Federal Republic of Nigeria

Federal Ministry of Works

Highway Manual Part 1: Design

Volume V:

Structural Design

2013

STRUCTURAL DESIGN

FOREWORD

The vision statement of the Federal Ministry of Works is to elevate Nigerian roads to a standard where they become National economic and socio-political assets, contributing to the Nation's rapid growth and development, and to make Federal roads functional, safe, pleasurable, and an avenue for redeeming Nigerians' trust and confidence in Government. This vision statement is in tune with the Transformation Agenda of the President of the Federal Republic of Nigeria, His Excellency, Dr Goodluck Ebele Jonathan, GCFR. Based on the foregoing, our mission is to use the intellectual, management and material resources available to the Ministry to make Nigerian roads functional all the time. The principal goal of the Ministry is to drive the transformation agenda by improving road transport infrastructure for the overall socio-economic derivable benefits and development of our great country, Nigeria.

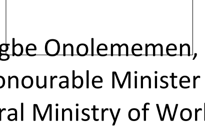
In exercising this mission and in discharging its responsibilities, the Ministry identified the need for updated and locally relevant standards for the planning, design, construction, maintenance and operation of our roads, in a sustainable manner. One of the main reference documents for this purpose is the Highway Manual, which previously included Part 1: Design and Part 2: Maintenance. Both current parts of the Highway Manual were first published in 1973 and 1980 respectively and have been subjected to partial updating at various times since then. The passage of time, development in technology, and a need to capture locally relevant experience and information, in the context of global best practices, means that a comprehensive update is now warranted.

The purpose of the Highway Manual is to establish the policy of the Government of the Federal Republic of Nigeria with regard to the development and operation of roads, at the Federal, State and Local Government levels, respectively. In line with this objective, the Manual aims to guide members of staff of the Ministry and engineering practitioners, with regard to standards and procedures that the Government deem acceptable; to direct practitioners to other reference documents of established practice where the scope of the Manual is exceeded; to provide a nationally recognized standard reference document; and to provide a ready source of good practice for the development and operation of roads in a cost effective and environmentally sustainable manner.

The major benefits to be gained in applying the content of the Highway Manual include harmonization of professional practice and ensuring uniform application of appropriate levels of safety, health, economy and sustainability, with due consideration to the objective conditions and needs of our country.

The Manual has been expanded to include an overarching Code of Procedure and a series of Volumes within each Part that cover the various aspects of development and operation of highways. By their very nature, the Manual will require periodic updating from time to time, arising from the dynamic nature of technological development and changes in the field of Highway Engineering.

The Ministry therefore welcomes comments and suggestions from concerned bodies, groups or individuals, on all aspects of the document during the course of its implementation and use. All feed back received will be carefully reviewed by professional experts with a view to possible incorporation of amendments in future editions.



Arc. Mike Oziegbe Onolememen, FNIA, FNIM.
Honourable Minister
Federal Ministry of Works,
Abuja, Nigeria
May, 2013

ACKNOWLEDGEMENTS

The Highway Manual has been updated by the Road Sector Development Team (RSDT), of the Federal Ministry of Works, with credit assistance from the World Bank's Federal Roads Development Project (FRDP). This update draws upon the original Manual, which was compiled between 1973 and 1980. The new Manual reflects recent developments in Road Design and Maintenance, in addition to latest research findings and updated references. Furthermore, it includes accepted practices that have been developed with the extensive effort of numerous organisations and people involved in the road sector. The assistance of all who have contributed is hereby gratefully acknowledged. Special acknowledgement is due to the following persons, who have been particularly involved and provided specific input that has been incorporated into the Manual:

Review Project Management Team:

Person	**Organisation**
Engr. Ishaq D. Mohammed	Director Highways/Unit Manager, RSDT
Engr. Chike Ngwuocha	Highway Engineer, RSDT

Peer Review Group:

Person	**Organisation**
Engr. B Giwa	Independent Consultant
Prof. Y. A Jimoh	University of Ilorin
Prof. K. J. Osinubi	Ahmadu Bello University, Zaria
Prof. L. Oyebande	University of Lagos
Dr. D. O. A. Osula	University of Benin

Thanks are also due to the following organisations that made staff available for the Stakeholder Workshop and other meetings, in addition to making direct contributions through comments and advice:

Public Organisations

Federal Ministry of Works – Highway Departments
Federal Ministry of Environment
Federal Roads Maintenance Agency (FERMA)
Federal Capital Development Authority
Federal Road Safety Corps
Nigeria Meteorological Agency
Nigerian Geological Survey Agency
Nigeria Police Force (Traffic Division)
Nigeria Hydrological Services Agency
Nigerian Meteorological Agency
Nigerian Society of Engineers
Nigerian Institute of Civil Engineers
Council for the Regulation of Engineering in Nigeria

Private Organisations

AIM Consultants
Aurecon Nigeria Ltd
Axion Consult Engineering Resources Ltd
Ben Mose & Partners
Dantata & Sawoe Construction (Nigeria) Ltd
Enerco Ltd
Etteh Aro & Partners
FA Consulting Services Ltd
Intecon Partnership Ltd
Julius Berger Nigeria Plc
Keeman Ltd
Multiple Development Services Ltd
Mansion Consulting Ltd

Property Mart Ltd
RCC Ltd
Sanol Engineering Consultants Ltd
Setraco Nigeria Ltd
Siraj International Ltd
Yolas Consultants Ltd

This update of the Highway Manual was compiled by the Road Sector Development Team of the Federal Ministry of Works with the assistance of the consultants Royal HaskoningDHV.

TABLE OF CONTENTS

List of Tables

List of Figures

Introduction

Description of the Highway Manual

Introduction to the Manual

The Highway Manual aims to guide members of staff of the Ministry and engineering practitioners, with regard to standards and procedures that the Government deems acceptable for the planning, design, construction, maintenance, operation and management of roads. The Manual directs practitioners to other reference documents of established practice where the scope of the Manual is exceeded; provides a nationally recognized standard reference document; and provides a ready source of good practice for the development and operation of roads in a cost effective and environmentally sustainable manner.

Arrangement of the Manual

The Highway Manual comprises a Code of Procedure and two Parts, each of which has been divided up into separate volumes, in the manner shown in Figure **Error! No text of specified style in document.**.1 **Error! Reference source not found.**.

Outline of Volume V: Structural Design

This volume is concerned with all facets of the planning, design and construction of bridges and other structures required as part of the Nigerian highway system. As concrete is the preferred material for the construction of highway structures in Nigeria, attention is focussed particularly on all forms of construction embodying reinforced and prestressed concrete components.

Emphasis is also placed on small to medium span bridges constructed by conventional methods, which are likely to constitute the majority of the significant structures on highways. Whereas large structures based on more sophisticated construction methods such as incremental launching, balanced cantilever and cable supported construction are briefly described in the text, particular design issues associated with these methods are not dealt with at this stage, but could be included in subsequent editions of the Design Manual.

Figure **Error! No text of specified style in document.**.1: Arrangement of the Highway Manual

Volume V is aimed at the achievement of **Safe, Serviceable, Economical and Aesthetically Pleasing Bridges** and provides guidance on the attainment of these goals and the broader concerns of **Sustainability**, by the application of sound engineering principles at each stage, from initial planning through to construction. This volume is based on the utilisation of the BS 5400 Code of Practice for the design of bridges, and serves to emphasise important aspects of the Code.

Highway Structures

Bridges and other structures are an essential component of highway systems and should be given special consideration in the investigation and planning of new roads, because of their potential vulnerability to a wide range of risks. In the case of very large river crossings and in certain other circumstances the planning and design of bridges is likely to have a significant influence on the location and geometry of the road, in order to optimise designs in terms of overall safety and economy.

However, in some instances bridges may constitute a modest percentage of the total cost of highways, especially when the cost of land acquisition is also taken into account. Whereas it is preferable to simplify bridge geometry when practical and economical to do so, the location and geometry of bridges should generally be treated as subordinate to the planning and design of the road, rather than vice versa.

Nevertheless, the bridge designer should always provide input from the Technical Feasibility stage of highway planning, as indicated in the **Code of Procedure** which accompanies this Manual.

Responsibility of the Bridge Designer

This Volume is not intended as a substitute for the application of sound engineering knowledge and judgement required for the successful planning and design of highway structures, which should be aimed at the achievement of the goals outlined in Section 1.1. The bridge designer shall conceive, analyse, design and prepare drawings for structures in such a way that the fundamental behaviour, durability and safety of the structures are within acceptable norms and are capable of remaining 'fit for purpose' during their intended design lives. The structures so designed shall be best suited to the site conditions, environment and other constraints, making due allowance for constructability and costs.

The onus rests with the bridge designer to acquire all data and investigate the determinants described in Chapter 2, required for or having an influence on the successful planning and design of the structures, and to obtain the necessary approvals from all parties concerned with the structures, in good time to avoid undue delays. In the interests of good practice and for record purposes, the bridge designer shall ensure that all issues which may have a material influence on the design and construction of the structures are well documented.

In keeping with this requirement, special attention shall be paid to the Design Statement which is to be included on the General Arrangement drawing of all structures. The information required in this context is outlined in Section 7.2 and should be comprehensive for construction purposes and for the cases of the future need to repair, strengthen or adapt existing structures.

The bridge designer shall employ a quality management system which embodies thorough and independent design checks by personnel competent in this field, to ensure compliance with the FMW's requirements.

Bridge Schedule

As soon as the location of a new route and the provision of bridges and other structures has been approved in principle by the FMW, it is required that the location, type and proposed geometric details of these structures be recorded on a **Bridge Schedule**, which is registered with the FMW to facilitate future management of the highway system.

The bridge schedule shall consist of a key plan and schedule on the same A0 drawing. The content and format of the drawing shall comply with the requirements specified in Section 9.2.1, and the drawing shall be numbered in accordance with the system specified by the FMW.

Prior to commencing with the DETAILED DESIGN PHASE the bridge designer shall review the original bridge schedule and affect any necessary reviews to standards and include any additions thereto. The updated bridge schedule shall be submitted to the FMW for acceptance.

Procedure and Guidelines

Procedures

The companion document to this HIGHWAY MANUAL is the **Code of Procedure**, which sets out the procedural steps required to be followed in the processes of investigation, planning, design and documentation of all facets of Nigerian highways.

This Volume V of the HIGHWAY MANUAL: PART 1 - DESIGN is arranged in a similar manner, in which the chapters follow the usual sequential steps from planning through to construction. In colloquial terms these chapters may be described as follows:

Chapter 1: Setting the scene

Chapter 2: Information required for design

Chapter 3: Waterway requirements for river structures

Chapter 4: Schedule of structures and components available for selection

Chapter 5: Principles to be met and goals to be achieved in the process of design

Chapter 6: Selection of the structural form best suited to particular circumstances

Chapter 7: Optimisation and refinement of the proposed structure

Chapter 8: Design and detailing issues to watch out for

Chapter 9: Documentation required for approval, tenders and construction

Guidelines

Whilst Volume V includes information about mandatory inputs and outputs in the process of design, it is not intended to be prescriptive about the application of the Design Code requirements. Rather the intention is to encourage the use of the Design Code in an imaginative way in the quest to optimise the selection and design of new highway structures.

This Volume may well serve as no more than an aide mémoire or useful check list on certain issues for experienced designers, but it is specifically aimed at assisting less experienced bridge designers to recognise potential design problems and avoid inadvertent omissions (such as discussed in Chapter 8), which could be difficult to rectify at subsequent design stages, or even during construction.

Emphasis is placed on the generous proportioning of bridge components at the conceptual design stage, in order to provide ample scope for optimisation and refinement at the detailed design stage. The bridge designer is encouraged to adopt simplified analytical and design procedures at the outset and to resort to more sophisticated procedures only at the detailed design phase, when it becomes essential to verify component strength and behaviour in terms of the applicable structural principles.

Furthermore, the bridge designer is advised to question given information and design assumptions at an early stage, and to carry out simplified manual checks at strategic stages during the design process, especially to avoid **systematic and gross errors** being carried forward to the next design phase.

Systematic and especially gross errors are not necessarily covered by the Design Code partial safety factors. Consequently it is imperative that such errors be eliminated by the application of proper checking procedures. Systematic errors arise, among other sources, from:

The unquestioning acceptance of information provided by others for design purposes.

The flawed interpretation thereof.

Reliance on assumptions, which may not or cannot be met in practice.

The inadequacy of the methods used to model, analyse and design structures.

Gross errors usually relate to the order of magnitude of the design actions and action effects, which are required for the determination of the size and strength of a structure and its component parts. However, both systematic and gross errors can occur throughout the entire process from conceptual design through to quantification and specification for construction purposes.

Applicable Codes and Standards

Bridge Design Code

The adaption of Eurocodes for application to Nigerian highway structures is under consideration, and among other benefits offers the country the advantage of a Code supported by ongoing research, mostly conducted and funded by other countries. However, on the evidence of the experience of others in this context, References 1.1 and 1.2, it is apparent that in order to adapt Eurocodes to Nigerian requirements, it will be necessary to undertake a significant amount of preparatory work to create the local framework needed to harmonise Eurocodes with Nigerian circumstances and the interests of the users of the Code. Until this process is well advanced it will remain preferable to continue to use BS 5400 for the design and construction of Nigerian highway structures.

The applicable Code of Practice for the design of bridges and other structures for Nigerian highways shall therefore continue to be the latest edition of:

BS 5400 : British Standard: Steel, Concrete and Composite Bridges, which comprises:

Part 1:	*General statement*
Part 2:	*Specification*
Part 3:	*Code of practice for design of steel bridges*
Part 4:	*Code of practice for design of concrete bridges*
Part 5:	*Code of practice for design of composite bridges*
Part 6:	*Specification for materials and workmanship, steel*
Part 7:	*Specification for materials and workmanship, concrete, reinforcement and prestressing tendons*
Part 8:	*Recommendations for materials and workmanship, concrete, reinforcement and prestressing tendons*
Part 9:	*Bridge bearings:*
Section 9.1:	*Code of practice for design of bridge bearings*
Section 9.2:	*Specification for materials, manufacture and bridge bearings*
Part 10:	*Code of practice for fatigue*

In addition, the latest editions of the other British Standards or Codes of Practice to which BS 5400 refers shall be deemed to apply.

Alternative Bridge Design Codes

A bridge designer who wishes to use any Code of Practice other than BS 5400, for the design of Nigerian highway structures, shall apply in writing to the FMW for permission to do so, fully setting out the reasons and justification therefore.

Neither Eurocodes, nor any other Code of Practice or part thereof shall be used for the design or construction of Nigerian highway structures, without the express written approval of the FMW.

FMW Standards

In addition to the British Standards deemed to apply as noted in Section 1.6.1, the FMW has adopted certain other Standards and Standard Details which shall apply to Nigerian highway structures, unless otherwise approved in writing by the FMW.

The following, for example, shall be applied as the minimum standards to be met:

1. The vertical clearance above roads or railway lines to the soffits of bridge decks, as specified by the FMW, and further discussed in Chapter 2.

The horizontal clearance to bridge deck supports as specified by the FMW, and further discussed in Chapter 2.

The design flood return periods and the associated free board limits for the design of river bridges and major drainage culverts, as determined from Chapter 3.

Any bridge component details issued by the FMW shall be treated as standard.

References

1.1 Nethercot D.A. MODERN CODES OF PRACTICE: WHAT IS THEIR EFFECT, THEIR VALUE AND THEIR COST? *Structural Engineering International 2/2012* 176 - 181.

1.2 Retief J.V. Wium J.A. PRINCIPLES AND APPLICATION OF STRUCTURAL DESIGN CODE DEVELOPMENT IN SOUTH AFRICA. *Structural Engineering International 2/2012* 182 - 194.

Design Data and Determinants

Introduction

Road Authority requirements and standards

The bridge designer is advised to become acquainted with the requirements and standards of the FMW before commencing with the project. This manual serves to provide an insight into the requirements of the FMW, but other relevant authorities should also be considered from the outset in order to provide a design which will satisfy all parties. This may be achieved by preliminary discussions with the relevant authorities and a study of their Codes of Procedure, Design Standards and Specifications, if relevant. When authorities do not have such documents available or are faced with uncertainties in the event of an unusual design situation, the bridge designer should assist in this regard by doing relevant research and providing innovative proposals.

Planning for present and future needs

Before embarking on gathering information about the design data and determinants required, the bridge designer should consider potential future changes to the structure, either planned or unforeseen. Planning for future extensions, widening or even obsolescence of a structure will be dictated by the FMW, but it is always prudent to provide a design which is amenable to future modification.

Available Information

Previous investigation reports and data gathered by others

All available records including correspondence, feasibility studies, basic planning, preliminary design, topographical surveys, geotechnical investigations, hydrological studies etc., prepared beforehand by the bridge designer or by others, should be identified and collected from the various project participants, past or present, and studied to determine if any further information is required. It should be noted that, during a dormant period of a project, it is possible that personnel previously involved may have been replaced by others with different design philosophies. It is also possible that the FMW's needs may have changed. It is therefore important for the bridge designer to be familiar with the relevance of available data in the light of latest requirements.

Road drawings (plan; longitudinal section; cross section; typical details)

If the road design has been completed, the plan, longitudinal section, cross section, typical details and other relevant drawings should be collected from the road designer as part of the information required to determine the bridge envelope in relation to the road. This will also assist the bridge designer to assess the extent of further investigations and surveys required.

Ideally, the development of roads drawings in the vicinity of proposed road structures should be a joint effort between the road and bridge designers. Once the road design has reached an advanced stage, drawings should be provided to the bridge designer to allow progress with the design process. As planning and design are continuous processes, no road drawing should be considered final until the bridge design has been completed. It is important for the two disciplines to liaise regularly so that design modifications are made known to each party.

As made records

As made drawings are invaluable for the design of modifications to and strengthening of existing structures. These drawings may be obtained from the FMW or the authority responsible for the original construction. The accuracy of such records should not be taken for granted and should be confirmed by comparing the drawings with the actual structure. **The importance of maintaining proper as made records by the FMW is self-evident.**

Site Inspection

Scoping, observations and records

The bridge designer shall make arrangements for inspection of the site with regard to the nature and scope of the work. Such a site visit is obviously good practice and should be mandatory by a professional design office.

The approximate position and alignment of the proposed structure should be known before a site visit is made. During the first site inspection, the bridge designer should inspect the terrain and record all relevant points of interest for the further planning of the project. A photographic record with geo-tagging at this stage is extremely helpful as an aide mémoire assisting the bridge designer to recognise all relevant physical features of the bridge site. Photographic records which indicate the position and direction of each photograph should be kept for inclusion in relevant reports. The use of a GPS device to record relevant co-ordinates on the proposed bridge site is strongly recommended.

Features to be observed which could have an influence on the proposed design include the following (where applicable):

Shape and nature of the terrain

Nature and extent of the obstacle to be crossed

Exposed potential founding material

Hydraulic constraints such as existing structures close to the proposed bridge, together with visible high water levels

Scour potential of the river bed material

Vegetation of the catchment area with particular reference to afforestation

Nature and details of existing structures

Existing services (water mains, sewers, drainage structures, electricity cables, railway support facilities, etc.)

Existing buildings and property fences

The position and direction of the nearest access roads

Position of and access to potential sites to accommodate the storage of large components such as precast beams or the contractor’s site establishment

Possible access for the transport and positioning of drilling equipment for geotechnical investigations and for construction plant.

Once the initial planning of the structure has been concluded, it is strongly recommended that a follow up site visit be undertaken to confirm initial observations.

Appraisal of existing structures

Existing structures fall into two categories:

1. Structures to be modified or strengthened

Such structures should be studied on site to determine possible deviations from as made records or to determine structural deterioration. A photographic record is essential.

Structural failures should be recorded meticulously as to position, extent and magnitude so that an experienced bridge designer may readily recognise and determine the cause(s) of failure.

On site observations and records should also focus on structural features which may be utilised for the modification and/or strengthening of the structure.

Structures to be either demolished or retained close to proposed new structures

This category of structure should be carefully considered on site to establish whether its proximity to the proposed new structure will influence the latter's function or capacity such as an existing structure directly up or downstream of a proposed new river bridge. Likewise the foundations of an existing bridge require careful consideration when bridge doubling is proposed.

Topographical Survey

Scope and extent

The two primary objectives of a site survey are to provide a comprehensive plan of the site showing all necessary details and data required for the design and accurate siting of the proposed structure, and to provide co-ordinated survey beacons to be used for setting out and control of construction.

It is important to carefully plan the scope and extent of the topographical survey to be undertaken. After the initial site inspection, the bridge designer should consider the range of options available regarding the type and size of the proposed structure and the extent of the proposed road embankments, and then decide on the required survey to be undertaken. Adopting details from the road centre line survey only should be done with extreme caution as essential detail may be missed.

The bridge designer should also consider the appropriate survey to be done for hydraulic design (if applicable) and determine the number and extent of water course cross sections required for the analysis of the proposed structure, as required by the hydraulic software used (refer to 3.3.1 of Chapter 3).

Survey standards

Structure site surveys should be undertaken in compliance with the guidelines indicated by the FMW which should cover the following:

Details to be surveyed

Requirements for the survey, including the extent of the survey, the required drawings and the required longitudinal and cross-sections

Requirements for the permanent survey stations and references pegs

Requirements for the horizontal and vertical control

Requirements for digital terrain models

Procedure to be followed when entering onto private property

The bridge designer is required to determine the survey accuracy required to plan the proposed structure and must indicate the density of spot shots and the appropriate contour interval required, as well as any further information required, e.g. railway details, position of services, etc.

Geotechnical Investigation

Scope and extent

The purpose of a geotechnical investigation is to provide an accurate assessment of subsurface conditions which will assist the bridge designer in the selection of suitable foundation types and the selection of appropriate embankment slopes, etc. It will also assist in the preparation of tenders and ultimately in the execution of the work.

The investigation is normally undertaken in several phases including initial desk studies and a site inspection which may include visual inspections and the digging of preliminary test pits. If feasible, geophysical methods such as resistivity or seismic refraction may be employed to assist in the choice of further detailed investigations. The bridge designer should also gather initial data from available sources such as topographical maps and aerial photographs (for the identification of geological faults, boundaries or dykes, drainage patterns and soil types, etc.), geological maps, investigations done by other authorities, and mining activities.

The bridge designer is referred to the Geological Map of Nigeria, produced by the Nigeria Geological Survey Agency, a copy of which is contained in Appendix A of Volume III: Pavements and Materials Design of The Highway Manual Part 1: Design.

Location, depth and type of exploration

The detailed geotechnical investigation should be undertaken at a stage when possible foundation positions and types have already been identified.

The bridge designer's judgement and experience is required to interpret the data gained in the preliminary geotechnical investigations, to determine whether further investigation is warranted. Should it be decided to continue with further detailed investigations, the extent and type of the exploratory works to be undertaken should be determined by a geotechnical engineer or a suitably experienced and competent bridge designer. This may include the following, as appropriate:

Auger trial holes

Test pits

Standard penetration testing

Dynamic cone penetration testing

Cone penetration testing

Rotary drilling, in-situ testing and sampling

Sampling

Rotary percussion drilling

Plate load tests

In-situ density tests

Geophysical techniques

Laboratory testing requirements

The laboratory testing requirements for soils are summarised in Appendix 2A, which has been extracted from Reference 2.1.

Content of geotechnical report

The geotechnical report shall be prepared to provide clear guidance to the bridge designer, to facilitate selection of the most appropriate solutions and foundation types. The report should quantify the parameters required for the design of the proposed foundation and approach embankment and should include the contents summarised in Appendix 2B.

Global Geometry

Conformity with road geometry

Once the geometry of a road has been established, the proposed structure should conform to the exact physical characteristics of the road in order to maintain continuity and the safe passage of traffic. The width of the bridge may differ from that of the road, depending on the requirements of the FMW.

Horizontal and vertical road alignments

The bridge designer should liaise with the roads designer during the preliminary design of the vertical and horizontal alignments, especially with regard to the depth of the bridge deck as this will impact on the allowance for vertical clearances. Once the road geometry has been finalised, the bridge designer may proceed with the preliminary design of the bridge to the same geometric configuration as the road. This will include the horizontal and vertical road alignments together with the cross section and its associated cross fall or superelevation.

Horizontal and vertical clearances

Grade separation structures are generally designed according to prescribed clearance diagrams indicating horizontal and vertical clearances which should be obtained from the relevant authority of the road or railway being crossed. Such diagrams will dictate the minimum deck span lengths and also the maximum allowable deck depths in relation to the vertical road alignment.

River bridges are designed according to a prescribed freeboard as set out in **Chapter 3: Waterway Requirements (River Structures)** and the deck depth will be similarly affected as for vertical clearance discussed above.

Structure geometric envelope

The proposed bridge is contained within a structure geometric envelope of which the boundaries are as follows:

Upper boundary: Vertical alignment less allowance for asphalt surfacing if applicable.

Lower boundary: Founding levels.

Horizontal boundary in elevation: Defined by the abutment positions (two possible options: closed abutments or additional end spans with spill through or bank seat abutments).

Horizontal boundary in plan: Maximum transverse width of the bridge, parallel to the road centreline.

In addition to the above, the envelope elevation may include clearance diagrams where no substructure elements are allowed.

Loads and Load Combinations

Types and combinations

Bridge loading is categorised into permanent and transient loads. Loadings shall be applied separately or in combinations as set out in BS 5400 in order to determine the most severe feasible combination of effects to be considered in the design of a structure

Permanent loads

Permanent loads are divided into two groups:

Loads due to the influence of gravity on the mass of the structure. These include the following:

Dead loads

Superimposed dead loads and retained earth pressure

Loads due to the configuration and stiffness of the structure, method of reinforcement and materials behaviour which, inter alia, may include the following:

Prestressing loads

Restrained deformation loads

Transient loads

Transient loads are temporary and may be divided into three groups:

Loads caused by the environment

Traffic loads (the nature of which must be confirmed by the FMW)

Construction loads

A summary of the various elements of bridge loading is shown in Figure 2.1 for reference. Further general information on this subject is provided by Reference 2.2, and about flood forces in Reference 3.3 (refer to Chapter 3).

The image cannot currently be displayed.

Figure **Error! No text of specified**

Environmental Constraints

Specific measures or controls

Project specific measures or controls will be set out in the Environmental Management Plan (EMP) as dictated by Volume VII: Environmental, and should be carefully considered at the Preliminary Design Phase of all structures and included in the contract documents as appropriate.

Access causeways; river diversions and general requirements

The bridge designer should always be sensitive to the environment and, in addition to the requirements of an EMP, strive to prepare a design which will minimise the construction "footprint" and harmonise with the surroundings. The following are important items to be considered in the design in this regard:

Temporary river crossings and diversions

Unnecessary destruction of vegetation and natural features

Method of construction, with particular regard to environmental impacts

Local Construction Industry Skills and Capacity

The following factors may have an influence on the choice of bridge configuration and materials when planning a project:

1. Access to and distance from major supply centres, concerning logistic problems and availability of skilled labour.
2. Availability of sophisticated manufactured elements such as bridge bearings, expansion joints, prestressed concrete components etc.
3. Possible FMW policy with regard to the use of local skills and labour.

References

2.1 Byrne G (Ed), Berry AG (Ed), A Guide to Practical GEOTECHNICAL ENGINEERING in Southern Africa, Franki-Johannesburg, (Fourth Edition - December 2008).

2.2 Ryall JM, LOADS AND LOAD DISTRIBUTION, ICE Manual of bridge engineering (Second Edition), Parke G (Ed) and Hewson N (Ed), Thomas Telford, London (2008).

ISBN : 978-0-7277-3452-5

Appendix 2 A

Guide to Laboratory Procedures and Requirements

This image cannot currently be displayed.

Appendix 2B

Guide to the Contents of Geotechnical Reports for Bridges

Introduction

Terms of reference

1. Purpose for which the investigation was conducted

Description of the site

Location

Accessibility to the site

Trafficability of the site for construction equipment

Listing sources from which data is available or was obtained from

Description of regional geology, vegetation, drainage and other general features of importance

Investigations carried out

Identification of all persons and firms responsible for the field work, interpretation of the geophysical work and the profiling

Record of dates on which work was conducted

Description of the types of field work undertaken and equipment used

Investigation results

Description of the soils encountered, identifying their stability or potential problems they may present e.g. heaving, collapse, settlement, etc.

Description of hard rock geology identifying the type, quality, degree of weathering, fracturing etc.

Potential for boulders and other obstructions to piling or caisson foundations

Description of markers for the projected scour depth

Description of the problems experienced or to be expected

Description of groundwater and expected variations

Field and Laboratory testing carried out, e.g.

types of test conducted on the respective materials and the results obtained.

Recommendations

Type of foundation best suited

Expected bearing capacity and settlement for the respective materials on which founding could be considered

Friction values and rock socket parameters for the design of piles

Construction sequences

References

Listing of standards used for the classification of materials in respect of soil condition and rock hardness

Annexures

Locality plan to appropriate scale

Results of geophysical investigations

Borehole, auger holes and test pit logs

Photographs of borehole cores recovered

Laboratory test results

Drawings to scale showing the location, including levels of all positions investigated, physical features of the site and setting out points.

River Bridges and Culverts

Introduction

Scope

This chapter provides technical guidance about the hydraulic design of river bridges and culverts. This information is based on the current knowledge of the related issues, which is well presented in the references listed at the end of this chapter. These documents are mostly freely available in the public domain, or alternatively from the publishing authorities.

The bridge designer is advised to study these references for more detailed information than provided in this chapter, in order to best guard against the additional risks to which river-related structures are subjected, when compared with equivalent "dry-land" structures.

Optimisation and evaluation procedures

The optimal design of drainage systems and hydraulic structures could be defined as that which maintains a proper balance between the cost of the project and the cost of the potential flood damage (economic risk). Various hydro-economic analytical techniques have been developed for this purpose, as outlined in Reference 3.1.

However, optimisation of the types outlined is usually warranted only in the case of large river bridges located on strategically and economically important routes (risk category 3) as summarised in Table **Error! No text of specified style in document.**.2. In most other cases of less important structures, simpler selection and design procedures are justified, as outlined in this chapter.

The minimum design standard

It is established practice to design a facility for a specific design event, commonly referred to as the "design flood" (Q_D). The minimum design life of the structure is determined by the choice of Q_D. A structure is designed to withstand a maximum flow expected with a return period T(=D), usually in the range from 10 to 100 years. The expectancy is determined by analysis of records of past rainfalls or flows, when available, or alternatively from accepted flood estimation techniques, such as the Rational Method which is commonly used in Nigeria, as outlined in Volume IV.

From an economic perspective the minimum design standard should be based on the design life of the road. The reason is that the majority of river bridges become obsolete long before they become structurally dysfunctional. The obsolescence should be ascribed to:

Increased traffic volumes exceeding the road traffic carrying capacity;

Road design standards that change, particularly with regard to lane and shoulder widths, and

Realignment of the road to provide more favourable geometric standards (safety) or to shorten travel distances.

Indicator flood

In the past various criteria were used to select the design return period of the flood for a particular site. From observations these criteria yielded inconsistent results, when for example the return period was based on the catchment area or some other characteristic of the catchment or region. Conversely it is evident that the rate of flow is the most significant factor in quantifying the potential damage to a road, and combined with the road class indicates the likely extent of traffic disruption or delay in the event of inundation or the need for damage repair.

For these reasons the peak flow rate for an **Indicator Flood** with a return period of 20 years (frequency = 1:20) is adopted as the basis for selection of the appropriate design return period.

Road classification

The strategic importance of the road (Road Class) is adopted as the second parameter used for the determination of the appropriate Return Periods, for which river bridges and drainage culverts should be designed. The use of Road Class in this manner will result in the adoption of longer return periods for the higher road classes, in recognition of the need to provide greater assurance against inundation and/or damage.

In order to cater for the design of drainage structures for lower classes of roads provision is made in this chapter for four road classes (A to D), in terms of the Road Classification System described in Chapter 2 of Volume I. The bridge designer is required to always confirm the applicable road class in writing with the FMW, prior to the commencement of the hydraulic design of river bridges and culverts.

Design Return Periods

The required design return periods (frequencies) are summarised in **Error! Reference source not found.** and depicted in Figure **Error! No text of specified style in document.**.3, which follows in Section 3.2.

Table **Error! No text of specified style in document.**.1: Proposed design return periods, T, for different road classes

1. Road class		Proposed return (T) based on the magnitude of the Q_{20} flood		
		$Q_{20} < 20\ m^3/s$	$20\ m^3/s < Q_{20} < 150\ m^3/s$	$Q_{20} > 150\ m^3/s$
A	National Trunk Roads	50	$T = 42.31 + 0.385Q_{20}$	100
B	Primary Roads	20	$T = 15.39 + 0.231Q_{20}$	50
C	Secondary Roads	10	$T = 8.46 + 0.077Q_{20}$	20
D	Minor Roads	10	10	10

The design return periods given in **Error! Reference source not found.** should be adapted if deemed necessary in accordance with the Risk Category, as obtained from Table **Error! No text of specified style in document.**.2. It remains the responsibility of the bridge designer to motivate the design return period to be used in the Technical Feasibility stage of planning, by allowing for the influence of possible future developments, such as urbanisation or dam construction on the flood peak.

Table **Error! No text of specified style in document.**.2: Factors to be considered in determining the risk category of the structure

Factors to be Considered	Risk Category		
	1	2	3
Extent of possible damage Potential damage to the road and associated cost of repairs	Low	Medium	High

Factors to be Considered	Risk Category		
	1	**2**	**3**
Potential other damage such as saturation of agricultural land, etc.	Low	Medium	High
Extent of loss of use			
Time needed for repairs to make route trafficable again	Short	Medium	Long
Availability of detours	Good	Medium	Long
Obstruction of traffic flow			
Period of flooding	Short	Medium	Long
Traffic density	Low	Medium	High
Depth and velocity of floodwaters	Low	Medium	High
Strategic and economic importance of route			
Strategic and economic importance: military, police, fire brigade, medical services, etc.	Low	Medium	High
Economic importance	Low	Medium	High

If the potential damage and the impact of disruption, due to inundation or failure of the structure are particularly high (Category 3 in Table **Error! No text of specified style in document.**.2), consideration should be given to increasing the design return periods in **Error! Reference source not found.** as deemed necessary, based on local incidences in the past. In the case of Risk Category 1 consideration might be given to alter the design return period after full consideration of all relevant aspects, and the motivation for alteration is accepted by the FMW.

Exceedance Probabilities

In the case of very large and strategically important structures, the possible influence of particularly large floods (such as the probable maximum flood) should be taken into consideration. In these instances the probability and potential effect of the exceedance of the estimated flood levels should be reviewed using Table **Error! No text of specified style in document.**.3 in conjunction with the factors listed in Table **Error! No text of specified style in document.**.2.

The probability P% of occurrence of a T year flood within a period of N years may be obtained from the values recorded in Table **Error! No text of specified style in document.**.3, which are based on

$$P = [1 - (1 - \frac{1}{T})^N] \times 100\%$$ ______________________**Equation** Error! No text of specified style in document.**-1**

Table **Error! No text of specified style in document.**.3: Probability of occurrence of a T year flood

Probability P%							
1. / 9.	2.	3.	4.	5.	6.	7.	8.
2	75	36	19	10	4	2	1
5	97	67	41	23	10	5	2
10	100	89	65	40	18	10	5
20	100	99	88	64	33	18	10
50	100	100	99	92	64	39	22
100	100	100	100	99	87	63	39
200	100	100	100	100	98	87	63

Design Flood and Freeboard Determination

Flood Peak Estimates

Flood peak estimates are required to be determined by the methods described in **Volume IV: Drainage Design**, or by any other hydrological analyses approved by the FMW. It should be noted that the return periods of the floods for the design of river bridges and culverts are not established in Volume IV, but are determined as outlined below following receipt of the estimated peak flow rates for floods with return periods T, of 2, 5, 10, 20, 50 and 100 years.

The peak flow rate estimates together with the summary of the catchment parameters, rainfall and runoff data shown in Table **Error! No text of specified style in document.**.4 are required for subsequent inclusion on the General Arrangement drawing, for future reference.

Table **Error! No text of specified style in document.**.4: Required Catchment and Flood Peak Data

Catchment Area A : km^2			Length of Longest Collector Lc : km			
Average Slope of Longest Collector So : %			Invert Elevation at Road C/L : m			
Time of Concentration T_C : h			Areal Reduction Factor □ : %			
Recurrence Interval T : yrs	2	5	10	20	50	100
Point Rainfall Depth d_T mm						
Runoff Coefficient C						
Peak Flow Rate Q_T m^3/s						

Design Flood Frequency

The value of the 20 year Peak Flow Rate (Indicator Flood) from Table **Error! No text of specified style in document.**.4 should be entered in Figure **Error! No text of specified style in document.**.3 in order to determine the Design Flood Frequency (T years) from the applicable Road Class line. Determination of T automatically establishes the return period 2T for the required check flood.

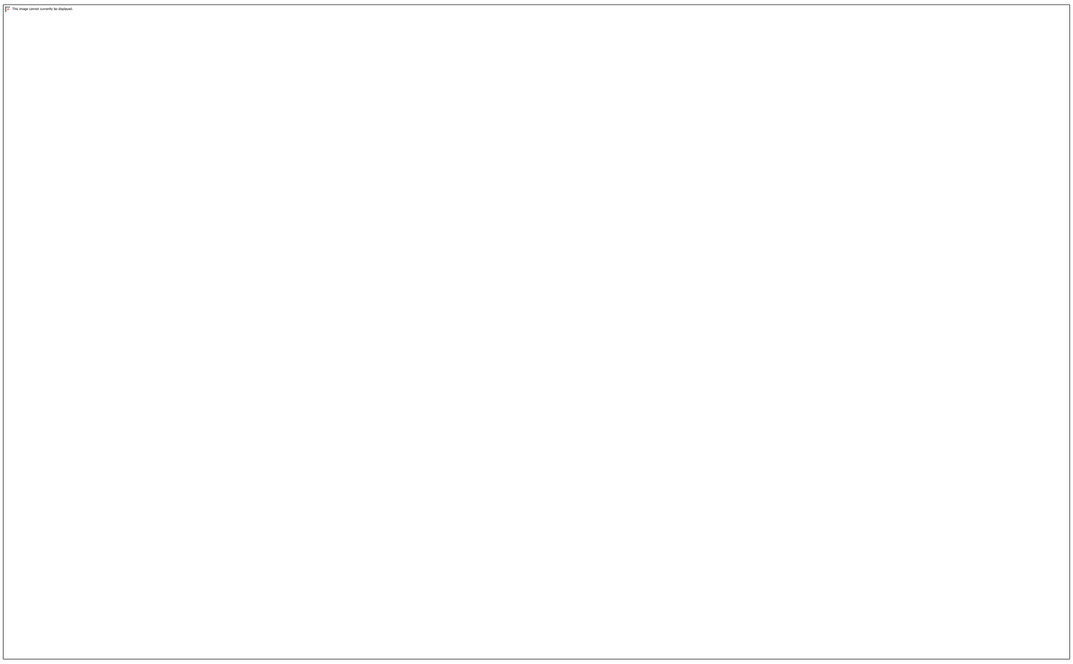


Figure **Error! No text of specified style in document.**.3: Design flood frequency estimate

Design and check floods

The design flood peak Q_T and the check flood peak Q_{2T} should be interpolated from a plot of the peak flow estimates obtained from Table **Error! No text of specified style in document.**.4, as illustrated in Figure **Error! No text of specified style in document.**.4

Figure **Error! No text of specified style in document.**.4: Illustration of Method of Determination of Design and Check Flow Rates QT and Q2T

Freeboard

It must be noted that the categorisation of drainage structures as bridges or culverts are for risk management purposes and do not refer to the form of the structure. Thus a bridge may take the form of a culvert and a culvert may operate in the form of a bridge in terms of freeboard and hydraulic performance, as illustrated in Figure **Error! No text of specified style in document.**.5.

Figure **Error! No text of specified style in document.**.5: Illustration of the hydraulic definitions

Distinction is made between two hydraulic types of conveyance and two hydraulic criteria which need to be met:

Free flow structures - structures with freeboard at the design flood and the friction loss in the flow is relatively small so that the water surface does not touch the soffit of the structures.

A prescribed freeboard, $\mathbf{F_D}$, which is the distance that the level of the design flood, $\mathbf{Q_T}$, below a deck soffit (underside of deck) should be maintained. $\mathbf{F_D}$ is shown in Figure 3.4 and is dependent on the magnitude of the design flood, $\mathbf{Q_T}$, the return period of which is determined from Figure **Error! No text of specified style in document.**.4 for the applicable road class; and

The level of afflux caused by a flood having **twice** the recurrence interval **(2T)** of the design flood shall be below the shoulder breakpoint $\mathbf{F_{SBP}}$ of the road (see also Figure **Error! No text of specified style in document.**.5). Please note this **is not** $2Q_T$ but a flood with magnitude of $\mathbf{Q_{2T}}$. The maximum afflux requirement to shoulder breakpoint, $\mathbf{F_{SBP}}$, shall generally only apply to Road Classes A, B and C.

Inlet control type structures - structures where the upstream water level of the design flood is above the soffit of structure.

The submergence of the inlet is usually limited to a ratio of $H_1/D = 1.2$ for the design flow rate of Q_T, where D is the internal height of the structure and H_1 is the water depth at the inlet measured from the invert of the structure; and

For the flow rate of Q_{2T} the maximum allowable submergence level is limited to the smaller of 2D or the SBP.

Figure **Error! No text of specified style in document.**.6: Required freeboard from the calculated backwater to the deck's soffit of bridges and culverts

The design flood, Q_T, and the freeboard requirements discussed above should be regarded as minimum standards. Indeed the bridge designer should always investigate the impacts of debris, standing wave action, and super-elevation of the flow on the freeboard to be provided. Additional freeboard shall be provided where sediment build-up is likely, e.g. if the structure is positioned upstream of an existing or a planned dam. Such build-up often extends above the full supply level of the dam.

In certain marginal cases sensitivity analysis as to assumed roughness and slope should be considered as changing these parameters may significantly affect flood levels. In addition where flow is verging on supercritical flow (Froude Number Fr > 0.8) and flow conditions may be unstable, care must be exercised. For bridges where approach flows are supercritical the bridge designer should aim to minimise the obstructions in the main stream flow. For this reason it is often ill-advised to place a pier in the main channel of the river or stream. It is always advisable to span the main channel. Generally the bridge designer should consider the natural stream flow without the road or bridge, as a guide on how to proceed with the aid of physical or numerical modelling of the flow patterns, as required.

Waterway Requirements

Bridge Hydraulics

The hydraulic capacity of bridges can be determined from the principles applicable to the conservation of energy, mass and momentum. Reference 3.1 provides a review of the methods of analysis of flow in natural channels (normal flow) as well as the basis for analysing the effects of bridges and embankments on flow in watercourses, which impede the natural flow. Whilst manual methods may suffice for bridge preliminary sizing, it is recommended that the well-known HEC-RAS software package, Reference 3.2, be employed to confirm the adequacy of the hydraulic capacity of river bridge proposals.

For adequate hydraulic analysis it is necessary to obtain accurately surveyed river cross sections at appropriate intervals upstream and downstream from the bridge crossing. In addition, an assessment of the Manning's roughness of each part of each cross section is required. This data should be acquired as part of the bridge site topographical survey which is outlined in Section 2.4 of Chapter 2 of this volume. The bridge designer should refer to the HEC-RAS Application Guide in order to define the location and extent of the river cross sections required as part of the survey, to be used in subsequent hydraulic analyses.

Backwater at bridge constrictions

For the purpose of establishing the required waterway opening it is necessary to first analyse the normal depth and width of flow in the river channel for the design flood Q_T. <u>The normal depth plus the required freeboard is the minimum permissible elevation of the bridge deck soffit</u>.

From this analysis it is necessary to assess whether normal flow at the bridge site is either subcritical ($F_r < 1.0$) or supercritical ($F_r > 1.0$) by determining the Froude number applicable to the flow from

Froude number: $F_r = \frac{(Q_T^2 . B_n)^{1/2}}{g . A_n^3}$ ______________________Equation

Error! No text of specified style in document.-2

Where: Q_T = design discharge (m^3/s)

B_n = total top width of normal flow (m)

A_n = total flow area for normal stage (m^2)

g = gravitational acceleration (m/s^2)

A bridge often reduces the available cross-sectional flow area of a stream, which leads to additional energy losses and causes upstream damming of subcritical flow (backwater). A typical subcritical flow pattern through a square crossing is shown in Figure **Error! No text of specified style in document.**.7. When the flow through the structure is supercritical the depth of flow increases where the cross section narrows, usually to a greater extent than in the case of subcritical flow.

Figure **Error! No text of specified style in document.**.7: Water levels and flow distribution at square crossings

When subcritical flow is maintained beneath the bridge the water surface does not drop below the critical depth of flow. Conversely in the case of supercritical flow through the bridge opening, the water surface dips below critical flow depth beneath the structure, which results in a hydraulic jump immediately downstream from the structure. Scour of the river bed is therefore likely to be more severe in the event of supercritical flow than for subcritical flow, particularly in the vicinity of piers.

Bridge waterway opening

It is recommended that the following criteria be applied:

Backwater just upstream from river bridges should generally not be more than 0.6m (Refer to Figure **Error! No text of specified style in document.**.5).

The design flow velocity through the constriction should generally be less than 4m/s.

The design flow velocity through the bridge opening should not be more than 1.67 times the normal (unobstructed) flow velocity, in order to limit the scour potential beneath and downstream from the bridge.

When determining the width of the waterway opening provided by river bridges, in the case of supercritical normal flow it will usually be preferable to ensure that the bridge abutments are located close to the limits of the normal flow top width, i.e. minimum constriction. In the case of subcritical normal flow it will often be found that a reasonable degree of constriction of the normal flow width will yield a safe yet economical design (bridge + road) without infringing the foregoing hydraulic design criteria.

In all cases the minimum permissible elevation of the bridge deck soffit must be arranged to ensure that normal flow depth plus backwater plus required freeboard is not compromised. For final design purposes the flow type and its profile should be confirmed by HEC-RAS or similar analysis.

The same analytical procedure must be applied to the check flood with a peak flow rate of Q_{2T}, to ensure that the total depth of flow at a bridge site does not cause backwater to the extent that the shoulder breakpoint is overtopped (refer Figure **Error! No text of specified style in document.**.5).

For more detailed guidance on this subject the bridge designer should study References 3.1, 3.2 and 3.3.

Culvert Hydraulics

The hydraulic performance of culverts is governed by the same principles as those applicable to bridges. References 3.1 and 3.4 provide detailed guidance about the hydraulic design of these structures and enable the designer to adopt designs based on a wide range of cross-sectional configurations. Modern culvert construction is based principally on precast or in situ reinforced concrete, as well as on corrugated galvanised tubular or plated metal sections.

The hydraulic capacity of culverts is predominantly governed by the conditions immediately upstream from the structures (inlet control), or at the downstream end (outlet control). Special culvert designs with improved inlet configurations may be governed by face control or throat control. Compared with inlet control, culverts operating under outlet control are usually found to occur in a small proportion of culvert projects in very flat terrain, when backwater from downstream obstructions or other conditions causes submergence or partial submergence at the outlet, and thereby reduces the discharge capacity of the structure.

Culvert performance curves

Reference 3.4 provides guidance on the development of performance curves for culverts with conventional cross sections (circular, oval, square, rectangular etc.) and particular wingwall, headwall and other inlet configurations, for both inlet and outlet control.

Performance curves portray the relationship between the headwater depth HW at the culvert entrance (or the ratio of headwater depth to the culvert internal height, HW/D) against the discharge capacity of the particular culvert size/shape, operating under inlet control. Similar curves can likewise be developed for culverts operating under outlet control, transitioning for greater headwater depths to inlet control.

Culvert size determination

Culvert performance curves enable the designer to very quickly determine the adequacy of alternative culverts to convey particular flood peaks Q_T or Q_{2T}, without infringing the applicable headwater depth (freeboard) limit for either case, as illustrated in Figure **Error! No text of specified style in document.**.8.

Figure **Error! No text of specified style in document.**.8: Culvert Size Determination

Notation

Q_T	:	Design Flood
Q_{2T}	:	Check Flood (shoulder break point limit)
$HW_{D+F/B}$	:	Permissible headwater depth = Culvert internal height - freeboard limit
HW_{SBP}	:	Permissible headwater depth = Height from inlet invert level to SBP level

Culvert Storage Routing

Storage at the inlet of a culvert is dependent on the depth of the headwater against the road embankment and the natural topography upstream from the structure. At most culvert sites the volume of water temporarily stored is a small proportion of the flood total runoff volume. However, in flat terrain storage may constitute a significant percentage of the runoff volume (15% or more is deemed significant). In these cases the temporary detention of a portion of the runoff volume has the effect of reducing the peak flow through the culvert compared with that arriving at the inlet. This is known as attenuation of the peak flow, which is illustrated in Figure 3.7

Flood peak attenuation is therefore related to the flood volume and not just the peak inflow rate. The storage curve needed for culvert storage routing reflects the increase of the stored volume with increased headwater depth, and is developed from a topographical survey plan of the flooded area.

In Figure 3.7 the inflow and outflow hydrographs are depicted as idealised triangular forms, merely for illustrative purposes. The inflow hydrograph is shown to peak at the Time of Concentration T_C, consistent with the assumptions of the Rational Method of flood peak estimation. The outflow hydrograph peak coincides with the falling limb of the inflow hydrograph and is translated by a time interval in relation to the inflow peak. The characteristic performance curve of the culvert reflects the flow rates discharged through the structure for a range of headwater depths, without the effects of storage. Such performance curves may be developed from the equations provided in References 3.1 and 3.4 for culverts of particular sizes and configurations. When storage is minimal, the required culvert size must be established from the characteristic performance curves of a range of culverts, of the preferred type, to ensure that the permissible headwater depths for peak flows of Q_T and Q_{2T} are not exceeded. Conversely, when storage is significant, for the same culvert size the peak flow through the structure will be attenuated and the maximum headwater depth will be reduced. This indicates that a smaller culvert size could be used to discharge the flood, without exceeding the permissible headwater depths for Q_T and Q_{2T}.

Figure **Error! No text of specified style in document.**.9: Flood Peak

References 3.1 and 3.4 describe the Storage-Indicator numerical flood routing procedure needed to determine the attenuated outflow peak and headwater depths for culverts with significant storage potential. Reference 3.1 also provides a computer software facility for this purpose. When the potential cost saving arising from culvert size reduction is small, bridge designers are advised to retain the original culvert size and treat the additional flow capacity afforded by storage as increased assurance against the uncertainty of flood peak estimation and the potential risks of sedimentation and debris blockage.

However, when significant cost savings are possible, in the case of larger culverts under very high embankments for example, downsizing on the grounds of storage could be considered provided the risks of such a step are carefully assessed and the step is approved by the FMW. Special care always needs to be exercised to ensure that the increased duration of ponding against a road embankment due to storage does not unduly increase the risk of culvert failure due to piping, especially in the case of corrugated metal culverts.

Other Hydraulic Considerations

Other important factors which must be assessed and guarded against in the hydraulic design of bridges and culverts include:

Sedimentation

Debris blockage

Scour

Piping and seepage

Overtopping of bridges and roadways

Hydraulic forces on bridge elements

(These forces are discussed in Section 3.5)

Sediment transport

Safe bridge design includes the recognition that river channels are not stationary, but that they may adjust their bed and banks during the life of the bridge. Factors that influence sediment transport include sediment properties, hydrology, watershed and land-use conditions, channel geometry and vegetation. Channel stability and sediment transport are complex processes that interact to produce the existing channel form and future channel adjustments.

These issues are beyond the scope of this Volume V. The bridge designer is advised to study Reference 3.3 in this context and in complex circumstances additionally to refer to the specialist literature quoted therein.

Debris blockage

When significant debris is anticipated from heavily wooded catchments a minimum freeboard of 0.3m should be provided above the estimated maximum backwater depth for all bridges and culverts. When large debris is anticipated, the recommended bridge minimum span length should be at least 7.5m for all roads but preferably 15m for Class 1 to Class 3 roads where the 20 year Indicator flood is 150m^3/s or greater.

Scour

Scour is a major mechanism responsible for the failure or partial failure of bridges and, to a lesser extent, culverts during flood events. This is a complex problem because of the number and variability of the factors giving rise to the various types of scour, and which include:

Total scour

Long-term scour

Short-term scour

Contraction scour

Local scour

References 3.1, 3.3, 3.4 and 3.5 provide detailed guidance on the identification of scour potential, analysis of the magnitude of scour and the design of countermeasures. It needs to be remembered that the added cost of making a bridge less vulnerable to damage from scour is small in comparison with the total cost of a bridge failure and replacement.

The bridge designer is advised to study the quoted references for guidance on analysis of the effects of scour and the design of the necessary countermeasures, which are beyond the scope of Volume V.

Piping and seepage

Piping is a phenomenon that begins with seepage along the outside of a culvert barrel, which progressively removes embankment fill material, forming a hollow similar to a pipe, hence the term piping. Fine soil particles are washed out freely along the hollow, and the erosion inside the fill may ultimately cause failure of the culvert and/or the embankment.

Piping may also result from the exfiltration of flow through open joints in the culvert barrel, which can be avoided by ensuring that these are additionally covered by strong flexible membranes on the outside of the barrel or otherwise prevented from opening, especially at the barrel invert.

Piping should be anticipated along the entire length of the culvert when ponding above the top of the structure is permitted and likely to prevail for a period, especially in the case of fine cohesionless material used as backfill or embankment. Countermeasures against piping include the installation of concrete apron slabs with cut-offs at the entrance and the installation of anti-seep or cut-off collars along the length of the barrel to increase the length of the flow path, decrease the hydraulic gradient and the velocity of flow, and thus the probability of pipe formation.

Further information on this subject is available from Reference 3.4.

Overtopping of bridges and roadways

In certain circumstances the FMW may permit the design and installation of low-level river crossings on low class roads which carry limited traffic volumes. A low-level river crossing is a submersible road structure designed in such a way as to experience no or limited damage when overtopped. This type of structure may be appropriate when the inundation of a road is deemed acceptable for a short period and significant cost savings can be achieved in comparison with the provision of a high level structure.

On higher order roads (Classes A to C) high-level bridges are generally required because of the unacceptability of disruptions due to flooding. However, even in the case of high-level bridges partial submergence of the deck may occur in the event of the check floods with a return period of 2T years, for which the maximum flood depth is permitted to reach the level of the shoulder break point. In the event of exceptional floods even these bridges may be fully submerged for a short period.

The bridge engineer is advised to study the principles outlined in References 3.1 and 3.3 for guidance on the analysis and design of bridges and roadways which are overtopped by flood flows.

Hydraulic Forces on River Bridges

Reference 3.3 provides detailed guidance on the evaluation of hydraulic forces on bridge elements, which include the following:

Hydrostatic forces

Buoyancy forces

Stream pressure and lift

Wave forces

Effects of debris

Effects of ice (not applicable in Nigeria)

Vessel collision (navigable rivers)

Backwater effects on bridge piers

Whereas scour is not classed as a load as it is caused by the combined effects of general and local erosion around the bridge foundations, it can result in amplification in the load effects on the foundations by removal of the lateral support to considerable depths around pile and caisson support in particular, which leave the pile or caisson caps exposed at a higher level than often assumed in design. Such effects should be conservatively analysed in determining the combined load effects on foundations.

In addition to hydrostatic forces imposed on bridge components, hydrodynamic forces on bridge piers, abutments and sometimes on bridge decks arise from the velocity of flood flows. The magnitude of these forces can be significantly amplified by debris impacts and when debris is entrapped at the deck soffit or against the parapets, especially in the case of highly afforested catchments. Reference 2.2 (Chapter 2) provides guidance on the estimation of these forces and lists further references for more detailed information.

When a bridge is designed to accommodate floods with a return period less than 50 years, the bridge designer is strongly advised to also check on the influence of the 50 year flood in terms of potentially increased hydrodynamic and debris impact forces as well as increased debris entrapment potential.

References

3.1 The South African National Roads Agency Limited, DRAINAGE MANUAL (6th Edition - Fully Revised), Pretoria 2013.

ISBN 978-0-620-55428-2 (Drainage Manual)

ISBN 978-0-620-55429-9 (Drainage Manual Application Guide – only available electronically)

3.2 US Army Corps of Engineers, Hydrologic Engineering Centre, HEC-RAS River Analysis System - Applications Guide, Version 4.1 (January 2012), Davis CA.

Unclassified.

3.3 US Department of Transportation, Federal Highway Administration, HYDRAULIC DESIGN OF SAFE BRIDGES, Publication No FHWA-HIF-12-018, (April 2012), Hydraulic Design Series No. 7, Arlington VA.

Unclassified.

3.4 US Department of Transportation, Federal Highway Administration, HYDRAULIC DESIGN OF HIGHWAY CULVERTS (Third Edition), Publication No FHWA-HIF-12-026, (April 2012), Hydraulic Design Series No 5, Arlington VA.

Unclassified.

3.5 US Department of Transportation, Federal Highway Administration, HYDRAULIC DESIGN OF ENERGY DISSIPATORS FOR CULVERTS AND CHANNELS (Third Edition), Publication No FHWA-NH1-06-086 (July 2006), Hydraulic Engineering Circular No 14, Arlington VA.

Unclassified.

Bridge Systems and Components

Introduction

Scope

This chapter briefly describes the purpose and function of bridges and schedules the various categories of these structures in terms of their types, components, materials, degree of articulation, method of construction and so on. These schedules provide a useful aide mémoire to bridge designers about the various options available to them, which are aimed particularly for use at the planning and concept stages of bridge design.

It is also intended to remind designers that the design and housing of the accessories needed in modern bridge engineering require early attention with regard to component sizing in order to avoid difficulties during the subsequent stages of bridge detailing. These reminders are again reinforced by Chapter 8 of this Volume.

Purpose and function of bridges

Simply stated: the primary purpose of bridges is to provide safe passage over obstacles.

Since historical times this objective has arisen from the needs of convenience, time and safety in the conveyance of people, their goods and possessions from one place to another. The subsequent development of civilisations has amplified these needs many fold as a result of population growth and concentration, the development of sophisticated modes of travel, and numerous other factors, all of which have led to urban congestion and the need for the separation of intersecting transport routes.

Categorisation of bridges

The circumstances discussed above have given rise to the need for bridges which may be categorised firstly by their function and loads to be carried, and secondly by the obstacles to be crossed:

Bridges categorised by their function and loading

1. Traffic

1. Road (various classes and traffic loads)

2. Rail (light; passenger; freight)

3. Pedestrian

4. Livestock and agricultural

5. Military and emergency (usually demountable)

Services

1. Pipelines and aqueducts

2. Conveyor systems

Combinations

1. Dual or multi-purpose

   (The loads to be carried and the forces to be resisted by bridges are summarised in Chapter 2).

Bridges categorised by the obstacles to be crossed

1. Natural

1. Plains

2. Valleys

3. Gorges/canyons

4. Rivers, streams and wetlands

5. Lakes and lagoons

6. Inlets and seaways

Man Made

1. Roads
2. Railways
3. Urban areas (various types and densities)
4. Airport runways
5. Canals
6. Dams
7. Harbours

Bridges may be further categorised by their type (primary and secondary structural systems), articulation, materials and methods of construction, each of which are briefly outlined in the following sections of this chapter.

Other highway structures

Pedestrian and livestock culverts (subways), and drainage and service conduits fall into the category of cellular structures beneath road and rail embankments and are briefly described in this chapter under the heading of cellular structures. However, ground mounted and overhead sign supports or gantries are presently excluded as beyond the scope of this chapter.

Bridge Types, Components, Articulation, Materials and Methods of Construction

Bridges categorised by their type

Bridges are broadly categorised as follows by their type, which essentially defines the manner in which these structures carry the imposed loads and self-weight to the supports and ultimately to the foundations.

Slab type bridges

Girder type bridges

Trussed girder bridges

Framed bridges

Arched bridges

Cable supported bridges

Cellular structures

Each of these bridge types may be configured in various ways, both in elevation and cross section. The more common types of small to medium span concrete bridges are the particular focus of this Volume and are further described in Section 4.3.

Bridge Components

With the exception of cellular structures and some of the special types, all bridges comprise the following main components:

Deck

Abutments

Piers (except in the case of single span bridges; pylons in the case of cable supported bridges)

Foundations

Accessories

All of the parts of the bridge above the bearings are referred to as the superstructure, while the substructure comprises all parts below the bearings. The main load carrying portion of the superstructure is conventionally termed the deck, which can be formed in numerous ways and spans longitudinally between the supports, as well as transversely. The numerous lesser components which form part of either the superstructure or the substructure are collectively termed bridge accessories.

Bridges categorised by articulation

Articulation arises from joints or hinges in or between structural members where bending moments cannot be transferred across the joint or hinge. The degree to which structures are statically determinate or indeterminate arises from the location or absence of the joints or hinges.

1. Statically determinate bridge decks and arches include:

1. Simply supported bridge decks: joints or hinges between adjacent spans at the supports, and separated by bearings from the supports.

2. Bridge decks with free cantilevers : joints or hinges in the decks but not at the supports, and separated by bearings from the supports i.e. spans which one or two cantilevers within the span and simply supported 'drop-in' portions between the joints.

3. Three hinged (or pinned) arches.

   In these structures the bending moments and shears may be determined at all sections entirely by consideration of the external forces acting, unaffected by variations in the section sizes.

Statically indeterminate bridges, described in terms of increasing degrees of indeterminacy

1. Continuous bridge decks (more than one span): decks separated from the supports by bearings and usually with vertical joints between the deck and the abutments only.

2. Portal frames and arches which have hinges (or other forms of articulation) only at the foundations or springing points.

3. Fixed structures with no articulation joints or hinges, such as: portal frames; cellular structures; fully integral bridges; fixed arches.

   In these structures the bending moments and shears cannot be determined by simple statical methods, and must be determined by analyses related to the relative stiffness and lengths of the adjoining members, with due regard to deflections and to the compatibility of rotations which occur at the supports or points of junction, hence the term statically indeterminate.

The question of fixity

The assumption of the achievement of full fixity should be carefully checked, especially when such fixity is reliant on soil-structure interaction, which is dependent in turn on the stiffness (or subgrade reaction) provided by the supporting ground. An example of such a case is the degree of fixity achieved at the bases of tall piers which are subjected to horizontal forces (braking or traction caused by traffic) imposed on bridge decks, and restrained deformations arising from temperature changes, shrinkage, creep and prestressing of the deck.

In these instances the restraint provided by piers to horizontal movements at deck level is directly related to the stiffness of the piers, which in turn is a function of whether the piers have cracked or uncracked reinforced concrete sections and the degree of fixity at the bases.

The case against articulation

As a general rule it is preferable to articulate bridge decks to the least degree possible. To increase the number of deck joints because of an unsubstantiated concern about differential settlement to the point that the advantages of deck continuity is lost, should be rejected. The best detailed expansion joints will almost invariably be inferior to continuous superstructures even in the event of moderate differential settlement.

The structural reserves of continuous bridge decks are lost by the adoption of statically determinate multi-span deck systems. However, construction of new bridges over heavily trafficked road or rail routes may dictate the adoption of simply supported spans comprising precast beam and in situ slab construction for the initial dead load condition. In these circumstances consideration should be given to the elimination of roadway joints by the introduction of 'flex slabs' which tie the spans together or by establishing continuity for live load effects as illustrated later in this chapter.

In numerous countries around the world there has been a discernible trend over the past 40 to 50 years toward the construction of fully integral bridges, in which the deck is monolithic with all supports, and roadway joints and support bearings are eliminated entirely. The total length of such structures is commonly 60 m or greater and life cycle costs to date are reported to indicate meaningful savings.

The main advantage of the elimination of joints and bearings, especially at piers, arises from the avoidance of the future cost and difficulty of replacing these components in circumstances of greatly increased traffic volumes in many instances. In the case of continuous in situ concrete decks, consideration should also be given to the installation of concrete hinges in place of bearings when the technical circumstances permit. Concrete hinges have a sound record of long term durability and offer the advantage of significant cost savings when compared with large steel bearings in particular.

Categorisation by materials

Bridges may be categorised by the materials of construction which have included the following since bridges were first built:

Concrete

Plain (mass concrete)

Reinforced (RC)

Prestressed (PSC)

Composite (RC + RC or PSC + RC)

Structural steel (SS)

Normal (various grades)

Weathering (various grades)

Stainless (various grades)

Composite (SS + RC or PSC)

Other

Timber

Stone

Brickwork and Blockwork

Aluminium

Carbon Fibre Reinforced Polymer strengthening

In Nigeria the use of structural steel is not favoured in highway bridge construction. The other materials listed are generally not frequently used in modern bridge construction. The use of structural steel and other materials are therefore not further discussed in this Volume, except in so far as some of these materials are commonly used in bearings and joints.

1. Concrete

In modern Nigerian highway structures the most frequently used materials for the major substructures and superstructure components are as follows:

Substructures: in situ RC components

Decks

either RC or PSC precast beams with in situ RC slabs

either RC or PSC for fully in situ work

In the interests of structural efficiency, economy and durability the minimum characteristic cube strength should be 30MPa for substructure components and preferably 40MPa or higher for superstructure components, especially in the case of prestressting. However, the development of concrete with very high strengths or 60MPa and greater are in the process of development in various countries around the world, the relative advantages or disadvantages of which should be investigated for future use in Nigeria for bridge works.

The most obvious threat to the long term durability of concrete bridges is that of the corrosion of the reinforcing steel arising from the deficiency or deterioration in the concrete cover. The bridge designer is advised to guard against these threats by the provision of minimum covercrete of 40mm (or more as specified in BS5400 for particular circumstances), together with careful detailing of reinforcement to prevent the loss of concrete cover. This must be followed up as further discussed in Chapter 8 to ensure the adequate compaction and curing of concrete during construction.

Reinforcing and Prestressing Steel

Reinforcing steel and prestressing steels are required to have the characteristic strength, ductility, bond characteristics, metallurgical composition and other qualities as required by BS5400 and assumed in the design. Special care needs to be exercised through adequate laboratory testing that all of the required criteria are fully met when these reinforcing steel bars or steel wire and strand for prestressing are imported from overseas.

Bridges categorised by the method of construction

Bridges are sometimes categorised by the method of construction, especially in the case of some of the more recently developed methods for the construction of bridge decks. The following is a list of modern construction methods.

Static falsework and formwork

applicable to in situ concrete work

Sliding and climbing forms

applicable to the in situ concrete construction of piers greater than about 30m in height.

Crane erection

applicable to erection of precast components such as beams, segments of decks, other precast components, or to the hoisting of concrete for in situ work.

Launching trusses, girders or carriages

similar to crane erection but usually applicable to long bridges with multiple repetitions.

Travelling forms, under or over

particularly applicable to the in situ construction of long arch bridges, often in combination with cable stays from temporary towers.

Balanced cantilever

applicable to in situ or precast construction of long span variable depth decks, advancing equally outwards from each pier and sometimes in combination with cable stays or additional temporary intermediate piers.

Transversely jacked structures

mostly applicable to the installation beneath an existing railway line by transverse jacking of an entire bridge above support level, during a very limited period of closure of the line (occupation over 2 to 3 days maximum).

Incrementally launched bridge decks

involving the prior construction of the bridge piers across the obstacle to be crossed, and the sequential construction of bridge deck segments attached by prestressing to previous segments, constructed initially behind one abutment and either jacked or pulled incrementally over the pier supports behind a leading steel girder system (nose) until the structure reaches the far abutment.

Cable supported systems

comprising suspension bridges, cable stayed bridges, or the more recently developed stress ribbon bridges.

The image cannot currently be displayed.

Figure **Error! No text of specified style in document.**.10: Common Structural Forms Suitable For Small to Medium Span Concrete Bridges

Bridge Superstructures

General

The configuration of bridge decks can be defined by their primary and secondary systems which function in different ways.

The primary system is required to carry all loads longitudinally to the supports and its form is mostly readily visible in elevation and often used to describe the type of bridge e.g. girder bridge, truss, arch and so on. The secondary system is the configuration of the cross section, the function of which is to distribute the imposed traffic or other deck loads transversely to the primary system.

With the exception of the cellular structural forms, which are usually small and used primarily as buried conduits beneath road embankments (culverts, subways etc.), the other bridge types recorded in Section 4.2.1 are listed approximately in relation to the increasing span ranges they are intended to serve.

Frequently used structural systems for small to medium span concrete bridges

A comprehensive list of bridge types is contained in Appendix 4A for reference. Many of these types are best suited to structural steel construction, or are otherwise deemed to be beyond the scope of this Manual and are not further discussed in the following sections. These include trussed girder bridges, arched bridges, cable supported bridges and girder type bridges constructed by incrementally launched or balanced cantilever methods. Reference should be made to specialist literature for information about the design of the bridge types excluded from this Volume.

Details of some of the bridge forms likely to be considered for small to medium span bridge projects are depicted in Figure **Error! No text of specified style in document.**.10 and Figure **Error! No text of specified style in document.**.11, the superstructures of which are discussed in this section.

1. Solid slab decks

Solid slab decks comprise a solid concrete slab section, without beams or voids, sometimes with thinner edge cantilevers for aesthetic reasons. This type of construction is comparatively versatile and is commonly used in the construction of short span in situ concrete bridges and culverts.

Reinforced concrete slab construction is particularly suitable for:

Conventional simply supported or continuous bridge decks, usually of uniform depth but with support haunches for longer spans.

Simple open frame portal bridges, over subsidiary roads for example.

Closed frame cellular culverts and subways under embankments.

Variable thickness prestressed concrete solid slab decks acting as part of strut frame type bridges can be economical for spans of up to about 35m.

The construction of solid slab decks, or portal and closed cell variants, is usually straightforward and uncomplicated, and the formwork is simple to construct. Reinforcement layouts seldom result in congested areas, and the placement of concrete should present no difficulties. Concrete volumes may be large.

Voided slab decks

Voided slabs are characterised by the presence of voids within the slabs, their function being to reduce the volume of concrete and thereby the self-weight required to be carried by the slab. The voids are generally cylindrically shaped, and constructed using either high density polystyrene formers or thin walled steel sections placed within the slab.

Voided slab decks are usually economical for total deck depths ranging from 900 mm to about 1500mm. Minimum slab thicknesses should be 180mm above the voids and 150mm below the voids, with concrete ribs between the voids of not less than 250mm (reinforced concrete) and 350mm (prestressed concrete). Voided slabs are usually not economical for deck depths of less than 900mm as the weight saving below that limit is relatively small.

In situ reinforced concrete voided slab decks are commonly used for continuous deck spans of:

Up to 25m for uniform depth sections.

Up to about 32m for variable depth sections.

For spans above 32m and up to about 45m variable depth voided slab decks are invariably prestressed.

Prestressed concrete voided slab decks with comparatively wide cantilevers are also suitable for twin spine beam type decks depicted in Figure **Error! No text of specified style in document.**.11.

Whereas the falsework and formwork required for voided slab decks is as simple as that for solid slab decks, this form of construction is rendered more difficult by the need to resist the significant uplift force on the voids during the concreting of these decks. The precautionary measures required for this form of construction are outlined in Chapter 8.

This image cannot currently be displayed.

Figure **Error! No text of specified style in document.**.11: Typical Deck Cross Sections for Small to Medium Span Bridges

Cast in situ beam and slab decks

Cast in situ beam and slab decks comprise deck slabs supported by cast in situ beams. The beams provide the strength and stiffness required for the span, and enable the slab to be relatively slender and economical to construct. The number and spacing of the beams utilised is dependent upon several factors, such as the width of the deck and the slenderness of the beams.

Formwork for cast in situ beam and slab decks is more complicated than that required for solid or voided slab decks. This form of deck construction is usually suitable for the same range of spans as the solid and void slab deck options, depending on the adoption of uniform or varying depth decks and the use of reinforcement or prestressing in the longitudinal direction.

These decks are sometimes designed with in span cross beams to improve the transverse distribution of live loads.

Precast beam and slab decks

Precast beam and slab decks are similar to cast in situ beam and slab decks, but utilise precast beams to support the deck slabs. During concreting of the deck slab, the formwork is usually supported by the precast beams, requiring that the precast section alone be designed to carry the combined weight of both beam and slab.

Occasionally, designs may require that the beams be temporarily propped until the slabs have been cast and gained strength. Under these conditions the self-weight of the slab only acts on the structure when the props are removed, and is therefore carried by the composite (combined) beam and slab section.

The formwork between the beams to support the slab during casting operations often comprises permanent fibre-cement boards or ribbed precast concrete planks. The strength and deflection characteristics of such permanent forms should be confirmed by testing in relation to the intended clear gap between the precast beam top flanges, which is usually not more than about 1.2m to 1.4m. Testing should allow for the weight of workmen in addition to the self-weight of the wet slab. Reliance should not be placed on fibre-cement boards to provide part of the cover to the slab bottom reinforcement.

In Europe and elsewhere various configurations of precast beams have been developed by specialist suppliers of these components, and among others include T-beams, I-beams, U-beam and M-beams (inverted T-beams) for example, which are illustrated in Figure **Error! No text of specified style in document.**.12. The suppliers provide design tables of the section properties of these beams, together with the intended span range for the beams, which usually varies from 10 m to a maximum of about 35m.

Figure **Error! No text of specified style in document.**.12: Examples of Precast Beams for Composite Beam and Slab Decks

With precast beams two types of deck are possible:

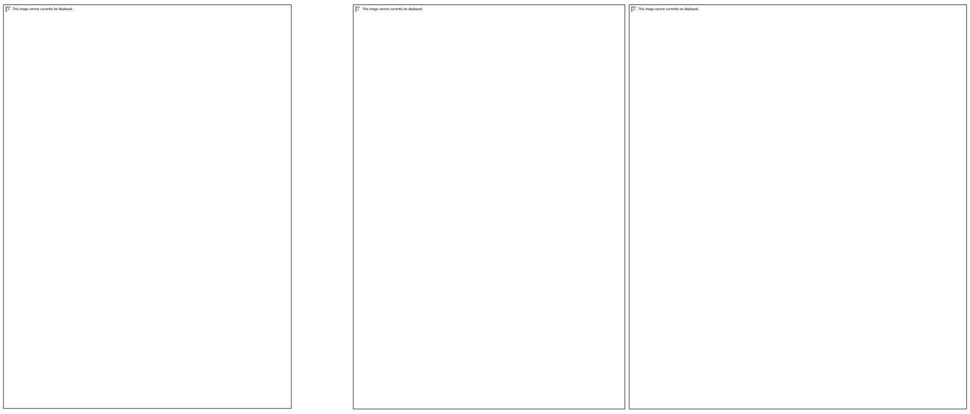
Small span bridges: 10m to 25m where the beams are normally pretensioned and of standard cross section dimensions.

Medium span bridges: 25m to 50m where the beams are normally post tensioned and are purpose made, with wider webs to accommodate the draped prestressing cables.

It is economical to design bridges with non-standard precast beams only when a large number of these components are required, usually in excess of 100. It also needs to be remembered that beams for medium span bridges are very heavy - of the order of 130 tonnes for 50m long beams for example - which require large cranes for erection purposes. The need to check on the availability of reasonable access and working space for beams of this size and plant of this magnitude is therefore self-evident.

In span cross beams should be avoided, especially in skew bridges, because of the difficulty and cost of the shuttering. Cross beams are invariably required at the supports, as part of the secondary structural system, and to provide space for the housing of the deck expansion joints.

At the pier supports of precast beam and slab decks, three articulation arrangements are feasible as illustrated in Figure **Error! No text of specified style in document.**.13:



(a) (b) (c)

Figure **Error! No text of specified style in document.**.13: Alternative Treatment of Joints at Supports of Precast Beam

1. Figure **Error! No text of specified style in document.**.13 (a):

Simply supported spans with provision for an expansion joint.

Figure **Error! No text of specified style in document.**.13 (b):

Expansion joint eliminated and replaced by a thin 'flex slab' which is heavily reinforced longitudinally to tie the spans together. This portion of the deck slab is usually cast about a month after the remainder of the slab, when part of the shrinkage of the in situ slab has dissipated. As the top surface of 'flex' slabs are prone to transversely cracking these should be protected by an adequate waterproofing membrane beneath the asphalt surfacing.

Figure **Error! No text of specified style in document.**.13 (c):

Deck simply supported for dead loads and made continuous for subsequent live loads. Bridge designers are advised to consult specialist literature for guidance on the means to achieve continuity.

Construction using M-beams results in the formation of multi-celled pseudo box deck structures. This form of construction with contiguous bottom flanges between the beams is particularly advantageous over electrified railway lines as it essentially provides a safe platform from the outset, except at the outer edges, where falsework and formwork must be suspended from the deck to permit safe construction of the edge cantilevers and the bridge parapets.

Concrete box girder decks

Concrete box girder decks utilise a section comprising one or more hollow boxes (usually rectangular or trapezoidal in shape). In its most common form, the deck has a single box (refer Figure **Error! No text of specified style in document.**.11) with comparatively wide cantilever slabs, but for wider bridges (width > about 13m), the configuration is extended to comprise a box with two and sometimes three contiguous cells. In the case of very wide bridge decks a further option is often selected, comprising twin cell or twin spine beam configurations with a wide slab between the separate cells or spine beams. This form is briefly outlined in subsection (f) below.

Box girder type decks have high torsional stiffness, and the presence of both top and bottom slabs provide capacity for both sagging and hogging bending moments. When constructing cast in situ concrete box girder decks, the lower slab and the webs are usually cast in a continuous operation, followed by casting the top slab a few days later.

Box girder bridges are predominantly continuous prestressed concrete, sometimes reinforced concrete for shorter spans. This form of deck construction is highly versatile and suitable for:

Conventional bridge decks with short to medium spans from 25m to 50m when using uniform depth decks cast in situ on stationary falsework and formwork, or alternatively constructed by the incrementally launched method.

Medium to long spans from 70m to 180m when using variable depth decks, either with precast concrete or in situ concrete segments, constructed by the balanced cantilever method.

Short to medium span portal frame or strut frame bridges.

Bridges with horizontal curvature.

Together with precast beam and slab deck forms, box girder construction is evidently the most frequently used bridge form for modern concrete bridge construction worldwide. Bridge designers who require detailed information about the configuration and design of box girder type bridges should consult references 4.1, 4.2 and 4.3 listed at the end of this chapter.

Twin spine beam decks

Twin spine beam decks utilise two spine beams to carry the deck slab and can be constructed from either reinforced or prestressed concrete. The spine beams commonly comprise solid concrete sections, but often utilise void formers or hollow boxes to reduce their cross-sectional area and increase their efficiency.

This form of deck construction is normally applicable to spans up to about 45m, and is useful in that the two halves of the deck can be constructed at different stages with a gap of about 1m to 1.5m in the central slab, which is cast one to two months after the second half of the deck to reduce longitudinal shrinkage and creep. This arrangement is well suited to projects which require the replacement of an existing bridge with a wider and longer new structure at the same location.

General considerations for superstructures

The importance for advance planning for the construction of bridge superstructures cannot be over-emphasised at the design stage.

Factors such as the precamber of all concrete bridge decks and the effects of elastic supports on prestressed concrete members must be given due consideration by the bridge designer and not left to the judgement of supervision personnel during construction.

1. Precamber

Reinforced concrete members usually require upward precamber to counteract the effects of elastic and creep deflections. Prestressed concrete members usually require downward precamber to provide for upward elastic and creep deflections. It is the bridge designer's responsibility to make an assessment of the necessary precamber of bridge decks and indicate these requirements on the drawings.

Effect of elastic support on prestressed members

In general, when prestressing is applied to members carried by rigid supports, the members lift off the supporting formwork when the prestress loading is applied. When this occurs the weight of the concrete is immediately transferred from the formwork onto the member itself, counteracting the upward forces imposed by the applied prestressing. This behaviour enables member stresses to be contained within acceptable limits.

However, when members are cast on formwork supported by very tall scaffolding towers or flexible beams/girders, the elasticity of the support influences the above behaviour, and the weight of the concrete may be only partially transferred onto the prestressed member. Unless this behaviour is properly evaluated and accounted for, tensile stresses at the top of the member may exceed allowable values causing cracking of the member. It is therefore essential that monitoring staff are alert to the potential for such situations and consult with their design office regarding the necessary prior evaluations.

Bridge Substructures

Substructure

The function of the bridge substructure is to support the superstructure at the required levels and safely convey the dead and live loads from the superstructure through the foundations to the ground and to provide stability against all actions on the structure. Reference 4.4 provides guidance about substructures.

Abutments and Wingwalls

The particular function of abutments is to support the ends of bridges and to provide smooth transitions between the superstructures and the approach embankments retained behind the abutments. Approach slabs are therefore always recommended behind abutment breast walls to reduce the effects of embankment settlement on the quality of rideability.

The selection of the type of abutment is often primarily dependent on whether a closed or an open form is preferred. Closed abutments are usually required for river bridges, but open forms are often preferred for bridges over major highways for example, in the interest of greater visibility beneath and beyond the structure (transparency). Such a choice also has an important influence on the aesthetics of bridges.

Following selection of the preferred form (open or closed), the configuration of the abutment and the associated wingwalls is dependent on the following factors:

Overall road geometry

Height from top of abutment to founding level

Height of retained embankment

Nature of retained material and the associated earth and surcharge pressures

Bearing capacity and other characteristics of the ground at and below the founding level

The types of structures commonly used for bridge abutments, wingwalls and independent retaining walls and other earth retaining systems are depicted in Figure **Error! No text of specified style in document.**.14, in which the following are shown:

1. Open abutment forms

Type 1: Bank seats, with adequate founding at a high level.

Type 2: Abutments integral with the deck, which can be either closed or open forms.

Type 3: Semi integral abutments, which can also be either closed or open forms.

Type 4: Spill through abutments.

Closed abutment forms

Type 5: Vertical cantilever abutments.

Type 6: Counterfort abutments.

Type 7: Cellular (or box) abutments.

Type 8: Mechanically stabilised walls with perched bearing seats.

This image cannot currently be displayed.

Figure **Error! No text of specified style in document.**.14: Bridge Abutments, Retaining Walls and other Earth Retaining Systems

Abutments with wingwalls parallel to the roadway

Abutment Types 1, 2, 3, 4 and 7 usually have horizontally cantilevered wingwalls (earwings) parallel to the roadway.

The wingwalls for abutment Type 8 are often a continuation of the abutment front face and are sometimes curved in plan, finishing parallel to the bridge deck.

Abutments with wingwalls angled relative to the bridge deck

Abutment Types 5 and 6 usually have wingwalls of the same structure type as the abutments, angled relative to the bridge deck.

Free standing retaining walls and retaining systems

The most common form of free standing wingwalls are the vertical cantilever Type 5, similar in form to the abutments but without the deck seating. For high walls the counterfort Type 6 is used.

Mechanically stabilised and gabion systems (Type 8 and 9) are also commonly used as retaining structures associated with highway engineering.

Piers

1. General

Piers usually comprise cast in situ reinforced concrete columns or walls and may have a variety of cross-sectional shapes, constructed using conventional formwork, except when these components are taller than about 30 m when sliding or climbing forms are utilised.

Piers provide intermediate supports for bridge superstructures consisting of more than one span. The superstructure may be monolithic with the piers or alternatively supported by bearings with varying degrees of freedom. Piers are mostly supported by conventional spot footings for single columns, or strip footings in the cases of a row of columns or walls. For deep founding piers are usually supported by pile groups and less often by caissons.

The configurations of piers are typically closely related to the nature of the obstacles being crossed by bridges, which can be broadly subdivided into three principal groups as follows, and depicted in Figure **Error! No text of specified style in document.**.15.

Figure **Error! No text of specified style in document.**.15: Pier Forms Suitable for Different Types of Locations

River crossings

Piers for river bridges are typically of the wall type, which should be shaped with cutwaters to minimise the resistance to flow or the entrapment of debris. The orientation of pier centre lines for river crossings should preferably coincide with flood high stage flow lines, which may not have the same direction as the low flow channel.

Road and railway crossings

Examples of typical cross sectional and elevation details for the piers of bridges which cross roads or railways are shown in Figure **Error! No text of specified style in document.**.15. Wall type piers are not suitable for low, wide bridges as these produce a tunnel-like effect. In highly skew bridges the number of columns for each pier should be reduced to the minimum to improve visibility beneath the structure and avoid the sensation of passing through a forest, as further discussed in Section 5.5.5 of Chapter 5.

Valley crossings

For high bridges (viaducts) over wide valleys the piers should preferably be designed with single or double cell box sections, examples of which are also depicted in Figure **Error! No text of specified style in document.**.15. The walls of such hollow piers should be at least 250mm to 300mm thick to permit the use of slip form or climbing formwork. However, the appearance of very tall piers is often improved by light tapering (1 in 70 recommended) from the bottom upwards, at least in one dimension, which will influence the method of construction.

Foundations

Conventional Foundations

1. Function

The primary function of foundations is to provide adequate support to the structures which they carry. This implies sufficient load bearing capacity to safely resist the effects of the various combinations of permanent and transient loads transmitted to the founding strata, without excessive deformation, which could otherwise compromise the integrity of the structure or impair its use. The safe or **allowable bearing pressure** is therefore a function of the ultimate load bearing capacity of the ground at the founding level and the load-settlement characteristics of the underlying layers.

In the case of rock foundations the allowable bearing pressure is determined from rock mechanics principles, with due regard to the degree of weathering, the inclination of the rock strata, the presence of shear planes, fissures and clay gouges in the bedding planes, among other factors. On intact rock foundations, bearing capacity may be less critical than other criteria, such as safety against overturning of earth retaining components for example.

The interaction between the structure and the ground when founding on compressible soils, is an important consideration regarding the articulation of the structure, usually to a greater degree than when founding on rock. The deformation of soils under load can vary greatly depending on the type, depth and characteristics of the soil, which are determined from in situ or laboratory tests. When it is uneconomical or unsafe to found at shallow depth due to the estimated magnitude of the deformation, it becomes necessary to resort to soil improvement or support on piles or caissons.

The safety of the foundations of river bridges and drainage culverts is also subject to the potential effects of scour. Resistance to undermining by scour is dependent on the nature of the support strata, the depth to founding and the protective measures, among other factors, which are discussed in Chapter 3.

Design and construction factors

The factors which relate to the foundations and which influence the design of structures and subsequently become the concern of the contractor can be summarised as follows:

Founding Material

Soil or rock.

Degree of compressibility, expansiveness, porosity etc.

Depth of Founding

Shallow: usually not more than about 3m.

Intermediate: usually between about 3m and 6m.

Deep: usually greater than 6m.

Location of Founding

Land environments, which are further subdivided into those which are:

remote from existing constructions, or

adjacent or near to existing services (roads, railway lines, pipelines etc.) or structures (buildings, bridges to be widened etc.).

Water environments, in which the structure is required to be founded below water level, or in waterlogged or unstable ground.

Method of Construction

Access to and drainage of the excavations.

Open unsupported excavations.

Excavations which require lateral support.

Underpinning of existing structures.

The method of excavation and removal of spoil.

> The circumstances which render it unsafe or uneconomical to construct conventional spread footings and preferable to resort to piled foundations or caissons can vary greatly from site to site and are likely to be influenced by the rate of progress needed to meet tight programmes, especially in water environments subject to flooding.

Piles and caissons

Piled foundations are usually the first alternative considered when it is impractical, uneconomical or unsafe to found conventional footings at shallow or intermediate depths below ground. There is a sufficiently wide range of piling systems available from which to select appropriate foundation solutions in most types of ground conditions for road bridges which require deep founding. Piling is usually considered from depths of about 7m and can reach depths of 50m or more.

Caissons provide an alternative means to achieve adequate founding at intermediate to significant depths in both land and water environments. This system has been frequently used overseas as the most practical means to found major bridges in deep water conditions and is employed in harbour engineering for the construction of wharves and quays. Whereas open caissons are now infrequently used for small to medium span bridges because installation is comparatively slow, this form of foundation construction can be a viable option for depths of about 5m to 10m because of the very low establishment costs involved. An alternative to cast in situ caissons is the smaller diameter shaft (1.5m to 2.5m dia.) comprising precast concrete outer rings filled with in situ concrete.

Ancillary Components

General

Ancillary components for bridges include:

Roadway joints

Bearings

Parapets

Deck and subsurface drainage

Attachments to structures

The ancillary components are functionally essential and aesthetically important in the design of bridges and have an important influence on the durability and life cycle costs of these structures.

The ancillary components as a whole usually constitute a disproportionally small percentage of the cost of a bridge compared with their importance to the performance, utility and safety of the structure, but are difficult and costly to replace in the event of damage or malfunction.

Bearings and Joints

It is evident that no other single component of bridges has caused as many service problems as expansion joints, which require intermittent replacement despite regular inspections and maintenance. Bridge bearings fall into the same category, but to a lesser degree, as these are not as directly exposed to wear and tear caused by the constant impact of vehicle wheels. However, bearings are considerably more difficult to replace because of their location and considerations of traffic accommodation.

Bridge designers are therefore advised to be conservative in the assessment of the loads and movements to be accommodated by bearings and joints and careful with the detailing of the members required to support or house them. Comprehensive information about the design and installation of commonly available types of bearings and joints is provided in Reference 4.5.

Parapets

The safety of bridges is highly dependent on the strength and degree of containment provided by parapets on bridges and railings or similar components on the approaches to these structures. The design and detailing of specific parapet or barrier systems is beyond the scope of this manual. The bridge designer is therefore advised to refer to the applicable section of BS5400 in this context as well as to References 4.6; 4.7 and 4.8.

Deck and subsurface drainage

The function of drainage is to dispose of sub-surface and surface water via designed outlets through various bridge components, in order to prevent the development of water pressure behind earth retaining structures or the accumulation of water on bridge decks which could prove hazardous to road users. In other locations drainage outlets are required to dispose of water which has percolated through joints and deck surfacing to avoid entrapment and durability problems.

The specific components which require the provision of drainage facilities are:

Abutments, retaining walls and culvert barrels, behind which drainage filters and pipes are required to collect ground water and dispose of this through weep holes.

Abutment girder beds which require the provision of collector channels and outlet pipes to remove water which has leaked through expansion joints or has arisen from driving rain or condensation.

Deck roadway surface subject to direct rainfall, which must be disposed of via drainage scuppers, supplemented in certain instances by grid inlets and concealed drainage pipes.

Deck concrete surface on the uphill side of concrete nosings, asphalt plug joints or proprietary joints, which cause the entrapment of water which has percolated through the asphalt surfacing and must be disposed of through small drainage pipes.

Drip notches in the underside of deck cantilevers, strictly in compliance with the configuration and positions shown on the drawings. The careless omission of drip notches can lead to the defacement of the soffits and sides of bridge decks through run off water laden with silt and other contaminants even before construction of the parapets.

Drainage of the voids of voided slab decks is discussed elsewhere in this manual.

Attachments to structures

Early provision should be made in the design process for attachments to bridge structures, such as light masts, signage and signalisation poles, etc. to avoid the need to make late changes to the components affected. In the case of light masts, these should be evenly spaced along the length of bridge decks in the interest of aesthetics.

References

4.1 Schlaich G, Scheef H, CONCRETE BOX-GIRDER BRIDGES, Structural Engineering Documents 1e, International Association for Bridge and Structural Engineering, Zürich, Switzerland (1982)

ISBN 3#85748#031#9

4.2 Menn C, PRESTRESSED CONCRETE BRIDGES, Birkhäuser Verlag, Basel, Switzerland (1989)

ISBN 3-7643-2414-7

ISBN 0-8176-2424-7

4.3 Hewson NR, DESIGN OF PRESTRESSED CONCRETE BRIDGES, ICE Manual of bridge engineering (Second Edition), Parke G (Ed) and Hewson N (Ed), Thomas Telford, London (2008)

ISBN : 978-0-7277-3452-5

4.4 Lindsell P, Clayton CRI, Xu M, Hewson N, SUBSTRUCTURES**,** ICE Manual of bridge engineering (Second Edition), Parke G (Ed) and Hewson N (Ed), Thomas Telford, London (2008)

ISBN : 978-0-7277-3452-5

4.5 Ramberger G, STRUCTURAL BEARINGS AND EXPANSION JOINTS FOR BRIDGES, Structural Engineering Documents 6, International Association for Bridge and Structural Engineering, Zürich, Switzerland (2002)

ISBN 3-85748-105-6

4.6 British Standard BS//EN//1317-2:1998

Road Restraint Systems, Part 1: TERMINOLOGY AND GENERAL CRITERIA FOR TEST METHODS

4.7 British Standard BS//EN//1317-2:1998

Road Restraint Systems, Part 2: PERFORMANCE CLASSES, IMPACT TEST ACCEPTANCE CRITERIA AND TEST METHODS FOR SAFETY BARRIERS

4.8 British Standard BS//EN//1317-4:2002

ROAD RESTRAINT SYSTEMS, PART 4: PERFORMANCE CLASSES, IMPACT TEST ACCEPTANCE CRITERIA AND TEST METHODS FOR TERMINALS AND TRANSITIONS OF SAFETY BARRIERS

APPENDIX 4A

Comprehensive List of Bridge Types

Slab Type Bridges (In Situ or Composite)

Solid

Voided

Twin spine types

Girder Type Bridges

Precast beam and slab decks

Tee beams

I Beams

U beams

R beams

M beams (Psuedo multi cell box)

In situ beam and slab decks

Single spine (Tee beams)

Twin and multi ribbed beams

U beams (Walk through type)

In situ box girder types

Single and double cell boxes

Twin box type (similar to 1.3)

Special box girder or forms, categorised by the method of construction

Incrementally launched: single cell

Balanced cantilever: single cell (in situ or precast segments)

Trussed Girder Bridges

Lattice (various configurations)

Vierendeel

Framed Bridges

Single and multi span portals (including semi or fully integral forms)

Vee framed: single or multi span

Strut framed (usually three span)

Arched Bridges

Spandrel

Open and closed forms; deck stiffened

Funicular

Tied forms

Bowstring suspension

Cable Supported Bridges

Suspension

Conventional (cables and hangers)

Stress ribbon

Cable stayed

Conventional

Harp or fan configurations

Extradosed

Cellular Structures

(Drainage culverts, subways, service conduits etc.)

Circular, elliptical etc. (steel or precast concrete)

Arched forms (precast or in situ concrete)

Portal forms (precast concrete, single or multi cell)

Square or rectangular (in situ concrete, single or multi cell)

Special Types

Lift bridges

Floating bridges

Fundamentals of Analysis and Design

Introduction

In this chapter the four fundamental goals of bridge design are identified, followed by an explanation of their prioritisation. A substantial portion of the text is devoted to means needed to achieve these goals. Consideration is also given to the important goal of sustainability.

The means required to achieve the goals of bridge design include appropriate methods of analysis accompanied by judicious decisions about the optimisation and refinement of bridges during the evolutionary process from initial concept through to final design. At each stage suitable checks are required to verify compliance with the applicable code requirements, with equal attention to the creative but less formal objectives of bridge design.

The final part of this chapter provides guidance about the alternative methods of bridge deck analysis best suited to the achievement of the primary goals of bridge design. The relative merits of the various methods of analysis are explained in References 5.1, 5.2 and 5.3, to which the bridge designer should refer for information beyond the scope of this chapter. However, in this regard the words of Albert Einstein should always be kept in mind:

"Everything should be made as simple as possible, but not simpler"

The Goals of Bridge Design

General

The primary goals to be considered are those of **Safety** and **Serviceability**, which constitute the **Functional** goals of bridge design, concerned with the safety of these structures and of those who use them on the one hand, and about the need for bridges to remain fit for purpose during the intended design life on the other. These goals are required to be verified in terms of the principles codified in BS5400 in the case of Nigerian highway structures.

The complementary fundamental goals are those of **economy** and **aesthetics**, which are not codified and require the attributes of good judgement and creativity on the part of the bridge designer, in order to create bridges which are cost effective as well as aesthetically pleasing. Accordingly economy and superior aesthetic qualities may be described as the **creative** design goals.

An over-arching consideration in many fields of human activity is that of **sustainability**, which has come into sharper focus in recent years in recognition of the need to conserve scarce natural resources and protect the environment for the benefit of future generations.

In the context of bridge design various procedures are in the process of development, which are aimed at assisting the bridge designer to meet the objectives of sustainability. Whilst noting that the four fundamental goals of bridge design are important components of sustainability, practical design procedures embodying other facets of this goal, such as life cycle costing, are not yet formulated in a universally accepted manner. Whereas formal design procedures to achieve the goal of sustainability or evaluate the results are not included in Chapter 5 at this stage, the elements of sustainability are summarised in Figure 5.1 for future reference.

The relative importance of the bridge design goals

In Reference 5.1 the relative importance of the fundamental goals (objectives) of bridge design are well explained by Christian Menn as follows:

"The fundamental objectives of bridge design are ***safety, serviceability, economy and elegance****. A design can be considered successful only when all four of these goals have been achieved. The relative importance of the objectives is defined by the consequences arising when they are not achieved. These vary from the unpleasant feelings evoked by ugly bridges to the loss of life and property caused by unsafe bridges. The order in which the objectives have been listed above can thus be regarded as hierarchical, beginning with safety as most important.*

Safety and serviceability are achieved through the systematic application of scientific principles. They thus depend on the analytical skill of the engineer. The criteria used to determine whether these objectives have been met are codified in design specifications and standards. Proficiency in designing safe and serviceable bridges can be acquired through an understanding of the underlying scientific principles.

Economy and elegance, on the other hand, are achieved through non-scientific means. They depend almost entirely on the creativity of the engineer. Economic and aesthetic criteria have not been codified and are largely subjective. Useful guidelines are available to help in improving the cost-effectiveness and visual form of bridges. Proficiency in designing economic and aesthetically pleasing bridges can nevertheless be acquired only through direct design experience, critical observation of completed structures, and full utilization of the engineer's creative talents.

Visual elegance and economy are to some extent interdependent. Aesthetically pleasing bridges are distinguished by transparency, slenderness, and the lack of unnecessary ornamentation, all of which result in an efficient use of materials and hence low construction cost. It is incorrect to infer, however, that the most economical design is necessarily the most elegant."

The four fundamental goals of bridge design together with that of sustainability are illustrated in Figure 5.1. Whereas these goals are numbered 1 to 5 to reflect the sequence in which they are discussed below, it is essential that they be treated interactively from the outset and throughout the design process. The various issues which are the subject of concern of these goals are listed under each relevant heading, and discussed in the subsequent sections of this chapter.

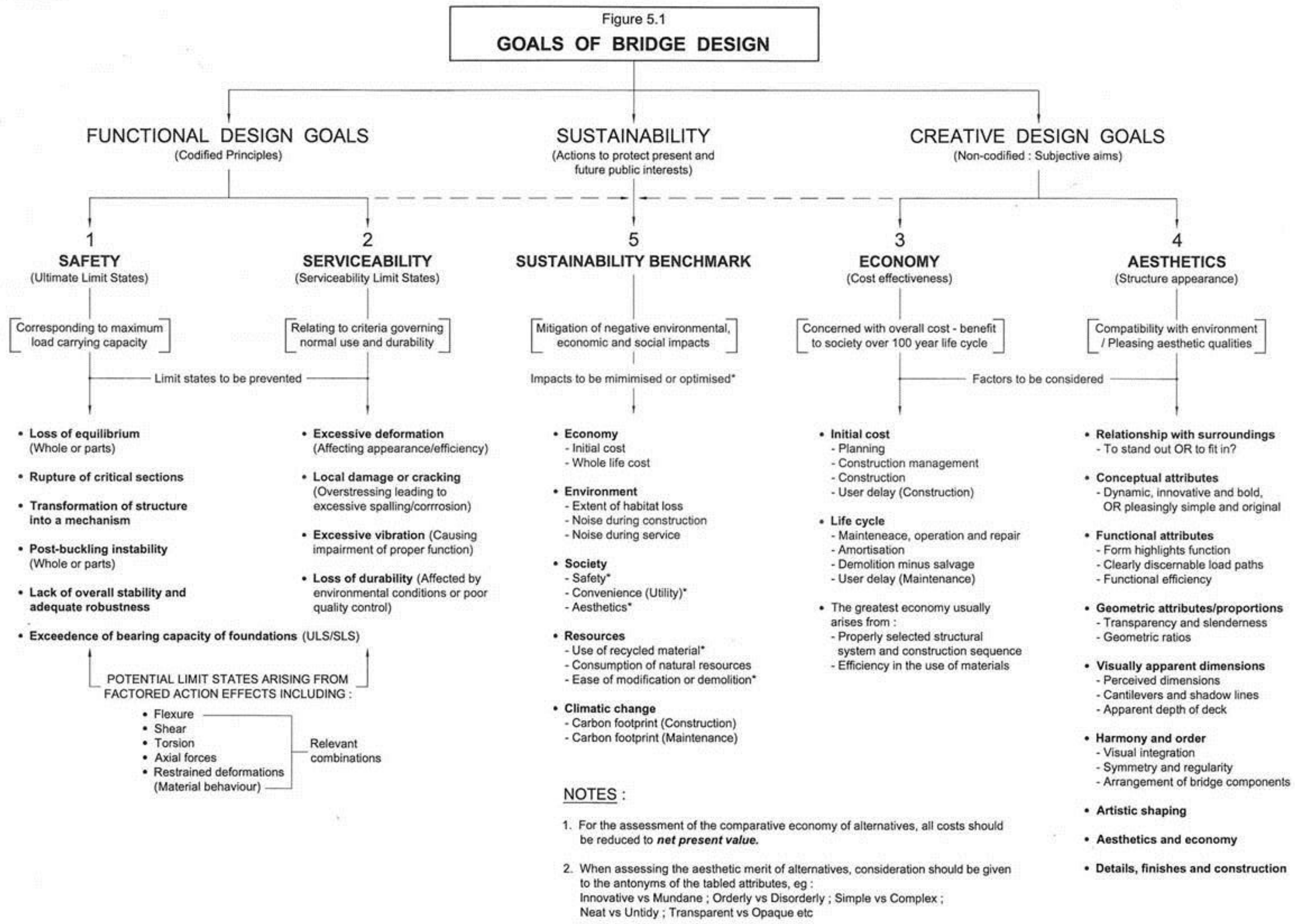


Figure **Error! No text of specified style in document.**.16:

Design for Safety and Serviceability

The BS 5400 design code applicable to bridges and other highway structures in Nigeria under the jurisdiction of the FMW, is based on the **Limit State** design method. This code requires that structures and their components should achieve acceptable degrees of safety or structural reliability against reaching either:

Ultimate Limit States, which are concerned with safety against various categories of potential failure, as listed in Figure 5.1, or:

Serviceability Limit States, which are concerned with prevention of excessive deformation, cracking, vibration etc. and the long term durability of the structure, as listed in Figure 5.1.

Thus before any structure may be built, its safety and serviceability must be proved during the design phase. This is accomplished by comparing quantitative measures of the safety and serviceability of each structural component and system to minimum acceptable values defined by BS5400. Since they are often independent of each other, safety and serviceability must be verified separately.

Safety

Design for safety involves the usual design steps in terms of BS5400 limit state code requirements as follows:

Appropriate elastic method of analysis to establish the critical sectional forces due to the applicable combination of the loads, prestressing and restrained deformations, including the appropriate partial safety factors to determine the design sectional forces ΣF_d.

Calculation of the applicable ultimate resistance of the cross-sections, diminished by the appropriate partial safety factors to determine the safe design resistance R_d

Confirmation that the safe design resistance equals or exceeds the related design sectional forces for all parts of the structure i.e.:

$$\Sigma F_d \leq R_d$$

Detailing of the reinforcement and the prestressing.

In carrying out the necessary ultimate limit state analysis and sectional design checks the bridge designer should take the following into account as appropriate:

The analysis should always be based on a lower bound elastic method of analysis (as outlined in Section 5.7.2). Methods of analysis based on the *kinematic method of plasticity*, which yields an upper bound for the ultimate load, should never be used for design. Such methods may, however, be useful as a means of checking elastic designs based on the statical method.

The parasitic effects of prestressing in the case of indeterminate structures should have been taken into account in the analysis including the long term effects on the prestressing, for sectional checks at different stages. Prestressing is essentially equivalent to a load case which improves structural behaviour through induced stresses and deformations. The differences in the behaviour and effects of bonded and unbonded prestressing is an important consideration.

Long term effects arise from:

creep in concrete;

shrinkage in concrete; and

relaxation in prestressing steel.

Redistribution in sectional forces arises due to a change in the structural system in cases such as:

the cracking of concrete sections of indeterminate structures, which effects stiffness; and

structures which are determinate for dead loads but are made indeterminate for live load cases.

1. Restrained deformations arise as a result of temperature change, shrinkage, creep and support displacements, and induce sectional forces that are directly proportional to stiffness, and hence highly sensitive to any change in stiffness.
2. At ultimate limit state, the sectional forces due to restrained deformations disappear completely with the formation of plastic hinges. Restrained deformations need therefore only be considered in systems of limited ductility, where the assumption of plastic deformations is not valid, and for serviceability checks.

Serviceability

Limit states at which bridges are structurally safe but otherwise unfit for service are called *serviceability limit states*. The most important aspects of serviceability are durability, function and appearance. *Appearance* refers here to the prevention of unsightly defects such as water stains and visible cracks, rather than to the aesthetic aspects of design. The *function* of individual components, such as expansion joints and bearings, must also be considered. *Durability* is closely related to function and appearance, both of which can be severely impaired by deterioration of concrete and reinforcement. It is also related to structural safety, since deterioration of reinforcement can result in a serious loss of resistance.

Satisfactory behaviour under service conditions cannot be verified incontestably during design in the same way as safety, since the direct relationship between safety and equilibrium has no analogy in the context of serviceability. The following strategies are therefore used to ensure that bridges will perform well during service:

Behaviour that can be readily quantified and calculated can be considered acceptable when it is within an acceptable range of values. Acceptable values may be specified in codes and standards or may be determined by the owner in consultation with the engineer, for a specific project. They are often based on non-technical considerations and are usually not absolute. Vibration criteria, for example, are normally more severe for bridges that are used by pedestrians as compared to those used by vehicles alone. This category includes most aspects of structural behaviour under service conditions, including deflections, vibrations and cracking. Many aspects of non-structural behaviour, for example the run-off of rain water on the deck, are also included.

Behaviour that is not easily quantified and calculated cannot be verified against acceptable values. An example from this category is the corrosion of reinforcing steel. A quantitative description of the corrosion process, although possible, is not well suited to the needs of bridge designers. Undesirable behaviour of this type is best prevented by properly specifying materials, careful workmanship and inspection during construction, and good detailing practice. Details that facilitate inspection, maintenance, and replacement of defective components are of particular importance in this regard.

Serviceability evolves over time, due to changes in the actions to which bridges are subjected and the ability of bridges to withstand them. Regular inspection is therefore of utmost importance for the early detection of potential problems. *It is desirable that on completion of the construction of all bridges a formal detailed program of periodic inspections should be initiated, valid for the entire service lives of bridges.*

Serviceability issues which should be checked in terms of the BS5400 requirements include the first three listed in Figure 5.1 as follows:

Excessive deformations

> These affect appearance and efficiency. BS5400 provides guidance on maximum recommended span: deflection ratios for example. Issues such as the long term deflection of cantilever retaining walls, allowing for the effects of creep, need to be

considered. The bridge designer must confirm whether the member is in a cracked or uncracked state in the calculation of deformation.

Local damage and cracking

All of the methods used to calculate crack widths and deformations of structures in the cracked state are based on the assumption that the stresses in the reinforcement are known beforehand or can be calculated. In reality, however, it is impossible to know the exact value of the steel stresses under service conditions. Stresses in the steel are a function of many different factors, some of which are subject to considerable variability. The most important of these include prestressing losses, the redistribution of sectional forces, self-equilibrating states of stress, and restrained deformations.

It thus follows that whatever accuracy promised by an "exact" calculation of steel stresses under service conditions is illusory. Simplifications based on rational models of structural behaviour should therefore be used to calculate steel stresses, crack widths and deformations. It also follows that the criteria used to evaluate cracking behaviour and deformations need not be regarded as "exact" values, but rather as rough, conservative estimates.

Cracking due to live load normally has no influence on durability or appearance and is thus not considered in bridge design. Of much greater importance are cracking caused by the combined effects of permanent load (dead load plus prestressing), restrained deformations, and self-equilibrating stresses. The prestressing concept is usually selected to prevent tensile stresses in the concrete due to permanent load. Concrete stresses in excess of tensile strength are therefore the result of the additional effects of restrained deformations and self-equilibrating stresses.

BS5400 provides information about minimum reinforcement to limit crack widths in beams and slabs, and simplified equations to design reinforcement for the same purpose in the case of various combinations of tension, bending, restrained shrinkage and temperature gradients in structures.

In the case of lateral displacement imposed at the tip of a column (imposed deformation) the sectional forces thus produced are a function of the stiffness of the column and cannot be easily calculated. The design of the reinforcement should not be based on the bending moment diagram, but rather on the deformation of the system. The reinforcement is designed to limit crack width to a given allowable value. It must then be verified that the deformations corresponding to the chosen reinforcement are compatible with the given tip displacement.

Excessive Vibration

Vibrations are normally of secondary importance, especially in prestressed concrete bridges. Dynamic analyses are therefore only required in exceptional cases, for example very slender bridges, heavily loaded bridges with pedestrian traffic, or single purpose pedestrian bridges. When there is a likelihood of the structure being subjected to excessive vibration from causes such as wind forces or moving traffic, resulting in resonance, appropriate analyses based on specialist literature should be carried out and measures taken to prevent discomfort or alarm, or impairment of its proper function.

Bridge designers who require more detailed information about the fundamentals of analysis and design, with particular regard to safety and serviceability are advised to refer to Chapter 4 of Reference 5.1.

Economy

General

The cost-effectiveness of bridges should preferably not be judged on the basis of construction cost alone. Bridge costs are best compared on the basis of *life-cycle cost*, defined as the total cost of construction, operation, amortization, and demolition, including the costs and benefits arising from changes in existing traffic patterns. Attempts to reduce the consumption of construction materials through optimization of span lengths and cross-section dimensions will have a limited effect on the total life-cycle cost. Cost-effectiveness is rather a function of overall concept, characterized by a properly chosen structural system, cross-section, foundation system, and construction sequence.

In many instances the length of bridge spans may be dictated by horizontal clearance requirements and permissible pier positions in the cases of bridges over roads and railway lines. Likewise the location of piers for river bridges may be dictated by hydraulic requirements or very deep foundations. In these cases optimisation should be investigated in terms of overall structure length, taking the cost of abutments (open or closed) and approach embankments into account, for the alternative structural forms considered.

Life-cycle costs

Operating expenses are incurred as a result of annual inspection, annual maintenance and periodic rehabilitation. Yearly operating costs can be calculated by expressing the cost of rehabilitation as an equivalent annual expenditure. Total yearly operating costs of highway bridges are expressed as a percentage of the construction cost as outlined below. These figures are valid for bridges that have been designed and constructed to minimize life-cycle costs; annual operating expenditure will usually be higher for bridges that have been designed to minimize construction cost.

Bridges are removed from service as a result of changes in the transportation system, increases in legal live loads beyond the capacity of the bridge, or excessive maintenance and rehabilitation costs. For planning purposes, the lifetime of a bridge is normally assumed equal to 100 years.

Annual operating costs of highway bridges (inspection, maintenance, rehabilitation averaged over the lifetime of a bridge) can vary to a significant degree from country to country depending on the regularity of these operations and on local financial interest and inflation rates.

For approximate budgetary purposes it is suggested that Total Average Annual Operating Costs as a percentage of Construction Cost of between 1.0 and 1.5% should be adopted. These figures, including potential savings arising from ultimate salvage, should be brought back to net present value for summing with estimated construction costs when comparing alternative schemes.

The lower annual percentage (1%) would usually be applicable to bridges in which the cost of replacement of ancillary items is likely to be minimal.

Construction Costs

The total construction cost of a concrete bridge can be categorised as shown in Figure **Error! No text of specified style in document.**.17.

Figure **Error! No text of specified style in document.**.17: Elements of Construction Costs

Contractor's Mobilisation and Establishment is defined as the work required before construction can begin, for example providing access to the construction site, provision of site facilities and procurement of plant and equipment.

Structure costs include the cost of the direct physical work in constructing the bridge, excluding the contractor's overheads (Items 1 and 3). Labour costs are usually not extracted separately and therefore form part of the costs of the components or the materials. Falsework applies to in situ concrete bridge construction, whereas erection applies to precast beam and slab construction.

Contractor's Construction Management costs provide for all monthly overhead expenditure, including items such as management personnel, insurance costs, finance costs, statutory fees and taxes, establishment running costs, accommodation of traffic, etc.

Design and Contract Administration costs are usually excluded from the cost of construction and allow for all costs expended by the FMW on their own direct inputs, or for payments to consulting engineers and specialists for site investigations prior to design (survey, geotechnical etc.), planning and design, contract administration and supervision.

Annual life-cycle costs are therefore expressed as a % of the sum of costs 1, 2 and 3.

Depending on the remoteness of the site and the difficulties of access, the sum of the contractor's mobilisation and management costs (1+3) will often be of the order of 17% of the structure costs, thus:

1 + 3:	Contractor's overheads	:	17%
2:	Structure	:	83%
Σ:	Deemed construction cost	:	100%

1. Sub-division of structure costs

Estimates of structure costs should be based on cost records of similar recently completed bridges.

For planning purposes with average founding conditions, deck widths in the range from 11m to 15m and open abutments, the sub-division of the cost of conventional concrete bridges will usually be of the following order:

Substructure	:	30%
Superstructure	:	60%
*Accessories	:	10%

Total structure : 100%

*Accessory costs can usually be assigned approximately 3% to the substructures and 7% to the superstructure.

Preliminary estimates of superstructure costs

Superstructure costs can be fairly reliably estimated with the help of the *geometrical average span length l_m* defined by:

$$\boldsymbol{l_m} = \frac{\Sigma \boldsymbol{l}_{\mathbf{i}}^{\mathbf{2}}}{\Sigma \boldsymbol{l_i}} \text{ (m)}$$ Equation **Error! No text of specified style in document.**-3

The summation is over the total number of spans; the length of span i is denoted l_i

For girder depths h (m) in the following range:

$$\frac{1}{20} < \frac{h}{l_m} < \frac{1}{15}$$

the volume of concrete in the superstructure (V m^3) may be obtained by multiplying the total deck surface area (m^2) by the effective girder depth h_m (m), defined by the following expression)

$$h_m = 0{,}4 + 0{,}0045\,\boldsymbol{l_m} \text{ (m)}$$ Equation **Error! No text of specified style in document.**-4

This equation yields a reliable deck average structural thickness (h_m) for box girder type decks in the span range from 20m up to about 50m, and a reasonably reliable average thickness for other commonly used concrete deck forms.

The remaining information required for deck cost estimation purposes includes the following obtained from analysis of completed similar bridge decks:

Concrete : cost per m^3 (C)

Falsework : cost per m^3 (S)

(Total cost ÷ Total deck concrete)

Formwork : cost per m^3 (F)

(Total cost ÷ Total deck concrete)

Reinforcement : cost per m^3 (R)

(Total cost ÷Total deck concrete)

Prestressing (tendons and anchors)

: cost per m^3 (P)

(Total cost ÷ Total deck concrete)

The all in unit cost of the superstructure concrete will thus be:

$\Sigma C = C+S+F+R+P$ per m^3 Equation **Error! No text of specified style in document.**-5

and the total estimated cost of the superstructure in the appropriate currency will be:

Deck Cost = $\Sigma C.V$ Equation **Error! No text of specified style in document.**-6

By reference to the foregoing subdivision percentage costs of the superstructure, substructure and accessories and the contractor's average overhead percentage, the total structure and construction costs may be obtained.

This process may be repeated for alternative bridge configurations with different span and total lengths for preliminary comparative purposes.

Finally, life-cycle cost estimates should be checked by using an appropriate range of annual discount rates based on historical data.

Aesthetics

Introduction

The achievement of aesthetically pleasing bridges is concerned with a number of inter-related issues which are outlined in Figure 5.1 and discussed in the following parts of this section. In general it will be found that the best designs are based on simple concepts in which the functional and creative aspects of bridge design complement each other and evolve through a continuous process of functional optimisation and aesthetic refinement.

Seldom will preconceived ideas yield the most satisfactory overall result. Indeed the boundary conditions imposed by the determinants discussed in Chapter 2 of this volume invariably provide clues to the most suitable structural type, configuration and method of construction for a particular site. The most successful designers are those who develop the talent to interpret the design possibilities in such a way as to create refined, elegant or even beautiful bridges, which are also functional and cost-effective, in spite of the adverse physical challenges of the site.

Bridge designers who require more detailed guidance on this subject than the brief outlines which follow, are advised to study References 5.4; 5.5 and 5.6 in particular. There are innumerable other easily accessible documents about bridge aesthetics, however those which advocate a tick-box approach to aesthetic design should be avoided as this seldom yields satisfactory results.

Relationship with the surroundings

It is generally agreed that the first aesthetic issue to be considered is whether a bridge should be designed to <u>stand out</u> or to <u>fit in</u> with its surroundings?

A bridge can be perceived as an independent entity or as an element of a larger landscape. Aesthetics in bridge design can thus be considered as a function of both abstract structural form and the relationship between structural form and environment. These two aspects of bridge aesthetics are often independent of each other. Structures that are aesthetically pleasing as independent objects are not always suited to their surroundings. Conversely, integration into the environment may be achieved in spite of shortcomings in structural form. It is therefore important that neither aspect be neglected by the designer.

The appearance of a proposed design must be evaluated from all possible viewpoints. The use of a large-scale model or three-dimensional computer graphics is strongly recommended; two-dimensional orthographic views alone are insufficient. The relative importance of the viewpoints should be considered. Frequently occurring views are normally more important than those that occur infrequently. The appearance of the bridge as seen from the most important viewpoints should be designed and evaluated with special care.

Bridge aesthetics should be considered in relation to the alignment of the highways or railways which they serve. Topographical features that may enhance the appearance of bridges should be identified and, if possible, incorporated into the alignment. Viaducts along mountain slopes should be gently curved to follow the contours of the landscape.

The character of the landscape should be reflected in the structural form. A subdued form is preferable in flat or gently rolling terrain that lacks conspicuous topographical features. Prominent obstacles such as wide rivers or deep canyons are best crossed by structures in which one span has been given particular visual emphasis. Bridges in built-up areas should normally be unobtrusive. In certain cases, however, a positive visual effect can be achieved by a structure that stands out from an urban environment, particularly in areas of industrial or other decay where renewal is planned.

Conceptual attributes

The outcome of the bridge designer's deliberations about the bridges relationship with the surroundings leads directly to a decision about the preferred form of the bridge, i.e. whether the structure should be innovative and bold or pleasingly simple and original.

Certain types of bridges, such as strut frames and open spandrel arches are comparatively dynamic in their form, which gives the impression of a giant leap over space and are therefore well suited to spanning over deep gorges and valleys and at height above rivers.

Conversely, low bridges over bland terrain appear static and should preferably be simple structures with slender continuous decks and commensurately proportioned abutments and piers. Further comment about the treatment of low, wide bridges is given in Sections 5.5.5 and 5.5.6.

Functional Clarity and Efficiency

It is generally accepted that the expression of function is the basis of good design, and conversely in the field of bridge engineering that form should not be dominated by function to the point of austerity. It is therefore desirable that the form of the structure should demonstrate its function by clearly discernible load paths, which are not unnecessarily contrived and indirect. In short, it should be easily apparent to the observer how the structure works.

The technical and aesthetic aspects of bridge design are closely related through the concept of efficiency. Our perception of elegance in bridges has been conditioned by familiarity with structures in the natural world, where beauty and efficient use of materials are inseparable. From Roman times to the present day, bridges that have become renowned for their elegance have almost without exception been remarkable for their efficient use of materials. The visual expression of efficient structural function is thus a fundamental criterion of elegance in bridge design. It is one of the primary distinguishing factors between structural engineering, art and architecture.

The role of efficiency in bridge aesthetics is illustrated in Figure **Error! No text of specified style in document.**.18. It is apparent that the bridge of Figure **Error! No text of specified style in document.**.18a, a slender inclined-leg frame, requires considerably less concrete than the girder and retaining walls of Figure **Error! No text of specified style in document.**.18b. The former structure thus accomplishes the same function as the latter but with a more efficient use of materials. The massive, heavy appearance of the bridge of Figure **Error! No text of specified style in document.**.18b is directly related to its lack of efficiency. Its counterpart appears much lighter in comparison.

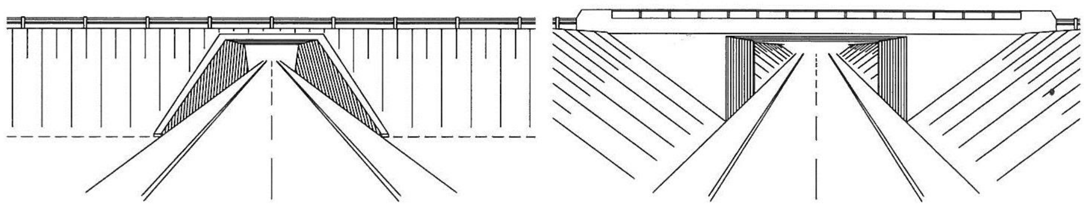


Figure **Error! No text of specified style in document.**.18: Visual Expression of Functional Efficiency

Transparency and slenderness

Two of the most important ways of expressing efficiency are transparency and slenderness.

Bridges that lack a suitable degree of transparency appear as solid walls from a wide range of viewing angles. For exceptionally low bridges, transparency is a function of girder depth. For most bridges, however, transparency is a function of the number and width of the columns. It is most effectively enhanced by reducing column width; even long bridges with many short spans can be given an adequate degree of transparency provided the columns are suitably narrow.

Maximum transparency is achieved using one column per support axis. Round columns, which are most effective in this regard, often appear to lack lateral stability and thus create a disturbing impression. Rectangular columns are a reasonable compromise between transparency and perceived stability. Single columns can be used regardless of bridge height provided the width of the superstructure, *B,* is less than 12m. Column width, b, should be chosen to ensure that the ratio *B/b* is between 3:1 and 3.5:1. Single columns can also be used for wider bridges (*B* > 12m), provided the bridge is sufficiently tall and the superstructure consists of a single cross-section. For these bridges, the ratio *B/b* should be chosen between 3.5:1 and 4.0:1.

Two columns per support axis must normally be used for low, wide bridges (*B* > 12m) and for all twin bridges. The columns should be slender and have a compact cross-section, for example a circle or a flattened hexagon. The lateral spacing between columns should yield a balanced moment diagram in the superstructure diaphragm. The use of three or more columns per support axis severely reduces transparency and is therefore not recommended.

The transparency of long bridges is drastically reduced by hammerhead columns and multiple-column bents. Their use should therefore normally be avoided. Two-column frames have been successfully used, however, for bridges with a small number of relatively long spans. An acceptable degree of transparency is obtained provided the individual spans can be recognized.

Slenderness is primarily a function of the superstructure arrangement. It is normally expressed quantitatively in terms of an *effective slenderness* parameter λ, defined as the ratio of span length to girder depth. The parameter λ is most useful as a rough measure of the relative economy of projects. The most economical superstructure is obtained when λ is chosen between 15 and 20. Although girders with effective slenderness ratios as high as 30 are possible, they are invariably considerably more expensive.

Visually apparent superstructure dimensions

The parameter λ is not, however, a reliable measure of visual slenderness, which is a function of the *visually apparent* superstructure dimensions. Visual slenderness can be defined as the ratio of the perceived uninterrupted length of the superstructure to the perceived superstructure dimension perpendicular to length. Depending on the location of the observer relative to the bridge, this dimension can be either girder depth or girder width. Span length therefore plays a subordinate role in the visual perception of slenderness, provided the continuity of the superstructure across the intermediate supports is not interrupted.

Apparent superstructure depth is of greatest significance in connection with low bridges. It is also important for short high bridges viewed from afar. Long high bridges always appear slender when observed from a distance, regardless of depth. Apparent depth can be effectively reduced through the use of wide deck slab cantilevers. The shadows they cast onto the girder webs create contrasting parallel strips of light and dark that accentuate the long dimension of the superstructure. Varying the depth of the girder can also be beneficial. The visual slenderness of a long three-span bridge, for example, can be substantially improved by haunching the main span and tapering the side spans. Haunched rigid-frames are preferable to low, single-span simple-beam bridges, which appear heavy for values of λ as high as 25. The interaction of superstructure depth and bridge height must also be considered; an adequate degree of transparency and slenderness cannot be achieved unless the ratio of bridge height to girder depth is greater than 4.

Apparent superstructure width is primarily of importance for the visual slenderness of high bridges. The critical viewpoint is relatively close to the structure; the critical direction of view is upward at a slightly oblique angle. Superstructure width is normally fixed by traffic requirements. The visually apparent width can, however, be reduced by an appropriately chosen cross-section. Single-cell box girders with wide deck slab cantilevers are particularly well-suited in this regard.

Harmony and order

All of the components of a bridge should be harmoniously integrated into one coherent, organic entity. This is accomplished by both visual means (providing symmetry, order and regularity) and technical means (properly defining the structural function of each component).

Our perception of harmony in bridges, like our perception of efficiency, has originated from familiarity with naturally occurring structural forms which have grown using minimum energy and materials. These forms invariably possess symmetry; the visual impression of balance and stability they create is always matched by a state of stable equilibrium and low stress. A symmetrical structural system forms the basis of almost all of the bridges that have been acclaimed for their elegance, from Roman times to the present day.

The concept of order is related to the orientation and arrangement of bridge components, many of which can be considered one-dimensional. The number of different inclination angles of similar structural members should be as small as possible, unless the members form a regular, gently curved envelope. Otherwise, the structure may appear ambiguous or unstable from certain viewing angles. Twin bridges that are parallel in plan should have identical longitudinal grades. If possible, the roadways should be arranged so that only one of the two bridges is visible from the most important viewing location.

Both the span arrangement and the cross-section must possess a high degree of regularity. Spans of equal length and a constant cross-section are thus desirable from a visual point of view. They have the added advantage of requiring the least amount of material and producing the most favourable construction conditions. Bridges of varying height should, however, be given spans of varying length. The ratio of span length to bridge height can thus be maintained constant, which results in a more balanced appearance than would be obtained using spans of equal length.

Artistic shaping

The raw structural form required for safety, serviceability and economy is rarely the most elegant. It can normally be refined into an elegant form, however, through artistic shaping of the structural members. The associated additional cost is insignificant. Shaping that follows the flow of internal forces is recommended in most cases. Forms that disregard the flow of forces normally produce a chaotic effect and should thus be avoided. In the hands of a gifted designer, however, member shaping based on purely aesthetic considerations and ornamentation can produce particularly charming results.

Aesthetics and economy

Any modification of the structural form made to improve appearance will be reflected in the total construction cost. Due to the relation between aesthetics and efficient use of materials, some modifications may result in cost reductions. Others, such as the artistic shaping of members, may increase the total cost. The combination of all structural modifications made for aesthetic reasons, excluding increases in span length, will normally result in a net change in construction cost of not more than about 2 percent.

Cost increases can be substantial, however, if span lengths are increased for aesthetic reasons. Any savings in substructure costs that may result are usually outweighed by increases in superstructure costs. The most economical span length is relatively short, and appears mediocre and overly cautious in most cases. Longer spans, on the other hand, substantially enhance transparency and convey an impression of efficiency and boldness of conception; the overall visual impression is greatly improved provided the spans remain in proportion to the surrounding landscape. It is therefore recommended that spans slightly longer than the economical minimum be provided, especially for prominently exposed bridges. A cost increase of up to about 7 percent of the cost of the most economical solution should be allowed for this purpose.

Details, Finishes and Construction Issues

The aesthetics of bridges which basically have pleasing geometric proportions and visually well balanced components can be undermined by the lack of equal attention to the detailing of ancillary components, for example the quality of the finishes and certain facets of construction.

1. Details

Details which require particular attention include the following:

The housing of bridge decks at abutments where the vertical faces of abutments should preferably continue to the underside of deck cantilevers, without leaving unexplained voids or set-backs.

The space provided for bearings and bearing plinths at the deck soffit, which should not be excessively large at either the abutments or piers.

The provision of adequate drip notches in the underside of deck cantilevers are essential and come into play even before the construction of upstands or parapets. Care also needs to be exercised with the longitudinal gradient of decks to ensure that concentrated flows are not discharged onto piers or abutment faces, thereby causing staining.

Drainage scuppers from bridge decks should not discharge concentrated flows onto road or rail below.

The spacing of joints in parapets which should be evenly spaced between support centre lines or deck movement joints.

The spacing of attachments such as lighting masts should preferably be uniform, with some relationship to span lengths if possible.

Finishes

Concrete finishes include plain or patterned surfaces imparted by the formwork; bush hammering or other exposed aggregate treatments of concrete surfaces; colouring of concrete by the selection of cement or fine aggregates or by the inclusion of colouring agents in the concrete mix design; or by the painting of finished concrete surfaces. High quality plain finishes constitute the great majority of the finishes for concrete bridge construction. The bridge designer must decide whether timber, steel or other forms are required or acceptable. In the case of large, bland concrete surfaces, when some degree of relief is desirable, carefully selected patterns or exposed aggregate finishes can provide pleasing contrast to add interest to bridge aesthetics.

Well proportional copings often provide a pleasing top edge-trim to wingwalls and freestanding retaining walls.

Colouring or relief or contrast of concrete can provide pleasing bridge aesthetics whilst painting can result in a stimulating appearance if tastefully and skillfully done. However, colouring of concrete can be highly variable if the quantity of the colouring agent, the quality of concrete mixing and the uniformity of the fine aggregates are not very carefully controlled. Painting of bridges invariably requires some degree of maintenance and repainting over a period of time.

Construction

The main construction issues which affect bridge aesthetics are those of the quality and uniformity of the concrete and the surface finishes, and in particular the regularity and symmetry of construction joint lines on completion. The bridge designer is always advised to select the position of permissible construction joints in bridge substructures, to indicate these on the drawings and to detail the splicing of reinforcement accordingly. If possible, the joint spacings should coincide with the sizes of the standard forms available in Nigeria.

Sustainability

Sustainability is aimed at the mitigation of negative environmental, economic and social impacts as noted on Figure 5.1, and concerned with the various factors which affect these issues. Whereas methods for establishing the sustainability index for bridge projects have been formulated, until such time as these methods have been developed into practical design tools for use by the bridge designer, no further comment on this subject is included in this chapter.

Alternative Methods of Bridge Deck Analysis

Available methods

This section provides a brief review of the methods commonly used for bridge deck analysis which may be summarised in the following groups:

1. Elastic Methods of Analysis

Influence lines (idealised line beams)

Influence surfaces

Orthotropic plate theory

Grillage analysis

Finite-element method

Stiffness of supports (soil-structure interaction)

Structural dynamics

Plastic Methods of Analysis

1. Limit analysis

2. Lower bound methods

Hillerborg strip method

Moment redistribution

Upper bound methods

Yield line analysis

Limit analysis

It is useful to distinguish between the terms limit analysis and limit state design. Limit analysis is a means of assessing the ultimate collapse load of a structure, whereas limit state design is a design procedure which aims to achieve both acceptable service load behaviour and sufficient strength. Thus limit analysis can be used for calculations at the ultimate limit state in a limit state design procedure.

A concept within limit analysis is that it is often not possible to calculate a unique value of the collapse load of a structure. For the majority of commonly used structural forms, the collapse load will usually fall between so called upper and lower bounds. It is thus necessary to consider two distinct types of analyses within limit analysis, namely upper and lower bound methods.

An upper bound method can be unsafe in that it provides a value of the collapse load which is either greater than or equal to the true collapse load. The procedure for calculating an upper bound to the collapse load can be thought of in terms of proposing a valid collapse mechanism and equating the internal plastic work to the work done by the external loads.

A lower bound method is safe in that it provides a value of the collapse load which is either less than or equal to the true collapse load. Any elastic method of analysis is a lower bound method, in terms of limit analysis, because it satisfies equilibrium. In this sense equilibrium implies that the bending moments (M_x, M_y), twisting moment (M_{xy}) and shear forces, all per unit length of a plate element, are in equilibrium with the applied load.

Comments on some of the methods of analysis

Note: Refer to References 5.1; 5.2 and 5.3 for more detailed background

1. Influence Surfaces

Various authors have produced a number of sets of influence surface coefficients which are particularly useful for determining load effects from point loads such as wheels on bridge decks. The influence surfaces have generally been derived experimentally or by means of finite difference techniques.

Orthotropic Plate Theory

An orthotropic plate is one which has different stiffnesses in two orthogonal directions. Various methods of analysis are available which offer solutions to bases, slabs and walls in useful tabular form, based on finite difference equations, for uniform and linearly varying loads applied to members supported on two, three or four sides.

Grillage Analysis

In grillage analysis, the structure is idealised as a grillage of interconnected beams, without simulating the Poisson's ratio effects of continua. The method should strictly be used for grillage structures only. However, the method is very popular as it is easy to apply and yields results which are generally sufficiently accurate for the design of most bridge decks.

Finite-element Analysis

The finite element method of analysis is considered the most versatile of the available methods and, in principle, can solve almost any problem in two way spanning elements and for bridge deck analysis. There are numerous excellent finite-element packages available on the market, many of which are advertised in technical journals.

Stiffness of Supports: Soil-structure Interaction

Bridge decks are supported by piers and abutments, which in turn are supported by foundations resting on soil. The stiffness of piers, abutments, foundations and of soil are all significant in analysing the bridge for safety. Reference 5.2 provides useful guidelines in this context.

Structural Dynamics

References 5.1 and 5.2 provide guidance for the dynamic behaviour of bridge decks under the action of various types of dynamic loads.

Hillerborg Strip Method

This is an inelastic lower bound method as simply explained with a useful example of the design of two way spanning abutment walls in Reference 5.3. The method is not well suited to analysis for point loads.

Yield Line Analysis

Yield line analysis is a very versatile method in that it is amenable to any form of loading or plate configuration. However, this method should not be used for new designs but is well suited to checking existing structures, provided information about the reinforcement is known or can be obtained.

The method is based on the assumption of simultaneous formation of plastic hinges at postulated positions and on the further assumption that the structure is sufficiently ductile to permit this to happen. These assumptions may lead to violation of serviceability considerations as cracking may occur at certain locations prior to all yield lines forming through redistribution of load effects.

A further difficulty with this method is that the designer cannot be certain that the postulated yield line pattern is the most critical for the particular structural configuration. Other yield line patterns may indicate higher plastic bending moments, or conversely lower collapse loads.

Reference 5.3 provides valuable information on this method of analysis together with several useful design examples. Yield line analysis should not be used for the checking of members of varying thickness or depth. In these instances the designer should employ grillage or finite element analyses.

References

5.1 Menn C, PRESTRESSED CONCRETE BRIDGES, Birkhäuser Verlag, Basel (1990)

ISBN 3-7643-2414-7

ISBN 0-8176-2414-7

5.2 Shamnugam NE, Narayanan R, STRUCTURAL ANALYSIS, ICE Manual of Bridge Engineering (Second Edition), Parke G (Ed) and Hewson N (Ed), Thomas Telford, London (2008)

ISBN 978-0-7277-3452-5

5.3 Clark LA, CONCRETE BRIDGE DESIGN TO BS 5400, Construction Press, London (1983)

ISBN 0-86095-893-0

5.4 22 Authors, BRIDGE AESTHETICS AROUND THE WORLD, US Transportation Research Board National Research Council, Committee on General Structures: Subcommittee on Bridge Aesthetics, Washington DC (1991)

ISBN 0-309-05072-3

5.5 UK Highways Agency, Design Manual for Roads and Bridges (BA 41/98), THE DESIGN AND APPEARANCE OF BRIDGES, London (1998)

5.6 fib Bulletin 32, GUIDELINES FOR THE DESIGN OF FOOT BRIDGES, Lausanne, Switzerland (2005)

ISBN 2-88394-072-X

Preliminary Design Phase

General

The main aim of the preliminary design phase is to identify a bridge proposal which best suits the FMW's objectives and the primary goals of bridge design as illustrated in Figure 5.1, with due regard to the determinants outlined in Chapter 2.

The preliminary design process should be carefully planned as it may well overlap with the collection of important information which materially affects the design. This process should therefore be managed to ensure that important determinants are not inadvertently overlooked in the formulation of the design concept, which could lead to wasted effort later.

It should be noted that bridge design involves a continuous process of optimisation and refinement, which ends only with the compilation of the bridge working drawings. The guidelines within this chapter should therefore not be rigidly applied, on the understanding that design is an iterative process which may require a certain degree of backtracking. During the preliminary design phase it is therefore preferable only to identify a sound bridge concept which is deemed superior to the other alternatives in most respects, and to make further improvements during the final design phase.

One of the determinants which can easily be overlooked is that of the courtesy of keeping the bridge designer's counterpart in the FMW in the picture with regard to the line of thinking at an early stage in the formulation of design concepts, rather than to take a fait accompli approach which could lead to disagreement on presentation of the proposal report. Such consultation should not be viewed as abrogation of the bridge designer's duties and responsibilities but rather as sensible communication regarding progress and agreement in principle about the concepts being considered.

As previously noted in Chapter 1, this Volume is aimed particularly at the design of small to medium span concrete bridges. Projects which involve the design of large span bridges are not at this stage catered for, in which case the bridge designer should refer to appropriate specialised literature.

Concept Formulation

The factors which influence the selection of potentially suitable bridge types and which should be considered at this stage are those outlined in Chapters 2 and 3, which include the following:

1. Structure geometric envelope

   The geometric envelope essentially defines the geometric boundaries within which or beyond which the bridge components must be constructed, including hydraulic requirements in the case of river bridges. The minimum elevation of the bridge deck soffit in particular is defined by this envelope and the limit positions of the abutments are indicated, with some freedom depending on the selection of closed or open type abutments.

The permissible positions of piers is indicated in the case of bridges spanning over roads or railway lines, but is dependent on hydraulic considerations and overall economy (alternative span lengths) in the case of river bridges.

Nature and magnitude of the traffic and other imposed loads to be carried

The type of traffic to be carried must be confirmed by the FMW.

Obstacles to be crossed

These are self-evident from the design brief and confirmed by the site inspection and topographical survey.

Founding conditions

Whereas founding conditions are deemed in Section 2.6.4 to constitute part of the structure geometric envelope it may be necessary to adjust the foundation type during the preliminary design investigations.

Feasible construction methods

Feasible construction methods have a significant influence on the cost of bridge construction and in turn are dependent on numerous other factors.

Accommodation of traffic

The accommodation of traffic is essentially a component of the obstacles to be crossed during the construction stage.

Feasible Structural Alternatives

Selection of the primary structural form

Having determined the boundaries imposed by the structure geometric envelope, the following preliminary parameters should be established in preparation for the selection of potentially suitable alternatives:

Total bridge length and width; minimum deck soffit elevation; skew and curvature.

Possible positions of substructure components and span lengths.

In the case of river bridges the precautions recommended in Section 3.3.3 of Chapter 3 should be observed with regard to the location of the abutments, and countermeasures should be planned against the effects of scour as recommended in Section 3.3.4 of Chapter 3.

Founding conditions should be evaluated to identify possible foundation types and founding levels. Should more than one foundation type be suitable, these should be selected as part of alternatives to be compared against each other as described in Section 6.4.

Feasible construction methods are dependent on factors such as the location of the structure, the nature (urban, rural, dry land, water environment etc.) and topography (flat, steep etc.) of the bridge site, ease of access for major plant and equipment, each of which should be considered in the selection of the most advantageous type of deck construction e.g. in situ or composite with precast elements.

The need for the accommodation of road or rail traffic on busy routes will also have an influence on the method of construction. Limited height for the installation of traffic windows in falsework over heavily trafficked roads and restricted occupation durations for work over electrified railway lines will favour precast concrete construction, for example.

Feasible alternative structural forms should be outlined in sketches at this stage, with due regard to the influence of the foregoing determinants.

Preliminary configuration of components

Following selection of the feasible alternatives, details of the major components should be developed, as follows:

1. Deck (depending on span lengths etc.)

1. Cast in situ or precast elements.

2. Slab, voided slab, beam and slab, box girder.

3. Continuous or simply supported.

4. Reinforced and/or prestressed concrete.

For the reasons outlined in Section 4.2.3, continuous bridge decks offer several advantages over simply supported decks and should be favoured if practical. It should also be noted that for any given bridge opening and deck configuration, there is an optimum span length and support configuration in terms of total cost. This may require the preliminary selection and costing of several span arrangements in order to identify the preferred span length and cost premium.

Abutments

Options most frequently used include:

Bank seat (perched) types.

Spill through (open) types.

Closed types (vertical cantilever or mechanically stabilised up to about 10 m height; counterfort or cellular above about 12m height; special types including ground anchors for very high walls).

Deck articulated at abutments, or integral with abutments.

Reinforced concrete in all cases.

The low maintenance advantages of bridge decks integral with abutments (integral bridges) have been proven, and should always be considered for bridges up to about 60m length and skews of less than about 20).

Piers

Pier shapes and types are highly varied and include:

Circular, rectangular or featured columns or walls.

Singly, pairs or groups.

Integral with decks or articulated at the deck soffit by hinges or bearings.

Column or wall caps when decks have precast beams.

Special tapered column designs for very tall piers (above about 25m).

Reinforced concrete in all cases.

The methods of pier construction include conventional formwork up to heights of about 30m, and climbing or sliding shutters for heights greater than about 35m. The elimination of bearings at the tops of piers in favour of concrete hinges or piers monolithic with the deck offers the advantage of little or no maintenance or replacement of bearings during the service life of a bridge, and should therefore be favoured if structurally feasible.

Foundations

Conventional spread or spot footings on in situ material (of adequate bearing capacity), alternatively

Piled foundations (precast driven, augered and other types), or

Caissons or shafts.

The choice of foundation type is dependent on many factors such as depth, type and bearing capacity of soils, access and available construction methods.

Simplified analysis of member forces

It is not necessary to carry out complex analysis of the member forces at this stage and the bridge designer should have a good understanding of the flow of forces and the behaviour of structures so that simple structural components may be envisaged and analysed. The bridge designer should do various checks at critical points using simple but conservative methods. The analysis of a continuous bending elements, for example, may be done by assuming less than full fixity at the supports and treating each span separately. Appropriate structural software programmes may be used, but the emphasis should be on simplifying the computer models with scope for further development at the detailed design phase.

Preliminary member sizing

The preliminary depth of the deck may be determined by using suitable span/depth ratios for the preliminary span lengths and types of deck chosen. For initial purposes span/depth ratios ranging from 15 to 22 for simply supported and continuous decks respectively, may be used. Other depths may be required for incrementally launched decks for which specialised knowledge is required. The sizing of pier columns largely depend on allowable slenderness ratios and aesthetic qualities.

Preliminary sizing of bridge members is largely dependent on the skill and experience of the bridge designer. The comparison of a proposed structure with similar existing bridges is helpful, even for the more experienced bridge designer.

Preliminary quantities and estimate of cost

A database of bridge construction rates per square metre is essential for cost estimation during the initial phases of preliminary design. If the database is up to date, sufficiently accurate estimates may be made by applying rates from similar bridges. The types of foundation proposed could have a bearing on the estimate and should be considered.

At the stage when the favoured scheme is submitted to the FMW, the bridge designer should prepare a separate schedule of quantities and insert the latest available rates with adjustments for possible price increases. The relative costs of each alternative scheme may now be used for further evaluation.

Comparative Evaluation of Alternatives

It is recommended that a minimum of two but preferably three alternatives should be identified for appraisal in terms of the following criteria, in preparation for further development of the favoured scheme and submission for approval.

Strength and Safety

All facets of the perceived strength and safety of the alternatives should be evaluated for comparison and rating in terms of relative superiority.

Durability and serviceability considerations

A similar evaluation should be carried out for these criteria.

Economy and constructability

Preliminary cost estimates permit an immediate rating in terms of construction costs. However, the alternatives should be further evaluated in terms of potential life cycle costs of those components of the designs which are likely to require greater maintenance inputs or more frequent replacement (e.g. bearings and joints).

Aesthetic considerations

In general, structures which are slender, transparent and simple, with visually balanced component proportions are likely to be the most aesthetically pleasing. The alternatives should firstly be compared on this basis and thereafter evaluated in terms of particularly striking and innovative but tasteful features. Finally, if necessary, the more esoteric aspects of aesthetics, as outlined in Chapter 5 should be considered before ranking the overall aesthetic quality of each alternative.

Development of the Favoured Scheme

Submission of proposal drawing, report and cost estimate

The alternative preferred on the basis of the foregoing evaluation criteria should be further developed on a preliminary proposal drawing. This drawing together with a more accurate cost estimate of the proposal should be submitted to the FMW for approval in principle, together with a report on the grounds for the recommendation.

FMW approval in principle

Subject to approval in principle by the FMW, the preliminary proposal drawing should be amended to reflect any changes required and resubmitted for record purposes. Only after the FMW has approved the preliminary proposal in principle and instructed the bridge designer to proceed to the detailed design phase should the design be further developed to completion.

Detailed Design Phase

Introduction

By this stage of the planning and design process the type and configuration of the proposed bridge will have been established as generally outlined in Chapter 6, with due regard to the boundary conditions imposed by the site and the highway requirements. Approval in principle of the proposal and authority to proceed with detailed design should also have been received from the FMW.

The task facing the bridge designer is to further develop the preliminary design through a process of optimisation and refinement, without altering the basic form of the bridge originally proposed.

This is the stage during which the achievement of the design goals outlined in Chapter 5 should be the prime objective. It is also the stage during which the practical guidelines described in Chapter 8 should be implemented as appropriate, in order to facilitate high quality construction.

Before proceeding with the detailed analysis and design of a bridge, it is essential for the bridge designer to confirm that the geometry on which the preliminary bridge proposal was based is still compatible with the latest edition of the road drawings in all respects. Failure to carry out such a check may result in wasted work or other consequences later.

Issues which affect Analysis and Design

At the outset of this phase it is necessary to also reconfirm the various loads and induced effects which the structure will be required to resist, and to record these in a design statement as part of the formal design calculations and for later inclusion on the general arrangement drawing.

This statement should provide a summary of the applicable permanent and transient loads as shown on Figure 2.1 of Chapter 2. It is also desirable to summarise the intended materials strengths and characteristics for the various bridge components i.e. concrete, normal reinforcing steel, prestressing system including anticipated losses etc.

It is anticipated that the analysis will follow the normal sequence, starting with the superstructure and proceeding downwards to the substructure components and ultimately to the foundations.

Structural analysis consists essentially of simplified mathematical modelling of the response of a structure to the applied loading. Such models are based on idealisations of the structural behaviour of the material and are, therefore, imperfect to some extent, depending on the simplifying assumptions in modelling. Consequently the assessment of structural responses is the best estimate that can be obtained in view of the assumptions implicit in the modelling of the system. Some of the assumptions are necessary in the light of inadequate data; others are introduced to simplify the calculation procedure to economic levels.

It must also be recognised that the design loads employed in assessing structural response are themselves approximate. The method of analysis chosen should be adequate for the purpose and capable of providing the applicable sectional forces at an economical cost, for subsequent use in determining or verifying the adequacy of the sections in terms of BS5400.

In the case of bridge decks it is necessary to select a method of analysis which is based on structure modelling compatible with the stiffness of the deck in both the longitudinal and transverse directions, with particular regard to the transverse distribution of the traffic loads.

Consideration should also be given to methods of analysis in which the modelling yields easily understandable and useable results for:

Long continuous bridges which are constructed in stages, during which the method of construction including the effects of construction loads and temporary supports come into play.

Bridges which are curved in plan, in which special consideration of the type and orientation of the bearings become important.

Bridges with high degrees of skew, in which special attention must be paid to the configuration of the model with regard to the orientation of the deck slab transverse and longitudinal reinforcement, as well as the behaviour of the deck in the acute corners and the differential loads on the bearings, especially at the abutments.

In the case of bridges with high piers and abutments with varying degrees of stiffness, analysis is required to establish the deck 'fixed' point in order to determine the deck, bearings and substructure movements and rotations under the influence of the deck temperature, shrinkage and creep deformations. Likewise the transfer of the traction and braking forces on the deck via the bearings to the piers and abutments must be analysed.

Discussions about the most appropriate methods of analysis which follow are comparatively brief. Bridge designers who require more detailed guidance about the merits of alternative methods of analysis and structure modelling are advised to study References 7.1, 7.2, 7.3 and 7.4 in particular.

Superstructure Analysis and Design

Methods of analysis

A number of the developers of modern structural analysis and design programs offer suites of software packages which embody most of the methods of analysis outlined in Section 5.7 of Chapter 5, together with methods of design based on the design Code of Practice of choice i.e. AASHTO, Canadian, BS5400 etc.

Many of these programs embody influence line and influence surface modules in order to maximise traffic load effects for combination with permanent and other loads, in order to establish maximum bending moment and shear envelopes both longitudinally and transversely. Whereas these programs eliminate the need for lengthy manual calculations involving complex mathematics, the benefit of developing simple in-house spread sheets for minor design aspects and some checking procedures should not be discounted.

1. Common Methods of Analysis

All methods of design should be lower bound as set out in Section 5.7 of Chapter 5, and must satisfy equilibrium and suitably chosen compatibility conditions. The most commonly used methods are Line Beam, Plane Grillage Analysis and Finite-element Analysis.

1. **Line Beam Models** usually consist of an assemblage of beam elements. The entire superstructure can be modelled as a single beam, provided the superstructure can deflect under load without distortion of the cross-section, failing which, the superstructure must be modelled as an assemblage of several beams.

   Single cell box girders and double-T girder decks can always be modelled as single beams, provided the span length is large in relation to the cross-section dimension, in compliance with the following inequality (:

$$\frac{l_o}{2\,(b_o + h_o)} > 1$$ .... **Equation** Error! No text of specified style in document.**-7**

Where b_o and h_o are defined in Figure **Error! No text of specified style in document.**.19 and the effective span length l_o is defined as follows:

1. For simply supported spans: l_o is equal to the span length.

2. For cantilever spans: l_o is equal to twice the cantilever length.

3. For continuous spans: l_o is equal to the length between inflection points.

Figure **Error! No text of specified style in document.**.19: Idealised cross-section dimensions

When the superstructure is modelled as a single beam, the external loads are equilibrated by bending moments *M*, shear forces *V*, and torsional moments *T*, denoted collectively as *sectional forces*. An eccentrically applied load can be resolved into symmetrical and antisymmetrical components. The symmetrical component induces bending moments and shear forces. The antisymmetrical component, a force couple, is equivalent to an external torque M_t (concentrated loads) or m_t (distributed loads). Torque induces torsional moments in the structure.

Cross-sections can be classified into two groups, according to the mechanism by which torsional moments are resisted. *Closed sections* (e.g. hollow boxes) resist torsional moments by means of a closed shear flow; *open sections* (e.g. T-sections) resist torsion primarily by bending moments in the webs.

Plane Grillage Analysis is one of the methods suitable for use when the deck cross-section is subject to significant deformation due to transverse bending. This method is widely used for bridge deck analysis and is inexpensive and easy to use and comprehend.

The deformability of the cross-section is a function of several factors, including the form of the section, stiffening elements such as diaphragms or cross-beams, ratio of span length to cross-section width, support conditions, and load arrangement. Transverse bending is generally present in wide solid slab or voided slab decks, beam and slab decks with three or more beam webs, pseudo box decks, multi-cell box girder and twin spine beam decks. These cross-sections must therefore be modelled as plane grillages, or by finite element analysis as discussed below.

The plane grillage method involves the modelling of a bridge deck as a skeletal structure made up of a mesh of beams lying in one plane.

The basis of the modelling and analysis using this method is well explained in References 7.2 and 7.4, in which its application to the various forms of deck listed above are described, as well as the particular issues to be considered in the cases of highly skew decks.

Finite-Element Analysis

Notwithstanding the benefits which traditionally favoured grillage analysis, in today's environment of inexpensive, high-powered computers coupled with elaborate analysis programs and user-friendly graphics interfaces, the finite-element method has begun to replace the grillage method in many instances, even for more straight forward bridge decks.

The finite element method is a technique for analyzing complicated structures by notionally cutting up the body of the structure into a number of small elements which are connected at discrete points (nodes). The basic principle of subdivision of the structure into small, simple elements can be applied to structures of all forms and complexity and allows the method to be used without limit to the type and form of structure. This is the most versatile method of analysis and is easily applied. However, the use of this method is subject to the numerous different theoretical formulations of element stiffness characteristics and should be used with circumspection.

Design of reinforced and prestressed concrete bridges

Bridge designers are advised to refer to the following References 7.3, 7.5 and 7.6 for detailed guidance about the design of reinforced and prestressed concrete bridges in compliance with the requirements of BS5400.

Design of Bearings

Comprehensive information about all facets of the design of bridge bearings is available from References 7.7 and 7.8.

Function

The function of a bridge bearing is to:

Transmit loads from the superstructure to the substructure

Provide restraint to specific directional or rotational movements

Allow freedom of horizontal or rotational movements in specific directions

Bridge bearings are typically required to transmit vertical loads from the superstructure to the substructure supports, with specific restraint or freedom of movement in the horizontal plane in the designated longitudinal and transverse directions. Restraint of movement in any direction is associated with the transmission of a horizontal force from the superstructure to the particular support in that direction.

The design of the bearings for a bridge therefore reflects the degree to which the designer has decided to articulate the structure as a whole (continuous or simply supported spans) and the locations at which it is intended to resist longitudinal or transverse forces imposed by the superstructure on the substructure components.

Concrete hinges fall into the same category as bearings. Bridge designers are advised to refer to Reference 7.9 for the design of hinges.

Types and configurations

The most commonly used bearings include the following:

Elastomeric bearings

Line rocker bearings

Pot bearings

Spherical bearings

The use of several layers of roofing felt or other sheet material may be used for small bridges with minimal rotational or horizontal translation movements. Several other types of bearings have fallen into comparative disuse and are not discussed here.

Movement and restraint

Bearings are classified in accordance with their ability to restrain or allow movements as follows:

Fixed bearings which allow no horizontal movement in any direction

Unidirectional bearings which allow horizontal movements in one direction only

Multidirectional bearings which allow horizontal movement in all directions

Each of the bearings mentioned in 7.3.2 have rotational capacity in all directions, with the exception of line rocker bearings which only allow rotation about one axis.

Loads and movements

Loads and movements are determined by the analysis of the effects of various actions applied to the selected form of the structure and the degree of articulation required by the designer. The following categorizations of actions apply:

Permanent and long term actions including dead load (superstructure self-weight, superimposed dead loads, earth pressure, differential settlement of foundations, prestressing effects, creep and shrinkage).

Short term actions including temperature range applicable to the structure, the temperature gradient through the individual members and construction loads.

Transient actions including traffic loads, braking and traction forces, centrifugal forces, wind loads, water forces, vehicle impact and earthquake action.

Normally standard bearings with designated allowable movements and load capacities are available from the manufacturers who supply brochures which list their range of standard steel bearings of the different types (fixed, unidirectional, multi directional). These brochures may be used to confirm practical bearing seat sizes and levels and to design bursting reinforcement. The exception is the design of elastomeric bearings which are required to be designed in terms of BS5400 Part 9.1.

The bridge designer is required to determine the most unfavourable combinations of the maximum and minimum vertical loads and co-existent horizontal loads, together with the maximum values of the reversible (e.g. temperature) and irreversible movements and rotations (e.g. shrinkage and creep).

Table **Error! No text of specified style in document.**.5 provides an example of the tabular information required on the drawings for each of the bearings, which summarises the combinations of the most adverse vertical and horizontal forces to be resisted by the bearing, together with the positive and negative movement range required along appropriate axes, the rotational movements about the axes, and the amount of the required preset of the bearings in the longitudinal direction.

Table **Error! No text of specified style in document.**.5: Typical bearing design data

Adaptor plates

The use of adaptor plates attached permanently to the superstructure and the bearing seat to facilitate replacement of steel bearings should be mandatory.

Plan layout

The judicious selection of fixed, unidirectional and multidirectional bearings in their correct positions and orientations are essential to allow the deck to move and rotate while maintaining a stable position on the substructure. Generally, the superstructure should be allowed to expand and contract in the longitudinal direction whilst maintaining its position at a designated fixed point. Similarly, freedom of transverse movements should be controlled by lines of restraint.

A general principle is that the allowable movement of a unidirectional bearing should be orientated towards a fixed bearing. A typical layout is illustrated in Figure **Error! No text of specified style in document.**.20.

Elastomeric bearings allow a limited amount of transverse movement and rotation. Fixity of elastomeric bearings may be achieved by the use of fixing dowels between the superstructure and the substructure.

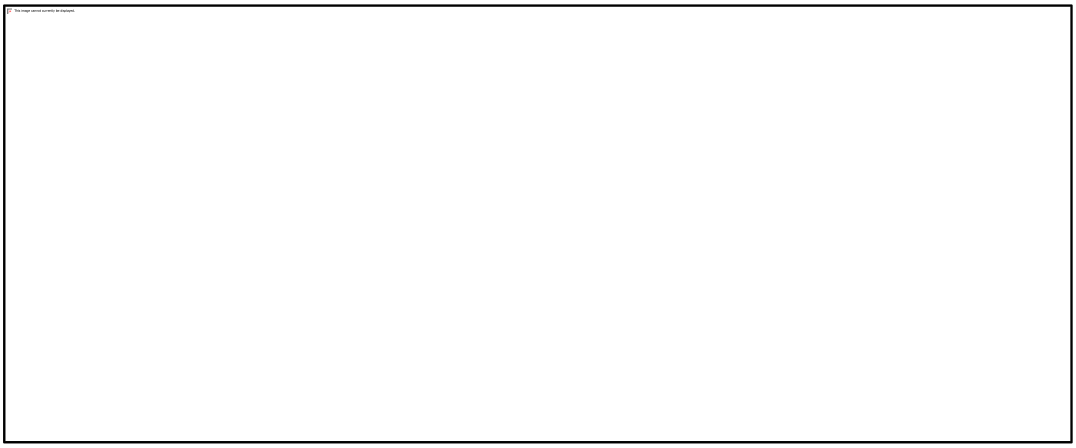


Figure **Error! No text of specified style in document.**.20: Typical Bearing Layout

Design of Abutments and Retaining Walls

Form and function

The alternative forms of bridge abutments and retaining walls are illustrated in Figure **Error! No text of specified style in document.**.14 of Chapter 4, and the function of these structures is described in the same chapter. For the purposes of this section two principal forms of abutments are discussed viz: open (or spill through) type abutments and closed abutments.

Most of the abutments illustrated in Figure **Error! No text of specified style in document.**.14 are of the closed type, in which the embankment is fully retained for the full height from road level to the underside of the base (or pilecap). Integral and semi-integral abutments are special types in so far as the bridge deck provides part of the horizontal restraint to the imposed earth or surcharge pressures. In the case of mechanically stabilised walls, earth pressure is resisted by various types of ties anchored in the embankment behind the wall. For information about the behaviour and design of integral, semi-integral and mechanically stabilised abutments the bridge designer should refer to Reference 7.9.

Spill-through abutments usually retain the embankment for the upper portion only; the lower portion of the wall (i.e. embedded columns) only partly retains the embankment.

The question which therefore arises is: what earth or surcharge pressures are imposed on the lower spill-through portion of the abutment? Whereas there are various theories about this matter, a basis for the estimation of the earth pressure forces in this context is provided below.

In this section attention is focussed on:

The various loads or forces (and combinations of these) imposed on bridge abutments.

Safety against overturning or sliding of abutments (i.e. stability).

Estimation of bearing pressures beneath conventional spread footings. (Piled foundations are dealt with separately in Section 7.7 of this Chapter).

The reinforced concrete design of the bases, columns, counterforts, walls and beams etc. of abutments is described in numerous specialist publications, to which the bridge designer should refer.

Reinforced concrete cantilever and counterfort retaining walls are a simplified version of the equivalent abutments and are therefore not dealt with additionally. However, some guidance is provided about the preliminary design of horizontally cantilevered wingwalls (earwings) in Section 7.5.5.

Forces on abutments and retaining walls

Typical diagrams of the forces imposed on cantilever abutments (embankment surface level or minor slope) and retaining walls (sloping embankment) are provided in Figure **Error! No text of specified style in document.**.21 and Figure **Error! No text of specified style in document.**.22.

The forces which arise in the case of abutments (Figure **Error! No text of specified style in document.**.21) are as follows:

Vertical forces due to self-weight, compacted earth fill (toe and heel) and deck forces at bearing level. Intermittent surcharge due to compaction equipment during construction and due to traffic wheel loads in service.

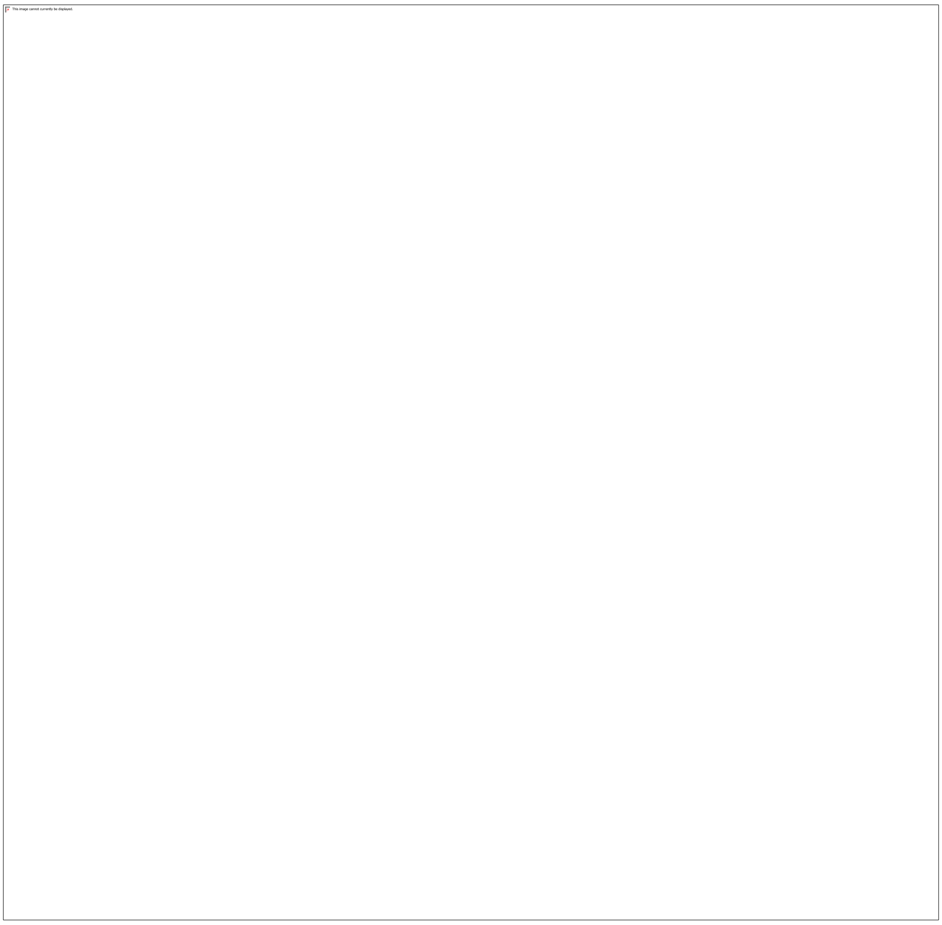
Horizontal earth pressure forces behind the wall (sometimes with a vertical component along plane HH_1 depending on soil characteristics: usually ignored), and due to traffic surcharge on the embankment.

Horizontal forces due to deck loads, applied at bearing level.

Flood forces including water pressure on the wall front face (if applicable), followed by water pressure on the earth face if the embankment is saturated and not adequately and rapidly drained (if applicable).

The bridge designer must consider the realistic combinations of co-existent maximum and minimum forces with regard to stability, bearing pressure (including settlement if applicable) and the design of the components. If a decision has been made to permit construction of the embankment up to bearing seat level to facilitate construction of the deck, this will constitute an early stage load case which will usually not be found to be critical, but which should nevertheless be checked.

In the case of an in situ prestressed concrete deck it will be feasible to complete the embankment and road pavement layers only following completion of the deck prestressing operations.



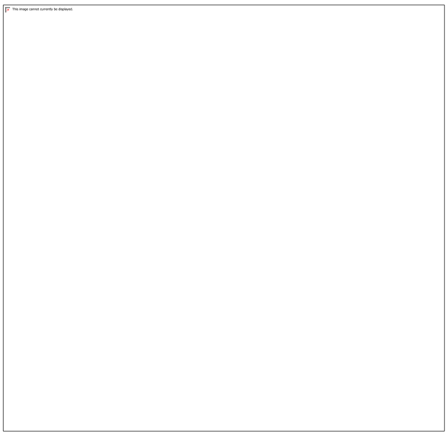
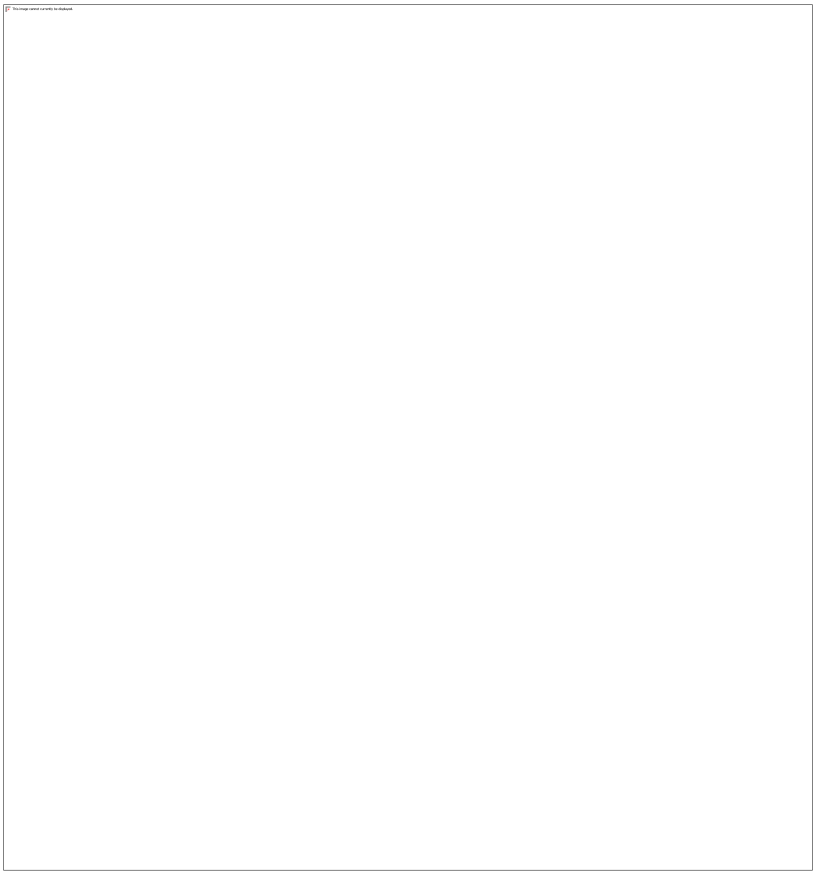


Figure **Error! No text of specified style in document.**.21: Forces acting on an abutment

The forces which arise in the case of a freestanding retaining wall are similar to those of abutments, with the obvious exclusion of deck vertical and horizontal forces. When the embankment behind a retaining wall is sloped as shown in Figure **Error! No text of specified style in document.**.22, for the purposes of the stability and bearing pressure calculations the earth pressures (horizontal and vertical components) must be applied to the entire vertical plane HH_1. However, for the design of the wall stem the earth pressure (excluding a vertical component) must be applied only to the actual height of the wall above the base.



NOTATION:

ΣV : Total vertical loads
ΣP : Total horizontal loads
P_h : Horizontal component of earth pressure
P_v : Vertical component of earth pressure
R : Resultant $(\Sigma V^2 + \Sigma P^2)^{½}$
T : Footing toe
H : Footing heal
$\left.\begin{array}{l} x \\ y \\ z \end{array}\right\}$: Lever arms
Bearing pressure reactions not shown

Figure **Error! No text of specified style in document.**.22: Forces acting on a retaining wall

Earth Pressure

This is an aspect of abutment and retaining wall design which is briefly outlined in Reference 7.10 and which justifies careful consideration on the part of the bridge designer.

Whereas it is common practice to assume active earth pressure against retaining structures on the grounds of granular fill and the ability of the structure to rotate sufficiently about its base or otherwise deflect away from the embankment, in the absence of these conditions the imposed pressures may be closer to those of the 'Soil at Rest' conditions, which is discussed in detail in Reference 7.11 and similar soil mechanics publications.

For example, on the assumption of a compacted soil density **v** of 19 kN/m^3, and various values of the angle of internal friction of the soil Ø, the following comparable soil pressures p = k.**v** **(**kN/m^3/m) arise:

Ø	Active (Rankine) $k_a = \dfrac{1 - \sin\phi}{1 + \sin\phi}$	At Rest (Jaky) $k_o = 1 - \sin\phi$
27°	p = 7,13	p = 10,37
30°	6,33	9,50
33°	5,59	8,65

If the bridge designer assumes well drained granular fill in the design calculations it is important to ensure that this will be achieved during construction. An embankment comprising a high percentage of cohesive materials may well result in earth pressures greatly in excess of the assumed values.

In the case of retaining walls with sloping fill, it is recommended that the Coulomb theory of earth pressure be adopted, in which the horizontal and vertical components of pressure act on the full height HH_1 shown in Figure **Error! No text of specified style in document.**.22.

In the case of rigid cellular abutments founded on rock, it is recommended that Soil at Rest earth pressures be adopted.

Stability and Bearing Pressure

1. Overturning

With reference to Figure **Error! No text of specified style in document.**.21 (or Figure **Error! No text of specified style in document.**.22) the moments acting on an abutment or a retaining wall are usually considered about the toe point T and comprise the total restoring moment ($\Sigma M_R = \Sigma V.x$) and the total overturning moment ($\Sigma Mo = \Sigma P.y$). The required factor of safety against overturning is achieved when the following inequality) applies.

$$FOS_O = \frac{\Sigma \boldsymbol{M_R}}{\Sigma \boldsymbol{M_O}} \text{ P } 1.5$$

Equation **Error! No text of specified style in document.**-8

It is usual to ignore any relieving earth pressure imposed on the front of the wall in checking this factor of safety.

Sliding

For a coefficient of sliding friction μ. the required factor of safety against sliding is achieved when the following inequality (applies:

$FOS_S = \frac{\mu.\Sigma V}{\Sigma P}$ P 1.5 Equation **Error! No text of specified style in document.**-9

When FOS_S <1.5 either the base width must be enlarged to increase ΣV, or an anti-sliding key should be incorporated beneath the base, to mobilise passive resistance against the vertical face of the key. It is undesirable to detail an anti-sliding key adjacent to the toe, where the ground in front of the base is prone to disturbance.

Bearing Pressure

The forces acting on an abutment or retaining wall result in the reactive bearing pressures beneath the base, and may be derived from which define the point at which the line of action of the resultant R intersects the underside of the base, i.e.

$\bar{x} = \boldsymbol{x} - \frac{\Sigma \boldsymbol{Mo}}{\Sigma \boldsymbol{V}}$ (where x, $\Sigma \boldsymbol{M_O}$ and ΣV are known) Equation **Error! No text of specified style in document.**-10

e = $\frac{\boldsymbol{B}}{\boldsymbol{2}} - \boldsymbol{x}$ Equation **Error! No text of specified style in document.**-11

Two cases of bearing pressure: e < B/6 and e > B/6, are illustrated in Figure **Error! No text of specified style in document.**.23 and are accompanied by the relevant toe and heel pressure Equation 7-7 or toe pressure Equation 7-6, respectively.

Equation **Error! No** Equation **Error!**

Figure **Error! No text of specified style in document.**.23: Bearing pressure configurations for varying eccentricity

It is preferable to proportion abutment and retaining wall bases to avoid high eccentricity (e > B/6) and thus reduce the potential for rotational settlement. If the maximum toe pressure exceeds the permissible bearing pressure of the foundation material, either the base width must be increased and/or the position of the wall on the base should be adjusted.

There are numerous software packages on the market which carry out analysis and design of all forms of abutments and retaining walls. However, it is often beneficial to develop in-house spread sheets for the purpose of preliminary proportioning and design of these components.

Spill-through abutments

Spill through abutments consist of two or more buried columns joined at the top by a transverse cap beam which supports the deck, and supported either by a continuous base or by individual spot footings in cases of sound rock founding.

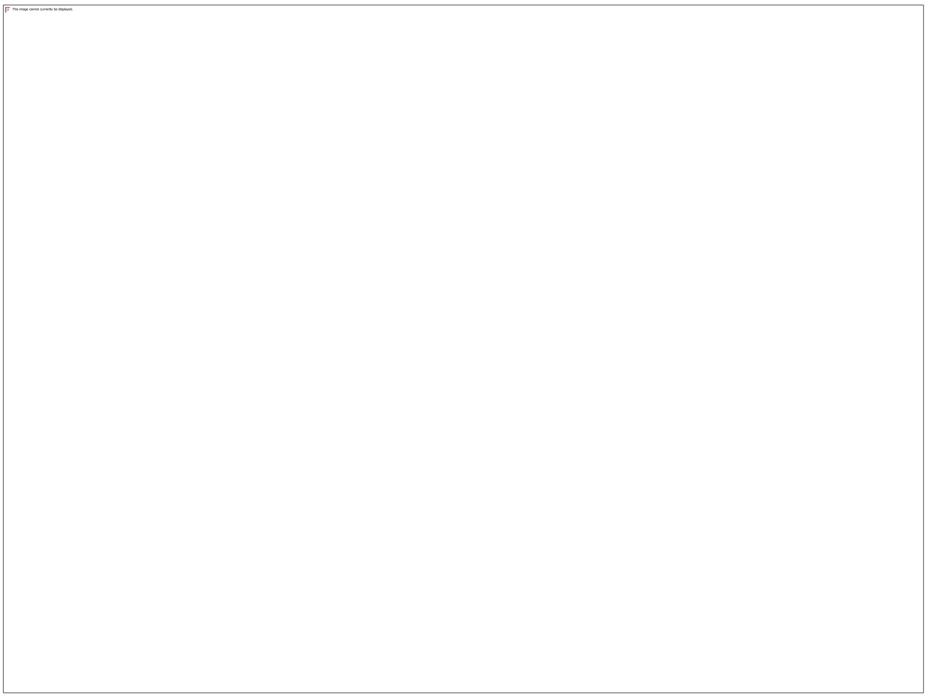
The columns are typically spaced at between 4m to 5m centre to centre and have minimum width of about 0.8m. In the direction of earth pressure the columns usually increase linearly in size from top to bottom, in which the 'depth' dimensions are proportioned in terms of vertical cantilever stiffness rather than strength only. The cap beam acts as a vertical beam in supporting the deck and resists earth pressure as a continuous beam horizontally. The cantilever stiffness of the columns is therefore of interest in the horizontal design of the cap beam. The horizontal forces imposed by earth pressure on the earwing walls at the outer edges (parallel to the road centre line) therefore also affect the horizontal moments and shears arising in the cap beam.

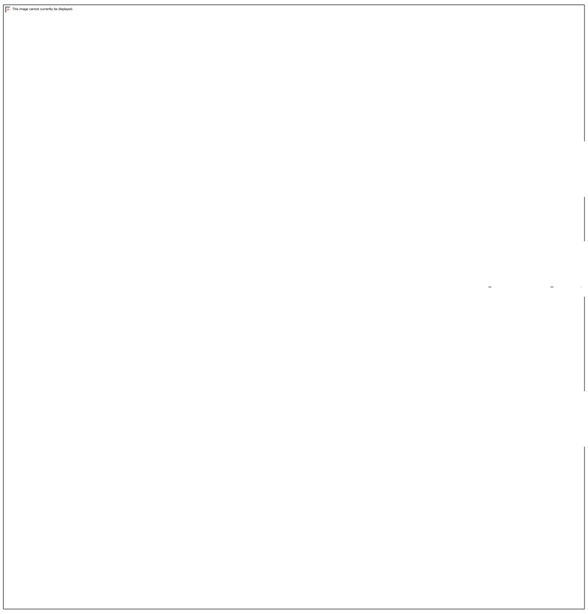
The top portion of the cap beam plus the breast wall is subject to earth pressure and traffic surcharge effects for the full width of the structure. From the underside of the cap beam to the top of the base/s it is recommended that active earth pressure should be applied to twice the width of the embedded columns, and that the relieving effect of earth pressure on the front of the columns should be ignored entirely.

When spill-through abutments have spot bases, for the purpose of confirming the stability of the abutment and calculating bearing pressures it is recommended that only the geostatic columns of earth directly above the bases should be considered as part of the stabilising forces. Reliance on additional earth loads arching between the bases is deemed to be unsafe.

Horizontally cantilevered wingwalls (earwings)

Earwing walls are subjected to variable earth pressures in proportion to their height and to surcharge pressures due to vehicles travelling close to the edge of the road. The following Figure **Error! No text of specified style in document.**.24 illustrates a typical wall of this type and includes Equation 7-10 to Equation 7-9 useful for the ultimate limit state design of these walls and the structure from which they cantilever.





Equation **Error!**

Equation Error!

Equation **Error! No text of**

Equation Error! No

Figure **Error! No text of specified style in document.**.24: Earth pressure applied to earwings

Design of Piers

Form and Loading

Piers act as vertical cantilever columns or walls, either articulated from the deck by bearings or hinges or monolithic with the deck. In the case of portal or strut frame bridges the piers comprise important vertical or raked portions of the primary structural system.

The following loads and forces are imposed on piers depending on the bridge type and location:

Hydraulic forces imposed by water pressure or the entrapment of debris, as outlined in Chapter 3 for river bridges.

Traffic impact forces which may have components parallel to and/or transverse to the direction of traffic flow beneath the bridge.

Wind forces and forces arising from seismic activity.

Vertical and horizontal forces imposed by the superstructure, with due allowance for intended or unintended eccentricity.

Bending moments and shears arising from frame action including the effects of restrained deformations imposed by the superstructure.

Slender columns

An important consideration in the analysis and design of slender pier columns and walls is the effect of buckling. BS5400 specifies minimum load eccentricity for the design of column and wall reinforcement. However, in cases when the bridge designer considers the potential deflection of pier columns to be significant, second-order analysis should be carried out.

Normal structural analysis software packages usually provide for such analysis. However, the bridge designer is encouraged to consult Reference 7.1, which provides very detailed explanation of second-order analysis of slender reinforced concrete columns, imposed deformations, special cases and flexible systems.

Relative stiffness and bearing restraints

In a multi-pier structure, the piers usually have differing stiffness arising from their various founding conditions, heights and bearing configurations. Whereas these circumstances will usually have little effect on the vertical loads imposed by the superstructure, considerable variations in the horizontal forces and deformations at the tops of the piers may arise, and require detailed analysis based on the individual stiffnesses of the piers and on the cumulative stiffness of the entire system.

Analysis and Design of Piles and Caissons

The use of piles and caissons as alternative foundation systems to conventional foundations is briefly outlined in Chapter 4.

Piles

Piles are slender structural members acting in groups and are installed vertically at a slope (rake) into the ground by various techniques (e.g. augering or driving). The resistance of the piles to the imposed loads is provided by end bearing on dense or hard material or by shaft friction or both.

Piles are normally analysed and designed by the Working Load method which includes a global factor of safety to cater for uncertainties in both the load effects and the pile/ground resistance. The bridge designer is reliant on the expertise of the geotechnical engineer for advice and guidance in this regard.

Piles groups supporting structures required to restrain horizontal forces in addition to vertical loads, include raking piles of which the slope should not exceed 1 in 4. In addition to the normal load effects for which pile analysis is carried out, there are other factors to consider of which the following are important:

1. Downdrag (negative skin friction)

   When the soil surrounding piles compresses slowly (consolidation) due to the stresses imposed by the embankment above, the piles are subjected to downdrag due to friction between the consolidating soil and the piles. This effect imposes additional loads on the piles and should be assessed with the assistance of specialist guidance. The potential effects of downdrag may be significant and should not be ignored. It is usual to de-rate the pile load capacity by the magnitude of the expected downdrag force.

Heave

When piles are installed in expansive soils subject to variation in moisture content, the upward forces acting on the structure may generate tensile forces in the pile shaft. If significant, these forces should be included in the design by the inclusion of appropriate reinforcement or by the installation of compressible materials such as polystyrene or straw bales under the pile caps. Measures to keep moisture content of the surrounding fill at constant levels are also a consideration. If possible, the material should be replaced with a suitable granular material.

Pile analysis and design

The analysis of a pile group involves the solution of a stiffness matrix comprising six unknowns for movements and rotations about the three conventional XYZ directions. In addition, the choice of fixity (fixed or pinned) at extremities of the pile has a bearing on the behaviour of the pile and should be carefully selected. The lateral resistance of the subgrade material also has a large effect. It can therefore be seen that the analysis can be complex and the designer is advised to employ the use of suitable pile analysis software. When not available, space frame analysis programs may be used where the lateral subgrade reactions are simulated by the introduction of lateral spring supports along the length of the pile shafts. The modulus of subgrade reactions for various soils types and conditions may be obtained from specialist literature.

The components of the resultant forces and moments acting on the pile cap should be according to the convention as used by the piling program. The convention normally followed is the "right hand screw rule" whereby the Z (vertical) axis is directed positively downwards and the moment vectors also follow the "right hand screw rule" for positive direction. All forces and moments must be resolved into components acting at the point of origin of the selected set of axes.

The data required for the analysis and design of the proposed scheme and for future record purposes includes the following:

Critical design-load combinations of the co-existent permanent and transient loads applied at the underside of the pile cap and the centroid of the pile group, relative to a defined system of axes, as shown in Figure **Error! No text of specified style in document.**.25 (tabulated on the drawings for each pile group). Load tables should clearly state whether the loads are service or ultimate.

Permissible movements which can be accommodated by the structure relative to the pile group centroid and system of axes, including displacements in the horizontal plane, deflection vertically, and rotation about any axis, as illustrated in Figure **Error! No text of specified style in document.**.25.

Information required for the calculation of down drag forces on piles when relevant.

Full details of the geotechnical investigation including borehole logs, test results, etc.

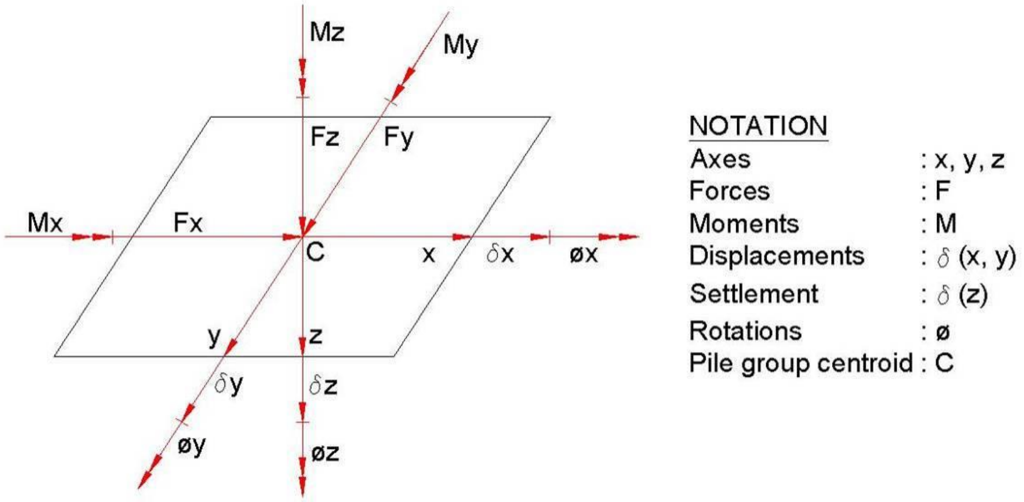


Figure **Error! No text of specified style in document.**.25: Illustration of Pile Group Design Data

Once the combinations of bending moments, shears and axial forces on each of the piles have been determined the design of the piles should be undertaken in terms of BS5400 for reinforced concrete columns.

Caissons

The bridge designer is advised to consult specialist literature for the design of caissons which are required to resist design load combinations in the same form as illustrated by Figure **Error! No text of specified style in document.**.25.

Design of Bridge Accessories

Expansion joints

The function of bridge deck expansion joints includes the following requirements:

Bridge the variable gap due to expansion or contraction of the deck in such a manner as to prevent unacceptable discontinuity in the riding surface of the carriageway.

Ensure the riding quality in terms of comfort and safety for all classes of road users for which the structure is designed.

Withstand the loads and movements due to traffic, vibration, impact, shrinkage, creep, settlement, etc. without causing excessive stresses in the joint of elsewhere in the structure.

Prevent the potential for skidding, excessive noise or vibration.

Prevent the ingress of water and debris, or to have provision for drainage and cleaning.

Be easy to clean and maintain.

Be economical in their total cost, including capital outlay and maintenance. The bridge designer should determine the range of maximum and minimum movements arising from temperature variations, creep, shrinkage, etc.

Typical joints found in modern bridges include:

Conventional filled joints to accommodate small movements due to concrete shrinkage, etc. These should be filled and sealed with suitable sealing compound. When required to prevent the leaching of backfill such as in abutment joints, they should additionally be covered on the earth face with appropriate flexible waterproof membranes.

Bridge deck expansion joints summarized by the range of movements catered for. The following may be used as guidelines:

Very small (less than 10mm) may be buried under the asphalt surfacing together with a waterproof bond breaker installed at the top of the deck to limit crack widths in the surfacing.

Small (10mm to 25mm) including concrete nosings with silicone seals or asphaltic plug joints.

Medium (25mm to 80mm) may include elastomeric cushion joints with anchored steel housings or claw type joints anchored into the adjacent concrete members together with preformed neoprene seals.

Large (> 80mm) including coupled elastomeric cushion joints, multi-element claw type joints, finger joints and rolling leaf joints.

The technical literature provided by the suppliers of proprietary expansion joints, including asphaltic plug joints, should be consulted to select the appropriate joint. For further information on issues such as skew, gradient, climatic conditions, etc., it is advisable to consult specialist suppliers.

Parapets

The design of bridge parapets or balustrades should be based on a standard design to ensure uniformity throughout Nigeria. The bridge designer's task should centre on the modification of standard parapet design to conform to the geometric form and size of the superstructure. In the absence of such a standard design, the designer should follow the loading specification as set out in BS5400. Care should be taken to ensure that the parapet is securely anchored to the bridge deck with appropriate reinforcement, especially in the case of thin cantilever slabs. In this regard, it is important to size the deck edge thickness to at least 250mm.

Drainage

The design of bridge drainage should be based on standard procedures to ensure uniformity of the design approach suitable for Nigerian conditions. The bridge designer should consult with the FMW to confirm the acceptability of such facilities.

Drainage in the fill behind abutments should comprise a geofabric system combined with perforated neoprene strips and horizontal pipes to drain water away from the road embankment.

Scuppers along the deck edges adjacent to the parapets should be suitably spaced to ensure that water is drained away from the deck without ponding beyond the width of shoulder lines. Such scuppers should also be large enough to ensure proper drainage without the build-up of debris.

References

7.1 Menn C, PRESTRESSED CONCRETE BRIDGES, Birkhäuser Verlag, Basel (1990)

ISBN: 3-7643-2414-7

7.2 Shanmugan NE, Narayanan R, STRUCTURAL ANALYSIS, ICE Manual of Bridge Engineering (Second Edition), Parke G (Ed) and Hewson N (Ed), Thomas Telford, London (2008)

ISBN: 978-0-7277-3452-5

7.3 Clark LA, CONCRETE BRIDGE DESIGN TO BS 5400, Construction Press, London (1983)

ISBN: 0-86095-893-0

7.4 O'Brien EJ, Keogh DL, BRIDGE DECK ANALYSIS, Taylor and Francis Group, London (2006)

ISBN: 0-419-22500-5

7.5 Jackson P, DESIGN OF REINFORCED CONCRETE BRIDGES, ICE Manual Of Bridge Engineering (Second Edition), Parker G (Ed) and Hewson N (Ed), Thomas Telford, London (2005)

ISBN: 978-0-7277-3452-5

7.6 Hewson NR, DESIGN OF PRESTRESSED CONCRETE BRIDGES, ICE Manual of Bridge Engineering (Second Edition), Parker G (Ed) and Hewson N (Ed), Thomas Telford, London (2005)

ISBN: 978-0-7277-3452-5

7.7 Ramberger G, STRUCTURAL BEARINGS AND EXPANSION JOINTS FOR BRIDGES, Structural Engineering Documents 6, International Association for Bridge and Structural Engineering, Zürich, Switzerland (2002)

ISBN: 3-85748-105-6

7.8 Kennedy-Reid I, BEARINGS, ICE Manual of Bridge Engineering (Second Edition), Parker G (Ed) and Hewson N (Ed), Thomas Telford, London (2005)

ISBN: 978-0-7277-3452-5

7.9 Marx S, Schact G, CONCRETE HINGES - HISTORICAL DEVELOPMENT AND CONTEMPORARY USE, 3rd fib International Congress (2010)

7.10 Lindsell P, Clayton CRI, Xu, M, Hewson N, SUBSTRUCTURES, ICE Manual of Bridge Engineering (Second Edition), Parker G (Ed) and Hewson N (Ed), Thomas Telford, London (2005)

ISBN: 978-0-7277-3452-5

7.11 Lambe TW, Whitman RV, SOIL MECHANICS, John Wiley & Sons, New York (1969)

ISBN: 0-471-51192-7

Specific Design, Detailing and Construction Issues

Introduction

Bridges are important public structures which are required to carry their own dead weight, additional permanent loads and dynamic loads which induce effects often of a similar magnitude to those caused by their own weight. Not only are these structures subject to a wide range of load combinations, but they are frequently required to comply with complex geometry involving curvature and high degrees of skew, and are constantly exposed to the vagaries of the climate.

Whereas the modern trend in bridge engineering is towards slenderness of bridge decks in the interests of appearance, bridge components should be sufficiently robust to ensure the stability of the structure as a whole, and generously proportioned to facilitate design and construction in regions of high force concentrations. Inadequately proportioned members can lead to unnecessarily congested reinforcement and difficulties with placing and compacting concrete, which may ultimately compromise the durability of the structure.

These issues are the focus of this chapter, in which the bridge designer is encouraged to favour generous and robust component proportioning in the interest of simple and practical design, detailing and construction, rather than tight design which will often prove to be false economy.

Avoidance of Tight Designs

Adequacy of space for placing and compacting concrete

The structural performance of reinforced concrete components is highly dependent on the adequacy of the placement and compaction of concrete, among other factors. It is often found that concrete components are proportioned without due regard to the minimum space required for concrete cover, fixing the various rows or layers or reinforcement in order to place and compact the concrete. The minimum guidelines given in detailing codes for the spacing of reinforcing bars should be applied circumspectly, not only to accommodate the aggregate size but also to provide adequate space for vibrators to compact the concrete and avoid honeycombing. It is evident that a well compacted reinforced concrete element with less but properly bonded reinforcement will perform better than a poorly compacted member with a larger amount of reinforcement.

The splicing of reinforcing bars occupies a large amount of space and adds to the congestion of the reinforcement. This should be recognised during the detailing stage to facilitate the placing and proper compaction of the concrete.

Provision of large scale details in areas of congestion: reinforcement, prestressing and joints

In many bridge components it is difficult to avoid congestion of reinforcement, such as in the vicinity of expansion joints, prestressing anchors and couplers and at high and low points of prestressing tendons. The liberal development of large scale sections to amplify the details (usually 1:5 scale) is often essential for error free detailing and will assist the steel fixer to place the reinforcement accurately and without frustration.

Reinforcement at junctions between precast and in situ concrete components

The design of composite construction in a bridge usually comprises precast beam elements carrying an in situ concrete deck slab. The former may be prestressed (pre-tensioned or post-tensioned) or reinforced as required.

The design of composite elements is complicated by the fact that, in addition to calculations for shear and flexure for both the precast unit and the composite section being required, interface shear stresses also have to be considered. Furthermore, differential shrinkage has to be considered, but the effect of this is reduced by creep for which a reduction factor is given in BS5400.

Interface shear reinforcement is required for the beam section to transfer shear to the slab thus allowing it to act compositely. The bridge designer should therefore be careful when considering the support conditions of the beam during placement of the in situ concrete, i.e. propped or unpropped. It should be noted that the beam can only carry loads compositely after the in situ concrete has hardened.

Adequacy of space for bearings

There are many patent bearings on the market and generally the contractor is at liberty to propose an alternative to that originally specified. The bridge designer should therefore provide for bearing seat level flexibility to accommodate possible variations in the thicknesses of the bearings.

The position and size of the bearings in plan should also be carefully considered when detailing the bearing seats. The space between the edges of the bearing adaptor plates and the bearing seat should be at least 100mm. Adequate reinforcement should be placed in the bearing seats to avoid edges spalling or splitting cracks to occur. Often the cross-sectional size of a pier column is determined by the plan dimensions of the bearings (including adaptor plates when applicable) to be supported. It is therefore essential to consider the plan dimensions of available bearings on the market when determining the size of the underlying support column or bearing seat.

Housing of deck expansion joints and prestressing anchors: reference to specialist supplier drawings

The suppliers of expansion joints and prestressing systems are required to prepare detailed drawings of these components. Often this is done without due regard to the reinforcement details in the deck diaphragms. The diaphragms usually have to accommodate large shear and bending stresses and are thus heavily reinforced.

Deck support bearings together with prestressing anchors at the deck end all contribute to a complex pattern of stresses and additional reinforcement in the diaphragm. The suppliers of expansion joints, prestressing anchors and bearings should carefully consider the reinforcement detailing in the diaphragm to avoid unnecessary congestion and clashing of reinforcement, which may result in the poor compaction of concrete. It is often advisable to separate the prestressing anchors from the diaphragm reinforcing cage by extending the deck webs beyond the diaphragm to avoid this problem, as illustrated in Figure **Error! No text of specified style in document.**.26.

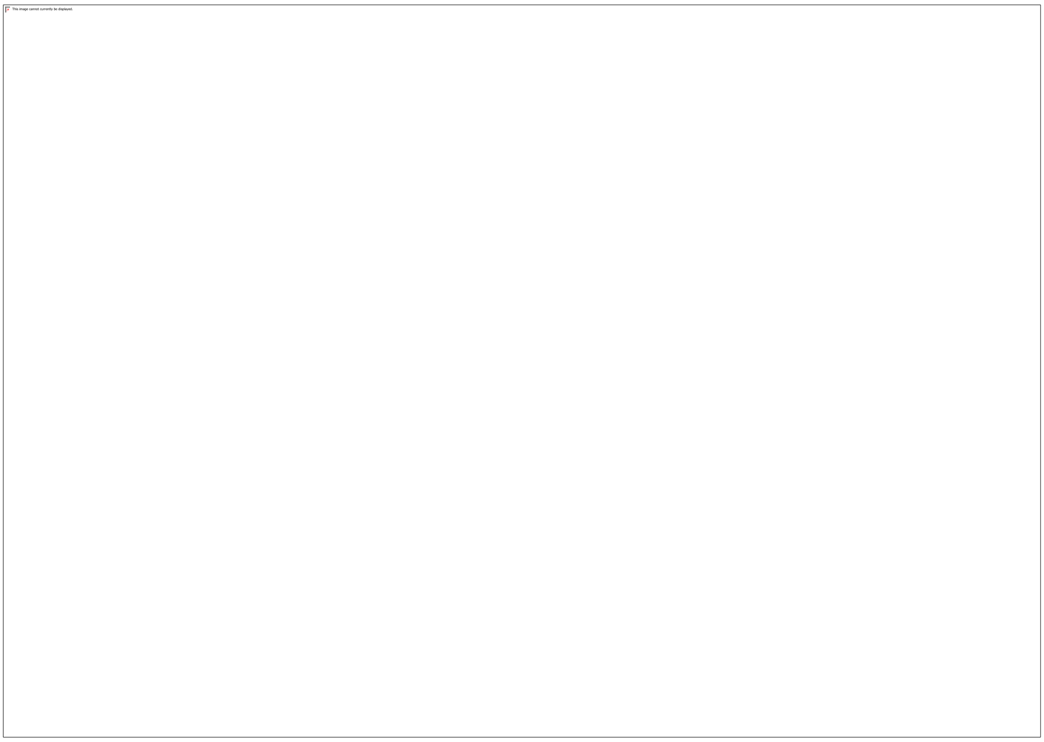


Figure **Error! No text of specified style in document.**.26: Extension of deck webs and expansion joint housing to avoid

Space required for operation of prestressing jacks and equipment

It is usual practice for the bridge designer to select a well-known and readily available prestressing system and base the prestresssing layout drawings on this system. The design of the bridge elements in the tendon jacking area should be carefully detailed to allow construction equipment to be positioned for safe and easy handling. Clearance diagrams for working space are obtainable from the suppliers and the bridge designer should ensure that this equipment can be accommodated in both the closed and extended positions.

The contractor may propose an alternative prestressing system, and if approved, the bridge designer should ensure that the above requirements are still met.

Tensioning of tendons involves large forces and constitutes areas of high risk during these operations. Construction workers should be prevented from standing behind a stressing point in case of a cable rupture which could result in serious injury or even death. A note to this effect should be added to all prestressing drawings.

Specific Reinforcement Design and Detailing Issues

Consideration of the flow of forces

When detailing reinforcement, it is often useful to consider the flow of forces by modelling the structural element as a truss with idealised concrete compression members and the reinforcement bars as tensile members. The internal forces obtained from such a model will satisfy equilibrium. The use of a truss model will assist in understanding and developing sound details for anchorage and splicing of reinforcement at joints in rigid frames and other components.

Skew slabs

The magnitude of skew is normally measured as the angle between the axis of the substructure and the line normal to the longitudinal direction of traffic.

For simplification of detailing and placing of reinforcement, transverse deck reinforcement is normally placed parallel to the skew edges for slabs with a skew of 10º or less. For skew angles exceeding 20º, transverse reinforcement is preferably detailed normal to the longitudinal direction of traffic and shorter bars should be used in the acute corners of the deck. For intermediate skew angles, practice varies and the detailer should adopt the method preferred by the bridge designer.

Avoidance of skew joints through cantilever slabs

Acute corners in any bridge element should be avoided if possible. This is particularly important in the case of skew joints ending in thin, cantilever slabs. Not only is it difficult to detail and place the reinforcement properly, but small, vulnerable sections in the acute corners are highly prone to damage.

All sharp edges should be squared off to at least 300mm perpendicular to the deck cantilever edge with due regard to the type of expansion joint and the cross-sectional width of the parapet on the deck.

Joints in rigid frames

Joints in rigid frames may be subject to opening and/or closing bending moments, for which appropriate reinforcement is required for each case. Closing moments generate compression on the inside faces and tension forces on the outside faces. Opening moments generate forces on the opposite faces. The principles of good reinforcement design and detailing for joints in rigid frames and the use of truss modelling is illustrated in Figure **Error! No text of specified style in document.**.27.

Figure **Error! No text of specified style in document.**.27: Reinforcement for opening and closing joints Detailing preferred and to be avoided

Reinforcement in acute corners

In addition to the guidelines given in 8.3.2 above, careful consideration should be given to the support conditions at acute corners of slabs and potential crack patterns identified. Local wheel loads shall be applied to generate shears and moments at such vulnerable positions and appropriate reinforcement detailed. Because of the vulnerability of acute corners, bridge designers are advised to design such areas conservatively.

Design for shear lag

The bridge designer shall take into account the effects of 'shear lag' in decks which have comparatively wide and slender cantilevers. This phenomenon arises from the transverse distribution of bending or axial compressive stresses via transverse interface shears, which results in lower compressive stresses in the outer extremities of cantilevers than in the main body of a deck cross section.

Shear lag must be taken into account in the assessment of the deck section properties for the purpose of analysis. This phenomenon also affects the length over which concentrated prestressing forces are distributed over the entire cross section some distance from the ends of a deck. In this case shear lag may result in diagonal cracking of wide cantilever slabs, which must be resisted by hairpin reinforcing bars, fixed perpendicular to the potential cracks and anchored well back into the body of the deck.

Bursting and splitting forces

In cases such as bearings on columns or prestressing anchorages, large concentrated forces are applied directly onto a member over a small area and then spread out over the cross section to a uniform compressive stress over a short distance. In the region of spread, high local stresses occur in particular large transverse bursting stresses (see Figure **Error! No text of specified style in document.**.28). In the case of post-tensioned decks, these stresses are at their maximum at the time of tensioning the cables and will decrease after the cables have been grouted. In the case of bearings, these stresses will vary according to live loads.

Various methods of analysis are available such as truss analogy described in Reference 8.1. This method is particularly useful to understand and design the deck in the region where the support bearings are close to the prestressing anchorages with complex stress patterns as a result. The guidelines for designing bursting reinforcement are well documented in BS5400 Part 4.

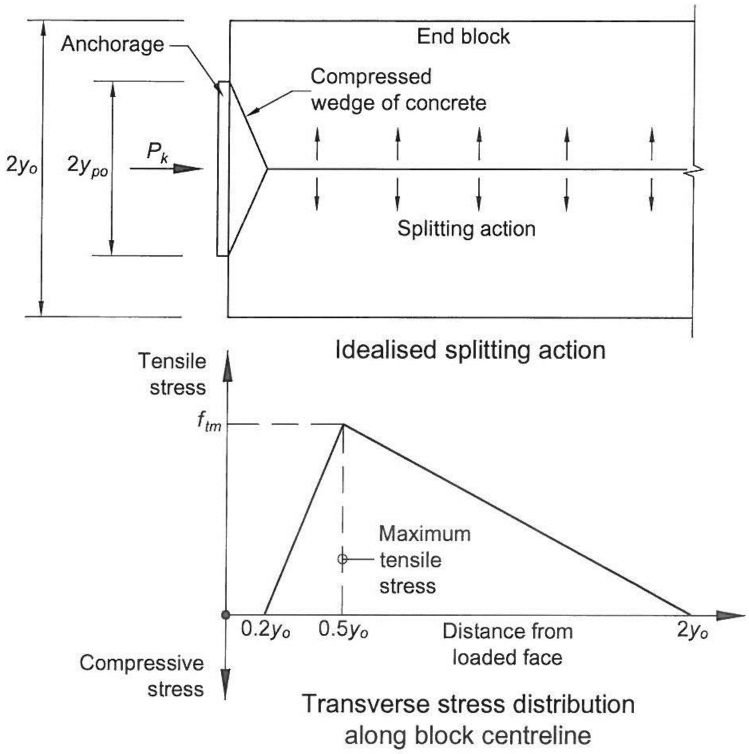


Figure **Error! No text of specified style in document.**.28: Tensile splitting stresses arising from concentrated prestressing

Cracked sections and crack width issues

BS5400 makes provision for the control of cracking in prestressed concrete members by the limitation of flexural tensile stresses for the three classes of prestressing. Design surface crack widths for reinforced concrete are based on considerations of appearance and durability and are prescribed for various conditions of exposure summarised in BS5400.

BS5400 further specifies a minimum percentage of reinforcement to control restrained shrinkage and early thermal movement cracks. These percentages should be regarded as the absolute minimum and should normally be rounded up when selecting bars sizes and spacing. The maximum bar spacing should not exceed 150mm. These amounts of reinforcement should be considered when calculating crack widths.

Practical guidelines with calculation examples for cracking based on BS5400 are given in Ref 8.1.

Design, Detailing and Construction for Durability

Durability

In this context durability implies compliance with the relevant clauses of BS5400 to ensure that bridges attain and maintain adequate durability during the design life of these structures. Durability is normally treated by attention to the detailed aspects of design, to the specification and control of materials and workmanship, with provision for protection and routine maintenance as dictated by environmental conditions.

The requirements of BS5400

The bridge designer is expected to be conversant with and to apply the various requirements of BS5400 concerning durability, which is an important criterion concerning the serviceability and function of concrete bridges. The factors which affect durability include the following:

Climatic conditions during and after construction with particular reference to the temperature range, rainfall and atmospheric moisture, salt and pollutant laden air; all of which influence the required concrete cover to reinforcement.

The design stresses, deformation and rotation of the bridge components, arising from the load effects and materials behaviour, with particular regard to the influence of cracking.

The details of the reinforcement and/or prestressing (type, quantity, bar spacing etc.) required:

for adequate strength in resisting bending, shear etc. and for crack control;

as minimum nominal steel to counteract restrained deformation such as caused by shrinkage and temperature.

The quality of the detailing and fixing of steel reinforcement in relation to the required concrete cover.

Details of the specified concrete mix such as the characteristic strength, anticipated density, characteristics of the constituent materials and resistance to the ingress of contaminants.

Control of the quality of construction concerning proportions, mixing, temperature, placing, compaction and curing, as evidenced by appropriate tests of strength, density, oxygen permeability, chloride conductivity and carbonation potential.

The provision of adequate movement joints in the structure and the control of the quality and location of construction joints.

Periodic maintenance and component rehabilitation of concrete bridges.

Minimum shrinkage and temperature reinforcement

In addition to reinforcement required to limit crack widths due to flexure and axial tension, reinforcement to prevent excessive cracking due to restrained shrinkage and temperature should be provided as required in BS5400 in the direction of such restraint.

Examples of components in which shrinkage and temperature reinforcement is required in the horizontal plane are:

The walls of large sized and tall cellular (hollow rectangular or similar) pier columns.

Cellular abutments in which the front walls are restrained by straight return wingwalls at the outer sides or by large counterforts.

Minimum concrete cover to reinforcement

The adequacy of the concrete cover to reinforcement is one of the main factors in ensuring the durability of concrete structures. In view of the permissible tolerances in the bending and fixing of reinforcement and in the accuracy of formwork, the nominal cover to reinforcement tabled in BS5400 for the particular conditions of exposure and various concrete grades should be treated as minimum.

As the covercrete is the last line of defence against corrosion of the reinforcement, the cover blocks separating the reinforcing bars from the formwork should have at least the same density and nominal strength as the applicable concrete component, and be furnished with galvanised tying wire embedded not closer than 0.5 x cover to exposed faces of the component.

Concrete mix design

Careful consideration should be given at the design stage to the specification of the concrete mix when a bridge is exposed to a corrosive environment such as industrial areas or coastal regions. Depending on the pollutants present, the concrete mix may be required to contain a minimum cement content or specific types of cement as appropriate. Specialist literature should be consulted if there is any doubt as to the effects of pollutants present.

The qualities of locally available aggregates (chemical composition and shape) are also a consideration as to the durability of a concrete mix design and should be carefully considered.

Compaction and curing of concrete

The durability of concrete is also largely dependent on the quality of compaction and curing of the concrete, as the presence of voids or cracks will allow the ingress of corrosive agents into the concrete. For this reason, the compaction of concrete should be undertaken by personnel experienced in the use of vibrator pokers to achieve a dense concrete. The curing of the concrete is also extremely important in the prevention of drying shrinkage cracks and weather conditions should be kept in mind when concreting large surface areas such as bridge decks (hot and dry windy conditions). Patent curing compounds may be applied, but the application of water to keep the concrete surfaces damp is arguably still the best method. Delaying the removal of shutters for longer than the specified minimum period is also a very effective method of curing.

Construction and movement joints

A construction joint is a temporary joint created to facilitate construction. Movement joints are created to accommodate contraction and/or expansion without generating significant internal structural forces in the contiguous members. The joints may be filled, sealed or fitted with seals and/or mechanisms to render them watertight and/or to create a smooth transition over the joint as appropriate.

When it is necessary to indicate construction joints on a drawing, careful consideration should be given to their exact location and due account should be taken of shear and other stresses. Construction joints should generally be at right angles to the direction of the member. The surface of the existing concrete may be treated in various ways before proceeding with the concreting, such as the application of patent "wet to dry" epoxy compounds, the wetting of the concrete and the application of a cement slurry etc. in accordance with the specification.

It is important to secure the continuation shuttering tightly against the existing concrete as all shuttering will kick to some degree under the pressure of wet concrete and may render the joint unsightly. The placing of a rectangular joint former at a construction joint will assist in making the joint as neat as possible without compromising the cover of the concrete to the steel. Triangular joint formers forming notched joints are discouraged for this reason.

Prior arrangements for emergency construction joints should be made to accommodate unscheduled stoppages.

Movement joints in a bridge substructure should be clearly indicated on the drawings and placed on one plane through the structure.

Voided Slab Decks

Materials for formation of voids

The provision of voids in deck slabs is common practice which serves to decrease the deck dead load whilst maintaining its shear and flexural strength. Depending on the size and shape, the following methods and materials may be used to form the voids:

1. High density polystyrene formers

Polystyrene may be pre-formed into any shape desired and is the safest and preferred method of forming voids in voided slab decks.

Spiral formed steel void formers

Generally the shape of this sacrificial shutter is restricted to a circular tube. The tubes are relatively heavy and voluminous and are difficult to transport and place into position. Since the formers are not waterproof provision has to be made for drainage. The tubes may be manufactured on site by light equipment thus reducing transport difficulties. The tubes must be braced internally by sacrificial timber cross bearers as shown in Figure **Error! No text of specified style in document.**.29 in the case of large void diameters.

Anchorage of voids during concreting

Often the magnitude of buoyancy forces of a void former in wet concrete during construction is not sufficiently appreciated. The buoyancy force generated by wet concrete is in the order of 24 MN/m^3 and, if the void formers are not restrained, they will float upwards to a position of equilibrium. For this reason, minimum anchorage comprising R10 bars or steel strap loops spaced at 1 m centres over the tube anchored by the bearers supporting the shuttering is required as also illustrated in Figure **Error! No text of specified style in document.**.29.

Drainage of voids

Voids shall be drained at the low points by 25mm Ø PVC pipes protruding from the soffit of the deck.

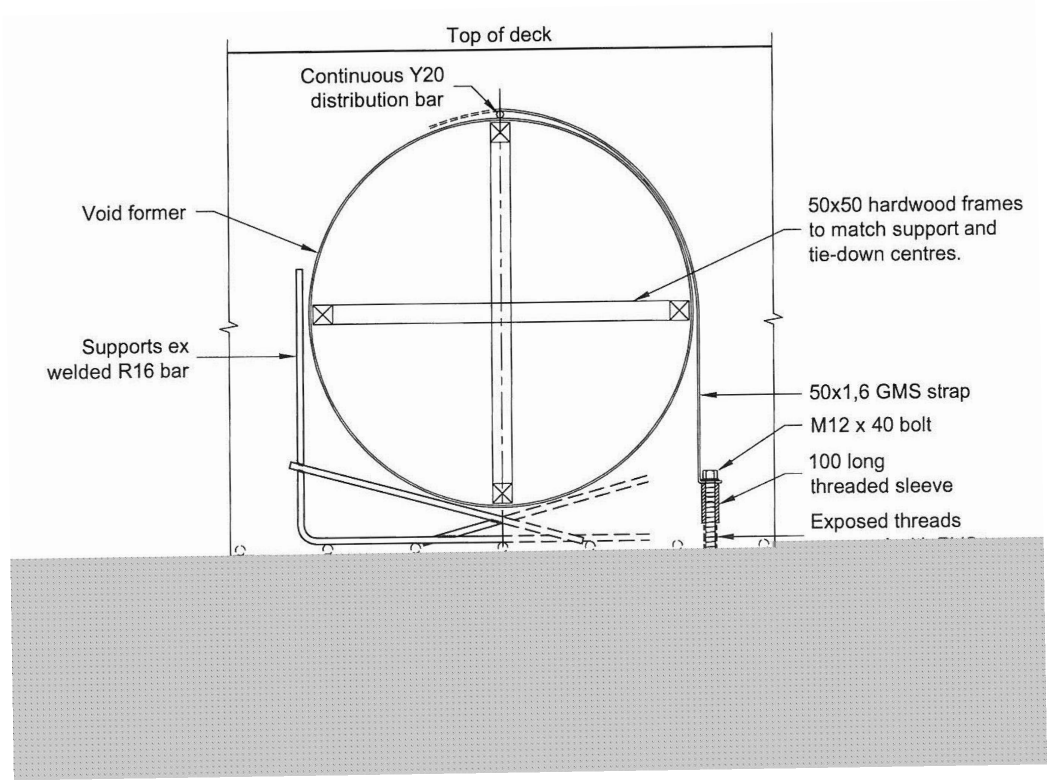


Figure **Error! No text of specified style in document.**.29: Typical void former support and holding-down detail (detail symmetrical about void former centreline)

Box Girder Bridges

Box girder forms

Box girders are a common and economical form of continuous prestressed concrete bridge deck construction for a wide range of spans. Spans of this form start at about 25m and extend to about 60m for uniform depth in situ or incrementally launched construction, and can extend to 150m or more when constructed by the balanced cantilever method using precast concrete segments to form variable depth decks.

Volume V is concerned principally with small to medium span bridges, which includes in situ and incrementally launched concrete box girder decks up to about 45m - 50m maximum span lengths. This section deals with certain design and construction issues concerning box girders in this range.

Construction sequences

In order to cater for the practicalities of box girder construction in stages it is necessary to design for both horizontal and vertical construction joints in this form of deck.

1. Transverse section: horizontal joints

1. For normal in situ decks it is preferable to construct the bottom slab and the webs in one operation and to provide for a horizontal joint between the webs and the top slab, with as short a time interval as practical between completion of the webs and casting the deck, in order to minimise differential shrinkage effects between the first and second stage concreting.

2. For incrementally launched decks it is usually necessary to cast the bottom slab, webs and top slab in three separate operations.

Longitudinal construction sequence

1. Span by span construction with vertical joints at 0.20 of the next adjacent span is favoured for normal in situ decks, with prestressing designed to have overlapping tendons. The analysis for this sequence should be done on an incremental basis in which each new span load, together with creep and shrinkage affect the cumulative load effects (bending moments and shears) in the spans previously completed. Overlapping of prestressing cables may be achieved by internal anchors, couplers or blisters.

2. For incrementally launched decks segments are usually half span in length with joints at the deck quarter points.

Access to voids through bottom slabs and diaphragms

The removal of internal shuttering and falsework of cast in situ box girders is achieved by the installation of access holes through the soffit slab and also through the diaphragms. The access hole through the deck soffit should be placed in a position of low compressive stresses and shaped to allow a smooth flow of stresses around it.

Deck and Subsurface Drainage

Deck drainage

The accumulation of rain water on traffic surfaces of roads and bridges is dangerous to traffic as this may cause planing on ponded surfaces. Side drains on the approaches to bridges will prevent water draining from the road onto the bridge and this may be sufficient provision for short bridges or bridges located on a hogging vertical curve. Deck drainage may be provided by the judicious spacing of scupper pipes along the deck adjacent to the parapets or kerbs. In all cases the accumulated water should not extend beyond the width of the shoulders into the traffic lanes. If the feature crossed is a railway line or a road, the water should be routed to the nearest suitable drain by downpipes at the piers or abutments. In all cases, the deck should have a minimum cross fall of 2% to direct the flow towards the drainage pipes or scuppers.

Drainage of bearing seats at abutments and piers

The accumulation of water from driving rain or leakage of roadway joint should be avoided on bearing seats to prevent contamination of the bearings and unsightly staining of the faces of the abutments or piers. The first line of defence is to maintain the expansion joints as waterproof as possible. However these joints are seldom 100% waterproof and bearing seats should be provided with drains to direct water away from the bearings and the vertical faces of the abutment walls or piers. At abutments, bearing seats should be drained by pipes into the subsurface drainage system behind the abutment wall. For piers, pipes placed inside the columns exiting at ground level are the most satisfactory method.

Subsurface drainage

The embankments contained by the abutment walls should be drained by a system of geofabric and perforated neoprene strips or a similar system, which conveys subsurface water to pipes discharging through weepholes just above the front face ground level.

Replacement of Bearings

The replacement of bearings is a normal maintenance operation required periodically during the service lives of bridges. Bearing replacement should preferably be catered for at the original design stage by providing suitable 'strong points' at which the superstructure can be safely lifted by jacks and which do not restrict access to the bearings which are to be replaced. However, in the case of precast beam and slab bridges and certain older forms of bridges it is usually necessary to re-analyse the structure in order to determine the location and means of lifting the superstructure.

Whereas the following steps relate specifically to the replacement of existing bridge bearings, the bridge designer should provide as much of the information required for this purpose as possible on new bridge drawings, to facilitate such replacement in the future.

For the purpose of planning the operations to lift a bridge deck in order to replace the bearings, the following information is required:

Copies of the As Made drawings and original calculations, if available.

The amount of lift required to remove the existing bearings and install the replacement bearings.

The form of the structure i.e. simply supported, continuous etc. and whether any of the pier columns are monolithic with the deck.

The configuration of the deck, i.e. slab, voided slab, beam and slab, box girder etc.

Investigate whether:

Jacking can be safely carried out from abutment bearing seats and pier caps.

Balustrade dowels or services across expansion joints may hinder lifting operations.

Deck expansion joints can be salvaged during lifting operations.

Replacement of the bearings must be carried out under full traffic, restricted traffic or without traffic.

The methods available for the lifting of bridge decks are many and varied and are beyond the scope of this manual. However, in all cases of the lifting of bridges certain information must be established from adequate analysis regarding specific requirements, limitations, or restrictions during the necessary operations to prevent damage to the structure and ensure safety. The information required by the contractor for purposes of planning, pricing and implementing the work includes the following:

The points at which jacking or support of the deck will be permitted, together with the loads to be lifted and the amount of the lift required.

The need for the strengthening of any parts of the superstructure or substructure, depending on the proposals submitted by the contractor.

The total amount of lift required to remove the bearings and the maximum incremental lift permitted at any lifting point and the maximum differential lift which will be permitted between designated lifting points in order to prevent the development of excessive stresses or rotations in the structure.

Any mandatory steps regarding the stability of the span being lifted or of the individual jacks, taking into account the longitudinal gradient of the deck and expansion or contraction of the deck due to temperature or other effects during the lifting or lowering operations.

The amount of space available to accommodate the jacking equipment. There are jacks available which may be inserted in gaps as small as 50mm. The use of steel shims to lift the deck in increments to the required amount is recommended.

All metal bearings should be installed with adaptor plates attached to the deck and underlying bearing seats to provide for ease of removal and replacement of the working parts of the bearing by jacking the deck. If adaptor plates are not present, the bridge designer should determine how the attachment of the bearing should be removed.

It should be noted that, in the case of lifting continuous decks, hydraulic jacks connected in series and operated by a single pump will each have equal internal pressure but not equal travel, depending on the resistance encountered at each lifting point. This problem may be overcome by the individual control of each jack to ensure uniform lift.

Ideally, replacement of bearings should not be carried out under traffic. If this is not possible, the temporary supports at the approved strong points should be designed to accommodate traffic as well as permanent loads.

The contractor is required to submit detailed drawings and a method statement to adequately describe the proposed method of work and demonstrate compliance with the design requirements. Following approval of these submissions, including any amendments required by the engineer, the work may proceed.

During the implementation of this type of work it is important that design requirements are met to ensure that restrictions or limitations are observed. In the event of any signs of distress in either the structure or the support work the design engineer should be promptly consulted on the steps to be taken. Thorough records must be kept of the jacking forces at each lifting point and the associated vertical displacements, for the purposes of further analysis if necessary.

References

8.1 Clark LA, CONCRETE BRIDGE DESIGN TO BS 5400, Construction Press, London (1983)

ISBN 0-86095-893-0

Further Reading

Menn C, PRESTRESSED CONCRETE BRIDGES, Birkhäuser Verlag, Basel (1990)

ISBN 3-7643-2414-7

ISBN 0-8176-2414-7

Drawings and Documentation

Draughting Standards

General

The standard of draughting shall comply with BS5070: Part 1 unless specified otherwise by the FMW.

The format of drawn or printed standard sheets for drawings shall have black line margins and standard title blocks as directed by the FMW.

Drawing sheets shall be A0 size, 0.075 mm polyester film unless otherwise authorised by the FMW.

Drawings are a vital means of communicating the design and construction requirements, which must be unambiguously conveyed to the users of these documents. The drawings must therefore be precise and with sufficient elevations, plans, sections and other information, arranged in a logical and systematic manner so as to facilitate implementation based on a sound understanding of the requirements shown on the drawings.

It is as important to avoid crowded, complicated and superfluous details as it is to avoid deficient details and 'short cuts'. Lengthy notes should also be eliminated in favour of providing explanatory details.

On completion of the working drawings, the bridge designer needs to be satisfied that: the contractor can successfully build the structure without the need for the provision of numerous additional details or frequent interventions and interpretations.

Line thicknesses and lettering sizes

Line thicknesses

1. Dimension drawings:	Preferably 0.50 mm (0.35 mm minimum for small or intricate detailing)
Dimension lines:	0.25 mm
Reinforcement Detail drawings:	0.35 mm (structural outlines)

	0.50 mm (reinforcement)
	0.25 mm (indicator lines)
Bending Schedules:	0.50 mm (sketches)
	0.25 mm (dimensions).

Lettering sizes

1. Titles:	5 mm
Notes, dimensions, etc.:	2.5 mm

Electronic drawings

CAD software packages available

There are many CAD packages available on the market, but the most widely used is Autocad which enjoys international recognition. Most other systems are Autocad compatible generics and drawings may usually be converted to the Autocad format quite readily. The designer shall confirm that his system is compatible with that of the FMW and that all the settings are mutually identical.

Approval of drawings

The procedure for submission and approval of electronic drawings shall be to the FMW's requirements and is beyond the scope of this document. A basic principle should be that, once a final drawing has been approved, it should be stored in a "read only" format to avoid changes by unauthorised persons.

Medium of submission

Drawings shall be submitted on compact disc to allow the FMW to archive the contents in a manner of choice.

Drawings at various Project Stages

Bridge schedule

The bridge schedule shall consist of a 1:50000 Key Plan and a schedule of all the structures on the proposed route drawn on an A0 sheet.

The Key Plan shall be in the form of a strip plan along the road, clearly indicating the individual positions of each structure included in the particular road project.

The bridge schedule shall contain longitudinal and cross sections of each structure under the project.

Proposal drawings

The proposal drawings form part of the bridge report about all the bridges included in the particular road project. The proposal drawing for each bridge will essentially be a preliminary general arrangement drawing for the structure and should essentially agree with the deck cross sectional details in the previously approved Bridge Schedule.

The purpose of the drawing is to adequately demonstrate the technical soundness of the proposal in terms of the FMW's objectives and the basic goals of bridge design with particular reference to functionality. The drawing should include an elevation, plan, longitudinal section and deck cross section to amply illustrate the intended form of the main components and the impact of the structure on the site and surroundings.

The positions of boreholes should be shown on the plan and the logs of the geotechnical investigation should be shown on the longitudinal section with annotations indicating the subsurface profile. This drawing should be furnished with leading dimensions and notes describing the design loads, materials types and strengths, etc., but omitting dimensions of small detail.

The content, format and naming of the Proposal Drawings for railway structures shall conform to the Code of Procedure of the relevant authority.

For river bridges, hydraulic data shall also be included as indicated in Chapter 3.

Tender drawings

The Tender Documents shall include adequate tender drawings (including typical standard and design plans, if applicable) of the bridges on the project. It is not always possible to provide final working drawings at tender stage due to time constraints. In such a situation, at least a Site Plan and General Arrangement Plan should be included together with as many concrete outlines as possible to allow the contractor to evaluate the project with the required accuracy. Any missing information essential for accurate tendering shall be described in the Project Document.

Working (construction) drawings

The working drawings shall include the following, generally in the order shown:

Site Plan including Key Plan, Survey Plan and a list of drawings

Longitudinal Sections of road(s) and railway line(s) as applicable

Subsurface Data including logs for boreholes and/or trial holes and other subsurface sounding results

General Arrangement including general notes, elevation of bridge, longitudinal section through bridge, plan of bridge and cross-section of the deck

Foundation Layout

Concrete Details of abutments, piers and deck showing sufficient details and dimensions

Prestressing Details clearly showing location of tendons in cross-section, longitudinal section and plan with relevant offsets together with coupling and anchorage details and any other relevant information required

Reinforcement Details of abutments, piers and deck

Parapet Drawings including concrete and reinforcement details

Miscellaneous Details: expansion joints, bearings, drainage, etc.

Bending Schedules

The format and style of the working drawings shall be agreed between the FMW and the bridge designer before commencement of these drawings.

Contractor's submissions (specialist supplier details)

Drawings submitted by the Contractor for approval of alternative ancillary components such as bearings, expansion joints, prestressing, etc., shall be to the same format and standard as the original drawings. These drawings shall be submitted timeously to allow perusal and approval by the FMW and delivery by the suppliers in good time for construction. The numbering system shall be agreed with the FMW to allow integration of the drawings with the final as made drawings on completion of the project.

As Made drawings

During a construction project, the supervisory staff shall record all changes and relevant data on a set of paper prints kept for the purpose of preparing formal “As Made” drawings at the end of the project. The format and medium of the “As Made” drawings shall be as prescribed by the FMW and may be stored on plastic film or electronically. All drawings shall be clearly marked “As Made” and, in the case of an electronic record, should be in a format such as PDF which cannot be modified.

Drawings for other Highway Structures

Major Culverts

The required working drawings are the following:

Site Plan which shall including a key plan, locality plan, longitudinal sections, typical section through road prism, subsurface data and typical cross sections of the waterway to determine the normal depth of flow

General arrangement and concrete details showing the elevation, longitudinal section, cross-section and plan of the structures together with relevant details required for construction. General notes and list of drawings shall also be included together with relevant hydrologic and hydraulic data

Reinforcement details

Bending Schedules. If sufficient space is available, these may be included on the drawing sheet for Reinforcement details

Other structures

Other structures such as retaining walls and lined canals shall be detailed in accordance with the requirements for major culverts, except that in the case of:

Retaining walls, the longitudinal sections forming part of the site plan may be omitted.

Lined canals, the longitudinal section of the road and the typical section through the road prism which form part of the site plan may be omitted.

Minor structures

Minor structures such as minor drainage structures (less than 2.5m^2 cross sectional area), service ducts and the like may be detailed in scheduled format and shall include the following:

Key Plan

Schedule

Details

Bending schedule

NOTE

Minor structures such as described above are usually designed as part of the road works (Volume IV) and excluded from work in terms Volume V.

Quantification and Documentation

Standard specifications and method of measurement

The method of measurement shall be in accordance with the Standard Specifications of the FMW. The pay items shall have the same numbering system and description as described in the relevant clauses for Measurement and Payment.

Bill of quantities

The Bill of Quantities shall be set out on an electronic spreadsheet either compiled “in house” or by software available on the market as preferred by the FMW. The Bill of Quantities normally consists of columns with headings for Item No, Description, Unit of Measure, Quantity, Rate and Amount. The Quantity, Rate and Amount columns are left blank and should be formatted to be interactive (Amount = Quantity x Rate). Further provision may also be made to accommodate Scheduled Quantities and Current Quantities during construction to provide an easy reference regarding progress.

Estimates of cost

The estimator shall complete the Bill of Quantities described above by inserting the relevant Quantities and Rates. The latter shall be estimated by gathering information from previous tenders and making adjustments for price escalation, if any. Provision shall be made for contingencies and other additions as specified by the FMW in order to provide a reliable estimate of the expected expenditure for budgetary purposes.

The use of electronic spreadsheets for the above allows the estimator to make minor adjustments quite easily for quick results and reporting.

Special provisions of contract

Conditions and requirements vary from project to project and special provisions must be made as relevant. These shall be included in the relevant sections in the project document according to the format and requirements of the FMW.