearings and drainage where access is from one side only

- REMARKS Dimension Z is crucial because it is needed to allow access around the whole of the bearing.
 - Where access is available from only one side, Dimension Z is particularly crucial to allow access to the bearing shelf drainage channel.
 - Relative dimensions for Y and Z given in the table allow access for arm, head and shoulders and whole person respectively, such dimensions being determined from the overall width of the bearing shelf.
 - The dimensions need to satisfy requirements for allowing removal and replacement of the bearings with the minimal amount of vertical displacement from jacking and the provision of jacking points.
 - Where access is available from both sides by virtue of a maintenance platform and an abutment gallery, the dimensions need to be considered in relation to the overall access provision and some reductions may be appropriate.
 - The required height can be provided by a supporting plinth, a downstand or a combination of both.
 - Refer also to Section 6.5 for relationship to other aspects of maintenance access.
 - Dimension P, defining size of plinth and downstand, value to suit particular bearings but typically a minimum of 100 mm to 200 mm larger than bearing. See also Detail 6.4.0-3.

Detail 6.4.0-2 Bearing and plinth



Typical detail for in situ concrete or steel beam superstructures

REMARKS

- Pockets could be rectangular on plan or alternatively be one large pocket.
 - Pockets for dowels must be located inside reinforcement.
 - Compressive zone within plinth to be contained by reinforcement.
- Must be sufficient clearance to remove bearing fixing bolts with minimal jacking.

Detail 6.4.0-3 Bearing plinth reinforcement



Sectional plan C-C

- Bursting stresses should be contained by suitable reinforcement.
 - Reinforcement for horizontal loads and dowel actions of bolts and short fixing sockets must be considered.
 - Load distribution at 45 degrees must be contained within the plinth reinforcement.

REMARKS

Detail 6.4.0-4 Bearing plinth at discrete column



- At the design stage, level X should be specified by assuming the suitable bearing with the greatest depth. On site the depth of the deck downstand may be varied to fit the bearing actually used.
 - Bearings that are bolted to the column/deck should be so arranged that the bolts can be removed and the bearing slid out of the gap with minimal jacking.

ACCESS TO BEARING SHELVES

The access requirements at abutments and bearing shelves relate to the need to:

- gain access around the whole of the bearing for ongoing inspection and maintenance
- clear out and maintain the drainage channel and outlet on the bearing shelf
- avoid the difficulties arising from the vulnerability of the faces of the deck end and back wall of the abutment to the corrosive effects of water and water-borne de-icing salts filtering through the expansion joints and service ducts.

Access to the abutment and end support bearings can be achieved in various ways.

- 1. From a platform in front of the abutment (see Detail 6.5.0-1).
- 2. From a gallery located between the end of the deck and the back wall of the abutment (see Detail 6.5.0-2).
- 3. From ground level by means of a ladder or hydraulic platform.
- 4. A combination of 1 and 2, or 2 and 3.

The choice will generally be made on the basis of:

- the form of construction, eg whether the bridge has side spans with sloping revetments and, if so, the length of the spans and steepness of the revetments
- whether the bridge lies on a routine winter gritting route and is therefore likely to suffer the frequent application of de-icing salts with their deleterious effects.

BD 57/95⁽¹⁾ and BA 57/95⁽⁸⁾ express a preference for access galleries. This provision will allow space for inspection and maintenance of bearings, expansion joints, ballast wall and deck ends in this highly vulnerable area. It is also expected to allow for the accommodation and safe operation of any future form of deck jacking.

Smaller bridges with shallow superstructure depth (say less than 1000 mm) could have their costs significantly increased by the inclusion of access galleries, however. An early decision whether or not to include an access gallery is essential.

Important detailing issues include:

- Confined Space Regulations 1997
- emergency escape
- traffic management
- security
- drainage
- ease of access including any relevant safety equipment
- provision of lifting points where it is possible that equipment may need to be raised.

Detail 6.5.0-1 Maintenance platform – dimensions



Section through maintenance platform

- If the bridge has side spans and the revetment can be built to get to the requisite level for a platform, then a platform should be provided.
- The height clearance should not exceed the dimensions given or this will necessitate a special ladder or platform for the inspector to see the bearings.
 - On construction using discrete beams the headroom clearance required is to the deck slab soffit, not the undersides of the beams, provided there is sufficient clearance between the beams.
 - The minimum width of platform of 1.5 m is to minimise the risk of people stepping back and falling down the slope.
 - Access to the maintenance platform should be from steps down from the road above or up from the road below.
 - If access to the platform is up a paved slope without steps then the slope and the paved finish must be suitable for use by maintenance staff.
 - See also Detail 6.2.3-1.



- PREFERRED If the road carried by the bridge is on a regular winter gritting route it is essential that an abutment gallery is provided.
 - If there is no maintenance platform provided in front of the abutment, or if access to the maintenance platform is difficult or disruptive to provide, an abutment gallery should be provided.
- REMARKS
 The primary purpose of the abutment gallery is to allow access for inspection and maintenance to as large an area as possible of the end face of the superstructure and the abutment back wall above the bearing shelf level.
 - The sloping soffits at the top of the gallery are crucial to ensure that the depth of the surfaces that are only apart by the thickness of the expansion gap is an absolute minimum.
 - The minimum depth of the gallery floor below the bearing shelf is to facilitate work on the bearings from the gallery.
 - Where the overall height of bridge superstructure is large, the relationship between gallery overall height and bearing shelf height needs to be considered in relation to the overall access provision.
 - 800 mm is the minimum suitable width for working and should be provided over the whole height of the gallery.
 - For choice of access provision, see Detail 6.5.0-3.
 - For Detail 6.4.0-1, bearing plinth at abutment, see Section 6.4.
 - Refer to Detail 6.5.0-1 for maintenance platform in front of abutment.
 - For drainage refer to Section 6.6.
 - For collection of sub-surface drainage refer to Detail 6.6.0-3

Detail 6.5.0-3 Access positions



- REMARKS When the choice of access is made, due consideration must be given to matters of security.
 - Size of access to be decided following assessment of equipment that may need to be carried in.



Detail 6.5.0-4 Abutment access from front

REMARKS

- In detailing, care must be taken that hinged doors do not foul the various adjacent angled surfaces.
- Due allowance must be made for screens and doors to be compatible with the relative movements between abutment and superstructure.

DRAINAGE OF BEARING SHELVES

A most important aspect of bridge detailing and maintenance is the management of water that inevitably reaches the bearing shelf. Access must be provided to allow blockages to be cleared and effective drainage to be maintained (see Section 6.5). Details 6.6.0-1 and 6.6.0-2 give guidance on the form of the drainage.

Detail 6.6.0-1 Bearing shelf – drainage

6.6



- PREFERRED Both options are appropriate for particular circumstances.
- REMARKS
 Where an access gallery is present, the preferred position of the drainage channel is at the back of the abutment shelf, which prevents salt-contaminated drainage water running over the front face of the shelf.
 - Where an access gallery is not present, cleaning and maintenance is more easily tackled from the front and hence the drainage channel is best positioned at the front. However, where adequate clearance is provided between bearings, putting the drainage channel at the front is optional.



- PREFERRED Semi-circular drain channel, Option A is preferred.
 - Semi-circular shape is normally compatible with circular drainage pipes.
 - The shape former could be either plastic or salt-glazed wear and should also be left in place as protection to the concrete.
- REMARKS Both shapes are easy to form, strip and clean.

Detail 6.6.0-3 Gallery – subsurface drainage collection and piping



• Gutter needs to be detailed to ensure that bearings are protected and the gutter is capable of being cleared and maintained.

Detail 6.6.0-4 Gallery drainage exit route – side exit



Detail 6.6.0-5 Gallery drainage exit route – level bridge soffit



Elevation on abutment





Elevation on abutment





Retaining walls

7

7.1 GENERAL

This chapter deals with the types of retaining wall that can be expected to be used on highway schemes. Some types may not be suitable for use on trunk roads or motorways.

Retaining walls can be broadly divided into the following forms of construction:

- cantilever
- embedded cantilever
- gravity
- reinforced soil.

The details to be found in this chapter are as follows:

7.2.1-1	Cantilever wall – cross-sectional shapes	
7.2.2-1	RC cantilever wall – stem shapes	
7.2.2-2	RC cantilever wall - with typical reinforcement	7.4
7.2.2-3	RC cantilever wall – stem base joint position	
7.2.2-4	RC cantilever wall – stem reinforcement	
7.2.2-5	RC cantilever wall – coping	7.7
7.2.2-6	RC cantilever wall – coping reinforcement	7.7
7.2.2-7	RC cantilever wall stem – movement joint	
7.2.2-8	RC cantilever wall stem – shear key	
7.2.3-1	Weep hole	
7.2.3-2	RC cantilever wall - drainage measures	
7.3.0-1	Embedded cantilever wall – drainage	
7.3.0-2	Steel sheet pile wall – capping	
7.3.0-3	Steel sheet pile wall - capping at revetment	
7.3.0-4	Steel sheet pile wall – revetment capping reinforcement	
7.4.0-1	Gravity wall - treatment at top	
7.4.0-2	Gravity wall - coping provision for parapet	
7.5.0-1	Reinforced soil wall – base	
7.5.0-2	Reinforced soil wall – coping	

Construction of a retaining wall often interferes with the natural drainage of the soil strata. There is always potential for water to build up immediately behind the wall in the interstices of the soil backfill.

Even with an impervious clay behind the wall, rainwater runoff has the potential to saturate the back of the wall to its full height. Consideration needs to be given to the effects of the trapped water, therefore, and an efficient dispersal system is usually incorporated into the construction.

7.2 CANTILEVER WALLS

7.2.1 Preamble

The form of construction covered in this section is where the vertical wall stem cantilevers from, and is monolithic with, a spread foundation. The wall is free-standing and its stability results mainly from the self weight of the structure and any permanent soil loading. "Inverted T-shape" and "L-shape" are popular ways of referring to the cross-section of these walls; see examples in Detail 7.2.2-1.

Usually the material used for construction is reinforced concrete, allowing the wall to be either cast *in situ* or precast.

The wall can have a concrete finish or be clad with another material such as brick or masonry. Concrete finishes can be as-cast plain, patterned, coloured or specially treated; the designer should refer to the Concrete Society report on concrete finishes.

Detail 7.2.1-1 Cantilever wall – cross-sectional shapes



REMARKS:

- The choice of shape of wall will be dependent upon design considerations.
- Option A is more suitable for supporting the sides of an excavation.
- Options B or C are preferred for retaining embankments.
- Provision of a downstand key is not preferred because the excavation for the downstand will expose the surrounding soil to potential deterioration from the effects of weather, which would result in reduced resistance to movement.
- Possible downstand key for Option A is shown positioned directly beneath the wall stem to simplify reinforcement detailing and reduce the risk of undermining the excavated slope during construction.

7.2.2 Reinforced concrete wall stems

The stem of a cantilever wall is a highly visible structure. It should therefore have acceptable appearance as well as provide the primary structural strength and be durable. The chosen shape should minimise construction difficulties.

High walls need their greatest strength at their base where, generally, greater thickness is provided. A stepped thickness wall has not been shown here in favour of the tapered wall stem for high walls.

Detail 7.2.2-1 RC cantilever wall – stem shapes



- REMARKS: Constant wall thickness, Option A, allows easier formwork details and the reinforcement can be fixed and checked more simply.
 - Option A is best suited for walls up to about 5 m in height where an illusion of toppling of vertical faces is not perceived.
 - Sloping-front face, Option B, is better suited for walls more than 5 m high to counter the illusion of toppling if vertical. It also uses material more effectively and economically, with the gradually increasing strength towards the base matching the increasing bending moment and shear force.
 - Sloping formwork for Option B can create geometrical (plane) problems if height varies. It is expensive to construct, particularly if the wall is curved on plan, as this requires formwork panels with a conical face.
 - RC cantilever walls are suitable for high, multi-lift, walls but require care at horizontal construction joints, where feature recesses should be considered.
 - Counterforts or buttresses may be required where the wall height exceeds 10 m.



AVOID

Fixing reinforcement into a concrete pour where it is "cover critical" in the next pour should be avoided.

• In the UK it is usual for vertical and top horizontal surfaces of reinforced concrete below ground level to be given a waterproofing treatment. This treatment, to improve the resistance of minor cracking to the penetration of moisture, conventionally comprises two coats of bitumen paint. Where conditions are particularly aggressive or aesthetic standards are particularly high (ie needing to positively prevent penetration of moisture through the wall) a sheet or sprayed membrane could be required. For the extent of waterproofing treatment, see Detail 7.2.3-2.

Detail 7.2.2-3 RC cantilever wall – stem base joint position



- PREFERRED A kicker, Option A, is preferred but it must be properly supervised on site to ensure that kicker position, and the concrete grade in it, are correct and that concrete compaction is satisfactory.
- AVOID
 Kickerless construction, Option B, can lead to shutter alignment difficulties and grout loss and should be avoided.
- REMARKS: Kicker should be cast integral with base slab.
 - Dimension K, height of kicker. Value to be between 50 mm and 150 mm, depending upon size of wall, shutter details and site preferences.
 - The detailer need only be concerned with the height of the kicker in order to plan the dimensions of the starter bars and reinforcement in the next lift of concrete. It is the choice of the contractor whether to follow these proposals.
 - Care must be taken to ensure that starter bars have the correct cover to any features that may be present and also allow for any enclosing links that may occur in later stage construction.
 - If bars in stem are constrained in length at the top by a concrete surface the lower ends of the bars should be detailed allowing for a tolerance on the constructed level of the kicker.

Detail 7.2.2-4 RC cantilever wall – stem reinforcement



PREFERRED • Placing the vertical bars in the outside layer (Option A) allows concrete to flow more freely at the face of wall during placing.

REMARKS

•

- Placing the horizontal bars in both front-facing layers (Option B) is slightly easier to fix.
- Where shrinkage stresses in the longitudinal direction along the wall are critical, placement of horizontal reinforcing bars on outside layer (Option B) can reduce crack widths in the visible and exposed face.

Detail 7.2.2-5 RC cantilever wall – coping



PREFERRED • Option A is simple to reinforce.

- Both options overcome the problem of the underside of the coping not coinciding with top of construction joint.
- Option B is used only when wall coping dimensions must reflect those of the deck string course or parapet beam.
 - Recommended fall across top of coping is 5 per cent.
 - Dimension A, vertical overlap of coping, is decided by consideration of concrete cover to coping reinforcement, but should be greater than 10 mm to avoid grout loss.

Detail 7.2.2-6 RC cantilever wall – coping reinforcement



Detail 7.2.2-7 RC cantilever wall stem – movement joint



- Refer to CIRIA Report 146, *Design and construction of joints in concrete structures* ⁽¹¹⁾, for further details of joints in walls.
 - Joints in walls should be minimised so far as possible. Spacing of joints should be compatible with designed reinforcement for crack resistance.





PREFERRED .

Option A, concrete shear key, is preferred.

• Care with the alignment of dowel bars, Option B, during concreting of joint is important to avoid "lock-up".

• Any dowel bars should be stainless steel. Normal steel is subject to corrosion and is therefore not acceptable and the serviceable life of galvanised steel dowels is unproven.

7.2.3 Drainage of RC cantilever walls

Walls often interrupt a natural flow of water in the soil and are subject to a build-up of water pressure behind. Small walls may rely upon weep holes (see Detail 7.2.3-1) to relieve the pressure, but primary walls on highway schemes should be provided with a drained permeable layer along the entire rear face (see Detail 7.2.3-2).

Detail 7.2.3-1 Weep hole



- A small, upwards, slope of the weep pipe towards the outside face of the wall (Option A) minimises slow weeping and is easier to rod.
 - Where positive drainage of the wall backfill is provided, the drain pipe should be lower than the weep holes (Option A). Weep holes are then maintained to provide relief of exceptional conditions only and not carry the bulk of the drained water.
- Use of weep holes adjacent to footways and hard verges where drained water from weep holes may become hazardous, particularly in freezing conditions, should be avoided. This particularly applies to Option B.
- Where possible, the back of wall drainage should be serviceable without relying on weep holes.
 - Option B tends to weep at all times even when carrier pipe is not blocked.
 - Weep holes are used, typically, at 3 m centres.
 - In urban situations the provision of covers over the weep pipes to prevent deliberate blockage should be considered.
 - Wall staining can be contained within a recess feature similar to that shown on Detail 6.6.0-7.
 - Consideration should be given to the use of vermin covers.
Detail 7.2.3-2 RC cantilever wall – drainage measures



REMARKS

Reference can be made to DMRB Standard BD 30/87, *Backfilled Retaining Walls and Bridge Abutments* ⁽⁴⁴⁾.

- A 500 mm-wide flat strip of fill alongside the top of the wall at the foot of the slope avoids tendency to overfill and can act for drainage collection.
- Weep holes are not intended to flow continuously. They should be provided only as a wall drainage failure indicator.
- A drainage channel in front of the wall can be provided to collect discharge from the weep holes.
- Large volumes of water should be dealt with separately and piped to a main drainage system.
- Ref S is to refer to the relevant specifications for waterproofing of buried concrete surfaces.
- Perforated pipe may have positive outfall and so could operate with or without weep holes. The perforated pipe should be provided with an independent rodding point, particularly where there is no weep hole.
- The effects of soil compaction adjacent to any drainage measures should be considered.

7.3 EMBEDDED CANTILEVER WALLS

The walls covered in this section are those that are formed in place either by boring, deep trench excavation or driving. DMRB Specification BD 42/94, *Design of Embedded Retaining Walls and Bridge Abutments*⁽⁴⁵⁾, should be used for reference.

Embedded wall construction can therefore take one of the following forms:

- contiguous bored piling
- secant piling
- diaphragm walling
- steel sheet piling.

Their features, typified by a bored contiguous pile wall, are shown in Figure 7.1.



Figure 7.1 Embedded cantilever wall – features

Reference should be made to specialist literature for information on how each construction form is completed. Each of these forms is mentioned in Chapter 6 as being suitable for use as bridge abutments as well as retaining walls.

Restraints at high level, or struts just below finished ground level, can be included in the design of the contiguous, secant and sheet piling to assist their resistance to lateral forces.

In most cases, the wall face revealed after excavation requires some form of finish to be applied. Installation tolerances, potential vehicle impact and surface roughness should be considered when designing the finish. Final finishes should then comply with the project's protective coating specification – see DMRB *Specification for Highway Works* ⁽⁴⁶⁾, Clause 1700. Unless the wall facing is designed for vehicle impact, safety fencing should be provided to protect the wall.

The use of secant (overlapping) piling, diaphragm walling and sheet piling are all effective in limiting the ingress of water into the excavation, but some provision should be made for scepage. Where discrete piles are used, more positive measures are necessary, see Detail 7.3.0-1.

Steel sheet piling is particularly effective in situations where ground disturbance and soil removal must be kept to a minimum and also where watercourses form part of a bridge works scheme. Alternative forms of capping for sheet pile walls are shown in Detail 7.3.0-2. For a revetment supported at the top of a sheet pile wall, an example capping is shown in Details 7.3.0-3 and 7.3.0-4.



- Option A provides protection to the vertical clay soil surface between piles. The perforated pipe is needed to prevent the build-up of water pressure, which otherwise would tend to burst off the shotcrete providing the protection.
 - Option B provides facing in front of the piles in granular soil. The perforated pipe provides drainage of the retained soil.
 - The ability of such wall face drainage to reduce loading on the wall is a subject for specialist design.
 - The geotextile retains fine soil particles from entering and blocking the porous pipe.
 - Ensure the drainage authority permits groundwater to discharge into its system. If not, a more substantial wall without drainage may present a more economical solution.
 - Drainage downpipes should be inspectable and maintainable.

Detail 7.3.0-2 Steel sheet pile wall – capping



RC capping

Steel capping



- PREFERRED Reinforced concrete capping, Option A, is preferred because it allows better load distribution and can cope with poor tolerances on sheet piles.
- REMARKS Steel channel capping, Option B, is more suited for urban use where it can be painted.
 - Timber capping, Option C, is used mainly in a waterway environment.

Detail 7.3.0-3 Steel sheet pile wall – capping at revetment



• Consideration should be given to the inclusion of a granular bedding layer to the revetment blocks above the geotextile.

Detail 7.3.0-4 Steel sheet pile wall – revetment capping reinforcement



REMARKS: • Dimension D, diameter of hole for reinforcement, to be 25 per cent larger than nominal size of reinforcement.

7.4 GRAVITY WALLS

A gravity wall relies on its self-weight to counteract any overturning or sliding effects resulting from pressure from the retained material.

In this category, walls are formed in unreinforced (mass) concrete, brickwork, timber, precast concrete unit crib walling or stone-filled wire gabions.

The Design Manual for Roads and Bridges, Volume 2, Section 1 (substructures)⁽¹⁷⁾ should be referenced in the design of gravity walls.

Mass concrete

CIRIA Report 155, *Bridges – design for improved buildability* ⁽³⁾, seeks to develop unreinforced elements for the construction of abutments and wing walls. Gravity walls, particularly those constructed in mass concrete, fulfil this aim. Gravity walls can only be used where sufficient ground space in the finished state is available. Cost savings result from not having to supply, fix and protect reinforcement embedded in the concrete. Counter to this is the cost of the additional concrete required. The principal drawbacks are the propensity for surface cracking and imperfections of a mass concrete wall, which, together with its sheer bulk, compromise its appearance. The use of fibre reinforcement in the concrete mix to control cracking of concrete and improve durability should be considered as should the use of low-heat cement (see CIRIA Report 135 ⁽⁴⁷⁾).

Examples of shapes of mass concrete retaining walls are shown in Figure 7.2.



Examples of mass concrete retaining walls

Figure 7.2 Gravity walls – concrete shapes

An estimate of the construction capability should be made at the time of the design. The location and number of construction joints in the mass concrete wall will be dictated by;

- the chosen maximum volume of concrete in any one pour
- the sizes of formwork panels
- the need to limit thermal cracking.

Suggested treatment of the foot of an embankment slopes at the top of a gravity wall is shown in Detail 7.4.0-1. Mass concrete gravity walls are not normally reinforced except sometimes at their tops in the support of parapet fixings, etc.

Detail 7.4.0-1 *Gravity wall – treatment at top*



REMARKS

- Guard rails may be required on top of the wall.
- Settlement behind wall could affect the precast drainage channel (Option B), which is therefore more suitable in areas of cut.
- Front face of wall could suffer discoloration if drainage measures are inadequate.

Mass concrete walls can be faced to improve their appearance, a durable facing being masonry, suitably fixed.

A permeable layer is usually provided against the rear face of the walls. The measures will be similar to those in Detail 7.2.3-2. If provided with a positive discharge route, the potential steady water pressure on the rear face of the wall will be lessened. This could allow the size of the wall to be reduced. It also lowers the chances of leakage through cracks and construction joints in the wall, which could deface the front surface or damage any finishes.

Detail 7.4.0-2 Gravity wall – coping provision for parapet



REMARKS

 Design of the precast slab and foundation must include the required design forces on, and from, the parapet.

- Shear and bending stresses must be checked at level of lower construction joint.
- Keys can be incorporated in construction joints to resist shear effects if necessary.
- Precast slab and foundation must be statically stable with required factors of safety, ie they should not rely upon a moment connection with mass concrete wall.

Crib walls

An alternative form of gravity wall is the crib wall, in which interlocking components are built up into a framework. The resulting open framework allows the fill material to be exposed on the feature face and thus be planted or seeded offering landscaping potential. They also allow the retained material to be free-draining, so clearance to the highway needs to be sufficient to allow for the inevitable spillage of soil. Refer to DMRB BD 68/97⁽⁴⁸⁾ and BA 68/97⁽⁴⁹⁾. Crib walls are usually proprietary structures and, as such, the details should be sought from the manufacturer.

Gabions

Another solution to stabilising and strengthening soil slopes, as well as combating problems of soil erosion, is the gabion wall. The gabions, or cages, are formed from wire mesh, wired together in their empty state, then filled with stones of larger gauge than the mesh to create the finished product. The life and durability of the cages must be considered, and the wire and wire mesh will normally have been galvanised or coated with PVC. The suitability of this form of construction is influenced by factors mainly connected with the stone filling. Availability, quantity and transportation costs of the stone should be researched at an early stage in the design process.

Examples of shapes of gabion walls are shown in Figure 7.3.

Some locations are not suitable for gabions, for example, alongside paved footways. Unauthorised persons, particularly children, can easily climb the stepped front faces of gabion walls and can remove the stone material. There are also restrictions on their use alongside motorway or trunk road carriageways. However, gabions have a proven track record over many decades and, where suitable for the purpose, are cost-effective.

Frictional anchor elements can be attached to gabions to give increased stability where required. Gabions can also be built level.



Gravity walls – gabions

Figure 7.3

REINFORCED SOIL WALLS

The reinforced soil system consists, basically, of proprietary reinforcing or anchor elements, made of galvanised steel or polymer, laid in a suitable fill material. Linked to the elements are proprietary modular facing panels (normally of precast concrete). The wall is built by repetition of a simple sequence of operations at successive levels; installing facing panels, placing and compacting fill, laying reinforcing elements and placing and compacting further fill, repeating the sequence until the desired height is reached. The finished wall is able to resist lateral pressure by friction along the reinforcing elements, which ensure the soil mass acts as a block as a gravity wall.

Examples of preferred types of reinforced soil walls, using discrete and full height interlocking panel systems, are illustrated in Figure 7.4.



Modular interlocking panels Full-height interlocking panels

Figure 7.4 Reinforced soil walls – basic types

Walls of variable height and formed to curved profiles, can be easily constructed. A series of walls can be provided where terracing and landscaping is desired. The asstruck surface of the panels of the types of wall shown provides a satisfactory finish.

Details should be compatible with the design guidance of BS 8006⁽⁵⁰⁾. The types of wall shown in Figure 7.4 can comply with this standard. The design of reinforced soil schemes is a specialist activity.

The position of the junction of flexible elements and inflexible/stiff elements (at connection with foundations, see Detail 7.5.0-1, or copings, see Detail 7.5.0-2) must be considered at an early stage in the design.

The design life and durability of the entire structure should be carefully considered, especially when the integrity of a bridge superstructure may depend on the tensile capacity of certain members or parts of the members.

Abutments constructed using reinforced soil techniques may need stricter and more limiting conditions to be specified.



PREFERRED • Non-channelled concrete strip base, Option A, is preferred, being easier to form on site.

REMARKS • Perforated pipes must discharge to an approved drainage system and must be roddable.

Detail 7.5.0-2 Reinforced soil wall – coping



- REMARKS •
- Service ducts should not be cast into copings.
- Design of the coping system with support slab must include the required design forces on, and from, the parapet.
- The coping system with support slab must be statically stable with required factors of safety, ie it should not rely upon a moment connection with top of wall panels.

Integral bridges

8

GENERAL

Integral structures are becoming a preferred form of construction for spans up to 60 m because they eliminate some of the major causes of deterioration that have become evident in the UK bridge stock in recent years. The deterioration is particularly related to the effects of movement joints in the deck, which are prone to leakage. Movement joints and bearings are an additional initial cost to the structure, so reducing or eliminating them can bring significant savings.

For the purposes of this book, an integral bridge is defined as a bridge with "abutments connected directly to the bridge deck, and without movement joints between spans or between end spans and abutments". This requires the superstructure of a multi-span bridge to be fully continuous across the intermediate supports that may or may not be built in to the deck. Methods of forming such continuity are discussed in this chapter. Continuous, jointless construction, but with bearings and a monolithic screen wall at the abutments (often known as "semi-integral"), is also included here.

It is recommended that the designer refer to the following documents from which the above definition is taken:

- The Design Manual for Roads and Bridges (DMRB) Advice Note BA 42/96⁽⁹⁾
- The Design of Integral Bridges ⁽⁹⁾
- Advice Note BA 57/95⁽⁸⁾ and Standard BD 57/95⁽¹⁾, both entitled *Design for Durability*.

Up to now, experience with integral bridge works in the UK has been fairly limited and standard practices are not yet established. This chapter therefore reflects experience gained in other countries combined with current thinking in the UK.

Integral bridges can comprise both steel and concrete elements, so reference should be made to the relevant chapters for other details for the individual bridge types, support structures etc. This chapter deals primarily with details connected with integral bridge construction that have not been covered elsewhere in this guide. Appropriate crossreferences are made to other parts of the book. The details to be found in this chapter are as follows:

8.2.3-1	Precast beam continuity (Type 1) – wide <i>in situ</i> integral cross-head 8.5
8.2.3-2	Precast beam continuity (Type 2) – narrow <i>in situ</i> integral cross-head8.6
8.2.3-3	Precast beam continuity (Type 3) – integral cross-head cast in
	two stages
8.2.3-4	Precast beam continuity (Type 4) – continuous separated slab
8.2.3-5	Precast beam continuity (Type 5) – tied deck slab
8.2.3-6	Diaphragms at piers
8.2.3-7	Joint for tied deck slab continuity (Type 5)
8.4.2-1	Frame abutment
8.4.2-2	Frame abutment, beam and slab – continuity concrete flush with wall 8.18
8.4.2-3	Frame abutment, beam and slab – continuity concrete wider than wall8.19
8.4.2-4	Frame abutment, beam and slab – beam bearing shelf
8.4.4-1	Piled abutment – integral bank seat with run-on slab
8.4.4-2	Piled abutment – integral bank seat
8.4.5-1	End screen – embedded (sheet pile) wall, semi-integral
8.4.6-1	Bank pad abutment – sliding type
8.4.6-2	Slip membrane – chamfer at edge
8.5.2-1	Run-on slab – interface with carriageway
8.5.2-2	Run-on slab – buried type
8.5.2-3	Run-on slab – fill backing to abutment
8.5.3-1	Run-on slab – connection types
8.5.3-2	Run-on slab connection – anchor bars near bottom
8.5.3-3	Run-on slab connection – anchor bars near top

8.2 CONCRETE SUPERSTRUCTURES

8.2.1 Preamble

The concrete superstructure of an integral bridge could take one of a number of forms. If the maximum length of such a structure were limited to, say, 60 m then the form of construction would be chosen from:

- solid slab
- voided slab
- precast pre-tensioned beams
- box girder.

For spans up to 15 m, solid slab construction is the most logical selection for economy and buildability, although precast beams may be appropriate for particular sites. Monolithic integral construction avoids elaborate bearing arrangements. Therefore, the threshold of competitiveness of precast beams should be reached at shorter span lengths compared with jointed construction.

For spans up to 30 m, precast prestressed concrete beams are the preferred form of construction. Voided slabs, either reinforced or prestressed, have been used in the past but, for reasons of buildability, this form of construction is now less used.

For bridges up to 60 m total length, spans in excess of 30 m are not often required except, for example, to avoid constructing an overbridge pier in the median between two carriageways. Precast pre-stressed concrete beams are successfully used for spans up to 40 m. Post-tensioned box girders can be used for spans up to 60 m and beyond.

Reference should be made to Chapter 3, as many of the details shown there are common to the superstructures of concrete integral bridges.

8.2.2 Construction sequence

It is important that bridge details permit the construction sequence of an integral bridge to be compatible with the assumptions made during the design. The drawings should clearly state the design assumptions. The likely sequence of construction will be:

- construct substructure up to deck soffit level
- place precast beams (where used) on supports (often on small, thin, rubber pads)
- cast deck slab
- cast superstructure and deck continuity at intermediate supports when present
- cast *in situ* stitches between deck and abutment
- backfill behind abutment.

This sequence will minimise the amount of movement at the abutment due to shrinkage of the concrete.

For short spans, the whole deck, including the monolithic connection with the abutments, can often be cast at the same time.

The construction drawings must also specify the sequence of backfilling behind the abutments where difference in fill levels or fill stages may affect the design assumptions.

8.2.3 Continuity of deck at intermediate supports

The principles of bridge superstructure continuity over intermediate supports are not unique to integral bridging. Any continuous construction can adopt the same details included in this chapter. They should be used in conjunction with other appropriate details found in Chapter 3.

There are different methods of achieving continuity of decks at intermediate supports when using precast concrete beams. Five solutions that have been used satisfactorily in the UK are shown in Details 8.2.3-1 to 8.2.3-5. The "Type" numbers refer to those used in BA 57/95⁽⁸⁾. Details 8.2.3-1 to 8.2.3-3 show *in situ* integral cross-heads, which may be designed to develop full continuity moments. Details 8.2.3-4 and 8.2.3-5 provide continuity through the deck slabs only (partial continuity). Detail 8.2.3-6 shows further options for the diaphragms at piers.

A form of continuity preferred by many designers and maintenance engineers is the fully built-in pier. The dimensions of the pier, its slenderness, the ability to accommodate movement due to shrinkage, temperature and braking loads, and its appearance, are all important issues. If the span arrangement is symmetrical then the central pier of a two- or four-span bridge can become the "null point" for movement, so consideration should always be given to "building-in" this pier. A built-in pier is efficient for resistance to impact, an additional asset for piers in the vicinity of the carriageway.

The designer must recognise the potential congestion of reinforcement and conflict between bars within these continuity details. It is advisable for reinforcement to be drawn at a large scale so that the relative positions of all the bars can be understood.

The arrangements requiring pairs of permanent bearings, namely Details 8.2.3-2 (optionally) and 8.2.3-4 to 8.2.3-6 have the disadvantage that, when jacking is required to replace bearings, space is not readily available for positioning jacks.

Details 8.2.3-5 to 8.2.3-6 have the disadvantage of a narrow gap between the two diaphragms, which is inaccessible should maintenance of those faces be required.

Details 8.2.3-2 (optionally) and 8.2.3-3 to 8.2.3-6 have the advantage of not needing temporary support during construction.

When two separate diaphragms are used, continuity is provided only in the deck slab at the top. The flexure of the superstructure (rotation at the ends of the beams) then results in relative longitudinal movement at bearing level. The design and detailing must take account of this longitudinal movement at the bearings as well as vertical shear generated in the connecting top slab.

Other disadvantages and features of the various types of precast beam continuity are set out in the notes with each detail.

Detail 8.2.3-1 *Precast beam continuity (Type 1) – wide* in situ *integral cross-head*



- Where full continuity is required, continuity Type 1 is preferred because of the avoidance of difficulties with lapping reinforcement and its ability to accommodate bridge curvature, both horizontal and vertical.
- REMARKS Beams are erected on temporary supports generally off pier foundations.
 - Permanent bearings are in single line.
 - Continuity reinforcement is provided in the slab and at the top and the bottom of bridge beams. The lapping of reinforcement is normally not difficult.
 - Only one set of jacks is required at each pier for raising deck for bearing replacement.
 - Beam embedment length, E, value typically about 1 m.

Detail 8.2.3-2 *Precast beam continuity (Type 2) – narrow* in situ *integral cross-head*



REMARKS

- Temporary supports are not required if the permanent bearing shelf is of sufficient width and continuous.
- Permanent bearings may be in single or twin line.
- Permanent bearings can sometimes be combined with temporary bearings as a wide single permanent elastomeric bearing.
- If outer pairs of bearings remain in place, design and detailing consideration must be given to potential relief of load on bearings when spans are alternately loaded.
- Continuity reinforcement is provided in the slab and at and the bottom of bridge beams. The lapping of reinforcement is difficult.
- Only one set of jacks is required at each pier for raising deck for bearing replacement.
- Beam embedment length, E, value typically about 1 m.

Detail 8.2.3-3 Precast beam continuity (Type 3) – integral cross-head cast in two stages



REMARKS

Beams are supported on first stage of cross-head during erection.

- Resulting cross-head is monolithic with pier.
- Cross-head soffit is normally lower than beam soffit.
- Temporary bearing pads are encased in surrounding concrete of Stage 2 cross-head construction.
- Reinforcement is similar to Type 2 (see Detail 8.2.3-2) depending on the cross-section of the Stage 1 cross-head.
- No bearing maintenance is required.
- Beam embedment length, E, value typically about 1 m.

Detail 8.2.3-4 *Precast beam continuity (Type 4) – continuous separated slab*



REMARKS

Temporary supports are not required.

- Separate bearings and end diaphragms are provided for each span.
- Deck slab is separated from support beams for a short length to provide rotational flexibility and this deck slab length is designed to flex.
- There is no continuity reinforcement between ends of beams and there is no moment continuity between the superstructures of the spans.
- Not recommended where air contamination may cause deterioration to inaccessible faces.
- Arrangements for jacking up the deck to facilitate replacement of bearings need to be considered by the designer and the provisions made described in the maintenance manual.
- Separated deck slab length, C, value typically about 1.5 m.
- Designer needs to be satisfied that bearings and continuity top slab can resist the effects induced by relative horizontal and vertical displacements (see Section 8.2.3).



REMARKS

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Temporary supports are not required.

- Separate bearings and end diaphragms are provided for each span.
- The tie reinforcement at mid-depth of the slab is debonded for a short length either side of the joint to permit deck rotation. There is no moment continuity between the superstructures of the spans.
- Not recommended where air contamination may cause deterioration to inaccessible faces.
- For joints for deck continuity see Detail 8.2.3-7.
- Designer needs to be satisfied that bearings and continuity top slab can resist the effects induced by relative horizontal and vertical displacements (see Section 8.2.3).



PREFERRED .

Option A, although both options are acceptable.

REMARKS

 While the structural dimensions of the diaphragm result from design, the principle of finishing the underside of the *in situ* concrete level with the underside of the beam as shown in Option A has the added advantage of being able to accommodate any arrangement for lateral movement restraint.

- Separate bearings and end diaphragms are provided for each span.
- Option A also allows the slotted holes in the beams to be used for transverse reinforcement.
- Option A does not need soffit formwork if beams are contiguous.
- Option A covers and protects the ends of the prestressing strands.
- Designer needs to be satisfied that bearings and continuity top slab can resist the effects induced by relative horizontal and vertical displacements (see Section 8.2.3).

Detail 8.2.3-7 Joint for tied deck slab continuity (Type 5)



- REMARKS
 Deck slab between spans is separated using compressible joint fillers, but deck waterproofing and deck surfacing are continuous and an extra membrane is provided over the joint for double protection.
 - Length of continuity bars to be at least sufficient to develop full anchorage bond into the adjacent spans from the ends of the debonded length.
 - Thickness of concrete between joint filler and crack inducer, through which continuity bars are centrally placed, is typically 140–180 mm.
 - Consideration may be given to the efficiency of the protective wrapping to be used and, if guaranteed, the substitution of conventional (HY) reinforcing bar instead of galvanised or stainless steel.
 - Waterproofing across the joint subject to slight cracking and movement will require special strengthening (see Detail 3.1.9-1).
 - Diameter of reinforcement, D, value to be established by designer, typically 25 mm.

8.3 STEEL SUPERSTRUCTURES

8.3.1 Preamble

The most appropriate deck type for integral steel bridges is steel/concrete composite construction. This form of construction is successful and economical where spans exceed 30 m. Whether standard rolled steel sections, fabricated plate girders or box girders are adopted, the principles are the same.

Reference should be made to Chapter 4 of this book, as many of the details illustrated there are common to the superstructures of steel/concrete composite integral bridges.

8.3.2 Construction sequence

The designer and detailer should define the construction sequence at the preliminary design stage and, as with concrete construction, all details and reinforcement should be selected with this sequence in mind. The drawings should clearly state the design assumptions and any limiting differences in fill levels.

It is preferable for all single and multi-span integral bridges to be designed and detailed to the following construction sequence:

- construct substructure up to deck soffit level
- place steel beams on temporary supports on piers and abutments
- cast deck slab
- form continuity of superstructure and deck over any intermediate supports
- cast *in situ* stitches between deck and abutment
- backfill behind abutment.

The different ways of making a composite deck continuous are discussed in Section 8.3.3 and the sequence of casting the concrete deck slab may be governed by this.

The details at the abutments may differ from those of concrete integral bridges. The use of an end screen may be more appropriate for steel/concrete composite construction, but a cast *in situ* concrete stitch is also common. The different types of deck/abutment layout are described in Section 8.4.

8.3.3 Continuity of deck at intermediate supports

Methods of achieving continuity of steel/concrete composite decks at intermediate supports usually involve the provision of full-depth continuity, although partial continuity has been used outside the UK. The former makes both the steel beams and concrete deck fully continuous; the latter makes only the deck slab continuous.

The simplicity of partial continuity can be attractive to the designer, particularly with regard to the ease and economy of construction. Its disadvantages are that it requires bearings beneath the end of each beam and a wide pier to support them. The continuous slab is likely to require some maintenance during the 120-year design life.

Although full continuity may incur greater construction costs, the continuous beam requires only one permanent bearing on which to be supported and piers can be slimmer and more flexible. This flexibility can be part of the bridge articulation using pinned bearings. Less maintenance is a feature of this form of continuity and of integral bridge construction philosophy as a whole.

8.4 END SUPPORTS

8.4.1 Preamble

The principal difference between integral bridges and conventional bridges is in the design of the end supports. In a conventional bridge, thermal movement, structural flexure, shrinkage etc are accommodated by a designed and clearly delineated movement joint. In an integral bridge, reliance is placed upon compliance of the soil behind the abutment with imposed movements of the bridge structure. Any required provision for movement in the carriageway is then placed outside the structure length where it will cause less deterioration to the structure.

Figure 8.1 shows three principal methods by which an integral bridge can accommodate movements of the bridge superstructure.

Figures 8.2 and 8.3 show different forms of end support following the recommendations from a study tour to North America by a task group from the Concrete Bridge Development Group⁽⁵¹⁾.

The main types of end support can be further described and categorised as:

- frame
- embedded wall
- pile
- end screen
- bank pad
- reinforced soil.

These are discussed in the following sections of the guide.

8.4.2 Frame abutment

The first illustration in Figure 8.2 shows the normal arrangement with a full-height vertical abutment face. The end support is constructed integrally with the deck and acts as a retaining wall for the approaches to the bridge. A sloping face is sometimes considered, but this is more difficult to build. The movements due to thermal effects and earth pressure are accommodated by flexure of the wall stems and slight movement of the soil backing to the wall.

The frame abutment in its most common form for underbridges and shorter-span structures is a portal frame structure. While apparently the simplest arrangement, the significant bending moments at the corners of the frame can cause difficulties with detailing and congestion of reinforcement. The difficulties increase where precast beams or steel beams are used. A typical example, showing a precast concrete beam superstructure, is shown in Figure 8.4 followed by appropriate details.



Movement by pile flexure - Type B



Rigid abutment and sliding bearings – Type C



Integral bridge abutments – accommodation of deck movements



Figure 8.2 Integral bridge end support types – Sheet 1



Semi-integral construction with bearings on top of a rigid retaining wall is a design method that can be used for full-height abutments for bridges of any length. Jacking of the deck can result in soil movement under the abutment soffit. This can obstruct the deck from returning to its original level

End screen (semi-integral)



Shallow abutments on spread footings are only considered to be suitable for situations where the foundation is very stiff and there can be no settlement problems. A granular fill layer should be placed below the footing to allow sliding.

Bank pad abutment



Piled bank seats are recommended for widespread use. The piles prevent settlement while allowing horizontal movement and rotation.

Piled bank seat



Bank seats can be designed as semi-integral abutments. The footing is not required to move horizontally and piled or spread footings can be used.

Piled bank seat with end screen (semi-integral)

Figure 8.3 Integral bridge end support types – Sheet 2











- Dimension B, length of precast beams, measured in line of beam to suit any skew required.
- Carriageway, footpath and verge dimensions should be given for measurements square to walls.
- Dimension W, thickness of wall, value to satisfy strength and flexibility design of frame.
- Dimension F, thickness of base slab, value to satisfy strength design of foundation.
- Level L, founding level, to satisfy strata depth and carriageway and footway cover requirement.
- Type of precast beam, P, to be chosen to suit design of bridge.
- Base of walls could be formed as pins, but are more complicated to construct.

Detail 8.4.2-2 Frame abutment, beam and slab – continuity concrete flush with wall



- Continuity concrete flush with wall face is preferred but additional wall thickness may be required. The additional frame moments resulting from the additional stiffness needs to be accounted for in the design.
- Dimension W, thickness of wall. Value to be compatible with continuity thickness and to satisfy strength and flexibility design of frame.
 - Dimension J, semi-width of top of wall for temporary support of beams, value typically 400 mm, but needs to be larger where the beams are skew to the abutment.
 - Dimension H, height of extension of top of wall for temporary support, value to be sufficient for lapping of reinforcement to ensure necessary continuity.
 - Dimension G, thickness of continuity concrete in wall, value to be sufficient for lapping of reinforcement to ensure necessary continuity.
 - Dimension E, length of embedment of beams. Value to provide sufficient strength of continuity and may extend to incorporate transverse reinforcement through holes in beams.
 - Waterproofing to extend below construction joint on outside of wall.
 - Type of precast beam, P, to be chosen to suit design of bridge.

Detail 8.4.2-3 Frame abutment, beam and slab – continuity concrete wider than wall



- Option A, rectangular bottom corner of continuity concrete. This option is preferred because of reduction in complexity of reinforcement.
- REMARKS Dimension W, thickness of wall. Value to satisfy strength and flexibility design of frame.
 - Bottom corner of continuity concrete wider than wall could be splayed at 45°, Option B, rather than rectangular, thus improving compaction activity but increasing complexity of reinforcement.
 - All other remarks, dimensions and details as for Detail 8.4.2-2.
 - For sectional view see Detail 8.4.2-2.



PREFERRED .

Option A, using bearing pad, is preferred.

- REMARKS Positioning and levelling of bearing pads in Option A can be carried out in advance of placing precast beams.
 - Mortar bed in Option B needs to be prepared at the time of placing beams with attention paid to quality control of mortar mix.

8.4.3 Embedded wall abutments

The second illustration in Figure 8.2 shows the basic principle of an embedded wall abutment. The end support is formed by a diaphragm wall (or a contiguous, secant or sheet pile wall), which has its toe embedded in ground below the lower ground surface and the top made integral with the deck.

The form of construction selected for the wall will depend on several factors such as:

- particular site constraints
- total length of deck
- existing, and future, ground conditions
- desirable finish to face of wall.

Concrete walls are more suitable when the movements are relatively small. Greater movements can be accommodated more readily by steel sheet pile walls, and high-modulus sections (universal beam sections welded to sheet pile sections) are particularly appropriate for this application. The walls flex under the influence of the movement and are restrained against rotation by their length of embedment.

8.4.4 Piled abutments

Where there is no requirement for the bridge clearance to come close to the abutment face, the ground may slope up towards the top of the abutment. The commonest and simplest form of integral bridge abutment for this situation uses a single row of piles but with various abutment arrangements. An example, piled bank seat, is shown in Figure 8.3 and in more detail in Figure 8.5.

Concrete or steel piles can be used, but the designer must carefully consider the forces and movements induced, not only from the deck, but from soil/structure interaction. Details 8.4.4-1 and 8.4.4-2 illustrate the use of steel piles (forms of piling such as steel H or tubular, precast concrete driven or bored cast *in situ* concrete are equally acceptable) with a concrete capping beam cast monolithically with the piles and the deck.

Piles can also be used successfully for semi-integral construction (where the superstructure is not built-in to the abutment), see the last illustration in Figure 8.3, piled bank seat with end screen, and Detail 8.4.5-1.



Figure 8.5 Integral bridge abutments – examples of pile flexure type
Detail 8.4.4-1 Piled abutment – integral bank seat with run-on slab



Detail 8.4.4-2 Piled abutment – integral bank seat



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8.4.5 End screens

An end screen is a wall constructed at the end of a bridge deck that extends below the soffit of the deck and is located beyond the end support of the deck. They use bearings and this form of construction is sometimes called "semi-integral". Nevertheless, it still satisfies the main objectives of BD 57/95 ⁽¹⁾ by having no movement joints in the deck. The first and last illustrations in Figure 8.3 are examples of this type. An embedded wall (sheet pile) abutment with end screen is shown in Detail 8.4.5-1. The end screen acts to retain the material supporting the approaches, and only transfers longitudinal loads.

The designer has to give careful consideration to the detail at the bottom of the end screen and may wish to limit the amount of movement. Maintenance engineers will also need regular access in the normal way to inspect the bearings and for their potential replacement.





8.4.6 Bank pad abutments

An integral bridge bank pad abutment is an end support constructed monolithically with the deck, which acts both as a shallow foundation for the end span, and as a shallow retaining wall for adjoining pavement and embankment. The second illustration in Figure 8.3 indicates the type.

The bank pad form of construction is probably the most buildable end support and so should be used wherever possible. It can be used where the foundation material is firm enough to prevent significant settlement problems. Some settlement can be accommodated in the design of the deck and a bank pad on a well-compacted embankment construction is feasible. If there is doubt about predicted settlement, or if ground conditions are unacceptable, a piled bank seat will be the choice (Section 8.4.4).

The abutments need to be designed and detailed to accommodate both sliding and rocking movements.

A bank pad for construction on rock is shown in Detail 8.4.6-1. The form is similar to the frame abutment (see Section 8.4.2) but with movement principally accommodated by sliding (and rocking), not flexure. To ensure freedom to slide, a low-friction granular fill layer and slip membrane are incorporated. This detail has, however, not yet received an industry consensus and is subject to further development.

Detail 8.4.6-1 Bank pad abutment – sliding type



REMARKS • Arrangement is suitable for both steel and concrete superstructures.

- Use of a run-on slab is optional.
- Dimension F, extent of fill supporting slip membrane beyond edges of foundation. Typical value 400 mm.
- G, gauge of high-density polyethylene sheet, to be chosen to provide required separation between underside of structure and low-friction granular layer. Consideration should be given to the need to wrap the granular material in a geotextile membrane to avoid washout.

Detail 8.4.6-2 Slip membrane – chamfer at edge



- REMARKS
- This chamfer at the bottom corners of bank pad foundations is recommended to help avoid damage to the slip membrane and to the support at the edges.
 - The chamfer can be simply formed using polystyrene packing as shown, which is not removed.

8.4.7 Reinforced soil

Reinforced soil is a quick and efficient form of embankment construction. It can be useful in locations that have restricted access for plant and where the phasing of construction work provides insufficient time for more conventional forms of embankment construction.

The last illustration in Figure 8.2 illustrates how reinforced soil can be used in integral bridge construction with piling providing the primary support for the bridge. The reinforced soil reduces concerns about the differential settlement of the embankment. The use of reinforced soil to support an unpiled bank pad is not considered to have reached sufficient development.

8.5 RUN-ON (APPROACH) SLABS

8.5.1 Preamble

Run-on slabs, otherwise known as approach or transition slabs, have been used in bridge construction for many years with varying degrees of success. Their purpose is to provide a smooth transition between the relatively flexible construction of the approach pavement and the non-flexible construction of the bridge superstructure. They are mostly designed and detailed as simply supported slabs. Figure 8.6 illustrates some examples of their construction form.

Difficulties have been experienced with run-on slabs, which have created significant problems for maintaining authorities. Several have failed, in both integral and non-integral bridge types, due to poor design and/or construction.

Most failures have occurred as a result of:

- failure of the connection between run on slab and abutment
- differential settlement of the fill supporting the free corners of the run-on slab.

In integral construction, the forces generated by diurnal and seasonal expansion and contraction of the bridge superstructure will be transmitted through the abutment, thus increasing the effects on the run-on slab.

Opinion is divided on the advisability of including run-on slabs in bridge work schemes.

8.5.2 Forms of construction

The three forms of construction shown in Figure 8.6 are:

- buried (at depth)
- simply supported
- cantilever.

The most common form is the simply supported version, which, because of the structural rotations at either end, still requires some sort of carriageway surfacing joint detail. However, it is preferred that the waterproofing system runs continuously from the bridge deck across the run-on slab, and this is shown in Detail 8.5.2-1.

The buried type (Option A in Figure 8.6) does away with such carriageway surfacing joints and still permits waterproofing membranes to be continuous on to the run-on slab. The inclination of the run-on slab allows for a variation in depth of road construction, which aids the transition effect. It is not yet known how successful these buried slabs are in the longer term.

A run-on slab must be designed to span over the fill material. The design should be on a "simply supported" basis but with effects anticipated as being reversible, with reinforcement detailed in both top and bottom faces. The run-on slab should always extend to the full width of the carriageway.

For details of run-on slab connection and support, see Detail 8.5.3-1.





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Detail 8.5.2-1 Run-on slab – interface with carriageway (continued)

- The run-on slab is sloped downwards away from the bridge deck. The top surface of the run-on slab, starting at the underside of the bridge deck surfacing, meets the underside of the highway pavement construction road base at its end remote from the bridge.
 - Dimension Y, thickness of the run-on slab. Value to be as required for strength to span a potential void beneath.
 - Sub-base thickness adjacent to the end of the run-on slab to be increased locally, if necessary, to match the thickness of run-on slab construction.
 - Dimension Z, length of run-on slab, to be sufficient to span across hand-compacted or uncompacted fill behind abutment with adequate length of bearing on formation (see Detail 8.5.2-3), but not less than 3500 mm overall.
 - Dimension P, thickness of compressible filler at remote end of run-on slab. Value to be appropriate to accommodate designed expansion movement over length of bridge.
 - Dimension B, thickness of blinding concrete under run-on slab. Value typically 50 mm to 100 mm.
 - Movement joint detail at remote end of run-on slab will need adaptation from bridge deck details in Chapter 3 to permit the detail to be incorporated in the road base instead of on the bridge deck.
 - For details beneath run-on slab, see Detail 8.5.2-3



Figure 8.6 Run-on slab – construction forms



- The run-on slab is sloped downwards away from the bridge deck. The level of the top surface of the run-on slab starts at the underside of the sub-base for the standard road construction thickness and falls at an appropriate slope to assist sub-soil drainage away from the bridge abutment. REMARKS
- Sub-base thickness is increased locally to suit slope of slab.
- Notes for dimensions Y, Z, P and B as for Detail 8.5.2-1.
- For details beneath run-on slab, see Detail 8.5.2-3.

Detail 8.5.2-3 Run-on slab – fill backing to abutment



- Depth of soffit of abutment and slope of hand-compacted granular fill will, in conjunction with the required bearing length on to embankment formation, define the length of the run-on slab required (see also Details 8.5.2-1 and 8.5.2-2).
 - Dimension B, thickness of blinding concrete under run-on slab. Value typically 50 mm.
 - Dimension C, width of drainage layer. Value typically 225 mm for a 150 mm-diameter porous pipe.
 - Dimension P, thickness of compressible sheet, value to suit expected movement of bridge.
 - Where run-on slab is to be used with a non-integral bridge the compressible sheet should be omitted and fill not left uncompacted.
 - Ref S is to refer to the relevant project specification clause number.
 - Refs 6N and 6P are to refer to the project specification class for appropriate granular fill to structures.
 - For details above underside of run-on slab, see Detail 8.5.2-1.

8.5.3 Connections between slab and abutment

The run-on slab is subject to forces from thermal and flexural movements of the bridge and the adjacent highway as well as the impact from the wheels of traffic. It is therefore essential that, where a run-on slab is used, it is anchored positively to the bridge abutment to avoid the risk of movement of the run-on slab away from the bridge.

Types of connection between run-on slab and abutment that have been found to be most suitable are shown in Detail 8.5.3-1.

Options A to D have steel reinforcement bars cast in, with the slab resting on a recess or corbel constructed in the back face of the abutment. Use of the recess, rather than a corbel, simplifies formwork and construction.

Option E is simple in that there are no cast-in details. A "shear key" is created to prevent horizontal movement.

A basic assumption should be made at the design stage that the surfacing will crack at the connections and that proper waterproofing and drainage will be necessary. The use of stainless steel for reinforcement in the area of the connection should be considered and details should be kept as simple as possible.

Where steel reinforcement crosses the construction joint between run-on slab and abutment the concentration of flexure at the joint dictates that the steel bar crossing the joint needs to be debonded over a significant length. This will allow excessive concentration of strain in the steel bar to be avoided. There is also the potential for a concentration of corrosion at this point. For this reason, stainless steel is often used. However, if the debonding material also provides a proven protection against corrosion it should be acceptable for conventional (HY) steel to be used.

Detail 8.5.3-1 Run-on slab – connection types



(A) Anchor bars near bottom



(B) Anchor bars near top







(D) Inclined anchor bars



(E) No anchor bars

• Options A and B, a single layer of horizontal reinforcement providing horizontal anchorage between run-on slab and abutment are preferred.

AVOID • Short dowels.

:

- REMARKS Run-on slabs in Options A and B are cast directly against rear of recess in abutment to form a construction joint across which only the anchorage bars are passed.
 - Anchorage bars are debonded over a short length adjacent to the construction joint to avoid concentration of stresses at the crack that would result from the flexing of the run-on slab.
 - Use of corrosion protection (Denso) tape to provide debonding will permit the use of conventional reinforcing bar for anchorage. Otherwise stainless-steel bar must be used for anchorage.
 - If stainless-steel bar is used for anchorage the stainless-steel bars must be provided with electric isolation from the remainder of the structural reinforcement in the bridge abutment to avoid the effects of electrolytic action between the dissimilar metals.
 - Reinforcement bar to be fully anchored into run-on slab and abutment.
 - Bearing shelf for run-on slab typically 200–250 mm wide.

Detail 8.5.3-2 Run-on slab connection – anchor bars near bottom



REMARKS

- Length of anchorage bars to be at least sufficient to develop full anchorage bond into the run-on slab and anchorage, typically 800 mm, from the ends of the debonded length.
 - Dimension X, run-on slab bearing, typically 200–250 mm.
 - Anchor reinforcement to be positioned to pass just above bottom mat of reinforcement in run-on slab.

Detail 8.5.3-3 Run-on slab connection – anchor bars near top



• Length of anchorage bars to be at least sufficient to develop full anchorage bond into the run-on slab and anchorage, typically 800 mm, from the ends of the debonded length.

- Dimension X, run on slab bearing, typically 200–250 mm.
- Anchor reinforcement to be positioned near the top of the middle third of thickness of run-on slab.
- Dimension C, height of compressible material in bottom of construction joint to be approximately half the thickness of the run-on slab and to suit the position of the anchorage bars.

8.6 OTHER FORMS OF INTEGRAL BRIDGE

8.6.1 Arches

The oldest form of integral bridge is a masonry arch. There are numerous arch structures, many of them centuries old, which are still functioning perfectly well even though the loadings may have far exceeded those that the designer had in mind, see Figure 8.7. To a large extent this must be due to the fact that the materials in these structures are in permanent compression and also there is no steel to rust.



Figure 8.7 Masonry arch (integral) bridge

Another popular form of integral construction uses precast reinforced concrete arch segments, which have been used for culverts, road underbridges and tunnels. However, the details for such bridges, except where proprietary products are adopted for smaller structures, are developed by the engineers specifically to suit the circumstances. They have yet to be proven by successful use over a significant period.

Today's designer should ask himself whether the site in question would be appropriate for a masonry or mass concrete arch. When whole-life costings are applied to the various options an arch may be very economic.

8.6.2 Boxes

Another very common form of integral bridge is the box, either single-cell or multi-cell, see Section 3.5. These structures, when buried, have no movement joints at road level and no bearings. When precast units are used, usually as a proprietary product, care must be taken with the joint details between adjacent segments.

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Procedure for feedback

Individuals/organisations are encouraged to submit recommendations to improve existing details in the guide and/or suggest alternative or additional details. The CAD library on the CD-ROM provided with this book has been designed to be updated easily and quickly.

CIRIA has established a holding file for contributions. Receipt will be acknowledged at the time, and the contents of the holding file will be reviewed annually by the Highways Agency and CIRIA. The address and format for contributions are set out below.

Should a correspondent consider that the implementation of a recommendation they are making is urgent their submission should be marked URGENT in bold capitals at the head of the front page.

Address

BRIDGE DETAILING GUIDE FEEDBACK CIRIA 6 Storey's Gate Westminster London SW1P 3AU

Format

ORGANISATION/NAME

Address Telephone Fax Email

NATURE OF FEEDBACK

1. Modification to existing detail(s) or advice

- identify detail number and/or section number
 - provide copy of existing detail or advice marked up with recommendations
 - provide brief background and/or justification to support the recommendation/s

2. New detail and related advice

- provide copy of the new detail
- recommend related advice to be included in the guide
- identify the need and justification for each new detail.

Note

Authorship of any new details and/or modifications that are adopted will be acknowledged.



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September 2001



The Quality Services Directorate of the Highways Agency manages a large research programme that assists the Agency in its primary role as network operator for the trunk road network. The research aims to support the Agency's key objectives by consolidating and improving their information, knowledge, ideas, tools and technologies for (i) corporate technical strategy and (ii) meeting the wider Agency needs.

The UK highways bridge stock has exhibited durability problems in recent years, many of which are attributed to poor detailing and a lack of appreciation of buildability by designers.

Although there are many sources of advice on good detailing, it is rare for such guidance to be collected together. Organisations engaged in bridge design and/or construction usually have their own preferences. As modern mass production methods are increasingly applied in the construction industry, details will tend to be repeated within projects and from project to project. It is therefore important to identify best practice and to eliminate deficiencies.

The book concentrates on those details that have proved to be reliable in everyday use for durability and ease of construction, inspection, maintenance, repair and demolition. Only details that can be clearly defined have been included. Supporting text explains the rationale behind the choice of each detail and considers durability and buildability issues. The details have been prepared in a way that allows them to be readily adopted by designers, but care will still be needed to ensure that the details, and developments from them, are correctly interpreted and applied. The details are also supplied in .dwg format on CD-ROM.

This book has been prepared for use by active members of the bridge engineering profession and is intended for consultants, contractors, bridge owners and their maintaining agents. It will be of direct use to trainee engineers (including graduates), technicians and incorporated engineers involved in detailing highway bridge designs. It should also be of value to chartered engineers as they develop designs, and to site staff, as it provides advice on the function and relative merits of various details.



