STRUCTURES ENGINEERING DESIGN MANUAL

Denmark River Bridge No. 86A

STRUCTURES ENGINEERING

Document No: 3912/03

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AUTHORISATION

As head of Structures Engineering of Main Roads Western Australia, I authorise the issue and use of this document.

RF Scanlon

…………………………………………..

SENIOR ENGINEER STRUCTURES

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PREFACE

This Structures Engineering Design Manual has been prepared by Structures Engineering Section of Main Roads Western Australia to provide a guide to the design of bridges and associated structures within MRWA.

The Manual is intended to fill a gap not covered by either the Bridge Design Code or standard text books. The Bridge Design Code provides detail of design principles, loads and standards to which all bridges designed by, or for, MRWA must comply. It does not, however, cover the application of these principles. On the other hand, standard texts cover structural analysis and design theory and application, but few are specific to bridge design in accordance with Australian codes. This area of specific application is what this Manual is designed to address. It is intended to be complementary to the Bridge Design Code and standard texts, but does not repeat material they contain, making use instead of a comprehensive list of references.

The Manual is also complementary to the Procedures in the Structures Engineering Management System. There is some unavoidable overlap between the two, but in simple terms the Structures Engineering Design Manual presents HOW things are to be done, whereas the Procedures detail WHAT is to be done, by WHO and WHEN.

This Manual is also complementary to the Bridge Branch Design Information Manual. Again, there is some overlap between the two, but basically the Information Manual is more mandatory, and presents MRWA specific variations or conditions that apply to the HOW things are to be done, as outlined in this Manual.

Each Chapter of the Manual has been written by an experienced, senior design engineer, with subsequent review and comment by other staff. It is intended for use by all members of Structures Engineering, but should be particularly useful as a training aid for new members of staff. It was this “young, qualified, but inexperienced engineer” that was the “target audience” during production of the Manual.

It must be stressed that the Manual is not a “cookbook” that can be freely used by all. It assumes a sound knowledge of structural engineering principles, awareness of the appropriate Codes and Standards and some background in bridge engineering. It must only be used by qualified engineers with some knowledge of bridge design, and/or under the supervision of an engineer experienced in bridge design. Although every care has been taken in the preparation of the Manual, because of the lack of control over its application, **NO RESPONSIBILITY WHATSOEVER** is taken for its use.

It is intended that the Manual shall be a dynamic document, subject to continual review. The need for change may occur due to the development of improved methods of analysis, new construction techniques, revised design standards etc. The Manual is the responsibility of the Senior Engineer Structures. However, all proposals for change shall be submitted in the first instance to the Structures Design and Standards Engineer.

The Manual is a controlled document as described in the Structures Engineering Document Control Procedure. In particular, a record shall be kept of the issue of the Manual to ensure that modifications are circulated to all.

CHAPTER 1

INTRODUCTION

CONTENTS

1.1 GENERAL

The purpose of this Chapter is to provide an overview and outline of the bridge design process. The detailed design process is developed in the following Chapters, each of which deals with a different aspect of the design process.

This Manual should be read in conjunction with Structures Engineering Management System (SEMS), in particular Procedure Document No. 3912/01/05 which details the different responsibilities and authorities required to approve and document the design process.

1.2 BRIDGE DESIGN

The design process can be considered as consisting of six stages:-

- Pre-design Activities
- Preliminary Design
- Final Design
- Preparation of Drawings
- Preparation of Contract Documentation
- Provision of design advice during Construction

Each of these stages represents a task with a readily identifiable output. These tasks often overlap in time but are normally progressed in a sequential manner.

1.2.1 Pre-Design Activities

A design brief or scope is prepared for approval in accordance with the SEMS. The brief will summarise the information to be used in the design and other conditions or requirements. Such information may include the following, depending on the situation:-

- Bridge location
- Road geometry
- Bridge configuration, length, width, clearance requirements, span lengths
- Design standards including barrier type
- Design loads
- Waterway requirements
- Aesthetic considerations
- Environmental, ethnic, archaeological and heritage considerations
- Services
- Local Government requirements
- Site restrictions

A detailed list of information that might be required for the design may be found in the CODE Part 1 Appendix A.

A major requirement at this stage is for a foundation investigation to obtain information on the sub-soil conditions at the site. This will usually be carried out through a specialist Geotechnical Engineering Consultant. The designer will need to specify the type and locations of bores and any special sampling or testing required. Depending on the complexity of the site and/or the structure, independent advice may be required from specialist geotechnical consultants to assist in specifying tests for inclusion in the brief. For further details see Chapters 5, 6 & 7 of this Manual.

1.2.2 Preliminary Design

The objective of the Preliminary Design stage (also known as 15% stage) is to investigate feasible options for bridging the site in sufficient detail to enable reliable comparative estimates of costs to be made.

This is the stage where most engineering judgement is exercised. Using the known information and constraints, a suitable location, length and span configuration is determined for the bridge and one or more structural solutions investigated. Guidance on possible structural options for a particular site is given in Chapter 2.

Preliminary Design usually commences with the superstructure and proceeds down through the substructure to the foundations. Subsequent Chapters of this Manual provide guidance on methods of analysis to use at these stages. The Preliminary Design provides approximate details of member sizes, reinforcement, prestress, piling and the various items of bridge furniture required, such as railing, expansion joints, bearings and lighting.

Possible methods of construction must also be investigated at this stage as they may affect the bridge type chosen and will have a strong influence on both tangible and intangible costs. This is especially true for bridges over existing roads, railways and large rivers.

A preliminary Bill of Quantities is prepared from the Preliminary Design(s) which can be priced, using rates obtained from the Quantity Surveyor for the various items such as formwork, concrete, reinforcing steel etc. To obtain proper comparison between alternative designs it is important that a fairly detailed estimate be prepared taking into consideration the method of construction, unit rates per square metre of deck area are usually not sufficient, (refer to Section 1.3 below).

When the preliminary designs and associated costs are completed, a report is prepared setting out the options that are available, with their advantages and disadvantages. This is then submitted for review (refer SEMS) which may result in final design commencing, or a request for alternative options be investigated. Cost is obviously a major consideration in making a decision on the design to be adopted but there are other factors to be taken into account, such as aesthetic appearance, future maintenance costs and ease of widening the structure in the future. When designing a major bridge it is important to seek architectural advice very early in the process to gain maximum benefit from this input, (refer to Chapter 2 clause 2.2.7 of this Manual).

It cannot be overemphasised that the preliminary design is a very, if not the, most important stage of the bridge design process. The correct identification of the most suitable structure for the particular site can produce economies much greater than will result from any refinements made later in the final design. This can only come about by investigation of alternatives at the preliminary design stage, including identification of the preferred method(s) of construction. To ensure a full and proper preliminary design is carried out, advice and input should be obtained from experienced engineers to provide guidance, especially on selection of the best structural form.

1.2.3 Final Design

Final design is commenced once the preliminary design has been approved. This involves the detailed design of all parts of the structure and is usually carried out by a design team working under the direction of a project designer.

Full details of the methods of analysis and design to be used for the different aspects of the structure are given in subsequent Chapters of this Manual.

1.2.4 Preparation of Drawings

Detailed Engineering Drawings are the main output of the design process and are the medium by which the designer communicates the design to the construction team in the field who will build the bridge. It is usual for the designer to work closely with drafting staff so the drawings are prepared while the detailed design is in progress, but care should be taken not to allow drawings to proceed ahead of design otherwise much drafting work can be wasted.

Information is given to drafting staff by the designer in the form of sketches, calculations, standard details and amendments to drawings of previous bridges. This information should always be provided in written form and a copy kept by the designer to assist in final checking. Well presented information will assist the draftsperson in producing a high quality drawing.

The designer should check progress on any drafting being prepared on a regular basis so that any errors or misunderstandings are corrected as soon as possible. Drawings are checked by the Project Officer Structures for accuracy of drafting and detailing but it is the responsibility of the designer to carefully check that all the engineering features shown are correct and in signing the drawing the designer accepts that responsibility.

1.2.5 Preparation of Contract Documents

The documentation for a bridge contract comprises the Conditions of Contract, Technical Specifications, Bills of Quantity /Schedules of Rates and the Drawings. The Conditions of Contract and General Clauses of the Specification are prepared by the Supply Branch of MRWA, but it is the responsibility of the designer to prepare the Technical Specification, based on standard specifications within the MRWA Tender Document Preparation system (TDP). The Conditions of Contract set out the contractual obligations of each party to the Contract and the Technical Specification defines standards of workmanship required. In addition, the designer must ensure that the Bills of Quantities or Schedules accurately reflect the works required. For further details refer to Chapter 21 of this Manual.

1.2.6 Bridge Construction

The designer's responsibility does not end once the Contract has been awarded and the job handed over for construction. The designer must be available to answer site queries and should make a point of visiting the site during construction, more than once if practical. It is only by doing this that information can be obtained as to whether all details shown on the Drawings are satisfactory. Informal conversations with site staff and workers can produce a lot of useful and timely information, which could be missed if the designer just relied on the "as-constructed" drawings.

There should also be a debriefing session with site staff after completion of the project, to highlight any issues with design details and design amendments which could improve constructability. It is essential that Drawing Office staff are also involved in this session.

1.3 COST ESTIMATES

Although they might not be considered a true part of the "design process", the preparation of estimates for bridgeworks is a very important task for the designer, as the cost estimate is usually the principal means of selection between design options and construction methods.

Estimates are prepared at different stages during the design of a structure and different methods and orders of accuracy are involved at each stage. Typically estimates would be prepared at the following stages:-

- Concept
- Preliminary Design
- Design
- Tender

Details of the estimates, their content, method of preparation and likely accuracy are given below:-

Concept - The first and most approximate estimate, the planning estimate, when the only information available will probably be the approximate length, width and location of the structure. Typically a rate per m^2 of deck area would be used. The estimator should try and ensure that the rate quoted is for a comparable structure, e.g. similar location, size, type, factored for inflation etc. although often only minimal information is available.

It is important at this stage to consider the construction method, as it could have a significant impact on the estimate.

In identifying the preferred concept, costs for ancillary works also need to be assessed. For example, costs of temporary works, detours etc may result in a bridge with more expensive construction costs being more cost effective when total project costs are considered. Other, intangible, costs such as traffic delays, adverse public reaction etc should also be considered.

The order of accuracy of a concept estimate is around $\pm$30 to 40%.

Preliminary Design - Prepared after more information is available and some preliminary design work has been carried out. Site investigation work may also have been completed. A more accurate unit rate can be used but to get a true comparison between alternative designs quantities should be calculated or estimated, and schedules prepared and priced, even if they are only approximate.

Again ensure that relevant rates are used and that all costs are included, including Bill 1, or general items, plus any Main Roads' overhead costs.

The order of accuracy of a preliminary design estimate will vary from $\pm$20 to 30%, depending on the information available and the amount of design work done.

This is probably the most important estimate, as it is the one which is used to compare alternative schemes and decide which to carry through to the final design.

Design - Continuous revision to previous estimates occurs during the design process as more accurate information becomes available. Any major changes to the scope of the structure that may occur during the detailed design phase should also be reflected in a revised estimate.

Order of accuracy increases from $\pm$20% to $\pm$10% as the design gets closer to completion.

Tender - From the final tender drawings and Schedule of Rates/Bill of Quantity. This should be within $\pm$10% of the final tendered price.

There are two main points which must be stressed concerning the estimates described above:-

1) The estimate is only as accurate as the information that goes into it. The accuracy of figures given must be borne in mind and pointed out to senior staff. If the estimate is to be used as the basis for decision making then appropriate contingency sums may have to be included to allow for estimating inaccuracies.

2) It is very important to record all estimates, including exactly what they are an estimate of, i.e. the state of the design at this stage and exactly what was allowed for in the estimate. For instance, were any earthworks included, service diversions, what construction method was assumed etc. This is very important when wanting to use these estimates as a guide for other jobs or when looking back and comparing the final cost of the job with the estimate.

CHAPTER 2

SELECTION OF BRIDGE TYPE

CONTENTS

2.1 INTRODUCTION

This Chapter contains a brief overview of the factors influencing the selection of bridge type and gives information on the types of structures most frequently used by MRWA.

2.2 FACTORS INFLUENCING SELECTION

The following are among the more important factors that should be considered when selecting the type of road bridge to be constructed at a particular site:

- Road geometry
- Bridge length
- Span length and configuration
- Method of construction
- Economics
- Durability/maintenance
- Aesthetics
- Possible future widening
- Type of crossing
- Site and foundation conditions
- Clearances (high/wide load route)

It should be recognised, however, that the above list is not exhaustive and the items are interrelated and selection of a bridge type should involve consideration of all relevant factors.

2.2.1 Road Geometry

Road geometry affects both the alignment and width of the bridge. The bridge designer should ensure that if possible, that the horizontal alignment of the bridge is straight or at the most on a large radius curve, such that it does not impact the choice of bridge type. Occasionally bridges have to accommodate roads with sharply curved alignments, such as the Freeway on-ramps at Manning Road and South Perth Interchange. In such instances the selection of bridge type is influenced by the high torsional loading. However in most cases the road alignment can be designed to accommodate a straight bridge.

2.2.2 Bridge Length

The length of a bridge is generally governed by the width of the obstacle to be crossed plus the clearances required.

The three types of crossing generally encountered are:

- Road over river or stream
- Road over rail (rail over road is normally the responsibility of the railway authority)
- Road over road (grade separation)

Generally, unless required for aesthetic reasons, bridge lengths are kept to a minimum because of the high cost of the bridge structure compared with road embankment.

In the case of a bridge over a river, the bridge length is determined by waterway / heritage requirements. Occasionally, bridge lengths over rivers may be required to span significant historical sites adjacent to river banks, making the bridge lengths greater than that purely for waterways requirements. For bridges over roads and railways, geometry and clearances usually govern.

2.2.3 Span Length and Configuration

The selection of span length requires careful consideration of the engineering, economic, environmental and aesthetic factors for any particular bridge. In general the greater the span the greater the bridge cost, but this is not always so, particularly when foundation costs are high and substructures are difficult to construct (e.g. in water).

Where aesthetics are important there is a tendency to select large spans so that an open, uncluttered under bridge appearance is achieved. Special care is needed when considering bridges that are highly skewed, very high or wide. Also, for aesthetic reasons the span should generally not be less than 1.5 times the height of the bridge above ground or water, (refer also Section 2.2.7 and Chapter 22).

For structurally continuous bridges, where possible the end span length is made around 80% of the internal spans, to equalise design moments.

Road over river - the spans of a bridge to be constructed over a navigable river are influenced by the size of the vessels that require to pass beneath the bridge. The size of the required navigable waterway is generally determined by the Department of Planning and Infrastructure (Marine Division) and the needs of that Department should be determined during the preliminary investigations for the bridge.

Where navigation is not a consideration then waterway requirements will govern and fix the overall bridge length. The individual span length is determined by the relative costs of deck and foundations, (shorter, cheaper spans require more foundations). Also consideration must be given to the maximum size of debris that is expected to be swept down the river during flooding.

Road over rail - the span lengths of rail bridges are determined by the clearances required by the relevant rail authority e.g. Public Transport Authority, WestNet Rail, Great Southern Railway, etc. A further consideration is the need to design substructures for the collision loads generated by a possible train derailment (see Clause 10.4 Part 2 of the CODE and Section 4.5 of this Manual). This can result in very heavy and expensive pier construction and consideration should be given to the economics of both moving the piers and increasing the bridge span or providing a single span with solid abutments, such as reinforced earth. The latter is the preferred option.

Road over road - the span lengths of grade separation structures are determined by the road cross-section and the minimum clearances required to piers and abutments (refer Chapter 12 of the Bridge Branch Design Information Manual). Future needs should also be considered.

2.2.4 Method of Construction

The likely method(s) of construction could have a significant impact on the selection of bridge type.

In general bridges are designed so that they can be constructed with a minimum of falsework.

This is because most rural bridges are over rivers with either permanent water or which may be subject to flash flooding. Hence, superstructures are often used which incorporate either precast prestressed concrete planks or beams, or steel beams. For reinforced concrete flat slab structures which are cast insitu the falsework is usually supported off the columns or piles. In special cases where bigger spans are required, incrementally launched superstructures, typically Tee-beams or boxes can be used.

Recent years precast prestressed concrete beams (Teeroffs) have been used for construction of many of the urban bridges in Western Australia. This facilitates reduced

construction times as beams can be lifted in place quickly to form the bridge span, as the beams provide a ready made working platform and permanent formwork for the casting of a reinforced concrete topping slab. The topping slab forms the deck running surface and provides composite action with the Teeroffs to carry the vehicle design loads.
The Teeroff has proved economical due to a number factors including reduced falsework and formwork requirements which in turn reduces the need for skilled personnel in these fields. One benefit is a consistent high quality 'off form" finish to the Teeroffs.
The Teeroff is also effective in minimising the impact of traffic disruption during the construction phase.

When contemplating the use of an insitu superstructure, which is to be constructed on falsework, the foundation conditions must be examined closely to ensure that excessive settlement will not occur during construction. It is good practice to test load the formwork/ falsework prior to placing concrete to ensure satisfactory strength and deflection control.

In environmentally sensitive areas the environmental impact of the falsework must be considered and if necessary, an alternative method of construction adopted. An example of this is where there are Aboriginal Heritage issues at a site, when construction in the watercourse may not be permitted.

2.2.5 Economics

All bridge designs should aim towards providing structures which give value for money. The value for money should include whole-of-life cycle considerations such as durability and maintenance costs.

That is not to say that the cheapest bridge to build is the one that is always selected. The particular bridge environment and hence, the emphasis placed on aesthetics, together with public needs and expectations all have to be taken into account. However, in many cases the initial cost is frequently the governing selection criteria.

It is usual to prepare concept designs for a number of technically viable proposals for each site and for each of these an estimate of cost is prepared. From these proposals a selection is made based on aesthetics, whole-of-life costs, maintenance requirements, etc.

To a certain extent the superstructures and substructures of MRWA rural bridges are standardised. This leads to economies in design, prefabrication and construction, through repetition and refinement of detail design and methods. It also allows standardisation of maintenance procedures.

In rural areas, transport and labour costs play a significant part in the selection of superstructure type. Steel I-beam superstructures have proved to be most economical in remote regions greater than 1000 km from Perth. It should be noted, however, that concrete options such as precast beams and planks should be considered as steel fabrication and surface treatment costs may in some instances exceed concrete manufacture and transport costs.

2.2.6 Durability/Maintenance

Durability and maintenance are important factors which have to be addressed during the design of a bridge. In WA we are fortunate to have a dry, warm climate with little air pollution. This, to a large extent has minimised the deterioration of our bridges when compared with cold climate countries that are forced to use de-icing salts to keep their roads open.

Durability and maintenance have not been significant factors, therefore, in bridge type selection, except in the following situations:

- It is MRWA’s practice to limit the use of steel beams or permanent steel formwork in bridges over large bodies of permanent water, whether fresh or saline. Where steel beams are used they should be corrosion protected, either with a paint system or by galvanising.

- Over streams in the South West, Wheatbelt and Goldfield Regions where runoff is saline, structures should be designed assuming the most aggressive environment.

- For piled substructures in saline conditions, concrete or composite steel and concrete piles should be used (see Chapter 7 of this Manual). Steel H-piles should be avoided if possible, due to corrosion problems. If used then a high quality, abrasion resistant corrosion protection system must be installed. Where reinforced concrete piles are cast insitu within driven steel tubular casings, it is generally the practice in design to ignore the contribution of the steel casing.

2.2.7 Aesthetics

MRWA places great importance on the aesthetics of its bridges, although the relative importance will vary depending on the bridge's location and the impact it will have on its environment.

Detailed aesthetic requirements are contained in Chapter 22 of this Manual. The Chapter outlines superstructure and substructure aesthetic guidelines for different category bridges. For urban bridges and other Category A structures it is desirable to have smooth, clean soffits and a minimum number of piers. It is also important to take into account the bridge environment and adjacent structures. For Category A bridges architectural input is mandatory.

For general guidance on aesthetics reference may be made to:

- Bruchen Bridges, Aesthetics and Design by Fritz Leonhardt
- The Aesthetics of Bridges, A reference Manual for Bridge Designers, Road and Traffic Authority NSW
- Bridge Design, Aesthetics and Developing Technologies edited by Bacow and Kruckemeyer
- The Appearance of Bridges, Ministry of Transport (UK)
- Bridge Aesthetics around the World, Transportation Research Board USA
- Bridgescape, The Act of Designing Bridges, Frederick Gottemoeller

2.2.8 Future Widening of Bridges

Frequently MRWA will stage road construction, widening the carriageway to its ultimate configuration some time in the future. The provision for future widening must be considered during the preliminary design phase of the project. Examples of this include the detailing of coupler reinforcement in the deck and overwidth footings. Often the accommodation of future widening is achieved by constructing a separate structure alongside the first stage. In some instances when construction of the second stage structure will be difficult, it will be necessary to provide for the ultimate road configuration in the first stage.

2.2.9 Type of Crossing

The type of crossing, over river, road or rail, has an influence on the type of structure, usually linked to the span lengths required and any special requirements for construction. These have mostly been covered in previous sections.

2.2.10 Site Conditions

For all new bridges a detailed site investigation / foundation investigation should be carried out in accordance with AS 5100 (CODE) and Chapter 5 of this Manual

Obviously the site conditions will influence the bridge type. In particular, subsoil conditions may affect the choice of foundation type, which may in turn affect the selection of superstructure type. For example, weak foundation conditions requiring deep, expensive piles may lead to a choice of a long span structure for economy, whereas there will be more options available where there are reasonable ground conditions and spread footings can be used.

Another site influence, particularly at river crossings, could be Aboriginal Heritage issues. This may preclude the use of driven piles, or in fact any form of ground disturbances in the river bed.

2.2.11 Clearances (High/Wide Load Route)

Certain routes have been declared high/wide load routes. On these, any overbridges must provide a minimum vertical clearance of 10.0 m unless there is an adequate bypass such a through a diamond shaped interchange at the intersection. This will influence the substructure type and possibly also the span lengths and overall bridge length. Refer to Main Roads Document 05/9235, Guide to the Design and Operation of High Wide Load Corridors.

2.3 SUBSTRUCTURE TYPES

2.3.1 Abutments

Abutments can be categorised into three main types:

- Spillthrough
- Reinforced concrete wall
- Reinforced soil

Spillthrough abutments - are generally used for most waterway structures. They are also used for other structures where selection is governed by economics and/or aesthetics. Spillthrough abutments to waterway structures are usually rock protected and frequently have guide banks attached to them (see Austroads Waterway Design Manual). Spillthrough abutments to road and rail bridges are usually protected from erosion by some form of treatment, such as stone pitching.

The roadway embankment slope is normally steepened up under the bridge, with the slopes of spillthrough abutments typically:

- Waterways structures - between 1.5h:1v and 2h:1v
- Other structures - between 2h:1v and 3h:1v

The form of a spillthrough abutment would normally be a high level spread footing or pile cap, or a spread footing at natural ground level with columns going up through the fill to a high level capbeam.

Wall type abutments - Where space is limited, e.g. at grade separations, or it is necessary to reduce span lengths, reinforced concrete wall abutments on spread or piled footings may be utilised. This type of abutment is generally not used for rail bridges as reinforced soil is usually cheaper and can be constructed more easily close to the rail track. Wall type abutments are also used to refurbish deteriorated timber abutments.

Reinforced Soil abutments - are generally used for rail bridges and as an alternative to reinforced concrete wall abutments in grade separations. In all cases, support for the bridge must be independent of the reinforced soil block, e.g. an abutment capbeam supported on piers, or piles founded below the block. The detailed design of the reinforced soil is normally carried out by the supplier with loads provided by the bridge designer.

2.3.2 Piers

Piers can be divided into three groups, those required for waterway structures, rail bridges and grade separations, each with different selection criteria. All types can be founded on spread or piled footings.

Waterway structures - Piers should be selected so that they are hydraulically streamlined and limit turbulence, can cope with the scour that is anticipated and can readily shed debris. They should also be able to sustain the dynamic stream flow and debris loading from flow skewed at selected angles, as the direction of flow can change during flooding. The ideal shape for this purpose is the circular column or pile. Wall shaped piers should be designed to take transverse loading from any anticipated skewed flow as indicated by Apelt (1965).

Rail bridges - Reinforced soil abutments are commonly used on railway bridges with a single, simply supported span, avoiding the use of piers altogether. Where this is not possible, piers that are located within 10.0 m of the rail centre line have to be designed for the collision loads generated by a derailed train. Additional requirements for use of reinforced soil walls are provided in the Bridge Branch Design Information Manual Section 2.

Grade separations - piers for grade separations are generally selected to suit aesthetic considerations.

Note also that piers located in areas viewed by the public are often given special architectural consideration (refer Section 2.2.7 above).

2.4 SUPERSTRUCTURE TYPES

2.4.1 Span length/Bridge type relationship

Figure 2.1 gives the range of span lengths applicable to the most common types of superstructure utilised by MRWA. For details see Sections 2.4.2 to 2.4.12

Bridge Type	Span (m)																								
	2	4	6	8	10	12	14	16	18	20	22	24	26	28	30	32	34	36	38	40	42	44	46	48	50
Timber	■	■	■	■																					
RC flat slab		■	■	■	■																				
RC planks			■	■	■	■																			
PSC flat slab					■	■	■	■	■																
PSC planks					■	■	■																		
PSC beam comp							■	■	■	■	■	■													
Steel beam comp						■	■	■	■	■	■	■	■												
PSC tees										■	■	■	■	■	■	■	■	■							
PSC voided slab												■	■	■	■	■	■	■	■	■					
PSC box																		■	■	■	■	■	■	■	■
Steel box																		■	■	■	■	■	■	■	■

Figure 2.1 - Superstructure Type v Span Length

2.4.2 Timber Bridges

Although timber bridges were once the most common form of construction, particularly in the south west of the State, and still comprise a very large proportion of WA's bridge stock, new timber bridges are no longer constructed. They are therefore not considered in this Chapter, although some details, especially of refurbishment works, are given in Chapter 14 of this Manual. Further information can also be found in Document No. 6706-02-2227, Load Rating and Refurbishment Design Manual for Existing Timber Bridges.

2.4.3 Reinforced Concrete Flat Slab Bridges

This design was developed for low cost, multiple short span bridges with minimum maintenance requirements. Span lengths are either 6 or 7.5 m with deck slab thicknesses of 330 and 350 mm respectively. Increased vehicle loadings such as MS 1600 to AS 5100 (CODE) will possibly require increased slab thicknesses to those shown, which are based on existing bridge stock. The deck slab is designed to be continuous over supports for dead and live loads. The deck kerbs are usually cast monolithically with the deck slab, which provides some edge beam stiffening to the slab, however, this is not generally allowed for in the design of the deck. Although once a fairly common form of construction, flat slab bridges are seldom considered for new bridges due to the necessity for intensive construction techniques including extensive falsework. Precast plank bridges would generally be a more economical option.

Longitudinal cracking can occur in the deck slab between piles, within the first few hours after casting, as lateral thermal shrinkage takes place with the dissipation of heat of hydration. At this time the concrete has not developed any significant tensile strength and restraint from the piles and formwork is sufficient to cause cracking. This is overcome by providing a minimum of 0.4% transverse reinforcement for the full length of the bridge, or AS 5100 Part 5 slab design requirements, whichever is the greater value.

The piers can comprise either driven piles or reinforced concrete columns cast into the deck slab. Piles can be of steel or prestressed concrete. Where foundation conditions permit, columns supported on spread footings are used. As the bridges are usually quite short they are generally fixed at the piers and expand towards each abutment. The piles/columns should be checked for the combined longitudinal and transverse loading, together with the effects of thermal expansion and shrinkage.

Depending upon the total length of the bridge and the region in which it is situated, spillthrough abutments are generally used with a number of different abutment details, (see Figure 2.2):

- Bridge lengths < 30 m anywhere in the State - no expansion joints are used. The deck is anchored in the abutment fill by a 2 m deep cut-off wall at the end of the deck slab which extends 1 m past the end support. Wall type abutments are also sometimes used (see Figure 2.3).

- Bridge lengths > 30 m and < 100 m - the end of the deck slab is supported on elastomeric bearings on an abutment capbeam with a rear wall and an expansion joint as shown on Figure 2.4.

- Structures > 100 m in length will require special consideration.

2.4.4 Precast, Reinforced Concrete Plank Bridges

MRWA in the past used a number of different types of precast, reinforced concrete elements in bridges e.g. Tees and inverted U beams. More recently a proprietary system has been developed, Rocla 'M' Lock which is applicable in the 6-12 metre span range. This is a fully precast system, comprising substructure and superstructure elements. A typical structure is illustrated at Figure 2.5.

However at the time of writing the use of 'M' Lock is not endorsed by Structures Engineering for use on Main Roads or Highways.

2.4.5 Prestressed Concrete Plank Bridges

Precast prestressed concrete plank bridges were introduced by MRWA to minimise on-site concrete pours in remote areas and hence, reduce costs. However, on-site batching plants have become more readily available in recent years and other forms of construction are proving to be more economic.

Precast prestressed concrete planks are currently identified as a viable option for new small span bridges in less remote areas mainly in the south of the state. Main uses identified are for replacement of short span timber bridges which have reached the end their useful life, new bridges and replacement for culverts located in aggressive environments.

Standard precast prestressed rectangular planks have recently been upgraded to the CODE and with standard lengths of 6 m, 9 m and 12 m.

The planks are designed to be simply supported. The bridge deck is formed by placing the planks side by side and transverse distribution of load is achieved with a concrete shear key between the planks and either transverse prestress or a reinforced concrete overlay. The latter is usually preferred. Where transverse prestress is used the hog of the beams is compensated for by placing an asphalt wearing course directly on top of the planks.

Each plank is supported on discrete rubber pads or a continuous rubber strip bearing. Dowels are provided at alternate ends of the planks to fix them to the capbeam supporting the deck. Part of the dowels just above the bearings is encased in plastic foam to allow relative rotational movement of the deck and pier. The gaps between the ends of the planks are filled with cement mortar.

Substructures comprise capbeams on either driven piles or columns on spread footings. Prestressed concrete, steel H-pile and concrete filled steel tubular piles are commonly used. Typical details of plank bridges are given on Figure 2.6.

2.4.6 Precast Prestressed Concrete I-Beam Bridges

Precast prestressed concrete I-beams with an insitu composite reinforced concrete deck slab have been utilised by MRWA where longer span bridges were required. However, they have now been replaced by the more economical precast concrete Teeroff sections, or steel I-beams. These notes are included for information only, as there are still many of these bridge types in use. The standard NAASRA Type 2 sections were mostly used but modified to increase the depth of the section from 900 to 915 mm. These have been designed to give a standard span length of 18 m with a beam spacing of around 1.5 m for T44 loading.

The beams are designed to be simply supported for dead load and continuous for live load. The continuity over the piers is achieved with non-prestressed reinforcement in the deck slab. Positive moments at supports due to a combination of temperature, creep and differential shrinkage are provided for by a welded connection at the level of the bottom flanges of the beams. This connection is effected by welding together a number of positive moment bars cast into the ends of the beams. The beam ends are generally cast into transverse diaphragms at supports.

The reinforced concrete deck slab is usually cast on permanent steel decking, e.g. Bondek. Composite action between the beam and slab is achieved through the roughened top surface of the beam and shear reinforcement.

The superstructure is supported on elastomeric bearings. Lateral restraint for the forces resulting from streamflow is provided by fabricated steel shear keys cast into the piers and abutments, and the diaphragms. No vertical anchorage is required against flotation during overtopping as the buoyant weight of the deck exceeds any upward forces, however, air release holes should be incorporated in the deck.

Substructures usually comprise a capbeam supported on either driven piles or, columns or wall type piers on spread footings

Typical details of a precast concrete I-beam bridge are given on Figure 2.7.

2.4.7 Steel I-Beam Bridges

Steel I-beam bridges utilising Universal or Welded Beams, of various masses, and composite reinforced concrete deck slabs have been used for bridges with spans of between 15 to 26 m.

The beams are designed as simply supported for their own dead load, continuous for the wet concrete of the deck slab and composite for live load. Continuity can be provided in the steel beams by welding or bolted joints. Live load continuity is achieved with tensile reinforcement in the deck slab. Cross bracing is often used to limit the critical buckling length of the compression flange, and/or provide stability during construction.

A transverse reinforced concrete diaphragm is usually provided at supports, although steel cross bracing can also be used. The reinforced concrete deck slab is usually cast on permanent formwork, either steel decking, e.g. Bondek or precast concrete, e.g. Hume Slab. Composite action between the beams and slab is achieved with shear studs welded to the top flange of each beam.

Steel bearing plates are welded to the bottom flanges of the beams which bear directly on elastomeric or sliding pot bearings. Substructures are similar to those utilised for the precast I-beam bridges, except that provision is made to not only restrain the superstructure against the forces resulting from streamflow, but to provide vertical restraint against flotation because of the light weight of the superstructure.

A typical steel I-beam composite bridge is shown at Figure 2.8.

2.4.8 *Prestressed Concrete Tee-Beam Bridges*

Prestressed concrete tee beam bridges have been frequently used in the past for continuous bridges up to spans of 35 m. They have been constructed both insitu and incrementally launched. More recently, simply supported precast concrete Teeroff bridges have been used.

a) Cast insitu and Incrementally Launched Bridges

Cast insitu and incrementally launched tee beam bridges typically consist of tee beams 1.2 m – 2.0 m deep at 7 m centres joined by an RC slab deck and outer cantilevers. As such it is ideally suited to a two lane bridge. Wider structures with more beams are possible, but primarily as cast insitu, it is difficult to launch more than a twin beam deck. For launching, the deck would normally have a constant depth, but with cast insitu bridges haunched construction has been used to assist with the high compressive stresses typically encountered over the supports.

The advantages of this type of structure lie in its simple and easy construction, because it generally has:

- constant cross sections;
- simple concrete pours; and
- casting of the section in one operation.

This ease of construction will generally reflect in savings in construction cost.

Disadvantages of this type of bridge are:

- less efficient span/depth ratio;
- compressive stresses over the supports govern the design; and
- a cluttered soffit, if used for wide bridges with a number of beams.

Span to depth ratios of up to the following values have been achieved for insitu structures:

Straight Tee-beams		**Two Span Haunched Tee-beams**	
End Spans	Internal Spans	Support	Mid Span
19	27	15	30

For incrementally launched tee-beams a span to depth ratio of 15 can be achieved.

A typical launched tee-beam structure is shown at Figure 2.9.

b) Precast Concrete Teeroff Bridges

In recent years a new form of precast, prestressed concrete section, the Teeroff beam, has come into use in Australia for medium span bridges. For full details on its development and use refer Bridge Beam Development – The Teeroff, John Connal, Austroads 1997 Bridge Conference.

In essence, it is a precast concrete tee section, with the body an open, hollow, box type section and with a wide top flange. The units are placed next to one another and touching to form an instant deck soffit form and safe working platform. A composite slab is then poured to complete the deck. Structurally each span is simply supported, although the slab can be made physically continuous to eliminate joints over the piers.

The standard Teeroff section is available in a number of depths from 750 mm to 2250 mm to suit different span ranges up to a maximum of 42 m. Span:depth ratios of 15-17 are normal. Typically the beams are placed at 3.5 to 4.5 metre centres.

For a typical standard and large Teeroff bridge refer Figures 2.10 and 2.11 respectively.

2.4.9 Prestressed Voided Slab Bridges

In the past MRWA has used insitu prestressed voided slab bridges for continuous spans up to 40 m and with span to depth ratios of up to 30. Voided slabs can provide elegant, slender bridges, but they have now generally become uneconomical compared with Teeroff type structures.

With a voided slab deck care has to be taken during casting to ensure that the void formers do not deform or deflect as a result of uneven pressures from the wet concrete. It is also necessary to consider and specify a careful pour sequence to ensure that longitudinal cracking of the concrete over the top of the voids due to concrete settlement does not occur.

The advantages of voided slabs are:

- more efficient span/depth ratio than tee beams;
- single pour construction; and
- a smooth soffit.

The disadvantages of voided slabs are:

- care is needed during construction to avoid displacement of the void former and consequential deck cracking;
- dead load of bridge deck is "high";
- falsework is required; and
- high labour costs.

A typical voided slab structure is shown at Figure 2.12.

2.4.10 Prestressed Concrete Box Structures

This type of bridge can comprise a single cell or multicell box.

It can be constructed by a number of methods and in WA we have examples of insitu construction, incremental launching and segmental construction.

Bridges with the following maximum span lengths (m) and span to depth ratios (shown in parenthesis) have been constructed in WA:

	Single Cell	**Twin Cell**	**Multiple Cells**
Single box - insitu	49 (l/d = 25)	-	46 (l/d = 25)
Twin boxes - insitu	44 (l/d = 25)	43 (l/d = 25)	-
Single box - segmental	-	76 (l/d = 25)	-
Single box - incremental	-	67 (l/d = 25)	-

For longer span bridges, box structures offer the most efficient span to depth ratios and are ideally suited for bridges where the depth of the superstructure needs to be kept to a minimum.

The advantages of box structures are:

- suitable for large span bridges;
- most efficient span/depth ratios;
- suited to a number of construction methods;

- suitable for carrying large torsional loads; and
- efficient use of structural materials.

The disadvantages are:

- usually require two stage deck pours; and
- complex construction and therefore expensive.

Typical box section decks are shown at Figures 2.13 and 2.14.

2.4.11 Steel Box Bridges

Steel box bridges with composite reinforced concrete deck slabs can have single or multiple boxes. This type of structure is generally used in situations where falsework cannot be utilised, such as in the construction of a bridge over an existing road which carries heavy traffic. The box (or boxes) are prefabricated and lifted into position, and the deck slab cast insitu with formwork supported off the box.

Very few structures of this type have been constructed in WA, because there have not been many situations where they have been required. Examples are the Causeway East Interchange Bridge, Lord Street grade separation and bridges over the Graham Farmer Freeway.

The advantages of steel box structures are:

- they are relatively light;
- they can be constructed without falsework; and
- they can be fabricated to match complex road geometry.

The disadvantages are:

- they require more maintenance than concrete structures; and
- they are currently more expensive than the equivalent concrete structure.

2.4.12 Special Bridge Types

For most locations the type of bridge selected will be one of those described above. Occasionally, however, a bridge is required to fulfil special criteria that are not met by the common bridge types utilised by MRWA. Examples of this type of bridge are the major road crossings of the Swan and Canning Rivers in the Metropolitan area, where considerable care has been taken with aesthetics and large spans selected. Because of the large spans and the cost of providing falsework, segmental construction and incremental launching have been utilised in their construction. Other forms of construction that could be utilised are cantilever construction and advanced shoring (travelling formwork).

The special bridge category is not restricted to prestigious bridges across major rivers and such bridges may be required where particularly difficult engineering, economic or aesthetic conditions have to be overcome.

Precast concrete arches, e.g. Techspan and the Bebo arch, are another form of special construction that might be suitable in certain circumstances. They are especially useful in construction over operating railways, as minimum closure periods are required for erection. The principal problem with them is that because of the shape of the arch, plus the minimum fill requirements, a greater clearance is usually required than with a conventional structure. They have their use in the right situation though. MRWA has made little use of corrugated steel arches for significant crossings, due to concerns about their behaviour under heavy load vehicles and long term durability.

2.5 FOOTBRIDGES AND PEDESTRIAN UNDERPASSES

Footbridges and pedestrian underpasses are facilities for the passage of pedestrians and cyclists. It is also usual to make provision for access by people with disabilities.

The specific clearance and design criteria for footbridges and pedestrian underpasses are given elsewhere in this Manual and the CODE. Requirements for cyclists are in Austroads Guide to Traffic Engineering Practice, Part 14 Bicycles and MRWA Policy for Cycling Infrastructure; and for access by the disabled in AS 1428.1.

Generally the choice between a footbridge or an underpass depends on the conditions at the site of the facility. However, MRWA has found that footbridges are generally perceived as less threatening than underpasses and are the preferred solution, except where an underpass can be kept short in length and clear unimpeded views can be achieved through the length and that of the approaches. In general, if underpasses are selected they should be spacious to prevent claustrophobic effects. Wall surfaces should be capable of being easily cleaned of graffiti and lighting should be vandal proof.

Complying with the disability access requirements of AS 1428.1, plus MRWA Cycling Policy, results in fairly long ramps. These would typically be a minimum of 120 metres for a footbridge and 70 metres for an underpass.

Because footbridges are very visible structures a great deal of care should be taken with aesthetics and architectural advice sought.

For spans up to 38 m MRWA has used a standard footbridge comprising a composite steel box with an insitu concrete deck slab. This is particularly useful for construction over operating roads. For footbridges requiring larger spans, which need to meet high aesthetic standards, prestressed concrete has normally been the medium for construction, usually with haunched spans. Alternatively, cable stayed construction in steel or concrete can be used, especially where the client wants a notable or “landmark” structure.

With underpasses, straight box sections are possible, but consideration must be given to longer spans to provide a more open, inviting aspect. Precast concrete solutions are also possible, boxes or preferably arches if there is the room available.

Some typical footbridges and underpasses are shown at Figures 2.15 and 2.16.

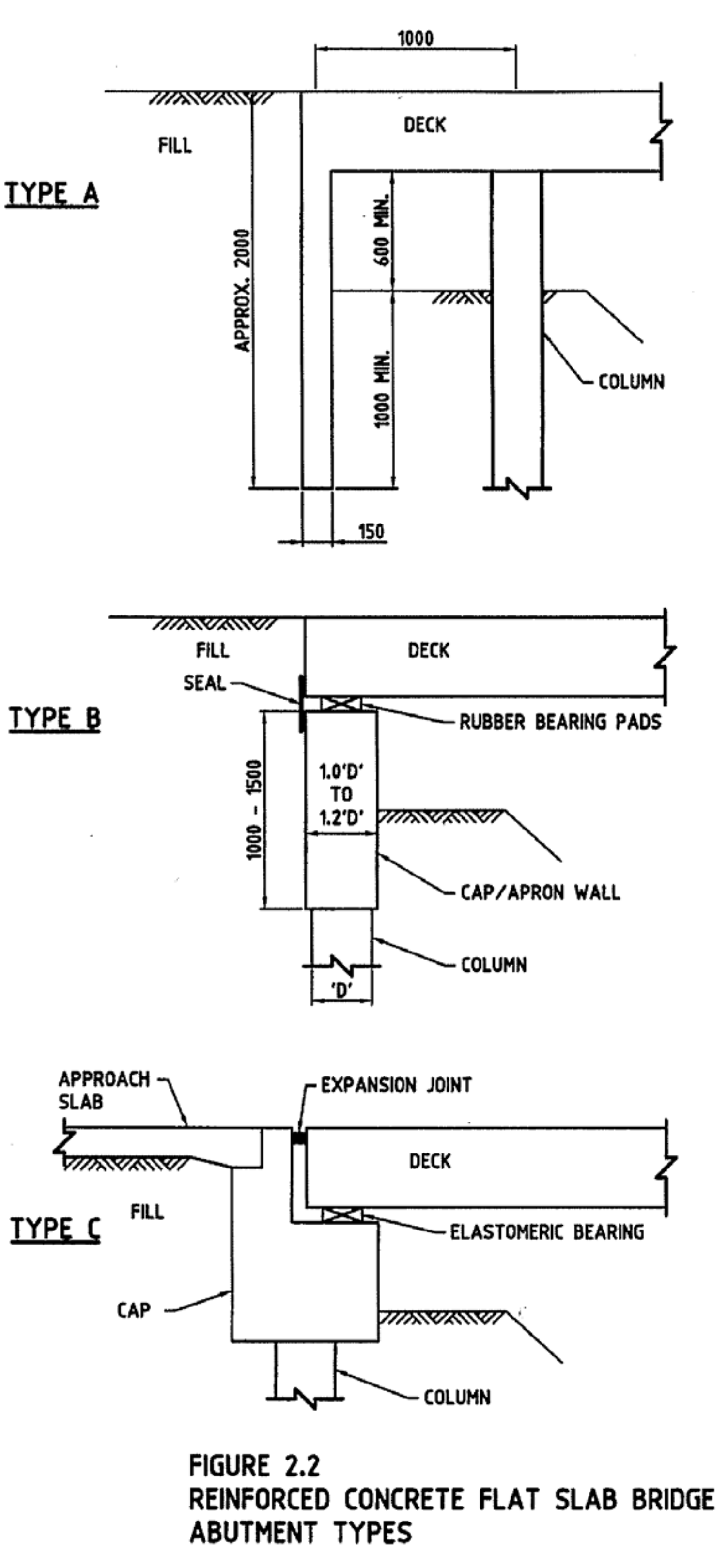


Figure 2.2 - RC FLAT SLAB BRIDGE ABUTMENT TYPES

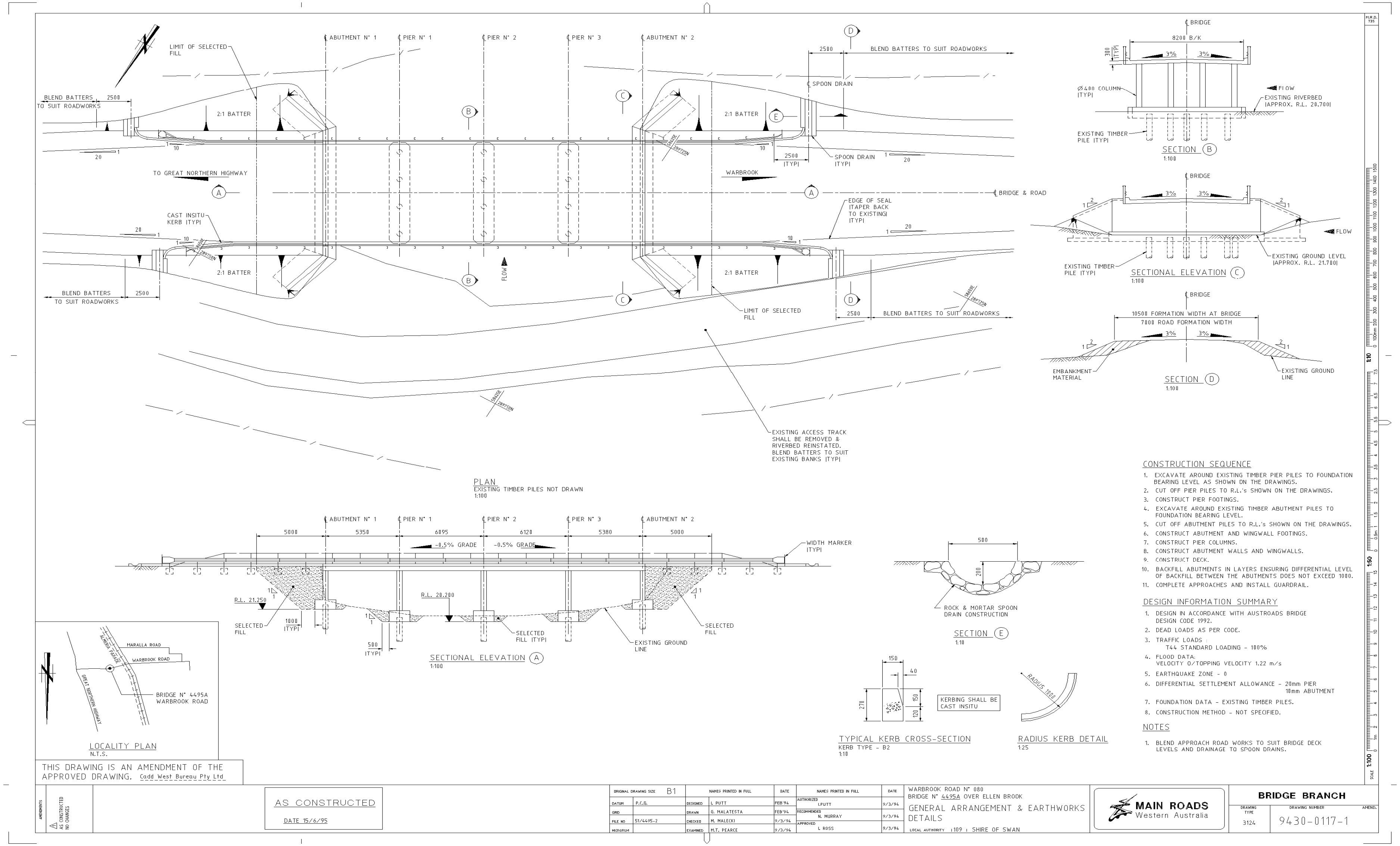


Figure 2.3 - FLAT SLAB BRIDGE

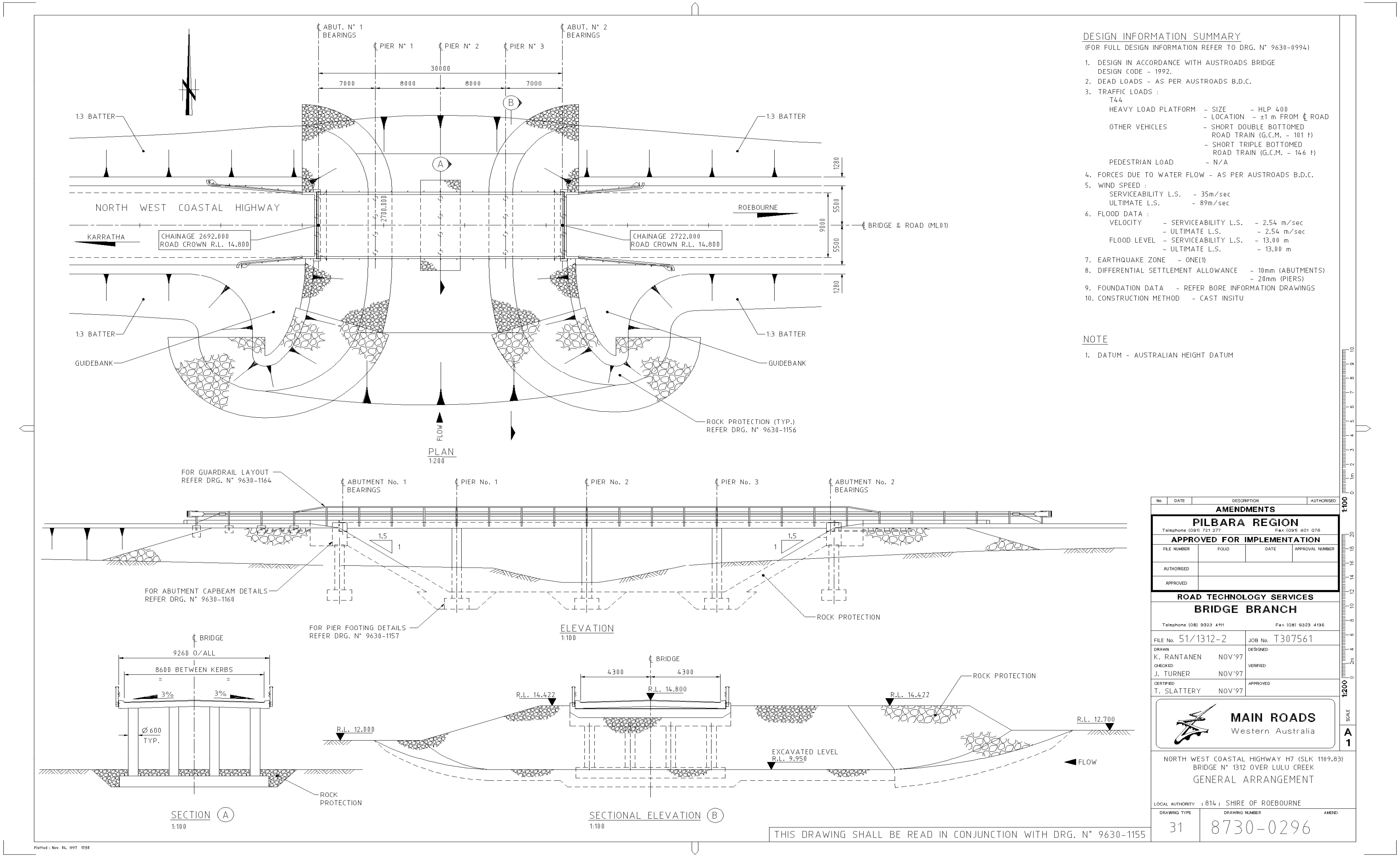


Figure 2.4 - FLAT SLAB BRIDGE WITH EXPANSION JOINTS

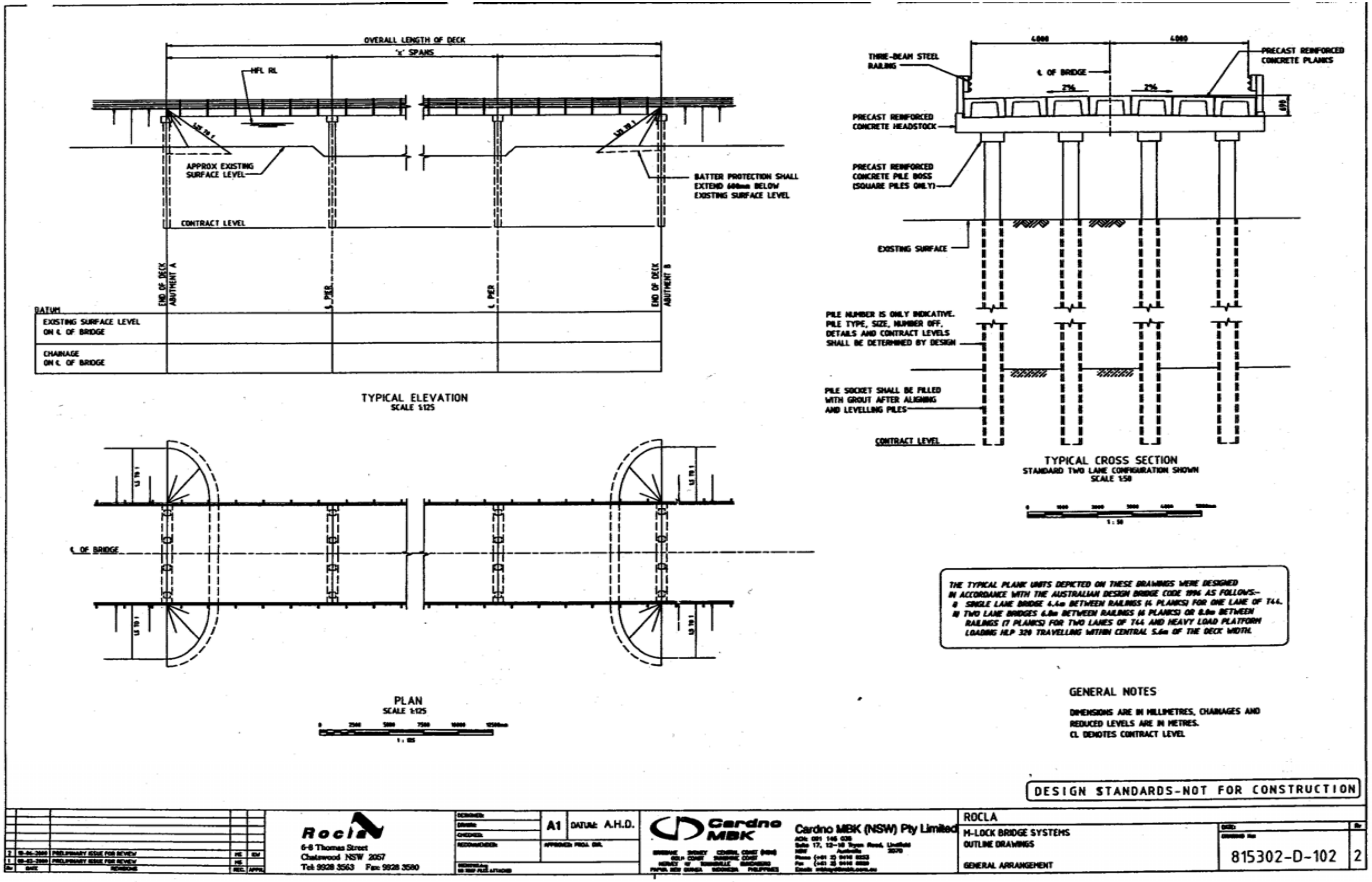


Figure 2.5 - ROCLA 'M' LOCK BRIDGE

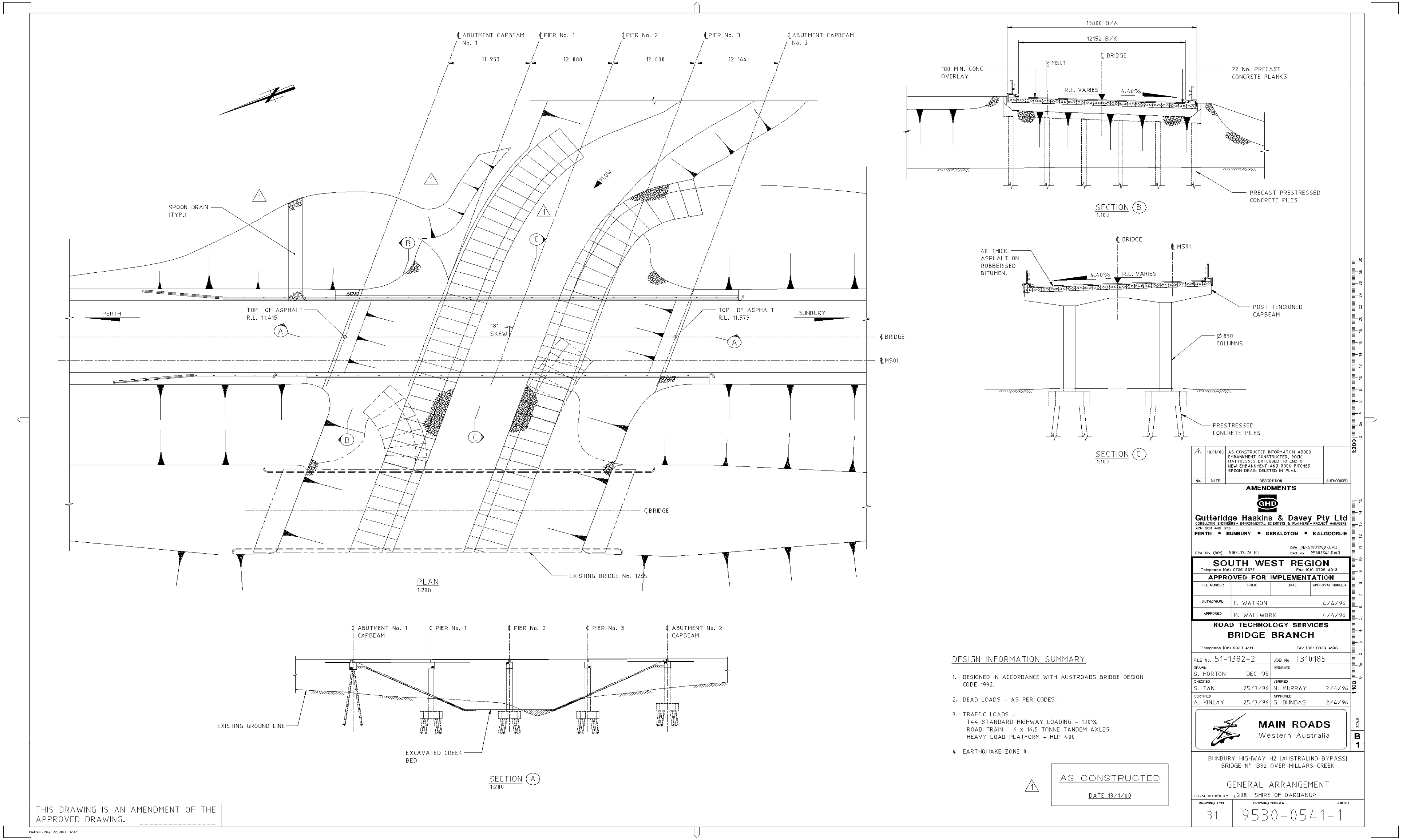


Figure 2.6 - PLANK BRIDGE

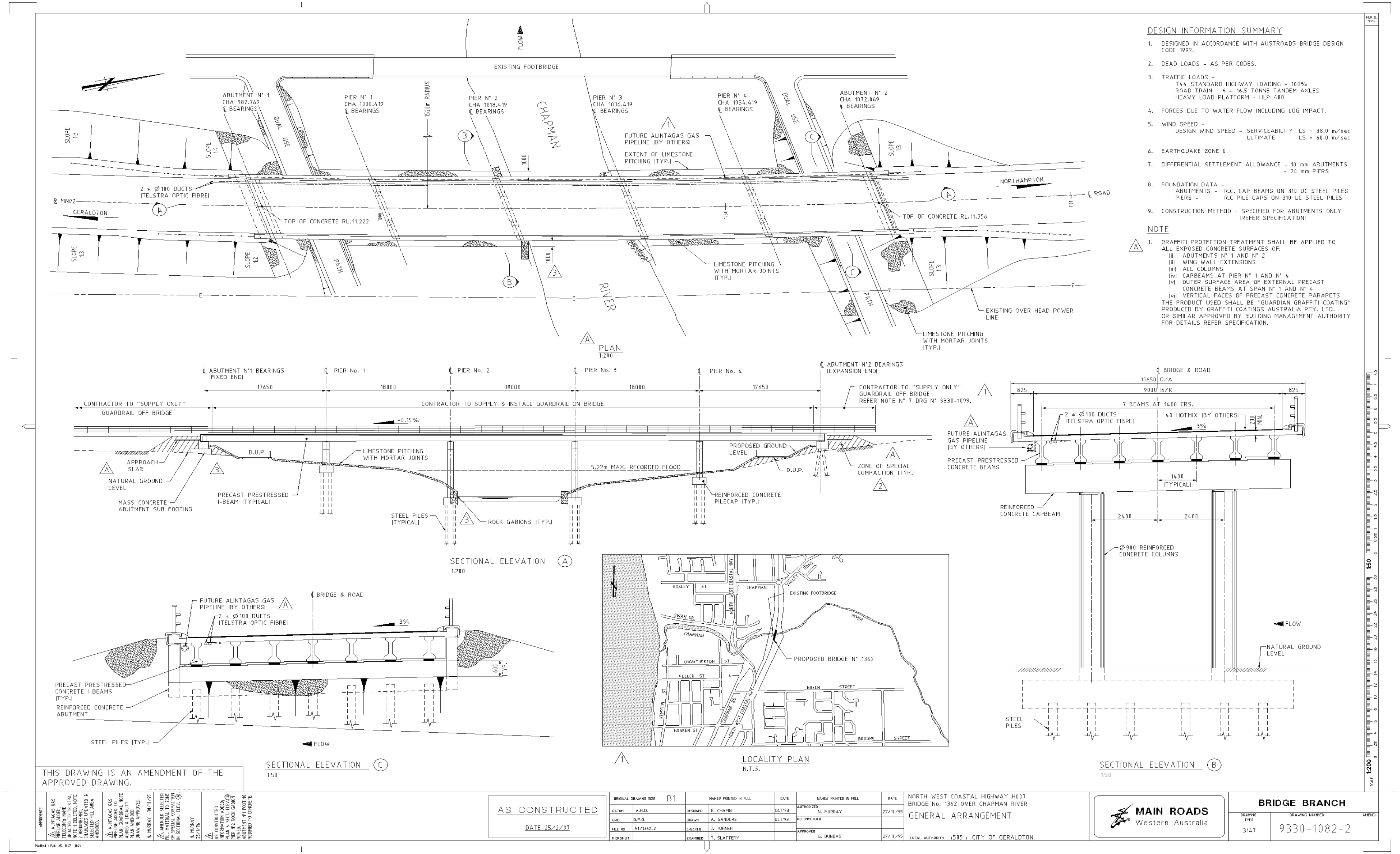


Figure 2.7 - PRECAST CONCRETE I-BEAM BRIDGE

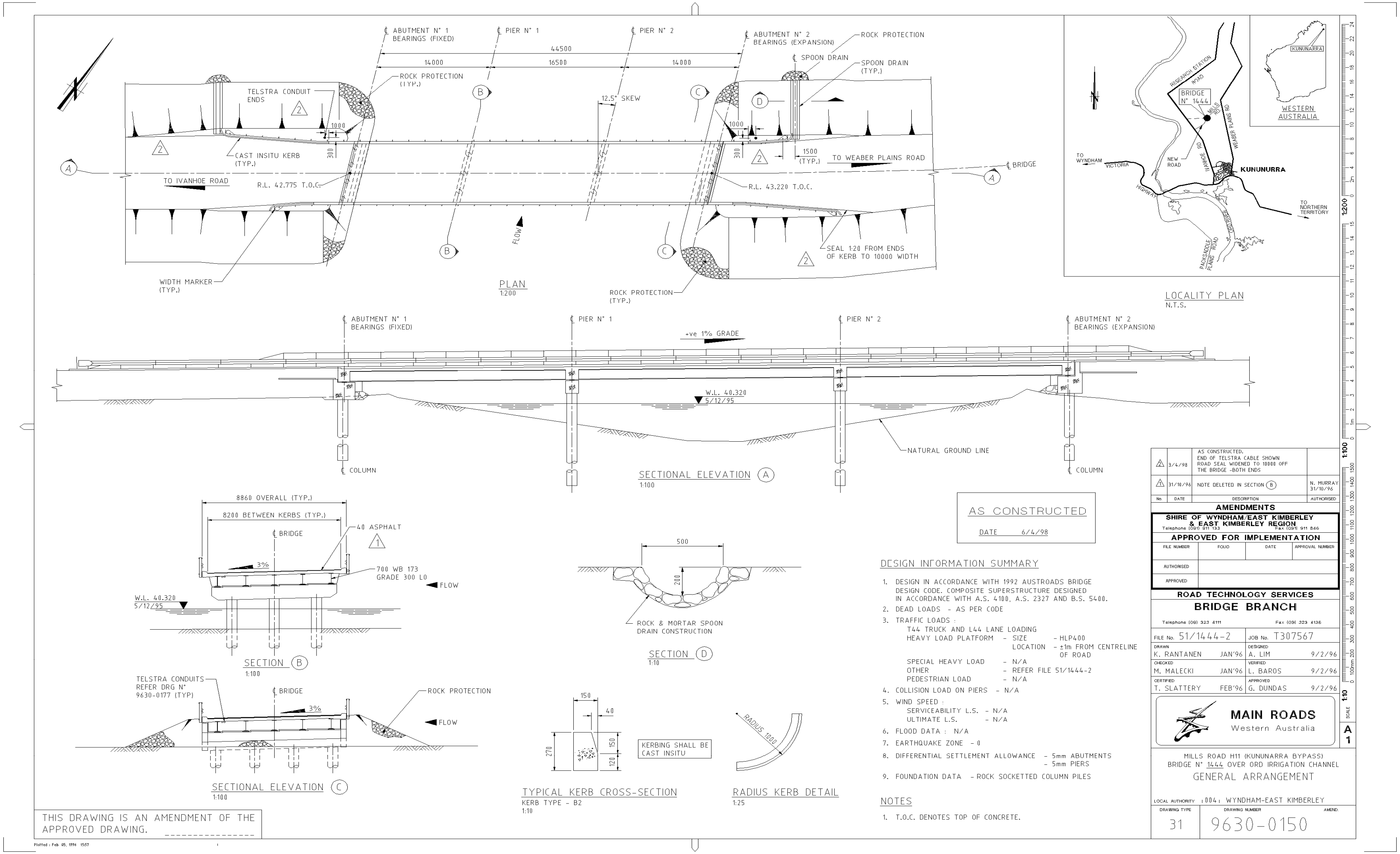


Figure 2.8 - STEEL I-BEAM BRIDGE

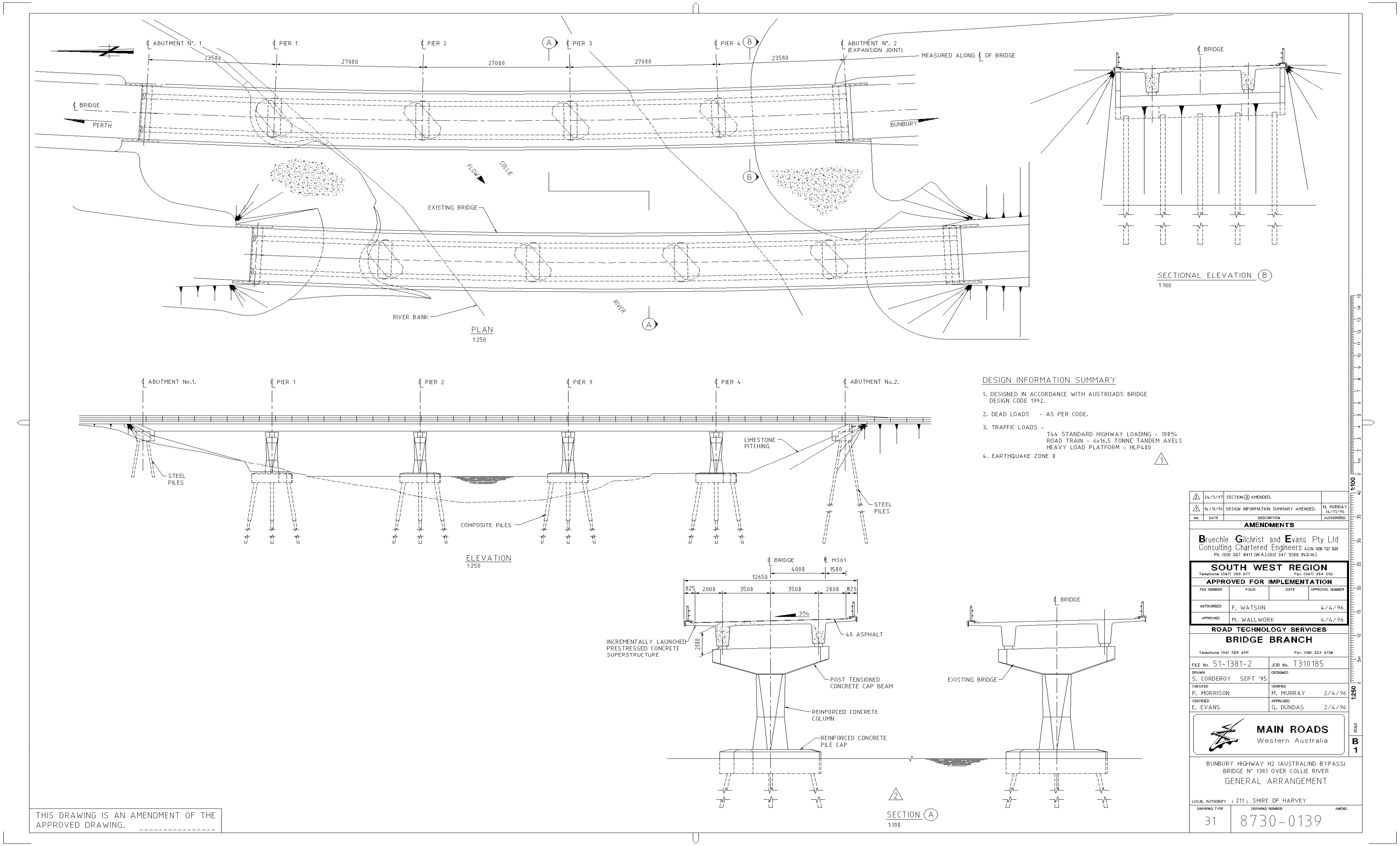


Figure 2.9 - PRESTRESSED CONCRETE TEE-BEAM BRIDGE

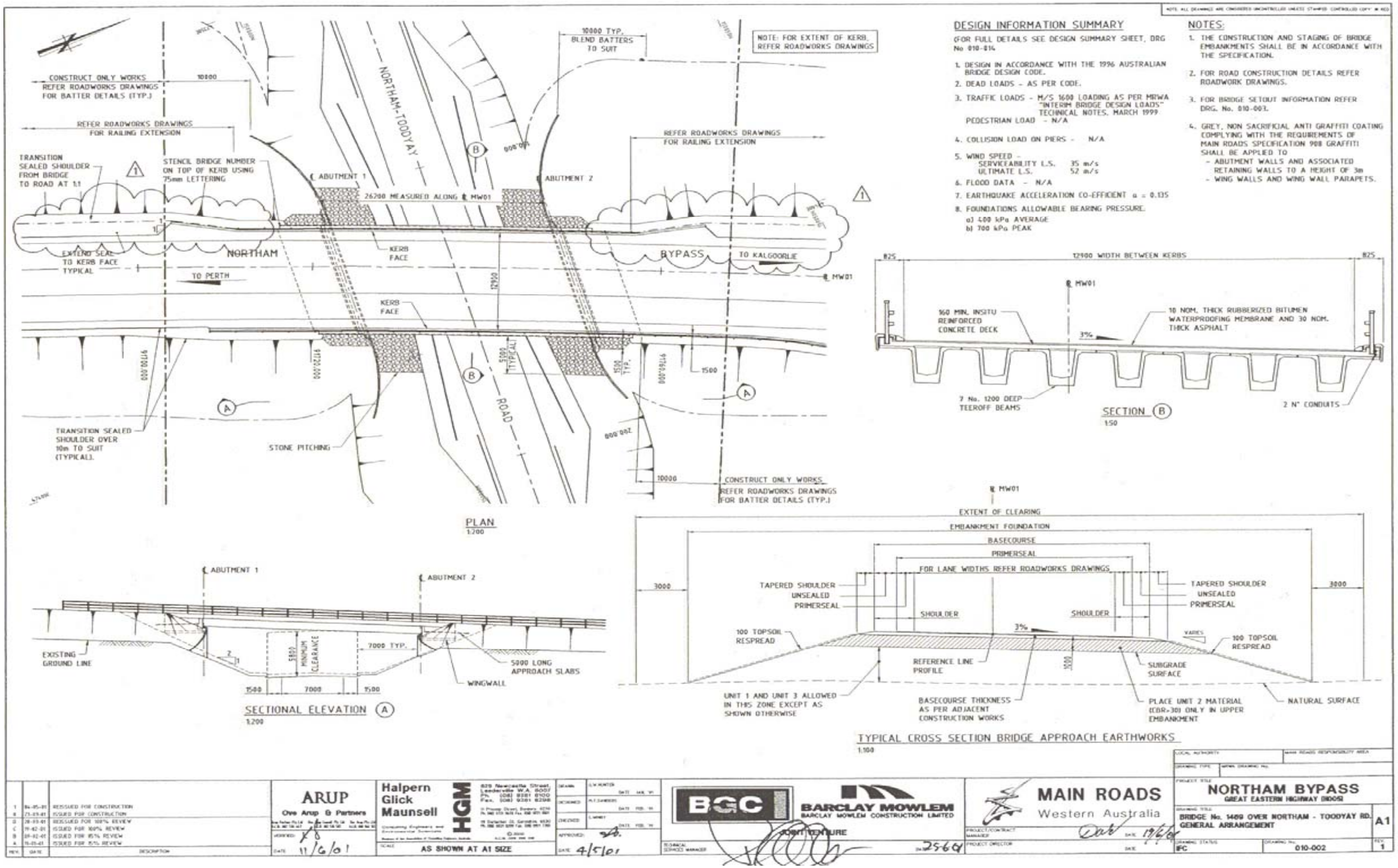
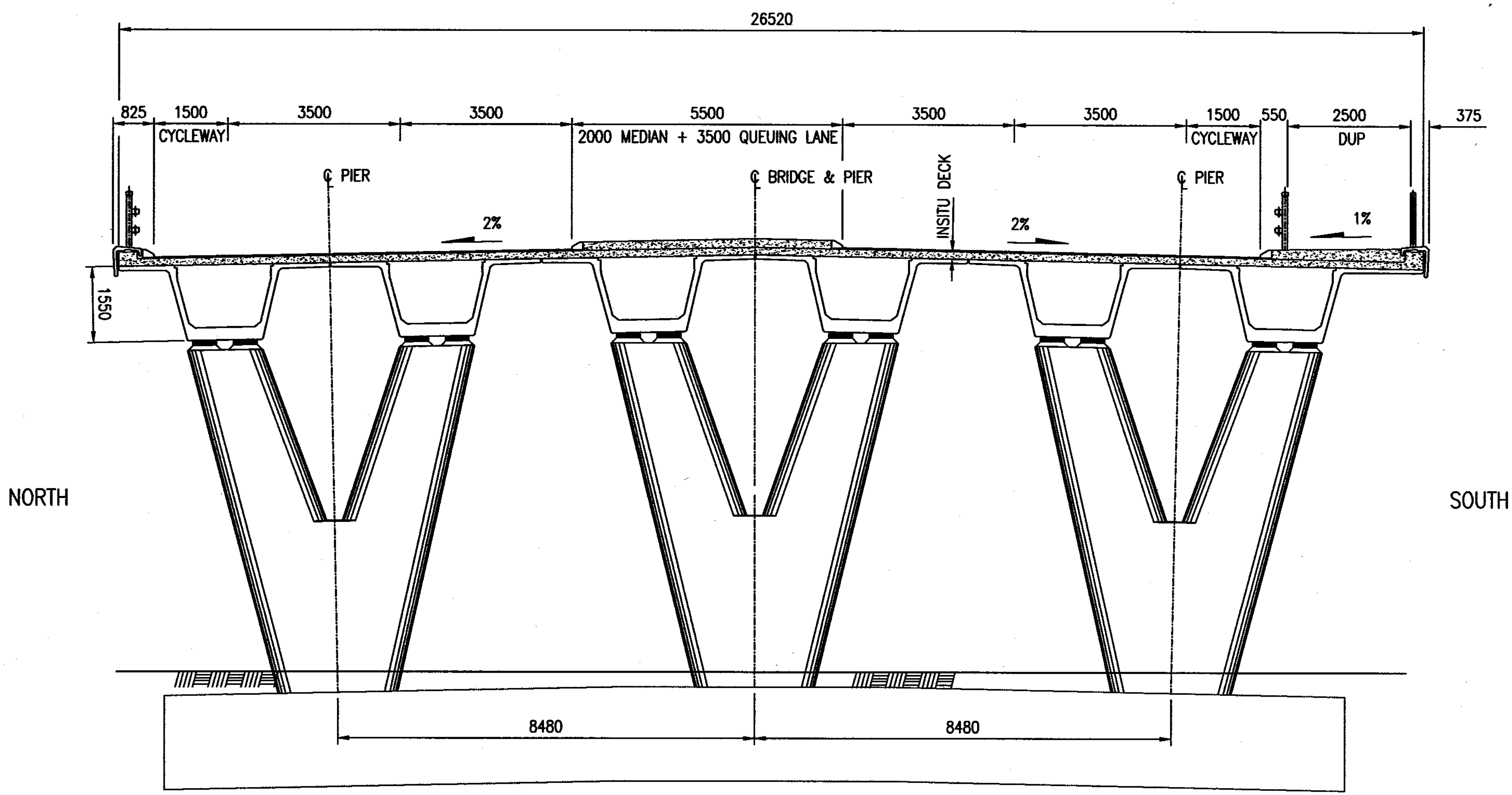


Figure 2.10 - PRECAST CONCRETE TEEROFF BRIDGE



BRIDGE 1455 CROSS SECTION AT PIER
1:125

Figure 2.11 - LARGE TEEROFF BEAM BRIDGE

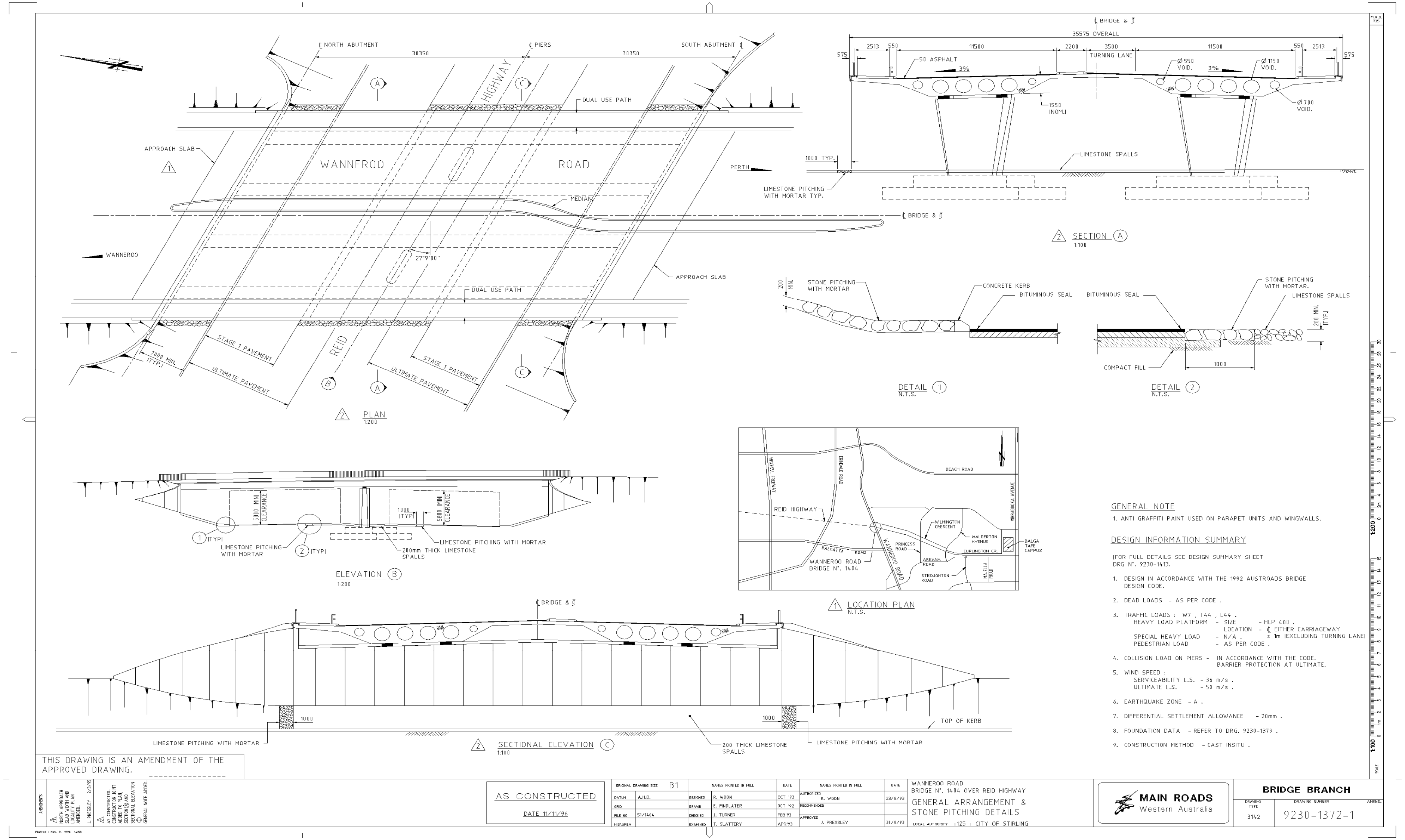


Figure 2.12 - PRESTRESSED VOIDED SLAB BRIDGE

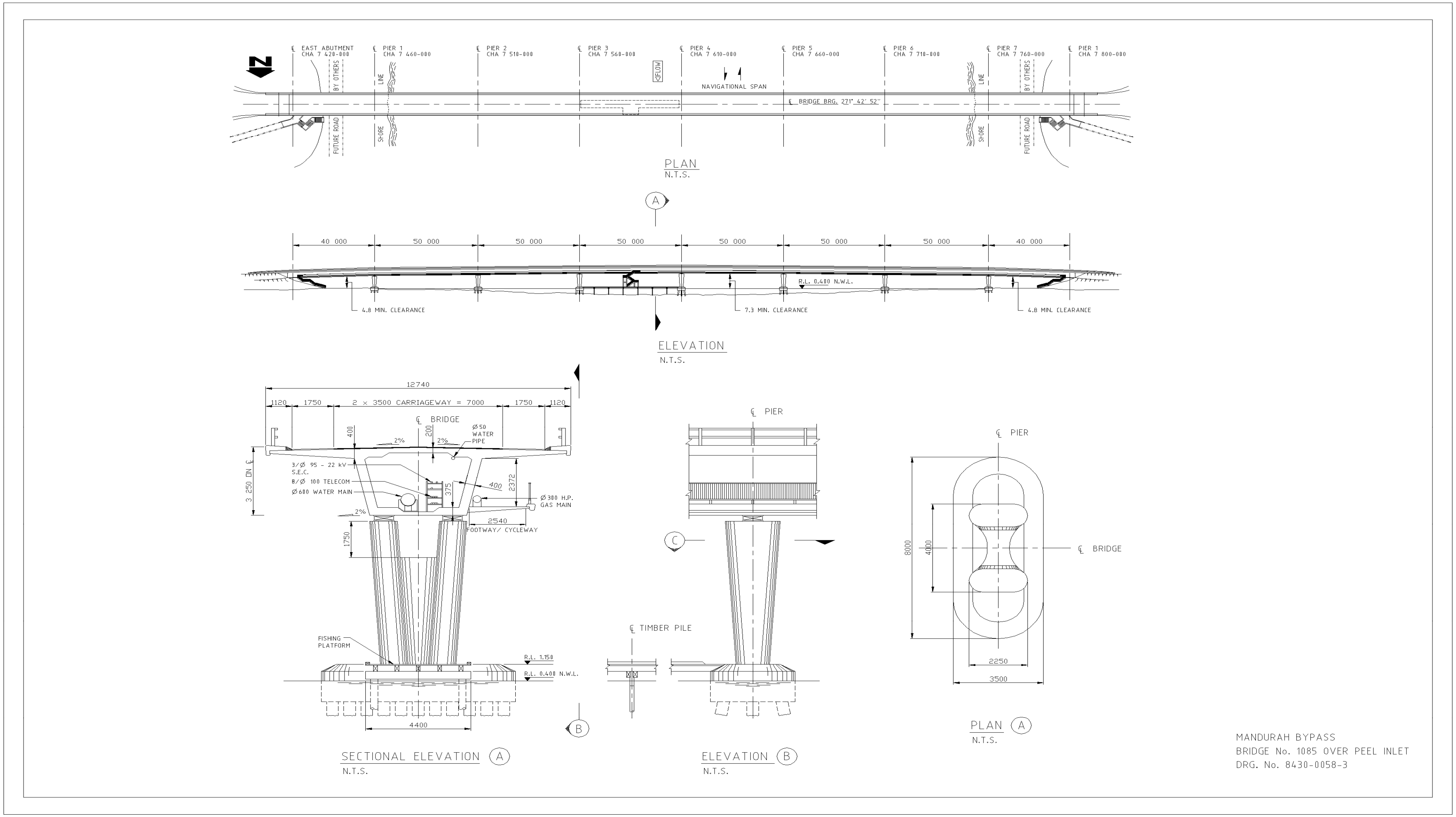


Figure 2.13 - SINGLE CELL PRESTRESSED CONCRETE BOX BRIDGE

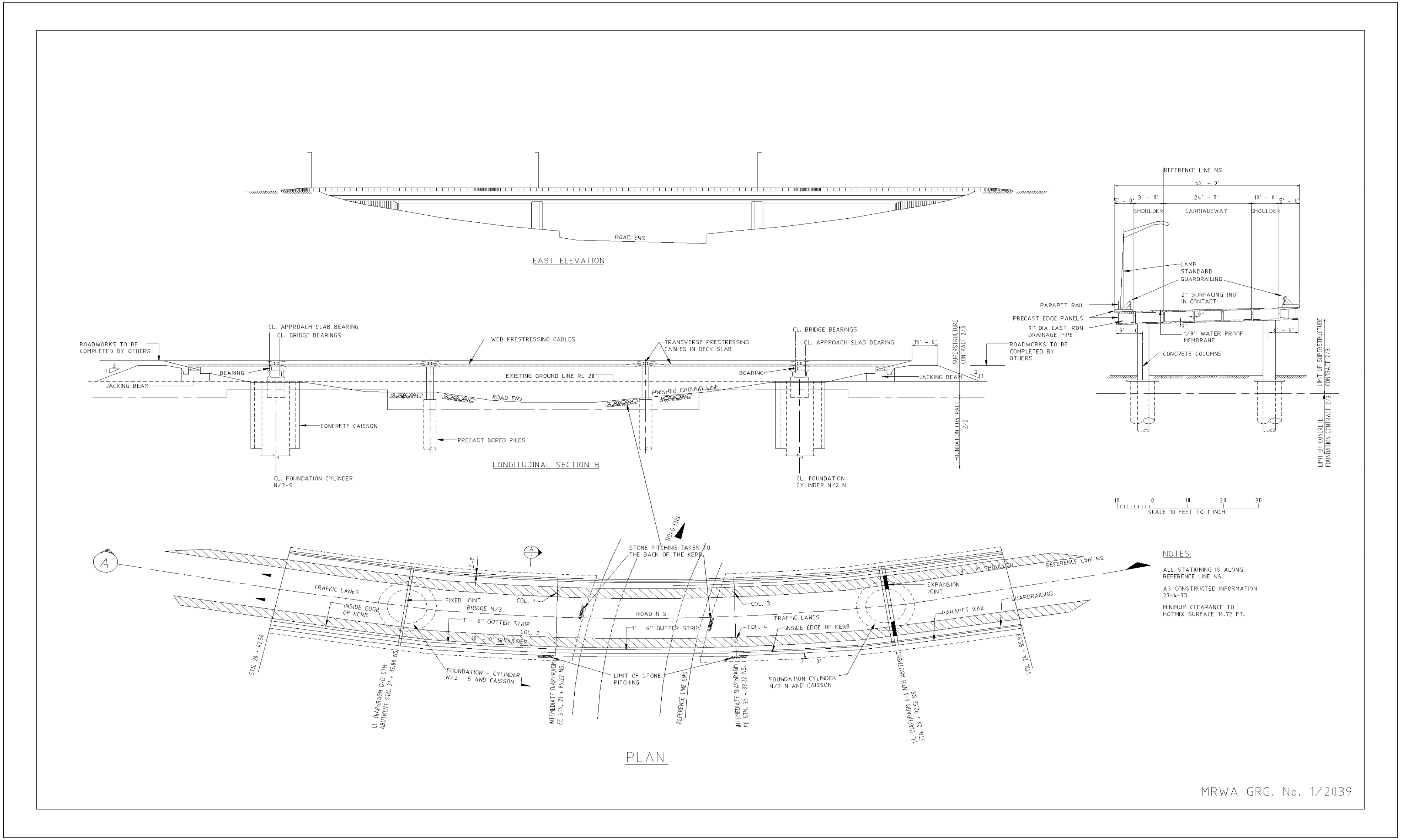


Figure 2.14 - MULTICELL PRESTRESSED CONCRETE BOX BRIDGE

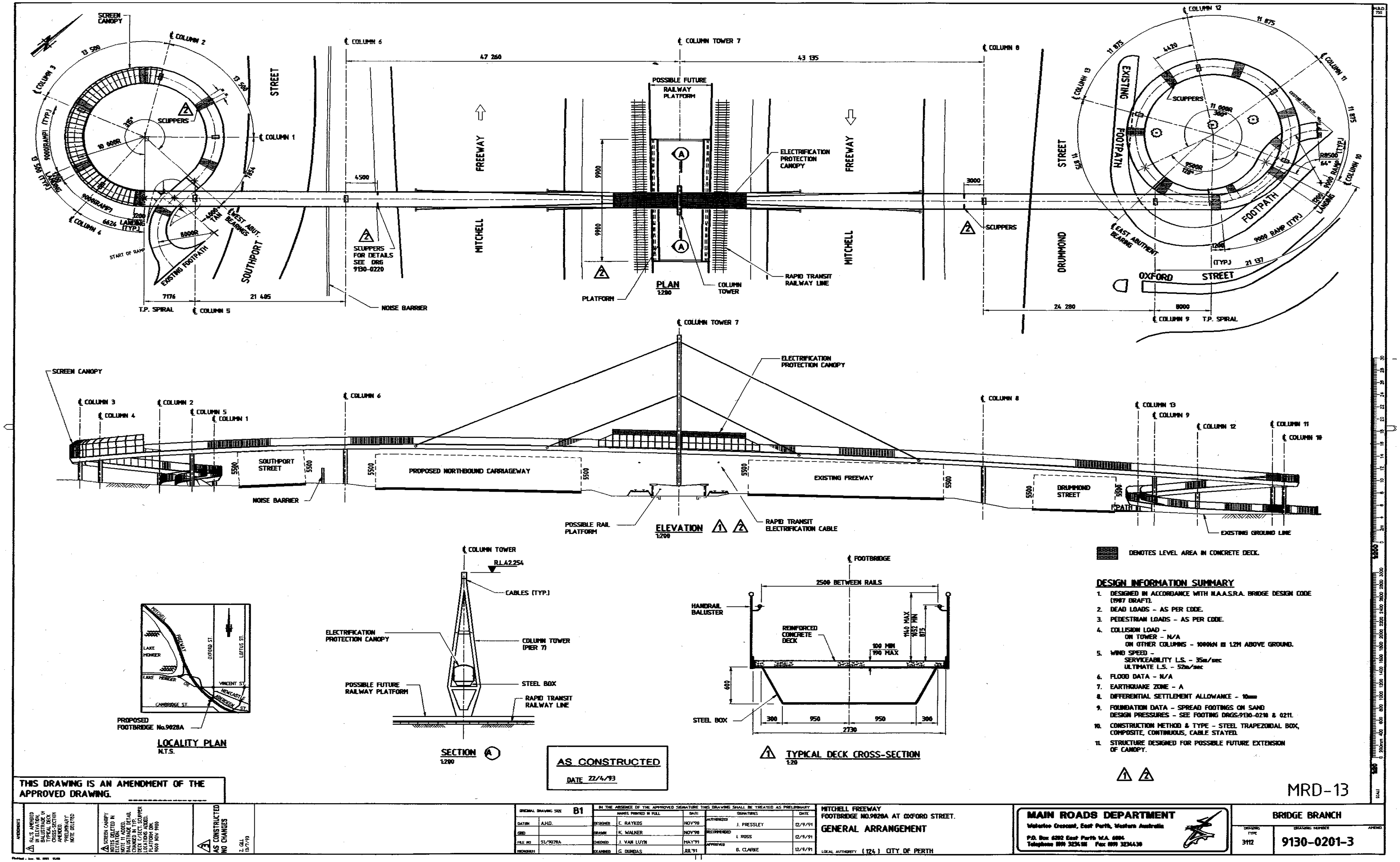


Figure 2.15 - CABLE STAYED FOOTBRIDGE

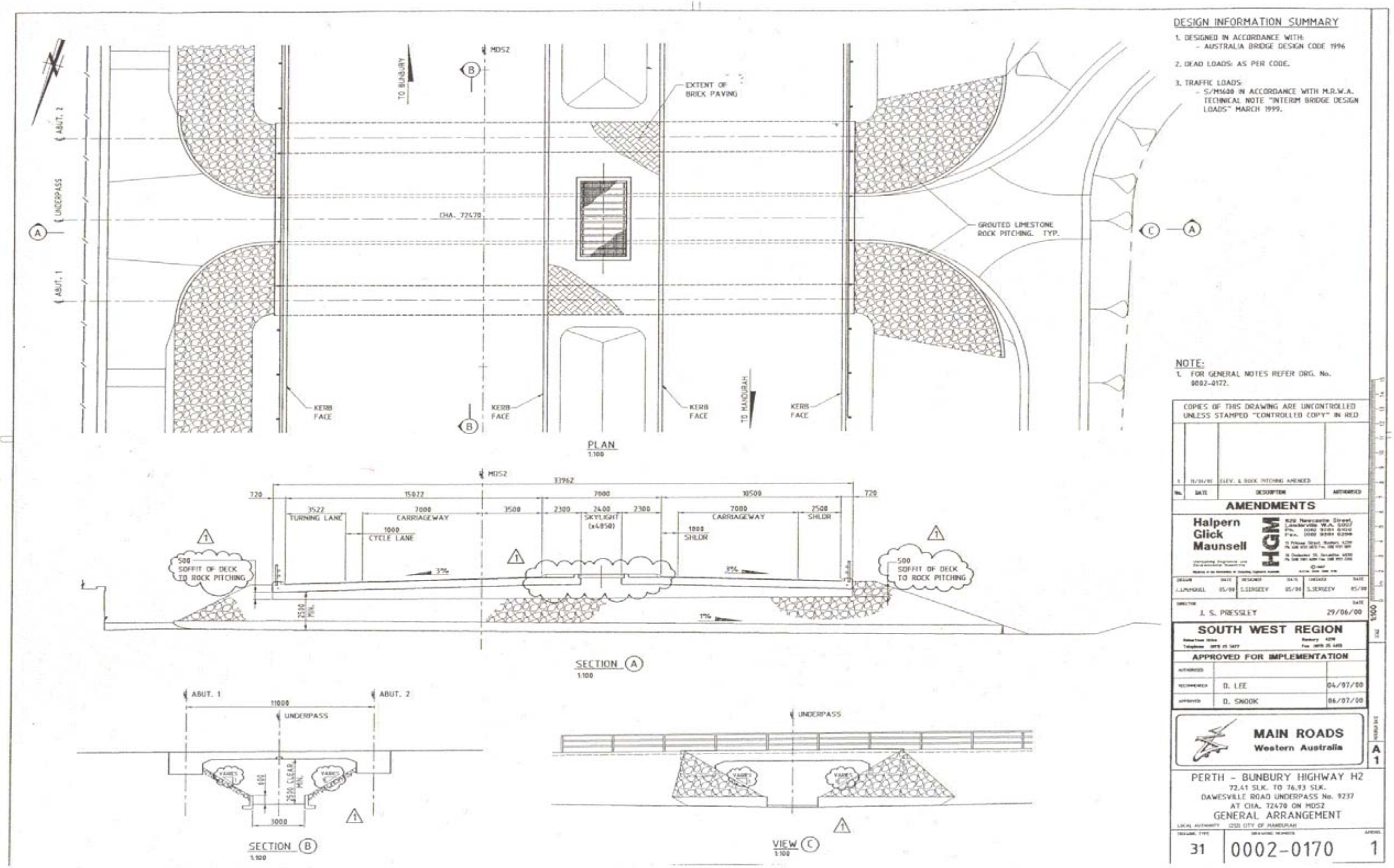


FIGURE 2.16 - PEDESTRIAN UNDERPASS

CHAPTER 3

PRINCIPLES OF DESIGN

CONTENTS

3.1 INTRODUCTION

The design process discussed in this Manual is based on AS 5100, Bridge Design (CODE). This Chapter gives a brief outline of the principles and application of limit state design upon which the CODE is based.

3.2 PRINCIPLES OF LIMIT STATE DESIGN

The general principles of limit state design are outlined in Clause 6 Part 1of the CODE. Essentially the aim of design of a structure is that it shall have a very low probability of reaching any of a number of ultimate or serviceability limit states during its design life. Both the applied loadings and the material properties are factored to ensure that this probability level is known and is consistent for different types of structures. Descriptions of the two limit states are:-

- ***Ultimate Limit States*:** are failure and safety related, satisfying these ensures that, during its design life, the structure will have a very low probability of failure, including collapse, through lack of strength or instability.

- ***Serviceability Limit States*:** are performance related and satisfying these ensures that, during its design life, the structure will remain fit for the purpose for which it was originally designed. Meeting these limit states ensures that the structure will not undergo unacceptable permanent deformations or damage and will not show any signs of distress, e.g. cracking or vibration, such as would alarm members of the public.

Obviously the results of attaining an ultimate limit state would be much more serious than reaching a serviceability limit state, and so for a satisfactory and economical design the two should have different probabilities of occurrence. There should be an extremely low probability of reaching an ultimate limit state, but a somewhat higher probability could be permitted for serviceability limit states. This is reflected in the CODE, where the load and material factors are chosen such that a serviceability limit state generally has a recurrence interval of 20 years, i.e. a 5% probability of occurring in any one year, (or a 99% chance in the 100 year design life of a structure), whereas an ultimate limit state has a recurrence interval of 2000 years, i.e. only an 0.05% probability of occurring in any one year, (or a 5% chance in 100 years).

It should be noted that the structure design life of 100 years used in the CODE is derived from the above probabilities. It does not mean that the structure will fall down after 100 years, in fact, with adequate maintenance it should still be perfectly serviceable at that time. It means that within this time period there is a 5% chance of failure (i.e. the occurrence of an ultimate limit state).

It is stressed that the limit state method of design does not imply the general use of non-linear analysis. The basic principles of linear elastic design still apply. The only aspects of plastic design permitted by the CODE are some limited redistribution of bending moments under certain conditions, and a few special cases of design of non-flexural elements.

The design requires calculation of the effects of the design actions or loads on the individual elements of the structure (bending moments, shears, deflections, etc.), and a check of the overall structural stability followed by calculation of the design resistance or strength of the structure or element. The final step is then to ensure that the strength of all elements is greater than the load effect, and the structure is stable.

The effects of the design actions are the bending moments, shear forces, torsions, reactions etc. produced in the various elements of the structure by the loads and deformations resulting from application of the different load cases, with the appropriate load factors on the individual effects. These load factors reflect both the likely accuracy of the loads, (i.e. the probability of the actual load exceeding the specified value), and the effect that they have on the structure. This means that load factors are usually greater than 1, except when increasing the load increases the safety of the structure, when they are less than 1. Load factors are obviously larger for ultimate limit state load cases than for serviceability ones; and for live loads as compared to dead loads.

The design resistance or strength is a measure of the strength of a particular element of the structure in bending, shear, torsion etc. This strength is calculated from the properties of the materials in the element, (e.g. failure stress, Young's Modulus) and its physical dimensions. Because of construction tolerances, variability of materials etc, the properties and dimensions cannot be known exactly for a particular element, only the statistically most likely or "characteristic" value, e.g. the "as-cast" dimension of an element, rather than the specified thickness; the "actual" compressive strength insitu, rather than the specified strength. Therefore, capacity reduction factors (Φ factors) are applied to the various resistances calculated from the theoretical (specified) properties to ensure that the figures used in design will have a very low probability of being more favourable than the actual values in the real structure.

3.3 ANALYSIS OF STRUCTURES FOR DESIGN ACTION EFFECTS S*

The analysis of structures involves the application of the relevant factored load combinations to the structural model to obtain the maximum design action effects (S^*) (bending moments, shears, torsions, reactions etc.), in the various elements of the structure (deck, columns, bearings, piles etc.), for the critical serviceability and ultimate limit state load cases.

In order to do this, a theoretical analytical model (or models) of the real structure is created, the relevant loads and load combinations applied and the design actions calculated.

The various load cases are given in Part 2 of the CODE and discussed in Chapter 4 of this Manual. There are many possible load combinations and it is important to recognise those that will be relevant for the particular element of the structure being considered, both to save unnecessary work and to ensure that the critical load cases are not missed.

The various structural models are discussed in Chapter 10 of this Manual. There are two basic types, or scales, that are used, **global** models and **local** models. As can be inferred from the names:-

- ***Global*** models look at the structure as a whole, or at least at a major part of it, and are used to obtain overall bending moments, reactions, distribution factors etc. They are the main analytical tools used in structural analysis and a number of different global models will normally be used for any one structure.

- ***Local*** models look at small, important or critical portions of the structure, e.g. diaphragms, slabs between beams, prestress anchorage zones, pilecaps etc. They tend to be used in the later stages of design, after the major dimensions have been fixed.

At least two global models will normally be used for any structure. A 2-dimensional line beam model to obtain overall, longitudinal design actions and then a more detailed 3-dimensional model to obtain transverse effects and the transverse distribution of the longitudinal actions. The first will typically be a PCBEAMAN model, with the second being an ACES grillage or finite element model.

Local models are usually finite element, or detailed grillage/space frame models of the particular portion of the structure of concern. In some instances, e.g. transverse bending in beam and slab decks, the global grillage model may be sufficiently accurate for local effects. Also, e.g. local wheel effects, prestress anchorage zones, there are design charts or standard design methods available.

A description of the software programs used by Structures Engineering is contained in the Bridge Branch Design Information Manual, Part 14.

3.4 ANALYSIS OF STRUCTURES FOR DESIGN RESISTANCE ΦR_u

After obtaining the design actions on the various elements of the structure for the critical limit states the next step is to calculate the design resistance ΦR_u or strength of all the elements of the chosen structure. This is covered in detail in Parts 5 and 6 of the CODE, and Chapters 11-15 of this Manual. The theoretical capacities are calculated and then the capacity reduction factor of the material (Φ) for each particular capacity is applied to obtain ΦR_u. Note that the Φ factors are different for the different resistances, bending moment, shear force, torsion etc., reflecting the different levels of accuracy of the strength calculations and also the likely result of attaining the particular limit state. For example, Φ for shear is less than Φ for bending as failure in shear is usually non-ductile, and therefore more sudden and potentially more catastrophic than failure in bending.

3.5 SATISFYING $\Phi R_u \geq S^*$

Having analysed the various parts and elements of the structure for the effects of the (factored) design actions (S^*), and calculated their (factored) design strengths (ΦR_u), the last step is to check that $\Phi R_u \geq S^*$ for all limit states (both ultimate and serviceability) for all elements. This is usually done in conjunction with the calculation of ΦR_u and is an iterative process as the first time through ΦR_u for some parts of the "guesstimated" structure will probably be less than S^* and so the structure will have to be changed and ΦR_u, and perhaps S^*, recalculated. Often however the changes required to increase ΦR_u, (e.g. dimensions or amount of reinforcement or prestress), will only be minor and S^* will not be affected.

The different parts of the structure have to be checked for both strength and serviceability limit states, i.e. that they will not fail under any of the ultimate limit states, and they do not contravene any of the serviceability criteria. Items to be checked will include:-

- ***Superstructure**:* Longitudinal and transverse strength of the various elements in bending, shear and torsion. Permanent and live load deflections.
- ***Substructure**:* Strength and stability of the columns, abutments and retaining walls.
- ***Foundation**:* Strength/stability/settlement of the footings or piled foundations.
- ***Miscellaneous Items**:* Strength and movement capacity of the bearings, expansion joints, guard rails etc.

The design is satisfactory when $\Phi R_u \geq S^*$ for all parts of the structure, for all limit states, and the deflection, vibration and detailing rules are satisfied.

CHAPTER 4

DESIGN LOADS

CONTENTS

4.1 INTRODUCTION

This Chapter is a review of the different loads and load groups that are specified in Part 2 Design Loads of AS 5100, Bridge Design, (CODE). It is only a brief review because the information given in the CODE and accompanying Commentary is fairly comprehensive. These notes do not set out to repeat details given in the CODE, but rather to give further information on each load and explain their application individually and in load groups in the analysis of bridges in Western Australia.

Before examining the individual loads it must be stressed that the Design Engineer should always be prepared to exercise judgement and discretion when dealing with loads and load combinations. This applies especially because:-

- On some unusual structures there may be loads which are not specified in the CODE, and the Engineer must be careful to identify these and include them in the analysis.
- The selection of load combinations must be done with great care. Intelligent choice of combinations is essential to ensure that impossible, extremely unlikely, or simply ridiculous ones are not included, but all possible ones are, so that the worst combination is identified. Also, it should be noted that there will probably be different critical combinations for different elements in the one structure.

It cannot be stressed too much that the correct identification and calculation of the loads and load combinations on a structure is of paramount importance, as it is essential to the subsequent analysis for the actions/load effects (S*). If the loads are calculated incorrectly, or the worst load combination not identified then the rest of the calculations will be in error and the resulting structure possibly unsafe.

The Clause numbers referenced are those in AS 5100.2, Bridge Design, Design Loads.

4.2 DEAD LOADS - (CLAUSE 5)

The structural dead load is detailed adequately in the CODE, and a range of material densities given in the Commentary. For reinforced and prestressed concrete in Western Australia it is normal to use a figure for density of 26 kN/m^3.

Superimposed dead load on a structure normally includes the bituminous surfacing; the kerbs, parapets, infills and railings; plus any pathways and services carried. The thickness of the asphalt surfacing for design purposes should be taken as a minimum of 100 mm for open graded asphalt or 75 mm for other road surfacings to allow for possible resurfacing. The weight of standard handrail or guardrail can be taken as 1 kN/m per line of rail.

One very important point in dead load analysis is to ensure that the method of erection of the structure is correctly allowed for. The dead load effects (e.g. bending moments and shears) are dependent upon the erection method. Only smaller structures are erected (or cast) as one unit. With other methods of construction the dead load effects will differ considerably. Some typical examples of this are:-

- ***Beam and Slab Construction*** - in either concrete or steel. The beams are usually erected first and initially will be simply supported for their own self weight. Then they may be either simply-supported or continuous for the weight of the wet concrete of the deck slab depending on the method and timing of establishing continuity. Finally, the combined, continuous structure will carry the weight of the superimposed dead load and live loading.

- ***Span-by-Span Construction*** - Typically used for large, prestressed concrete bridges, where one complete span and a short length of the next are built, stressed and stripped out and then the next span added to this with continuity established by coupling the prestressing cables.

There are other examples, in both concrete and steel and the correct structural model must be used in all cases.

If a particular method of erection has been assumed in the calculations and this is important for the distribution of dead load effects, then it MUST be specified on the Drawings, as the use of any other method without reference to the Designer could be dangerous.

4.3 ROAD TRAFFIC AND PEDESTRIAN LOADING - (CLAUSES 6 & 7)

Loading specified is composed of five separate loads, these are:-

- ***M1600/S1600*** vehicular loading. This is the standard traffic loading and the CODE requires that it should be applied in full to all structures. These vehicles are not real vehicles but design vehicles which reproduce the effects of the heaviest normal vehicles likely to use a structure. Occasionally for very minor or special purpose roads or for refurbishment of existing bridges, MRWA may allow design for reduced loading. All such instances MUST be approved individually by the Senior Engineer Structures (SES). This relaxation is used selectively as it may only result in a small cost saving and there is always the possibility of overweight trucks using the structure or the chance of a future change of road function.

- ***HLP 320/400***. The heavy load platform (HLP) is modelled on realistic vehicles presently in use on Australian roads. Use of these vehicles for design must be decided at the commencement with advice from SES.

- ***High Wide Loads***. The requirement of HWL shall be in accordance with Road and Traffic Engineering Branch's "Guide to Design and Operation of High Wide Loads Corridors" Doc. No. 05/9235.

- ***W80/A160***. These individual heavy wheel and individual heavy axle loads are used for the analysis of local effects and the design of certain deck elements.

- ***Pedestrian Loading***. This can be quite high but is considered justified, both because such loads are possible in very crowded situations, and because of the potentially catastrophic effects of failure of a crowded footbridge. Note that this load is for the design of footways and their supporting elements only. If the footway is on a road bridge the CODE requires both the road and the footway to be loaded simultaneously. The 20 kN isolated load will only be important for local effects.

Refer further the Bridge Branch Design Information Manual Document No. 3912/02/05, Design Vehicle Loads.

The other important factor associated with traffic loading is the dynamic load allowance (Clause 6.7). This is to allow for the "impact" effect of vehicles when traversing the bridge. For the HLP a constant figure for the Dynamic Load Allowance of 10% is used because of the slow, controlled speed at which such vehicles usually travel.

Application of traffic loading to a structure requires identification of the design traffic lanes. For an urban structure this should always be the maximum that can be fitted between the parapets (i.e. allowing for the possible conversion of footpaths to road carriageway where practical), to retain maximum flexibility for future use of the structure.

When the lanes are identified the load should be placed as eccentrically as possible across the structure, to get the worst distribution of loading. Note also that because of the effects of distribution and the Accompanying Lane Factors, the maximum load effects are not always when all lanes are loaded. Two or three lanes loaded is often the most severe.

4.4 HORIZONTAL ROAD TRAFFIC FORCES - (CLAUSES 6.8 & 9)

Horizontal forces due to traffic can be in two directions, laterally due to centrifugal effects and longitudinally due to braking.

The former is obviously only present when the bridge is on a curve, and even then the force is usually small and rarely warrants consideration. It is often adequately covered by the minimum lateral restraint requirement of Clause 9.

Braking forces can however be an important loading as the force is quite high. Because of this, careful consideration must be given to how the force is resisted. On structures with sliding bearings the force is resisted by the fixed abutment, apart from the proportion taken out in bearing friction. For structures using elastomeric bearings, or with pier columns directly connected to deck, the resisting force will be distributed between more members. The total force is shared between the piers and abutments in proportion to their stiffness (note: it is important that the total combined stiffness of each member, e.g. bearing plus column is used). It may be found that a pier attracts a high load which would require a massive column to resist. In many cases this may be unrealistic and engineering judgement is required. If the bridge has abutment diaphragms pushing directly against the embankment fill then passive pressure should be included in the longitudinal resistance. However, if there is a back wall to the abutment this will not be possible (without complete closure of the expansion joint) and so the columns, or fixed abutment, must be designed to take the total force.

The forces are unlikely to be a consideration at the ultimate limit state, as the embankment fill will prevent excessive movement of the structure longitudinally, as long as it is not washed out.

The minimum lateral restraint capacity specified, as 500 kN or 5% of superstructure dead load, whichever is greater, as an ULTIMATE load at EACH support, is included to prevent an unexpected force or movement (e.g. a minor earthquake or a collision), causing damage out of all proportion to the load e.g. knocking the structure off its supports.

4.5 COLLISION LOADS - (CLAUSE 10)

This Clause deals with the possibility of road traffic or trains hitting bridge supports. The results of this could be catastrophic if the structure came down on top of the errant vehicle or train and so the possibility of it happening should be considered carefully.

- ***Road Traffic*** - MRWA has a policy that for road over road bridges the supports should be a specified minimum distance from the edge of the pavement, otherwise a protective barrier must be erected. Refer further to the Bridge Branch Design Information Manual, Document No. 3912/02/02, Design of New Structures.

  All columns in the median and adjacent to shoulders must be designed to resist traffic impact forces, regardless of whether they are protected by safety barriers or not.

- ***Railways*** **-** Protecting bridge supports is even more important in the case of bridges over railways, because of the much higher impact forces involved. It is MRWA's policy wherever possible to cross over railways in a single span with solid wall abutments, reinforced concrete or reinforced earth, to avoid the issue of potential impact on piers. Refer further to the Bridge Branch Design Information Manual, Document No. 3912/02/02, Design of New Structures.

- ***Ships*** **-** Ship impact is no longer expressly covered in the CODE. It can, however, be critical for some large structures with ship impact an order of magnitude above that of traffic and trains, because of the mass of the moving body involved. There are few places in WA where this loading has to be considered. If it is required then specialist literature should be consulted. (Ship Collision Analysis by H. Gluver and D. Olsen contains proceedings of the interactions symposium on advances in ship collision analysis.) Not only the piers of the designated navigation span(s) may require protection. If the water is deep enough for navigation then there is the possibility of impact to any pier.

4.6 KERB AND BARRIER DESIGN LOADS - (CLAUSE 11)

The different barrier classifications are defined in Clause 10 of AS 5100.1. There are a number of standard barrier types used by MRWA that are deemed to comply with the respective CODE performance levels. These should be used where possible.

With all railings it is important that they are detailed such that in the event of an accident there are no lengths of rail that can break free and become potentially dangerous "spears" that could impale both vehicles and occupants.

4.7 WIND LOADS - (CLAUSES 16, 23 & 24)

The calculation of wind loads for normal structures is fairly straightforward. Once the design wind speed(s) are known all that is required are the exposed area of the structure and the drag coefficient, both of which can be calculated by following the CODE. The design wind speeds for the bridge location, at serviceability and ultimate, are obtained from AS/NZS 1170.2.

When calculating this loading note the following :

- Coincident wind and live loading need only be considered as a serviceability load and then the wind speed is limited to 35 m/s, with no requirement for wind on live load.

- Longitudinal and vertical wind loads are specified, although for most structures they are unlikely to be of concern.

- The details given in the CODE are for standard structures only, "unusual" ones, e.g. suspension and cable stayed bridges, will require special investigation.

When analysing the effect of the wind load on the bridge substructure care should be taken in assessing the amount of load which goes to each support, as this will depend upon their relative stiffnesses. The centre of effort of the wind loading will normally be above the bearing level so this must be allowed for in the analysis.

Wind loads on traffic signs, traffic signals and lighting structures are calculated in a similar manner to above, however lower design wind speeds are used because of the lesser consequences of failure.

The preferred failure mechanism is a gentle bending or yielding of the support column. It is essential that the sign face stays on its supporting beams and that the beams stay attached to the columns, or the sign becomes a dangerous missile.

For overhead sign gantries, these should not collapse in the ultimate design wind, as the gantry would block the road, which is undesirable in a post-disaster situation. Similarly the signs and their connections to the gantry must be strong enough to resist the ultimate wind load, or they will contribute to the airborne debris. In the ultimate case, the sign face may well be destroyed, but should not come off in large pieces to fly long distances. Refer further Clause 23 of AS 5100.2.

Wind loads on noise barriers are also treated differently as outlined in Clause 24 of AS 5100.2.

4.8 THERMAL EFFECTS - (CLAUSE 17)

Temperature change can affect a bridge in two ways, dimensional change caused by overall temperature variation and induced stresses and moments caused by temperature gradients within the structure.

4.8.1 Temperature Variation

On an unrestrained structure overall temperature change will cause expansion and contraction, and these movements must be allowed for when designing the bearings and expansion joints. On a restrained structure the movements will induce forces and moments. Ensure that ample allowance is made for dimension changes (including transversely on wide bridges) as these movements are easy to cater for in the original design, but very difficult to change in the finished structure.

For overall temperature change, Perth's shade temperature range should be taken as 44^{o}C to -1^{o}C. The calculated average bridge temperatures are long term temperatures for the finished structure and the use of different values should be considered if temperature loading is a significant factor during construction. Usually a lower temperature range would be used because of the reduced probability of getting the temperature extremes during the construction period, but a higher range could be justified in the case of steel structures exposed to the full intensity of the sun with no insulating layer of concrete or asphalt. In fact, temperature movements in steel bridges during erection can be significant and should be checked carefully, especially for curved bridges.

4.8.2 Temperature Gradient

Differential temperature effects constitute an important load case and the magnitude of the forces induced can be on a par with live load. When a temperature gradient develops across a section a potential strain is induced which matches the shape of the temperature gradient. However the actual strain that occurs will depend on the section's boundary conditions, and at any level the difference between the potential thermal strain and the strain that actually occurs is a strain that can be converted to a thermal stress. These thermal stresses are self-equilibrating and are termed the primary stresses (see Clark, LA 1983 *Concrete Bridge Design to BS 5400* for methods to calculate the primary stresses). If the thermal strain or curvature is prevented from occurring by the presence of external restraints, for example in continuous structures, secondary or parasitic effects also occur in addition to the primary stresses. Design temperature gradients are given in the CODE for both hot top (producing positive or sagging moments) and cold top (producing negative or hogging moments) cases, and both must be checked.

For certain bridge types (reinforced and prestressed concrete) the CODE permits a reduction in the calculated secondary effects. This is to allow for the effect of the reduction in stiffness when the concrete cracks, as the bridge is modelled using the uncracked gross section properties (see AS 5100.2 Commentary Clause C17.4).

Although the vertical temperature differential is by far the most important one it is also possible on a wide structure to get a transverse temperature differential and the effect of this may have to be assessed, probably more because of the movements it may cause than because of the forces induced. In addition, for wide structures the effect of a vertical temperature differential may be important for the transverse spans of the support diaphragms and can lead to redistribution of the reactions and possible overloading of the bearings.

4.9 FORCES RESULTING FROM WATER FLOW - (CLAUSE 15)

Forces on both substructures and superstructures due to water flow are an important load case for structures over rivers. In the North-West of the state, (North of the 26th parallel), which experiences cyclonic rainfall, bridges are generally designed for overtopping and this can be the most critical load case.

The design stream velocities for both serviceability and ultimate limit states will come from the waterway calculations. It is then a relatively simple matter of applying the formulae given in the CODE to obtain the forces on the substructure and superstructure. It is important to check for both the no scour and maximum scour cases, and stages in between. It is also necessary to check the angle of stream flow to the structure, as it may change with increasing depth of flow and will influence the orientation of the substructure.

It should be noted that the CODE has altered the previous relationship between the Lift Coefficient C_L and the angle of incidence of the flow θ_W. The change is one which greatly increases the lift force for flows of very low angle of incidence. Enquiries were not able to illicit an explanation for this change and it may be overly and unnecessarily conservative.

Where beam and slab structures are used and are designed to be overtopped, air release holes must be used in the deck between the beams to reduce flotation forces. Also, positive tie-downs and shear keys should be detailed at the supports.

Debris loading can be important at various stages in a flood and the critical load case is often stream force plus debris just before overtopping. However, because of the variety of vegetation in the State consideration should be given to the make-up of the debris mat and thus the magnitude of the design force

Stream forces calculated from the CODE will be distributed between the various supports depending on their relative stiffnesses and the stiffness of the deck.

4.10 EARTH PRESSURE - (CLAUSE 13)

Surcharge loading must be applied where traffic can pass close to a retaining wall. There are also possible dangers from the high loads generated during construction if heavy vibratory compaction equipment is allowed close to a wall (Ingold, 1979).

4.11 FORCES FROM BEARINGS - (CLAUSE 20)

The longitudinal force on the substructure is dependent upon the type of bearings used. With elastomeric bearings the force will be derived from the displacement of the deck at the support (caused by temperature, creep, shrinkage etc.) and the shear stiffness of the bearing. For sliding bearings, e.g. pot, spherical or rollers, the forces are governed by the frictional characteristics of the bearings. Coefficients of friction for the different types of bearing are given in Part 4 of the CODE and discussed in Chapter 16 of this Manual.

The principal type of sliding bearings used by MRWA are “pot” bearings. For these, which have a stainless steel/PTFE sliding surface, it is normal to take a maximum coefficient of friction of 0.05. The requirement to check for a seized bearing is not necessary for stainless steel/PTFE sliding surfaces, but it should be considered if steel/steel sliding or roller bearings are used.

4.12 EARTHQUAKE FORCES - (CLAUSE 14)

This Clause is based on the provisions of AS 1170.4 with some bridge specific amendments. At time of writing, AS 1170.4 had been recently updated and amended however the CODE had not yet been amended to maintain the previous consistency of formula and nomenclature. A set of interim earthquake design rules has been issued in Structures Circular SES 1/08 to be used until such time as the CODE is amended.

The Commentary for AS 5100.2 indicates that an Annual Probability of Exceedance of 1/500 (k_p = 1.0) is adequate for earthquake design. However Structures Engineering is maintaining the requirement for ultimate limit state design of a probability of exceedance of 1/2000.

The forces are an ultimate load case so damage is acceptable, although the structure should be usable for light traffic immediately, and for heavy traffic after emergency repairs. All that is usually necessary is to perhaps strengthen the fixed abutment and tie the deck to it. Otherwise the minimum horizontal force requirement covers most cases.

If it is necessary to design a structure for maximum earthquake loading then specialist literature should be consulted.

4.13 SHRINKAGE AND CREEP EFFECTS - (CLAUSE 18.1)

The shrinkage and creep of concrete produces strains in a structure which can have a number of affects, depending on the form of construction:-

- For unrestrained structures they will result in movements which must be allowed for at the piers and abutments.

- For restrained structures, e.g. portal frames, they will result in forces being induced in the members.

- In pretensioned and post-tensioned concrete construction they result in a loss of prestress force, (see Clause 6.4 of AS 5100.5).

- For composite construction (steel or precast concrete beams with an insitu deck slab), the restraint provided by the beams (complete restraint in the case of steel, but only partial in the case of concrete beams, as they will also shrink and creep, but at a different rate) will result in parasitic forces being induced in the structure. This is discussed in Freyermuth (1969).

4.14 PRESTRESS EFFECTS - (CLAUSE 18.2)

The prestressing of a continuous bridge with a profiled prestress cable induces secondary, or parasitic, bending moments in the deck. These arise because the prestress applies an effective upward force to the deck, but this is prevented from deflecting because of the continuity, and so a positive (sagging) moment is induced. It is important that these effects are properly allowed for as they can be large. The moment is always positive and so it tends to relieve the support moments and increase the midspan moments. The Designer can often use this to advantage, as by varying the profile the moments can be adjusted for optimum effect.

The CODE gives load factors of 1.0 at both serviceability and ultimate for parasitic effects, although the Commentary does qualify this somewhat. The problem is that the parasitic moments induced by the prestress are dependent upon the stiffness of the structure, and so if cracking occurs in some areas (as will always happen at ultimate and may happen at serviceability for partially prestressed structures) the moments will change. It is considered that 1.0 is a fair value to use at serviceability, but at ultimate consideration should be given to adopting a conservative approach and using 1.0 where the parasitics are additive and 0.5 where the parasitics are helpful.

4.15 DIFFERENTIAL MOVEMENT OF SUPPORTS - (CLAUSE 19)

Differential settlement of the piers and abutments of a bridge can be an important load case for continuous structures where the displacements will induce secondary, (parasitic) shears and moments in the deck. The magnitude of the induced forces will depend both on the amount of the differential settlement and on the stiffness of the structure, being higher for stiffer structures (i.e. those with deep decks and/or short spans). As any settlement will occur gradually the long term value of Young's Modulus should be used in calculations involving concrete structures.

It is also possible to get rotation in conjunction with settlement at a bridge abutment which could affect the expansion joint. This should be investigated if it is considered that it could be a problem, especially for high abutments.

For bridges on spread footings it is likely that the amount of settlement will be higher than for piled foundations. It is possible to calculate the anticipated settlements but for spread footings on non-cohesive soils it is usual to design for 20 mm differential settlement for piers and 10 mm for abutments, with 5 mm for piled foundations. It is usual to monitor the levels of all spread footings (at least in the Metropolitan area) so that if the settlements become excessive the deck can be jacked back to level and the bearings repacked.

A load factor of 1.0 should be used for differential settlement effects at both the serviceability and ultimate limit states. This is considered to be fairly conservative at ultimate, as cracking of the concrete will tend to reduce stiffness and therefore decrease the induced forces.

4.16 CONSTRUCTION FORCES AND EFFECTS - (CLAUSE 21)

These have been referred to in Section 4.2 above. The only additional comment is that for certain forms of construction, e.g. incremental launching, it may be necessary to check the structure at different stages during construction and in these cases it is also important to allow for construction loadings. The anticipated weight of the forms etc. should be used plus a general allowance of 2.5 kPa for construction live load. If this loading is critical it should be stated on the Drawings to prevent the possible overloading of the structure during construction.

4.17 LOAD COMBINATIONS - (CLAUSE 22)

Once all the individual loads have been calculated they must be assembled into load combinations and the critical load cases identified. The CODE and Commentary are fairly self-explanatory on the method of forming load combinations at both the serviceability and ultimate limit states. Engineering judgement is required in order to identify the critical cases with the minimum of effort.

Remembering that a serviceability load case is one with a recurrence interval of 1 in 20 years then some of the listed combinations can be dismissed immediately as being unlikely this side of Armageddon, e.g. flood + temperature + wind!! Usually for road bridges the critical combination for the superstructure will be:-

PE + LL + 0.7T, or
PE + T + 0.7LL

and for the substructure:-

PE + LL + 0.7W, or
PE + LL + 0.7FL

However these are only general observations and each structure should be treated on its merits. This is an ideal application for a spreadsheet, where all the loads can be listed and the combinations assembled.

At the ultimate limit state there will also probably be different combinations for different parts of the structure. For bridge superstructures the critical case will generally be:-

PE + LL + T

and for the substructure some combination of ultimate flood or wind load with serviceability traffic, if this is possible, (i.e. deck not submerged, or wind not too strong for traffic movement).

4.18 DYNAMIC BEHAVIOUR - (CLAUSE 12)

Dynamic behaviour is a serviceability limit state which has to be checked for both road and footbridges, although it is only likely to be a problem for footbridges.

Road bridges are usually massive enough such that vibrations are unlikely to be a concern, the only possible problem being where there is a footpath on a relatively light bridge. The susceptibility of a bridge deck to vibration is easy to check by the method given in the CODE, although if it does appear to be a possible problem the answer is not easy. The only solution is to change the structure stiffness or mass, or alter the span lengths, all of which are likely to be impractical, although some slight increase in slab thickness may be possible.

With footbridges, again especially light steel ones, vibration can be a serious problem. Fortunately, if analysis indicates that this may be a problem then it is usually possible to find a solution. If the structural parameters cannot be changed then a damping device can be installed. This will change the liveliness of the bridge and can usually be done fairly easily and cheaply. For typical details see the arrangement used on the Mandurah footbridge, Bridge No. 9108 (Drawing No. 8530-0622).

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CHAPTER 5

FOUNDATION INVESTIGATION

CONTENTS

5.1 INTRODUCTION

This Chapter of the Manual provides information on foundation investigation for bridgeworks. It does not however duplicate text book material or information given in the reference documents. It outlines present MRWA procedures in planning and executing foundation investigation works. It should be used as a guide by the designer in determining the type of investigation necessary at a site in order to obtain sufficient data for the cost effective design and safe construction of the bridge foundations.

Soil types and foundation conditions encountered at a site generally will have a significant influence on the selection and cost of the foundation structure and ultimately on the overall structural cost and so should be investigated thoroughly.

For a detailed description of site investigations and test procedures, reference should be made to AS 1289, Methods of Testing Soils for Engineering Purposes. The Bridge Branch Drilling Procedures Manual, although not maintained and updated, contains a great deal of practical information.

Further information on the planning, cost estimation, conduct and reporting of geotechnical investigations by MRWA can be found in MRWA Pavement Engineering Report No. 21, *Guidelines for Geotechnical Investigations of Bridge and Waterway Structures,* by Chowdhury and Sapkota.

5.2 FOUNDATION INVESTIGATION PROCEDURE

A Foundation Investigation should be carried out for all structures, including major culverts. The design engineer shall determine the extent and type of foundation investigation required. It is dependant on the size, importance and complexity of the proposed structure, the foundation conditions at the site and the type of load transfer member being considered to support the structure, i.e. spread footings or piles.

It is usual for foundation investigation to be carried out for MRWA by one of the specialist geotechnical engineering consultants in Perth. The services that they can provide include insitu and laboratory testing, logging of soil and rock samples and the preparation of designs, reports and recommendations. Assistance with writing the geotechnical Brief can be obtained from MRWA Materials Engineering section, who can also provide specialist advice.

In planning a foundation investigation the possible consequences of obtaining insufficient information should always be considered, as a small saving in investigation costs could lead to problems on site, possibly requiring extensive redesign and resulting in delays and costly contractual claims. The extent of the investigation should be commensurate with the level of the economic risk, especially for large structures with difficult ground conditions.

The first stage in planning an investigation is to carry out a desk-top study to examine all available information.

From the information at hand a decision may need to be made whether to seek geological advice concerning the site geology. It is at this stage that a site inspection should be carried out.

It is sometimes advisable to carry out a preliminary foundation investigation for a more complex site or structure. This will assist in determining the foundation conditions and hence in selecting the detailed testing required.

From the desk top study and preliminary foundation investigation a geological model can be created. The possible foundation designs can then be considered and the soil parameters that are required for the design identified.

This information will enable the preparation of a foundation investigation plan detailing locations, depths and types of testing to be carried out. Recommendations for test locations are given in Part 3 of the CODE Clause 6.2, but it is MRWA practice to carry out at least one probe at each pier or abutment position along the centre line of a proposed bridge.

Once an investigation plan has been prepared then the main investigation proceeds. It is important that during this stage the designer continues to monitor investigation results. The objective of the investigation is to determine the underlying geological formations to sufficient accuracy to enable confidence in the design. Changes to the investigation plan will be required should unexpected conditions be encountered. This close liaison will allow the designer to get a "feel" of the soil conditions and develop geotechnical expertise.

It is also important that conditions encountered during construction, e.g. soil types and densities, pile driving records etc are recorded for future reference.

5.3 SITE GEOLOGY

The site geology is usually established during the first stage of the site investigation, which encompasses the desk top study and may include a preliminary field investigation. This stage aims to establish the general distribution of materials on site and their condition. Information may be gathered from:-

- Engineering Geology Maps
- Aerial Photographs
- Previous Site Investigations
- Preliminary Probings

The last two are the most useful for Bridge Site Investigation.

A compilation of this information will indicate the surface geology and may indicate the sub-surface geology. The sub-surface geology will be in terms of the geological unit(s) and their likely thickness and engineering properties. The descriptive terms used for the soil and rock encountered need to be well-defined and consistent so that information passed to the designer has clear engineering meaning. The classifications in Table A1 of AS 1726 should be referred to in this regard. The description of soil follows the Unified Soil Classification System (USCS). The USCS classifies soil according to its particle size distribution, plasticity and amount of organic matter. At present there is not a formalised system for classification of rock. However, AS 1726 provides a classification based on strength, type, defects and weathering.

5.3.1 Engineering Geology Maps

Engineering geology maps are available for most of the State. However, their scale of 1:250 000 make them suitable only for obtaining general information about the site. The maps of most use are the Urban Geology Series and the Environmental Series, each at a scale of 1:50 000. At present, the maps only cover a limited part of the State. They may be obtained from the Department of Land Information (DLI).

The Urban Geology Series provide surface geology and at least one cross-section. The engineering geology is described on each sheet which indicates the likely site conditions and hence foundation type.

The Environmental Series provide a description of the surface soil or rock (i.e. lithology) rather than their origin (i.e. geology). The maps provide a general description of physical properties of materials and comments on suitability for specified uses. The maps also provide many logged boreholes and at least one cross-section. The maps provide sufficient detail to indicate the likely site conditions and hence foundation type.

5.3.2 Aerial Photographs

Aerial photographs are usually readily available, and can be interpreted to provide information valuable in both the design of site investigations and the interpretation of the results. They can often be used, particularly when examined stereoscopically, to identify and delineate specific ground features such as the distribution of soil and rock types, fracture patterns and spacings etc.

The features mentioned above may be important for the interpretation of site conditions. Early identification of major changes in soil and rock types and features that are likely to have a significant influence on the local groundwater regime can be of great assistance in establishing a geological model for the site and in the design of the main site investigation.

5.3.3 Previous Site Investigations

Previous site investigation data gathered at or near the site can be of great use. However, the following should be considered before using the information:

- the value of the data depends on whether the soil and rock descriptions used were those of a consistent, recognised system (such as in AS 1726). If not, then only broad information such as possible soil/rock interfaces can be obtained.
- the results of testing can differ if non-standard practices are employed. A check of the information should be made for this possibility.
- the groundwater level at the time of testing may affect the information obtained.
- site investigation data must always be co-ordinated (using eastings and northings) and related to Australian Height Datum (AHD), particularly so if the site character or surface levels have changed.

5.3.4 Preliminary Probings

A large scale site investigation will probably benefit from a preliminary field investigation. A general understanding of the site geology and groundwater regime will better enable the main field investigation to be planned.

Probings at regular positions across the site will delineate the general soil type and consistency, approximate groundwater surface and probable soil/rock interface. Probing may be carried out using auger probing, see Section 5.4.2, and/or the electric friction cone penetrometer (EFCP), see Section 5.4.4.

5.4 INVESTIGATION METHODS USED BY MRWA

Investigation methods used by MRWA can be categorised into the following five main areas:

- indirect methods
- direct methods
- sampling
- in-situ testing
- laboratory testing

The choice of test method is largely dependent on the location and geology of the site, the type of structure, likely foundation type and the magnitude of the loads to be supported.

5.4.1 Indirect Methods

The designer can usually gather a great deal of information regarding site conditions using indirect methods, see Section 5.3. These are usually carried out during the desk top study phase of an investigation and can result in a more cost effective foundation investigation.

Other indirect methods are the geophysical ones, such as seismic refraction, ground penetrating radar, resistivity / conductivity etc. These are more sophisticated and are only used for bridge foundation investigation to confirm abrupt changes in sub-surface features. These methods are listed in AS 1726 and their applications are described in specialist literature.

5.4.2 Direct Methods

Direct methods of testing allow the examination of the geological structure of a site and enable either disturbed or undisturbed samples to be taken. There are two basic methods:-

- Excavation pits
- Auger probing

(a) *Excavation Pits*

Excavation pits are mainly used where shallow foundations are anticipated. They are usually dug using a backhoe and are typically a maximum of 3 metres deep and can obviously only be used where the water table is below the anticipated foundation level.

They allow the foundation materials to be exposed and examined in-situ. Seepage zones and defects in both soil and rock can be clearly observed and zones of weathering identified. The excavated pit should be photographed for future reference.

High quality insitu samples can be taken with the minimum of disturbance. Samples can also be taken from the sides of the pit, allowing comparison of material properties in the horizontal and vertical directions. This is useful as material anisotropy may have an important bearing on the performance of the structure.

Insitu bearing plate testing can also be carried out at various levels during excavation to assess bearing capacity.

(b) Auger Probing

The various types of augers used are:

Solid Augers - Solid Auger drilling produces a disturbed sample (the soil coming up the auger flights) and is used mainly to determine the subsoil profile. It can give fairly accurate soil identification up to 10 metres deep. It is suitable for confirming the location of a hard layer or profiling a rock surface and is often used in the preliminary investigation stage.

Hollow Augers - Hollow augers are only used for the purpose of casing-off soil strata to enable insitu testing, sampling and diamond drilling to be carried out. They are generally best suited in sands up to 20 metres deep and clays to 35 metres, although they can be used in favourable conditions in sand/clay materials of uniform consistency to depths of 70 metres.

More detailed and useful information is available from the Bridge Branch Drilling Procedures Manual. The manual is available from the DAC library or Structures Engineering CD library.

5.4.3 Sampling

Subsurface sampling provides either a disturbed or undisturbed sample. Disturbed samples are generally obtained from sampling surface materials in an excavation or from material brought to the surface by auger drilling. Undisturbed samples for laboratory testing are generally obtained from cohesive material by tube sampling.

Disturbed samples are used for general observation, inspection and soil classification but usually not for laboratory testing.

Undisturbed samples are used for soil classification tests as well as density and moisture content determination, triaxial shear or unconfined compressive tests and consolidation tests.

Hard clays and rock are usually sampled by using diamond drilling techniques. Core samples can be obtained in lengths of 1.5 or 3 metres. These samples need to be identified by location, hole number, depth and date. Storage and transport needs to be carried out with great care.

Samples are also obtained during SPT testing. These are usually only subject to visual inspection as they are moderately disturbed, the amount of disturbance varying according to the soil type. These samples can also be used to determine the particle size distribution and Atterberg limits.

5.4.4 Insitu Testing

Insitu testing is carried out by introducing a device into the soil or rock mass being examined. The amount of disturbance caused depends on the type of test being performed. The various penetration tests, (SPT and EFCP), can cause great disturbance as they are driven into the soil, whereas the Pressuremeter Test causes very little. The importance of the degree of disturbance depends upon what use will be made of the test results.

(a) Standard Penetration Test (SPT)

The standard penetration test is described in AS 1289.6.3.1 and various texts. It has application in all soils, including highly weathered rock.

SPT values quoted on bore logs typically contain three figures. The first number is the blows required to drive the standard sampler tube 150 mm, the “bedding” length to get past the disturbed zone at the base of the bore, the next two numbers are blows per 150 mm of penetration. The sum of these latter two is the SPT value at that depth.

In sands the test is commonly used to indicate soil density or strength. In clays the test provides an indication of soil strength and type, but undisturbed sampling with laboratory testing is required for precise evaluation of soil properties. Table 5.1 and Figure 5.1 provide correlations between SPT values and undrained shear strength in clays. A correlation between soil modulus and SPT values is given in Figure C3.5(C) of the 1996 Austroads Bridge Design Code Commentary.

Consistency Test	Field Test	N (SPT)	Undrained Shear Strength (kPa)
Very Soft	Extrudes between fingers when squeezed	0-2	0-10
Soft	Easily moulded by finger pressure	2-4	10-25
Firm	Moulded by moderate finger pressure	4-8	25-50
Stiff	Dented by strong thumb pressure	8-15	50-100
Very Stiff	Dented readily by thumbnail	15-30	100-200
Hard	Difficult to dent	>30	>200

Table 5.1 - TYPICAL UNDRAINED SHEAR STRENGTHS FOR COHESIVE SOILS

SPTs are usually taken at 1.5 m intervals for the first 15 m of a bore and 3 m intervals thereafter.

In highly weathered rocks an SPT will generally reach practical refusal within 500 mm of penetration, whereas in soft rock it can penetrate long distances. Nevertheless test results provide a useful guide as to relative strengths e.g.:

- 100 - 300 mm penetration for 50 blows represents very weak rock,
- 30 - 100 mm penetration for 50 blows represents weak rock, and
- Virtual refusal represents medium strong to strong rock.

Generally SPT values are not taken above 130 blows/300 mm in the field for fear of damage to equipment, also this type of material may be easily diamond drilled. A blow count of 80/150 mm is generally taken as an indication of rock.

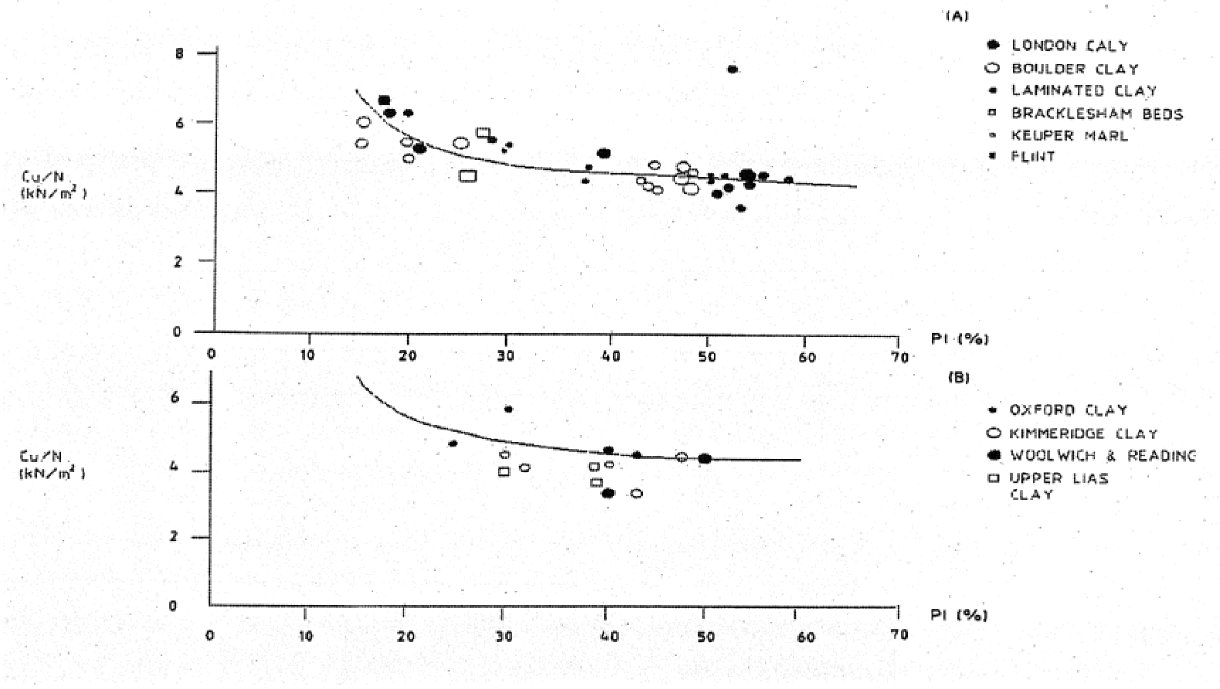

Figure 5.1 - CORRELATION OF SHEAR STRENGTH WITH SPT VALUE (STROUD 1974)

(b) Cone Penetration Test (CPT)

The Cone Penetration Test was used in the past by MRWA. For this a standard cone was attached to an E rod (33.3 mm diameter), inserted into the hollow auger and lowered to the bottom on removal of the SPT equipment and driven into the ground similar to an SPT. It is now no longer used, being replaced by the EFCP test.

(c) Pile Penetration Test

This is another test used by MRWA in the past. It involved driving a flat ended cylindrical B rod of 48.4 mm diameter into the ground using the SPT hammer. The test commenced approximately from ground level and was performed continuously until resistance equivalent to 130 blows/300 mm was obtained. This is a non-standard test and has been replaced by the SPT and EFCP tests.

(d) Electric Friction Cone Penetrometer (EFCP)

The electric friction cone penetrometer (EFCP), is described in AS 1289.6.5.1. The test and its interpretation is described in Ervin (1983) and in Meigh (1987).

This is usually the first choice test for site investigation, as, provided access is available, it can provide a great deal of information easily and cheaply giving both an accurate sub-surface profile at the site and design information.

The test provides a continuous record of the variation of cone resistance and sleeve friction with depth. It is also usual to plot the friction ratio (sleeve friction divided by cone resistance) variation with depth. A typical site plot is attached at Appendix A and a chart for interpretation of these results shown in Figure 5.2. This provides an indication of soil type, soil density, and clay consistency.

The test can also be used to:

- estimate the undrained shear strength of clays, for use in calculation of slope stability or for an estimate of pile shaft friction, and/or
- estimate end bearing resistance and shaft friction for piles in homogeneous sands.

Fleming et al (1994) provides the relationships for the above.

For accurate interpretation the results should be calibrated against laboratory testing of material sampled from the site.

An expression relating undrained shear strength (c_u) to cone resistance (q_c) has been developed for normally consolidated Swan River silty clays:-

$$c_u = q_c/10$$

Modern cone penetration techniques, where a friction sleeve is used immediately behind the cone, enables the skin friction to be estimated directly for full displacement piles i.e:

skin friction = value of friction measured by the friction sleeve

Piles will generally have a somewhat rougher surface texture than the friction sleeve. However, differences due to this will be balanced by scale effects and the presence of rather higher normal stresses on the sleeve just behind the advancing cone, than will be present around the pile shaft.

Cone resistance can be related to SPT blows as given in Figure

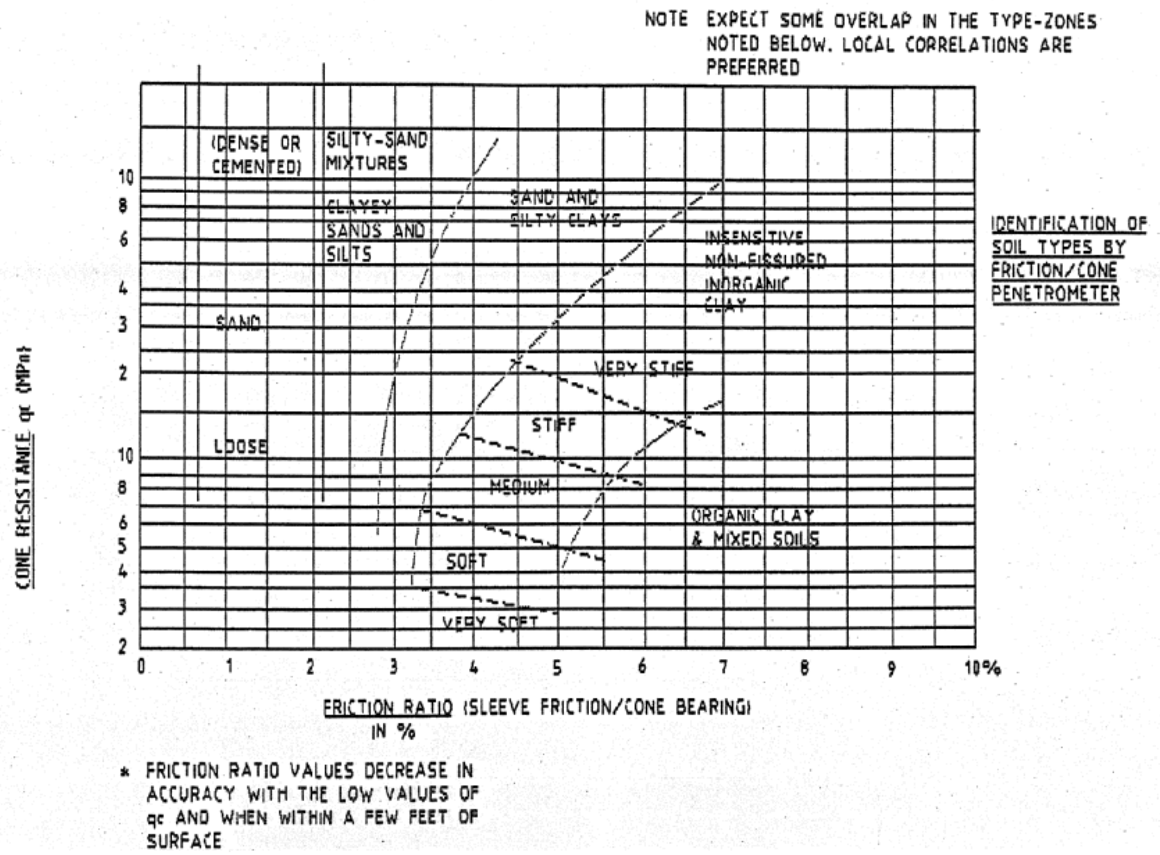

Figure 5.2 - SCHMERMANN'S SOIL CLASSIFICATIONS CHART

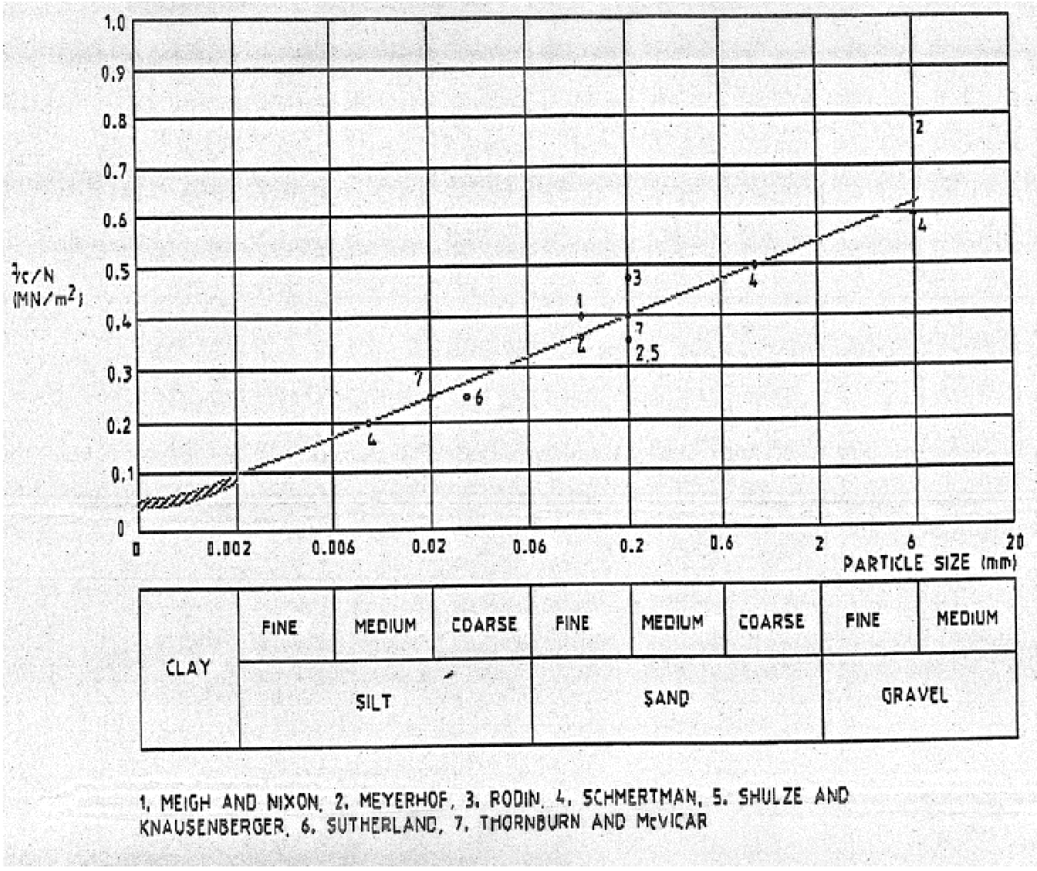

Figure 5.3 - CORRELATION OF RESISTANCE WITH SPT VALUE

(e) Pressuremeter Test

The pressuremeter test is described in Ervin (1983). The test has application in all soils and weak rock. It can be used for the determination of:

- stress-strain behaviour and shear modulus of all materials,
- undrained shear strength of cohesive materials, and
- friction angle.

The pressuremeter normally used is the self-boring type. It consists of an inflatable cylinder that is inserted to the required test depth inside 125 mm ID hollow augers.

The auger is drilled to the desired depth and the pressuremeter is lowered to the bottom from where it drills itself forward into undisturbed material, ready for testing. The cylinder is then expanded and response in the horizontal direction measured.

The test usually provides high quality data. It has particular application for testing material insitu that is difficult to sample. Its big advantage is that it measures soil properties insitu with minimal disturbance, however it is a slow and relatively expensive test.

There is also a non self-boring, high pressure, pressuremeter available for the testing of stronger materials, such as weak rocks.

(f) Screw Plate Test

The screw plate test is described by Kay and Parry (1982). The test has application in stiff clays. It can be used to determine the undrained shear strength and elastic modulus (and hence the bearing capacity).

The test is carried out via 125 mm ID hollow augers drilled to the required depth. The plate is helical, which allows it to be screwed into undisturbed material ready for testing. The screw plate measures response in the vertical direction.

The test is slow and therefore relatively expensive and is rarely used.

(g) Vane Shear Test

MRWA uses the Nilcon vane borer for the determination of the undrained shear strength of cohesive materials. A good description of vane shear strength testing is provided in Ervin (1983).

The Nilcon vane borer by itself can test on the surface or at a shallow depth. If testing is required at depth then a drilling rig must be used to drill to the required depth with 125 mm ID hollow augers.

The Nilcon vane borer is only suitable for testing materials to firm consistency (c_u < 75 kPa). Stiffer materials must be tube sampled for laboratory testing.

5.4.5 Laboratory Testing

Laboratory testing is carried out on material sampled during foundation investigation to determine or verify the soil classification and its engineering properties.

The engineering properties of soils are determined by such factors as parent material, mineralogical composition, organic matter content, age, degree of consolidation, structure and moisture content.

The sampling process involves some disturbance. If the disturbance is expected to be great then insitu testing should be considered.

A sample is generally small in size. The properties of the sample determined in the laboratory are assigned to a much larger soil or rock mass. The validity of this extrapolation depends upon the structure and fabric of the material being tested. In other words, if the material contains discontinuities much smaller than the sample size then the properties determined may well represent the properties of the mass. If however the discontinuities are larger than the sample size then they may dominate the engineering behaviour of the mass. Examples of the latter are joints in rock and silt/sand lenses in soft clay.

The laboratory testing environment allows the conditions of testing (e.g. stress, boundary conditions and orientation) to be varied. The deformation or failure mode can usually be observed during testing. Ervin (1983) provides a good introduction to the above.

Details of the more common laboratory tests carried out on soil samples obtained from foundation investigations are described in Appendix C.

5.5 TYPICAL FOUNDATION INVESTIGATION

A "typical" foundation investigation carried out by MRWA for a bridge site would be composed of:-

- Rural Bridge - Test pits for minor structures, otherwise Auger probes plus SPT testing. This might be augmented with diamond drill coring.

- Normal Urban Bridge - Auger probes plus SPT and/or EFCP with possibly some laboratory testing for clayey soils.

- Large Structure - Auger probes plus SPT and EFCP with laboratory testing and probably pressuremeter and/or screw plate tests. Possibly full scale test pile and pile load testing.

It cannot be overemphasised that any site investigation will only test an extremely small proportion of the material at the site and so considerable variations in soil conditions must be expected and allowed for in design.

5.6 FOUNDATION CONDITIONS AND TYPES

Foundations can be categorised into two types, directly related to the depth of the load transfer member(s) below ground. The two are:-

- Shallow foundations; and

- Deep foundations.

Table 5.2 gives an indication of the type of foundation investigation appropriate for various ground conditions and types of foundation

Condition Expected	Type of Foundation	Test
Shallow Rock * See note 1.	Spread footing. Socketed, cast in place pile.	a) Expose by trenching, or auger bores. b) Drill and core and unconfined compression test if doubt about bearing capacity. c) Assess scour potential induced by proposed structure if soft rock - especially if this is of uncemented nature, e.g. dry, very hard clay.
Soft to firm sediments	Driven or bored, cased pile - may be supported predominantly by end bearing in sand or by skin friction in clay.	a) Preliminary auger bores with visual and immediate tactile inspection. Take note of comments of experienced driller. b) SPT or EFCP. c) Unconfined or triaxial test on undisturbed samples. d) Consolidation test.
Firm to stiff clay	Spread footing. Driven pile (friction and end bearing). Bored pile (friction and end bearing).	a) Preliminary auger bores with immediate tactile inspection. b) SPT or EFCP. c) Unconfined or triaxial test on undisturbed samples.
Cobble	Spread footing if no scour risk. Bored, cased piles (use percussion drilling).	a) Drill with percussion gear. b) If piles are not founded in predictable material below shingle, test pile advisable.
Sand	Spread footing if no scour risk. (Sand may be densified). Driven pile.	a) SPT or EFCP.

Table 5.2 - FOUNDATION INVESTIGATION

NOTES to Table

1. Streams often follow a fault line. Occurrence of rock on one side may not indicate a similar condition on the other side. If comprehensive drilling is not planned a seismic survey should be considered where there is rock evident on one side and not on the other or where there are different types of rock on each side. Some drilling is normally needed to calibrate the seismic interpretation.
2. In all cases it is of great importance to ensure water table levels and dates are recorded as these can have a considerable effect on design and construction, and on possible contract claims.
3. Whenever piled foundations are to be used the possibility of carrying out a pile test in addition to the above tests should be considered.

5.6.1 Shallow Foundations

Spread footings are the most common load transfer member for shallow foundations. They are generally founded a maximum of three metres below ground on sands, firm to hard clays, shingle or rock. For further details see Chapter 6 of this Manual.

At river crossings, footing levels should be set below any anticipated scour level. Weathered rock or hard clay may be quite hard when dry, but could break down when immersed in water, even for a short period of time. This is easy to check and should always be tested. It is important that test results should allow for the worst conditions likely to occur.

Settlement as well as bearing capacity of the soil is a major consideration for spread footings on clays and sands. Adequate field and laboratory tests should be carried out to enable the best estimate of the required design parameters to be made.

A safe bearing capacity correlation with SPT blow counts for clayey soils has been developed by MRWA and is given at Figure 5.4. Reference should be made to Teng (1962) for calculation of safe bearing capacity for non-cohesive soils.

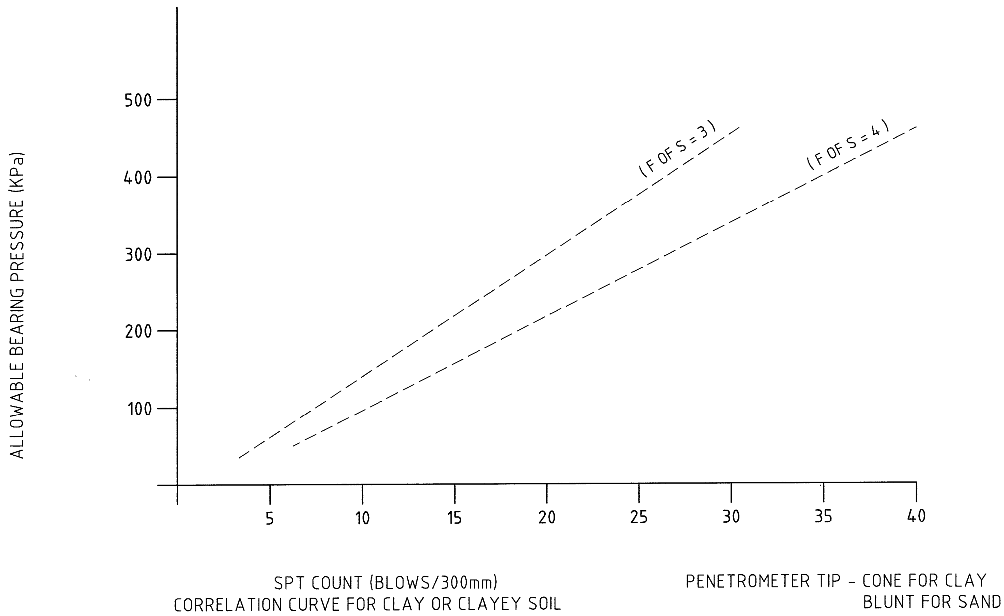

Figure 5.4 - BEARING CAPACITY OF CLAY SOILS

The determination of elastic settlement usually requires the estimation of the elastic modulus of the soil (Es). For sands the relationship of Schultze and Sherif (1973) has been found to be adequate. For clays, the relationship between (Es) and STP blow count is less precise. It is recommended that site specific relationships be developed from laboratory testing or plate load testing.

Soil parameters necessary for the design of shallow foundations can generally be derived from the various field tests with reference to empirical relationships described in various texts.

The depth of drilling for shallow foundations should be down to material which is dense and will not contribute to any settlement. As a general rule drilling should be taken to a depth below anticipated footing level 3x the maximum dimension of the footing, or 10 m, whichever is the greater, or to refusal with the probe augers. However some holes should be taken deeper to ensure weak or compressible material does not underlay the stronger founding strata.

For footings founded on rock it is recommended that core samples of the rock be taken to a depth of three metres to ensure the test hole is not on a "floater" and also to determine the strength properties of the rock. The core can also indicate the character and composition of the various strata with evidence of spacing and tightness of joints, faults and other structural details. Lack of core recovery can give a good indication of the scourability of the material. Where footings are to be founded on rock it is important to establish its surface profile over the extent of the foundations.

5.6.2 Deep Foundations

Deep foundations generally have piles as the load transfer member. These are designed to transfer load from the structure to a satisfactory founding strata at depth. They transfer the load either in end bearing, side friction or a combination of the two. For further details see Chapter 7 of this Manual.

The type of investigation required for pile design and installation are adequately covered in AS 1726 and many textbooks.
(e.g. Pile Foundation Analysis and Design by Poulos HG & Davies EH)

Where piles are to be founded on or in rock, extensive testing of overlying strata is generally not warranted, except where large displacement piles are used. Here it is important to know the material properties of the strata to determine whether the pile can be driven through these layers. If not, jetting, predrilling or blasting may be required.

As a rough guide to the depth of investigation required, driven concrete piles will generally not penetrate hard layers more than 1 m in thickness having SPT blow counts greater than about 60 and will reach refusal in granular materials rated as slow penetration for a probe auger with a rock bit or very slow penetration with a spade bit.

When founding in weak founding rock with steel H piles, deriving their resistance from skin friction, the depth of investigation should be 20 metres of diamond drilling to determine the strength of the underlying rock and the depth the pile is required to penetrate. As a rule of thumb, steel H piles will drive into weak rock having SPT blow counts up to 120 and stop at the point of refusal for a probe auger with a rock bit.

REFERENCES

Australian Standard 1726: Geotechnical Site Investigations.

Australian Standard 1289: Methods of Testing Soils for Engineering Purposes.

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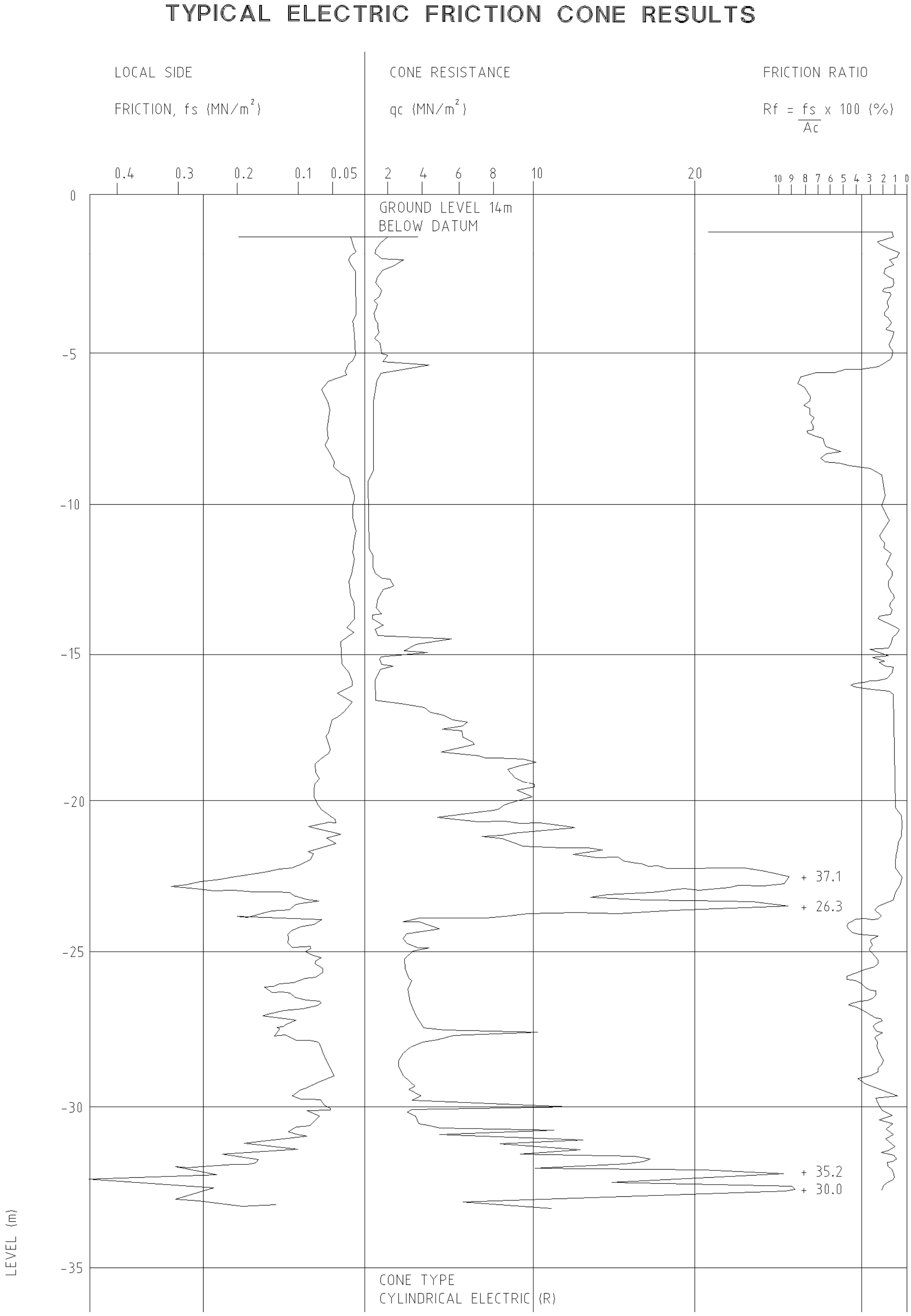
APPENDICES

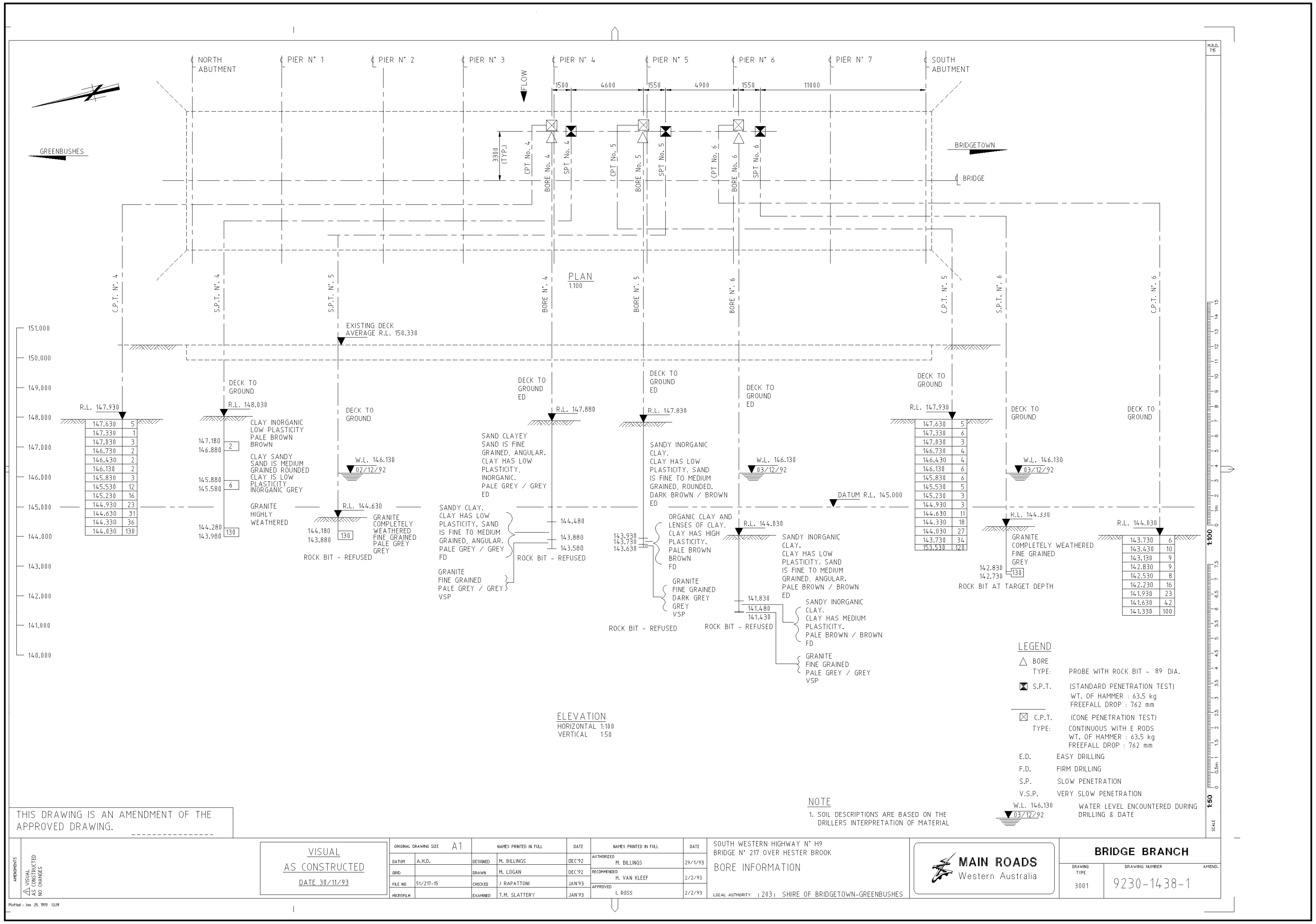
A TYPICAL ELECTRIC FRICTION CONE RESULTS

B TYPICAL BORE LOGS AND BORE INFORMATION DRAWINGS

C TYPICAL LABORATORY TESTS

APPENDIX A - TYPICAL ELECTRIC FRICTION CONE RESULTS




NORTH ABUTMENT
PIER N° 1
PIER N° 2
PIER N° 3
PIER N° 4
PIER N° 5
PIER N° 6
PIER N° 7
SOUTH ABUTMENT
FLOW
GREENBUSHES
BRIDGETOWN
BRIDGE
PLAN
1:100
EXISTING DECK AVERAGE R.L. 150.330
DATUM R.L. 145.000
ROCK BIT - REFUSED
ROCK BIT AT TARGET DEPTH
ELEVATION
HORIZONTAL 1:100
VERTICAL 1:50
LEGEND
BORE TYPE: PROBE WITH ROCK BIT - 89 DIA.
S.P.T. (STANDARD PENETRATION TEST) WT. OF HAMMER : 63.5 kg FREEFALL DROP : 762 mm
C.P.T. (CONE PENETRATION TEST) TYPE: CONTINUOUS WITH E RODS WT. OF HAMMER : 63.5 kg FREEFALL DROP : 762 mm
E.D. EASY DRILLING
F.D. FIRM DRILLING
S.P. SLOW PENETRATION
V.S.P. VERY SLOW PENETRATION
W.L. 146.130 03/12/92 WATER LEVEL ENCOUNTERED DURING DRILLING & DATE
NOTE
1. SOIL DESCRIPTIONS ARE BASED ON THE DRILLERS INTERPRETATION OF MATERIAL
THIS DRAWING IS AN AMENDMENT OF THE APPROVED DRAWING.
VISUAL
AS CONSTRUCTED
DATE 30/11/93
SOUTH WESTERN HIGHWAY N° H9
BRIDGE N° 217 OVER HESTER BROOK
BORE INFORMATION
SHIRE OF BRIDGETOWN-GREENBUSHES
MAIN ROADS Western Australia
BRIDGE BRANCH
3001
9230-1438-1

TYPICAL BORE LOGS

Golder Associates

REPORT OF BOREHOLE: BH5

CLIENT: MRWA	BOREHOLE LOCATION: Refer Site Plan	SHEET: 1 OF 3	
PROJECT: Bridge 631A	SURFACE RL: m DATUM: AHD	DRILL RIG: GDR 650	
LOCATION: Goomalling - Toodyay Road	INCLINATION: -90°	LOGGED: DBM	DATE: 28-12-99
JOB NO: 99640332		CHECKED: GD	DATE: 17/02/00

Drilling					Sampling		Field Material Description					
METHOD	PENETRATION RESISTANCE	WATER	DEPTH (metres)	DEPTH RL	SAMPLE OR FIELD TEST	RECOVERED	GRAPHIC LOG	USC Symbol	SOIL / ROCK MATERIAL DESCRIPTION	MOISTURE	CONSISTENCY DENSITY	STRUCTURE AND ADDITIONAL OBSERVATIONS
Rotary Mud			0					SP	SAND Silty grey white, fine to medium grain size minor fine to medium gravel			
			1	1.20				SC	Clayey SAND Light brown, medium to coarse with some fine gravel, low plasticity fines, medium dense			
					SPT #1 1.50 - 1.95 m N= 28 Rec= 300 mm							
			2	2.50					Loose			
			3		SPT # 2 3.00 - 3.45 m N= 5 Rec= 450 mm							
			4	4.20					Dark grey, medium to coarse			
			5		SPT #3 4.50 - 4.95 m N= 1 Rec= 450 mm							
			6		SPT #4 6.00 - 6.45 m N= 9 Rec= 250							
			7	7.20					Light brown yellow			
			8		SPT #5 7.50 - 7.95 m N= 9 Rec= 250							
			9	8.80				GP	GRAVEL Well cemented medium grained size gravel with sand lenses			
					SPT #6 9.00 - 9.45 m 6/20, 20 for 75 mm Rec= 150 mm							
			10	9.80				MH				

Report of borehole must be read in conjunction with accompanying notes and abbreviations

FULL PAGE BH1-8.GPJ GLDR_AUS.GDT 17-2-00

GAP Form No. 9

REPORT OF BOREHOLE: BH5

Golder Associates

CLIENT:	MRWA	BOREHOLE LOCATION: Refer Site Plan		SHEET: 2 OF 3	
PROJECT:	Bridge 631A	SURFACE RL: m	DATUM: AHD	DRILL RIG: GDR 650	
LOCATION:	Goomalling - Toodyay Road	INCLINATION: -90°		LOGGED: DBM	DATE: 28-12-99
JOB NO:	99640332			CHECKED: GD	DATE: 17/02/00

Drilling					Sampling		Field Material Description					
METHOD	PENETRATION RESISTANCE	WATER	DEPTH (metres)	DEPTH RL	SAMPLE OR FIELD TEST	RECOVERED	GRAPHIC LOG	USC Symbol	SOIL / ROCK MATERIAL DESCRIPTION	MOISTURE	CONSISTENCY DENSITY	STRUCTURE AND ADDITIONAL OBSERVATIONS
Rotary Mud			10		SPT #7 10.50 - 10.95 m N= 10 Rec= 350			MH	Sandy SILT Green brown, very stiff to hard, high plasticity fines			
			11	11.50	SPT #8 12.00 - 12.45 m N= 20 Rec= 250				Silty CLAY Grey white, with some sand and gravel, very stiff to hard, extremely weathered granite type rock			
			12, 13	13.10	SPT #9 13.50 - 13.95 m N= 22 Rec= 450				Yellow brown, very stiff			
			14	14.60	SPT #10 15.00 - 15.45 m N= 25 Rec= 420			SM MH	Silty SAND/Sandy SILT Yellow brown, high plasticity fines, extremely weathered dolerite type rock, dense to very dense			
			15, 16, 17		SPT # 11 16.50 - 16.95 m N= 47 Rec= 450							
			17	17.60	SPT #12 18.00 - 18.45 m N= 45 Rec= 450				Into distinctly weathered rock			
			18, 19, 20		SPT #13 19.50 - 19.95 N= 54 Rec= 390			SM				

Report of borehole must be read in conjunction with accompanying notes and abbreviations

GAP Form No. 9
RL 5.4

Golder Associates

REPORT OF BOREHOLE: BH5

CLIENT: MRWA	BOREHOLE LOCATION: Refer Site Plan	SHEET: 3 OF 3	
PROJECT: Bridge 631A	SURFACE RL: m DATUM: AHD	DRILL RIG: GDR 650	
LOCATION: Goomalling - Toodyay Road	INCLINATION: -90°	LOGGED: DBM	DATE: 28-12-99
JOB NO: 99640332		CHECKED: G.D.	DATE: 17/02/00

Drilling					Sampling		Field Material Description					
METHOD	PENETRATION RESISTANCE	WATER	DEPTH (metres)	DEPTH RL	SAMPLE OR FIELD TEST	RECOVERED	GRAPHIC LOG	USC Symbol	SOIL / ROCK MATERIAL DESCRIPTION	MOISTURE	CONSISTENCY DENSITY	STRUCTURE AND ADDITIONAL OBSERVATIONS
Rotary Mud			20						Into distinctly weathered rock			
			21	21.45	SPT #14 21.00 - 21.45 m N= 81 Rec= 450							
									END OF BOREHOLE @ 21.45 m TARGET DEPTH GROUNDWATER NOT OBSERVED			

Report of borehole must be read in conjunction with accompanying notes and abbreviations

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GAP Form No. 9
RL 5.4

APPENDIX C – TYPICAL LABORATORY TESTS

1. INSITU MOISTURE CONTENT

The insitu moisture content is a measure of the mass of water which can be removed from a sample by drying to constant mass. It is expressed as a percentage of dry mass and is determined by WA 110.1 or WA 110.2.

2. CLASSIFICATION TESTS

MRWA has adopted the Unified Soil Classification System and classification is based on particle size distribution and plasticity properties. The system is described in Appendix A of AS 1726.

Particle Size Distribution - Particle size distribution defines the proportions of component particles of each size constituting a soil. The distribution in a soil sample is determined by WA 115.1 and is expressed in terms of the percentage mass of sample smaller than each size.

Liquid Limit and Plastic Limit - The liquid and plastic limits are useful as an aid to soil classification. The tests are performed on fully remoulded samples with the liquid limit being determined by WA 120.1 or WA 120.2 and plastic limit by WA 121.1. They are extremely useful as indicators of the variability of conditions over a large site. A cohesive soil with an in-situ moisture content of the same order as its liquid limit will, in general, be a very soft material, while a cohesive soil with an in-situ moisture content of the same order as its plastic limit will, in general, be a stiff material.

Plasticity Index - The plasticity index indicates the magnitude of moisture content range over which the soil remains plastic. It is determined by WA 122.1 and is the range lying between the liquid and plastic limits. A low value of plasticity index indicates a low cohesion.

3. PERMEABILITY

A knowledge of permeability is required in considering problems such as seepage and groundwater lowering. The permeability of cohesive soils is, in general, very small but can be affected by sand or silt lenses and there is a need to note the presence of any lenses during investigation. Permeability can be determined in-situ (WA 335.1) or in the laboratory (AS 1289.6.7.1).

Permeability may vary in orthogonal directions (up to two orders of magnitude) and orientation of the sample during testing can be important. Hazen's equation (Lambe and Whitman, 1969) can be used to estimate the permeability of sands.

4. STRENGTH TESTS

Where tests are undertaken on samples taken from the field, correct sample handling is imperative. Change in moisture content will result in a change in strength and sealing of the sample must be carried out immediately after sampling. Sample numbers and condition should be noted prior to testing.

Triaxial Testing - The laboratory measurement of shear strength under controlled conditions of drainage is largely dependent on the triaxial test. The test may be performed in various ways and the testing undertaken should relate to site conditions. A comprehensive survey of techniques used in the triaxial test has been prepared by Bishop and Henkel (1962). The two most common tests used are the Unconfined Compressive Test and Consolidated Undrained Test.

Unconfined Compressive Test - This test is carried out on diamond drill samples and can only be employed for cohesive soils. It is a quick test that determines shear strength (c_u). The test method is indicated in BS 1377 (Test 20) and the shear strength is taken as one half of the stress at failure.

Undrained Consolidation - This test is used to determine the parameters of c' and φ' (effective cohesion and friction angle). Pore water pressures are measured during the test (WA 151.1).

Direct Shear Test - The strength parameters of cohesion and friction angle can be obtained from this test (WA 144.1). It is typically used for testing sands. Clays can be tested, however test conditions are vague (pore water pressures cannot be measured and there is complete sample disturbance) hence, results are only indicative. The friction angle of sands and silts can be estimated from their grading as indicated in Table 11.3 of Lambe and Whitman, 1969.

Point Load Index Test - This test is used for the determination of the unconfined compressive strength of rock. It is a quick test and the rock strength is determined through correlation. The test method used for this test is International Society for Rock Mechanics.

5. CONSOLIDATION

The magnitude and time rate of consolidation settlement are determined from odometer tests (WA 150.1). Although the magnitude can usually be reliably predicted, the rate depends on the length of drainage paths which in turn lies in identifying drainage layers in the field.

CHAPTER 6

SHALLOW FOOTINGS

CONTENTS

6.1 SCOPE

The purpose of this Chapter is to provide a basic introduction to the design for shallow footings[1]. Typically, for in-house design of structures, the allowable design parameters are determined by specialist Geotechnical companies in the course of their site investigations.

Shallow footings are provided for in Australian Standard AS 5100 - Bridge Design, Part 3 in particular Clause 10, Shallow Footings. The reference 'the CODE' has been used throughout and refers to AS 5100 and specifically Part 3, Foundations and Soil-Supporting Structures, unless otherwise noted.

6.2 DEFINITION

Shallow footings comprise all types of spread footings such as pads, strips and rafts for structures, including retaining walls. The CODE defines a shallow footing as "one that is founded at shallow depth and where the contribution of the strength of the ground above the footing level does not influence the bearing resistance significantly." As a general rule, a shallow footing will be founded less than 3 m below the natural ground surface. However in riverbeds with consideration of future potential scour, the depth may extend to 5 or 6 m below bed level.

6.3 LIMIT STATES

The design of shallow footings must satisfy both ultimate and serviceability limit states. Loads to be taken into account are given in Section 8.2 of the CODE, including those specified in AS 5100.2 and loads induced by soil movement, construction effects and water.

Both limit states are related to the interaction between the structure as a whole and the supporting ground. In practice, experience will often show which limit state(s) will govern the design and once this is established, then analysis of most of the other limit states may be limited to approximate control checks.

The limit state concept has not changed the basic methods of calculating ultimate bearing resistance, settlement, earth pressures and many other manifestations of soil/structure interaction; it has merely altered the format of the equations by introducing material factors, reduction factors, load factors and geometrical parameters which in combination replace the old working stress approach using safety factors.

Characteristic geotechnical design values should be based on a careful assessment of the range of values which might be encountered in the field. The selection of characteristic value for a particular project will depend on the specific requirements and constraints of the project as referred to in Section 7.3.4 of the CODE. Further comments are outlined below:

- Experience indicates that a moderately conservative approach is to set the characteristic value of soil stiffness and strength such that 25% of the measured values are less than the selected characteristic value.
- Careful consideration must be given to the potential influence and presence of zones or strata of lower strength within the soil or rock profile.
- Selection of a conservative design value may involve a value either greater than or less than the mean value, depending on the effect of the parameter on the design. For example, soil density may be included in the dead load applied to a structure.

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[1] A distinction between a "footing" which rests on a soil "foundation" is made in this Chapter.

6.3.1 Ultimate Limit State

a) A limit state at which a mechanism is formed in the ground, for example, a slip circle type of failure (geotechnical strength).

b) A loss of static equilibrium or rupture of a critical section of the structure (structural strength) due to:
   i) rupture through the footing caused by the applied structural loads; or
   ii) movements in the ground causing structural damage that can render the structure non-functional and require it to be closed for major repairs or reconstruction.

The ultimate geotechnical resistance of a footing (R_{ug}) can be estimated from the soil properties and footing geometry. The soil properties to be used are characteristic values[2] in accordance with CODE requirements, refer to Section 7.3.4 of the CODE. These are generally moderately conservative values using local knowledge and experience to interpret results of laboratory or field tests. Well known bearing capacity equations for determination of bearing capacity are used to calculate R_{ug} e.g. Bowles (1996) and Lambe and Whitman (1969).

The relationship $\phi_g R_{ug} \geq S^*$ must be satisfied, where $\phi_g R_{ug}$ is the generalised design geotechnical resistance or strength and S^* is the generalised design action. ϕ_g is the geotechnical strength reduction factor. Recommended ranges for the value of ϕ_g are given in Table 10.3.3(A) of the CODE. The range of values for ϕ_g is dependent on the extent of the site investigation and other factors. A guide for assessment of ϕ_g is provided in Table 10.3.3(B) of the CODE.

6.3.2 Serviceability Limit State

A state when the loss of serviceability is caused by excessive deformation. The structure fails to perform to the required standard of utility, appearance and public comfort etc., e.g. failure at expansion joints, rocking, vibrations and unsightly deformation or deflection (including possible cracking in concrete structures).

The serviceability limit state must be chosen with consideration to the effect on the structure being designed. The settlement of shallow footings at serviceability limit state is typically limited to about 25 mm.

[2] In general, the characteristic value of a geotechnical parameter should be a conservatively assessed value of that parameter. Engineering judgment needs to be exercised in making such an assessment, with geotechnical engineering advice being obtained as required.

6.4 DESIGN OF SHALLOW FOOTINGS

6.4.1 Footing Depth and Proportions

6.4.1.1 Founding Level

The following factors should be considered in determining the founding level, i.e. level of underside of footing:

a) The founding level is on a stratum with an adequate geotechnical resistance;

b) For structures over water or in seasonally active watercourses, the anticipated depth of scour must not undermine the foundation or reduce embedment beyond limits required for bearing capacity or sliding;

c) In clayey soils, the foundation level is located in a zone that is free from seasonal moisture changes which may cause shrinkage and swelling of the soil. The zone should also be free from any disturbance caused by roots of trees and shrubs for both clay and sand soils;

d) The founding level is below the level of any future nearby excavations or trenching for utility services;

e) Anticipated ground movements especially in sloping ground, will be within allowable limits;

f) The seasonal high groundwater level's impact on geotechnical resistance; and

g) Construction considerations, e.g. temporary works and dewatering requirements for deep versus shallow excavation.

6.4.1.2 Size of Footing

The width and length of the footing is controlled by the following factors:

a) The footing pressure must be within allowable limits;

b) The size of the load supporting members e.g. columns and walls, which have to be accommodated will fit on the footing;

c) Practical and economic considerations for excavation, and minimum working space requirements; and

d) Clearances required to adjacent roadways, services etc., especially during construction.

The overall size of a footing is dependent on the design loadings, both vertical and inclined, the point of action (eccentricity), the ultimate geotechnical design resistance of the soil at the founding level (R_{ug}) and total and differential settlement limits.

Generally the vertical load will be applied at the centroid of the footing. However, bridge footings subject to large overturning moments from flood loading may be designed such that the point of action from the pier acts off centre from the footing towards the upstream side. The purpose of this is to create a more even load distribution across the footing base under flood load.

At the ultimate limit state, the toe pressure can reach the ultimate geotechnical resistance of the soil and there can be uplift at the heel, but at the serviceability limit state it is recognised good practice that the vertical load resultant should lie within the middle third with no resultant uplift at the heel.

6.4.2 Ultimate Geotechnical Strength (R_{ug}) of Shallow Footings

Geotechnical strength requirements that must be addressed are specified in Section 10.3.3 of the CODE. The modes of failure to be considered are:

- ultimate bearing failure;
- overall (global) instability; and
- sliding failure.

6.4.2.1 Ultimate Bearing Failure

General bearing capacity equations account for the important design variables. These involve:

- shape (square, rectangle, strip);
- depth of embedment;
- inclination and direction of inclined loads;
- eccentricity of load (could be one-way or two-way);
- sloping ground surface;
- tilted footing base;
- stratification in foundation; and
- scour (taken into account through reduced embedment).

Reference to Bowles (1996) indicates how these factors can be accounted for by using a range of reduction factors. It should be noted that the presence of relatively small horizontal loads can significantly reduce the ultimate geotechnical resistance.

Eccentricity will tend to reduce the effective area of a footing. Moments applied to the footing need to be resolved to components at right angles to the width and length of the footing (sometimes referred to as "biaxial" loading). The effective width of a footing is defined by B' = B - 2e, where B is the width of the footing (m) and e is the eccentricity (m) applicable in that direction. Similarly the effective length of a footing is defined by L' = L - 2e, where L is the length of the footing (m) and e is the eccentricity (m) in that direction. For a strip footing or a footing where L >> B it is only necessary to consider the eccentricity in the direction perpendicular to the footing length. The reduced effective footing size not only results in higher applied bearing pressures, but in reduced ultimate bearing capacity.

For footings where the length is similar to the width it is necessary to calculate the ultimate bearing capacity based on an effective footing area that considers both L' and B'. This is usually undertaken using a computer program for analysis but hand calculations using tabulated factors can also be used. It must be realised that the resulting pressure diagram underneath the footing will not be square, with shape dependent on the relativity of eccentricity in both directions.

In applying the various bearing capacity formulae, consideration needs to be given to the choice of effective stress (drained) or total stress (undrained) soil parameters. The choice will need to take into account the conditions prevailing at the particular site. However, the following notes are provided as a guide.

- For a footing on saturated clay, the available bearing capacity will usually be lowest immediately following loading. The undrained shear strength s_u (total stress) should be used;
- For footings on sand, dissipation of excess pore pressure will usually occur rapidly and drained (effective stress) strength parameters should be used;
- For footings to be constructed on over-consolidated clay subjected to excavation, the undrained strength potentially exceeds that which would apply once drainage has occurred and both states need to be considered in analysis.

Various formulae have been developed for calculation of ultimate bearing capacity, such as the methods developed by Terzaghi, Meyerhof, Brinch-Hansen, Vesic and Bowles. The most appropriate methods for design are considered to be Brinch-Hansen and Vesic since they include allowance for the important design variables noted above.

6.4.2.2 Overall (Global) Instability

Overall global instability must be considered, accounting for possible modes of failure as follows:

- footings on or near an inclined slope (including buried rock profile beneath alluvium);
- footings near an excavation or retaining structure;
- footings near a river, lake, canal, reservoir or the sea shore;
- footings near mine workings or buried structures; and
- possible change in profile due to future construction, scour, erosion, etc.

6.4.2.3 Sliding Failure

Horizontal forces acting above the horizontal bearing surface of the foundation are resisted by the frictional resistance on the base of the footing, by passive resistance against the vertical face of the footing and any shear keys, and by counterforts and ground anchors.

Care should be taken when relying on passive resistance, as a considerable deformation (typically about 30 mm to 50 mm for typical footing dimensions) is usually required to activate full passive pressure. In addition, caution should especially be exercised in utilising passive resistance in clayey locations susceptible to moisture movements as a vertical gap can develop between the soil and the footing as a result of shrinkage. Generally to minimise potential displacement, no more than 50% of the total passive resistance after the application of a geotechnical reduction factor ϕ_g is to be taken to provide resistance against horizontal movement.

In the case of a rock footing, critical sliding failure may occur along a defect surface (e.g. joint in rock, clay filled shear surfaces etc.).

Sliding resistance may be reduced under flood conditions and submerged unit weights below groundwater or flood river levels need to be considered when calculating frictional resistance.

6.4.3 Assessment of Scour

The potential of materials in the river bed to scour should be assessed in order to determine a founding level that cannot be undermined through erosion by water during flood events. In addition, if a minimum embedment is required to achieve the ultimate bearing resistance (R_{ug}), the material into which the footing is embedded must also not be subjected to scour. The Senior Waterways Engineer of Structures Engineering shall be consulted regarding advice on scour.

Depth of scour for areas of constricted flow is primarily a function of the channel flow velocity and bed material grain size, assuming a cohesionless soil. A number of empirical approaches are available for estimation of scour depth, such as the Competent Velocity Method (MRWA, 1982) in which the average constriction scour level is based on the D_{50} of the bed material, mean flow velocity through the waterway opening and a calculated 'competent flow velocity' for the adopted D_{50} and estimated flow depth. Alternatively, hydraulic models such as HEC-RAS (USACE, 2008) can be applied. This model also requires estimates of bed material particle size distribution (D_{50}) as well as design flow and river channel cross-sections. Given the empirical nature of the formulae in these approaches, the derived value shall be checked against previous MRWA experience and with confirmation from the Senior Waterways Engineer.

In the case of a footing on rock, the stream power (P) (kW/m^2) for the design flood needs to be assessed and an erodibility index (K) assessed for the rock in the river bed. Scour Technology (Annandale, 2006) presents guidance on designing for scour. In this publication, an erosion threshold based on the erodibility index and stream power is presented.

Consideration should be given to measuring the parameters that contribute to the erodibility index when planning the site investigation. These include the rock mass strength, the defect frequency and rock quality designation (RQD), number of defect sets, defect roughness, state of alteration and orientation. To obtain this information, quality diamond core drilling and detailed geotechnical logging is required. The final and most accurate assessment of scourability will come from field inspection of the final footing excavation and all designs for spread footings on rock in river beds need to have inherent flexibility so that footings can be lowered, modified or scour protection measures incorporated to cover actual site conditions differing from those inferred from investigations.

For example, the majority of a rock mass might be demonstrated through investigation to be non scourable. It is however, still necessary to allow in the design options for dealing with localised areas around the perimeter of a footing excavation where scour could potentially affect the performance of the footing. Such measures may include removal of scourable material and replacement with mass concrete, deepening of the footing, reinforcement of the rock mass through steel dowels grouted into the rock mass, scour aprons, etc.

The footing excavation in rock in itself exposes the rock around the edge of the excavation to a greater vulnerability to scour than would be the case for a level rock surface. For this reason, it is generally good practice to backfill the overbreak between the formed edge of the footing and the rock excavation with lean-mix concrete.

6.4.4 Serviceability Limit State Design

The stiffness of the foundation will generally control the extent to which serviceability limit state controls design. Section 10.3.5 of the CODE indicates that displacement components of immediate, consolidation and long-term creep need to be considered.

Any possible settlement caused by self-compaction of the soil shall be assessed.

Section 10.3.5.1 of the CODE indicates that a geotechnical strength reduction factor does not need to be applied to the characteristic design values for soil stiffness when assessed on the basis of appropriate field tests or laboratory tests, or by evaluating the behaviour of similar structures.

Tilting of the footing due to large and repetitive load eccentricity, which may cause “doming” of the stratum resulting in “rocking” of the footing, needs for be considered, refer Section 10.3.5.2 of the CODE. This may occur if the peak footing contact pressure below the edge of the footing is more than about 50% of the average bearing pressure at load levels sufficient to exceed the elastic limit or to lead to compaction of the soil below the edges of the footing.

6.4.4.1 Settlement of Shallow Footings

For shallow footings three mechanisms of settlement must be considered

- immediate settlement;
- primary consolidation (time dependent) settlement; and
- long term creep settlement.

Immediate settlement occurs in both sands and clays. In stiff clays and sands, immediate settlement predominates and it is usually sufficient to estimate such movements using so-called closed form equations, Poulos and Davis (1974) or numerical analysis based on elasticity adapted for computer applications e.g. FLEA or PLAXIS. Schmertmann (1970) presents a method of elastic analysis which uses strain based influence factors at depth as the basis for calculating surface settlements.

Estimation of design elastic modulus values can be based upon in-situ test results from Standard Penetration Test (SPT), Cone Penetrometer Test (CPT), Marchetti Dilatometer Test (DMT), shear wave velocity (V_s) tests and pressuremeter tests. Fahey, Lehane and Stewart (2003) have discussed correlations for design modulus with these in situ tests from experience at sites in Perth.

Numerous empirical methods have been published using SPT results to arrive at settlement directly. These may have limited applicability unless correlated with local conditions.

Differential settlement across a larger footing or between adjacent footings also needs to be assessed to restrict structural repercussions. These are estimated using elastic theory after consideration of cross-influence between new or existing footings.

Consolidation (time dependent) settlement is a behaviour which occurs in both sands and clays, but it can only be isolated as a separate phenomenon in wet clays. Winterkorn and Fang (1975) give details of how calculations of consolidation are made. Experience indicates that estimates of rate of consolidation from laboratory test data are grossly conservative as most clays in the Perth Basin have higher horizontal than vertical permeability structure and microfractures which promote drainage.

Creep settlement occurs following the almost complete dissipation of excess pore pressure. This component of settlement, particularly if it is associated with horizontal movement can control long-term (up to 100 year) serviceability. The risk is more pronounced for relatively weak (<40 kPa undrained shear strength) foundations below embankments and footings. Analysis involves settlement estimates based on the results of laboratory one-dimensional consolidation tests to define the value of the coefficient of secondary compression, C_α. The creep settlement estimates can be calculated or plotted on a log-time basis which can then be extended to the applicable design life (e.g. 100 years).

In cohesionless soils, the presence of collapsible soils should be assessed. Collapsible soils such as silty sands of relatively low density are generally stable when dry, but ‘saturation collapse’ can occur resulting in a sudden compression of the ground under a sustained load occurs due to loss of weak inter-granular cementation. Pindan Sands prevalent within the Pilbara and Kimberley region of Western Australia have been found to be prone to saturation collapse under certain conditions. Some loose calcareous sands may also show collapse when saturated or when particles are crushed. Refer to Section 6.4.5 for further discussion.

In cohesive soils, consideration must also be given to displacements caused by volume changes in foundation soils resulting from moisture changes. This effect can be minimised by founding a spread footing below the depth at which seasonal moisture content variations are significant. Further comments are provided in Section 6.4.6.

The influence of stress history on settlement should be taken into consideration. For footings constructed on unconsolidated fill material, consolidation of both the in situ and fill material under the self weight of the fill material need to be assessed. For footings constructed on over-consolidated soils resulted from excavation, unloading and reloading construction stage may need to be considered.

6.4.4.2 Allowable Bearing Pressure of Perth Sands

A large number of bridges in the Perth metropolitan area are founded on spread footings on Perth Sands. Application of normal soil mechanic methods of analysis for Perth Sands may lead to an underestimation of allowable bearing pressures unless appropriate values of soil stiffness are adopted.

As discussed earlier the actual allowable bearing pressure is a factor of footing size, depth of embedment and depth to water table. However, from experience over a large number of structures the following figures may be used for preliminary design:

- Serviceability Limit State average bearing pressure 350 kPa.
- Serviceability Limit State peak bearing pressure 450 kPa.

Provided that:

- Perth sands only;
- Minimum blow count of 10 blows per 300 mm for 750 mm depth below founding level when tested with a standard Perth sand penetrometer in accordance with AS 1289.6.3.3;
- Minimum footing dimensions 2.5 m;
- Minimum depth to underside of footing 1.5 m; and
- Minimum depth of groundwater below underside of footing 1.5 m.

The above bearing pressures should ensure both an adequate factor of safety against bearing capacity failure and an acceptable level of settlement. They are based on a substantial history of settlement monitoring of bridges in the Perth area.

Any cases outside the above limits or other cases where a more detailed analysis is required shall be analysed fully.

Use of the Perth sand penetrometer is not suitable for assessing the relative density or achieved level of compaction in clayey sands and sands with lenses and pockets of clay. Such materials are commonly exposed in areas of Guildford Formation or shallow Bassendean Sands over Guildford Formation. A direct or indirect measure of soil strength is preferred in these conditions.

6.4.4.3 Stiffness Based on SPT Data

Massey (1976) compared the results of many empirical methods with observed settlements on a number of bridges. He concluded that for the leached sands of the Spearwood dune system typical of the Perth coastal plain, the best estimate for elastic modulus was given by the method of Schultze and Sherif (1973).

Schultze and Sherif gave:

$$E_m = 1.68\, N^{0.87}\, B\, (1 + 0.4\, D/B) \qquad \text{Eqn 1}$$

where E_m is soil elastic modulus in MPa

N is the SPT blow count for 300 mm (uncorrected)

B is the footing width in metres

D is the footing embedment depth in metres

The value of modulus from equation (1) can be used in the estimation of settlement using elastic theory.

6.4.4.4 Stiffness Based on CPT Data

The stiffness of Perth sands is commonly estimated from CPT results. For typical natural Perth sands, the relationship $E = a \times q_c$ is commonly adopted. The value of the parameter ‘a’ depends on the geological history and stress history of the soil deposit.

Data presented by Baldi, et. al. (1989) indicates that the parameter ‘a’ typically varies:

- between about 2 and 4 for geologically recent normally consolidated sands; and
- between about 6 and 12 for geologically aged or over-consolidated sands.

Cocks et. al. (2003) analysed settlement monitoring data from two sites in Perth and derived values of the parameter ‘a’ as follows:

- between 12 and 15 at loads below a “threshold stress”; and
- between 3 and 4 for loads in excess of the “threshold stress”.

The threshold stress is not easily determined in advance and was found to be between about 60 kPa and 140 kPa for the two sites investigated. This phenomenon is likely to be a result of non-linear stress-strain behaviour of the soil, as discussed in detail by Fahey, Lehane and Stewart (2003).

For engineering design purposes for natural geologically aged Perth sands in a medium dense or denser condition, the parameter 'a' is often assigned values in the range 6 to 8. For relatively loose sands, the parameter 'a' may be as low as about 1 to 2. These lower values are consistent with the parameters recommended by Schmertmann (1970).

6.4.4.5 Stiffness Based on Advanced In Situ Test Data

Soil stiffness can be estimated from the results of relatively advanced in situ tests where warranted. Typical more advanced test methods that are currently available include:

- shear wave velocity;
- flat plate dilatometer; and
- self-boring pressuremeter.

The use of the above methods for estimation of the stiffness of Perth sands is described by Fahey, Lehane and Stewart (2003).

6.4.5 Collapsing Soils

Significant settlements can occur following saturation for spread footings founded on low density sands and silts. Soils of this nature occur in the Pilbara and Kimberley areas. This mechanism needs to be considered in Aeolian deposits (e.g. dunes) in arid areas. Specialist advice should be sought when the potential for this mechanism of settlement is suspected.

Collapsing soils can be recognised from low SPT or PSP (Perth sand penetrometer) blow counts. In particular, blow counts below 5 blows/300 mm warrant further investigation.

The Water Corporation has published a report on collapsing soils; refer Water Authority of Western Australia (1990). This publication deals with the identification and design considerations surrounding these materials.

6.4.6 Expansive Soils

Although expansive soils are more significant for lightly loaded shallow footings, in parts of Western Australia, highly expansive soils with swell pressures in excess of applied pressures to the ground beneath footings do occur. In these cases, the serviceability limit state due to heave may need to be examined.

Potential for settlement or heave can be assessed using the principles set out in AS2870, *Residential slabs and footings – Construction*. The guidance on footing design however in this Standard is for low rise residential structures and is inappropriate for the design of bridge footings. Nonetheless if large shrink or swell values are assessed, the magnitude of seasonal movement can be controlled by founding the footing at depth to reduce the amount of movement due to seasonal moisture change. If the footing is founded at the depth of seasonal moisture change then the potential for settlement or heave due to either desiccation or swelling of the clay foundation soil becomes negligible unless large trees are present nearby.

6.4.7 Footings on Rock

6.4.7.1 Field/Geological Assessment

Footings on rock require detailed field and geological assessment and perhaps a greater level of supervision during construction than for footings on soils. The reason for this is the degree to which the level, strength and nature of rock can vary in a short distance both vertically and horizontal due to differential weathering and preferential erosion along faults and fractures.

As a minimum, there should be one diamond cored borehole per pier and abutment. If water conditions permit, probing is useful to profile the surface of the rock and anticipate level variation within footing excavations. The designer should be cognisant of the possibility of sudden drops in the rock surface. This is particularly common in riverbeds where river erosion may have resulted in a cliff or vertical face in the rock surface that has been subsequently buried beneath alluvium. Exploration of ground conditions outside the footing base during investigation is necessary where sudden variations are expected.

During construction, an experienced geotechnical engineer should monitor the footing excavation and inspect the final base. This is particularly important so that scourable material can be identified, the footing level deepened or made shallower if necessary and base and perimeter treatment undertaken to ensure soft weathered zones are removed. These areas should be refilled with make-up concrete, to minimise unnecessary rock breaking. Advice on treatment necessary to prevent deterioration of rock subject to slaking or softening on exposure to water or atmosphere could also be required.

6.4.7.2 Bearing Pressures and Footing Design

Assessment of ultimate bearing pressures for rock requires careful consideration. Presumptive bearing pressures published in many soil mechanics text books can overestimate the ultimate bearing pressures. An approach that takes account of the rock mass properties is recommended e.g. Wyllie (1999). In some instances if a rock is highly fractured the rock mass may be treated as a dense granular soil and ultimate bearing capacity obtained via soil mechanics bearing capacity equations. This is often the approach taken for weak rock, e.g. limestone.

The position of the footing relative to the rock profile needs careful consideration. This includes the potential of a footing slide on a surface defect, proximity of abutment footings to potentially scourable or unstable river cliffs, both exposed and buried, beneath alluvium.

6.4.7.3 Scour

Footings on rock are sensitive to scour as rock ledges have the tendency to constrict the stream channel resulting in higher velocities and stream energy. The influence of the bridge pier and footing may further augment the stream erosive power. It is important that weak zones beneath footings on rock are removed and refilled with concrete. Further information on scour is given under Section 6.4.3.

6.4.8 Passive and Active Anchors and Dowels

The difference between an anchor and a dowel is that an anchor is designed specifically to take tension. Anchors in footings may be vertical or inclined and are commonly used to resist overturning forces and reduce large footing base eccentricities. The term dowel refers to a shorter anchor not necessarily designed to carry tensile loads, but instead may be a connection. For instance, to connect a footing to horizontally laminated layer of rock or to reinforce a fractured rock mass.

Passive anchors are not prestressed, whilst active anchors are prestressed. The choice of using passive or active anchors is generally governed by structural design issues and relates to the type and amount of movement of the structure before the anchors engage. However, the potential for ‘unzipping’ of long bonded anchorage lengths in a passive anchor may result in an active anchor being chosen that requires a shorter bonded length at a greater depth.

Unzipping is a mechanism that can occur if too much elastic extension occurs when load is applied to an anchor. If unzipping occurs, the geotechnical strength of the anchor will be less than expected from the peak soil/rock contact strength. Unzipping can be reduced to some extent by reducing the elastic extension of the anchor. This can be achieved through:

a) increasing the section stiffness of the anchor (i.e. a larger diameter anchor or by bundling multiple bars or strands);

b) reducing the length of the anchor; or

c) reducing the applied load.

Generally options b) and c) involve having a greater number of shorter anchors.

Section 12 of the CODE presents design and testing requirements. Further guidance on the design of Ground Anchors is provided in Appendix B of AS 4678-2002 Earth-Retaining Structures.

For anchors in rock or soil, the design must consider the following:

a) uplift of soil or rock mass;

b) tendon design;

c) soil or rock to grout and grout to tendon bond (grouted anchors); and

d) durability.

Anchorages must also be designed for other relevant factors such as:

a) creep movement of soil under sustained load – this is primarily an issue in relatively weak cohesive soil, or for anchors with a relatively low factor of safety;

b) level of groundwater and possibility of changes in the level which may reduce the effective stress in the ground;

c) provision for drainage around the anchor head, so that water does not affect anchorage performance;

d) depth of anchorages relative to global stability of the structure, so that the anchorage zone extends beyond the extent of any global stability mechanism or the active soil block;

e) rigidity of the structure being supported so that the extension of the anchors as they are loaded does not permit excessive distortion;

f) possibility of movement of the structure;

g) group effects, where interaction between adjacent anchors leads to lower anchor group capacity than the sum of the individual anchors;

h) in the case of anchorages in soil, the behaviour of the soil due to the anchor loads, e.g. strain softening, breakdown of inter-granular bonding, particle crushing etc.;

i) in the case of anchorages in rock, anisotropy, inhomogeneity, fracturing and discontinuities of the rock leading to reduced resistance across rock defects;

j) method of installation, so that ground is not disturbed excessively during installation;

k) geometry of the anchorage to take account of geotechnical stratification, rock defects etc.;

l) non-uniform stress distribution, particularly the potential for ‘unzipping’ failure for anchors with long bond lengths; and

m) strength of mechanical anchorage for mechanically secured anchors, e.g. strand, connection and anchor plate strength.

Limit state principles apply with design factors, material factors and importance category reduction factors applied to the various design checks.

The capacity of an anchor is governed by the weakest link in the following list:

- strength of tendon;
- strength of grout to tendon bond;
- strength of grout to rock/soil bond;
- weight of pullout cone or other shape[3].

Anchor design requires close liaison between the geotechnical and structural designers to take account of factors such as unzipping and the amount of displacement that is permissible before uplift loads are transferred to anchors.

6.4.9 Bridges over Waterways, Road or Rail

A slightly different approach to footing design is often required for bridges over waterways to that for bridges over rail or roads.

In the case of a bridge over a waterway, the potential for the river or stream to scour the foundation materials is of paramount consideration. If scourable material is present, it is necessary to protect the foundation from scour, or more commonly to deepen the footing, to a level such that resistance of materials to scour induced erosion improve or the stream energy capable of causing scour decreases or a combination of the two.

The result is that footings may need to be deeper than can easily be constructed and piled foundations become the more practical option. The intermittent or permanent presence of water in a river may also become a construction consideration that significantly influences the decision making process for footing design.

A depth of about 6 m is considered to be a maximum practical depth of excavation for shallow footings before piling may become a more practical and economic option. This depth may be shallower if groundwater or interaction with adjacent structures, services etc., is a major issue. Generally beyond a depth of 6 m, temporary support of the sides of the excavation becomes a major consideration.

Investigation costs over water may also be much higher.

6.4.10 Bridge Approach Embankments

Bridge approach embankments built on susceptible soils can result in considerable settlement of the underlying soil. This settlement can have immediate, long-term consolidation and creep components. If settlements are principally immediate and the embankment fill is placed before the superstructure is constructed, then effects will be minimal. Otherwise, proper allowance should be made for the staging of construction and associated settlements. Preloading of the embankment

[3] Where multiple anchors are used, it is possible that the pullout cones will overlap. It is therefore necessary to consider group effects. The effective weight of rock or soil available to an individual anchor is reduced where the pullout cones overlap. It is necessary to sketch the shape of the block of soil or rock that is available to resist pullout when calculating the resistance of individual anchors. For uniformly spaced anchors, the resistance will vary depending on whether the anchor is located in the middle (least resistance), along the edge of a group, or at the corner of a group (most resistance).

will help reduce post-construction settlements. If preloading is not an option, ground improvements through surcharge loading, stone columns, deep soil mix columns, piles or similar may be used.

6.4.11 Seismic Effects

Significant seismic events may impart cyclic loading to the foundation soil. This cyclic loading can lead to an increase in pore water pressure, which may lead to the following effects:

- a loss of strength, possibly resulting in liquefaction of the soil;
- ground settlement during and after the earthquake caused by densification; and
- lateral ground movement.

The likelihood of liquefaction occurring during an earthquake should be assessed during the design of the structure. Non-cohesive soils that are in a very loose to loose state are most likely to be susceptible to liquefaction or other potentially adverse effects following an earthquake. Methods of assessing the likelihood of earthquake liquefaction occurring are provided by Youd et. al. (2001).

Where earthquake liquefaction or other adverse effects following an earthquake are identified as being significant, piled footings may be considered to support the structure, or ground improvement may be carried out to mitigate against seismic effects.

6.4.12 Durability

Consideration should be given to the detrimental effects of the environment on the materials used for the construction of the footings (refer to Clause 9 of the CODE). Consideration must be given to the possible deterioration of structural components as a result of:

- aggressive substances in soils or rocks;
- aggressive substances in groundwater, seawater and water in streams; and
- the abrasive actions of debris and grit in fast flowing streams.

Specific aggressive environments that may need consideration include:

- pH;
- sulphate;
- chloride;
- electrical resistivity;
- sulphate reducing bacteria; and
- acid sulphate soils.

Aggressive environments are typically dealt with by increasing the concrete cover, specifying a higher concrete characteristic strength or using sulphate resisting cement. Section 4 of AS 5100.5 provides details of design for durability for concrete structures.

6.4.13 Utilisation of Geotechnical Design Information for Structural Design

The structural designer must ensure that the interactions of the structural elements with the soil are compatible with geotechnical design limits. As the final design actions are unlikely to have been determined prior to investigation, it is desirable for the geotechnical report to provide sufficient information to cater for a variety of design solutions. Hence the report should contain:

- ultimate bearing capacity (R_{ug}) for various footing sizes at expected embedment depths;
- recommended geotechnical reduction factor Φ, based on the CODE;
- load - settlement tables or plots; and

- other pertinent factors applicable to the site conditions and proposed structure.

The structural designer may then compare the ultimate load actions to the value ΦR_{ug} and in addition estimate the settlement using the serviceability load combinations or working loads load in conjunction with the settlement tables or plots.

6.5 CONSTRUCTION CONSIDERATIONS

6.5.1 Foundation Improvement

Where settlement is likely to be excessive or bearing capacity inadequate there are a number of techniques which can be used to improve foundation soils so that spread footings can still be used. These include:

- compaction with conventional plant to improve the stiffness of the upper zones of the foundation;
- surcharge preloading for accelerating compression of weak zones;
- improving stiffness of the foundation by constructing compacted columns in situ beneath the footing using coarse aggregate (stone columns);
- chemical grouting injected into soil voids or cavities for improving strength by cementation;
- vibroflotation to increase the relative density of granular soils;
- deep soil mixing for improving soil stiffness and strength by in situ mixing of soil and cement;
- reinforcement through provision of geogrid or structural materials to improve strength;
- dynamic compaction to increase the relative density of granular soils; and
- removal of unsuitable material and replacement with suitable material.

Specialist geotechnical advice should be sought where foundation improvement is warranted.

Shallow footings may not be the correct choice for the site and piled foundations should be considered if extensive foundation improvement is required for the satisfactory performance of shallow footings (refer Chapter 7 of this Manual).

6.5.2 Footing Base Preparation

6.5.2.1 Clay and Sand

In preparation of sections of the specification which refer to footing base preparation, the designer must consider the following:

- need for blinding layer to preserve strength and integrity of ground prior to concrete placement – particularly for clay foundations;
- requirements for compaction control at footing base level. Need for inspection of footing base by a geotechnical engineer before release of any hold point;
- cleaning of base;
- levelling of base.

The relative importance of these factors depends on the importance of these factors in design analysis. For example, the required undrained shear strength of the upper level of the foundation may be critical in determining settlement.

As almost all spread footings are of reinforced concrete, it is obvious that construction requires the standard practice of excavation, compaction of the base, pouring blinding concrete to protect the bearing surface and support reinforcement, forming sides for pouring concrete and finally stripping and backfilling the over-excavation. At certain locations, it may be necessary to dewater the site to carry-out the above operations.

It is often prudent to leave the last 300 mm of material in a base until immediately before the site visit from the engineer to inspect the excavation base. The reason for this is because undue exposure of the footing base prior to pouring blinding concrete leaves the inspecting engineer with no option but to request the founding level to be deepened, if undue softening has occurred due to early exposure.

It is essential to adequately dewater a footing base and allow sufficient time to both dewater and depressurise the base of the excavation before advancing an excavation. Inadequate dewatering is one of the most common reasons for base preparation not meeting required standards. Advancing an excavation too fast can result in disturbance (loosening and softening) of the ground several metres below the footing base. If not rectified this may result in excessive settlement occurring.

6.5.2.2 Rock

In hard rocky grounds where it is necessary to excavate to place the footing below the existing rock level, possibly because of the presence of a layer of weathered rock, then the footing should be cast against the excavated rock surface without forms if the footing can be neatly formed without overbreak.

For bridges founded on rock but not in an environment subject to scour or erosion, base preparation is less critical than for footings in a river bed that may be subject to scour. Nonetheless, the footing base will need to be cleaned of loose material, clay, fines, etc. Leaving a layer of weak material, often pulverised material from the excavation process itself introduces not only a layer of potentially compressible material, but a plane of weakness that may not provide adequate resistance to the horizontal forces acting on the bridge that need to be transferred into the ground to prevent sliding.

In the case of a footing on rock in a riverbed subject to scour, the level of base preparation becomes more critical. If weak zones around the perimeter are identified that may be scoured these need to be chased out and replaced with make-up concrete. All loose or softened material or material disturbed or fragmented during the construction process should be removed. A large excavator is not an efficient tool for base preparation, and generally a first pass at base preparation may be undertaken using a mini excavator, followed by hand techniques, such as shovels and stiff brushes. Use of airlifts and airlances provides a still higher level of base preparation. Water jetting may be used to clean hard rock footing such as fresh granite. Water however must be avoided for highly fractured rocks, rocks with clay filled defects and for rocks such as mudstones that may be susceptible to slaking or swelling.

6.5.3 Dewatering

Although dewatering systems are generally part of the temporary works and are designed by the contractor, the requirement for dewatering is also an important part of the designer's decision making process in assessing the founding level for spread footings or the need for piled footings. Similarly, if dewatering is required, consideration needs to be given to the potential areal extent and level of groundwater drawdown and the impact that this may have on local wetlands, settlement of nearby structures, etc.

Dewatering systems require space to install, and a license is generally required to extract and discharge the water. The potential presence of acid sulphate soils become a major consideration as does any pre-existing contamination of the groundwater when considering discharge of the extract groundwater. Information on dewatering systems, design consideration etc., is provided in the CIRIA publication - Control of Groundwater for Temporary Works (Somerville 1986).

It is important that a dewatering system that is appropriate for the ground conditions is designed and installed. The dewatering system should be allowed to operate for a sufficient length of time to adequately dewater and depressurise the footing or pile cap bases before excavation proceeds.

Wellpoints are commonly used in Perth to dewater footing bases where the soils are predominantly sand. Experience has shown that the sacrificial wellpoints are sometimes needed below the centre of large footing bases as the wellpoints around the perimeter may be incapable of adequately drawing the groundwater level down beneath the centre of a large base. If a sacrificial wellpoint is used the wellpoint spear should be grouted up at the time the system is turned off.

6.5.4 Preservation of Base Conditions

Excavation should proceed in a timely manner to minimise potential disturbance to the footing base. Once the footing base has been approved, the footing base should be blinded with lean mix concrete and the footing formed as soon as practical. Backfilling around the footing should take place to a level at least 1 m above the equilibrium groundwater before any dewatering system is turned off.

Common reasons for construction related disturbance to the footing base include:

- excavating before sufficient time to adequately dewater the ground has occurred;
- dewatering system is not appropriate or sufficient for the ground conditions;
- attempts to compact the footing base are undertaken before the footing base has been adequately dewatered (dewatering to approximately 1.5 m below the footing base is generally required);
- careless excavation resulting in overbreak (the footing should be carefully excavated in layers as final level approaches); and
- too long a period between excavation and blinding, resulting in the footing base being exposed to climatic effects.

6.5.5 Temporary Works

The design of temporary works is normally the responsibility of the contractor. Nonetheless, the constructability of the footing system is an important design consideration.

Where an excavation may stand vertical, consideration may be given to the footing being cast directly against the side of the excavation, negating the need for formwork. This can however only be undertaken if the sides of the excavation are stable and safe. The nature of the materials also must be such that a neat vertical excavation face can be formed with minimal overbreak. If too much overbreak occurs, the footing will need to be formed in order to comply with maximum cover requirements over reinforcing elements.

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CHAPTER 7

PILE FOUNDATIONS

CONTENTS

7.1 INTRODUCTION

The aim of this Chapter is to provide some guidance on the design of piled foundations with specific reference to the types commonly used by MRWA. The pile types used by MRWA are described and details given to assist in the selection of the most appropriate type for a given situation.

The Chapter does not try to be comprehensive; many types of piles in common use elsewhere are only mentioned briefly herein, and design is not covered in full detail. However, cross referencing to both the AS 5100.3 - 2004 Australian Standard Bridge Design Code, particularly Part 3, Foundations and soil-supporting structures and to various standard texts is provided. The cross references provide additional information and detail not presented herein.

Only permanent, structural, mainly axially loaded piles are discussed here. For special types of piles e.g. sheet piling, and stone columns, and ground improvement works such as vibroflotation and deep soil mixing, the reader is referred to specialist literature.

The main function of a piled foundation is to transfer loads from the bridge substructure to a lower level bearing stratum capable of adequately sustaining the loads. This can be accomplished using a variety of types of pile foundations. This Chapter is intended to be applicable to piles made of a range of materials and installed by a variety of methods.

Pile foundations are used as alternatives to other types of foundation systems, (e.g. spread footings), to avoid rupture of weak soil layers, excessive settlements or the possible effects of scour. Piles transfer loads to a lower level bearing stratum by skin friction (adhesion) along the sides of the pile permanently embedded in the ground, end bearing at the base of the pile, or a combination of the two.

The proportions of the load taken by skin friction and end bearing will depend on the soil type and the method of installation of the pile. All these factors are covered in later Sections of this Chapter, but it is worth bearing in mind that skin friction is usually mobilised by very small movements of the pile relative to the surrounding soil, whereas the full mobilisation of pile end bearing resistance requires much greater strains in the soil. Pile base deflections of up to 10% of base diameter or more may be required to fully mobilise all of the available base resistance. This is not generally a problem for serviceability, however, where long purely end-bearing piles are used, the design needs to account of larger displacements where the base resistance is more than say 80% of the ultimate capacity.

It is stressed that this Chapter is not intended to be, and most certainly should not be used as, a "recipe book" for pile foundation design. With the range of structural types used by MRWA, and more importantly the enormous variety of sub-soil conditions encountered, each pile design is unique and must be dealt with strictly according to its merits.

The nomenclature used throughout this Chapter is in accordance with Part 3 of AS 5100, unless otherwise noted.

7.2 TYPES OF PILE

Piles can be classified either in accordance with the material they are made from (i.e. timber, concrete, steel or combinations thereof), or the method of installation (i.e. driven or bored).

The majority of piles used by MRWA are driven and the principal types in use are: -

- Concrete - precast, prestressed and usually either 350 mm square with a normal working load of 700 kN or a 450 mm octagon with a working load of 1000 kN. Also, some precast normally reinforced segmented concrete piles have been used;
- Steel - H sections or steel tubes in a wide range of sizes and capacities;
- Composite - combined steel and precast concrete; and also
- Timber - round, untreated Jarrah with a working load of around 500 kN. Note that for a number of reasons including the lack of suitable quality jarrah, timber piles have not been used for many years.

There are increasing opportunities and/or requirements to use bored piles, either CFA (continuous flight auger) or bored, rock socketed pile types. The opportunities have arisen due to availability of dedicated plant and improved construction skills.

In addition to those pile types listed above, there are a large number of other possible pile types that could be used, but since these are not in common use in WA they are not discussed in detail here. The different pile classifications are presented in Figure. 7.1. Reference should be made to one of the standard piling texts in the general reference list for information on these piles.

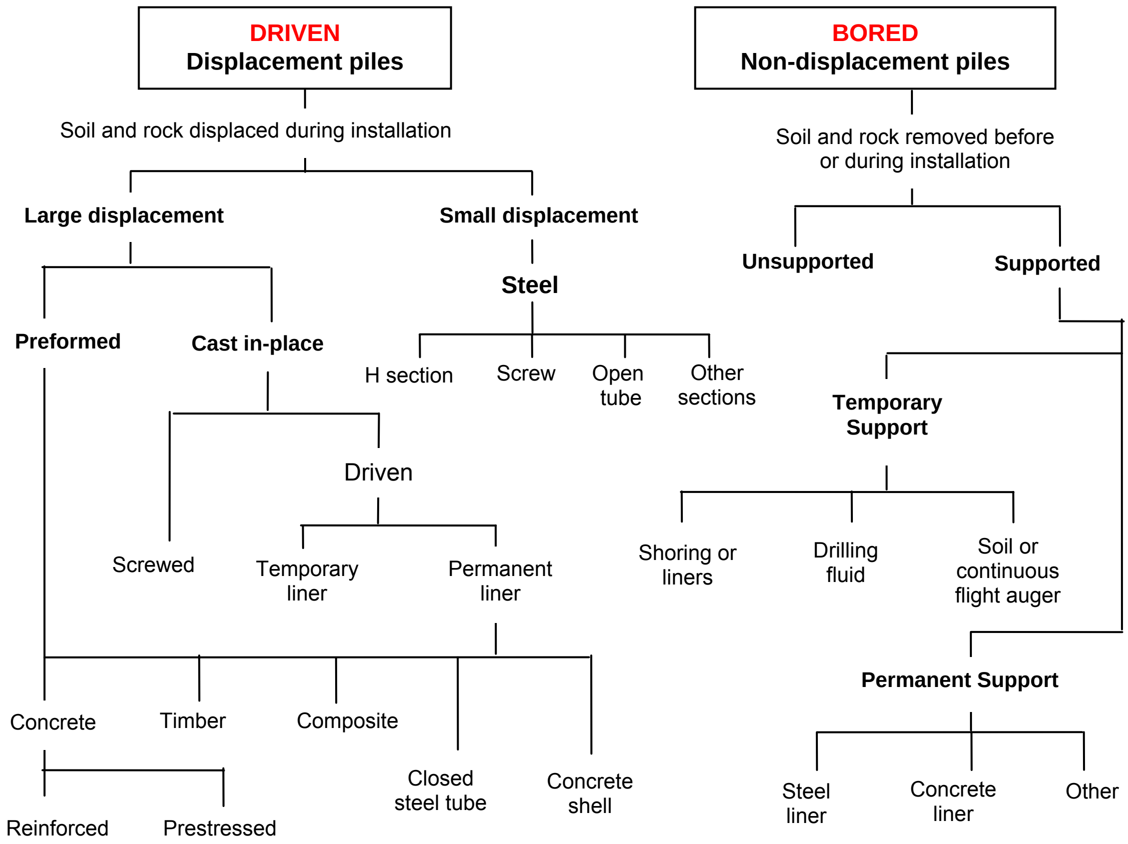


Figure 7.1 – CLASSIFICATION OF PILE TYPES

Further details of the different pile types and the effects of the different methods of pile installation are presented in Appendices A, B and C.

7.3 STAGES IN A TYPICAL FOUNDATION DESIGN

The different stages in a typical foundation design are shown diagrammatically in the flow chart presented in Figure 7.2 (for driven piles), and are outlined below. It should be stressed that this is specifically the approach used by engineers in MRWA. It has been found that this method of analysis is the quickest, easiest and most logical for the structures and soils found in WA, it may not necessarily be the best approach in other areas.

For pile types other than driven piles, a similar design process is normally followed to that outlined in Figure 7.2 and the text below. While pile driving characteristics would not be considered for other pile types, pile installation and QA/QC requirements specific to that pile type would need to be considered.

The Sections in the Chapter follow the same sequence as the flow chart and this is different to most text books which tend to cover all the single pile calculations (vertical and lateral load capacity and deflection by static and dynamic methods), before considering piles in groups. However, it is felt that the approach used here is valid as piles for bridges are nearly always used in groups and in the vast majority of cases the vertical load carrying capacity is by far the most important criteria.

There may be other matters which arise such as constraints due to nearby structures, environmental conditions and ethnographic considerations. These must be recognised and appropriate design remedies adopted. This present discussion concentrates on geotechnical considerations.

A typical foundation design process for a bridge would consist of: -

a) Calculation of Relevant Loadings

These usually come from the overall structural analysis of the complete structure, but the loads have to be transferred down to the pile cap, usually to the underside, and thus must include the weight of the piers and the pile cap. All six components of load must be considered; vertical, longitudinal and transverse forces, moments about the longitudinal and transverse axis, plus any torsion (moment about the vertical axis), that may be under both serviceability and ultimate limit state load conditions. The reader is referred to AS 5100-Parts 2 for more details on the calculation of design loads.

b) Assessment of Pile Type to be Used (Refer Section 7.4)

Decide on the most appropriate type of pile that is suitable for the site geotechnical conditions and the structure. If there is no strong indication in any particular direction, consider a number of options and make the decision on economic grounds after carrying out alternate designs.

c) Calculation of Pile Ultimate Capacity (Refer Sections 7.5 and 7.6)

For the chosen type of pile, or piles, using the available soils information and current geotechnical practice, calculate the ultimate geotechnical static capacity for vertical load of a single isolated pile (R_{ug} in the nomenclature of AS 5100, and $R_{d,ug}$ and AS 2159 2009 (the Australian Piling code).

Having calculated the ultimate single pile geotechnical capacity (R_{ug}), calculate the pile design geotechnical strength $R_{d,g}$ by multiplying the pile ultimate geotechnical strength by the appropriate geotechnical material strength reduction factor Φ_g.

The appropriate value of Φ_g can be calculated in accordance with Clause 4.3.1 of AS 2159.

d) Trial Pile Group Geometry

Considering the ultimate limit state bridge design loads make an "educated" guess at a suitable pile arrangement. It will usually be necessary to provide at least 1.5 times the required vertical pile design geotechnical strength resistance to allow for horizontal forces and bending moments. Consider including raked piles if there are significant horizontal forces which lead to deflections or bending moments that are larger than acceptable. These should be on the outside of the group, with a preferred rake of no more than 1 horizontal to 5 vertical, (absolute maximum 1:4), as any more can make the piles difficult to drive.

e) Analysis of Pile Group (Refer Section 7.7)

Analyse the chosen group. There are a variety of design methods available, but the PIGLET program (Randolph, 1994) is a suitable relatively user friendly program.

Analyse the pile group for the worst combination of ultimate limit state loads to assess pile structural actions. Analyse the group under serviceability load cases to assess expected deflections.

f) Check of Maximum and Minimum Pile Loads

The maximum pile compression load under ultimate limit state loading will be the usual design criterion. Check that there are no unacceptably high pile tensions or bending moments and shear forces, particularly if there are significant ultimate limit load state load combinations that include bending moments and/or horizontal loads acting on the group, especially in conjunction with a low vertical load.

g) Check that Pile Cap Deflections are Acceptable under Serviceability Limit State Loads (Refer Sections 7.8 and 7.9)

These will come from the PIGLET output for serviceability limit state load combinations. Check calculated deflections against allowable values. Typically at serviceability limit state pile cap deflections should be not more than about 15 mm to 20 mm. Different structures may have differing settlement tolerances.

Consider likely differential settlement tolerances for the type of structure both longitudinally and transversely. The settlement analyses should take into consideration any restraint provided by the soil around the pile caps if it can be relied upon in the long term and scour is not a consideration.

Note that for most common structures, deflections are very rarely a limiting criteria as long as the ultimate limit state load capacity criteria are met.

h) Iterate Previous Steps (d) to (g) to Obtain a Satisfactory Pile Arrangement

The number of iterations required will depend on the accuracy of the first layout considered, the complexity and number of the ultimate and serviceability limit state load cases and the acceptable deflection criteria.

i) Select Pile Type

If a number of different pile types were identified initially, carry out preliminary designs for all of them and choose the preferred alternative.

j) Structural Check on Pile and Pile/Pile Cap Connection for Calculated Loads and Moments (Refer Section 7.10)

Using the worst combinations of calculated pile ultimate axial load shear load and bending moments obtained from the final PIGLET output, check the structural design of the chosen pile type and ensure that the shear forces and bending moments can be carried by the proposed piles. Consider column effects for piles that are subject to scour or which extend from ground surface to crosshead level.

Also check that the calculated pile top moments and shears can be transferred into the pile cap. If not, then the pile type may have to be changed or the foundation re-analysed assuming a pinned connection between the piles and the cap. A pinned connection will probably make little difference to the vertical pile loads, but may result in higher, and perhaps unacceptable, horizontal deformations under serviceability loads and a peak pile bending moment that is present some distance down the pile shaft rather than at the pile head.

k) Structural Design of Pile Cap (Refer Section 7.10.3)

Use the worst combination of ultimate limit state pile loads from the final PIGLET analysis to carry out the structural design of the pile cap.

l) Check of "Driveability" of Pile using GRLWEAP (Refer Section 7.11)

For driven pile solutions, assess that the piles can be driven and tested to attain the required ultimate geotechnical axial compression capacity with the proposed piling equipment.

Also check the driving stresses that are likely to be encountered during pile driving.

The driveability analysis will give an indication of the pile set or pile penetration per hammer blow to achieve a given ultimate pile geotechnical capacity, and an estimated pile toe level at which the design ultimate geotechnical capacity may be achieved.

For concrete piles, the driveability analysis should include an allowance for a pile top cushion material between the top of the pile and the pile driving hammer helmet. Typically up to three sheets of 19 mm plywood are used.

For piles other than driven preformed piles, consideration should also be given to the need for pile testing to confirm pile capacity and/or integrity. The pile test results can be useful in confirming that construction control exercised by driving response (e.g. records of set and temporary compression at end of driving) is a valid indicator of pile capacity. This process would be applicable to all normal types of pile foundation, although it is of course possible that special foundation conditions or large, important structures may require further analysis, especially for settlement or lateral resistance, and a full scale pile load test may be warranted.

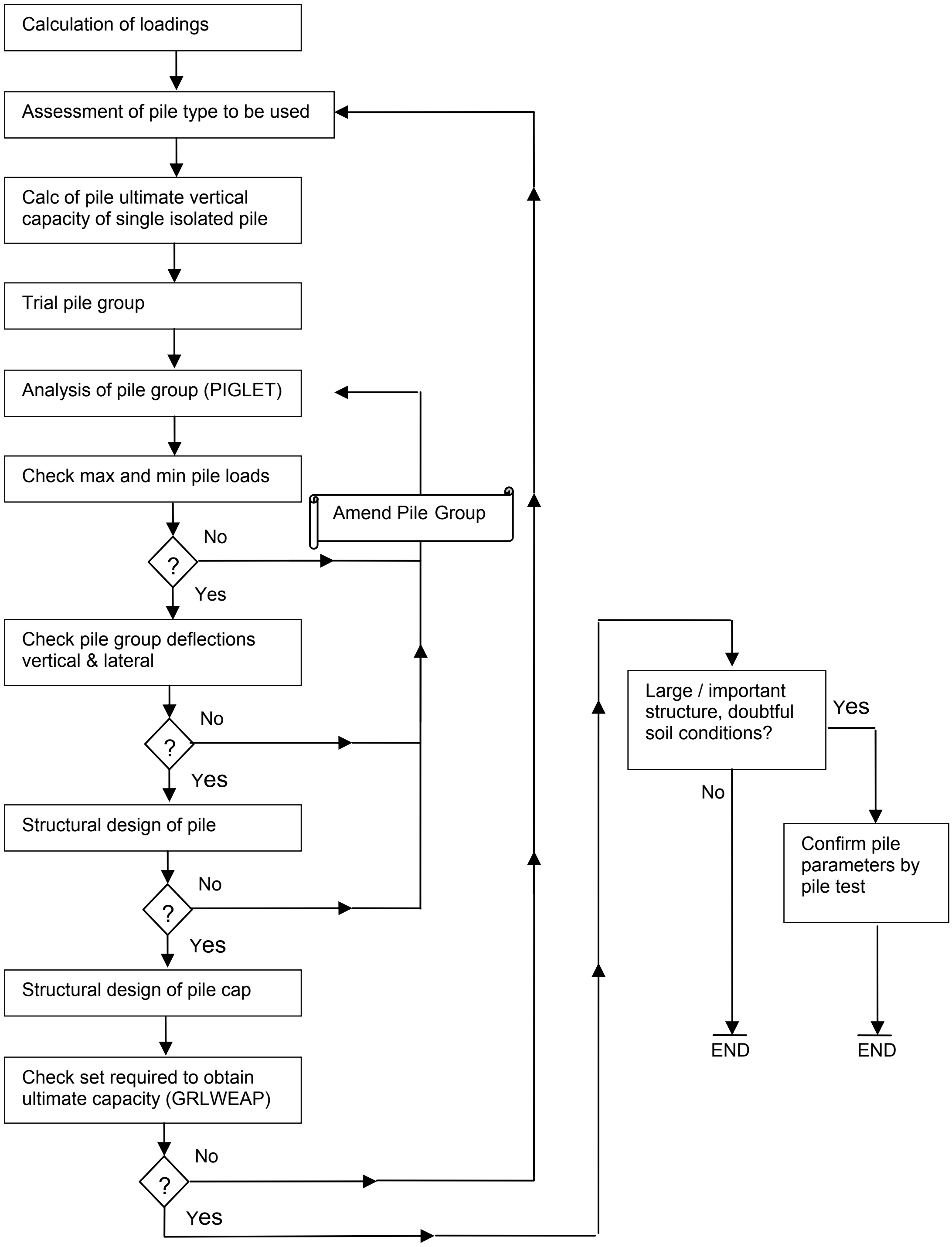


Figure 7.2 – STAGES IN A TYPICAL DESIGN FOR DRIVEN PILES

7.4 SELECTION OF PILE TYPE

This is the first item in the foundation design process to be undertaken once the loads are known and bore logs available. The material, type and the method of installation of the proposed piles have to be identified as they influence the subsequent calculations. These depend on a number of factors, such as the type of structure and its location, the size of the loads to be carried, and most importantly the nature of the bearing stratum and overburden. These are shown diagrammatically at Figures 7.3 and 7.3a, and discussed below.

7.4.1 Sub-Soil Conditions

The single most important factor influencing pile selection is the sub-soil conditions, as revealed by the site investigation, i.e. boring and testing and/or previous pile driving in the area. This will show whether the proposed pile will carry the load by end-bearing, friction, or a combination of the two, and also whether the ground is suitable for bored piles.

A hard founding layer at a reasonable depth will indicate that end-bearing piles are practical; driven precast concrete piles if the loads are moderate and the overlying strata easy to penetrate, otherwise driven steel piles, with low displacement H piles the choice if there are very hard layers to penetrate.

If the overlying soil is suitable and ground water is not an issue, e.g. medium/stiff clays above the water table, then uncased bored piles may also be a possibility.

If ground conditions are such that bored piles are likely to require temporary shaft support to prevent collapse, consideration may be given to driven cast in place enlarged base piles (Frankipile) providing that a suitable end bearing layer is present within 12 to 18 m from the ground surface. Alternatively, continuous flight auger piles (which provide their own shaft support) or bored piles drilled with an appropriate drilling fluid (bentonite or polymer slurry) may be considered. These types of pile must be load and/or integrity tested.

Generally it will be found that a driven pile is more economical on a cost/tonne of load capacity than other types of piles, unless there are very high loads, deep scour requirements or corrosion issues to be addressed in the design process.

For driven preformed piles, the depth to the founding layer is important because of the difficulties of extending some types of piles. If precast concrete piles need to be longer than about 12 m to 15 m some form of proprietary steel joint for precast concrete piles will be required. Several piling contractors have joints that have been approved by the Victorian and NSW road authorities. If joints are not required, consideration may be given to prestressed concrete piles (which can be handled in longer unjointed lengths) or alternatively steel preformed tube or H sections which can be extended by welding.

With driven piles the likely resistance to driving must also be assessed to check on driving stresses and the size of pile driving hammer that will be required to driven the piles. Driven steel piles may have to be used (or bored piles), if the driveability assessment indicates that concrete piles may be damaged during driving or dynamic testing.

Another factor to consider when selecting a pile type is the potential presence of sea water, brackish water or aggressive ground water which could lead to a durability/corrosion problem. Durability issues may require perhaps the use of concrete rather than steel, and possibly even the need to use special cements or cement additives such as silica fume.

If driving conditions are hard then this may preclude the use of driven normally reinforced concrete piles. Prestressed concrete piles may provide a more durable pile in marine environments. Driven steel tube piles may be provided with suitable protective coatings in the zone of potential corrosion. Alternatively, driven steel tubes, subsequently cleaned out and filled with reinforced concrete may be used where the outer steel lining is considered to be sacrificial and the reinforced concrete infill provides for long term durability.

For most ground conditions, any re-strike of a driven pile after a period of days will require some increase in driving energy for the remobilisation of the pile. This is referred to as “set-up” of the pile.

7.4.2 Loading

The magnitude and type of loading and the required type of pile load testing will also affect the choice of pile.

Ultimate axial loads of up to about 3000 kN to 4000 kN per pile can satisfactorily be catered for by a number of types of pile. In order of increasing load capacity: precast normally reinforced segmental concrete piles, driven cast in place enlarged base piles, prestressed concrete driven piles and steel H sections.

Larger ultimate design loads will usually require steel tube piles, composite piles or large diameter bored or CFA piles.

Large tension or lateral loads will require special consideration. Tension piles with an enlarged base (with adequate reinforcement) can provide significant uplift resistance, as can bored or CFA piles socketed into rock. In hard rock consideration can be given to the use of passive or active rock anchors to develop uplift resistance.

The potential for negative skin friction to develop caused by the settling of soft soils around the piles should be considered where there the site sub surface profile has deep deposits of soft normally consolidated soils which may be subject to creep, or where additional loads (such as embankment fills) are to be imposed on soft soils in proximity to piles.

Negative skin friction can increase the working load in piles, or can create additional pile settlements at working load. This may necessitate the use of larger capacity piles capable of resisting additional axial or in some cases lateral load due to horizontal movements under embankments built on soft soil.

For driven piles, the piles can be coated with a bitumen slip coating that can reduce the magnitude of down drag loads significantly. Raked piles are generally not suitable where the surrounding soil is settling, due to the bending that is introduced to the piles.

7.4.3 Site Conditions

The physical conditions at the site can influence the choice of pile type in a number of ways.

Site access for piling plant can be a major factor. It may not be possible to get a large piling rig into position (e.g. a steep creek crossing), or there may be restricted headroom (especially important when widening or strengthening an existing structure), requiring the use of short lengths of pile jacked into place, or a small bored-pile rig.

The site itself may be distant from Perth and so the transport costs of precast concrete piles may become excessive as compared to steel piles which may be more economical.

The presence and type of any adjacent structures may also have a bearing on the choice of pile type.

Installation, particularly of uncased bored piles, close to an existing structure may reduce the available capacity of the existing structure’s piles if they develop significant capacity from shaft friction.

Driven piles may not be feasible in built up areas or close to vibration sensitive structures, because of possible vibration and noise issues. Steps such as pre-boring can reduce ground vibrations and newer hydraulic piling hammers come with noise reduction shielding.

7.4.4 Structure Type

The main influence of the structure type comes from the stiffness of the structure, plus any aesthetic considerations. A continuous structure, especially if it has a deep stiff deck and/or relatively short spans, can tolerate little differential settlement across or between piers. Relatively stiff piles installed to a hard bearing stratum reduce the risk of large differential settlements. Alternatively, a simply supported structure is more tolerant of differential settlements.

The proposed pile design should consider not only pile capacity but also vertical and lateral deformation at working loads. Due allowances must be made in the design of the structure for the anticipated differential settlements.

Aesthetic considerations may rule out some pile types, e.g. concrete or steel piles extending to the underside of cross head above ground level would only be used as combined pile/columns in the country and other less publicly visible areas.

7.4.5 Assessment of Pile Type

When considering the influence of the above factors, the first major choice will be between bored non displacement and driven piles. In WA, with our mainly non-cohesive soils, driven piles are by far the most common choice for small and medium span bridges, except for the special uses of Frankipiles, or bored piles in the North-West. Bored piles may be a good option for structures in remote and inaccessible sites, as lightweight, truck-mounted rigs can be used provided hard rock drilling is not required.

With bored piles there are a number of specific issues which could affect their capacity and these must be taken into consideration in design, specification and construction. Integrity of the shaft is a key issue. If there is any doubt about the ability of the ground to stand up uncased, then permanent or temporary casing or consideration of using CFA piles (if appropriate) should be specified. Inspection of boring and grouting records is essential and pile integrity tests are valuable in these circumstances.

Special measures must be taken when placing and compacting concrete in pile shafts. Tremied concrete, provided as a self compacting high slump, high cement content mix, can provide reliable results provided it can be sourced from an experienced reputable concrete supplier.

Another potential problem, is contamination of the base of bored piles. It is very difficult to clean the base, and then to inspect to prove it is clean. Often the use of purpose built cleaning tools is required and inspection by a suitably qualified geotechnical engineer may be required to verify satisfactory base cleaning.

Contamination of the base of a bored pile with cuttings, or the presence of disturbed soil or un-displaced drilling fluid, can result in excessive displacement of the pile before its full design capacity is mobilised, particularly for piles where a majority of the load is to be developed by end bearing. This problem is reduced if CFA piles are used where concrete or grout is injected under pressure to the pile base before the CFA auger withdrawal commences.

If driven piles are to be used then the next consideration is whether they will develop their design load capacity in shaft friction, end bearing or a combination of both. Shallow friction piles in sands with low/medium loads would permit the use of PSC piles, which are usually cheaper than a steel alternative. Longer piles would require segmental concrete piles with suitable joints, longer prestressed piles or spliced steel H piles. Driven steel tube piling is usually necessary for higher loads and if hard driving is likely to be required.

A final possibility is the use of composite piles, where the high vertical load capacity, penetrating ability and ease of extension of a lower steel pile section is combined with the stiffness and corrosion resistance of an upper concrete section. These would tend to be used for larger structures over water.

Large structures may require a special type of piling. e.g. the concrete cylinders surrounded by articulated caissons at the Narrows Interchange, but by far the majority of structures in WA will use some form of medium capacity driven pile.

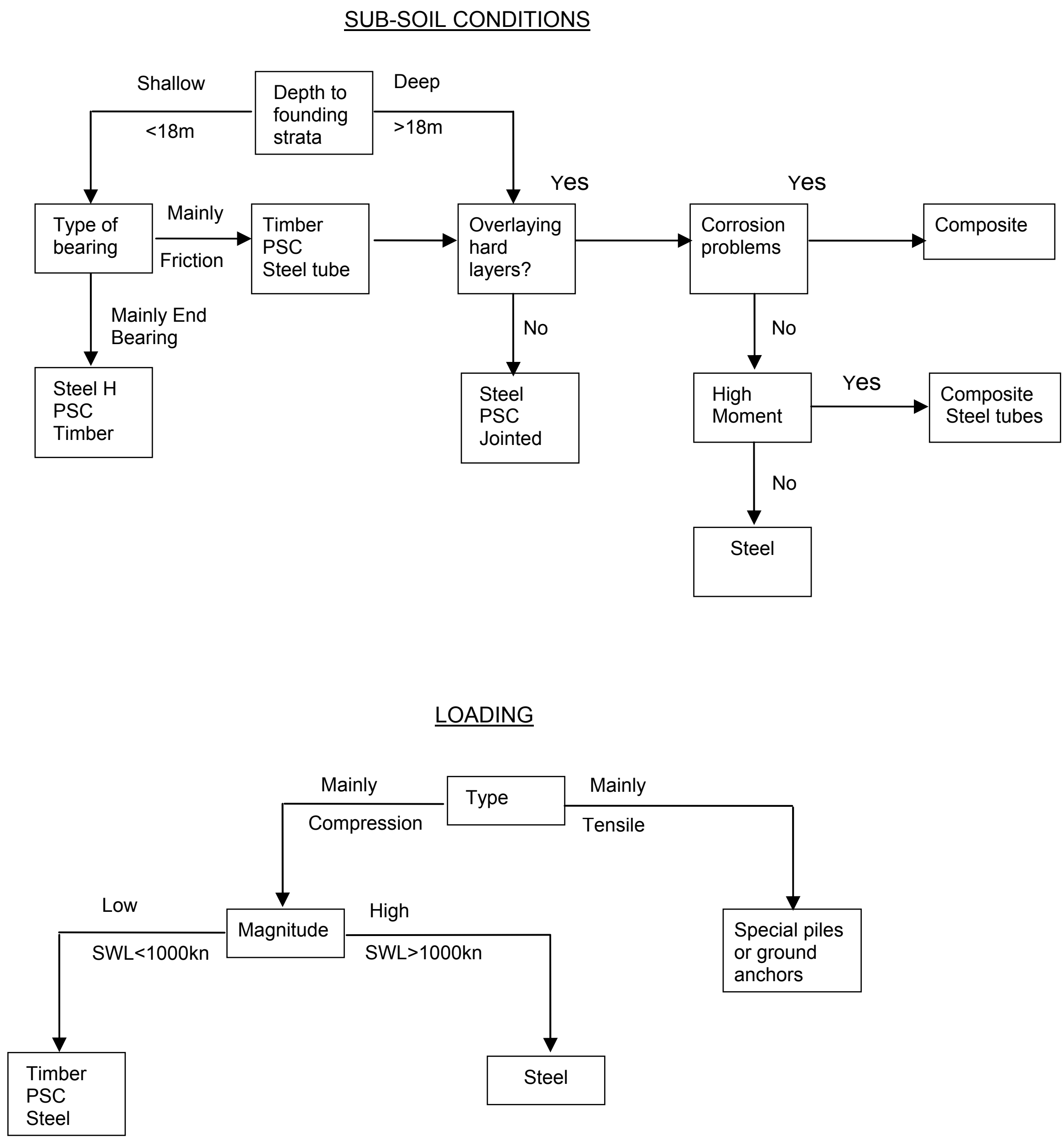
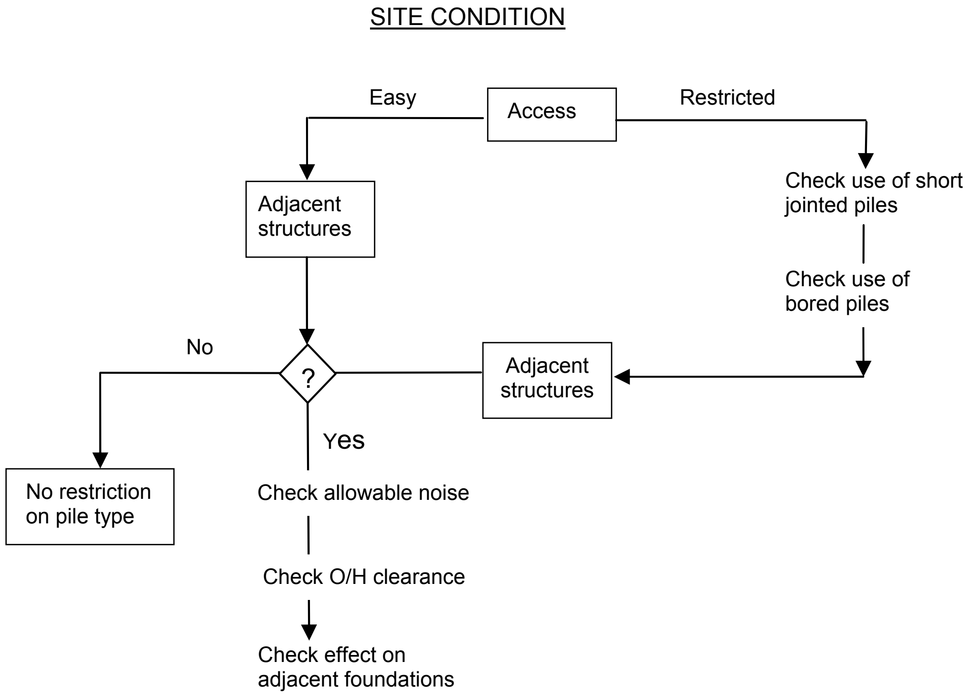


Figure 7.3 – SELECTION OF A PILE TYPE



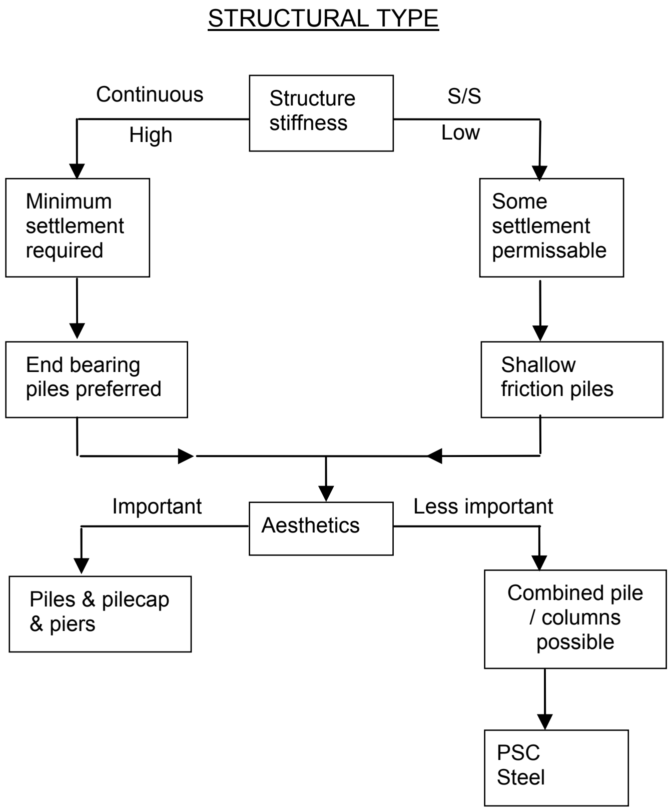


Figure 7.3a – SELECTION OF A PILE TYPE (CONT)

7.5 ULTIMATE LOAD CAPACITY OF A SINGLE PILE

7.5.1 General

The calculation of the design ultimate geotechnical strength of a single, isolated pile is probably the most important step to be taken in assessing the likely performance of piled foundations. Although piles are nearly always used in groups the calculation of the ultimate geotechnical strength of a single pile is a necessary prerequisite for estimating group performance. Also, although other factors such as settlement and lateral capacity are important in foundation analysis, vertical load capacity is usually the most important consideration for piled foundations.

In this context the design ultimate geotechnical strength is the maximum load that a single pile can carry or the load which generates a pile head settlement of the order of 50 mm. At this load all of the available pile soil resistance is fully mobilised. The pile is incapable of supporting more load and settlement.

AS 5100 Part 3, Clause 11.3) requires the ultimate geotechnical design strength of piles and the appropriate geotechnical strength reduction factor to be in accordance with AS 2159.

The ultimate design geotechnical strength of a single pile is the sum of the soil friction on the pile shaft and the resistance on the base of the pile. The basic pile capacity equation is as follows:

$$R_{ug} = \bar{f}_s A_s + f_b A_b \qquad 7.5.1$$

The background and derivation of this expression is presented in the nominated references.

7.5.2 Application of Geotechnical Strength Reduction Factors (AS 2159 Clause 4)

When calculating the pile ultimate limit state capacity it is necessary to apply strength reduction factors as in normal ultimate analysis. The differences between structural limit state design and geotechnical limit state design are:-

Structural:

1) Calculate the ultimate strength of the section (R_u) for the required action (bending, shear etc), using specified dimensions and characteristic material properties.
2) Factor this by the Structural Strength Reduction Factor (Φ_s) appropriate to that action to determine the design strength ($\Phi_s R_u$).

Geotechnical:

1) Calculate the ultimate geotechnical strength of the pile ($R_{d,ug}$) for the required action (bending, shear etc), for the specified pile type and using the design geotechnical parameters.
2) Factor this by the Geotechnical Strength Reduction Factor (Φ_g) appropriate to that action to determine the design geotechnical strength ($\Phi_g R_{d,ug}$). AS 2159 provides guidance on the appropriate value of Φ_g for various situations.

7.5.3 Piles to or into Rock

These are not commonly used in Perth, but AS 2159 Clause 4.4.2 gives guidance where necessary. Where they have been used around the Metropolitan area the founding strata is a siltstone or limestone, which is classified as a weak rock. Bored, rock socketed piles have been used in the North-West of the state. The degree to which these rocks are weathered controls the settlement of the piles. Settlement can occur in sockets drilled into these weathered rocks.

Bored pile rock socket capacities are often governed more by the expected settlement at serviceability load rather than by the ultimate geotechnical strength, particularly for end-bearing piles.

Where serviceability settlement is to be estimated, rock mass stiffness and strength are important design parameters. Where ultimate geotechnical strength is required, rock mass strength is the most important design parameter.

Bored pile rock socket design is often based on unconfined compression strength (UCS) tests carried out on diamond drill core samples. Point load index testing (Is_{50}) on rock core may be correlated to UCS test results to provide additional information. Refer AS 4133.4.1 and AS 4133.4.2.

Care must be exercised when assessing core samples that the sample is representative of the rock, especially if the material *in situ* is fractured. If a better assessment of *in situ* strength or modulus is required, consideration may be given to carrying out *in situ* testing in boreholes such as pressuremeter testing in relatively weak rock.

7.5.4 Other Considerations

The preceding gives details of how to calculate the ultimate vertical load capacity of a single pile. There are also a number of other vertical load effects although they are usually less important, e.g. design uplift capacity and, in relatively soft soils, potential negative friction and pile buckling. It is usually only necessary to calculate these for a minority of structures and because of this, plus the fact that they are covered in the AS 2159 (Clause 4), they will only be briefly mentioned here.

a) Uplift

For a tension pile without an enlarged base, the uplift capacity is simply the sum of the pile shaft friction, i.e. the average shaft adhesion multiplied by the shaft surface area, plus the pile weight.

For piles in clay the adhesion is usually taken as being the same as for downwards loading.

For piles in sand or weak rock close to the surface, a check should be made not only on shaft friction but also on cone pullout where instead of the pile shaft pulling out of the ground, the shaft plus a cone of soil lifts out.

For piles with enlarged bases assuming the pile itself has sufficient structural strength, then failure can occur in two ways, either as a straight pull out of the pile shaft or as a cone pullout extending upwards from the enlarged base as presented in Figure 7.4.

Both failure modes should be checked and the minimum adopted for design. This can be particularly important for pile groups where consideration should be given to the pull out failure of a block of soil encompassing the pile group occurring. Guidance for uplift design is provided by Poulos and Davis (1980) and by Tomlinson (1994).

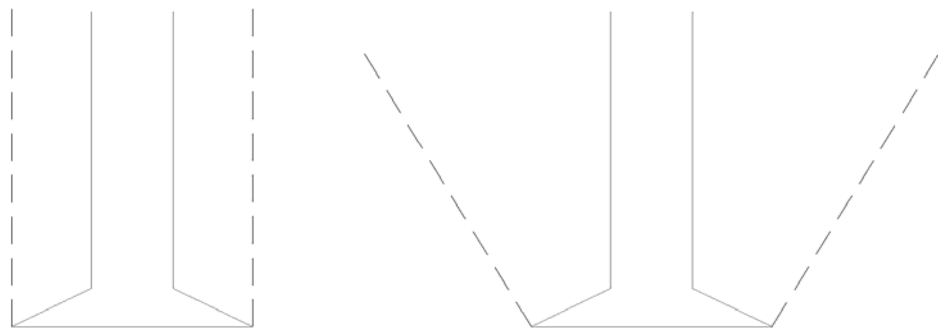


Figure 7.4 – UNDER-REAMED TENSION PILE – MODES OF FAILURE

b) Negative Friction

This can occur typically when an end bearing pile is driven through soft clay to a hard bearing stratum. Consolidation of the clay, especially at the abutments where it is loaded with embankment fill, can induce additional axial load in the pile which can be quite substantial.

The additional load is termed negative skin friction or down drag. This load does not affect the ultimate geotechnical capacity of the pile, but it can have a serious effect on pile settlement at serviceability load (Poulos 2008). The pile also needs to have sufficient structural strength to resist not only the applied design load but the additional down drag load induced into the pile shaft as well.

The magnitude of potential down drag forces are best estimated by carrying out a soil structure interaction analysis. The computer program RATZ (Randolph 1986) allows for such an analysis for a single pile where an estimated vertical ground settlement profile over depth can be input and pile settlement under service loads plus the effects of down drag can be assessed. The piling code (AS 2159) also presents approximate methods that can be used to assess down drag loads in the absence of a soil structure analysis.

If the additional down drag load presents a problem it is possible to reduce the magnitude of down drag for driven preformed piles by the application of a bituminous slip coating to that part of the pile shaft that lies within the soft soil. Additional information on the efficiency of bituminous coatings has been provided by Bjerrum et al (1969) and Walker and Darvall (1973).

It may be possible to delay pile installation under abutment fills until consolidation settlement under the added weight of the fill is complete. It may also be possible to isolate piles from downdrag soil movement, driving them inside a steel or concrete shell which effectively isolates the load carrying pile from the moving soil.

Another potential problem that may occur in piles installed through abutment fill constructed on soft clay or weak ground is associated with horizontal movement of the soil beneath the embankment. This could affect abutment piles that are installed whilst vertical and lateral movement of soft soil beneath abutment fills is still occurring. This is a difficult problem and requires a high level soil structure interaction analysis to assess in any detail.

If it is proposed to install abutment piles in this situation, consideration should be given to assessing potential effects on abutment piles. This may include:

- Carrying out a high level soil structure analysis to quantify the problem.

- Using some form of ground improvement to limit expected vertical and horizontal soil movement beneath the abutment fill to acceptable levels.
- Adopting pile types that have a significant inherent bending capacity such as concrete filled steel tubes or large diameter bored cast in place piles.
- Consider construction of the embankment using light weight fill to limit vertical and horizontal settlement.
- Consider installing the bridge abutment piles in casings which separate the piles from lateral soil movements providing the piles can act as unsupported columns over the sleeved depth.

c) Buckling

Buckling of slender steel or precast concrete piles can be an issue if the piles have long unsupported lengths above ground level. In these cases the pile shafts need to be designed as structural columns over the unsupported length of the piles.

It is also theoretically possible for heavily loaded, slender steel H or circular section piles to potentially buckle if installed in very soft soils. This is generally very unlikely to be an issue, as only a very small lateral pile-soil resistance is required to prevent buckling unless low moment capacity pile joints are present close to the ground surface. Guidance for the assessment of pile buckling is presented by Poulos and Davis, 1980.

7.6 EMPIRICAL METHODS OF ULTIMATE LOAD CAPACITY ESTIMATION

7.6.1 General

As well as the theoretical approaches to single pile ultimate load capacity calculation detailed above, there are also a number of empirical methods available. These range from the purely empirical, based mainly on local knowledge, to pseudo-scientific methods based on empirical observations.

Examples of the former are the knowledge that exists in MRWA on the performance of common types of pile, e.g. 350 mm square PSC concrete, 310UC steel, in local ground conditions. For these situations, an experienced engineer will be able to assess the reports of boreholes and from the SPT results, make an estimation of the depth to which a pile may be driven to develop a required design strength.

For instance, concrete piles will not be able to be driven very far into material with SPT values of around 60 and above or below where cone penetrometer cone resistances in excess of 50 MPa are measured, providing the soils are not calcareous in nature.

Caution is, however, required to ensure that the material beneath the founding layer is sound, particularly where pile groups are concerned as whilst individual piles may be driven to the required capacity, the load applied by the pile group to the soil below the founding layer may be significant and the capacity of the pile group may be governed by block failure rather than the sum of individual pile capacities.

The semi-scientific methods of pile geotechnical strength assessment are all based on established correlations with standard soils tests such as the Standard Penetration Test (SPT), the Cone Penetrometer Test (CPT), and to a lesser extent in Australia, the pressuremeter test (PMT).

7.6.2 Standard Penetration Test (SPT)

The SPT test is perhaps the most commonly used *in situ* test. Numerous empirical correlations between SPT N value and shaft friction and end bearing have been developed.

These correlations take the form:

Pile shaft friction: $f_s = A_N + B_N N$ (kPa) 7.6.1

Pile end bearing: $f_b = C_N N$ (MPa) 7.6.2

The most widely used empirical method is that presented by Meyerhof (1956) for driven piles in sand with $A_N = 0$, and $B_N = 2$ for displacement piles and 1 for small displacement piles and C_N=0.3. Limiting values of f_s of 100 kPa for displacement piles and 50 kPa for small displacement piles and about 12 MPa to 15 MPa end bearing in dense sands have been recommended. Poulos (1989) has summarised a number of other empirical correlations for various soil types.

7.6.3 Cone Penetrometer Testing (CPT)

Use of CPT data offers the chance of improving estimates of pile axial capacity as this test measures shaft friction and end resistance directly and separately and provides a continuous profile of soil resistance which is much less sensitive to drilling operations.

Full details of the use of the instrument and interpretation of results are given in Chapter 5 of this Manual but a brief summary of the main points relevant to pile design are given below.

Bustamante and Gianeselli (1982) have provided a comprehensive review of the estimation of pile capacity for a variety of pile types based on CPT results. Their review is based on the results of 197 static load tests on 48 sites comprising a variety of soil types. Their correlation takes the form:

Pile shaft friction: $f_s = q_c/\alpha$ (kPa) 7.6.3

Pile end bearing: $f_b = q_c k_c$ (MPa) 7.6.4

Where q_c is the cone point resistance. Values of α and k_c vary with soil cone resistance and pile type.

For the assessment of end bearing an average value of cone resistance over a depth ranging from 1.5 pile diameters above and below the pile base is used. Cone resistance values higher than 1.3 times and lower than 0.7 times the cone resistance under the pile toe are eliminated from the calculation of the average cone resistance. Additional details are presented in the reference noted above.

Typical α and k_c values for driven displacement piles are presented in Table 1.

Table 1: CPT Correlations for Driven Displacement Piles after Bustamante & Gianeselli

Soil Type	Cone Resistance (MPa)	α	Limit shaft friction (kPa)	k_c
Silt and loose sand	< 5	60	35	0.5
Medium dense sand	5 to 12	100	80	0.5
Dense sand	>12	150	120	0.4

For stiff to hard cohesive soils, the cone point resistance (q_c) can be used to obtain the undrained cohesion, c_u, from the relationship: -

$c_u = q_c/N_c$ 7.6.5

With the value of N_c normally in the range 15 to 20. The assessed value of c_u can then be used to estimate the appropriate value of pile shaft adhesion (f_s) and pile end bearing resistance (f_b) in the usual static bearing capacity formula (refer Section 7.5.1). Reference to Fleming et al 1992 provides relationships between f_s and c_u. For end bearing capacity N_c is in the range of 6 to 9 depending on depth of penetration into the founding layer.

7.6.4 Pressuremeter Test (PMT)

The results of pressuremeter tests usually require expert interpretation. A correctly executed pressuremeter test can provide information on soil or rock *in situ* modulus and strength.

Various empirical methods have been developed which relate the pressuremeter limit pressure to ultimate pile base and shaft resistance. These are summarised by Clarke (1995) who describes the latest recommended method developed in France (LCPC-SETRA, 1985) which provides guidance for the assessment of end bearing and shaft resistance for a variety of soil types for both bored and small displacement piles and full displacement piles.

7.7 PILE GROUP ANALYSIS

7.7.1 Introduction

Piles in bridge foundations are almost invariably used in groups rather than as isolated piles. Therefore, the calculation of pile group capacities, for both vertical and horizontal loads is a more important consideration than the behaviour of single piles.

It is, however, necessary to first calculate single pile behaviour as pile group behaviour is estimated from these. It must be stressed that in nearly all cases, the group capacity and deflections will differ from those of the individual piles making up the group. In the majority of cases, the geotechnical ultimate capacity of the group will be less than the sum of the individual pile capacities and the deflections of the pile group will be greater than the deflection of a single pile carrying the average load of a pile within the group. Where the pile spacing is greater than about 3 pile diameters, the geotechnical strength of the pile group may not be reduced significantly due to interaction effects, however, settlement of the pile group should always consider interaction effects.

Another thing that must be borne in mind at all times with foundation calculations, and especially with pile groups, is that the answers obtained will not be exact. No matter how scientific the method of analysis may seem, the output can only be as good as the input to the analysis, and in the case of piles and pile groups in particular, this input must, because of the nature of the problem, be approximate only. For example: -

- ***Soil Properties*** - can never be known exactly. Only a very small proportion of the soil on a site will be tested and even that may not indicate the actual *in situ* properties of the soil or rock at the site as it will probably be affected by the manner of sampling and testing and may behave differently under *in situ* load. Also, the soil will probably be anisotropic, although for simplicity in analysis it is usually assumed to be isotropic.
- ***Pile Positions and Rakes*** - will not be known accurately as there is always some variation in the position and dimensional tolerance that can occur during pile installation.
- ***Soil Interaction*** - there will probably be some soil contact with the soffit and sides of the pile caps, but this is often not taken into consideration during pile group analysis.

Notwithstanding these qualifications and the inherent complication of the 3-dimensional pile system which has to be analysed, there are a number of methods available which seem to provide adequate, if perhaps somewhat conservative estimate of the loads and deflections in pile groups. These methods have provided results which indicate that they are suitable and acceptable for pile design for bridges.

7.7.2 Analysis Methods

The usual approach, having estimated the ultimate capacity of a typical single pile is to then analyse a "guesstimated" pile group under general 3-dimensional loading to check that the individual pile capacities are not exceeded, in either compression or tension and that group deflections under serviceability loads are acceptable. Methods available for the analysis of pile groups are outlined below. Further details of various methods are summarised by Poulos and Davis (1980), Fleming et. al. (1992) and Tomlinson (1994)

- **Simple Static Methods**

These are very simple, usually 2-dimensional only “bolt group” type analyses which ignore the presence of the soil and assume the piles are pinned to the pile cap so that the structure can be analysed as statically determinant. Pile–soil interaction effects where the lateral resistance of the soil is applied to the pile are ignored.

Graphical or simple spread sheet methods of solution are also available.

The use of this method is not recommended except for use as a rough first pass at estimating the number of piles likely to be required within a group.

- **Equivalent Bent Method**

This takes the above method one stage further. It assumes a somewhat more complex equivalent frame that can be 3-dimensional and can have piles built into the pile cap. This can then be analysed on a standard structural frame analysis program, e.g. ACES, Microstran, etc.

Soil properties are modelled to some extent by modification to the lengths and stiffnesses of the actual piles when deriving the equivalent piles for the model, e.g. by assuming a depth to fixity for the piles but the methods are not exact. Pile group interaction effects are not included and estimates of soil spring stiffness are required.

- **Elastic Analysis Methods**

These methods use elastic theory for the effects on the individual pile and the interactions between the piles themselves and the pile and the soil. They are a great improvement on any of the above methods as they can be made to allow for load effects in 3 dimensions and for pile/soil interaction. They are computer based because of the extensive computation required.

MRWA uses the PIGLET (Randolph, 1986) program which can analyse full 3-dimensional loading for either a rigid or flexible pile cap using a soil stiffness profile which can be varied with depth.

Other commercially available programs include Repute (www.geocentrix.co.uk) and DEFPIG (Sydney University Geotechnical Application and Research at www.civil.usyd.edu.au).

- **Finite Element Analysis**

This is not often used because of the complexity and size of the models required. Recent developments of the PLAXIS suite of 2D and 3D geotechnical computer programs (www.plaxis.nl) now provide a user friendly geotechnical finite element analysis package that can be used to assess a variety of pile soil interaction problems where the complexity of the problem justifies the use of this approach.

Three dimensional finite analyses are complicated and time consuming to run so where possible it is better to use 2D analyses if at all possible. Correct analyses require good estimates of soil properties to provide meaningful results.

Having assessed the general loading behaviour and deflections of the proposed pile group, it is important to realise that the structural and geotechnical design strengths of single piles within the pile group calculated previously may require modification to account for interaction between the piles before the pile group design can be finalised.

7.7.3 Group Effects

As stated earlier, a group of piles behaves differently to a single pile in the same situation. This "group effect" is different for each design action.

- **Ultimate Load**

In cohesive soils, piles driven close together can generate high pore pressures (with accompanying reduction in the effective strength of the surrounding clay soil) which may take a long time to dissipate, especially from the centre of the group (perhaps a matter of years).

The ultimate vertical capacity of the pile group can be considerably less than the sum of the capacities of the individual piles. In this case, it is usual to take the group ultimate load capacity as the lesser of the sum of the individual pile capacities or the capacity of the whole foundation block which encompasses the pile group. This is covered in AS 2159 Clause 4.4.4.

In cohesionless soils, driving groups of piles at close centres can result in increases in relative density and therefore increased lateral pressures in the sand, which can increase pile skin friction and thus pile capacity. However, it is usual to take a conservative view and assume the group capacity will be represented as the sum of the individual pile capacities. For bored piles in either material, there may be a reduction in capacity for the group due to the relaxation of lateral stress as a result of the method of installation.

- **Group Settlement**

There are a number of alternate approaches available to estimate pile group settlement as outlined above in Section 7.7.2.

The settlement of a pile group is usually linked to the settlement of a single pile under the same average load as a pile in the group by a group settlement ratio (R_S). Poulos and Davis (1980) present charts and elastic solutions that allow the estimation of group settlement for isotropic elastic conditions. R_S is dependant upon the length: diameter ratio of the piles, (or equivalent diameter in the case of non-circular piles), the Poisson's ratio of the soil, the variation in Young's modulus of the soil, and a relative stiffness parameter (K_{rb}), linking the pile and soil stiffnesses.

Alternatively pile group settlement estimates can be obtained directly from the output of elastic analysis programs such as PIGLET, Repute and DEFPIG.

- **Group Lateral Capacity**

There are two possible modes for lateral failure of a pile group. If the piles are widely spaced (at least 3D to 4D), they are unlikely to interfere with one another and so the ultimate lateral capacity will be the sum of the lateral capacity of the individual piles within the group. If the piles are closely spaced, then the group capacity will be that of an equivalent pier containing the piles and the soil. The lesser of the two values should be used. In the second case, only short-pile failure, (see Section 7.9), need be considered, the idealised "pier" being unlikely to fail "structurally". See AS 2159 Clause 4.4.7.

Lateral capacity of individual piles can be assessed using elastic methods (Poulos and Davis, 1980); limit state equilibrium methods such as those proposed by Broms (1965) or so called p-y methods such as those presented by Reese et al (2006).

- **Lateral Deflection**

There will usually be increases in pile deflections and rotations as a result of group action. There are a number of possible approaches to this calculation, but the easiest is that of Poulos using elastic theory. He has tabulated group deflection and rotation ratios which are used in a similar manner to the group settlement ratio for vertical settlement (see Poulos, 1979).

7.7.4 Summary

Although the above may seem complex, it must be remembered that the program PIGLET makes allowance for the interactions between the piles in a group and so all that is necessary, (assuming the correct loads, soil properties, etc, are input), is to check the maximum and minimum pile loads from the program output against those acceptable for a single pile and deflections and rotations against whatever is judged to be acceptable for the particular structure.

7.8 SETTLEMENT OF PILES

Whereas the pile capacity calculations presented above are an ultimate limit state check, this section deals with the estimation of the settlement of piles and is therefore essentially a serviceability check. Again only single pile settlements are presented here; settlement of pile groups has been discussed above.

The differential settlement of the supports of a bridge can be an important load case, depending on the properties of the structure. For continuous decks, especially those with stiff cross-sections and short spans, it will be more important than for simply supported or more flexible structures. It is normal to think of piled foundations as rigid and assume there will be no settlement of the supports. However, this will only be really true for short, end-bearing piles to a rigid sub-strata. For other types and especially friction piles, there will be the chance of some settlement, although this will of course usually be much less than for spread footings. There is also the elastic deflection of the piles themselves to consider. This could be quite large for long piles, but if the piles are all of similar length it will tend to be the same for all supports, so will produce little structural distress. In WA, all structures on piled foundations should be designed for a minimum differential settlement of 5 mm and then, as long as the vertical capacity is adequate, it will usually be unnecessary to specifically check pile settlement. However, in certain instances, calculation may be required.

The subject of pile settlement is complex. For a full treatment refer to Poulos and Davis, 1980, Chapter 5. There are a variety of methods available for pile settlement calculations, but the most popular is elastic theory, for which there are tables and charts available to assist in calculations. A few such graphs are given in Poulos and Davis, (1980), or other publications referred to in the AS 2159 Commentary.

The simplest elastic solutions consider an incompressible pile in an infinite half space with a completely homogeneous soil having a constant Young's modulus (E_s), and Poisson's ratio, and assume there is no slippage at the pile/soil interface. Also elastic theory normally only calculates the immediate, elastic settlement of the pile, it does not readily provide information on any long-term consolidation settlement that may occur. However, these approximations are usually acceptable. A greater problem which can have a profound effect on the answer obtained is the correct choice of E_s. The triaxial test was used to derive values, but these resulted in gross overestimates of pile settlement. The only reliable way is to carry out an *in situ* test, loading a test pile, plotting the load settlement relationship and then back calculating the soil modulus. It is not necessary for the test pile to be the same cross-section as that to be used in the final structure, but it must be the same length so that it goes through the same soil layers. Failing this, there are a number of empirical correlations available, refer to AS 2159 Commentary.

7.9 LATERAL LOADING OF PILES

7.9.1 Introduction

Lateral loading on piles has to be checked for two limit states as for vertical loading; the ultimate limit state of lateral load capacity and the serviceability limit state for lateral deflection. These are covered in AS 2159 in Clause 4.4.7.

Lateral load capacity is usually less important than vertical load capacity and it tends to be treated in a similar manner to vertical pile displacements and often not evaluated specifically. This is because the lateral loads on a pile group are, in the majority of cases, considerably less than the vertical loads and are usually catered for by raking some of the outer piles in the group and then checking the overall group capacity to ensure that no single pile is overloaded. Horizontal deformations can be checked in a similar manner, but are rarely critical, except in the special case of ship impact for bridges over navigable waters. They are usually only significant at the abutments, specifically the fixed abutment, and this will have extra raking piles to carry the loads. In addition, the abutments are usually surrounded by fill, whose restraining effect is not usually allowed for in the analysis.

7.9.2 Lateral Resistance

There are two possible modes of failure for a pile subject to static lateral loading, refer Figure 7.5. These are: -

- ***Short-pile Failure*** - where the pile rotates bodily about some point along its length and reaches its maximum lateral capacity when the soil fails.
- ***Long-pile Failure*** - where failure is by yielding of the pile itself. The pile breaks before the soil fails.

In the case of a long pile, plastic structural pile failure will occur at some depth down the pile and only the upper part of the pile will experience significant displacement. See Poulos and Davis, 1980, for formulae and charts for the cases of a free-head pile and fixed-head pile lateral loading condition.

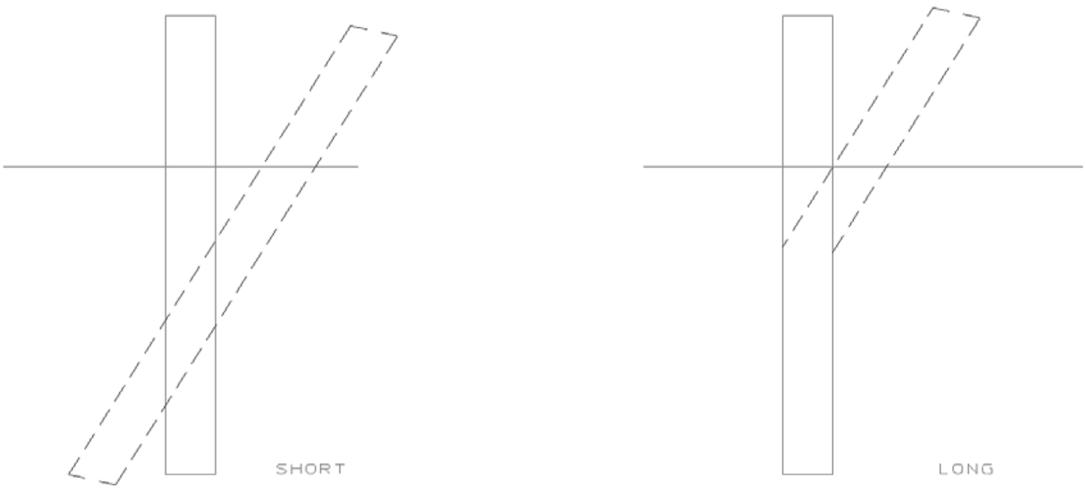


Figure 7.5 – MODES OF LATERAL PILE FAILURE

7.9.3 Lateral Deflection

This may be estimated from the formulae references given in AS 2159 Commentary. Care is needed because the calculated deflection is again depends on the value of soil modulus (E_s), used, although it is not as sensitive to this as is axial settlement. The modulus for lateral deflections will usually not be the same as that for vertical loading. The best way of estimating the modulus is via a lateral load test, which should be carried out on a pile of the same length, if not the same diameter, as is to be used in the structure. However, these are carried out infrequently as lateral deflection is rarely a critical design case.

Pressuremeter tests may also give a reasonable estimate of E_s, but if all else fails, it will be necessary to use one of the available empirical relationships. For clays, the long term tangent modulus may be taken as about $200c_u$, For sands, E_s can be estimated from SPT results, as approximately 1.6 N.

7.10 STRUCTURAL DESIGN OF PILES AND PILE CAPS

7.10.1 Introduction

The structural design of piles and pile caps is sufficiently different to normal reinforced concrete design theory to warrant some separate consideration. The structural design of the different types of piles is covered fairly extensively in AS 5100 Part 3 Clause 11.4 and Commentary. Pile caps are covered in the concrete section in AS 5100 at Part 5, Clause 12.1, with additional comments given in the commentary to this section in AS 3600.

7.10.2 Piles

Piles have to be designed for the bending and tensile stresses likely to be generated during handling and driving, as well as for the imposed moments in service.

Handling stresses are fairly easy to calculate, given assumed lifting points on the piles. Allowance should always be made for impact loading during handling, and if handling stresses are critical, (usually only in concrete piles), the design lifting points clearly identified.

Driving stresses are somewhat harder to quantify. They can be calculated by wave equation analysis, but are dependant on assumptions made about a number of variables, including the dolly/pile helmet/cushion combination, and so may not be that reliable. Allowable stresses during driving are given in the Commentary to AS 5100, Part 3 Clause C11.4, but since these are instantaneous stresses they have in the past been interpreted quite liberally. In some cases, wave equation analysis has indicated pile compressive stresses above yield, but there have been no obvious signs of distress during driving. The only qualification to the above is that great care should be taken to eliminate significant tensions from concrete piles. These are a definite danger as they can result in piles cracking during driving. Although the prestress in the pile will usually close the cracks, they could still be a corrosion hazard. The main way of controlling pile stresses occurring during driving is by controlling the hammer energy and thus limiting the pile penetration per blow. The specification of the properties of the driving cushion between the helmet and the pile head can also have a significant effect. Timber packing is usually used and this gradually deteriorates as driving progresses, so that in some cases it may be necessary to specify that it be renewed at stages during the pile driving, especially near the end if driving is hard.

There is also the real possibility of damage to the pile head during driving, damage that can be so severe that further driving is not possible until the head is repaired, a long process with concrete piles. It is difficult to deal with this by normal methods of analysis and it is more a case of experience, correct detailing and frequent changing of pile top packing.

Points to be considered with the different types of pile are:-

- ***Timber:*** Very resilient, but care is necessary with the handling of long piles. Heads should be protected to prevent splitting during driving; a steel driving band is usually used.
- ***Steel:*** Again, usually not a problem. Heads and toes may require stiffening if driving is hard. With tubular piles driven open-ended, any stiffening ring should be on the inside, because if on the outside it will lead to a reduction in skin frictional resistance. However, care will be required if cleaning out inside the tube to ensure that the grab does not go past the toe and get stuck.
- ***Concrete:*** Can be a problem with both handling and driving stresses. High handling stresses can usually be overcome by the specification of the correct lifting techniques and pile reinforcement and prestress, and the provision of lifting hooks at these points. Failure of the head is more of a problem. Adequate stirrups must be provided and in addition, a driving band may be required if driving is likely to be hard, see Figures 7.7 and 7.9.

7.10.3 Pile Caps

Because of their dimensions, pile caps must usually be designed as deep beams and not by normal bending theory. However, to apply truss analogy to a large pile cap, supporting say two or three columns and with 30 or more piles underneath, becomes very difficult. In such circumstances, the best approach is to analyse the group for the critical load cases and obtain the loads in the piles, then consider elevations or sections of the pile cap and group together numbers of piles on that elevation or section and then derive an analogous truss. This may seem a simplification, but given the complexity of the actual structure it is an easy and safe way of dealing with it. Ultimate loads must be used for this analysis to suit the design methods of Part 5 of AS 5100.

Do not be put off if the analysis shows that seemingly large quantities of reinforcement are required. Two or three layers of 32 mm diameter bars may appear a lot, but because of the large size of most pile caps, the actual steel % will usually be found to be low. Also, do not forget the top steel. If there are two or more columns on the cap, this can be quite heavy too.

7.10.4 Pile/Pile Cap Connection

Finally, there is the pile to pile cap connection to consider. This is essential, in order to transfer the load from the pile into the cap. The type of connection will depend on the type of pile and whether full moment continuity is required or just transfer of axial load.

- ***Timber Piles:*** are most often used in timber bridge construction and standard bolted connections will be used. If they are used in composite construction, going straight into a concrete deck, then a nominal inset into the concrete of say 50 mm to the bottom steel is usually all that is needed, to transfer axial load.
- ***Steel Piles:*** are used both with pile caps and directly into concrete crossheads or decks. Moment transfer is usually required and is obtained by welding adequate reinforcing bars to the pile and then continuing them into the pile cap or deck. In this case, the pile itself will only penetrate as far as the bottom layer of reinforcing steel. It is also possible with pile caps to omit the reinforcing bars and instead extend the pile into the cap to obtain continuity. This, however, results in more disruption of the bottom reinforcement in the pile cap.

  A check should also be made for punching type failure, especially with steel piles cast directly into a relatively thin RC deck slab. In cases where this is a problem, a capping plate may be required.
- ***Concrete Piles**:* are used in similar situations to steel. In this case, continuity is obtained by stripping back the concrete above pile cut-off level to expose the reinforcing bars and extending these and the prestressing strands into the cap. Again, the pile itself will only penetrate a nominal 50 mm into the cap.

7.11 ULTIMATE LOAD CAPACITY OF SINGLE PILES FROM DYNAMIC ANALYSIS

The ultimate load capacity of a driven pile can also be estimated by the use of dynamic pile formulae. There are a number of approaches, all based on the same two assumptions, which unfortunately are not fully valid:-

- The resistance to driving of a pile can be determined from the kinetic energy of the driving hammer and the movement of the pile under the blow; and
- The resistance to driving is equal to the ultimate bearing capacity of the pile for static loads.

A large number of driving formulae have been developed over the years and Poulos and Davis, 1980 give details of eight of the most widely used ones, with comments on their accuracy. Basically, none are very reliable and the only one that is used by MRWA with any degree of confidence is the Hiley formula. Details can be found in the above reference, but the formula should only be used for relatively short piles, less than 12 metres long and even then with caution. As it requires the actual hammer efficiency and the temporary compressions of both the pile and the ground it is most appropriate for checking capacity in the field, when these factors can be measured directly, although approximations (default values) can be used in analysis.

A much more satisfactory prediction of ultimate load capacity by dynamic means can be obtained by the use of Wave Equation Analysis. This is a computer based method of capacity estimation which considers the transmission of stress waves through the pile. The program GRLWEAP is available to carry out this analysis and the manual gives the theoretical background to the method and provides guidance on the choice of parameters. There are a number of different properties of both the soil and the pile required and these must often be estimated from a minimum of information.

GRLWEAP is also useful for checking the "drivability" of a pile, enabling an estimate to be made of the size of hammer required for driving and giving the final set and toe R.L. necessary to obtain the specified ultimate capacity. The analysis also gives stresses in the pile during driving which are useful, especially to check for possible tensions in concrete piles.

A development of Wave Equation Analysis is the Pile Driving Analyser (PDA) which allows real time monitoring of pile top strains and accelerations This utilises electronic measuring devices fastened to the pile during driving, which measure accelerations and strains, the resulting information being fed into a portable data analyser which provides an actual measure of hammer energy, pile top stresses and pile integrity and an estimate of the pile capacity during driving. It is a relatively easy and low cost test to carry out and provides a reasonable degree of assurance as to the pile capacity *in situ*. Further details of the method can be found in Balfe, 1984.

A more accurate assessment of pile capacity can be obtained by inputting selected pile top acceleration and strain data into the commercially available software suite known as CAPWAP. This provides a more accurate assessment of pile static capacity, an estimate led pile top load deflection curve and an estimate of the distribution of resistance acting on the pile shaft and base at the time of testing.

A value of geotechnical strength reduction factor, ϕ_g that is appropriate to dynamic analysis should be used, in accordance with AS 2159.

7.12 PILE LOAD TESTS

7.12.1 Introduction

Of course the best way to forecast the likely performance of a pile, either the ultimate load capacity (vertical or horizontal), or the load/deformation characteristics, (again in either direction), is to carry out a full scale load test on a representative pile at the actual site. This will obviously be superior to theoretical design methods, as all variations in soil properties, effects of installation, etc, will automatically be allowed for. With all its advantages, it may seem surprising that few load tests are in fact carried out, but there are good reasons for this, or at least one very good reason, and this is the high cost of any full scale test. This means that it is only economical to perform a load test on the larger and/or more important structures, where the foundation cost is high and therefore the potential savings significant. On most small and medium sized structures, it is cheaper to perhaps somewhat over-design the foundations rather than carry out a test.

A pile load test is usually carried out to confirm calculations based on normal soils tests and theoretical design methods as outlined above, rather than as the sole method of design. Often the load test is to determine the depth of driving required to obtain the required capacity more than to find the size/type of pile to be used. It will basically involve driving a pile of the size most likely to be used in the structure, with the equipment to be used and to the design depth. The pile is then loaded by jacking (either vertically or horizontally), against some form of reaction system to check for the required capacity.

The normal type of pile test is a vertical load test, which may be an ultimate load test or just a proof load test to ensure that deformations are acceptable at the normal working load. Either test can include the determination of load/settlement behaviour, which can then be used to back calculate soil properties. Other less frequent types of test carried out are for tension, lateral load and torsion (Poulos and Davis, 1980).

Reaction systems used for load tests are either kentledge, i.e. a physical mass of sufficient magnitude to take the pile to the required load, or some arrangement of tension piles or ground anchors. It may be possible in some cases to arrange things so that the test pile and the reaction piles are incorporated into the final structure, but this is often difficult due to pile location layout.

With either system there is the possibility of the loading system influencing the test. Tension piles may interact with the test pile and kentledge may increase lateral soil stresses on the pile near the surface or cause ground heave as it is lifted up.

A value of geotechnical strength reduction factor, ϕ_g that is appropriate to pile load tests must be adopted. Full details of the requirements for various types of pile load testing are given in Section 8 of AS 2159 and Appendices A to D.

7.12.2 Test Methods

There are two basic types of pile load test:-

- ***Maintained Loading Test*** - in which the load is increased (and decreased) incrementally with each load applied for a given period of time, or for a given penetration, and then stepped to the next increment.
- ***Constant Rate of Penetration Test*** - a quicker test which is not included in AS2159 in which the load is continuously varied to maintain a given rate of penetration. This is mainly used to obtain an estimate of the ultimate geotechnical strength of a pile.

With either test, it is important that the correct procedures are followed, including the proper isolation of deflection measuring devices, either levels or datum beams, and proper calibration of jacks. The maintained loading test is the most common.

The interpretation of load tests requires care. Because of the shape of the load/deflection curve, the actual definition of the point of "failure" may be difficult. Fleming et al (1992) defines a settlement of 10% of the pile diameter as "failure". The load/settlement behaviour can also be used to determine a modulus value for the soil.

Care must be taken if the bridge is long and/or the soil conditions vary across the site as it may then be necessary to carry out more than one load test.

Lateral load tests are usually carried out by driving two piles and then jacking their heads apart or pulling them together; ensuring that the piles are far enough away from one another (about five diameters) so there is no interaction.

As well as the full pile load test outlined above, there is also: -

- ***Pile Driving Analyser* (PDA) and CAPWAP analysis**, which can give a good estimation of the likely ultimate load capacity of a pile measured during driving or on restrike.
- ***Rapid Load Testing*** which uses a long force pulse imparted to the head of a pile by means of impact of a cushioned dropped mass or by reaction against an accelerating mass.
- ***Bi-directional load testing*** which involves casing one or more hydraulic jacks into the shaft of a cast in place pile which can provide a reaction force to apply downwards load to the pile base and upwards load to all or part of the pile shaft.
- ***Pile Integrity Tests***, which can help to assess pile shaft integrity by means of low strain impact applied to the pile top or transmission and receipt of sonic signals between a transmitter and a receive lowered down access pipes cast into the pile shaft.

These tests are outlined in more detail in AS 2159 Section 8 and its Appendices. Test equipment is available commercially and can be hired, along with an experienced technician to operate the instrument and engineers to interpret the results. Their limitations are mainly due to equipment malfunction and an inability to strike a pile with significant energy so that it is mobilised sufficiently. Specialist consultants are available to advise on and interpret these tests.

MRWA has carried out a number of pile load tests, in all cases for major structures. Full scale loading tests were carried out on the piles for the Burswood, Stirling, and Mount Henry Bridges, and the Pile Driving Analyser was used on test piles driven for the Mandurah Estuary Bridge, Reid Highway (Swan River) and Narrows Duplication Bridges. Reports on all these are available, see Forman and Gill, 1975; Michael and Wade, 1973; Jewell, Michael and Smith, 1984; and Bott, 1984.

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APPENDICES

A - PILE MATERIALS

B - METHODS OF INSTALLATION

C - EFFECTS OF INSTALLATION

APPENDIX A - PILE MATERIALS

A brief outline of the different materials piles can be made from was given in Section 7.2. This will now be expanded and each type described, with discussion of its advantages and disadvantages.

A.1 TIMBER

In the past, timber piles have been used extensively in WA, especially in the South West, however, they are now no longer used because of durability problems and the fact that suitable materials i.e. timber types and sizes, are no longer available. However, their special properties of flexibility and resistance to shock make them especially useful for fender piles, dolphins, etc.

The following information is provided to enable proper assessment to be made of existing and possible future use of timber piles.

Durability must be taken into account when considering the use of timber piles. If the piles are permanently below ground water level then they should have an indefinite life, the problem area is around surface level, where there is alternate wetting and drying. Here the piles will be susceptible to decay and destruction by termites (white ants), or fungi unless appropriate protective measures are taken.

In addition, timber piles in warm, tidal waters are very susceptible to damage from marine borers. This applies to all saline river estuaries in WA. The only proven method of protection for bridge piles is complete encasement to prevent contact between the water and the pile. In the past this was affected with some form of sleeve around the pile, either concrete or fibreglass, which is then filled with sand, see Figure 7.6. However, this has been found to cause rot in some cases and so should be used with caution. An alternative treatment is wrapping with Denso “Seashield” or similar. For lesser structures, e.g. dolphins, fishing platforms, etc, chemical pre-treatment by pressure impregnation could be considered, Koppers, (1985).

The timber species considered suitable and used in the past as bridge piling in Western Australia are Jarrah, (Eucalyptus marginata), WA Blackbutt, (Eucalyptus patens), and Yellow Tingle, (Eucalyptus guilfoylei) but they are not now readily available.

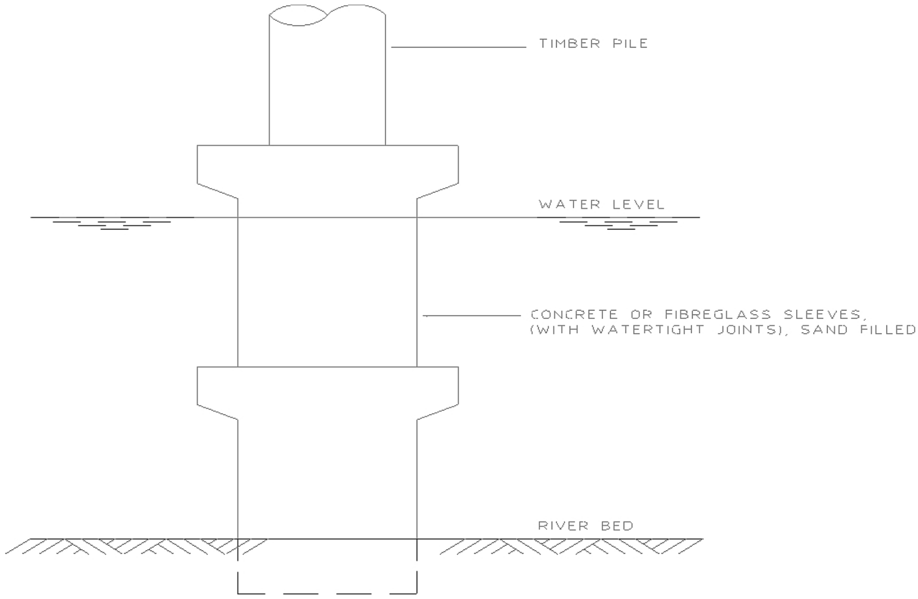

Figure 7.6 – PROTECTIVE SLEEVE FOR TIMBER PILES IN TIDAL WATERS

Jarrah piles would typically be 12-15 metres long with a minimum butt diameter (the head of the pile), of 450 mm and a crown (the toe of the pile), between 310 mm and 360 mm. Such a pile would have a safe working load of around 500 kN. Longer piles are sometimes available, up to a maximum of around 18 metres, but above this the pile would have to be spliced.

The advantages of timber piles are:-

- easy to handle, as they are relatively light and flexible, (compared with concrete);
- easy to drive, as they are flexible and so do not damage easily and to some extend can be "bent" if they go out of position during driving; and
- easy to cut-off at the required level.

Disadvantages are:-

- access to forests to win timber difficult in some areas and at some times of year, because of dieback restrictions;
- durability, as mentioned above;
- difficult to extend, see above;
- limits to lengths available, see above; and
- can be damaged by hard driving, especially the head.

A.2 CONCRETE

Concrete piles can be either precast or cast-in-place. Precast piles are either reinforced or prestressed, and cast-in-place piles either cast inside a previously driven casing, which may be left in or extracted as the pile is concreted, or cast in a pre-bored unlined hole.

A.2.1 Precast

Piles are usually available with either a square or octagonal section, although square ones are most commonly used by MRWA. They are durable and cheap and have reasonable load capacity.

The size most often used is a 350 mm square prestressed concrete pile, see Figure 7.7. These can be driven to an ultimate geotechnical capacity of up to 3000 kN or more, with a normal working load of around 700 kN, depending on the amount of moment to be carried. Pile lengths of 10-15 metres are most common, although they have been used up to 18 metres. A permanent prestress of about 7 MPa is used, which helps to prevent cracking when handling and driving. These piles are also used as combination pile/columns and cast directly into a reinforced concrete deck slab or pier cap beam.

Larger, higher capacity piles, e.g. 450 mm octagons with a 1000 kN working load, have been used, but these are heavy and therefore more difficult to handle and drive, and in addition are not as readily available. Also worth considering is the use of shorter reinforced concrete piles, either 400 mm, 350 mm or 300 mm square, which are now available, precast segmental piles from Frankipile or Wagstaff Piling. These are usually a maximum of 10-15 metres long but include a proprietary metal splice so that any length can be made up.

The difficulties of splicing are one of the major disadvantages of concrete piles. Unless an acceptable proprietary metal splice is available it is necessary to stop driving for a considerable time whilst a concrete or epoxy splice is made and cured. However, these are rarely used because of the delay caused. Alternatively, it may be possible to extend the pile after driving. Other problems with concrete piles are:-

- They are heavy and therefore difficult to handle and require a fairly large hammer for driving;
- Long piles are a problem because of splicing difficulties, see above;
- They are difficult to cut off. The concrete has to be broken back carefully to expose the reinforcement which is then cast into the pile cap or deck to provide continuity and moment transfer; and
- With heavy driving the head of the pile can break, making further driving difficult. Reinforcement in the head must be detailed carefully to avoid this, including perhaps the use of an external steel band, see Figures 7.7 and 7.9.

On the other hand, their advantages are:-

- Usually cheap, unless a long way from Perth, when transport costs may be high;
- Excellent durability even in highly corrosive environments, if designed and manufactured properly; and
- Good load capacity for cost.

Precast piles of moderate length, e.g. 10-12 metres, may be driven with a small diesel hammer, D12 or K13, but the larger ones will require a Delmag D 22 or equivalent. If possible it is preferable to drive precast concrete piles with a hydraulic hammer such as a Banut or Junttan with a 5 to 7 tonne ram. The hydraulic hammers provide more control over drop height and can thus better control driving stresses during installation.

Precast piles are large displacement piles and as such usually carry most of their load in friction unless they are driven to rock. This also means that they will not readily penetrate through hard layers to deeper founding material.

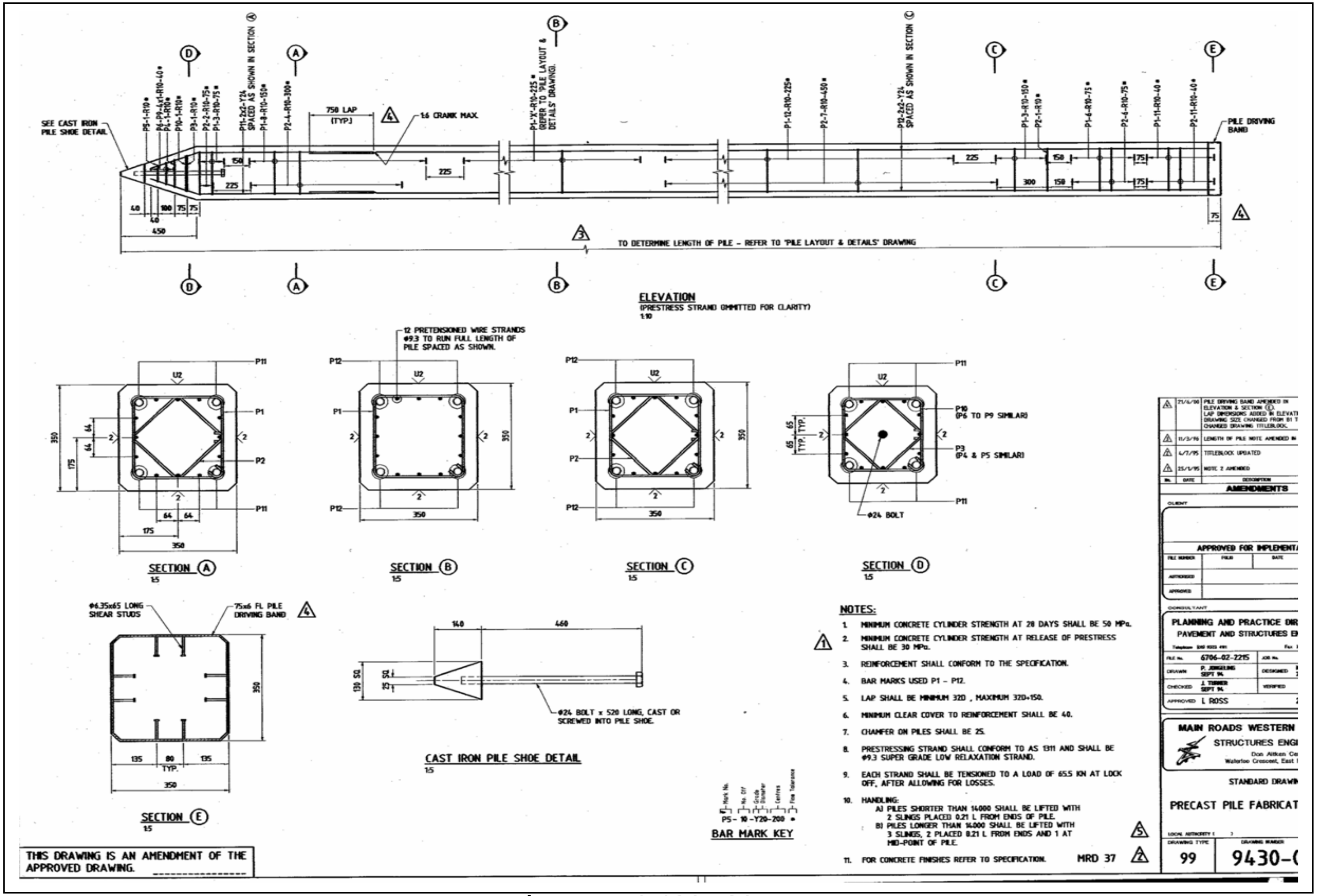

Figure 7.7 - 350SQ PSC PILE

A2.2 Cast-in-Place

Cast-in-place piles usually involve the driving of a steel shell, which is subsequently filled with reinforced concrete. The shell may be driven either closed or open ended and may be left in the ground or extracted when the permanent concrete pile is cast. Alternatively the "hole" for the piles may be bored, either with or without drilling mud (bentonite).

The main type that has been used in the past by MRWA are steel shell piles of up to 960 mm diameter which are driven open-ended, left in the ground and concrete filled. These have been used mainly in the North West, often where driving is hard but it is necessary to penetrate to a specified depth for scour protection. The shell is cleaned out progressively to ease driving, although at all times leaving a soil plug at the pile toe, to prevent the ingress of water. The tubes usually also serve as columns and are extended into the pier capbeam. Extension of the steel shell during driving can easily be achieved by welding. The pile casing is usually considered to be sacrificial and not taken into consideration for capacity calculations, although the section above ground is usually given corrosion protection treatment. Working loads for these piles vary from around 3000 kN for 600 mm diameter, up to 8000 kN for 960 mm diameter.

Open ended tubes are driven from the top, usually with large diesel hammers, Kobe K 35 and above or larger hydraulic hammers such as Junttan 10 or 12 tonne hydraulic hammers. Cleaning out inside after driving, or as driving progresses, is by screw auger or grab bucket. Closed ended tubes are little used because of penetration difficulties.

Cast-*in situ* piles in which the steel tube is extracted have been little used by MRWA. The most common type is the "Frankipile", in which the steel casing is driven to the required depth by a cylindrical internal drop hammer falling on a temporary plug of gravel at the bottom of the casing. When the casing is at the required depth it is restrained, to prevent it sinking further, and the plug is driven out. A bulbous base under the casing is then formed by compacting successive batches of semi-dry concrete by means of blows from the hammer.

Finally a reinforcing cage is installed and the shaft of the pile concreted as the tube is extracted. Details are shown at Figure 7.8.

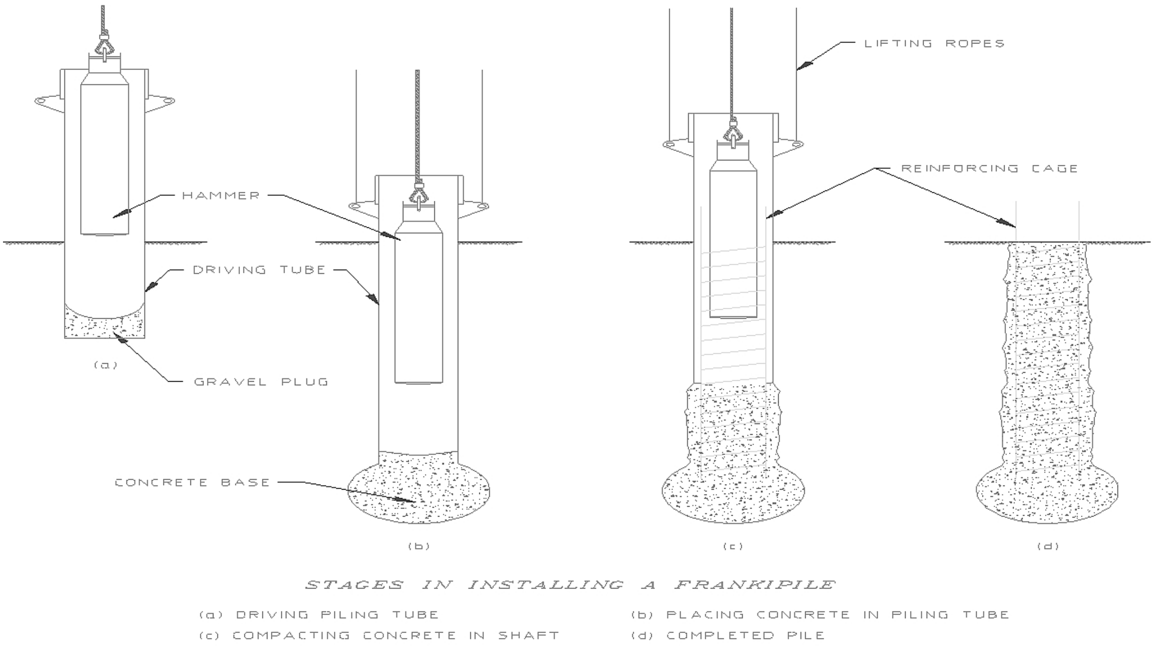

Figure 7.8 – FRANKIPILE INSTALLATION DETAILS

The standard Frankipile has a working load of around 700 kN. They have also been used as tension piles, but the bulbous base should not be relied upon unless the reinforcing cage penetrates the base. A double based pile can be created where an upper base is formed and the pile casing is then driven through it and another base is made which allows the reinforcement cage to penetrate the upper base for tension resistance.

They have been used infrequently by MRWA although use has increased in recent times. The main reason for this is doubts over the structural integrity of the pile shaft after concreting, because of the possible collapse of the surrounding soil or ingress of water as the shaft is concreted. However, pile integrity testing as well as proof load testing is now available and so their use should not be ruled out, (CIRIA, 1977 and Frankipile).

Other types of extracted tube, or even tubeless, piles have been used only rarely by MRWA, as this technique is less suitable for the predominantly sandy, highly permeable soils occurring in WA.

One instance where MRWA has successfully used uncased bored piles is in sand as contiguous piles to form a retaining wall across an existing road to enable a pedestrian underpass to be constructed, (refer Drawing No. 8330-0565). As long as the piles are concreted soon after drilling the sand stands up and the technique is quite successful.

A2.3 Continuous Flight Auger Piles (CFA)

CFA piles installed to depths of 25 m or more can be arranged using dedicated equipment and plant. A continuous auger string is used to bore a hole of diameter about 0.7 to 1.2 m. At the design depth concrete is pumped through the auger pipe to the end of the augers. These are withdrawn slowly as concrete is continually pumped to form the pile shaft. Records of concrete volumes pumped, drilling resistance and torque per metre are obtained. A reinforcement cage is lowered through the concrete to complete the pile. Reinforcement can be easily placed to about 15 m depth, but it is difficult to place full depth reinforcement for deeper piles unless in the form of a group of bars placed centrally.

A.3 STEEL

Driven steel piles take the form of either preformed sections (predominantly H sections), or steel tubes, which are usually purpose made.

Standard H sections as produced by OneSteel are the type most frequently used by MRWA. A variety of sizes are used, from the light 250 and 310 UBP sections, to the heavier 310 UC. Capacities obviously vary depending on the pile, from an ultimate for purely axial loading, of 2500 kN on a 310UBP78.8, up to 7500 kN for a 310UC240.

H section piles are a low displacement pile and can carry their loading by skin friction, end bearing, or more usually a combination of the two. Because they have such a low displacement they are very useful for penetrating through hard strata to reach an underlying founding layer. Other advantages include:-

- available in a wide range of sizes/capacities;
- consistent quality of materials;
- light and therefore easy to handle and transport and can be driven with small hammers;
- have a high capacity for their size/weight;
- are easy to extend to any length, by on-site welding;
- easy to cut off at the required level; and
- can be extended up to deck level to also act as columns, in a similar manner to concrete piles.

However, disadvantages are that:-

- the piles are expensive;
- they are susceptible to corrosion and so require some form of corrosion protection treatment;
- H section piles have a low moment capacity; and
- there is the possibility of a buckling type failure for piles with little lateral support.

Tubular piles have been covered above. They can have a higher capacity than H sections, in both friction and end bearing, and also do not have the problems of low moment capacity and possible buckling associated with H piles. However, they are more difficult to handle and more expensive.

Steel piles are usually top driven using a diesel or hydraulic hammer, the size depending on the weight and required capacity of the pile. Sealed tubes can also be bottom driven with a drop hammer, (e.g. Frankipile), and very light H piles can be driven with an air hammer, similar to those used for sheet piling.

Corrosion protection of steel piles is most commonly achieved by the use of galvanising or high strength epoxy coating systems. It is only necessary to treat steel exposed to air and water, piles permanently in soil below water table are unlikely to corrode, unless aggressive groundwater or anaerobic bacteria are present. Refer to Chapter 18 of this Manual for further details.

A.4 COMPOSITE

The final option, which combines the advantages of concrete and steel, with only some of their disadvantages, is to use a composite pile. This has a steel lower section with a precast concrete, usually prestressed, upper section. It therefore has the high capacity, easy extension and penetrating ability of steel in the lower, non-corrosive area, with the durability, bending moment capacity and stiffness of concrete in the upper, corrosion prone zone.

The steel portion is usually an H section, typically a 310UC158 or 240. This is driven first, extending as necessary, and then the concrete section added, there usually being a specified minimum R.L. for the bottom of this to ensure that the unprotected steel is not in the corrosion zone. The precast section is cast with a short length of steel section protruding so that it can be welded onto the previously driven steel pile. A typical pile is shown at Figure 7.9.

A large hammer is usually required to drive these piles as they are typically of high capacity and are quite heavy in their final, full length. A diesel Kobe KB 45 or even a KB 60 have been used in the past but a large hydraulic hammer such as a Junttan HHKA 9 to 16 tonne is now preferred because of the better control over driving stresses. Because of the large hammer, high capacity and therefore hard driving, special details are necessary to prevent a bursting type failure around the steel/concrete joint or head failure of the PSC section, see typical details at Figure 7.9.

One problem with this type of pile is that it is necessary to determine fairly accurately the final drive depth to ensure that the concrete upper section has sufficient embedment to protect against corrosion. This may be difficult to achieve, especially with the change in drivability when the large concrete upper section is added.

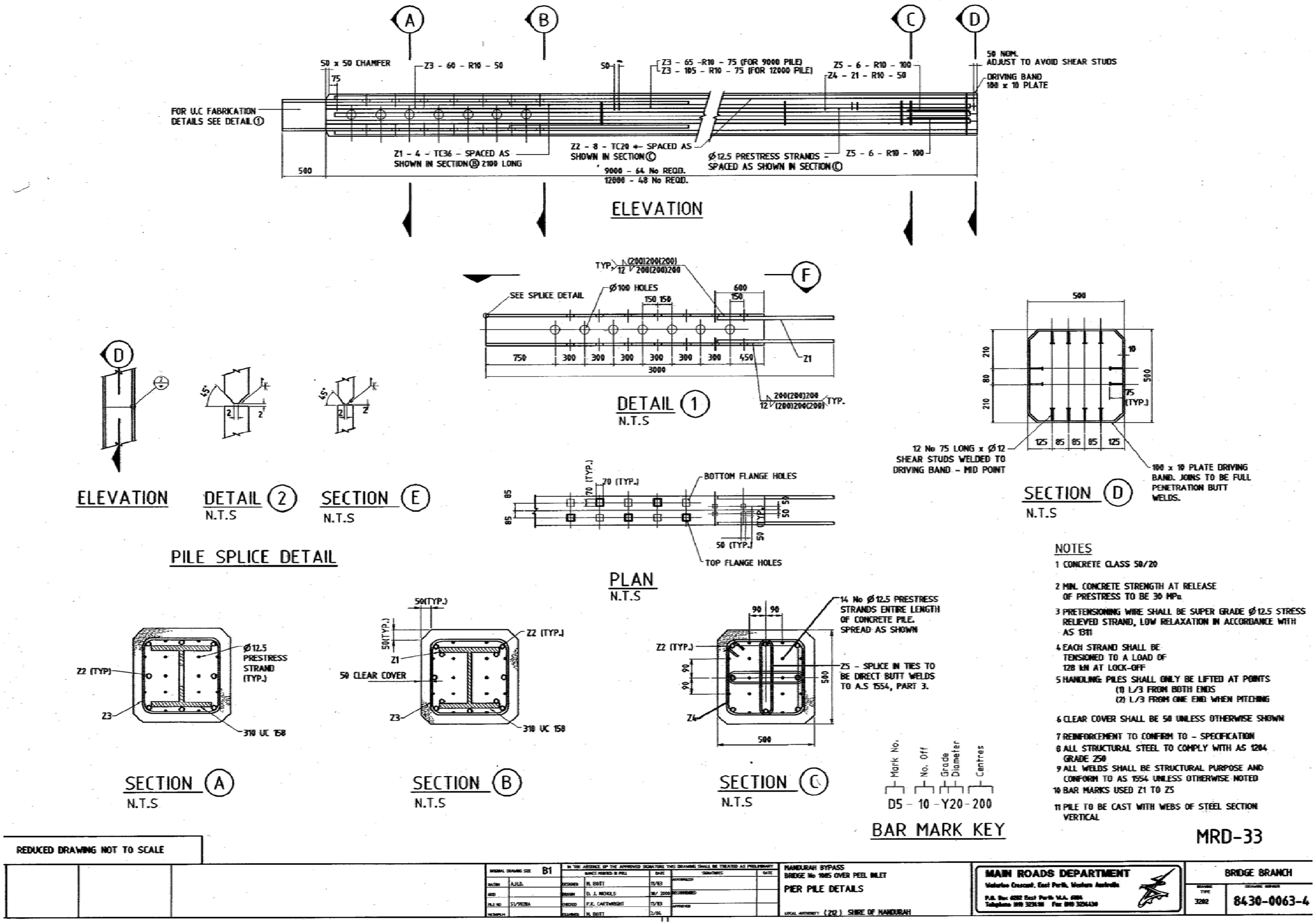

Figure 7.9 - COMPOSITE PILE

APPENDIX B - METHODS OF INSTALLATION

As mentioned previously, piles can be installed by either driving or boring. In WA by far the most common method of installation would be driving, which is used for precast concrete and steel H section piles as well as some steel tubes.

B.1 DRIVING

There are a number of different types of hammer and ways of supporting the pile and hammer during driving but MRWA principally uses diesel hammers, in conjunction with fixed leads on a crawler crane. Drop hammers and piling frames were the standard driving technique, but now all piles are driven using diesel hammers and crane mounted leaders. The hammers readily available in WA are Delmag D12 and D22 and KOBE K13, KB35, KB45, K60. More modern hydraulic hammers such as those manufactures by Junttan, Ice or BSP are now readily available for rental. Full hammer information is available from manufacturers catalogues, or from the GRLWEAP program.

To reduce the possibility of the impact of the hammer causing damage to the head of the pile, special pile helmets and cushioning materials are used. The helmet should be the correct size for the pile, so that the pile is controlled during driving. The helmet is preferably of forged rather than welded construction for durability. The cushion between the hammer and the helmet can be of dense hardwood timber or more often a special plastic material, "novasteen" or "micarta".

For precast or prestressed concrete piles a pile top cushion, usually of timber or plywood, must be used between the underside of the helmet and the head of the pile to prevent damage during driving. This must be regularly changed during driving to prevent damage to the pile top.

It may occasionally be necessary to use a long "dolly", a temporary extension between the pile and the helmet, if it is required to drive below the bottom of the leaders, see Figure 7.10.

Selection of hammer size is dependant upon pile weight and ultimate capacity. Some guidance is given in AS 2159, and the British BS 8004, but typically the weight of the falling part of the hammer, the ram of the diesel hammer piston, should be not less that half the weight of the pile being driven. The best way to check the choice of hammer is by use of the wave equation analysis program GRLWEAP, discussed previously.

Some problems associated with pile driving are the ground vibrations caused, which could damage adjacent sensitive structures, and the noise, especially of a diesel hammer, which can lead to restrictions on working hours in built up areas. Also, care should be taken when driving high displacement piles adjacent to existing structures, not just because of vibrations but because of actual ground displacements, both horizontally and vertically, which may occur.

To aid penetration into stiff soils it is possible to pre-auger a hole and then to pitch the pile in this and drive it on. Also, in dense sands jetting can be used, with high pressure water jets around the pile toe, but this must be used with caution as it results in considerable disturbance to the soil and therefore doubts about pile capacity unless this can be confirmed by restrike PDA testing after jetting installation is completed.

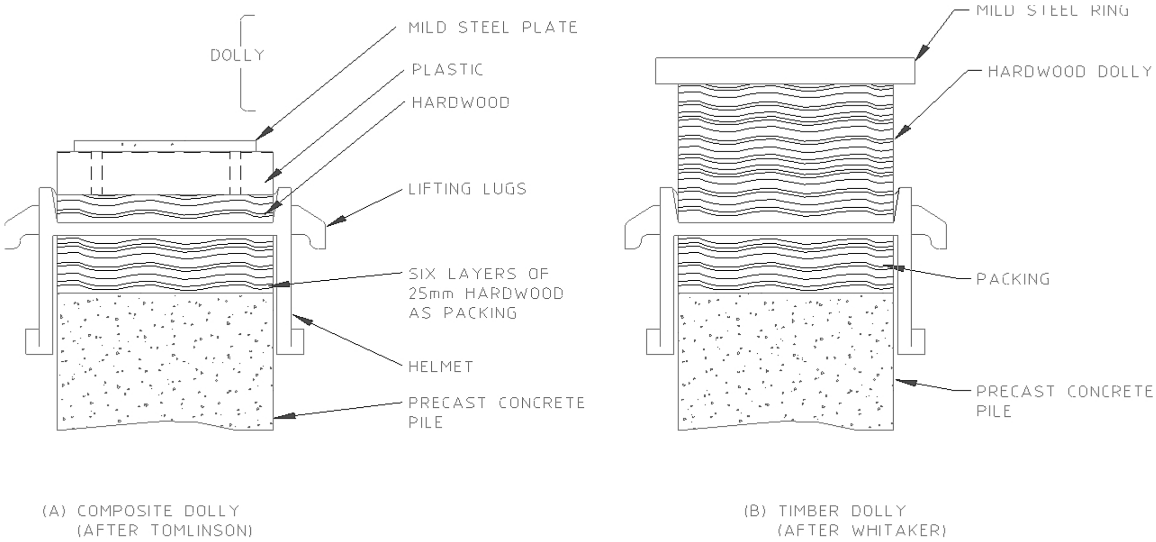

Figure 7.10 – PILING HELMET FOR PRECAST CONCRETE PILES

B.2 BORING

Some of the noise and vibration problems can be avoided by the use of bored piles, cased or uncased or CFA piles. With bored piles, excavation is by rotary drilling, bucket grab or percussion drilling and there is reduced noise and minimal ground disturbance. Some ground conditions are more difficult than others in which to construct bored piles e.g. boulders or cobbles in alluvial material, cavities in calcrete and variable rock in terms of strength and depth to competent material limit application. Other construction problems can include caving in of the hole, necking of the pile, segregation of concrete, displacement of reinforcement. Quality control of the pile construction requires close attention by an experienced geotechnical engineer where variable ground is encountered.

APPENDIX C - EFFECTS OF INSTALLATION

The act of installing a pile, whether driven or bored, must cause disturbance to the soil in the immediate vicinity of the pile. In addition, when driving groups of piles, the installation of adjacent piles may cause horizontal or vertical soil movements, which can effect previously driven piles. This means that there will be stresses locked into the pile by driving and the surrounding soil will no longer be the uniform mass that is assumed in theoretical analysis. These disturbances may be helpful, e.g. possible densification of sand brought about by driving, or detrimental, e.g. weakening of clay adjacent to driven or bored piles. Although it may not be possible to fully allow for all these effects the important thing is to recognise they occur and they should always be borne in mind when carrying out a theoretical analysis.

C.1 DRIVEN PILES IN CLAY

The driving of piles into a cohesive soil results in four major changes to the soil:-

- remoulding of a thin layer of soil immediately surrounding the pile;
- alteration of the stress state for the soil immediately surrounding the pile;
- creation of excess pore pressures in the soil around the pile, which subsequently dissipate with time; and
- long term strength regain in the soil disturbed by driving, partially due to dissipation of the excess pore pressures mentioned above, and partially to thixotropic effects, (i.e. restoration of structural bonds in the clay destroyed during driving).

Remoulding effects on clay are very difficult to quantify but there will be some loss of undrained shear strength (c_u), in the soil immediately surrounding the pile due to remoulding at constant water content. Immediately after driving the soil will have a low strength, but this will usually be regained, at least partially, although it may take a matter of days or weeks. Remoulding effects an area within about 1.5-2 diameters of the pile surface, although this will vary depending upon the sensitivity of the clay. The soil at the pile/soil interface will be fully remoulded, with a progressive reduction away from the pile.

Undoubtedly, the most important effect of pile driving in clay is the development, and subsequent dissipation, of excess pore pressures during driving. Measurements have shown that the pore pressure increases to a maximum within 2-4 diameters of the pile surface, and then gradually drops off back to the natural value at a distance of 15-25 diameters. The maximum value in the plastic, remoulded zone adjacent to the pile can be up to twice the initial overburden pressure. It is possible to calculate the maximum value and the drop-off rate theoretically, which can be used to estimate the rate of dissipation of excess pressure (Poulos and Davis, 1980). This will then give some idea of the likely rate of increase of pile/soil adhesion.

A further effect of driving large displacement piles into clay is that it can cause substantial displacements of the surrounding soil, which could have a detrimental effect on adjacent structures, or previously driven piles. There is an initial heave, followed by consolidation. There are a number of theoretical methods of calculation available for quantifying this effect (Poulos and Davis, 1980).

In addition, it should be noted that steel H piles in clay generally drive as an H, but fail statically as a box section as soil is trapped between the flanges and the web effectively forming a pile of rectangular cross section. This has to be considered when calculating the ultimate pile capacity.

C.2 DRIVEN PILES IN SAND

Driving piles into sand usually results in compaction of the soil by displacement and vibration, so in loose soils there is an increase in capacity. The affected region is up to 3-4 diameters around the pile shaft and up 5-6 diameters below the pile tip. There are some fairly simple theoretical methods for estimating this effect (Poulos and Davis, 1980).

With H piles in cohesionless soils there is also soil entrapment and the pile will again normally fail statically as a "solid" section.

C.3 BORED PILES

The main effect of installation on bored piles in clay is the reduction of pile/soil adhesion to a value less than the undrained adhesion before installation due to softening of the clay caused by:-

- Stress relaxation of the bore hole walls as the pile shaft is excavated
- absorption of water from the wet concrete;
- movement of water in the clay to the lower stressed area around the open hole; and
- water poured into the hole during the boring operation.

In addition, if drilling mud is used and is not fully displaced during concreting there can be substantial reductions in adhesion (down to zero!), due to the thin coating of bentonite left on the surface of the hole if a smooth pile bore is created.

The above factors, which cause problems at the pile shaft, also apply to the pile base. In addition this may be contaminated by cuttings and debris falling in during construction and not being properly cleaned out prior to concreting. This can lead to a reduction of bearing capacity and increased pile settlement. It is important for the base to be properly cleaned and inspected before concreting is carried out, either directly if the hole is dry or using a remote control inspection device (Williams and Smith, 1984).

There is little information available on bored piles in sand as they are not often used. They will usually require either casing or the use of drilling muds. The principal effect of installation will be a loosening of the soil of both the shaft and the base.

C.4 PILE GROUPS

With a group of piles driven into clay, the zones of increased pore pressure will overlap and, depending on the distance apart of the piles, there will be an extensive area of increased pore pressure throughout the group, with a pressure gradient only at the group boundary. This will result in a slow dissipation of excess pore pressure and therefore only a slow regain of strength.

In sands, if the soil is initially loose then around and between the piles it will become highly compacted and the group may have a higher capacity than the sum of the individual piles. On the other hand, if the soil is already highly compacted then driving may loosen it and result in a reduced group capacity, but this is less common.

It is normal to specify a minimum centre-to-centre distance for piles in groups, to minimise the above effects. The desirable minimum is 4 times the pile diameter, (D), with an absolute minimum of 2.5D (AS 2159 Clause 4.4.3.1).

CHAPTER 10

BRIDGE SUPERSTRUCTURE AND DECK ANALYSIS

CONTENTS

10.1 INTRODUCTION

This Chapter presents detailed information on the analysis or the derivation of the design loads and forces (the ‘design action effects’ (S^*)) of bridge superstructures and decks.

Before commencing it is important to be clear on the terminology employed. Although the terms "deck" and "superstructure" are often taken to be synonymous and used interchangeably, there is in fact an important difference. To be strictly correct, "superstructure" is the general term for every part of the bridge above the level of the supports, whereas "deck" refers purely to the top slab. This implies that the "superstructure" includes the "deck". These meanings will be maintained in this Chapter.

The importance of this distinction is that, for the purposes of analysis and design, the **deck** is considered to carry the loads and distribute them transversely to the longitudinal elements in the **superstructure**, (i.e. the beams and webs), which then transfer the forces to the supports.

Having made this distinction it is important to note an exception. With a flat slab deck, the slab in fact performs two tasks, it not only distributes the loads transversely but also carries them longitudinally to the supports. Therefore flat slab decks, (or more correctly flat slab superstructures), are designed as slabs in two-way bending.

Another important point to remember when reading these notes is that linear elastic behaviour is assumed throughout. This is in fact all that is permitted by the CODE, except for limited allowance for redistribution in a few special cases, (refer CODE Part 5 Clauses 7.2.8 and 7.2.9). Also, most analysis models consider the superstructure and deck as monolithic for all loadings. If they are not, e.g. timber superstructure, or non-composite steel/concrete decks, then allowance will have to be made in the model.

10.2 MODELLING

When analysing bridge superstructures and decks for the results of the design actions and forces, (i.e. deriving S*), it is important to realise that what is in fact analysed is not the actual deck but an **idealised mathematical model**. This may seem an obvious and unnecessary statement, but it is very important that this qualification is remembered at all times during the analysis. Any model is trying to represent three very complex factors; the behaviour of the materials, the geometry of the structure and the actual loading. Therefore, it can obviously only approximate the "true" situation. Different models have different characteristics, which must be understood and taken into consideration when interpreting the results of any analysis.

There are a number of different levels and complexities of models that can be used in analysis, depending upon:

The stage of the design - Simple, overall models tend to be used initially, with more complex, detailed ones later when the main design parameters are fixed.

The importance and/or complexity of the structure - The more important or complex the structure the greater the number and complexity of models that are likely to be used.

What effects are being studied - Some effects, such as overall bending and shear caused by dead and live loading, temperature, differential settlement etc., can be analysed satisfactorily on fairly simple, general models. However others, such as load distribution, transverse bending and local load effects, need more detailed and complex ones.

The different types of model can be classified in a number of ways, simple and complex, 2-dimensional and 3-dimensional etc., but the method adopted here is to consider two basic "generic" types. These are **global** models, and **local** models.

- ***Global*** - models can be either 2 or 3-dimensional and are usually of the complete structure, or at least a very large part of it. They range from simple 2-D line beam models to complex 3-D space frames, folded plate or finite element models. They are used for studying overall effects on a structure, both longitudinal and transverse, such as moments and shears caused by dead and live load, temperature, prestress, construction load sequences, stream force etc., and the distribution of these effects.

- ***Local*** - models can also be 2 or 3-dimensional and are used for studying specific areas of a structure, usually loaded by special actions. Typical examples would be; local wheel loads on slabs, prestress anchorages, punching type failures, diaphragms, deep beams etc. The models used can vary from detailed grillages, to space frames and finite element models.

Note this is for the **ANALYSIS** of the structure, (calculation of S*). The **DESIGN** of the superstructure and deck, (calculation of $øR_u$), will vary depending upon the construction materials, and is covered in detail in Chapters 11 to 15 of this Manual, although some guidance is given in this Chapter at Section 10.7.

10.3 GLOBAL MODELS

The general classification of global models can be subdivided into two dimensional or line beam models, and three dimensional models, and this is the order in which they are generally used in a design.

10.3.1 Line Beam Models

The first approach to the design of most bridges is to set up global line beam model(s) of the proposed structure. These are used to obtain the overall longitudinal bending moments, shear forces, torsions and reactions etc. for a number of load effects. The deck is modelled as a single centreline member, in 2-dimensions for a straight structure and 3-dimensions if curved. The boundary conditions for the model of a simple structure will usually be pins at the pier and abutment positions with an infinitely stiff support. The actual piers can be incorporated into the model if they are likely to be important in the distribution of the forces, e.g. if built into the deck. Also, the actual bearing stiffness can be used at the supports, although this is usually not necessary. The model must allow for any variations in moment of inertia of the deck, in order to get the correct longitudinal distribution of forces.

Global line beam models will be used for the analysis of:

Dead Load - This includes self-weight and superimposed dead load. These two effects are usually kept separate as they have different distributions and load factors. Typically a PCBEAMAN line beam model would be used, or an ACES model if the structure is curved or not of uniform cross-section. It is important to make proper allowance in the analysis for the construction method and ensure that the correct model is used. The different types of construction e.g. for beam and slab, span-by-span or cantilever construction, will have different and variable additions on the permanent effects and so the final bending moment diagram, shear force distribution etc. will be different to that of a similar structure cast all in one go. For the special case of incremental launching, ACES has a dedicated module that can be used.

Differential Settlement **-** Again a PCBEAMAN or possibly ACES model is usually used. It is important to allow for the settlement of any combination of supports. The recommended method is to analyse the structure with the settlement at each support considered as a separate load case, and then take the worst combination for each point along the deck. Positive and negative moments should both be calculated, plus shears and reactions.

As differential settlement is usually a long-term load, extending over a considerable period of time, it is important that, for concrete superstructures, relaxation effects are considered. This is usually done by using a long-term value of Young's Modulus in the analysis, (see CODE Part 5 Clause 6.1.2).

Overall Temperature Change **-** If this is considered important structurally, e.g. if the superstructure is built into the piers, then the analysis can be done using the temperature change facility in ACES. Normally temperature movements are only required for the design of expansion joints, bearings etc., and are easily calculated manually.

Differential Temperature **-** Differential temperature effects (secondary or parasitic moments) are calculated by obtaining the fixed end moments caused on the structure by the differential and applying these to a PCBEAMAN model (straight structures) or ACES model (curved structures). Note that parasitic effects only occur in continuous structures or portal frames, not in simply supported or statically determinate ones.

Creep and Shrinkage **-** Overall creep and shrinkage, if important structurally, can be analysed on an ACES model using the overall temperature change facility, as the effects are analogous. Otherwise movements required for expansion joint and bearing design are usually calculated manually. The method of construction must also be considered, e.g. staged construction will reduce the final overall movements.

Differential Creep and Shrinkage **-** Differential creep and shrinkage effects, such as resulting from casting RC slabs on steel or precast concrete beams may generate restraint moments. These can be calculated in a similar manner to differential temperature parasitics (see above). This subject is also covered in Appendix E of Part 5 of the CODE.

Prestress Parasitics **-** Straight structures can be analysed on the PARTIAL program, once an approximate cable profile has been determined. Alternatively a PCBEAMAN line beam model can be used with the prestress applied as an equivalent loading. The advantage of this method is that it is applicable to both straight and curved structures. For straight bridges it is adequate to obtain the equivalent loading by considering the vertical component of the prestress force along the structure resulting from the profile of the cable (see Hambly, 1976, Sec 11.7-11.9). For curved structures all components of the prestress, i.e. load and moment effects, must be considered.

Vehicle Loading **-** Moments and shear forces caused by traffic loading on straight structures can be analysed on PCBEAMAN. PCBEAMAN will generate overall moment and shear envelopes for the CODE standard vehicle loadings, or any other vehicle specified by the user. It gives an envelope of effects at different points along the structure and the corresponding vehicle positions. The latter is required when locating the vehicle on the structure to analyse the transverse distribution of the loading.

For curved structures the analysis is more difficult and an ACES line beam or grillage model should be used. The load positions for maximum effects will have to be found by trial and error, from an influence line for the structure or using a PCBEAMAN run for an equivalent straight structure to provide guidance.

Pedestrian Loading - This can be analysed on a PCBEAMAN or ACES model, choosing the loaded lengths as appropriate to give the worst effects.

10.3.2 Three-Dimensional Models

The above are all essentially 2-dimensional global models which give overall effects but do not take into consideration transverse effects, or the transverse distribution of longitudinal effects. These distribution effects are most important in bridges. Although dead load, temperature effects etc. are often considered to be uniformly distributed over the superstructure cross-section this is not always the case, and is seldom true for traffic loadings. Because traffic lanes can be positioned anywhere on the deck, the effects from vehicle loading, e.g. bending moments and shear forces, will be unevenly distributed between the longitudinal load carrying members. Satisfactory analysis for distribution effects requires a different model to the line beam one discussed so far. Either a different type of 2-D model (e.g. a plane grid rather than plane frame) or a full 3-D model. Note that even a grillage model is really a 3-D model. The geometry may only be defined in the x, y plane, but the load is out of plane, in the z direction.

The main reason for using these more complex models is to examine the distribution of loading to the different parts of the structure so that the maximum effects can be identified (see Section 10.7). This distribution is influenced by a number of factors:

- The position of the load on the deck; i.e. the eccentricity of the load (live or dead load) with respect to the supporting element.
- The structure type; number and position of beams, webs etc. and shear lag (see Appendix A).
- The effects of any skew or curvature.

The choice of analytical model is very important as the model used will influence the distribution of the loads, and could produce incorrect results. The model must be made to behave in the same manner as the real structure. For instance, with skew bridges the elements used in the model should span in the same direction as the reinforcement/ prestress in the actual structure, both longitudinally and transversely as this is the way the load will be carried.

There are a number of different models available to carry out distribution analysis, (and transverse bending analysis). The main ones used by MRWA are summarised below and discussed in more detail in Section 10.4.

Grillage - Grillage analysis is a popular method of structural modelling. The real, contiguous structure is replaced by a model composed of a grid, or grillage, of discrete beam elements spanning longitudinally and transversely. The number of subdivisions used, both longitudinally and transversely, will depend upon the form of the structure and the level of accuracy required.

The method can be used for all types of bridges, although it is more difficult to apply to boxes. Analysis of the grillage is carried out on the ACES program, using a plane grid. It is suitable for both straight and curved bridges and is particularly applicable to skew and tapering bridges, which are more difficult to model properly by other methods.

Analysis for dead load should also be approached with caution, as double counting is possible if the automatic density provision in the program is used.

The main limitation with grillage analysis is that it does not allow properly for shear lag, although an approximate allowance can be made through the use of the "effective width" approach, discussed later.

Finite Element - The finite element (FE) method is a very powerful, general method of analysis that can be applied, with appropriate selection of element and structural modelling, to almost any form of structure or structural element. For global models of bridge superstructures to analyse for load effects and load distribution, plate or shell elements are usually used. These are obviously directly applicable to flat slab type decks, but may also be combined with beam elements to model a composite beam and slab type deck, or even arranged to model a box section.

Finite element models are particularly useful for heavily skew bridges, where they generally produce better results than a grillage model.

Selection of the appropriate size of element is important. Too coarse a grid may miss peak values, whereas too fine a grid will increase computation time and volume of output. It is also important to understand the type of output produced, as some elements average out values across the element, so again peak values may be missed.

Qualifications aside, the finite element method is very powerful, and modern computers and programs with graphical pre and post processors produce speedy and useful results. It may be that, depending on the type of structure, one FE model will do for all analysis. A coarse model for overall effects, with finer, more detailed areas for local effects.

There are a number of purpose made FE programs, some set up specifically for bridges, e.g. LUSAS, ANSYS, LARSA, LEAP, however MRWA mainly uses the FE capability of ACES, which is adequate for most local situations.

Folded Plate - This method of analysis idealises the structure as a combination of beams and plates. It can be used for the same types of decks as grillage analysis, but it is especially useful for box section decks, which are difficult to model adequately as a grillage.

MRWA has the program STLBEAM, available to perform this analysis. The program has some limitations. It assumes the structure has a constant prismatic cross-section and is square at the abutments, which has rigid diaphragms.

Folded plate models can be used to analyse multi-span skew structures, as long as the spans are of reasonable length in order to get away from the end effects caused by the assumption of square abutments.

STLBEAM is used mainly to assess transverse load distribution in a structure and to calculate transverse bending in the slabs. It should **not** be used to calculate the absolute values of longitudinal bending moments as it is difficult to identify all components of the moment, since it is carried both in bending moments in the beams and in bending and membrane forces in the plates. A good measure of the distribution of load between the beams can be obtained however. This is usually assessed by comparing the stresses in the different beams, the compression face stresses usually being used. The distributions are likely to be different for sagging and hogging moments, and also for bending moment and shear force. The best measure of distribution of shear force at the supports usually comes from the reactions.

The program can also be used to analyse for transverse bending moments resulting from vehicle loads. The usual approach is to generate an influence line by moving a single vehicle transversely across the deck, and then get the maximum moment by finding the worst combination of loaded lanes. When doing this, both midspan and support positions should be analysed, as they may be significantly different. Also, make sure to allow for the

Dynamic Load Allowance on live load and to include dead load, superimposed dead load and prestress effects.

10.4 CHOICE OF MODEL

Although, as discussed above, a line beam model is adequate for the investigation of many load effects, it is necessary to use a more complex model to analyse for load distribution effects. While the line beam model gives overall values, the 3-D models give the transverse distribution of these values and so the peak design effects. The correct choice of model for a particular structural form is therefore important. It is theoretically possible to use any model with any type of structure, but some are more suited to one structural form than another.

Within Structures Engineering an ACES grillage or beam and slab model would normally be the first choice as the program is applicable to a wide range of structural types, is quick and easy to use and produces reliable results. However, in some cases, a STLBEAM model would be more appropriate, e.g. steel box section. It is also useful to have alternative models available when carrying out design checks.

The choice of models generally is discussed by Hambly (1976) and Cusens and Pama (1975) and to a lesser extent by O'Connor (1971) and Rowe (1962) including the use of some less common types. However, the best models for the superstructure types commonly used by MRWA are outlined below.

10.4.1 Beam Bridges

These are the simplest type of structure. The only bridges used by MRWA that can be treated as beam bridges are footbridges, both prestressed concrete and composite. Most road bridges are so wide that live load distribution effects have to be considered and a simple beam model is not adequate. For further details see Hambly (1976) Chapter 2.

Simple beam bridges can be analysed by hand using flexibility methods but it is usually easier and quicker to use a computer program, e.g. PCBEAMAN or ACES, modelling the superstructure as a line beam. As discussed above, this approach is also usually suitable for many of the effects on more complex structures, e.g. self-weight, finishes, temperature etc., on beam and slab, tee-beam and box section decks, provided they are symmetric.

10.4.2 Slab Superstructures

These would typically be solid reinforced concrete slabs, solid or voided prestressed concrete slabs or precast plank bridges (made continuous with an insitu reinforced concrete topping slab or transverse prestress). Solid decks act as isotropic slabs and the others orthotropic. Voided slabs can be analysed as solid slabs as long as the ratio of void diameter to overall depth is less than 0.6. Precast plank decks without an insitu RC slab act as an articulated plate, with shear transfer only between adjacent beams. This can be modelled on a grillage by the correct choice of nodes and releases (see Hambly 1976, Chapter 6).

An ACES grillage is the usual model used, or perhaps a finite element model. STLBEAM can also be used for constant section square ended slabs. For heavily skewed slabs an ACES shell element model produces the best results, particularly at the acute corners.

The choice of members in the grillage, their position and properties is important as this can affect how the model behaves, and whether or not it is a true representation of the real structure.

Longitudinal Grillage Members **-** are usually fairly easy to model as they represent beams in the actual structure, or a specific width of slab. The direction the beams should span should be obvious, even for skew bridges, as it will nearly always be parallel to the edges of the deck. Section properties [Section area (A), moment of inertia (I) and torsion constant (J)] of the model members are calculated in the usual manner for the member they represent in the real structure. It is usual to consider the gross section for these calculations, ignoring the reinforcement and the effects of cracked concrete, but see note below re J.

Transverse Grillage Members **-** will represent a width of slab. The number of transverse subdivisions used will depend on the level of accuracy required, and the need to realistically model the load, but normally a spacing of round about 3 times the slab depth will be adequate. The transverse members should usually be normal to the longitudinal ones, even for skew structures, unless the reinforcement is to be placed on the skew. This is normally only done in decks with a high skew, i.e. greater than 30 degrees, or where required for satisfactory reinforcement detailing, e.g. where there are diaphragms present. In this case the model members must also follow the skew in order to get a true representation of the action of the real structure. For all transverse members A and I are calculated as above, but the effective J for transverse members is reduced and it is usually adequate to take J = 2I for slab type bridges. Refer note below. For further details see Hambly (1976) Section 3.3.2.

> **Note**: Torsional stiffness - The value of J used for both longitudinal and transverse members must be chosen carefully. If a J value equivalent to the full solid section is used then the analysis will indicate an increased torsional moment with a consequential reduction in the moments about the other member axes. However if the section is not adequately reinforced for the resulting torsional moments it will crack resulting in a substantial reduction in J and therefore redistribution of moments to the other axes for which they may not be adequately reinforced. The CODE in fact effectively puts a limit of 20% of the full J as the maximum that can be used in analysis (Part 5 Clause 7.2.5(c)). Even this should be used with caution, and setting J = 0 or torsionless design seriously considered.

Loading **-** With vehicle loading, the loads applied to the structure will rarely fall upon the members of the grillage and they are usually transformed into nodal loads by distributing statically, first to the adjacent members and then to the nodes. The ACES program will perform this step automatically, however, it is important to be aware that the local moments associated with distributing the wheel loads to the nodes are not included in the results.

Output **-** The output from the analysis, e.g. bending moments, shear forces and reactions, can usually be used directly, but ensure that the width of the member represented is taken into consideration, and that coarseness of the grillage does not produce significant errors. See Hambly (1976).

For information on skew, tapered and curved grillages see Hambly (1976) Chapter 9. For further details on grillage modelling of slab decks generally, see Hambly (1976) Chapter 3.

10.4.3 Beam and Slab Bridges

This would include composite bridges with either precast concrete or steel beams, and concrete tee-beam structures. Both can be analysed using a grillage, beam plus FE plate/shell elements or a folded plate model, although the grillage is less suitable for tee-beams, due to shear lag effects.

Grillage **-** The modelling is very similar to that described above for slab bridges. Longitudinal members will usually represent the individual beams, plus the appropriate width of slab.

Longitudinally, section properties should be calculated about the centroid of the member they represent, any difference in level of the members being ignored. If the beams are far apart then shear lag may become important and this can be approximated using the effective width approach, see Appendix A. The torsion constant (J) for the beam and slab combination can be calculated by summing the individual Js of the beam and effective slab width. Again assume the concrete is uncracked and use transformed sections for steel/concrete composite construction.

Transversely a minimum of 8 members should be used per span, with properties as above, i.e. $I = bd^3/12$ and $J = bd^3/6$. The earlier note for flat slab bridges, on the maximum value of J that can be used also applied here, but is even more important. In fact, with concrete beam and slab bridges it is often better to adopt torsionless design (J = 0 for the main longitudinal members), as otherwise the torsion forces can be high and difficult to design for, even with the 20% limit.

One common modelling problem is if the main longitudinal beams (webs) have significant width. Modelling the transverse members as effectively spanning from centreline to centreline of the beams neglects the width of the web, and load effects will be over estimated. To better model this situation, rigid links must be inserted transversely for the width of the web to reduce the span of the transverse members.

Beam and FE Plate **-** Beam and slab bridges are probably most accurately modelled using a grillage, beam and FE plate model. The main longitudinal members are modelled as before as grillage members, but purely using the properties of the beam itself, there is no associated width of slab. The transverse members are now finite element plate or shell members. This can be modelled on ACES.

This is usually a more accurate model, as it more closely reproduces the behaviour of the actual structure. Effects of shear lag are also taken into account and transverse bending results given.

Again for structures where the beams have a significant width, rigid plate elements should be used directly over the beam with width equal to the beam width. Rigid elements are obtained by using a high value of Young's Modulus for these elements.

Folded Plate **-** This is an alternate way of modelling beam and slab decks with large spacings between the beams, e.g. double tee-beam construction, and it allows automatically for shear lag effects. The program STLBEAM is used.

For further details on analysis of beam and slab decks refer Hambly (1976) Chapter 4.

10.4.4 Box and Multi-Cell Bridges

This includes both "pure" box sections with 2 or 3 webs and multi-cell decks.

It is possible to analyse these types of deck, especially the multi-cell decks, using a modified grillage, or a space frame (see Hambly 1976, Chapters 5 and 7) but normally a folded plate model would be analysed using the STLBEAM program.

An outline to the theory behind the method is given in Hambly (1976) Chapter 12.

10.5 LOCAL MODELS

The analysis of local effects in bridge decks is not as well developed as the global analysis described above, but there are a number of approximate methods available which perform adequately.

For local wheel loads on slabs the method of Westergaard (1930) the influence surfaces of Pucher (1964) or for a number of special cases the formulae given by Roark (1965) may be used. If necessary a detailed grillage or finite element analysis can be carried out on ACES.

For wheel loads on cantilever slabs, STLBEAM gives good results although a much quicker and just as accurate analysis can be obtained using the manual method derived by Jaramillo, given in Appendix B.

For prestressing anchorages and areas around bearings the methods given in the CODE, (Part 5 Clauses 12.2 and 12.3) are usually adequate. Any further detail would have to be obtained from a finite element model, although specialised elements would need to be used as the concrete would probably be cracked in these regions.

Special models may also be required for pier and abutment diaphragms if their dimensions are such that they are classified as deep beams, see CODE Part 5 Clause 12.1, and Park and Pauley (1975). The usual approach is to use a strut and tie model (see CODE Part 5 Clause 12.1.2). The equivalent strut and tie model is set up and analysed as a plane frame or space frame in the ACES program.

For any other special local models, a finite element analysis would be the usual approach, however great care is required in the choice of element, and specialist literature should be consulted.

10.6 APPROXIMATE METHODS

Approximate or hand methods of analysis were once used extensively, especially for preliminary design work. However, now that computers are widely available and programs relatively user-friendly it is more usual to use the same methods of analysis for preliminary design as for final design. The only difference will be that approximate properties and coarser models will be used for preliminary analysis. For example, a coarse grillage mesh might be used; conservative guesses at live load distribution made; parasitics estimated etc.

Approximate methods based on statics are still important for checking. It is also important to have an idea of the likely magnitude of the principal effects before embarking on detailed analysis. This will form a check on the behaviour of the theoretical model.

10.7 APPLICATION OF RESULTS

This Chapter of the Manual is intended to cover analysis of bridge superstructures and decks and this is what has been discussed above. However, it is appropriate at this point to also consider the application of the results of the methods of analysis described. This is important because with bridge superstructures there are two special effects which have to be considered which mean that it is not usually possible to just design the whole cross section for average forces. These two effects are load distribution and shear lag.

Load distribution is well understood and applies to all types of bridge superstructures. However, in addition, when analysing superstructures made of beams and slabs, e.g. steel or precast concrete beams with RC slab decks, prestressed concrete tee-beams and boxes of both steel and concrete, consideration has also to be given to the effects of shear lag. It must be stressed that the two are completely different and separate. In brief they are:

Distribution **-** The uneven distribution of load effects across the superstructure resulting from non-symmetry of the loading, e.g. bending moments and shear forces caused by eccentric dead load and/or vehicular load. This means that, depending upon its location in the structure, a beam may carry more bending moment, shear force etc. than would be expected by just taking an average figure, i.e. the total effect on the deck divided by the number of beams.

Shear Lag **-** This is a result of the failure of normal Engineer's Bending Theory to allow for the in-plane distortion of the slabs in beam and slab bridges. It results in a peak stress in a member substantially higher than what might be expected from just taking the bending moment on the member, its total section modulus and applying normal bending theory. It is discussed more fully in Appendix A.

Designs in the past tended to use the "effective width" approach to allow for this effect, the method being included in the CODE Part 5 Clause 8.8.2 and Part 6 Clause 4.4.1. This ascribes a width of slab to each beam of the structure. The resulting model was then used to analyse the structure for load effects and the results obtained used to design the "Design Member", (i.e. the worst loaded beam and its effective width of slab) and from this the required prestress, reinforcement etc. obtained.

This approach however fails to properly allow for the fact that the shear lag effect varies with:-

- location on the structure; e.g. it is usually more severe close to the pier than in midspan; and
- the size of the structure; e.g. usually more severe for shorter spans.

Because of the above, in more complex structures the effective width and design member method is now not recommended for use. Instead Structures Engineering recommends the use of the "Stress Concentration Factor" (SCF) approach. This does not have the failings listed above and also is theoretically more logical and justifiable, although involving more work. Use of the method is however reliant upon having the proper analytical tools available. Whereas the effective width approach can be used in manual methods of analysis or on simple structural models, such as grillages, the use of the SCF method needs a more realistic, complete and therefore more complex structural model. This is usually a folded plate, finite strip or finite element model and necessarily implies the use of a computer analysis. The model is composed of an assemblage of beams and slabs with full slab action permitted (i.e. the slabs can distort, so that plane sections do not remain plane), and therefore shear lag effects are reproduced.

When used in analysis the SCF method allows for the effects of both load distribution and shear lag and in essence entails obtaining factors to scale up the results of the individual non-distributed load effects to allow for distribution and shear lag, and then using these figures in design.

Application of the method involves the following stages:-

1) Obtain the overall load effects for all the different loadings from normal line beam analysis on ACES, PCBEAMAN etc., as appropriate.

2) Analyse the critical load patterns on an ACES beam and slab or STLBEAM model. (Usually only the dead and live load effects are considered, not differential temperature, prestress, differential settlement etc., as the SCFs for these can be taken as unity).

3) From the compressive stresses in the beams obtained from the STLBEAM model obtain the SCFs for the different loadings. The SCF for a particular beam with a

particular loading is the ratio of the maximum stress in that beam to the average stress obtained from consideration of the total moment acting on the whole section behaving as an elastic solid in accordance with Engineer's Bending Theory, i.e. stress = M/Z.

Although to be strictly accurate the SCFs should be obtained for the full moment envelope, for all loadings, at all locations along the bridge, it is usually sufficient to calculate them only for +ve and -ve effects at midspan and the supports. The most appropriate figure is then used at other positions.

Note: The compressive stresses in the beams are normally used at this stage rather than the tensile stresses, although the latter often indicate higher SCFs. This difference is covered by the distribution of the reinforcement (see step 7 below).

If for some reason only the load distribution factors are required without the shear lag effect, then these can be obtained by comparison of the stresses in the beams with **each other** rather than with the "average" bending stress.

4) Apply these SCFs to the overall moments obtained at step 1. This will give the design moments, including the effects of load distribution and shear lag.

5) Obtain the design load cases by combining the above as per Part 2 Clause 22 of the CODE.

6) Use these scaled moments to design the superstructure, using the **whole** section and not just a design member.

7) When the required prestress and reinforcement are obtained, it is necessary to reflect the shear lag effects by locating the reinforcement in the cross section to suit. This means ensuring that the full amount of reinforcement required is placed in the "effective width" for each beam. The effective width can be estimated from the CODE or it may be worked out more accurately by considering the transverse distribution of longitudinal stress from the STLBEAM runs, see Appendix A.

   It should be noted that, as it is not usual to vary the layout of reinforcement across a section, the maximum required will normally be repeated all the way across. Also, although prestress should be treated in the same manner it is not usually necessary as it is normally concentrated at the beams anyway, although for large bridges this may require further consideration.

The above is a summary of the use of the SCF design method. However, although this is a more correct approach than the effective width method it must be remembered that, especially for reinforced and partially prestressed concrete, it is still only an approximation and so should not be pursued down to the finest detail. The main reason for this is that the model which is analysed assumes monolithic, linear elastic behaviour and for concrete structures, with cracking and its associated change in stiffness, this can obviously only be an approximation. It will be fairly accurate at the serviceability limit state, but what about ultimate when there will be wholesale cracking and progressive yield?

It might be considered that this will invalidate all of the above and at ultimate, as failure can only occur when the section has yielded, there will be an even distribution of all load effects across the section with no shear lag. This is true to some extent, however it relies on assumptions about the transverse distribution of the longitudinal effects as ultimate is approached and at present there is little information available on this. It is therefore safer to use the same SCFs for both the serviceability and ultimate limit states.

The above only covers the flexural design of the superstructure. Shear design is somewhat simpler. Shear is always assumed to be carried only by the beams and so shear lag effects will not be relevant. Distribution will still be important however. It is usual to assume that the shear force distributes in a similar manner to the bending moments and so the compressive beam stresses in the STLBEAM model will provide a good approximation of the shear distribution. However, in structures which have supports under each beam it may be more appropriate, in the critical region near the support, to assess the shear force distribution by consideration of the reactions.

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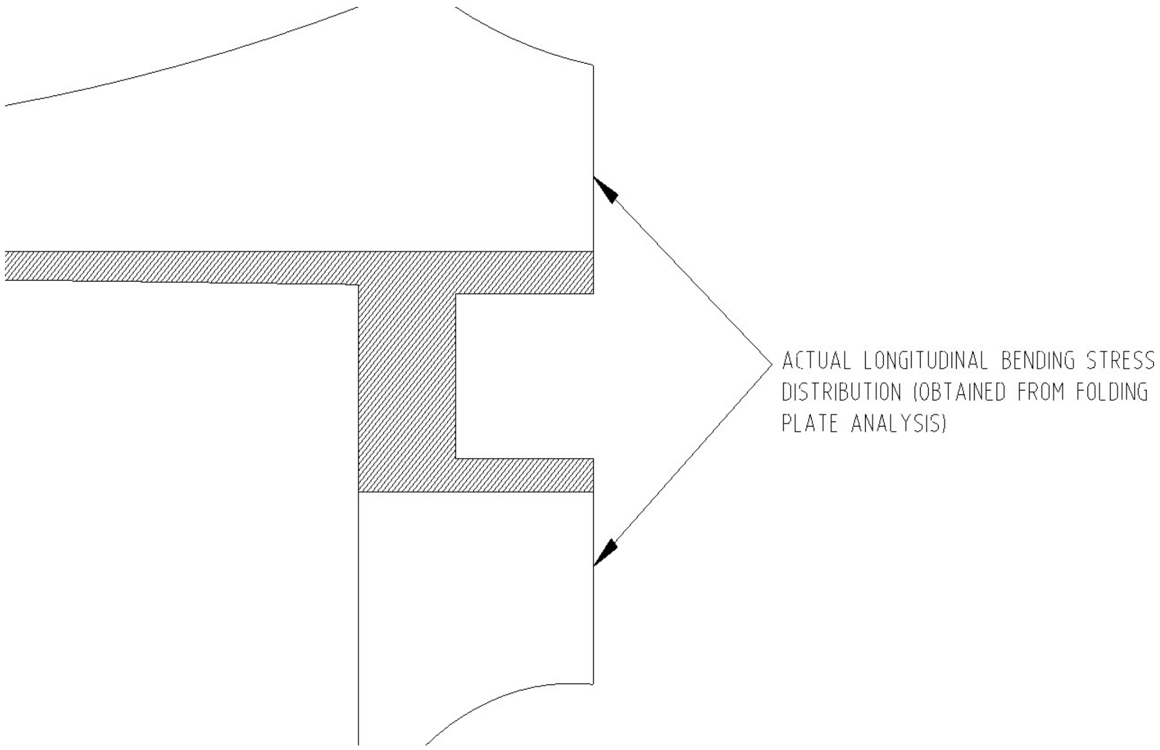
APPENDICES

A SHEAR LAG

B ANALYSIS OF CANTILEVER EFFECTS BY JARAMILLO'S METHOD

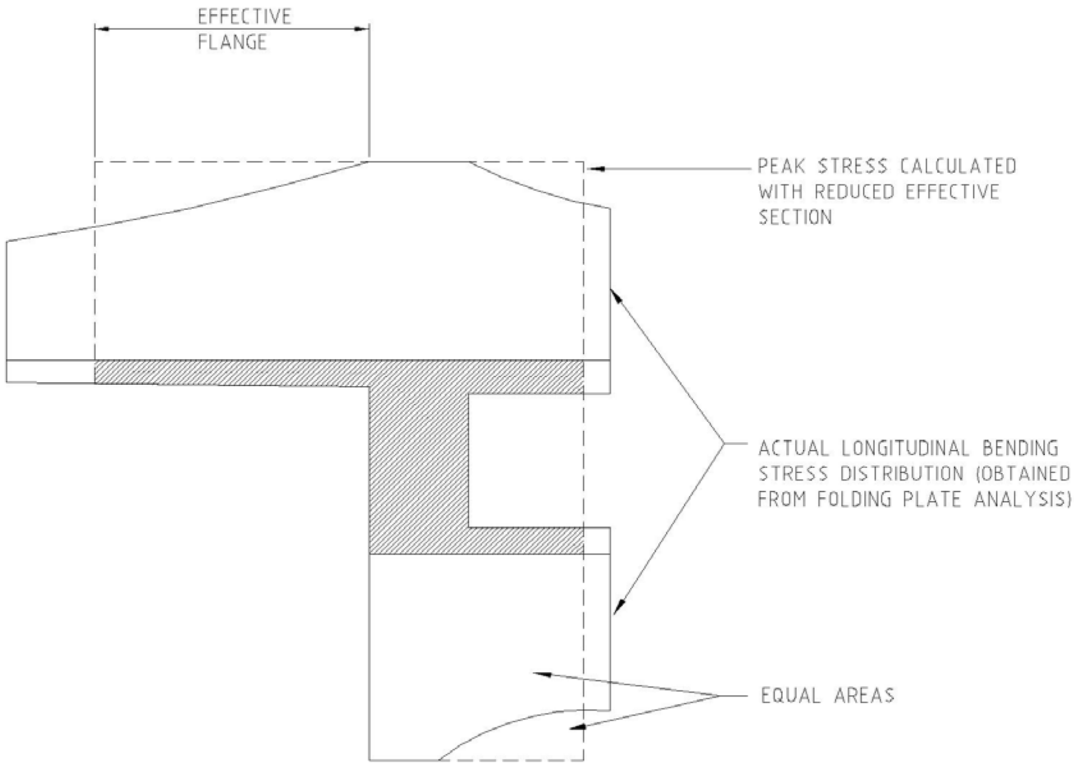
APPENDIX A - SHEAR LAG

Shear lag occurs in the slabs of beam and slab, tee-beam and box girder bridges. For these, simple beam theory, with its basic assumption of plane sections remaining plane breaks down because the flange distorts and therefore the distribution of bending stress becomes non-uniform, (see sketch below).



The effect becomes more pronounced as the slab width associated with the individual beams increases, as the slab gets thinner and as the span length decreases. Also, for continuous bridges the effect varies for different positions in the span and is more pronounced, (i.e. the peak stress relatively much higher than the average stress), at the piers than at midspan.

For the calculation of member properties in a grillage, the "effective flange width" is needed. This is the width of slab required at a uniform stress equal to the peak stress in the actual slab in order to provide the same total longitudinal force as is provided by the full width of slab associated with the beam at the actual, varying stress (see sketch below).



As an approximation, the effective flange width can be obtained by using the "rules of thumb" given in the CODE:-

The sum of the width of the beam plus for each side;

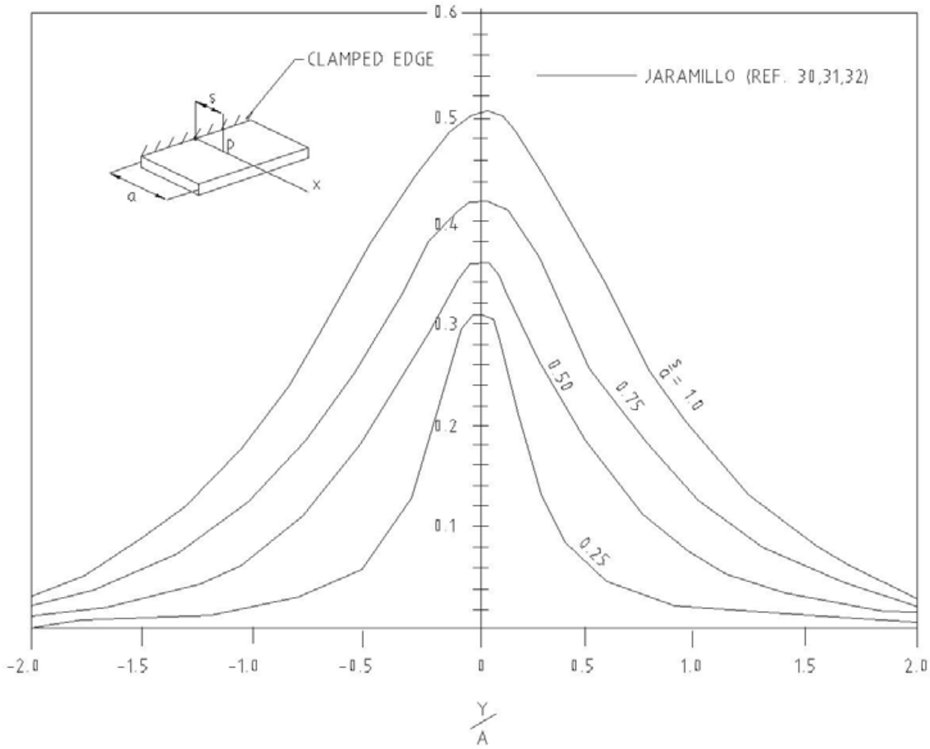
- The actual width of the overhanging flange, (or half the distance to the next adjacent beam); or
- 1/10 of the effective span length of the beam; or
- Six times the flange thickness

whichever is the lesser.

If using the STLBEAM program then the effective width can be calculated more accurately by plotting the transverse distribution of longitudinal force in the relevant slabs as shown on the diagram, integrating to get the total force and applying the definition above.

For more information on shear lag see Hambly (1976).

APPENDIX B - ANALYSIS OF CANTILEVER EFFECTS BY JARAMILLO'S METHOD



DISTRIBUTION OF MOMENT PER UNIT LENGTH (M) AT THE ROOT OF A CANTILEVER SLAB DUE TO A CONCENTRATED LOAD (P)

CHAPTER 11

CONCRETE DESIGN (Superstructure)

CONTENTS

11.1 INTRODUCTION

This Chapter provides guidance to designers on the materials, construction methods and tolerances typically expected in the construction of reinforced and prestressed concrete bridge superstructures in Western Australia. It also gives guidance for the design process, particularly preliminary design e.g. where to start, what computer programs to use, what shortcuts are acceptable. As with the rest of this Manual, it is not intended as a replacement text book for the basics of design, but it does aim to give guidance to engineers new to Structures Engineering in the methods used through appropriate references.

Design of structural concrete members is dealt with in detail by Part 5 of the AS 5100 (CODE) and associated Commentary. The Clauses of these are numerically aligned with Clauses of AS 3600 where appropriate, and there has been a deliberate effort to make the provisions of the CODE align with AS 3600 wherever possible. This has the advantage of uniformity in the industry. However, it is important to remember that the design life of bridges is typically 2 or 3 times that of buildings and so the durability aspects of design are accordingly more important.

11.2 MATERIALS

The materials described here are those which are typically used in bridge construction in WA because they are economically available. Rather than being the things that can be produced locally, they are the things that industry is willing to produce locally, combined with the things that are worth importing.

11.2.1 Concrete

The CODE has adopted the same set of preferred classes of concrete as set out in AS 3600. The MRWA Tender Document Preparation System concrete specification lays down details for concrete Classes S35/10, S40 and S50, plus S50M for marine environment. Class S35/10 contains 10 mm aggregate to facilitate precasting of fine components such as precast parapet panels. The maximum aggregate size in other classes is 20 mm.

The S classes are specified to ensure durability. In particular:

- Minimum cement content is specified for durability, particularly the control of steel corrosion.
- Mix proportion limits are specified to minimise shrinkage and creep.
- Limiting alkali content of constituents (usually as equivalent Na_2O) is also specified as a guard against Alkali Silica Reactivity (ASR or AAR). This is a long-term reaction of some aggregates with alkali in the concrete causing cracking and a potential risk of further degradation. The specification requires aggregates to be tested for ASR. Aggregates classified as having potential for slow/ mild ASR may only be used if the concrete mix is modified with a blended cement, either flyash or granular blast furnace slag. Aggregates classified as having potential for substantial ASR shall not be used.

Class N concretes are suitable for small elements (such as culvert headwalls and bases) in rural situations where project control of the mix design is not warranted. However, only S class is used for bridge construction. Class N20 is the only normal class from AS 3600 which is used in bridges, it is used for non-structural concrete such as blinding, guardrail post bases etc.

A review of the performance and economics of concrete classes in 1994 resulted in a general increase in the strengths specified for durability purposes. Table 11.2.1 is provided as a guide for selecting concrete class for typical structural elements.

Structural Element	Concrete Class
Precast Piles	S50
Precast Culverts	S50
Precast Beams	S50
Precast Parapet Panels	S35/10
Substructures - urban (non-aggressive)	S40
Substructures - in salt or splash zone	S50M
Cast-insitu Superstructures	S40 or S50

Table 11.2.1 - GUIDE FOR SELECTING CONCRETE CLASSES

S50M is a special class of concrete for “marine” applications. It contains silica fume in addition to cement, typically about 7-8%. The silica fume helps create a denser, more impermeable concrete, which improves overall durability. In addition, for large, thick elements such as pilecaps, a blended cement containing pulverised blast furnace slag is normally used rather than straight ordinary portland cement (OPC). The blended cement has a lower heat of hydration, therefore it is easier to control temperature gradients in the element, and so reduce the chance of early thermal cracking. Against this, it has a slower strength gain and requires longer curing times than OPC concrete.

11.2.2 Reinforcement

The commonly available reinforcement (deformed bar to AS/NZS 4671) is now Grade 500. It is important to specify N class reinforcement, Normal ductility. This is essential to ensure adequate ductility in a section at ultimate, L class is not adequate. Also, although bending and welding are acceptable practices, these together will reduce the available ductility and fatigue life. Consequently, welding in the vicinity of a bend is not advisable.

Welded wire fabric is used in concrete overlays of timber bridge decks but not often in major superstructures.

11.2.3 Prestressing Systems

Two types of prestressing tendon have been typically used for longitudinal prestressing of major bridge superstructures. These are:

> Φ7mm wire and Φ12.7mm 7-wire super strand to AS/NZS 4672.1 (Φ15.2mm is also available)

Brochures by Structural Systems and VSL give useful guidance on standard sizes of anchorages, trumpets and ducts, clearances for jacking, minimum radii of bends and other such details needed to prevent site difficulties with post-tensioning systems.

For the last three decades the use of prestress in WA has been almost exclusively as bonded cables. In post-tensioned construction, this is achieved by grouting of ducts with a neat cement slurry as detailed in the Specification. In pre-tensioned precast elements, the concrete is cast directly against the clean tensioned tendons. More recently, some overseas failures of bonded cables have prompted a renewed interest in unbonded tendons using new materials to guard against corrosion. However, these have only had limited use in WA.

Supergrade bars to AS/NZS 4672.1 are used for shorter applications such as tie-downs at abutments, prestressing pier and abutment crossheads and for tying structural elements together.

11.2.4 Miscellaneous Fittings

Various proprietary fittings are used to connect components to concrete and to couple reinforcing bars across a construction joint. Technical information on these is held in the Structures Engineering Technical Library.

In particular, the following are useful:

- Reid Swiftlift - lifting lugs for precast components.
- Schroeder Sockets - as blind nuts or couplers for connecting standard bolted items (note that as a coupler, a bar must be shown to pass through the eye).
- Ramset sockets for holding down expansion joint cover plates or similar.
- Alpha splices for coupling bars where there is insufficient space for simple laps, or to keep space free for other construction operations such as for prestressing jacks. There are screwed and swaged types - swaging provides transfer of full bar strength but it requires a machine. This is suitable for factory preassembly but not always suitable for site applications.
- Tapered thread connectors (e.g. Lenton splices) are particularly suitable for column starters.

11.3 CONSTRUCTION CONSIDERATIONS

11.3.1 General

Do not attempt to reinvent the wheel, look at drawings of previous designs for appropriate ideas and details. When you find an idea or detail worth copying, ensure that you check the final 'As Constructed' version of the Drawing and also look up the bridge file for construction feedback. These may point out variations that should be made.

11.3.2 Structural Continuity

Either continuous spans or simply-supported spans may be used and both have their advantages and disadvantages. Continuity of superstructures over piers provides a margin of robustness not explicitly recognised by the CODE, and also provides a smoother appearance and allows higher span to depth ratios for given deflection criteria. Continuity is also valuable in multi-span structures over railways as it provides some redundancy in the event of train impact with the piers. Simply supported spans require extra attention at the joints over the piers to ensure water tightness and durability.

The design of continuous superstructures must include an allowance for differential settlement (as a permanent effect) and differential temperature effects in accordance with the CODE. Values for these need to be established in the preparation of the Design Criteria Sheet. These compatibility effects may affect the serviceability limit state directly but only affect the ultimate limit state if insufficient ductility is available in the structure. Computer programs PARTIAL or CONKS can be used to evaluate the serviceability effects of these compatibility loadings.

With continuous structures, the CODE provides empirical rules to allow a limited amount of "Moment Redistribution" at the ultimate limit state, depending on the amount of ductility which is assumed to be directly related to the value of k_u at the section where the moment would otherwise be critical. It is important to note that this redistribution is a shift of the bending moment diagram from hog to sag (or vice versa) only where static redundancy will allow it - it is not a reduction in the overall moment, nor does it imply a non-linear analysis.

Recently simply-supported precast concrete Teeroff beam bridges have been used extensively. Although structurally simply supported, the deck slab is continuous to eliminate the need for joints over the piers.

11.3.3 Method of Construction

Although details of the construction method are generally specified as the responsibility of the Contractor, the designer must ensure that the design is both practical and safe to construct and the Drawings must demonstrate the overall method of construction assumed in the design. This is not to say that temporary works need to be detailed, but construction joints, precast units and sequences for launching and jacking must be specified where they affect the net capacity of the structure.

Design considerations for the main construction types used in Western Australia are introduced below.

Insitu - When the deck can be readily supported on falsework during construction, such as for new freeway construction, insitu casting of decks is possible. This provides the advantages of continuity without the analytical complications of staged construction. Construction joints may need to be detailed if the volume of concrete exceeds practical limits for a single pour. Differential shrinkage between such stages of casting is generally small enough to neglect, providing that the deck is continuously supported on falsework and the usual amounts of surface steel are provided for crack control.

Proof load testing of the falsework is recommended to ensure that capacity, settlements and deflections have been adequately considered.

Staged Construction - Some bridges which are cast insitu have the initial stages prestressed and/or falsework released prior to the casting of subsequent stages. The effect of this staged construction can be assessed by separately modelling each stage of construction with only the dead load of the latest section included at each stage, and then summing the results.

Precast Beams - Precast pretensioned concrete beams are manufactured to standard designs. The design of these requires knowledge of composite behaviour, which is dealt with in Chapter 12. In simple terms, the beams are generally designed to carry all the dead load in a simply supported state combined with carrying live load through composite action with a cast insitu deck. Normally the composite structure is made continuous, so allowances must be made for settlement differentials from pier to pier, and shrinkage and thermal differentials between the deck and the beams.

Steel/Concrete Composite - Steel/concrete composite superstructures are very similar to precast beam bridges in behaviour and the same considerations apply. The steel beams are erected, usually made continuous with welded or bolted splices and then carry the weight of the wet concrete of the cast insitu deck. The final superimposed dead loads, live loads, differential shrinkage and other effects are carried on the composite structure. Alternatively, the beams are left non-continuous and carry the loads as simply supported composite members.

Incremental Launching - Construction of concrete bridges by the incremental launch method is now common in Australia. Apart from allowing construction over traffic or difficult terrain with minimum falsework, substantial economies are gained when the bridge is long enough for the savings in falsework and formwork to offset the costs of setting up the casting bed and jacking systems. There are other advantages and pitfalls that can be found in various reports on the subject, particularly, Incrementally Launched Bridges, Design and Construction, Bernhard Göhler and Brian Pearson.

The casting bed should be designed to minimise construction lack-of-fit effects. This is achieved by judicious choice of the distance from the abutment to the casting bed and/or ensuring that adjustment of the casting bed level is provided for.

Construction loading and stress limits to be used in the design of incrementally launched structures are given in the Bridge Branch Design Information Manual, Section 9.

Segmental Construction - This method of construction has mostly been superseded by the incremental launch method. Mount Henry Bridge and Stirling Bridge were of segmental construction.

11.3.4 Clash Prevention

When considering detailed constructability, it is important to know that deformed bars are 10% larger than their nominal diameter and that the outside diameter of post-tensioning duct is 6 mm greater than the inside diameter. This, combined with normal placing tolerances can possibly cause problems in highly congested areas and should be checked carefully.

11.4 PRELIMINARY DESIGN

Structural design is generally an iterative process and the aim of preliminary design is to establish the general form that the structure is to take. This usually entails producing a preliminary estimate of the cost with various possibilities also costed. Consequently, sufficient detail must be assessed to be able to estimate the costs of options and to optimise overall sizes, but time should not be wasted on details that belong in the detailed phase of design.

The following sequence for the design process is presented in an endeavour to minimise wasted effort in the iterative process of preliminary design of each concrete bridge option:

- Estimate Spans
- Estimate Depth
- Choose a Structural Type
- Estimate Permanent Load Effects
- Compute Live Load Envelopes

- Estimate Distribution of Load
- Estimate Live Load Deflection
- Estimate Main Prestress and Reinforcement
- Check Shear Feasibility

11.4.1 Estimate Spans

The main spans of larger bridges are generally determined by clearances from roads etc. as set out in the Design Information Sheets. Allow for typical pier and abutment dimensions and for the effects of skew if any.

For bridges over waterways, if navigational clearances are not required, the relative economics of different span configurations should be considered taking hydraulic and foundation factors into account. When using prefabricated members, consideration needs to be given to a number of factors including transport, weight, ease of handling and location.

If possible, end spans of continuous multi-span bridges should be between about 75% and 85% of the length of adjacent spans for structural efficiency, economy and appearance.

11.4.2 Estimate Depth

In selecting a preliminary overall section depth D, some trial and error iteration is usually required. However with the software available this is neither difficult nor time consuming. As a starting point, for a simply supported prestressed beam $D \geq L/17$.

Incrementally launched bridges need larger depths (L/D ratios around 16 are typical) because of the restriction to concentric prestress only during launching.

In the preliminary design, do not attempt to be too ambitious in achieving shallowness of superstructure. Refining the depth of a structure in the detailed design may turn out to have benefits, but choosing a shallow depth at the early stages of design can cause unnecessary problems at the detailed design stage, especially if road levels have to be changed.

11.4.3 Choose a Structural Type

Chapter 2 of this Manual provides guidance on the most appropriate structural type for various situations.

11.4.4 Estimate Permanent Load Effects

Once again, guidance from previous designs provides a starting point for estimating dead load per beam. An allowance must also be made for superimposed dead load, construction effects (if applicable) and settlement. (With the latter, the amount must be defined on the Bridge Design Criteria Sheet - usually 20 mm differential at any pier and 10 mm at the abutment for spread footings, and 5 mm at all supports for piled foundations. Also, as this is a sustained load effect use the long-term modulus as described in Part 5 Clause 6.1.2 of the CODE).

11.4.5 Compute Live Load Envelopes

PCBEAMAN produces envelopes of moment, shear and reactions for standard vehicle loadings.

11.4.6 Estimate Distribution of Load

To design a beam, it is necessary to calculate the moments carried by that beam.

Approximations are useful checks. Examples include:

- For twin beam bridges with long spans, distribution based on transverse statics neglecting torsion can be used.

- The 1976 NAASRA Bridge Design Code gave a set of distribution formulae which can be used as a first estimate of the distribution of standard loading among beams.

Chapter 10 of this Manual treats this topic in detail for more precise methods of determining distribution.

11.4.7 Estimate Live Load Deflection

Before proceeding too far with the design it is important to check live load deflection. With the increasing live loads on bridges, and increasing refinement of design leading to lighter structures, this issue is becoming more important. The Part 2 Clause 6.11 of the CODE nominates this as a Serviceability Limit State with a limit of 1/600 of the span (refer Part 5, Clause 2.7 and Part 6, Clause 3.3.2 of the CODE for the calculation of the deflections).

11.4.8 Estimate Main Prestress and Reinforcement

At this stage (if not earlier), the exposure classification, the strength of concrete and the cover must be established and checked that they meet the requirements of Part 5 Section 4 "Design for Durability" of the CODE (allowing for the difference between nominal covers in the tables and minimum cover to be specified). Guessing an average diameter of steel, allows the designer to then estimate the effective depth from the compression face to the steel centroid.

Although approximate manual methods are available, it is more usual to utilise the PARTIAL or CONKS programs to check the section design. Although the ultimate limit state is the principal item to check at this stage, the tension steel stresses (SLS) should also be reviewed (refer CODE Part 5, Clause 8.6).

11.4.9 Check Shear Feasibility

Web thickness should be checked for shear at an early stage to ensure that the dead load estimate was reasonable.

11.5 DETAILED DESIGN

11.5.1 General

If the preliminary design was done using the guidelines above, the detailed design is a matter of ensuring that all the rules in the CODE are followed and that all details have been thought out to ensure ease of construction and maintenance.

Detailed design of Concrete superstructures is divided into two main zones:

- Flexural zones where plane sections remain plane and CODE formulae for the contribution of the concrete to the shear capacity can be applied.
- Non-flexural zones where Part 5 Section 12 of the CODE precludes the use of the beam and slab sections of the CODE.

11.5.2 Flexural Zones

Computer programs PARTIAL, CONKS, and COLDES are useful for detailed calculation of bending capacities at ULS and SLS where bending governs. Refer to the Bridge Branch Design Information Manual Section 14 for more details of these programs (simple principles are given in Section 11.4.8).

PARTIAL is also useful for determining cable friction losses and parasitic moments (i.e. the moments that occur in a continuous prestressed beam to maintain compatibility with statically 'redundant' supports). It can then be used to estimate the temperature effects allowing for the attenuation of these due to cracking. The TAB mode of PARTIAL can be used to produce envelopes of section capacities at the various limit states.

11.5.3 Non-flexural Zones

End zones of prestressed concrete bridges have bursting and spalling effects resulting from the dispersion of concentrated forces from prestressing anchorages. The free-body diagrams in Part 5 Appendix G of the CODE (which are based on the strut and tie model) illustrate how a balancing moment is required over the zone through which the force disperses to being uniformly distributed over the cross-section. This applies both vertically and horizontally. The moments should be calculated for each direction.

The CODE provides formulae in Part 5 Clauses 12.2.4 and 12.2.5 for the forces associated with these moments and Part 5 Clause 12.2.6 gives rules for working stresses. Part 5 Clause 12.2.6 gives rules for locating this steel, but it is also important to ensure that the steel is placed where it can resist the moment mentioned above and that it is located and detailed such that concrete can be placed and compacted.

11.6 CHECKING

As design criteria are in a limit state format, the results of the check should also be documented in limit state terms. Presenting these graphically not only concisely documents the results (with suitable precision - if you can't see the difference in a plot, then it is not significant) but also helps to avoid overlooking any weak spots. In particular, it simplifies the rules for truncation of steel for moment and shear at 'd' beyond where it is otherwise required (Part 5 Clauses 8.1.8 and 8.2.12 in the CODE).

The following graphs should be presented on a longitudinal base:

- Ultimate factored envelopes of Moment within factored Moment Capacities
- Ultimate factored envelopes of Shear within factored Shear Capacities
- Serviceability envelopes of Moment within serviceability limit state Moment Capacities

Similar plots should be presented for each beam type and for transverse effects, if significant truncation of steel is used.

When combined effects (such as torsion with moment) become significant, the CODE generally presents the limit state in a format such as:

$$\frac{M^*}{\phi M_u} + \frac{T^*}{\phi T_u} \leq 1$$

In such cases, the left-hand side of the inequality could be plotted using absolute values. However, this format can be misleading if the values plotted do not allow for the 'd' offset mentioned above. A practical way to allow for this is to plot extra 'x' values at 'd' from the points at which the capacities were calculated and to linearly interpolate the M* and T* values for the above expression. The plot can look messy at the low points which are non-critical but it will identify any point which exceeds unity.

CHAPTER 12

COMPOSITE DESIGN

CONTENTS

12.1 INTRODUCTION

Composite construction in the form of a reinforced concrete deck slab acting compositely with steel main load carrying members is an economical and popular form of superstructure, particularly for bridge sites that are remote from the Perth Metropolitan area. This type of construction may be used for a wide range of span lengths. This Chapter however, is only concerned with composite decks comprising a concrete slab and steel I-beams, the form in which composite construction is most commonly used by MRWA. Other steel/concrete composite superstructures such as boxes, trusses and arches with composite slabs are not covered.

Composite construction using various forms of precast, pretensioned concrete beam, with an insitu deck slab, is also common. Whilst much of the content of this Chapter is also applicable to concrete, composite construction, as stated above, the Chapter concentrates on steel I-beam/concrete deck composite design.

This Chapter is to be read in conjunction with Parts 2 and 6 of the AS 5100 Bridge Design CODE. It is intended to be complementary to Part 6 of the CODE and provides guidelines for the actual design process, detailing requirements and some important construction issues associated with steel/concrete composite superstructures.

12.2 PRELIMINARY DESIGN

The main purpose of the preliminary design is to determine the most economic span-to-depth ratio and the number of girders required for a selected slab thickness.

The span lengths are generally determined by waterway requirements (for bridges over rivers) or by minimum clearances from roads. Then, for a given span the designer has to select:

- a steel I-beam (generally, directly from the OneSteel (ex BHP) catalogue as a first guess). There is a fine line between the number of girders, which improves the live load distribution, and the economic effectiveness of the structure;
- a slab thickness to ensure that the composite girder has sufficient stiffness and the span-to-depth ratio is within the recommended range; and
- the spacing of the steel girders to match the selected slab thickness and minimise or eliminate the effect of shear lag in the composite section. It is advisable at this stage to have also selected the type of the formwork to be used for the deck slab, as this may have an impact on the final thickness of the slab (refer Section 12.3 below).

With the introduction of the SM1600 live load, the live load deflection limit and control of footpath girder vibration have become governing criteria in composite design. Based on recent research, the recommended span-to-depth ratio for a composite girder to meet the above criteria should be around 15 – 20 (note that this ratio is of course influenced by the girder spacing).

At the preliminary design stage the designer should also consider the costs and practicality of transporting the beams from the supplier to the fabricator and from the fabricator to the site. This may influence the overall economics of the structure. However, for the more remote sites, steel/composite construction, is still likely to be a more economic solution than precast on insitu concrete.

12.3 PERMANENT FORMWORK

Experience has shown that the use of permanent formwork for the deck slab, rather than temporary formwork can have significant economic advantages. There are a number of types of commercially available permanent formwork suitable for bridge deck construction. The two major groups are:

- precast concrete panels; and
- steel (BONDEK II, CONDECK HP, CONFORM, TRUSSDECK, etc.).

Concrete panels have the advantage of providing a complete concrete finish to the deck soffit. It can also incorporate formwork for the kerb upstand, providing an extra cost advantage. Care is required in detailing however to avoid clashes with shear studs. Issues that have arisen in the past with the concrete formwork and its manufacture include obtaining sufficiently accurate tolerances and neat alignments of the kerb edges.

Since the concrete panel will become part of the final composite slab, care should be taken in selecting the overall slab thickness, for two main reasons:

a) thickness of the concrete panels varies depending on the girder spacing and the amount of in-situ concrete it will have to carry; and

b) the slab bottom structural reinforcement is normally incorporated into the concrete panel in addition to the formwork reinforcement. Allowance, therefore, should be made for the bar diameter and the minimum concrete cover in specifying the final panel thickness. The top cover may be reduced to as little as 20 mm, as it is only required for the construction stage. Since the concrete panels are placed on high-density polystyrene strips (e.g. 25 x 25 mm) running along the beam edges, 10 mm nominal allowance should be made for the compressed strip in determining the final deck level.

Final design of the concrete formwork panels will be carried out by the supplier, however a spreadsheet program is also available from the supplier to enable the designer to carry out preliminary design and ensure the proposal is feasible. In WA the licensee for Humeslab is Humes.

Steel permanent formwork is normally considered in MRWA's design practice as sacrificial. In other words, any composite action between the formwork and the concrete is disregarded. A reduction of the bottom reinforcement cover may however be warranted. The "metal" corrugated deck soffit appearance may be a deterrent to its use in areas of high public exposure, and also possible corrosion may be an issue for near coastal sites. Forming the deck cantilever and kerb upstand will also be an additional expense, as in most situations it would require use of additional formwork.

The main suppliers provide structural information e.g. section properties of their product to enable the designer to check its suitability.

12.4 STRUCTURAL MODELLING

12.4.1 General

Adequate structural modelling is essential for the proper determination of live load distribution. There are two ways that the designer can go about structural modelling.

One way is to set up a grillage or a 'grillage and FE slab' model (refer Chapter 10 for further details) and use it directly for analysis of all load combinations. In this case the model should have the capacity to calculate the effects of support settlement, secondary effects due to differential temperature and shrinkage, and enveloping those with the dead and live load effects.

Another way is to use the grillage or 'grillage and FE slab' model for determination of live load distribution only, complemented by a line beam analysis model capable of calculating and enveloping all feasible load effects.

The former method will usually be required for skew bridges, but the latter, simpler approach, is often adequate for straight, square structures.

With either approach, the load effects in the model elements are calculated assuming elastic stress distribution. Although the ultimate plastic hinge stress distribution is quite different from its elastic counterpart (see Figure 12.1), according to the CODE it is acceptable to assume that the load distribution at the ULS remains unchanged to that in the elastic model.

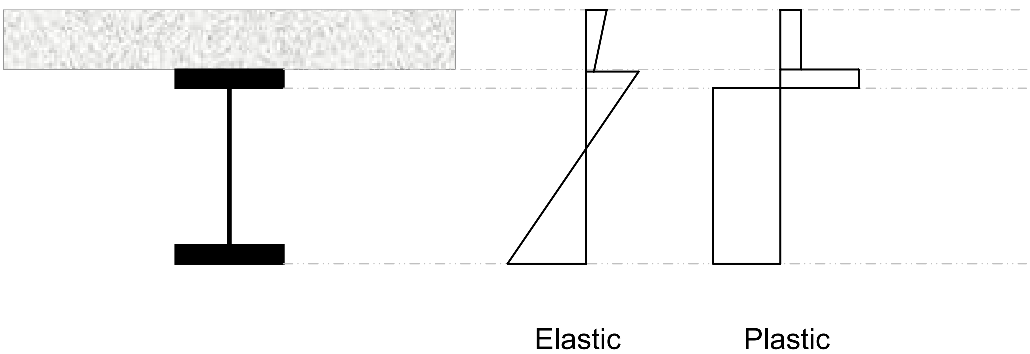


Figure 12.1 - STRESS DISTRIBUTION IN COMPACT COMPOSITE SECTION

12.4.2 Available Useful Software

There are a number of software packages licensed to MRWA, or available as free issue, which can be utilised in modelling and analysis of a composite superstructure. Those are:

- ACES and STLBEAM – for 2D analysis of a composite superstructure;
- PCBEAMAN – for a line beam analysis of an effective girder;
- BHP BRIDGEBM – for analysis of an effective composite girder;
- SMORGON ARC – HumeSlab Panel Excel spreadsheet.

12.4.3 Effective Girders

When setting up the analytical model, the main beams and their associated width of slab (the effective girder B_{ef}) form the longitudinal members. The effective width for beams is taken as the same for both hog and sag regions. The calculation of the effective width for both is based on the same 'effective' beam length. The effective length is taken as the distance between supports for simply supported beams and 0.7 times the distance between supports of internal spans for continuous beams. For advice on determination of B_{ef} refer to Part 6, Clause 4.4.1 of the CODE.

A continuous composite girder of uniform section may have variable stiffness over the supports due to the potential cracking of the concrete in the hog region. This cracking or "softening" in the hog region will lead to a longitudinal moment re-distribution from hog to sag region.

To ensure that the worst loading cases are modelled, MRWA policy is to utilise two longitudinal models for analysis of continuous composite design:

1. Using an uncracked I value throughout the length of the bridge, to determine the HOG moments; and
2. Varied I values, using an uncracked section in SAG regions and cracked section in HOG regions, to determine the SAG moments.

In the uncracked sections, the width of the concrete slab is converted to an equivalent width of steel by multiplying by the ratio of E_c / E_s for composite analysis. In the HOG region (for calculating SAG moments) the longitudinal reinforcing steel over the extent of B_{ef} is included into the composite cracked section (and the concrete ignored). Refer further Figure 12.2 and also Part 6, Clause 4.4.3 of the CODE.

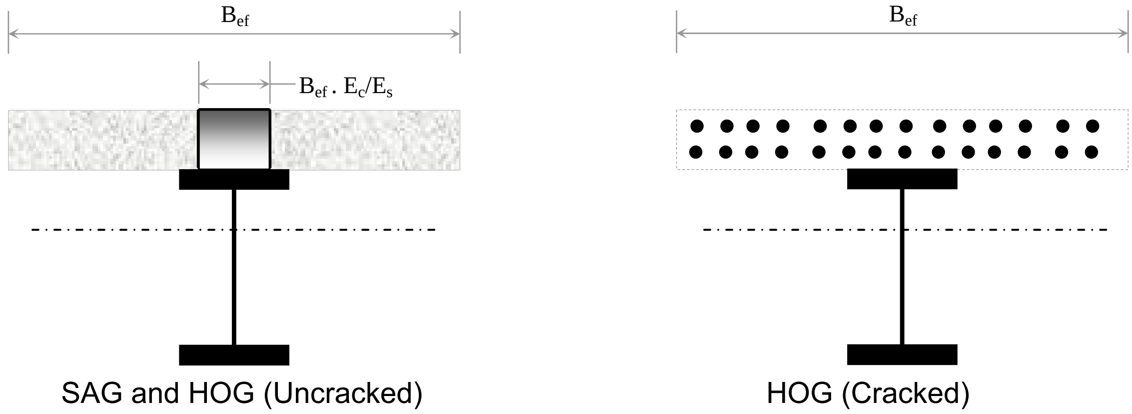


Figure 12.2 - EFFECTIVE GIRDER

12.4.4 Transverse Members

Continuity/distribution is modelled in the transverse direction and is provided by either discrete transverse members (slab members with beam properties) or, in a ‘grillage and FE slab’ model, by the finite elements.

It should be noted that selection of the particular type of a model is left to the designer. It seems, however, that the ‘grillage and FE slab’ model is to be preferred over the traditional grillage. This is due to the fact that the former is free of some of the issues inherent in the grillage model, where it is necessary to assign transverse beam properties, such as the out-of-plane moment of inertia and torsional moment of inertia, to model what is essentially a strip of a continuous slab.

12.4.5 Support Diaphragms

Lateral restraint to the longitudinal girder at the supports is provided by either a steel (by way of cross-bracing) or a reinforced concrete diaphragm.

With steel bracing, concerns exist over:

- construction costs: manufacturing and installation of cross-bracing are rather labour intensive operations;
- maintenance: making sure that the bolts remain tight for the design life of the bridge; and
- the aesthetics of cross-bracing, particularly for bridge sites with a high public exposure.

Indeed, the current composite design trend is to avoid cross-bracing altogether by ensuring the effective girders are sufficiently stocky for both construction and the final stage. In situations where bridges may be subjected to overtopping by floodwater, the use of cross bracing is also to be avoided, as it could be a debris "catcher".

Concrete diaphragms are normally considered more aesthetically pleasing than steel and may also have a potential cost saving as the resultant structure may require fewer bearings. They also make incorporation of a shear key, to provide resistance against overtopping, easier. Selection of a concrete diaphragm does, however, require large holes to be drilled through the beam webs to allow for threading of the diaphragm reinforcement, awkward formwork between the beams, and possibly temporary bracing during construction.

In modelling the diaphragms, it important to note the following:

- if the diaphragm structural reinforcement is not adequately anchored in the slab, in other words if there is no structural continuity between the diaphragm and the slab (such as is the case where concrete formwork panels are laid continuously on top of beams and diaphragms, for instance), the members modelling the diaphragms in the 2D model should be removed. This approach is somewhat conservative, as the diaphragms still provide lateral and torsional restraint to the beams;
- if the diaphragm is designed to be structurally continuous with the slab, relying on full torsional stiffness of the diaphragm may assist with better live load distribution between the effective girders. However, unless diaphragm torsional cracking is controlled in accordance with the CODE, the torsional rigidity of the diaphragm in the 2D model is recommended not to exceed 10% of its theoretical "uncracked" value. Note, however, that proper torsional crack control requires larger amounts of longitudinal diaphragm reinforcement to be threaded through the beam webs than would otherwise be necessary.

12.4.6 Differential Shrinkage

In a continuous composite structure, shrinkage of the cast insitu reinforced concrete slab can induce substantial secondary (or sometimes termed as "parasitic") effects and must be taken into consideration when compiling the critical serviceability and ultimate load combinations. The sketch below demonstrates one of the ways to model the differential shrinkage effects in a line beam analysis.

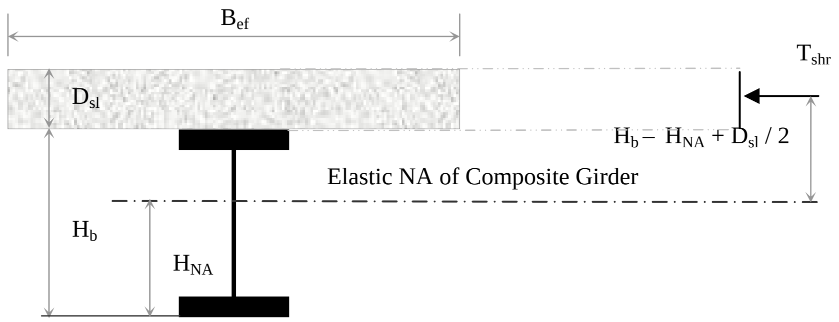


Figure 12.3 - MODELLING OF DIFFERENTIAL SHRINKAGE LOADING

$T_{shr} = B_{ef}.\ D_{sl}.\ E_{c\ long\ term}\ .\varepsilon_{cs}$

$M_{shr} = T_{shr}.\ (H_b - H_{NA} + D_{sl} / 2)$

where ε_{cs} = is the design long term shrinkage differential.

This primary moment shall be applied as uniformly distributed over the entire length of the effective girder in the grillage or line beam model. As differential shrinkage is considered to be a permanent effect, the parasitic moments and shears calculated by the model are then combined with the superstructure dead load to produce the maximum load effects.

12.4.7 Differential Temperature

The differential temperature effects in a continuous girder are calculated in a similar manner to those of differential shrinkage by applying a primary moment calculated per unit length of the girder to the model and compiling the secondary effects. The secondary effects should be included in both serviceability and ultimate combinations as required by the CODE.

The differential temperature primary moments are calculated thus:

$$M_T = F_T \, e_T$$

where M_T = primary moment per unit length;
F_T = resultant "thermal force" applied to a cross-section;
e_T = eccentricity of the resultant "thermal force", being the distance between the centroid of the thermal force and the elastic NA of the composite section;

$$F_T = \underbrace{\underbrace{\underbrace{\alpha_T \, T}_{\text{Strain}} \, E}_{\text{Stress}} \, A}_{\text{Force}} \quad \text{with } \overbrace{T\,E\,A}^{\text{integration of temperature gradient diagram}}$$

α_T = material coefficient of thermal expansion;
T = effective temperature (refer Part 2 Figure 17.3 in the CODE);
E = modulus of elasticity (short term for concrete);
A = area of cross section.

Both "hot top" and "cold top" conditions shall be considered and it is necessary to sum the components from the steel I-beams and the concrete slab.

12.5 DESIGN OF EFFECTIVE GIRDER

There exists a fundamental difference in the design of simply supported and continuous steel-concrete composite girders. Simply supported girders are usually fully compact with the compression flange continuously restrained by concrete. The girder ultimate bending capacity is, therefore, governed by the girder section moment capacity. In continuous girders, however, the ultimate girder capacity is frequently dictated by section slenderness ratio and lateral - torsional buckling of the girder in the HOG region.

The latest revision of Part 6 of the CODE has seen the deletion of the provision for "slender" composite sections. Thus the CODE is now consistent with existing MRWA policy of prohibiting the use of slender sections in MRWA composite bridge design.

12.5.1 Simply Supported Girder

To cause collapse of a simply supported effective girder, a plastic hinge must develop in a critical cross section (see Figure 12.4). The location of the neutral axis in the hinge, and subsequently the section plastic moment capacity, is affected by the degree of shear connection at the section. In other words, the depth of the concrete flange in compression that develops the stress equivalent to 0.85 f_c' to counteract the tensile forces developed in the steel section corresponding to full yield, is dependant on the compound capacity of shear studs between the end of the girder and the section under consideration.

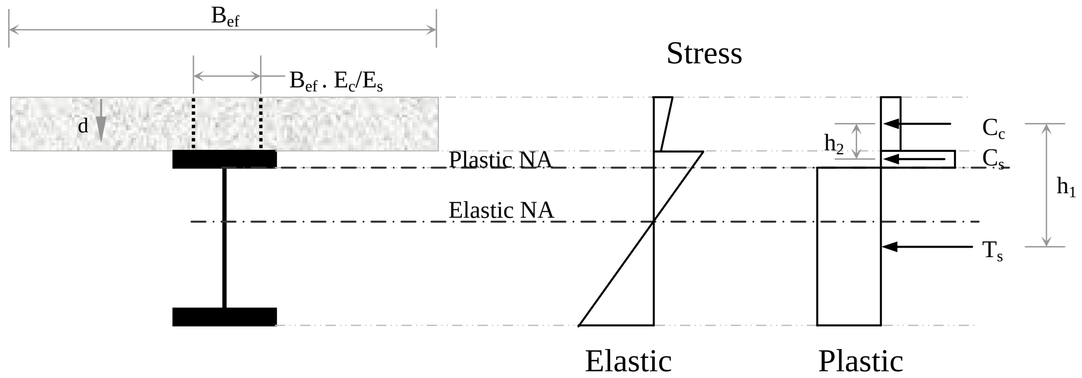


Figure 12.4 - COMPOSITE SECTION IN SAG REGION

Location of plastic NA is determined by $T_s = C_c + C_s$

$C_c = 0.85\ f_c'\ B_{ef}\ d$, where d depends on the degree of shear connection (with full shear connection d = depth of slab)

Ultimate girder capacity in bending is then $M_u = T_s\, h_1 + C_s\, h_2$

To rely on the compression capacity of the entire concrete flange, full shear connection must be achieved at the given section.

Since the plastic hinge implies stress redistribution, it is not necessary to sum separate stress distributions for each stage of construction.

12.5.2 Continuous Girder

The design of continuous composite girders is primarily governed by lateral - torsional buckling over an internal support, where the bottom flange and part of the web are in compression, but continuous restraint is provided by the slab at the top flange. The beam may also be restrained by a diaphragm at the support.

The beam section capacity is again calculated based on a degree of shear connection at a given section. The SAG region is analysed similar to simply supported structures, whereas in the HOG region the Designer must make an estimate of the girder effective length and draw a bending moment diagram for <u>each</u> critical load combination. It is important to note that in order to determine the moment modification factor α_m a resultant (dead, live and other loads) bending moment combination diagram should be drawn for a given position of the live load (Figure 12.5). Conventional enveloping of the load cases will not assist in this situation.

Unless restraints are provided directly by a diaphragm or bracing at both ends of a girder segment, the effective length should be calculated in accordance with Part 6 Clause 5.6.4 of the CODE

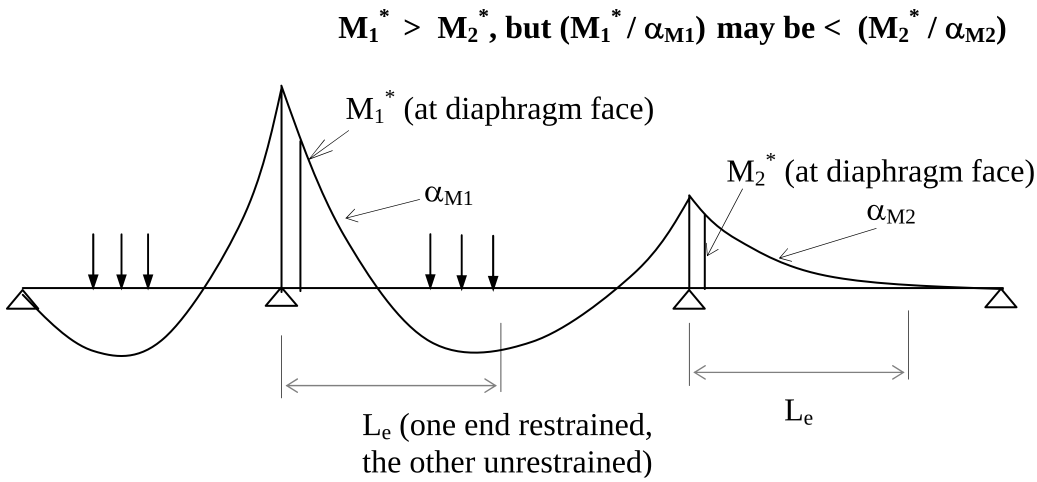


Figure 12.5 - EFFECTIVE LENGTH AND MOMENT DISTRIBUTION FOR GIRDER BUCKLING ANALYSIS

If there are reinforced concrete diaphragms present then the maximum HOG bending moments should be calculated at the face of the diaphragm.

For preliminary design an approximate, if conservative, approach is to take
α_m = 1.3 for all cases.

12.5.3 Shear Connectors

There is a direct relationship between the extent of composite action and the degree of shear connection at a given section. In other words, the amount of concrete in the slab and/or its reinforcement acting compositely with the steel beam is directly determined by the magnitude of the compound longitudinal shear resistance provided by the shear connectors at the interface, built up to the cross section under consideration.

Modern composite design is based on the theory of ductile shear connection - the shear connectors are able to plastically deform such as to ensure a uniform longitudinal shear resistance over a substantial length of the beam. It is therefore not necessary to group the connectors at the ends of the beams. Instead, the CODE requires uniform distribution of the shear connectors along the entire beam length.

Design for longitudinal shear is covered by Part 6 Clause 6.6 of the CODE. It should be noted that shear studs are designed to the Serviceability Limit State, not Ultimate.

The conventional type of shear connector is a simple headed stud. The studs may be welded to the beams at a workshop or on site. Welding on site is sometimes cost effective, but will cause damage to the beam protective coating, and requires a substantial power source for the stud welding machine. For remote bridge construction sites, it is advisable to require workshop welding of studs, followed by application of the protective coating to the entire assembly. In this instance, particular care must be taken to make sure that no clashes will occur on site, for instance, between the studs and the permanent concrete formwork reinforcement.

12.5.4 Site Splices

If site splices are required preference should be given to a bolted splice ahead of a welded splice, unless the splice is designed to be over a pier, where any damage to the protective coating will be covered by casting the splice in concrete. The splice over a pier, however, will have the maximum load effects, but it does eliminate any requirement for temporary propping.

Finally, whatever its location, any site splice must comply with Part 6, Clause 12.3.1 “Minimum Design Actions on Connections”, with the composite member design capacity determined in both the HOG and the SAG mode, if applicable.

12.6 SLAB DESIGN CONSIDERATIONS

The reinforced concrete deck slab contributes to the bending resistance by being part of a composite girder and also acts transversely in bending to distribute the wheel loads between the girders.

The slab longitudinal reinforcement may comprise one or two layers of bars and is determined by analysis of the composite girder in HOG for both SLS and ULS.

The slab transverse reinforcement comprises two layers of bars and is determined by a combination of the slab global bending (generally transverse deck curvature is more pronounced in midspan) and local bending (most influential at or close to the piers). Moments and shears are obtained from the grillage or ‘grillage and FE slab’ model, (refer to Chapter 10 of this Manual for guidance on modelling).

In determining the slab reinforcement, the designer also should be aware of the minimum slab reinforcement requirements specified by Part 6 Clause 6.1.4 of the CODE.

12.7 SUPERSTRUCTURE LATERAL RESTRAINT

If a physical restraining system is required to ensure structural integrity of the composite superstructure for identified lateral forces, e.g.. force due to water flow or earthquake, it may be achieved by one or a combination of the following means:

- use of guided pot bearings;
- use of dowels; and/or
- use of shear keys.

The latter two normally require construction of concrete superstructure diaphragms over the supports.

12.8 CONSTRUCTION STAGES

Composite structures are usually constructed in stages, e.g.:

1. place individual beams;
2. splice beams to make them continuous (if applicable); and
3. place concrete.

For all these stages the beam alone, either simply-supported or continuous, must carry its self-weight and the weight of wet concrete plus any construction live load. Composite action is only available for superimposed dead load, live load, differential settlement, differential shrinkage and temperature, once the concrete has reached its specified strength.

Therefore in the analysis of construction stages the following must be considered:

- lateral - torsional buckling of a simply supported steel girder carrying its own weight; and
- lateral - torsional buckling of a simply-supported or continuous steel girder carrying the weight of the in-situ concrete.

A further option for construction staging is to pour the deck concrete in stages. If the section over the piers is poured first then this establishes continuity in the beams for carrying the weight of the remaining deck pours. Forming the construction joints can be awkward, but this option usually has merit.

Unless permanent bracing is being provided, the beams may have to be temporarily braced at the construction stage. In some cases, particularly with longer spans, upward precamber of the steel girder for its own weight and the weight of the in-situ concrete may also be warranted.

12.9 DESIGN FLOWCHART

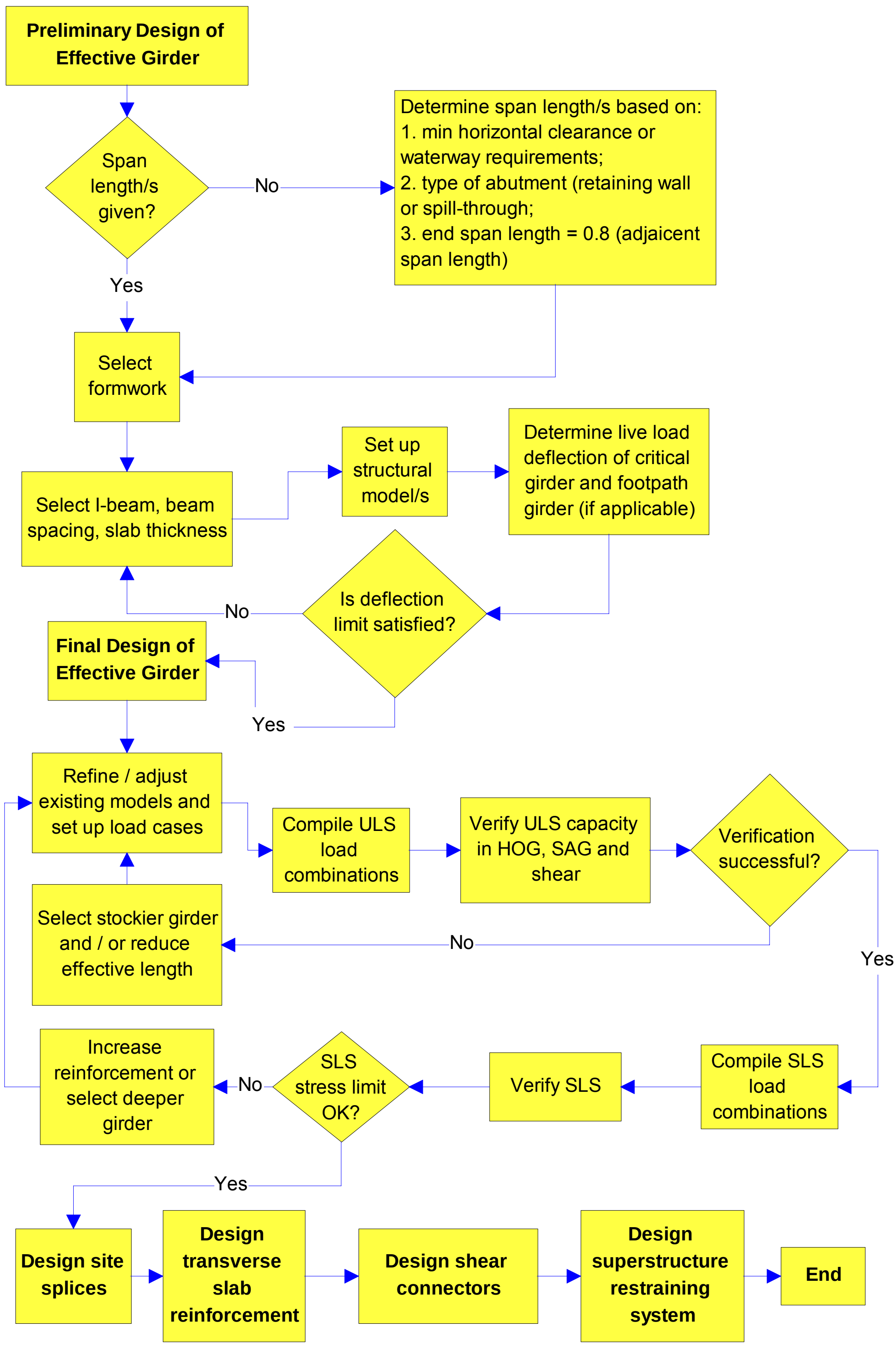


Figure 12.6 - COMPOSITE DESIGN FLOWCHART

CHAPTER 13

STEEL DESIGN

CONTENTS

13.1 INTRODUCTION

This Chapter is intended to provide some guidance on the design and detailing of steel structural elements/members, that may form part of a bridge superstructure or substructure. Hence, this Chapter is complementary to and shall be read in conjunction with both Chapter 12 of this Manual “Composite Design” and Part 6 “Steel and Composite Construction” of the Australian Standard Bridge Design AS 5100 (CODE).

This Chapter does not, however, cover the following:

- Cold formed members other than those complying with AS 1163;
- Steel members for which the value of yield stress used in design exceeds 450 MPa; and
- Steel elements, other than packers, less than 4 mm thick.

13.2 TYPES OF STEEL STRUCTURES

13.2.1 Steel I – Beams

Steel beams are available as rolled or welded sections. The OneSteel (ex BHP) Hot Rolled and Structural Steel Products Catalogue contains the necessary information to assist the designer in selection of an appropriate section. In addition to these products, imported steel sections are sometimes available at competitive prices. Alternatively, a unique steel section specifically suited to a particular design can be fabricated. Modern specialised "beam line" facilities available in WA ensure that welded beams can be fabricated out of steel plates at a cost that is very competitive to OneSteel or imported steel sections. This also gives the designer an opportunity of optimising the section for a particular design. The final selection however, is likely to be left with the Contractor (in searching for the most cost-effective proposal) provided that this selection complies with the design.

It should be noted, that a number of the larger sized sections are only produced infrequently and stock may be limited. Hence pre-ordering is advisable.

Quite often “off the shelf” beams come in standard lengths shorter than those required by the design between the site splices. In these instances, the designer must review the proposed shop splice detail and specify the location of the shop splice away from any high stress zones. A fatigue analysis of the proposed detail may be warranted.

Rolled and welded sections come in Grade 300PLUS as standard, but stronger grades are also available. Higher-grade steel sections are only likely to be cost competitive when a substantial tonnage is required.

Standard plates are readily available in a large variety of plate thicknesses and lengths for Grade 250.

Some steel sections and plates are also available in different low temperature impact Grades of L0 and L15. The grade indicates the temperature down to which the specified Charpy V-Notch impact strength is guaranteed. L0 is down to zero degrees, L15 to –15 degrees. For the WA environment Grade L0 will generally suffice. However, should particularly low temperatures in a given area become a concern, selection of Grade L15 may be warranted. It should be noted, however, that it is more expensive and may not be readily available, so pre-ordering is essential.

The most common use of steel beams in bridges is in steel/concrete composite superstructures, covered in Chapter 12 of this Manual.

13.2.2 Steel Boxes

Steel boxes are mainly used in WA in steel/concrete composite footbridge superstructures, only occasionally in road bridges. The boxes provide a ready-to-walk surface with minimum requirement for falsework. The deck slab is cast insitu to form a composite cross section (provided that longitudinal shear transfer is ensured by an appropriate number of shear connectors). Steel boxes feature superior lateral torsional buckling performance, which is essential for cable stayed structures, where significant cable induced compressive forces are combined with large bending moments. Examples of this type of design include the Mitchell Freeway footbridges and Claisebrook footbridge over Graham Farmer Freeway in East Perth. Road bridges with steel box superstructures include the Lord Street Grade Separation in East Perth and the bridges carrying the Graham Farmer Freeway over the Mitchell Freeway in the Hamilton Interchange.

The designer must take into consideration how the box section will be fabricated, i.e. how it is to be welded. It is important to ensure that satisfactory access is available for welding inspection and site splices. This will be especially important around the bearing stiffeners. Difficult welds should also be avoided, e.g. external corner welds, which may have to be ground to achieve the correct shape. In addition, folding of the plates is a much cheaper option than welding and can be used in light boxes, provided the shape and the plate thickness are appropriate, i.e. a section which is constant or changes linearly and the plate thickness not exceeding 16 mm.

Shear lag and local plate buckling may have a significant effect on steel box design. The most cost-effective method of reducing the adverse affect of these phenomena is to increase the plate thickness. Then the potential increase in material cost is offset by a substantial reduction in the labour intensive welding of plate stiffeners or extra internal webs. Additionally, welding of plate longitudinal and/or transverse stiffeners may cause heat-generated distortion of the box section.

Figure 13.1 - CLAISEBROOK FOOTBRIDGE

13.2.3 Steel Columns/Towers

For steel columns and/or towers, MRWA has chosen to avoid the use of open sections. Instead welded steel box sections fully or partially filled with unreinforced concrete are generally utilised. There are two main reasons for the use of concrete infill:

- to improve resistance to lateral impact; and
- to prevent plate local buckling.

Composite action is not usually considered, but can be assessed in accordance with Part 6 Clause 10.6 of the CODE.

It should be noted that, when concrete infill is used, staging of the concrete pour is absolutely essential to avoid bulging of the column plates due to the high hydrostatic pressure of fresh concrete. Should staged concreting be impractical, use of internal plate stiffeners is an alternative.

13.2.4 Steel Bracing

Steel bracing is common and comprises angular sections assembled in a triangulated strut-and-tie system.

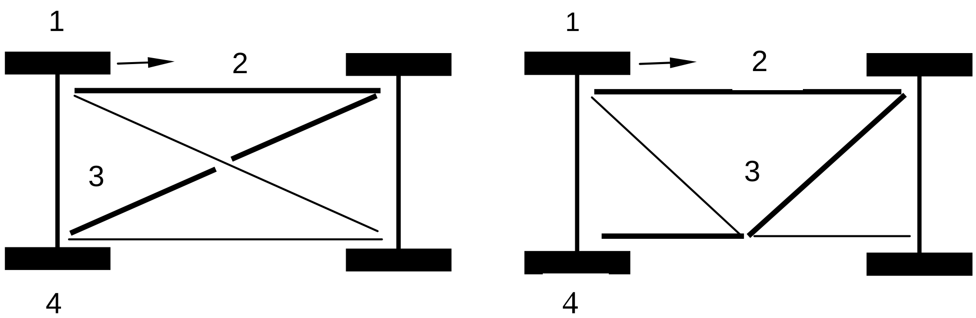


Figure 13.2 - EXAMPLES OF CROSS BRACING

Highlighted is a commonly adopted load path in the bracing, which includes:

1 - compression flange, prone to buckling;
2 - compression strut;
3 - tie;
4 - tension flange, serving as a restraint.

13.2.5 Sign Gantries

Sign gantries over wide roadways are generally configured as a double-legged portal structure comprising of either welded or cold formed rectangular box sections. Such a shape is perceived to be least visually intrusive, stable for wind loads (the critical gantry load case), and the most resilient to the aggressive effects of the environment (rainwater and dust).

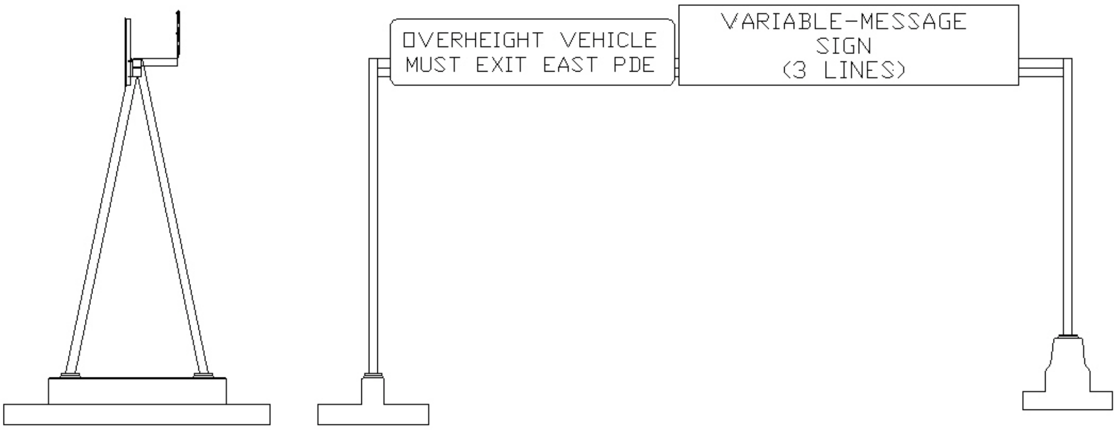


Figure 13.3 - TYPICAL PORTAL FRAME TYPE SIGN GANTRY (ELEVATION VIEW)

13.3 STRUCTURAL STEEL

13.3.1 Properties of Steel

The strengths of steels are given in Part 6 Table 2.1 of the CODE. The other properties that are essential in the design process are assumed to be invariable for all grades of steel, (see Part 6, Clause 2.2.5 of the CODE):

- Modulus of elasticity E = 200×10^3 MPa;
- Shear modulus G = 80×10^3 MPa;
- Poisson's ratio ν = 0.25; and
- Coefficient of thermal expansion $\alpha = 11.7 \times 10^{-6}$/°C.

All structural steel selected for the bridge design shall comply with the following Australian Standards and their latest amendments as appropriate:

- AS 1163 Structural Steel - Hollow Sections;
- AS/NZS 1594 - Hot-rolled Steel Flat Products;
- AS/NZS 3678 Structural Steel - Hot-rolled Plates, Floorplates and Slabs;
- AS/NZS 3679-1 Structural Steel - Hot-rolled Bars and Sections; and
- AS/NZS 3679-2 Structural Steel - Welded I Sections.

Certified mill test reports should be requested for all structural steel. Where unidentified steel is used, the yield stress shall be taken as no greater than 170 MPa, and the tensile ultimate strength as no greater than 300 MPa.

13.3.2 Fasteners

Steel bolts, nuts and washers and screws shall conform to the following Australian Standards:

- AS 1110 ISO Metric Hexagon Bolts and Screws – Product Grades A and B;
- AS 1111 ISO Metric Hexagon Bolts and Screws – Product Grade C;
- AS 1112 ISO Metric Hexagon Nuts;
- AS 1237 Plain Washers for Metric Bolts, Screws and Nuts for General Purposes;
- AS/NZS 1252 High Strength Steel Bolts with Associated Nuts and Washers for Structural Engineering; and

- AS 1420 ISO Metric Hexagon Socket Head Cap Screws.

13.3.3 Welding

All welding shall comply with AS/NZS 1554.1 Structural Steel Welding - Welding of Steel Structures.

13.4 DESIGN OF CONNECTIONS

13.4.1 General

Connections used on bridge steel structures are designed in accordance with Part 6 Clause 12 of the CODE.

The general simple rule in designing connections is that they should match the capacity of the actual members they connect. It must be noted, however, that there are also minimum design actions on connections detailed in Part 6 Clause 12.3.1 of the CODE to be taken into account.

There are both workshop and site connections/splices. While welding is the most common type of workshop splices, bolting is preferable for site splices, as it does not damage the steel protective coating, does not require a highly skilled workforce, and is not influenced by the weather.

Due to significant variation in the stiffness of bolted connections and welds, connections combining welds and bolts in one assembly should be avoided.

13.4.2 Design of Bolted Connections

Bolted connections are designed in accordance with Part 6 Clause 12.5 of the CODE. This includes both friction-type and bearing-type connections, comprising of the following bolting categories:

- Category 4.6/S, referring to commercial bolts of Strength Grade 4.6 (nominal tensile strength of 4 x 100 = 400 MPa; nominal yield stress of 0.6 x 400 = 240 MPa). Symbol “S” stands for “snug-tightened”, essentially meaning use of a standard wrench.
- Category 8.8/S, referring to any bolt of Strength Grade 8.8 (nominal tensile strength of 8 x 100 = approximately 800 MPa; nominal yield stress of 0.8 x 800 = approximately 640 MPa), snug-tightened.
- Category 8.8/TB and 8.8/TF. “8.8/T” refers to Grade 8.8 structural bolts fully tensioned in a controlled manner. Symbol “B” stands for bearing, where essentially no reliance is made on friction between the contact surfaces, caused by bolt clamping, to withstand the design actions. Symbol “F” means that at SLS the design actions are transmitted completely by friction between the contact surfaces. At ULS both TF and TB categories are designed to rely entirely on the bolt bearing capacity.

Note that both TB and TF Categories require a controlled tension at installation. Methods of bolt tensioning are described in Part 6, Appendix H Clauses H2.4 and 2.5 of the CODE.

The advantage of using a TF type connection lies in the better control of fatigue in the connected elements. This is because the forces in the joint are taken by friction between the contact (sometimes called faying, or mating) surfaces, thereby completely removing the high local stresses in the vicinity of the holes – the stress concentrators. The holes in a TF type connection are slightly oversized compared to TB type connections (2 – 3 mm larger than the bolt diameter). A typical bolted site splice using Category 8.8/TF bolts is shown in Figure 13.4.

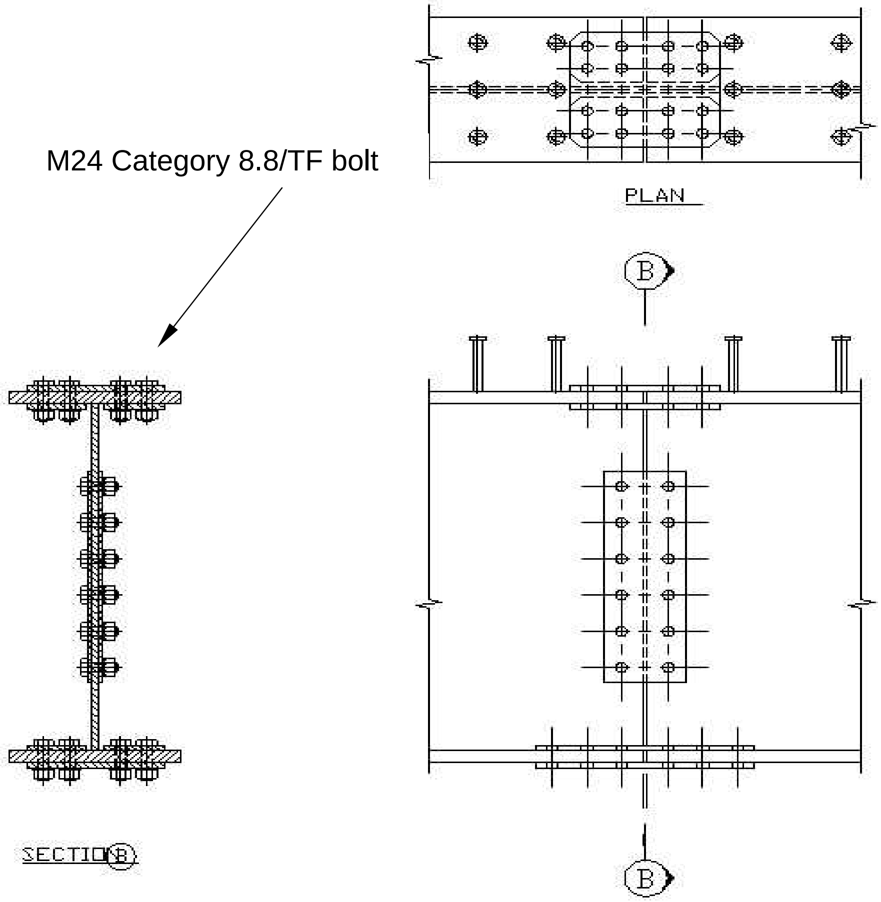


Figure 13.4 - TYPICAL BOLTED SITE SPLICE

13.4.3 Bolting Connections of Category 8.8/TF in Shear

The no-slip requirement addressed by Part 6 Clause 12.5.4.1 of the CODE is applicable to serviceability load effects only - it would be totally unrealistic to expect the no slip condition to continue under ultimate loads.

The number of bolts for a TF type connection is, therefore, calculated to satisfy both the SLS and the ULS, and that which requires the largest number of bolts is selected.

In the SLS analysis, treatment of the contact surfaces (reflected in μ, the slip factor) has a significant effect on the resultant shear resistance of the connection. The following values may be adopted:

Surface Treatment	Average Slip Factor
Uncoated:	
Clean as-rolled	0.35
Flame cleaned	0.48
Abrasive blasted	0.53
Painted:	
Inorganic zinc silicate	0.50
Hot-dip galvanised:	
Clean as galvanised	0.14
Weathered	0.20
Wire brushed	0.31
Grit blasted	0.31

Table 13.1 - SUMMARY OF SLIP FACTORS*

* Sourced from AISC "Bolting of Steel Structures" 3rd Edition by A Firkins and T J Hogan; and "After-fabrication Hot-dip Galvanising to introduce AS/NZS 4680" by Galvanisers Association of Australia.

13.4.4Modern Alternatives to High Strength Bolts

As an alternative to bolts, HUCK ® fasteners are gaining more and more popularity amongst Contractors, especially for jobs where a substantial number of high strength friction grip connections have to be assembled. A HUCK bolt/rivet is essentially a high strength Category TF fastener, which requires special tooling to install, but is extremely reliable in connections subject to high fluctuating loads. These fasteners are available in a variety of forms and diameters (maximum 20 mm).

Figure 13.5 - Metric HUCK-SPIN ® Fastener

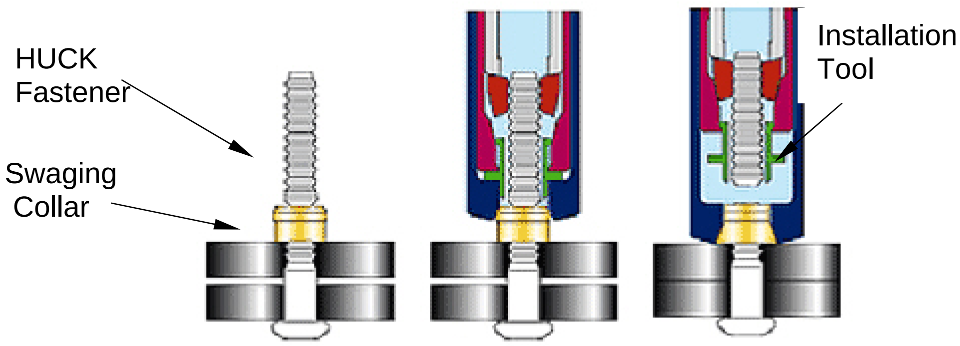


Figure 13.6 - INSTALLATION SEQUENCE

For more details on HUCK bolts refer to www.huck.com.

13.4.5 Design of Welded Connections

Weld types common in bridges are fillet welds, butt welds, or a combination of both called a compound weld. Butt welds can be full penetration or partial penetration welds. There are also slot or plug welds being a variation of a fillet weld. The weld quality is designated as either structural purpose (SP), or general purpose (GP) welds. Bridge structures usually require SP welds in accordance with AS/NZS 1554.1. The welding of shear studs shall be in accordance with AS/NZS 1554.2.

Weld design is covered by Part 6, Clause 12.6 of the CODE.

(a) General

Welding is a process in which two metal components are fused together using molten metal. A good weld has a good mix between electrode and parent metal and minimal ingress of impurities within the weld.

Hand welding is versatile but expensive. Semi-automatic or fully-automatic welding is cheaper but less versatile.

In weld design, important points to remember are:

- ***Butt Welds*** - there are two types of butt welds:

  Complete penetration butt welds are assumed to be of equal strength to the parent metal, provided non-destructive testing has confirmed achievement of the required quality. Generally, for plates thicker than 16 mm edge preparation is recommended, either a single or double Vee.

  Incomplete penetration butt welds (Clause 12.6.2.4 (b) of the CODE) are only allowed for longitudinal joints to connect the elements of built-up members. In this weld, the weld metal does not penetrate the full depth of the joint.

  Butt welds must be installed with the use of run-on and run-off tabs to ensure the same quality of the weld throughout.

- ***Fillet Welds*** - the weld material is placed into the corner created by two adjacent plates. The weld size is specified by the length of the weld leg, t_w. Failure occurs through the thinnest dimension of the weld, designated as throat thickness t_t. Fillet weld design is either carried out by nominating the size of the weld and calculating its required length, or, wherever the weld length is set, by calculating the minimum required weld size.

  It is important to note that fillet welds with t_w larger than 8 mm require multiple welding passages, and, hence, are rather expensive and may cause some distortion in the steel members by recurring heat.

- ***Plug and Slot Welds*** - are a type of fillet weld around the circumference of a hole or slot, or where the hole or slot is filled with weld material. These welds are designed in accordance with Clauses 12.6.5 and 12.6.7.3 of the CODE.

The strength of a weld is determined by both the parent metal and the weld metal. Hence, appropriate designation of welding materials is essential. In other words, the metal in the selected electrode or wire should at least match the strength of the steel elements being welded.

Welding consumables are specified as follows:

- for Manual Arc Welding E41XX or E48XX electrodes are specified, where E designates manual arc welding, 41 represents the nominal tensile strength of the electrode F_{uw} (in this case it is 410 MPa), and XX is usually specified by the fabricator as a two digit code for weld type eg. vertical up, overhead, etc. Manual arc welding shall be in accordance with AS/NZS 1553.1.

- for Automatic Welding W40X or W50X is specified, where W designates the automatic welding process, 40 represents the nominal tensile strength of the electrode F_{uw} (in this case 410 MPa) and X is again usually designated by the fabricator for the type of welding - submerged arc (to AS 1858.1), flux cored arc (to AS 2203), or gas metal arc (to AS/NZS 2717.1).

(b) Weld Inspection/Examination

Completed welds are inspected/examined in accordance with AS/NZS 1554.1. The usual procedures are incorporated into MRWA Standard Specification 830 "Structural Steelwork" and include:

- Visual Scanning;
- Visual Inspection;
- Radiographic Testing;
- Ultrasonic Testing;
- Magnetic Particle Inspection;
- Dye Penetration Testing;
- Hardness Testing; and
- Micro-structure Replica Examination.

(c) Detailing of Welds

Welds are detailed on the structural drawings in accordance with AS 1101.3 "Welding and Non-destructive Examination". Weld symbols and notes should be consistent throughout.

It is important to emphasise that careful consideration needs to be given to the detailing of the connections in the design phase as this will contribute significantly to ease of construction and reduce the chance of a fatigue failure.

13.5 DESIGN FOR FATIGUE

In addition to satisfying the basic design criteria, members and connections subjected to reversal of stress must be designed for fatigue. There are two types of fatigue – high cycle and low cycle fatigue. High cycle fatigue is that which occurs as a result of a large number of stress cycles with stresses substantially below f_y. Low cycle fatigue failure occurs when the steel stresses exceed the value of f_y over a limited number of cycles. The former is directly relevant to bridges, where the extreme load cases assumed for design occur very rarely, but moderate levels of load occur frequently. Low cycle fatigue is not common, since the CODE directs that the SLS stresses must not exceed $f_{y.}$ In other words, the structure must remain elastic for any conceivable combination of service loads.

Fatigue cracks are usually initiated at points of high stress concentration. The stress concentration may be caused by or associated with boltholes, welds, defects in materials or abrupt changes in the geometry of a member. Members subjected to fluctuating loading should be detailed such that stress concentrations are either completely removed or made as small as practicable.

Steel structures fatigue analysis is covered by Clause 13 of the CODE, except for the following:

- Reduction of fatigue life due to corrosion or immersion;
- High stress - low cycle fatigue;
- Thermal fatigue; and
- Stress corrosion cracking.

Although directly related to structural failure, a fatigue analysis is performed for serviceability load effects. The number of cycles and the live loads, which may cause stress fluctuations resulting in fatigue failure, are specified in Part 2 of the CODE.

The following procedure is recommended for a bridge fatigue assessment:

1. identify possible stress concentrators;
2. calculate the minimum and maximum live load effects at the identified location;
3. determine the stress range in those concentrators;
4. if this stress range exceeds the “exemption level” (Clause 13.4), determine the applicable stress category;
5. perform fatigue analysis (generally constant stress range analysis in accordance with Clause 13.7.2 is sufficient).

No matter how elaborate the fatigue analysis is, it is impossible to overestimate the importance of careful detailing in order to avoid obvious stress concentrators. For example, using friction grip type connections in lieu of bearing type connection, grinding butt welds smooth, “flattening” or “concave rounding” of transverse fillet welds in high stress zones, etc, will significantly reduce the likelihood of a fatigue failure.

CHAPTER 14

TIMBER BRIDGE REFURBISHMENT

CONTENTS

14.1 INTRODUCTION

This Chapter provides a brief overview and background to the design of refurbishment works to existing timber bridges. This is a subject not explicitly covered by text books, especially the specific repair/strengthening details which have been developed by MRWA. Detailed methodologies and requirements are provided in the following MRWA publications which should be read in conjunction with this Chapter:

- Load Rating and Refurbishment Design Manual for Existing Timber Bridges
- Structures Engineering Practice Notes
- Timber Preventative Maintenance Standards
- TIMBAR Procedure Manual

14.2 GENERAL REFURBISHMENT METHODS

There are approximately 1450 timber bridges in Western Australia, of which about 360 are on Highways and Main Roads. Many of these bridges are deteriorating due to weathering and fungal attack, and refurbishment methods have been developed to preserve and where necessary, strengthen these structures.

The main method of prolonging the life of a timber bridge is by placing a reinforced concrete overlay onto the deck. The main effect of the concrete overlay is to provide a weatherproof deck to protect the timber decking and stringers, and it also increases stiffness and reduces vibration. As the roadway width of most timber bridges is below current standards, the bridge is often widened at the same time, and this is incorporated into the concrete overlay. The widening also provides a “veranda” effect for the outside timber stringers, the most exposed bridge members, thus reducing weathering of the timber. Under these conditions, the life of the bridge timbers is prolonged significantly.

Early concrete overlays were designed as thin, flexible running surfaces providing a weatherproof cover to the timber deck. Consequently many of the overlays carried out in the early 1970s are now failing, having badly cracked due to their flexibility under traffic loading, and are leaking water. Concrete overlays designed today are thicker and contain more steel reinforcement than the earlier overlays. With increasing guardrail requirements, another important consideration for the concrete overlay is to provide adequate connection for a guardrail appropriate to the current standards.

Load tests of bridges with concrete overlays indicate that partial composite action may be developed (although this effect is not included in the design).

Repairs to the superstructure (i.e. stringers) and substructure (i.e. piles, halfcaps, wingwalls, etc) are also a major consideration when refurbishing a bridge. It is preferable that this work is carried out together with the concrete overlay.

As nearly all timber bridges are of identical construction (refer Appendix A for typical timber bridge details) most repair and concrete overlay details have been standardised. Structures Engineering has prepared a manual of standard details called the Structures Engineering Practice Notes, which is issued as a controlled document. This manual should be referenced for more detailed information on the standard repair and concrete overlay details discussed below.

14.3 DESIGN OF TIMBER BRIDGE REPAIRS AND REFURBISHMENT

14.3.1 Background

The design philosophy adopted by Structures Engineering for timber bridges is that the loading should be treated the same as for other bridge types, such as reinforced concrete and prestressed concrete bridges, except that timber bridges are to be analysed using the working or allowable stress method rather than the Limit State method of analysis used for concrete and steel bridges.

Although Australian Standards updated the Timber Structures Code to limit state design in 1997, MRWA has chosen not to adopt this Standard but instead has documented its own process and procedures in the Load Rating and Refurbishment Design Manual for Existing Timber Bridges (6706-02-2227).

The MRWA methodology of designing and load rating timber bridges is based on AS 5100 for general bridge design principles and design loading, in conjunction with the now superseded AS 1720 1988 Structural Timber Code, with some modifications, variations and guidelines as set out in Load Rating and Refurbishment Design Manual for Existing Timber Bridges (LRRDM).

14.3.2 General Requirements

In general, timber bridges are analysed as simply supported structures, with any continuity which may result from corbel to stringer interaction being ignored. Timber bridges with concrete overlays are also designed as simply supported structures, as the concrete overlay is assumed to be cracked over the piers. This is ensured in practice by saw cutting a crack initiator over the piers soon after the concrete has reached initial set, and before shrinkage and creep effects become significant. This is usually within 24 to 48 hours after the concrete has been poured.

Composite action is not included in the design, as the longitudinal shear transfer at the concrete overlay to timber deck interface necessary to develop composite action cannot be relied upon, either initially, or in the long-term.

The design approach used follows the same logical procedure as for other bridge types. The only difference is that the dead and live loads are unfactored, except for the DLA which is applied to live loading only. These load effects are then used to calculate bending moments, shears and reactions, and the resulting stresses in the timber elements. These are then compared to the allowable stresses shown in the LRRDM.

The methodology can be expressed in a simple form by the following equation:-

$$\mathbf{(DL + LL \times (1+DLA))_{stresses} \leq F_a} \quad \text{.............................}\mathbf{(1)}$$

where
- DL = dead load
- LL = live load
- DLA = dynamic load allowance
- F_a = allowable stresses (refer Section 14.4 below)

AS 5100 specifies a minimum distance from the edge of the design lane to the nearest wheel for T44 and M1600 loading. For refurbishment designs this approach is generally followed. However when construction stages are checked and lane widths have to be reduced, the minimum distance is often reduced to an offset of 200 mm (half wheel width).

14.4 ALLOWABLE TIMBER STRESSES

14.4.1 Introduction

In deriving the allowable stresses for the timber elements, the correct species of timber in the existing structure must be used. This will generally be given in the Detailed Inspection Report for the bridge. Jarrah is the most common type of timber found in WA timber bridges, with the next most common being wandoo and then karri.

As a guide for differentiating between jarrah and wandoo at a bridge site, the jarrah species can be identified by its fine parallel grain running the length of the member. Wandoo has wavy grain, giving it a gnarled appearance. Also, the bark of jarrah logs tends to separate away from the trunk and the underlying timber has a clean, smooth look, whereas the bark of wandoo timber remaining on the logs takes on a crumbly, shaggy look. Karri is generally only used for halfcaps and is a light "yellow" colour internally, in contrast to the reddish internal colour of jarrah. Other timber types, such as blackbutt and yellow tingle, are much harder to identify. If these are mistaken for jarrah, the result is conservative, as these timbers are usually in a higher strength group.

Once the species of timber has been identified, it is then classified as seasoned or unseasoned. Existing timber elements in bridges will almost always be seasoned, as most existing timber bridges are reasonably old. Very few timber bridges have been built in the last two decades, and none are likely to be built in the future. This is due to the absence of quality timber of the right size, and the increased cost.

Refer to the Load Rating and Refurbishment Design Manual for Existing Timber Bridges (LRRDM) for guidance on the generally accepted allowable stresses for the various timber types in conjunction with the differing structural elements.

These stress grades have been determined on the basis that the timber elements are seasoned and in good condition. Therefore, they are a guide only and a pre-design site visit to the bridge is required to assess the actual condition of the bridge timbers as some stringers, piles and halfcaps, can be in a poor condition with numerous splits and knots, but otherwise drill solid. A heavy spiral grain or saturated timber may also reduce the timber grade.

Once the appropriate stress grade has been determined, the associated basic working stresses for bending, compression, tension and shear can be taken from tables. This information as well as additional modification factors such as for duration and stability is provided along with worked examples in the LRRDM.

Where the timber is deteriorated due to fungal attack, and some or all of the timber section is rotted or friable, the allowable stresses are further modified to account for the weakened timber. This is discussed below.

14.4.2 Stress Reduction Factors for Deteriorated Timber

In many instances, the Detailed Inspection Report will indicate that a particular bridge member has drilled with a proportion, or all of the section as dry rot or friable. Friable timber is timber in a stage of deterioration between solid timber and dry rot. At present there is no information available giving guidance on appropriate stresses for deteriorated timber (either rot or friable) but experience shows that it does take some load and should not be totally disregarded in design or load rating.

When elements with deteriorated timber are being analysed, the following approach has been adopted for calculating the section properties and allowable stresses:

i) for timber elements with some remaining solid or friable timber, any dry rot timber is ignored (i.e. considered as a void) in the calculation of the section properties to be used in the analysis.
ii) if one or more of the drillings indicate friable timber then the entire section (but not the entire element) is considered as friable. Friable timber is included with reduced allowable stresses as given below.
iii) in the unlikely event that a timber element has no remaining solid timber (i.e. drilled as all dry rot), then the whole section is taken as dry rot. This situation is highly theoretical and for most practical situations the member is either ignored (i.e. redundant) or replaced/strengthened.

The basic allowable stress is then modified (see LRRDM). In the absence of reliable test data, the stress levels given for deteriorated timber, have been estimated from engineering judgement.

14.4.3 Derivation of Shear Stresses

When calculating the shear stresses due to dead and live loading, for stringer ends and sawn timber elements, the sectional area resisting shear forces is multiplied by ⅔. The stringer ends are so treated because the squared off section required for seating the stringer onto the corbels results in the cross-section approximating a rectangular section.
The shear stress distribution in a rectangular section is not uniform and the average shear stress is ⅔ of the peak shear stress. The shear distribution in a circular section is uniform through the section.

14.4.4 Determining Timber Elements Requiring Analysis

It is important to note that in assessing an existing timber bridge, only deficient timber elements are investigated, to reduce the time taken to carry out load ratings and refurbishment designs. It is assumed that timber elements having sufficient remaining solid timber are structurally adequate provided that the structural configuration of the bridge (such as stringer diameters, spacing, span length etc) are within standard limits.

As a guide, the following remaining solid timber sections are considered structurally adequate and would normally not require further investigation for a load rating or refurbishment design:-

Jarrah stringers and piles:-

100 mm solid annulus, with a minimum diameter of 400 mm for stringers and a maximum span length of 6.3 m, and a minimum diameter of 300 mm for piles, with a maximum free length of 5 m.

Wandoo stringers and piles:-

80 mm solid annulus, with a minimum diameter of 350 mm for stringers and a maximum span length of 6.3 m, and a minimum diameter of 300 mm for piles with a maximum free length of 5 m.

Members with drillings showing the amount of solid timber remaining to be less than the above values are to be checked for structural adequacy under the appropriate dead and live loading.

14.5 TIMBER DETERIORATION

The fungi responsible for the rot of timber can only grow and attack the timber fibres in the presence of both moisture and oxygen. Where there is too little of one or the other, the rot cannot sustain itself. Thus for dry timber, where the internal moisture content is low, rot will generally not occur. This is evidenced by internal timber stringers of a bridge with a deck which provides a good weatherproof cover (i.e. concrete overlay) having no, or very little rot. Whilst the outside stringers open to the weather and subject to drainage outfall from poorly located deck scuppers, will almost always have some degree of rot within the timber.

Timber piles located in semi-permanent water, or ground adjacent to water where the soil is moist, are the most prone to fungal attack, as this environment tends to have the ideal mix of available oxygen and moisture. It is these piles that generally have rotted timber at the ground line, generally extending down to about 1 m below ground, below which the oxygen level is lower and rot cannot proceed.

For timber piles located in permanent water, there tends to be a lack of oxygen available below the waterline to sustain the rotting process, rot can still occur up to 1 m below the mean water level.

For piles in seawater, or saline river estuaries there is danger of attack from a variety of marine borers, pile protection is essential.

14.6 STANDARD DETAILS

14.6.1 General

Over the last decade Structures Engineering has developed a series of standardised details for various timber bridge repairs and refurbishments. These details have been enhanced and modified to achieve the most economical design utilising common materials and available construction expertise, whilst maintaining acceptable design standards. This continual improvement of the standards is now at the stage where they are issued as a controlled document, to allow updates and modifications to be sent to each document holder, enabling all users to have the most up to date and recent design details referred to as, the "Practice Notes".

The following sections discuss some of the more important and common repairs such as pile repairs, new steel stringers and halfcaps, and deck widening by cantilevering from the existing structure.

It should be noted that the Standard Details apply to "normal" situations, and standard or typical timber bridges. Each Engineer applying these details should check that the detail being used is applicable for each particular bridge and loading.

14.6.2 Pile Repairs and Replacements

It is quite common to repair or replace timber piles on bridges because these elements are in contact with the ground which is a very high risk area for rot and decay to occur.

Where a timber pile has deteriorated to the extent that it is under capacity for the design loads, then it must be repaired or replaced. A repaired pile incorporates a new steel column section joined onto the remaining solid timber pile below the ground, making use of the original timber foundation. Replacing a pile generally involves driving a new steel pile alongside the deteriorated timber pile. Figures PN30-2103 to PN30-2133 of the Practice Notes show typical timber pile repairs.

The remaining timber section of a repaired pile is generally retained if it is in good condition and where the height to the halfcaps exceeds 2.5 m. Where the height is less than 2.5 m, then the steel column section is extended all the way up to the halfcaps.

Axial load capacity is maintained by the concrete and steel column sections, and moment capacity by the concrete pot. This type of repair has been carried out on a large number of timber bridges over many years and has proved very successful and economical. The halfcaps need to be adequately propped during the repair works, and also, where possible, all traffic directly above the pile being repaired should be diverted to the other side of the bridge, and a speed limit imposed.

This type of repair has been designed to be equivalent to the original timber pile, and is capable of safely carrying an axial load of 500 kN, and a moment of 150 kNm, based on a height of pile above ground not exceeding 3.5 m.

Splitting of timber piles below the halfcaps is very common, due to bearing and seating problems at the pile notch. Splits can also occur at other locations along the height of the pile. Where such splits occur, the piles are banded with steel to radially compress the pile and close the split, and to prevent the split expanding under load.

14.6.3 Cantilever Widening

During construction of a concrete overlay, additional bridge width can be obtained at relatively small cost by cantilever widening on one or both sides of the bridge. Where widening is such that the reinforced concrete overlay cannot attain the required widening by cantilevering, then new steel stringers are installed supported by cantilevering the halfcaps with new steel channel sections. This is a cost effective solution and is quite common. A more expensive alternative is driving new steel piles and installing new steel halfcaps. This is usually only required when the widening is quite substantial, or is on one side of the bridge only. The Practice Notes show typical details of a cantilevered widening with new steel stringers and halfcaps.

The concrete slab widening is effected by the use of permanent formwork (typically Bondek) or in corrosive environments normal timber formwork will need to be used. The deck spans transversely from the centreline of the existing outer timber stringer or the new steel stringer, cantilevering past the stringer as required. The spacing between the existing outer timber stringer and the new steel stringer is generally kept to the maximum typically used for timber stringers, which is about 1.0 to 1.2 m.

Cantilevering of the concrete beyond the steel stringer is kept to a maximum of 500 to 600 mm, based on the capacity of the steel stringer/halfcap extension.

14.6.4 Guardrails

The kerb details are shown in the Practice Notes, and allows the existing substandard timber post and rail system to be replaced with a new steel post and Thriebeam guardrail system. The Thriebeam with top rail is only to be used if cyclists regularly use the bridge or the height from the bridge level to ground/bed level below the bridge exceeds 6.0 metres. Top rails on narrow bridges in rural areas can restrict the easy movement of wide agricultural machinery across the bridge and this needs to be weighed when selecting the guardrail system. The kerb to post fixings allow damaged posts to be replaced with little or no damage to the concrete kerb, and are therefore quick and economic repairs.

14.6.5 Stringer Strengthening/Replacement

Where a load rating analysis indicates that an existing stringer is under capacity, then the stringer is either replaced with a steel beam or a steel beam is placed alongside the deteriorated stringer. Replacing the stringer is usually preferred unless the stringer that is under capacity drills solid.

The standard steel replacement used in stringer strengthening repairs is the 410UB54 (although the beam size will need to be checked for each strengthening design). This size beam is of a similar size to the existing timber stringers, has similar stiffness and generally provides adequate strength. As the stringer height at the corbel is typically around 350 mm, the depth of the new steel stringer is reduced at the corbel seating to fit into the available space.

All steelwork used in timber bridge refurbishments is hot dipped galvanised for corrosion protection. All welding is special purpose structural welds and requires a qualified and skilled welder. As such, site welding is minimised with many of the steel connection details bolted.

14.6.6 Halfcap Strengthening/Replacement

New steel halfcaps can be installed either beneath the existing timber halfcaps, (halfcap strengthening), or installed to completely replace the timber halfcaps (halfcap replacement). The condition of the existing halfcaps determines which of these details is used.

Where the existing timber halfcaps are in reasonable condition then the new steel halfcaps are generally installed beneath the existing timber halfcaps with steel packers if required. The new steel halfcaps are bolted horizontally at each pile and both sets of halfcaps are bolted together vertically through the packers. The timber piles are notched to provide seating for the steel halfcaps, and may require supplementary steel plates or channel sections for this purpose if the piles are out of alignment and sufficient seating cannot be gained. Refer PN30-2328 of the Practice Notes for details.

Where the existing timber halfcaps are in poor condition then they are generally totally replaced by new steel halfcaps. The steel halfcaps are bolted horizontally to the pile (similar to strengthening) while angles are welded either side of the stringer and bolted horizontally to restrain it. For halfcap replacements, the corbels and stringers require propping and jacking to allow removal and installation of the halfcaps. Propping is obviously difficult for piers over permanent water and in these situations it is preferable to strengthen halfcaps if possible, rather than replace them.

14.7 LOAD RATING PROCEDURE

Following the detailed inspection of an existing timber bridge, the inspection report is forwarded to the Engineering Associate Bridges who carries out a separate site visit and visual inspection of the bridge with the detailed report available for reference. This enables an accurate summary report to be produced, which becomes part of the detailed inspection report. This is then passed to the Bridge Condition Manager who carries out work assessments for each bridge. It is at this stage that the need for a load rating analysis becomes apparent, when the extent of deteriorated timber elements is confirmed. The Bridge Condition Manager then requests the Engineer Bridge Loading to carry out a load rating.

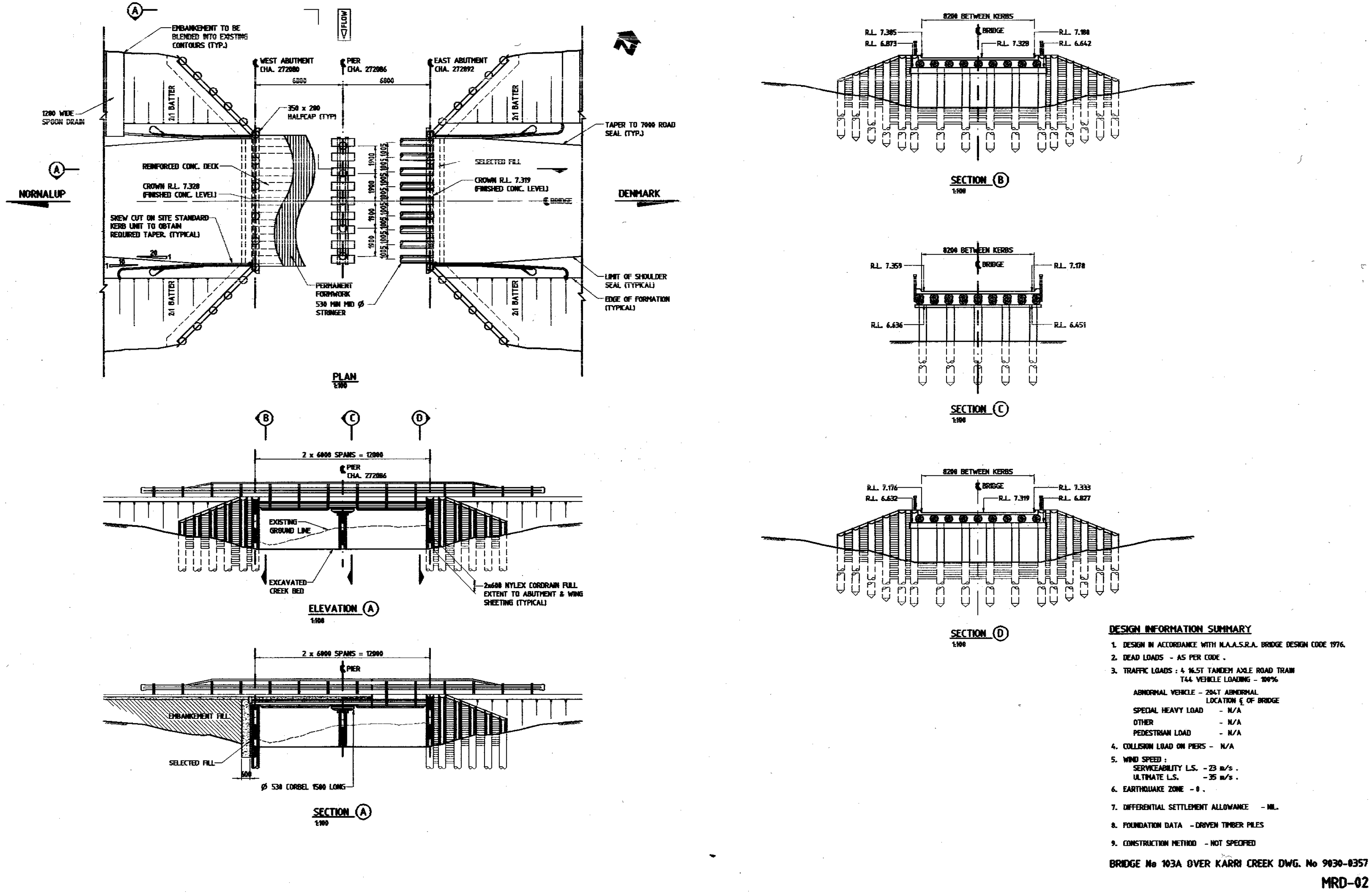
The LRRDM includes guidance on the load rating of timber bridges, and sets out the criteria and conditions to be used in the analysis. Reference should be made to this manual for full details on timber bridge load rating.

To assist in carrying out the load rating analysis, a computer program called TIMBAR and Excel spreadsheets have been specifically developed by MRWA. A Procedure Manual has been written to provide guidance on the use of TIMBAR and associated Excel spreadsheets.

APPENDICES

APPENDIX A Typical Timber Bridge Details

APPENDIX A


EMBANKEMENT TO BE BLENDED INTO EXISTING CONTOURS (TYP.)
FLOW
WEST ABUTMENT CHA. 272080
PIER CHA. 272086
EAST ABUTMENT CHA. 272092
1200 WIDE SPOON DRAIN
2:1 BATTER
350 x 200 HALFCAP (TYP)
TAPER TO 7900 ROAD SEAL (TYP.)
REINFORCED CONC. DECK
SELECTED FILL
CROWN R.L. 7.328 (FINISHED CONC. LEVEL)
CROWN R.L. 7.319 (FINISHED CONC. LEVEL)
NORNALUP
DENMARK
SKEW CUT ON SITE STANDARD KERB UNIT TO OBTAIN REQUIRED TAPER. (TYPICAL)
PERMANENT FORMWORK
530 MIN MID Ø STRINGER
LIMIT OF SHOULDER SEAL (TYPICAL)
EDGE OF FORMATION (TYPICAL)
PLAN
1:100
2 x 6000 SPANS = 12000
EXISTING GROUND LINE
EXCAVATED CREEK BED
2x600 NYLEX CORDRAIN FULL EXTENT TO ABUTMENT & WING SHEETING (TYPICAL)
ELEVATION (A)
1:100
EMBANKEMENT FILL
SELECTED FILL
Ø 530 CORBEL 1500 LONG
SECTION (A)
1:100
8200 BETWEEN KERBS
SECTION (B)
1:100
SECTION (C)
1:100
SECTION (D)
1:100
DESIGN INFORMATION SUMMARY
1. DESIGN IN ACCORDANCE WITH N.A.A.S.R.A. BRIDGE DESIGN CODE 1976.
2. DEAD LOADS - AS PER CODE .
3. TRAFFIC LOADS : 4 16.5T TANDEM AXLE ROAD TRAIN
T44 VEHICLE LOADING - 100%
ABNORMAL VEHICLE - 284T ABNORMAL
LOCATION ℄ OF BRIDGE
SPECIAL HEAVY LOAD - N/A
OTHER - N/A
PEDESTRIAN LOAD - N/A
4. COLLISION LOAD ON PIERS - N/A
5. WIND SPEED :
SERVICEABILITY L.S. - 23 m/s .
ULTIMATE L.S. - 35 m/s .
6. EARTHQUAKE ZONE - 0 .
7. DIFFERENTIAL SETTLEMENT ALLOWANCE - NIL.
8. FOUNDATION DATA - DRIVEN TIMBER PILES
9. CONSTRUCTION METHOD - NOT SPECIFIED
BRIDGE No 103A OVER KARRI CREEK DWG. No 9030-0357
MRD-02

CHAPTER 16

BEARINGS AND JOINTS

CONTENTS

16.1 INTRODUCTION

This Chapter is intended to provide guidance in the design and selection of bearings and joints for road and footbridges. This topic is also covered in the AS 5100.4 - Australian Standards Bridge Design Code, Part 4 (CODE). This Chapter follows a similar layout to the CODE, but concentrates on the types of bearings and joints used by Main Roads Western Australia (MRWA).

A very good general review of this topic is presented in a 2002 IABSE publication, Structural Engineering Document 6, *Structural Bearings and Expansion Joints for Bridges* by Gunter Ramberger.

This Chapter is only intended as a support document and reference must be made to the CODE when designing any Bearings or Joints.

16.2 GENERAL

16.2.1 Function

Bearings - provide the interface between the superstructure and the substructure of a bridge. They transfer loads from the deck to the supporting columns, abutments and foundations and permit relative movement between the two. Whilst little can be done to affect the vertical loads, the type of bearing chosen can have a significant influence on the horizontal loads transferred to the substructure and must therefore receive careful attention.

Joints - occur at the ends of bridge decks to provide a seal and a trafficable surface across the gap between the structure and the abutment. They can also occur at other openings in the bridge deck, e.g. where a structure comprises of a number of simply-supported spans.

16.2.2 Loads and Movements

Vertical loads on bearings result from the various loadings given in Part 2 of the CODE and are derived during the normal superstructure analysis. Horizontal loads on bearings come from both horizontal loads on the superstructure, e.g. wind, braking, stream force, earthquake and movements of the superstructure, e.g. temperature, creep and shrinkage. These are not so readily obtained, as they depend to a considerable extent on the type of bearing used, e.g. a free sliding bearing will only transfer a small frictional force, whilst the horizontal load transferred by an elastomeric bearing will depend on both the amount of movement and the shear stiffness of the bearing.

Elastomeric bearings are designed for serviceability limit state loads and movements, (CODE Part 4, Clause 12.4.1). It is impractical to design for ultimate loads and it is assumed that these can be taken by overstress in the bearing and that in the unlikely event of failure it would be non-catastrophic and the bearings could be replaced. Pot bearings are designed for ultimate limit state effects (CODE Clause 13.1).

Both bearings and joints must however be designed for the maximum possible movements as these are usually fairly easily catered for and the results from not allowing for this could be serious.

16.2.3 Replacement

It is important to make provision for possible future replacement and also for resetting if settlements are considered likely, e.g. typically for structures founded on spread footing on Perth sands. Provision is usually made by ensuring :-

- that the deck can be jacked up, preferably by providing space on the column top for jacks, but otherwise from the foundations; and

- that cast in sockets and retaining bolts are used to fix bearings rather than cast in studs and dowels, which make removal difficult.

Durability is an important aspect and all ferrous metal surfaces of both joints and bearings should be galvanised. Also, when detailing it is prudent to assume that joints will **not** be waterproof, as they very rarely are, and detail accordingly, with provision for drainage of the area beneath the joint.

16.3 BEARINGS

16.3.1 General

The different bearing types used by MRWA are described individually in this Section, but first a number of items common to all types of bearings are discussed:-

Alignment - Bearings are almost always installed level. In addition, with restrained or elastomeric bearings on curved or skew bridges, care must be taken that they are correctly oriented in the direction that movement will occur, e.g. for a curved bridge movement will be tangential out from the point of fixity. Also on wide bridges allowance must be made for lateral movements due to shrinkage and temperature.

Installation - Bearings are usually installed on mortar pads, either poured in place before installing the bearing or dry-packed under the bearing following positioning on temporary shims. It is important that the mortar is strong enough for the bearing stresses imposed and a non-shrink cementitious mortar with a minimum cube strength, (70 mm cube), of 40 MPa should be specified, or a similar epoxy mortar. Thickness of mortar should be a minimum of 25 mm, for ease of placing, with a maximum around 75 mm. Edge clearances between the bearing and the edge of the structural element supporting it should be such that the 45^{o} load-dispersion line from the edge of the bearing is adequately contained within the reinforced section of concrete.

Anchorage - The method of physical anchorage provided for bearings varies depending on the bearing type.

For elastomeric bearings MRWA has had evidence in the past of bearings "walking" under cyclical movements and have therefore used dowels to provide fixity. These are cast into the abutment or pier and protrude into the first layer of steel in the bottom of the bearing . The problem with this is two fold; it increases the effective shear stiffness of the bearing and also makes subsequent replacement difficult, as the structure has to be lifted significantly to clear the dowels. A better method of providing fixity is by the use of external stops bolted to the substructure, (see Figure 16.1), or placing the bearing in a shallow recess.

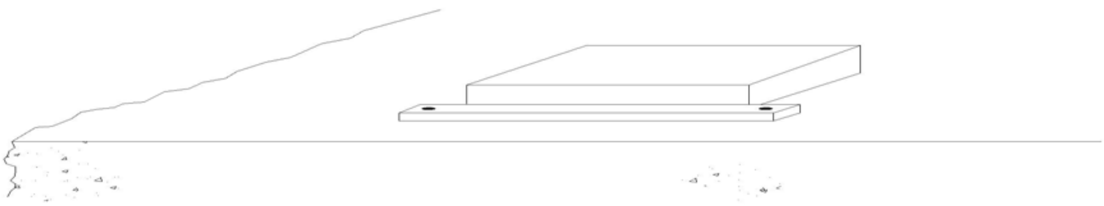


Figure 16.1 – Example of Fixing Detail

Pot bearings, are usually bolted in position using cast-in fittings. A cast in socket and bolt arrangement is preferred, for ease of removal, but with thin bearings there may be insufficient vertical clearance to get the bolts in. The CODE does permit reliance on friction only for pot bearings for incrementally launched bridges (Clause 10.1).

Replacement - See 16.2.3 above.

Friction - An important parameter in the use of sliding pot bearings is the value of the coefficient of friction. Values given in the CODE should be used, (Clause 11.2), with bearings always specified to be permanently lubricated. A design case that should be considered, but will only usually be important for incrementally launched bridges, is with the coefficient of friction at zero.

16.3.2 Elastomeric Bearings

Elastomeric bearings are the most common type used by MRWA. They consist of a multi-layered, bonded sandwich of steel plates and rubber. MRWA also uses strips of plain rubber, but only in non-critical applications, such as for the supports of approach slabs. Elastomeric bearings for bridges must have 2 or more layers of steel.

Elastomeric bearings are popular because they are easy to install, require no maintenance, are very forgiving if overloaded and have a long history of trouble free use.

It should be noted however, that they do not provide positive lateral connection. Therefore any lateral forces, e.g. from stream flow on a submerged deck, will have to be taken by other means, such as shear keys, dowels etc.

The design of elastomeric bearings is covered extensively in the CODE, but, since there is a large range of "standard" bearings available it is only very rarely that this process has to be followed.

Using the Tables in the CODE Appendix A, selection of the appropriate bearing is fairly straight forward. Knowing the maximum vertical load and anticipated movements and rotations, a number of bearings can be identified with differing thickness and shear stiffness, from which the most appropriate one can be chosen. Normally the thinnest (and therefore cheapest) would be chosen, but the effect of this on the substructure has to be taken into account as the thinner the bearing the stiffer it will be and therefore the more force will be transferred to the substructure from any deck movements.

The CODE also gives details of load testing for elastomeric bearings, (Clause 12.9 and Appendix D), however MRWA's test requirements are more stringent. Details are given in MRWA Tender Document Preparation (TDP) Specification 860, but basically they are:-

- Compression Test - as per CODE;
- Stability Test - This is additional to the CODE, and is required as a check on bearing fabrication. The test detects bearings with non-parallel plates, as these will tend to be

unstable and under a purely vertical load will deflect more in one direction than another;
- Proof Loading - This is an additional and fairly severe test involving simultaneous application of vertical and horizontal loads, with each varying over its full range; and
- Stiffness - The measurement of shear stiffness undertaken in accordance with the CODE.

16.3.3 Pot Bearings

These are the principal type of proprietary bearing used by MRWA. The advantages over elastomeric bearings are that they can support much higher vertical loads and they transmit much lower horizontal loads. The basic construction is shown in
Figure 16.2.

A pot bearing comprises:

- a base plate with a steel pot containing a rubber pad; and
- a piston bearing on the rubber with the top of the piston surfaced with PTFE which in turn bears against a stainless steel sheet fastened to the top plate.

The base plate is fixed to the substructure and the top plate to the superstructure.

Under load the rubber pad becomes semi-fluid and permits rotation about any axis. Low friction translation is effected by the stainless steel sliding on the PTFE. Bearings can be fixed against movement, guided to move in one direction only, or free to slide in all directions.

Pot bearings are individually designed by the manufacturer, based on the loads, movements and rotations specified by the designer.

Bearing design information specific to the structure is usually provided on a Bearing Data drawing which gives details of:

- Vertical load – maximum and minimum (SLS and ULS);
- Horizontal loads – maximum longitudinal and transverse (ULS); and
- Horizontal movements – longitudinal and transverse, maximum +ve and –ve; and
- Bearing orientation, fixing details etc are also shown.

Whilst pot bearings are not usually available “off the shelf”, most manufacturers have a "standard" range, which should be used wherever possible.

Design requirements for pot bearings, e.g. allowable bearing pressures, clearances, coefficients of friction, material properties etc are given in the CODE Clause 13. Specification details are given in MRWA TDP Specification 860 which also covers testing, which is generally in accordance with the CODE.

An interesting development with pot bearings is the use of load indicating bearings. These have load cells incorporated into the construction and can provide a direct read-out of the load on the bearing. This is especially useful for skewed bridges, as it enables a check to be made on the design assumptions, especially the load distribution. For further details refer to a paper by Graham Davidson to the 1991 Austroads Bridge Conference.

16.3.4 Other Types of Bearing

Although elastomeric and pot bearings are the principal types of bearing used by MRWA, others are available and are occasionally used for special purposes.

* ***Rocker and Roller Bearings*** - These are all metal bearings providing respectively for rotation only or rotation + translation. They are now rarely used because of problems with maintenance and corrosion, however they may still be found in older structures, e.g.: Perth's Causeway Bridge. Some information on these is given in the CODE Clause 15.3.

* ***Knuckle and Leaf Bearings*** - These are again infrequently used but do have an application as a high capacity fixed bearing for larger bridges, where pot bearings are insufficient. They would normally be purpose designed and made for each application. Details are given in the CODE Clause 15.4. Examples of use of such bearing include Mt Henry Bridge, (Drawing No. 7930-127-2).

* ***"Simple Bearings"*** - A number of manufacturers produce a very simple sliding bearing, comprising a thin, plain rubber pad with a bonded PTFE surface and stainless steel sliding plate, (e.g. *Granor Slipjoint*). These are low load bearings, (maximum 500 kN/m), and have only a limited rotational capacity, but they may have an application in footbridges and other small structures.

* ***Concrete Hinges*** - Concrete, (or Freyssinet) hinges are very specialised items which comprise a narrow throat of concrete, subject to very high compressive stress (up to 100 MPa). The concrete behaves in almost a semi-fluid manner and permits quite large rotations. They were used on the Narrows Interchange Bridges, (e.g. Bridge N/1 Drawing No. 4/1077), but are now not covered by the CODE and specialist literature should be consulted.

* ***Launching Bearings*** - For the incremental launching method of construction special bearings are used during launching. These usually comprise a reinforced concrete base topped by a machined steel bearing plate surfaced with a stainless steel sheet. Thin elastomeric bearing pads, faced with PTFE are fed continuously in-between the soffit and the bearing as the launch proceeds. For details refer to Mandurah Estuary Bridge Drawing Nos. 8430-0083 to 8430-0085.

* ***None*** - It should not be forgotten that having no bearings is always an alternative, and a cheap, maintenance free one. If the structure is small enough so that movements will only be small and/or the columns are flexible enough to take the necessary movements without inducing high bending stresses, then columns and/or abutments can be built into the deck and portal action used.

Useful information on any of the above, plus elastomeric and pot bearings is given in "Bearings in Structural Engineering" by J E Long.

16.3.5 Bearing Selection

When selecting a particular bearing for a job, the usual process is:-

* No bearings - check if this is an option, i.e. movements can be taken by deflection of columns. Failing that;

* Elastomerics - the first choice of bearing type, but depends on magnitude of vertical load (maximum around 4000-5000 kN) and space available. If not feasible;

* Pot Bearings - would be the likely other choice, as they are readily available with capacities up to 30,000 kN and can be specially made for even higher loads.

These standard bearing types should cover the great majority of cases, with more specialised types described at 16.3.4 above available as required required.

16.3.6 Commercial Availability

The standard range of elastomeric bearings is available from a number of suppliers, e.g. Ludowici, Granor, Advanx. Pot bearings are available mainly from Ludowici and Granor, with other companies occasionally tendering, e.g. PSC.

The supply of bearings is usually included in the construction contract for a bridge, or alternatively, to save time, they can be purchased in advance and supplied to the Contractor by the Principal. In either case, the designer must check the availability and time for delivery when selecting a bearing system.

MRWA TDP Specification 860 covers the supply and testing of both types of bearings.

16.4 JOINTS

16.4.1 General

Selecting the joints for a bridge is often thought of as the easy job you do when the hard, exciting bit of the main bridge design is finished. Just work out the movements, choose a joint from a catalogue and that is it. However, joints require much greater attention than this, because of all the features of bridges, this is the one that can create the most problems, both directly and indirectly.

Directly because joints often get full of dirt and so do not function properly; they may have open gaps or be slippery and so represent a danger to pedestrians or cyclists; or they may even come loose. Indirectly, because they are never 100% waterproof and so leak water, dirt and debris through onto the substructure (and bearings) beneath. Therefore, the proper design, detailing and installation of joints is essential to ensure they do not become a difficult, expensive and disruptive maintenance problem later.

Joints are provided in bridge decks to cater for movements between adjacent structural elements. This normally means between the deck, which can move, and the abutments, or approach slab, which does not. In addition, there could be joints within the bridge deck itself if simply supported spans are used. MRWA normally avoids these by using continuous spans, or at least a continuous deck slab, as more joints only means more possible problems!

16.4.2 Design

Deck joints have to be designed for the loads to which they will be subject and the movements they will have to accommodate.

Loads

Design loads are given in the CODE, however, it has been found difficult to apply normal design methods to joints, mainly because of the unknown magnitude and effect of impact loading. Being at the end of a bridge, level differences between the deck and the approaches and/or misalignment of the joint itself can result in high and unknown impact factors from wheel loads. It is therefore usual to design joints based on detailing rules (refer Clause 17.3.2 in the CODE), or past experience of what has been successful.

Movements

The main requirement when calculating the design movements for a joint is to ensure that the maximum amounts of all possible movement are included. Although longitudinal movements will obviously be the main ones, allowance must also be made for any transverse movement (due to temperature or shrinkage of a wide deck and/or effects from curved or skewed decks), and vertical movements/rotations (especially if there is an approach slab and the approaches are likely to settle).

Longitudinally, temperature, creep and shrinkage movements must be accommodated. Obviously, the joint must never close up completely or open excessively, therefore, it is essential that the worst possible combination of movements is considered, i.e. the hottest possible day occurring immediately after the joint is installed (so creep and shrinkage movements are a minimum) and the coldest possibly day after all the creep and shrinkage movement has occurred.

The other design consideration is to ensure drainage is properly allowed for. With a theoretically sealed joint a sloping bearing shelf with the bearings on a raised mortar pad may suffice, but with an open type of joint a positive drainage system will be required, usually some form of collection troughs, made from stainless steel as maintenance will be difficult (refer Mandurah Estuary Bridge Drawing Nos. 8430-0130 and 8430-0131 for typical details).

16.4.3 Joint Types

There are a large number of different joint types available, both purpose made and "off-the-shelf", the principal ones, especially those used by MRWA, are described below.

None

The first alternative, which should not be ignored, is to have no joints at all. Although of limited application, this automatically gets rid of a lot of the problems associated with joints, although unfortunately introduces a number of others.

If the bridge is short, so that anticipated movements are small, then the deck can finish with a simple curtain wall, which is allowed to expand against the surrounding dirt. MRWA uses this non-joint, typically in RC flat slabs and overlays, but restricts its application to bridges under 30 m long.

The main danger with this joint is that cyclic movement against the soil will result in a gap opening between the deck and the road and this will create a potential erosion problem.

For further details refer “Integral Bridges” England, Tsang and Bush - Thomas Telford, UK.

Cover Plates

Simple galvanised steel cover plates over an open gap are another possible almost 'non-joint'. They are obviously not waterproof so drainage will be required. They are used occasionally on footbridges and on footpaths where cover plates are used in conjunction with other types of joint (typically the rubber compression seal), to prevent pedestrians tripping in the joint. Where plate surfaces are used they shall be made from non-slip materials (e.g. checkerplate).

Sealant

A simple sealant joint (silicone, polyurethane, bitumen/rubber, epoxy/rubber, as examples), is possible for small movements, but all sealants have a limited life and so this type of joint should not be used in critical locations or where replacement will be difficult. Movement capacities are limited (usually a maximum of 25% of the joint width) and joint widths should be kept below about 30 mm because of dangers to pedestrians and cyclists with a wider gap and the risk of material getting embedded in the joint and jamming it. A compressible backing rod should always be used to improve the efficiency of the joint (refer Figure 16.3).

The “XJS Expansion Joint System” from Granor is a typical proprietary joint and is used for rehabilitation of traffic bridges having small movement.

Rubber Compression Seal

There are a number of joints available based on preformed rubber compression seals. MRWA has developed rubber compression seal around the Wabo range of seals, although similar seals available from other manufacturers, e.g. Miska. The joint uses steel nosing angles to reinforce the adjacent concrete and provided all the standard details are used, especially the angle size (125 x 75 x 10) and number and size of anchor straps, the joint should perform well and has an excellent long term record in Western Australia.

For typical details see Figure 16.4.

These joints are used to cater for movements of up to 50 mm, i.e. a maximum joint width of around 100 mm, as above this the joint gets too wide and becomes noisy and a possible danger to cyclists, (refer CODE Clause 17.3.5). They are used at both deck to abutment back wall and deck to approach slab joints.

A range of smaller, lighter duty rubber seals is also available from Granor, (the Fermaseal range) and these have application in small, lightly trafficked joints, e.g. footbridges, where they can be used without reinforcing angles and are a preferred alternative to the sealant type joint, (refer Figure 16.5)

Strip Seals

There is a large range of what could generically be called "Strip-seals" available from different manufacturers, e.g. Granor Type AC or Miska BJ6 single element joint, (refer Figure 16.6).

These joints cover a similar range of movement to the Wabo joint above. There are a number of possible problems with these joints, e.g. lack of self-cleansing, puncturing of the membrane and difficulties with replacement. To attempt to get over some of these issues MRWA will only accept the use of this type of joint with a multiple web seal not the simple single membrane seal.

Again there is a maximum size limit of around 100 mm as for rubber compression seals.

Finger Joints

For movements in the range 100-200 mm MRWA has normally used open finger joints, e.g. Mt Henry and Mandurah Estuary Bridges. Provided they are properly installed these can provide a simple, maintenance-free joint for large structures. However, they cannot be used where cyclists have access to the bridge, because of the danger of bicycle wheels being caught in the gaps between the fingers. Attempts have been made in the past to weld transverse metal straps across the joint to prevent this, but because of the problems with welding and the continuous impact from heavy traffic this has never proven 100% successful and if the straps come loose they can be dangerous.

Such joints are purpose made for each structure with the plate either cut from heavy, specially machined steel plate or cast. Correct detailing of the plates, the holding down bolts and the method of installation are vital. The design of the fingers and the holding down bolts should be such that the operational stress ranges are low and hence the risk of fatigue is minimised. Access for maintenance of the holding down bolts should be provided for in the design. For long concrete bridges, consideration must be given to the ultimate creep and shrinkage contraction of the bridge and the consequenctial increase in maximum the gap size. The allowance must be made in design to cater for simple and economical re-setting of expansion gap for the future maintenance work. The fingers must also obviously be capable of supporting a standard wheel load.

Typical details are shown in

Figure 16.7, but the above bridge drawings and specifications should be referred to for full information.

This is an open joint, so positive drainage is required underneath.

Another possible danger with this joint is that incorrect installation or subsequent settlement, can result in a sloping plate with the fingers sticking up proud of the road surface. However, notwithstanding this, the finger joint is an excellent, simple joint for larger movements. Due to its nature, the fingerplate joint does not generate a lot of noise in operation and compares favourably with modular joints.

Modular Joints

For even larger movements, or where finger joints cannot be used because of cyclists, the modular joint is favoured. This is a proprietary joint comprising a number of rubber compression seals, refer Figure 16.8. It is available from a number of manufacturers, e.g. Granor Wabo Modular Expansion Joint System and Miska D.S. Brown Steelflex System.

The modular expansion joint must not generate significant increase in average noise levels. If a modular expansion joint is to be used on a bridge located within a 2 km radius from residential development, then the joint must be fitted with a noise reduction system and must be approved by RTA.

This joint is capable of catering for very large movements, up to 800 mm, depending on the number of modules it contains. Design and installation is a specialist task and is the responsibility of the manufacturer. However, in recent times it has become apparent through in-service failure of certain modular joints throughout Australia that simply meeting the CODE requirements is insufficient to ensure a long service life for the modular joint. The Road Traffic Authority (NSW) have undertaken research on modular joints and have prepared specification standards that should be referred to when specifying modular joints.

With this joint, the actual sealing element can usually be either a rubber strip seal or a box type compression seal. Again MRWA prefers the box compression seal for durability and reduced maintenance.

Larger Joints

For even larger movements specialist joints are required, e.g. roller-leaf, and manufacturers should be consulted. It may even be necessary on very long bridges to divide the superstructure so that a number of smaller joints can be used.

16.4.4 Joint Selection

The selection of a joint for a particular structure is mainly dependent upon the movements expected, which in turn obviously depends on the bridge type and length. The following is provided as a very rough guide to preliminary selection of joint type for road bridges.

Bridge Length (m)	Joint Type

0 - 30	None
30 - 100	Compression Seal or Strip Seal
100 - 750	Finger Joint or Modular
> 750	Modular or Special

Special features, such as curvature and skew must also be allowed for.

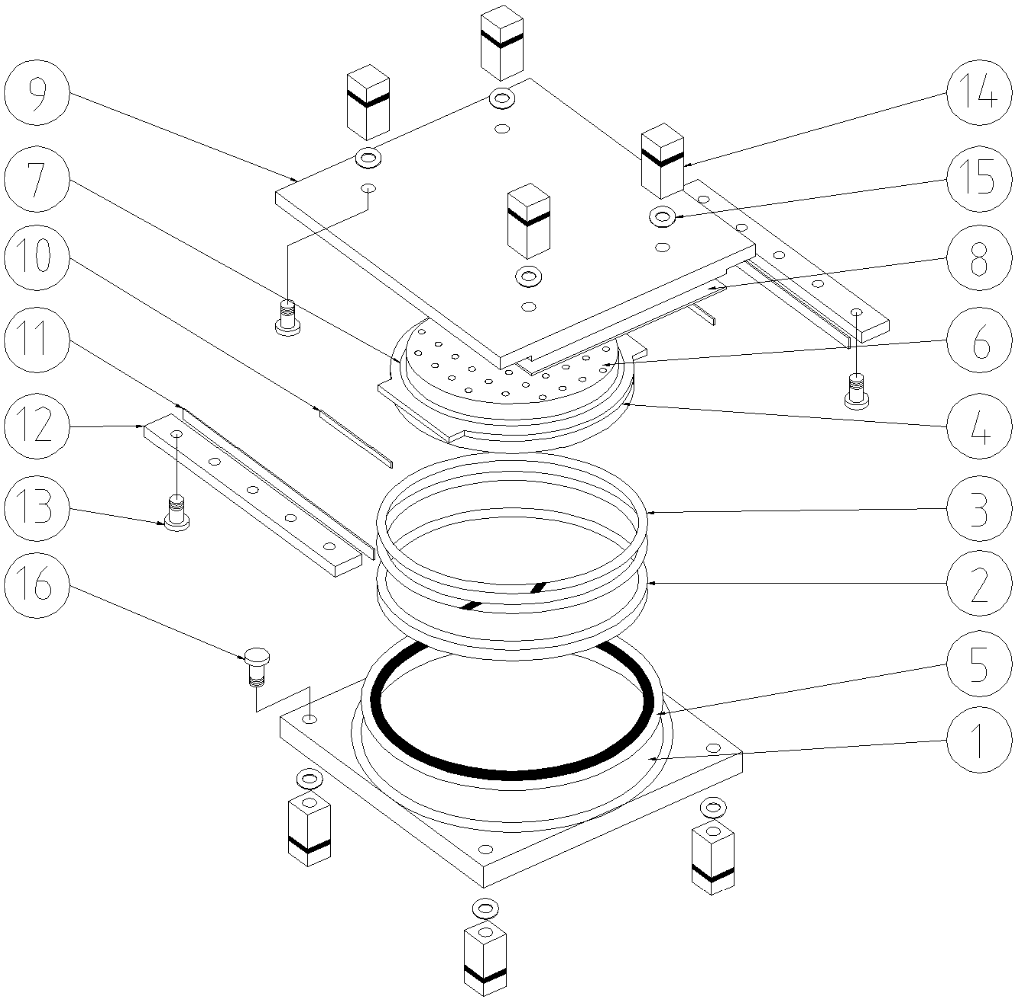
On large, or otherwise unusual structures it may be worthwhile going to a performance specification and letting the specialist manufacturers suggest the most appropriate joint.

Also, note that the above does not include concrete overlays on timber bridges. With these it has been found that expansion joints are not required for bridge lengths up to 30 m. Above this the rubber compression seal type of joint is used.

NB: A note of warning concerning the use of new, unproven joints. This should only be done with great caution because of the number of potential problems that can occur with joints. Any maintenance work will require road closure and disruption to traffic. On busy highways and freeways this can be a substantial imposition. Modular joints are typically manufactured in large sections and the replacement of one would require the closure of a number of lanes. A dollar saved during construction may lead to having to spend many dollars on maintenance or replacement later.

16.4.5 Commercial Availability

There are a number of manufacturers producing the different types of joints, e.g. Granor, Ludowici, Miska, ICL, Watson Bowman Acme. Unless there are special reasons why a specific joint must be used it is usual to call up a particular type on the bridge drawings, but with the proviso "or similar approved" and leave the supply of the joint to the Contractor.



No.	PART	DESCRIPTION
1.	POT	MILD STEEL
2.	ELASTOMERIC PAD	NATURAL RUBBER Z50
3.	SEALING RINGS	PHOSPHOR BRONZE
4.	PISTON	MILD STEEL
5.	ROTATION SEAL	SILICONE SEALANT
6.	BEARING SURFACE	P.T.F.E.
7.	DUST SEAL	EXPANDED POLYETHYLENE
8.	SLIDING SURFACE	STAINLESS STEEL
9.	TOP PLATE	MILD STEEL
10.	BEARING SURFACE	ACETAL
11.	SLIDING SURFACE	STAINLESS STEEL
12.	GUIDE	MILD STEEL
13.	BOLT	HIGH TENSILE CAPSCREW
14.	SOCKET	MILD STEEL
15.	WASHER	NEOPRENE
16.	BOLT	HIGH TENSILE HEXAGON BOLT

Figure 16.2 - TYPICAL POT BEARING

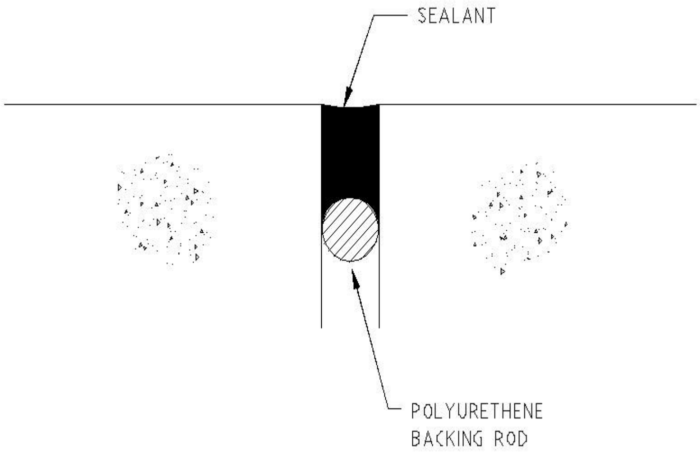


Figure 16.3 - SIMPLE JOINT SEAL

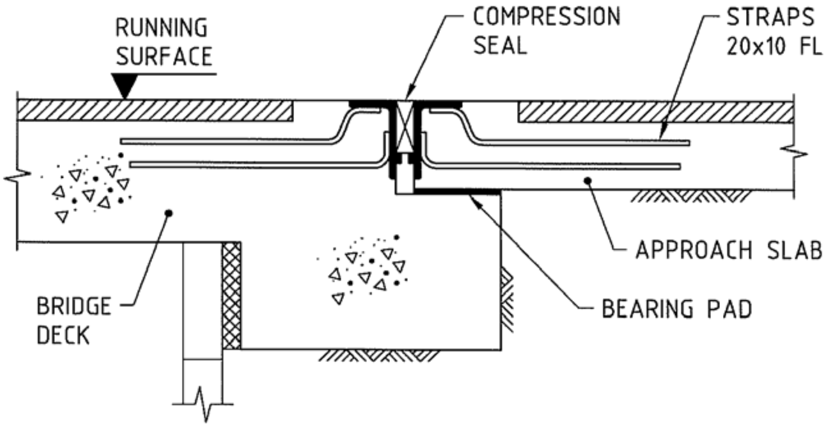


Figure 16.4 - RUBBER COMPRESSION SEAL

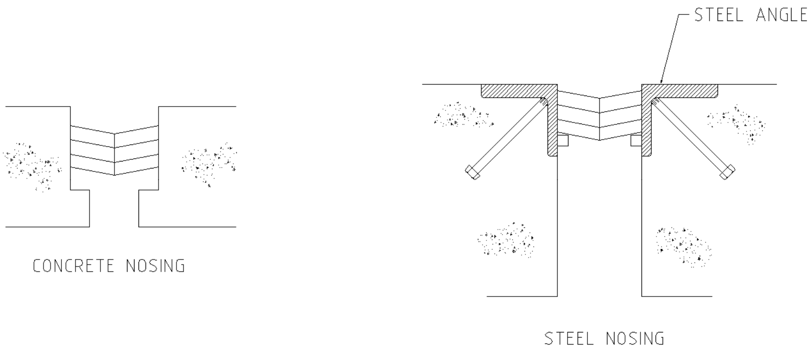


Figure 16.5 - GRANOR FERMASEAL

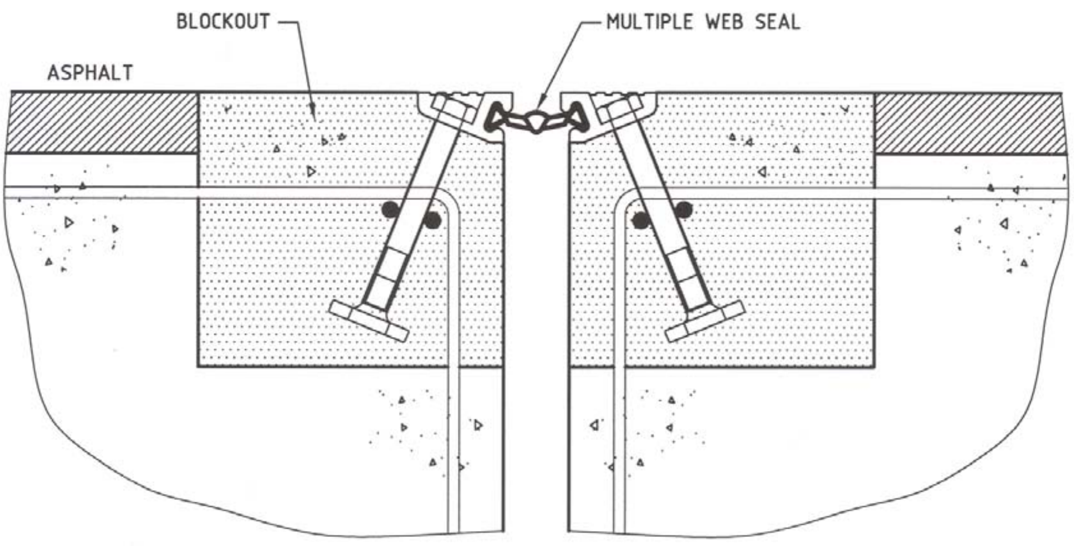


Figure 16.6 - STRIP SEAL

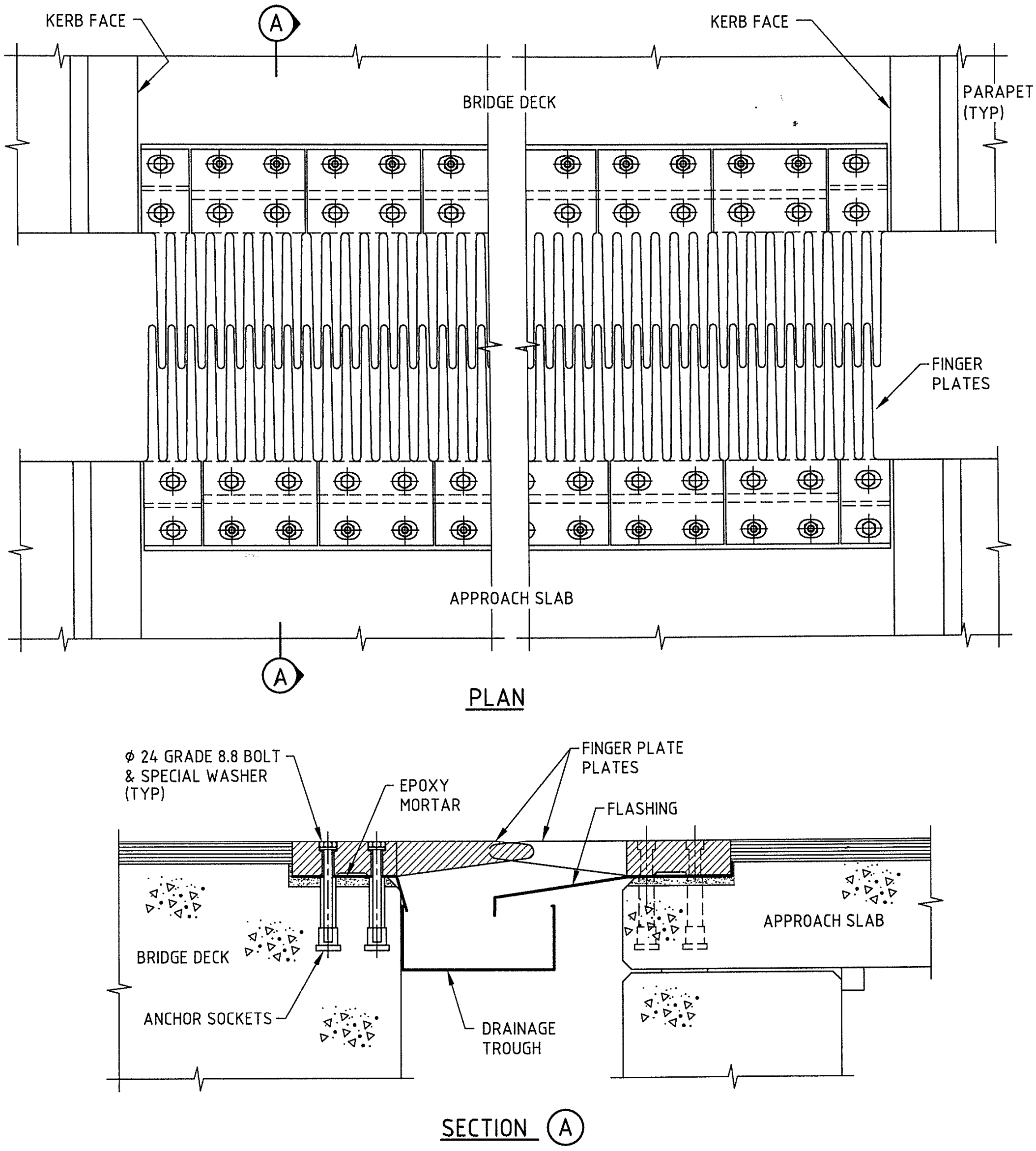


Figure 16.7 - FINGERPLATE JOINT

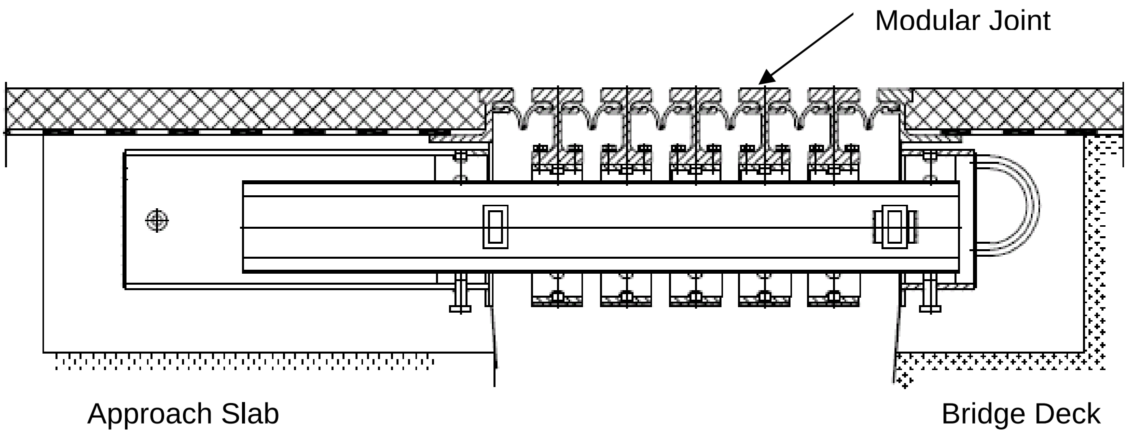


Figure 16.8 - MODULAR JOINT

CHAPTER 17

BARRIERS & PARAPETS

CONTENTS

17.1 INTRODUCTION

This chapter aims to provide information on the function, level of service, design and specific types of bridge barriers and parapets used by Main Roads WA. In addition a review of Australian and overseas literature has provided a summary of policy and practice that should help to promote discussion and clarify issues on the use of traffic barriers associated with bridges. Reference to research documents and codes is made within the text. Refer to the Bibliography and the Main Roads Library for further details.

Basically two types of barriers exist on bridges, these being:

- traffic barriers; and
- pedestrian barriers.

Pedestrian barriers are typically the post/baluster type. Traffic barriers however come in a variety of configurations to suit particular circumstances, including flexible, semi-rigid, rigid beam and post, and rigid concrete barriers.

Parapet panels are usually located along the outside edges of a bridge and sometimes extend along abutment embankment wingwalls.

17.2 PEDESTRIAN BARRIERS

Pedestrian barriers on road bridges and footbridges are required to safeguard pedestrians and/or cyclists.

The Australian Standard Bridge Design AS 5100 (CODE) Part 1 specifies geometric requirements in Clause 12 and Part 2 of the CODE specifies design loads in Clause 11.5.

The minimum railing height for pedestrian only footbridges is 1100 mm. Where cyclists may use the footbridge, e.g. DUP, minimum railing height shall be 1300 mm.

If the footbridge is required to meet ACROD/Australian Standards for people with disabilities then there are additional requirements - see AS 1428.1 for details.

MRWA does have a standard design detail for bridge balustrades suitable for paths wherein there is the possibility of use by cyclists (Principal Shared Paths Drawings 0530-1087 and 0530-1307) based on the modification of a previous standard. This detail does not provide a separate handrail that would conform to AS 1428.1 requirements for handrails on a ramp and hence its application is limited to bridges with grades of not more than 1:50.

The surface finish on the barriers can be either a paint system for architectural requirements or galvanising for maintenance considerations. For successful hot-dip galvanising balusters must be made from solid bar as shown on the standard drawing.

17.3 TRAFFIC BARRIERS

The following section provides a review of MRWA existing traffic barriers used on bridges and compares current Australian and overseas practice. In addition the CODE is explained with respect to the application of the design principles and how they relate to MRWA standard bridge barriers.

Bridge railings are normally installed on bridges to prevent errant vehicles from running off the edge of the bridge or to protect pedestrians or other motorists in adjacent areas. Almost all bridges in WA have some form of bridge railing and only a very limited number of structures without bridge railings have been sanctioned by MRWA. These structures tend to be low-level flood crossings rather than “bridges” that are subject to frequent overtopping and tend to be located on low traffic volume roads.

Previous Australian bridge design codes (Austroads Bridge Design Code 1992, and the NAASRA Bridge Design Specification 1976), required bridge barriers to be designed for a given static load, at key locations on the railings and satisfy certain geometric requirements.

The AS 5100 Bridge Design Code (CODE) still provides static design loads, however these loads are only provided to allow the design of prototype barriers for crash testing, for the design of deck cantilevers supporting the barriers and designing the bolted anchorage connections. A new emphasis is given to the use of barriers that have been crash tested to a particular standard.

Both of the Australian Codes are based on crash testing requirements developed by the United States Transportation Research Board (National Cooperative Highway Research Programme (NCHRP) Report 350). Table 17.1 provides a comparison between AS/NZS 3845 and TRB Report 350.

Test Requirements to AS/NZS 3845					**Test Requirements to NCHRP 350**				
Test Level	Vehicle Mass (kg) & Type	Speed (km/h)	Impact Angle (Φ)	Centre of Gravity Height (mm)	Test Level	Vehicle Mass (kg) & Type	Speed (km/h)	Impact Angle (Φ)	Height of Centre of Gravity (mm)
0	820 C 1 600 C	50 50	20 25	550 550					
1	820 C 2 000 P	50 50	20 25	550 700	1	820 C 2 000 P	50 50	20 25	550 700
2	820 C 2 000 P	70 70	20 25	550 700	2	820 C 2 000 P	70 70	20 25	550 700
3	820 C 2 000 P*	100 100	20 25	550 700	3	820 C 2 000 P*	100 100	20 25	550 700
4	820 C 8 000 S	100 80	20 15	550 1 250	4	820 C 8 000 S	100 80	20 15	550 1 250
5	820 C 36 000 V	100 80	20 15	550 1 850	5	820 C 36 000 V	100 80	20 15	550 1 850
6	820 C 36 000 T	100 80	20 15	550 2 050	6	820 C 36 000 T	100 80	20 15	550 2 050
Legend to AS/NZS 3845 Test Vehicles: C = Small car P = Four wheel drive or utility truck S = Single unit van truck T = Tanker type semi trailer V = Van type semi trailer					Legend to NCHRP 350 Test Vehicles: C = Small car P = Four wheel drive or utility truck S = Single unit van truck T = Tanker type semi trailer V = Van type semi trailer				

Table 17.1 - COMPARISON OF AS/NZS 3845 AND NCHRP REPORT 350 TEST REQUIREMENTS

* This is a standard test vehicle in all NCHRP 350 Test Levels although not shown here in TLs 4, 5 and 6.

17.3.1 The Impact Severity Index (IS)

The “Impact Severity” index is used in both NCHRP 350 and in the European Union code EN1317 (although it is described as “Containment Level”) for comparisons between individual test within a testing regime to account for tolerances in the various variables i.e. the mass of the test vehicles, its speed and angle of impact. The Impact Severity formula measures the available kinetic energy just prior to vehicle impact.

The Impact Severity is defined by the following formula:

$$IS = \tfrac{1}{2} M (V \sin\theta)^2 \quad \text{(joules)}$$

where:

M	=	Mass of vehicle (kg)
V	=	Velocity (m/s)
θ	=	Angle of Impact

The IS is a valuable tool as it can function as an equivalence measure and be used to evaluate tested barriers against other crash test standards. This approach is valid where the nature of the test vehicles (for the relevant categories) and other acceptance criteria (i.e. occupant risk and post impact vehicular behaviour) are similar.

With all other parameters being equal, the resulting kinetic energy or IS value of the test vehicle can be used to rank or derive the equivalency of a tested barrier under an alternate crash standard. The IS cannot be used in-lieu of a crash test and therefore cannot be used to increase the rating of a tested barrier. The IS does not examine the performance of a bridge rail directly, rather the performance of a bridge rail is inferred from the highest test level that was passed. Hence, while a particular barrier may be able to satisfy the requirements of a higher test level, the IS cannot be used to demonstrate this potential nor can it be used to predict the post impact behaviour of the vehicle. This point is particularly relevant to the containment level of a tested barrier and its ability to prevent a colliding vehicle from tripping over the barrier.

17.3.2 Summary of International Test Levels

The American NCHRP Report 350 and the European Union EN 1317 are currently the two most widely recognised and accepted crash-testing standards.

The American Federal Highway Administration (FHWA) issues letters of approval for bridge rails that comply with NCHRP 350 and maintains a website of all approved bridge rails.

At this stage, a similar central repository of Comité European de Normalisation (CEN) approved bridge rails does not appear to exist. It is understood that CEN stipulates that compliant systems have to be crash tested to the requirements of EN 1317 but does not issue certificates of compliance. Without such a system, it would be difficult for a road authority, such as the MRWA, to verify CEN compliance of any proposed barrier system.

The nearest equivalent to a database of compliant EN 1317 bridge barriers is to be found in Tables provided in Appendix A of the Austroads 2002 document “Study of Overseas tested Bridge Barriers”.

AS 5100 Performance Levels	Equivalent Crash Test Category (See Note 1)	Other Acceptable Categories (See Note 2)	Test Standard
Low	TL-2	TL-3	NCHRP 350
Regular	TL-4 H1	 H2 H3	NCHRP 350 EN 1317 EN 1317 EN 1317
Medium	TL-5 H4a	 TL-6 H4b	NCHRP 350 EN 1317 NCHRP 350 EN 1317
Special Category 1 Category 2 Category 3	 TL-6 Note 3 None	 H4b Note 3 None	 NCHRP 350 Note 3 NA

Note:

1. Other Test Levels with similar Impact Severity (IS) values but have test vehicles with lower Centre of Gravity (COG) are not considered "Equivalent".
2. "Other Acceptable Categories" include crash test categories where the IS values are within the tolerance range and the test vehicles have a similar or higher COG. Any barrier rated in a higher category is acceptable by default.
3. Depending on the crash test requirements for the Category 2 44t vehicle, the EN 1317 H4b may be suitable.

Table 17.2 - OTHER CRASH TESTS EQUIVALENT TO AS 5100 CATEGORIES

17.3.3 Function

The aim of traffic barriers is to improve site safety. This aim equates to minimal damage and injury to impacting vehicles, vehicle occupants, others on the bridge and traffic, property, roadways, railroads or waterways below the bridge, consistent with the assessed level of risk at the bridge site.

Assessment of the level of risk is the critical component in determining the type of traffic barriers that are suitable for a particular bridge site. Some factors which contribute to the assessment of risk include:

- total traffic volume;
- types and proportions of vehicles in the traffic population;
- identifying the type of vehicle to be contained;
- road alignment;
- bridge width;
- general site conditions; and
- the consequences of not containing the identified vehicle within the roadway.

A good example of the selection procedures used to determine the performance level required of a railing at a particular bridge site is the AASHTO Guide Specification for Bridge Railings 1989. The CODE has adapted the AASHTO guide to prepare a version modified for Australian circumstances which is included in Appendix B of Part 1.

To manage the assessed risk and its associated factors the CODE provides a range of **Performance Levels** that are to be applied to each particular bridge site. Simply put, a bridge is assessed as requiring a traffic barrier to meet a particular performance level and a traffic barrier suitable to that performance level is chosen and specified.

17.3.4 Code Requirements

Part 1, Clause 10 of the CODE deals with the requirements of "Road Traffic Barriers" on bridges. The scope is defined as being applied to "traffic barriers for new bridges and replacement traffic barriers for existing bridges; and, the design requirements for road traffic containment barriers on bridges."

Clause 10.3 describes the features that acceptable bridge traffic barriers need to display and should be referred to for full details. The CODE implies the designer must factor into the design a wide range of inputs such as level of risk, vehicle crash behaviour, ease of repair, robustness and more.

Structural adequacy of barrier systems is an important factor in successful performance. Therefore in design, consideration must be given to possible failure mechanisms. Rigid concrete barriers must be adequately anchored to the bridge deck by reinforcement and the deck must be able to sustain the impact force without distress. Open beam and post barrier systems require continuity of the longitudinal rails to be maintained across any connection. This can be achieved by using spigot and socket joints which allow bending stiffness to be maintained across the connection. The joints must also be capable of carrying tension so that "ribbon action" is maintained during impact. This is generally achieved by bolts through oversize holes in the spigot. Longitudinal rails are no longer welded because of the difficulty and expense of testing the required full penetration weld. MRWA requires longitudinal rails to be supplied in maximum stock lengths and to be installed with 10 mm gaps between these lengths so as to take up any differential temperature between the barrier and the bridge deck. Because of this requirement the major expansion joint in a barrier coincides with the expansion abutment/s and is constructed using a solid insert.

17.3.5 Code Acceptance Requirements

The acceptance criteria for bridge traffic barriers are defined under Part 1, Clause 10.4 of the CODE. To be acceptable bridge rails must be crash tested and comply with the acceptance criteria as set out in National Cooperative Highway Research Programme (NCHRP) Report 350 or to other appropriate Standards as determined by the relevant Road Authority. However, the CODE in tacit acknowledgement that it is not practical or economically feasible to test every barrier and component, has provided other acceptance criteria besides full-scale crash testing. Barriers are also acceptable if they can be evaluated as being both geometrically and structurally equal to a compliant crash-tested system. Alternatively, approval may be given based on the evaluation of performance of an existing barrier to the Authority's requirements.

With the development of better crash simulation software, it is anticipated that modified or new barriers will be developed through crash simulation and verified with limited full-scale crash testing.

Table 10.4 of the CODE summarises the crash testing criteria for different performance level barriers. This Table also relates performance levels with the NCHRP 350 Test Levels. Part 1 also contains an informative section, Appendix B, which provides for larger vehicles designated as 'Specials'.

17.3.6 Barrier Performance Levels

In evaluating the type of barrier to be used it is critical to note that the Road Authority must carefully investigate the degree of risk, clearly identify the types and mix of vehicles that are to be contained and determine the Performance Level required at the bridge site.

Five levels of performance (None, Low, Regular, Medium and Special) are defined in the CODE. Part 1, Clause 10.5 outlines key physical parameters that influence the appropriate performance level for a specific site.

Low, Regular and Medium are the levels of service that will be applicable to nearly all bridge sites. Appendix B provides further detail and selection tables that may be used to estimate the appropriate barrier performance level required for a bridge. The selection tables are based on values of projected AADT figures and adjusted for road type, grade, curvature and deck height and under-structure conditions using adjustment factors.

a) No Traffic Barriers

Clause 10.5.2 of the CODE makes provision for the omission of barriers from low-level bridges subject to frequent flooding provided a number of conditions are met. In addition the CODE recognises that for certain bridges or culvert sites, conditions may be such that the traffic barrier may constitute a higher risk than not providing any barrier.

As previously mentioned, only a very limited number of structures without bridge railings have been sanctioned by MRWA. Even on structures that meet the criteria for omission of a barrier, designers should consider incorporating other features that aid traffic containment such as non-mountable kerbs or castellated kerbs.

The CODE does not specifically differentiate between a bridge site and a culvert site. The consequences of running off the road at a culvert site can be severe although the probability is reduced by the increased formation width and room for recovery.

b) Low Performance Level Traffic Barriers

These barriers are to be used in situations of low risk as described by the conditions (a) to (d) of Clause 10.5.3. The Low performance level traffic barrier is required to contain a level of impact typical of a 2.0 tonne utility vehicle (light vehicles) at 70 km/h and an impact angle of 25 degrees and is based on the NCHRP Report 350 Test Level 2.

The efficient flexible W-beam (ARMCO type) roadway traffic barrier was used as a standard by MRWA for many years and is a suitable Low performance level traffic barrier for use on bridges. This type of barrier has been fully crash tested under this level of impact and has successfully contained passenger cars. However the ARMCO W-beam has been largely replaced with the Thrie beam railing.

MRWA has a standard Low performance level traffic barrier specifically for use with concrete overlays on timber bridges. (Drawings 0430-0768 to 0778). This barrier consists of Thrie beam railing supported off 150 x 110 ‘C’ section steel posts.

The Thrie beam with top rail is only to be used if cyclists regularly use the bridge or the height from bridge level to ground/bed level below the bridge exceeds 6.0 metres.

MRWA still retains its 2 and 3 Traffic Rail Guardrail standard details. Although these systems have not been crash-tested in accordance with NCHRP Report 350, they are deemed as meeting the requirements for a Low performance level barrier based on theoretical capacity analysis and successful long-term performance in service. However, use of these barriers requires the prior approval of the Senior Engineer Structures. Refer to

Standard Drawings 9630-0631, 0632 and 9430-0062 for 2-Traffic Rail Guardrail details and the 3-Traffic Rail Guardrail system may be found on Standard Drawings 0030-0310 to 0312.

c) Regular Performance Level Traffic Barriers

The Regular performance level traffic barrier is required to contain a level of impact typical of an 8.0 tonne truck at 80 km/h and an impact angle of 15 degrees and is based on the NCHRP Report 350 Test Level 4.

Clause 10.5.4 of the CODE states that "Regular performance level barriers shall be provided for the effective containment of general traffic on all roads". This is interpreted to mean that the regular barrier will be applicable and appropriate to the majority of bridge sites.

MRWA has prepared two sets of standard barrier details that are deemed to conform to the Regular performance level.

The first is a Thrie beam and steel post detail based on the crash-tested ‘Delaware Thrie beam’. It is generally used as standard for rural bridges. Reference drawings 0530-1098 to 1101.

The second is a post and rail detail which has been based on the crash-tested State of New York Department of Transportation steel bridge railing. Reference drawings 0430-0760 to 0767 and 0430-1213.

d) Medium Performance Level Traffic Barriers

The Medium performance level traffic barrier is required to contain a level of impact typical of a 36.0 tonne articulated van at 80 km/h and an impact angle of 15 degrees and is based on the NCHRP Report 350 Test Level 5.

Medium performance level traffic barriers are discussed in Clause 10.5.5. of the CODE. The CODE states that "Medium performance level barriers shall be provided for site-specific, medium to high risk situations for the effective containment of medium to high mass vehicles and buses on all roads.”

The emphasis is on successful containment of the vehicle on the bridge to mitigate against injury to third parties at risk off or below the bridge. It is important to distinguish the requirement of identification of the type of vehicle to be contained as a critical factor.

The cost of crash-testing 36 tonne trucks is large and subsequently there are fewer crash-tested barrier systems available for selection. Concrete barriers have proved capable of containing substantial forces.

The “F” Type concrete barriers are based on the AASHTO profiles and are considered to be compliant with NCHRP 350. It is understood that both the 810 mm (32”) and 1060 mm (42”) tall barriers are used in WA. The 810 mm tall barrier is rated as TL-4, while the taller 1060mm high barrier has a TL-5 rating.

The New Jersey Concrete Barriers are also NCHRP 350 compliant, with the 810 mm (32”) tall barrier rated at TL-4 while the taller 1060 mm (42”) barrier is TL-5 compliant.

The Constant Sloped barriers do not appear to have been tested to TL-5, however it may be possible to infer its performance as a Medium Performance Level Barrier. A constant sloped barrier has the advantage of not being affected by the change in relative height of the barrier to the running surface when road resurfacings are carried out. It should be noted that the Constant Sloped Barrier (9.1° Californian Profile) is one of the MRWA standard road safety barriers.

As an alternative to the solid concrete barrier, Main Roads has a preference for the low level concrete barrier in combination with steel posts and top railing and is preparing a standard that is deemed to comply, based on the crash-tested Texan type HT Bridge Railing.

e) Special Performance Level Traffic Barriers

The provision of a Special performance barrier is at the discretion of the Authority (Clause 10.5.6 of the CODE). The CODE identifies three different categories of Special performance (Table B3) although only the first two categories include performance critieria, that is vehicles, test speeds and impact angles. It is considered that these Special categories will have very limited application, if any, in Western Australia.

The first Special category differs from the Medium performance level in that the specified truck is a 36t tanker as compared to a 36t articulated truck. The tanker has a higher centre of gravity and rigid vehicles are generally considered to impart higher energy at impact than an articulated vehicle of the same weight. The NCHRP Report 350 test level is TL-6. There is currently only one TL-6 rated barrier, the Texas Modified T5 Bridge Rail for Tanker Type Trailers and this barrier can be adopted without modification.

The second Special category identifies a 44 t articulated van at a test speed of 100 km/h which is almost double the energy or severity of impact of the first Special category. For the American NCHRP 350 compliant barriers, there is currently no barrier rated higher than TL-6.

Potentially, a compliant EN 1317 H4b barrier may be suitable as a Special category 2 barrier because its IS value is about 30% higher than the TL-6. The suitability of the H4b barrier will depend on the actual requirements of the Authority and these will include the height and location of the centre of gravity of the test vehicle, the collision speed and the angle of impact.

The German Maxi-rail is a proprietary barrier system, which is a compliant H4b barrier. This barrier may be suitable as a Special category 2 barrier where the centre of gravity of the T44 vehicle is less than 1.9 m and the impact speed is limited to 80 km/h at a maximum impact angle of 15^{o}.

The third Special category is to be specified by the Authority.

17.3.7 Replacement Traffic Barriers on Existing Bridges

The CODE addresses the replacement of traffic bridges on existing bridges by providing for the Authority to determine the performance level on the basis of a risk assessment, refer Part 1, Clause 10.1 Note 2.

The AASHTO Guide Specification for Bridge Railings 1989 considered this question and provided the following comments which are still applicable now:
"The guide specifications are applicable to railings for new bridges and for bridges being rehabilitated to the extent that railing replacement is obviously appropriate. They are not applicable to determining the adequacy of existing railing, when the existing railing should be strengthened or replaced or level of strengthening appropriate for upgrading a substandard existing railing. Such determinations require special study, the outcome of which will depend on site specific factors such as the condition of the existing railing, its performance record (at the site and elsewhere), traffic volume and mix, costs to effect various levels of upgrade, expected time to major rehabilitation or replacement of the bridge etc. It may be appropriate to do nothing with an effective existing railing with a performance level significantly below that which would be indicated by the guideline specifications because the cost to improve the railing would not be justified by the improvement in safety achievable."

As a consequence of the substantial increase in the containment forces being considered, many existing bridge decks do not provide the capacity to fully fix the higher performance level barrier rails. In particular, concrete overlays on timber bridges are generally inadequate to fix a Regular barrier.

17.4 MEDIAN BARRIERS

Numerous median and roadside safety barriers exist. Roadside barriers are not included here as they are comprehensively covered in the Main Roads "Guide to the Design of Road Safety Barriers" (Document No. 67-08-7) available on-line.

The purpose of these barriers is to make highways and roadways safer by reducing accident severity. The barriers accomplish this by:

- preventing vehicle penetration, thus excluding vehicles from crossing medians and reducing accident severity by excluding vehicles from entering dangerous areas;
- redirecting impacting vehicles in a direction parallel to traffic flow thereby minimising the danger to following and adjacent traffic;
- minimising the hazard to vehicle occupants; and
- minimising vehicle damage so that the vehicle can still be manoeuvred.

The "F" Type concrete barrier and New Jersey barrier have been used in WA on major structures that carry high volume and high speed traffic. These barriers are considered to be rigid barriers. These barriers have been used since the mid 1970s in high risk locations and in locations where the barrier deflections had to be minimised, e.g. as a median barrier. Examples of the use of this barrier include the Dawesville Bridge (No. 1224) and Windan Bridge (No. 1324). While these barriers have generally been used as a median barrier, the two or three rail barriers are typically used along the edge of the bridge for both aesthetic purposes and to maintain a view from the bridge.

17.5 END TREATMENTS AND TRANSITIONS

17.5.1 Barrier End Treatments

An untreated end of a roadside barrier or median barrier or obstruction is extremely hazardous if hit, e.g., a beam element of a roadside barrier which may penetrate the vehicle compartment. A number of options exist when a longitudinal safety barrier is terminated.

Only end treatments that have been successfully crash tested to NCHRP 350 or AS/NZS 3845 and have been approved for use by MRWA, shall be used. There are a number of different products that exist to terminate a longitudinal safety barrier. Refer to MRWA Document No. 67-08-7 for a list of these end treatments and restrictions on their use. Structures Engineering Branch compiled a thorough review into end treatments entitled 'Review Report and Investigation into Standard Safety Barriers Part 1: Barrier End Treatments' which is a comprehensive reference document.

For high speed (100 km/h and over) high traffic volume urban roads, it is recommended that only proprietary NCHRP 350 TL3 compliant treatments be used. The ET 2000 Plus, SKT350 and X350 are all proprietary gating end treatments which have been tested to TL3 under NCHRP 350. Refer to standard drawing 0330-1646 for layout. For other applicable situations, the WAMELT is available for use as the MRWA standard non-proprietary end treatment. The WAMELT is in the process of being modified to replace the timber posts with steel. Refer to standard drawing 0330-1646 to 1651.

Another type of end terminals are crash cushions (or energy attenuators) which are used where there is limited room such as in a narrow median. This type can be employed to absorb the energy of impact that would otherwise be applied to an obstruction such as the end of a median barrier.

17.5.2 Transitions

Transition sections are necessary to provide continuity of protection by a gradual change in stiffness when two different barriers join, e.g., when a roadside barrier or end terminal joins a bridge barrier. The gradual stiffening of the transition section will reduce the possibility of vehicular snagging, pocketing or penetration. There are a number of standard Structures Engineering Branch guardrail transition detail drawings as follows:-

- Thrie beam regular performance barrier transition to W-beam refer to drawings 0530-1098 and 1099.
- Post and rail regular performance barrier transitioning to Thrie beam to W-beam refer to drawing 0430-0765.

Standard drawings for a concrete barrier to Thrie beam are still to be prepared.

For detailed information on the principles of transition design and available crash-tested transitions refer to Section 7.8 of the AASHTO Roadside Design Guide.

17.6 PARAPETS & PRECAST PANELS

Parapets usually come in the form of precast concrete panels. Their main purpose is to provide an architectural delineation of the bridge deck. This generally is heightened by the different surface finish of the parapet panels compared to the bridge superstructure. On concrete bridges the precast parapet panels have an exposed aggregate /sandblasted finish as opposed to the off-the-form superstructure thus visually enhancing a strong horizontal band when viewing the bridge in elevation. Parapets also serve to hide the fixing details for traffic and pedestrian barriers, the edge beam/kerb and any staining of the edge that may occur.

Important points to consider in the design and detailing of parapet panels are:

- location of centre of gravity so that precast units are stable;
- location of levelling screws so that precast units can be aligned simply and accurately;
- location and detail of cut-outs to suit (barrier) posts;
- galvanised reinforcing steel is used in precast units to allow reduced cover and hence lighter units; and
- consideration to casting insitu parapets, especially on significantly curved bridges.

Generally precast units offer better control of quality. They can be sandblasted away from the bridge site under factory conditions and the use of standard steel moulds ensures good tolerances are maintained. In addition precast parapet units offer cost savings over cast insitu parapets. This is because insitu parapet formwork is usually time consuming and detailed thereby slowing down the overall superstructure works. By comparison precast parapets can be manufactured off-site and can be installed in a relatively short space of time during a non-critical phase of the bridge works. Reference drawing is standard drawing 0430-1314.

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APPENDIX A – IMPACT SEVERITY VALUES

AS/NZS 3845 & NCHRP 350						EN 1317						AS 5100					
TL	Mass (kg)	Type	Speed (km/h)	Angle (Degrees)	IS (kJ)	Level	Mass (kg)	Type	Speed (km/h)	Angle (Degrees)	IS (kJ)	PL	Mass (kg)	Type	Speed (km/h)	Angle (Degrees)	IS (kJ)
1	820	C	50	20	**9.3**												
	2000	P	50	25	**34.5**												
						N1	1500	C	80	20	**43.3**						
2	820	C	70	20	**18.1**							Low	820	C	70	20	**18.1**
	2000	P	70	25	**67.5**								2000	P	70	25	**67.5**
						N2	900	C	100	20	**40.6**						
							1500	C	110	20	**81.9**						
						H1	900	C	100	20	**40.6**						
							10000	R	70	15	**126.6**						
3	820	C	100	20	**37.0**												
	2000	P	100	25	**137.8**												
4	820	C	100	20	**37.0**							Regular	820	C	100	20	**37.0**
	8000	S	80	15	**132.3**								8000	S	80	15	**132.3**
						H2	900	C	100	20	**40.6**						
							13000	B	70	20	**287.5**						
						H3	900	C	100	20	**40.6**						
							16000	R	80	20	**462.1**						
						H4a	900	C	100	20	**40.6**						
							30000	A	65	20	**572.0**						
5	820	C	100	20	**37.0**							Medium	820	C	100	20	**37.0**
	36000	V	80	15	**595.4**								36000	V	80	15	**595.4**
6	820	C	100	20	**37.0**							Special Cat. 1	820	C	100	20	**37.0**
	36000	T	80	15	**595.4**								36000	T	80	15	**595.4**
						H4b	900	C	100	20	**40.6**	Special Cat. 2	820	C	To Be Specified by		
							38000	A	65	20	**724.6**		44000	V / T	Road Authority		
												Special Cat. 3	820	C	To Be Specified by		
													To be Specified		Road Authority		

Legend of "Vehicle Type"

A = Articulated Heavy Good Vehicle
B = Bus
C = Small Car
P = Four Wheel Drive, Utility
R = Rigid Heavy Goods Vehicle
S = Single Unit Van Truck
T = Tanker Type Semi Trailer
V = Van Type Semi Trailer

The data is arranged in the order of increasing Test Levels within a Standard and increasing "Impact Severity" across the Standards

CHAPTER 18

CORROSION PROTECTION

CONTENTS

18.1 INTRODUCTION

This Chapter presents a general and brief introduction to the subject of the corrosion protection of steelwork, and other metals, in bridges. It is brief, as the specific topic is well covered in other texts (see Section 18.2 below), and also MRWA has a fairly definitive position on corrosion protection, down to the detailed specification of the types of treatment to be used, so there is less for the design engineer to determine. Having said that, it is still important that engineers are aware of the mechanisms of corrosion, and the results of the different treatments, so they are able to make an informed judgement on each case, some of which may not be covered by the "standard" approach.

The scope of the Chapter is to address the corrosion protection of bridge superstructures, substructures and foundations, for both new works and maintenance works. While concentrating on the corrosion protection of steelwork, other metals are also mentioned.

18.2 REFERENCES

18.2.1 Introduction

There are a large number of references on corrosion protection, both general and bridge specific, but the most useful are:

- HB73.1 Handbook of Australian Paint Standards;
- AS/NZS 2312 Guide to the protection of structural steel against atmospheric corrosion by the use of protective coatings;
- BHP Coatings Guide for New Steel Bridges;
- MRWA Standard Specification 835 Protective Treatment of Steelwork;
- Australian Standard Bridge Design Code AS 5100; and
- Australian Paint Approval Scheme specifications and approved product lists.

These are discussed below. Manufacturer's catalogues may also provide a useful source of reference such as from the following companies:
Orica, British Paints, Taubmans, International, Denso, Petro Coating Systems, Galvanisers Association.

18.2.2 AS/NZS 2312

Although first introduced in 1994 and now contained in Part 1 of the Handbook of Australian Paint Standards, this Standard is still very relevant, and provides an extremely good coverage of the topic. It is a "Guide" rather than a "Standard" and so contains a lot of useful non-mandatory information on the topic. It covers:

- Atmospheric Environments - Classification of the various environments a structure may be in and which will have an influence on the optimum corrosion protection system. However for bridgeworks, note should also be taken of the different environment classifications in the CODE.

- Corrosion Protection Systems - Definition of a range of corrosion protection systems applicable to different exposure classifications and nominated design lives.

- Planning and Designing for Corrosion Protection - Hints and tips on how to design to reduce the corrosion hazard.

- Surface Preparation Treatments - Description of the different methods available for surface preparation prior to surface treatment.

- Metallic Coatings for Corrosion Protection - Guidance on the use of the different metallic coatings available for corrosion protection.
- Coating Systems for Corrosion Protection - Guidance on the use of the different painting systems available for corrosion protection.
- Painting and Paint Application Methods - Details on the factors involved in paint application.
- Wrapping Tapes for Corrosion Protection - Information on the specialist area of the use of wrapping tapes for corrosion protection.
- Maintenance of Protective Coating Systems - Repair and replacement of corrosion protection systems.
- Inspection and Testing - Details of requirements for inspectors and inspecting corrosion protection systems.
- Preparation of Coating Specifications - What to put in a corrosion protection system specification.
- Health and Safety - Very important given the hazardous nature of some coatings.

The Standard also has a number of appendices, including one giving a good description of the different types of paint and their uses.

In addition, it provides a very comprehensive list of other applicable standards covering corrosion protection related topics, including surface preparation, paints and testing.

18.2.3 BHP Guide

This booklet was produced by BHP (now OneSteel) in 1998 to specifically address the topic of the corrosion protection of steel bridges. Obviously BHP have a vested interest in the use of steel bridges and they have produced this document because of their concern that the proper and appropriate corrosion protection systems were not being specified, often penalising steel bridges, both from the point of view of cost and perceived durability. The guide is complementary to AS/NZS 2312, but is bridge specific, in that it concentrates on long-life coatings. It also presents the results of BHP research into the subject.

The guide’s coverage is similar to AS/NZS 2312, but from a bridge perspective and with greater emphasis on costs, particularly the long-term, or “whole-of-life-cost” of the different systems. Specifically it covers:

- Classification of Atmospheric Environments
- Coating Systems
- Coating Maintenance
- Sealing of Joints and Gaps
- QA Standards and Inspection
- Safety, Health and Environment
- Coating Specification Types
- Guarantees
- Contractors and Contracts
- Economics of Bridge Coatings

Again there are some useful appendices, particularly one giving a brief description of the corrosion process, and another with a description of the different coatings, what they are, and their physical and chemical properties.

18.2.4 MRWA Specification 835 – Protective Treatment of Steelwork

This specification is the standard used on MRWA bridge projects and contains details on surface preparation, coating types, application of coatings, testing and site repair. It covers most new MRWA structures, and would normally be used as-is, with only minor job specific changes. Any major departures would require the approval of the Structures Design and Standards Engineer.

Basically the Specification specifies hot-dip galvanising wherever possible, or otherwise a three coat paint system (blast, inorganic zinc silicate primer, epoxy micaceous iron oxide intermediate coat and acrylic, decorative top coat). More details on these are provided below.

18.2.5 Australian Standard 5100 Bridge Design Code

This Code and associated Commentary, addresses corrosion protection in a number of areas, e.g.

Part 3 Clause 9.4	Provides information on the durability of steel components of foundations and retaining walls and gives indicative corrosion rates in different circumstances.
Part 6 Clause 3.7	The Commentary provides some guidance, mainly on design details. Reference is also made to Appendix C of AS 4100, which gives general advice on corrosion protection.

18.2.6 Australian Paint Approval Scheme

The Australian Paint Approval Scheme (APAS) is the largest and most widely recognised paint scheme in the world.

APAS is administered by Scientific Services Laboratory (SSL) which is part of the Australian Government Analytical Laboratories (AGAL). APAS tests and certifies paints and coatings to ensure they meet stringent performance specifications. The APAS List of Approved Products contains more than 3000 approved products and is a comprehensive guide for specifiers.

MRWA refers to the APAS specifications in Specification 835, to ensure high quality products. Also, as MRWA is a member of APAS it has access to the List of Approved Products and can use this to check paints proposed by contractors. For the major paint companies the majority of their products should conform and the manufacturer will quote this on their data sheets.

Note that APAS used to be known as the Government Paint Committee (GPC) and some references may still be to GPC specifications.

For further information see the APAS web site at www.apas.gov.au.

18.3 THE NEED

This Chapter is about the corrosion protection of steel and other metals in bridge structures, but before presenting more detail on the topic, it is necessary to answer the question, why are corrosion protection treatments necessary?

The simple answer is because steel corrodes. No matter what the circumstances, steel always tends to transform into its oxides and hydroxides, or rust. The main variable being the rate of this transformation. As this process results in a reduction in section thickness and therefore strength, protective measures are usually necessary to prevent/control the reaction.

The chemistry of the process is covered in AS/NZS 2312, the BHP Guide and various texts, but basically, given the presence of oxygen and water, steel will corrode. The rate at which the process/conversion takes place is influenced by the presence of such things as chlorides (e.g. sea water), pollutants (e.g. industrial effluents), and bacteria (e.g. anaerobic sulphate reducing bacteria). Therefore, some form of protective treatment against corrosion is usually required.

In the past, MRWA has built bridges in the arid inland and north-west of the State using bare steel, but nowadays some form of surface treatment is usually used.

18.4 SUPERSTRUCTURES

With respect to corrosion protection the term “superstructure” applied to steel bridges usually means the steel beams and boxes making up the main load carrying members of the bridge, even if they are used in conjunction with a reinforced concrete deck.

MRWA’s first preference for the corrosion protection of superstructure elements is hot-dip galvanising (HDG), as this normally provides the longest in-service life, and lowest whole-of-life-cost, notwithstanding the possibly higher initial cost. HDG is easily applicable to the small components of a bridge superstructure, e.g. guardrail post sockets and expansion joint angles, but becomes more difficult when applied to the main structural members. Because of the process involved, where the element has to be immersed in a “bath” of molten zinc, the HDG treatment of the large beams and box sections used as the main load bearing members of a bridge is necessarily limited by the size of bath available locally. The largest bath currently (2007) available in WA is 12.6 m long x 1.4 m wide x 2.7 m deep. Therefore, even with double-dipping, where opposite ends of the section are alternately submerged, the maximum size that can be treated is limited to a 20 m length of 1000WB.

If HDG is not feasible, MRWA’s next preference is for a “three coat paint system”. This comprises a class 2½-3 sandblast, followed by an inorganic zinc silicate (IZS) primer (75 μm), an epoxy micaceous iron oxide intermediate coat (150 μm) and a top coat. The latter is usually purely decorative, in which case an acrylic is used (typically 60 μm), otherwise, if a high degree of protection is required, a urethane may be specified (75 μm). This system is covered by MRWA Specification 835.

Issues which must be borne in mind when specifying superstructure coatings include:

Sandblasting - A freshly sandblasted surface corrodes quickly, so it is essential that the primer is applied as soon as possible after blasting, certainly no longer than a four hour delay.

Edges, Corners and Details - On a steel section in service the first areas which usually breakdown are exposed edges, e.g. the outer edges of flanges on beams; corners, including the flange/web joint on a beam; and around details, such as bracing connections. This is because most paint is applied by spray and it is very difficult to properly coat the above areas. To avoid this it is usual to specify double lap spraying of these areas, or even applying a brush coat before spraying. This is particularly applicable to edges. Inspection should also concentrate on these areas.

Curing - Different paints require different curing conditions, and it is important to ensure these can be complied with. The most important one is the curing of the IZS prime coat. This requires the presence of moisture to cure properly. Atmospheric moisture is adequate, but adequate cure time before transport and erection is also important to prevent handling damage, see handling below.

Friction Grip Joints - A common splice detail with beam and box section decks is a friction grip bolt joint. This relies on interface friction for its capacity at the serviceability limit state. Therefore anything which might reduce this friction is to be avoided. In principle this means that the IZS primer must be left uncoated in these areas. If the units are galvanised, then some form of surface roughening is necessary. Details are given in the CODE Part 6, Clause 12.5.4.1

Handling - It often happens that there are delays in fabrication and units are wanted urgently on site and so the contractor tries to reduce the full curing times for the specified coatings. This must be avoided because of the damage that can occur during transport and erection. IZS primers must be fully cured before transport, especially if top coated. In fact, if early handling is likely to be an issue it might even be preferable to prime with a zinc rich epoxy (ZRE). On its own ZRE is not as good as IZS, but it cures more quickly and bonds to top coats better, so in some situations it may be preferred.

Welding - Apart from the friction grip joint, the other main method of joining sections on site is by welding. In this case it is important to leave the ends of the sections that are to be welded uncoated. Otherwise the paint will burn during the welding process and this can affect the quality of the weld. Also, the fumes from some paints are harmful to health.

Paint Repairs - It is just about unavoidable that some site repairs will be required, because of site welds or handling damage. Again, this is covered in Specification 835. The main considerations are the same as for the initial painting, correct surface preparation and adequate paint thickness. A different primer has to be used for repair work, as IZS is only really suitable for spraying, which means large areas. Also, IZS requires top quality surface preparation and does not bond well to existing paintwork. For these reasons, ZRE paint is usually specified for repair work.

BHP Research - It is interesting that recent research by BHP, described in the guide discussed above, has found that uncoated IZS performs better than IZS with top coats, and consideration may be given to its sole use, where a decorative finish is not required.

18.5 SUBSTRUCTURES AND FOUNDATIONS

The corrosion protection treatment of substructures, typically steel columns, is very similar to that discussed above for superstructures. However, foundations are very different, both because of the environment they are in (i.e. below ground, possibly in contact with a variety of pollutants), and their methods of installation (e.g. driven piles).

It was always considered that steel piles situated a reasonable distance below ground level (say > 3 metres) should not deteriorate (or at least deteriorate only very slowly), as there will be very little oxygen, so that the rusting reaction cannot take place. This is still generally true, however there are some exceptions. The main one is where there is the presence of sulphate reducing bacteria (SRB). These occur in anaerobic (i.e. without oxygen) conditions

in soils which have some organic matter for the microbes to feed on. They excrete sulphuric acid, which is very harmful to bare steel. They also tend to congregate in colonies, so that localised, deep pitting corrosion can result, rather than a general section reduction. Fortunately SRB are usually only present in estuarine situations, but tests should be carried out if there is the possibility of activity. If SRB are present in sufficient concentrations to be a problem, then the only reliable ways of protection is cathodic protection, or considering the tubular pile casing as sacrificial and filling with reinforced concrete.
The other main area of concern is the top section of the pile. There are a number of possible treatments. The first alternative is no treatment at all. If the soil and any ground water are clean, then it may be that the rate of corrosion will be low and can be allowed for in the design. Refer to Part 3, Clause 9.4 of the CODE for guidance on corrosion rates.

If some form of corrosion protection is required then a coating would be the next choice. The problem here is that coatings tend to be damaged during pile driving, both because of the rough handling associated with pile driving, and the abrasion involved in penetrating the soil. In some instances a damaged coating can be worse than none at all, as corrosion will tend to concentrate in scratches and chips and deep pitting corrosion may result. Of the coatings, galvanising is preferred by MRWA as it is less likely to be damaged. UC and H section piles can usually be treated quite easily, larger tubes may be difficult. As mentioned before, only the upper section of piles needs to be treated, but given the usual uncertainties over founding depth, ensure a good safety margin. HDG coatings should not however be used on piles in a marine environment as accelerated breakdown occurs in the presence of sea water.

For paint coatings on piles, flake filled polyesters are usually preferred.

Alternatives to coatings, apart from designing for a corrosion allowance as discussed above, are principally concrete filling, composite piles and cathodic protection.

Steel tubes can be filled with reinforced concrete, in which case the tube is usually assumed to be sacrificial. In fact the result is effectively a reinforced concrete pile, the steel tube becoming just a method of creating the hole in the ground. This option is usually satisfactory, but can be expensive, as the steel tube will have to be fairly thick to resist driving stresses, but is then largely ignored in capacity calculations.

Composite piles are a combination of a steel lower section, well below ground where corrosion will not be an issue, and a reinforced, or prestressed concrete upper section in the more corrosive area. For further details refer Chapter 7 Appendix A4 of this Manual.

Cathodic protection is used in a more aggressive environments, e.g. seawater splash zones. If done properly it is usually very effective, but it can be expensive, and requires on-going maintenance. There are two principal alternatives, sacrificial anodes and impressed current. In the former a piece of more reactive metal is attached to the pile, e.g. zinc, magnesium or aluminium, and this corrodes preferentially to the pile. In the latter, an electrical potential is applied to the pile to prevent corrosion. For further details consult specialist literature such as AS 2832 “Cathodic Protection of Metals”.

18.6 MAINTENANCE

This section covers two separate issues. The on-going maintenance of protective coatings applied to new structures, and the corrosion protection of elements used in maintenance work, particularly on timber bridges.

To deal with the latter first; steel elements used in timber bridge refurbishment works are principally 410UB sections for stringer replacements and 200UC, 250UC or occasionally

310UC sections used for piling. These sections are relatively small and the universal treatment is hot-dip galvanising. Any repair works necessary due to damage or site welds would be done using zinc rich epoxies.

Whilst the above is the accepted practice in essentially fresh water waterways, in salt affected areas of the Wheatbelt it has been observed that HDG coatings fail prematurely, in some cases within three years. In these situations a high build epoxy coating is recommended.

The maintenance of existing coatings on bridges is a more complex topic, and consideration has to be given to a number of factors:

Timing and Extent - First and foremost is when to carry out maintenance, and directly associated with this, how much work to do. At the two extremes, one can either carry out frequent and regular maintenance, washing off pollutants and touching up any corrosion as soon as it starts; or do absolutely nothing until the coating has deteriorated completely, and then replace it. The choice between "repair or recoat" and the various intermediate combinations is influenced principally by availability of finance and how easy it is to access the structure, although a further factor is the integrity of the existing coating. If it is adhering well to the substrate then repair is always possible, but if not, then replacement may be the only option. Which approach is most economical on a whole-of-life basis will vary from structure to structure, although usually some form of on-going maintenance will have economic advantages, as it will extend the period up to full replacement, as long as access is not expensive.

Access - This is the key to the proper maintenance of coatings to steel structures. If access is made relatively easy, then regular maintenance is more likely to be done, as it can be carried out economically. Especially with larger structures, this may require special provisions to be "designed in", e.g. access walkways, maintenance cradles, etc.

Recoating - Another factor which must be borne in mind is the recoatability of the original coating. Some top coats, especially some epoxies and urethanes, are very hard and it is difficult to get a repair coat to adhere without extensive surface preparation.

Red Lead Paint - One issue with older structures is that they will probably have been painted with red lead paint. The removal of this is a health hazard both to the operator and the general environment, and special measures have to be adopted. Fortunately most bridges with red lead have now been repainted, but if this is an issue then Road Traffic Authority NSW have guidelines to follow for the removal of lead based paints.

Coal Tar Epoxy - Coal tar epoxy, which used to be used as a protective coating mainly on piling, is another coating whose removal should be planned with care, as it presents a potential health hazard.

Hot Metal Spray - An option that has been used by MRWA for maintenance works on the older bridges over the Swan River is hot metal spray. This involves complete removal of the old coating and application of a metallic coating similar to hot dip galvanising insitu using specialist equipment. For further details see Section 6 of AS/NZS 2312. This is a very durable coating, but expensive. Where it has been used it has been justified by the high cost of access.

18.7 MISCELLANEOUS

The above sections have covered the topic of the corrosion protection of steel, but there are a number of other corrosion issues which have to be considered by bridge engineers. These include:

Stainless Steel - Although not often used because of its high cost, stainless steel does have application in highly aggressive environments, especially for important elements which cannot easily be repaired or replaced, e.g. fixings and hangers. Another use in could be for pile caps or piers in 'splash zones'. Marine grade stainless steel should always be specified.

Aluminium - Aluminium is another material that can be used on bridges in aggressive environments, as it is much less susceptible to corrosion than ordinary mild steel. However, aluminium has a significantly lower strength than steel, and this must be allowed for in design. Generally, the use of aluminium is restricted to handrails. Even then, it should be anodised to guarantee long life.

Dissimilar Metals - Where dissimilar metals are in contact there is always the increased risk of corrosion through galvanic reaction. This is particularly so where the metals are far apart on the galvanic table. Positive use is made of this in passive cathodic protection, but normally ensure dissimilar metals are not in direct contact, e.g. use insulating washers under bolts. For further details refer Section 4.3.3.1 of AS/NZS 2312.

18.8 CONCLUSION

This Chapter has given a brief introduction to the topic of corrosion protection, particularly as it is applied to bridges, and specifically MRWA bridges. Although it has presented a large amount of relevant information, it has not set out to be a comprehensive coverage of the subject, but rather to acquaint engineers with the issues, and direct them to where further information can be obtained.

It may be thought after all this, that why use steel with all its corrosion problems, it must be better to use concrete which is thought to be inherently more durable. However, concrete also is subject to deterioration, e.g. alkali silicate reaction, carbonation, chloride penetration etc, and so may not have a longer maintenance free life than steel. The "advantage" of steel is that if it does start to deteriorate at least the deterioration is immediately obvious, so maintenance can be organised and planned. With concrete, corrosion of reinforcing steel can proceed to quite a degree before any external symptoms are obvious. Therefore in some respects steel is a "safer" material to use, with the correct protection.

CHAPTER 19

HEAVY LOADS

CONTENTS

19.1 GENERAL

Heavy Vehicle Operations (HVO), within the Road Network Services Directorate, receives permit applications from the Transport Industry for the movement of oversize or over weight vehicles. Over weight vehicles which exceed the stipulated levels advised by Structures Engineering (SE) (refer 05/4210), are forwarded to the SE Heavy Loads Group for assessment.

The permit applications received by the Heavy Loads Group are of two kinds: Period Permits, or Single Trip Permits. Period Permits cover vehicle movements for a period of time, generally on specific routes, but are usually for an area, such as a Local Authority, or a specific radius from a town. Single Trip Permits cover a single movement of a vehicle on a specific route only, and this type of permit makes up the bulk of permits assessed by SE.

The work carried out in the Heavy Loads Group consists of the following:

a) assessment of Period Permits

b) assessment of Single Trip Permits

c) bridge load rating

d) bridge load limit posting

e) assessment of proposed heavy load movements

Each is different and requires a different approach and methodology, and is described in detail below.

Working stress methods of analysis are still used in the Heavy Load Route Analysis System (HLR) program for a first pass assessment and for assessments of timber bridges. Reference should be made to the 1976 NAASRA Bridge Design Code for details, including load combinations, allowable stress levels etc.

19.2 PERMIT SYSTEM

19.2.1 Types of Vehicles

The Vehicle Standard Regulations 1977 sets out the maximum permitted legal loads of various types of vehicles. Vehicles are licensed by the Police in one of three classes as follows:

- Class A Licence

Goods carrying vehicles which are within the mass and dimensions laid down in the above Regulations.

- Class B Licence

Goods carrying vehicles which do not conform to the mass or dimensions laid down in the Regulations.

- Class C Licence

Vehicles designed to carry fixed plant (bulldozers, scrapers etc.), and which cannot be readily modified to become a goods carrying vehicle, and mobile cranes. These are known as "Non Load Carrying" vehicles.

19.2.2 Overload / Oversize Permits

Where a vehicle with a Class A licence is required to carry a load which exceeds the weight or dimension limitations stipulated in the Regulations, the vehicle operator must obtain either an Extra Mass Permit or Oversize Permit or both, before the load is permitted to move. In the case of vehicles with a Class B or Class C licence, a permit issued by HVO is required for all movements on public roads. Only Extra Mass Permit applications outside HVO's delegated authority are forwarded to SE for assessment.

19.2.3 Period Permits

Period Permits are issued for the movement of a number of licenced vehicles, including truck and trailer combinations, road trains, B-double vehicles, mobile cranes, drilling rigs and front end loaders.

Currently, the maximum axle group loads under different permit conditions for truck and trailer combinations are shown below for the different axle configurations.

No. Wheels per Axle	Vehicle Axle Configuration	VSR Loads (t)	Concessional Loads (t)	Class 1 Over Mass Loads (t)
2	Steer axle, single wheels	6	6	6
4	Single axle, dual wheels	9	9	9
2 2	Twin steer axle, single wheels	11	11	11
4 4	Tandem axle, dual wheels	16.5	17	18
4 4 4	Triaxle, dual wheels	20.0	23.5	27
4 4 4 4	Quad axle, dual wheels	20.0	23.5	36

19.2.4 Single Trip Permits

Single Trip Permits are issued for the movement of indivisible loads on defined routes only, up to a maximum of 2.25 tonnes per tyre (except for steer group axles, which are permitted slightly higher tyre loads to retain steering control). The tyre load limit is based on pavement capacity, and was included in the 1985 NAASRA Review Of Road Vehicle Limits (RORVL) Study recommendations.

HVO has been delegated authority to issue permits up to 18 t tandem, 27 t triaxle and 36 t quad and 484-quad axle groups using data in the Integrated Road Information System (IRIS). The Heavy Loads Group of SE is responsible for providing these axle ratings for every bridge on public roads within the State.

HVO also has been delegated authority to issue permits for standard tri and 484-quad axle floats on the Heavy Haulage routes and common transport routes as per 05/4210. The Heavy Loads Group of SE is responsible for keeping this bridge load capacity data current.

19.3 PERMIT ASSESSMENT

19.3.1 Assessment of Period Permits

All bridges on nominated routes, or within the requested area of travel, must first be identified and listed. Once this is done, each bridge must be checked individually. This is usually done by comparing the effects of the requested vehicle to the effects of the T44 design vehicle using HLR.

When assessing Period Permits, if the existing bridge ratings were calculated using the working stress method, this method must also be used to calculate the vehicle effects, to ensure the comparison is valid.

As Period Permits allow unrestricted travel on approved bridges, in terms of vehicle speed and location on the bridge, the requested vehicle is treated the same as the T44 design vehicle, in that it is analysed using the same loading combinations. These group loading combinations can have a combined stress level of 125% of the basic allowable stresses. Where these values are very close or the rating or design vehicle envelope is exceeded, a detailed bridge analysis may be required.

Any bridges which cannot be used by the requested vehicle are excluded on the permit, or that road excluded from the requested route. Where a particular vehicle or route results in many bridge restrictions, the permit is generally rejected, and travel on the requested routes or areas will be on a Single Trip Permit basis only. Also, travel outside the route specified on the permit is not allowed, except on a Single Trip Permit basis.

19.3.2 Assessment of Single Trip Permits

Assessment of Single Trip Permits is very similar to that used for Period Permits described above, except that the vehicle in this case can be treated as a heavy load platform or 'abnormal' design vehicle, which can be permitted up to 140% of the basic allowable stresses. This is because of the relative infrequency of moves with all movements of Single Trip Permit vehicles being central within the lanes in the direction of travel (or any other location on the bridge as stipulated on the permit) to minimise the transverse distribution effects, and at a speed of no more than 25 km/h. For very heavy loads, the speed of the vehicle crossing specified bridges can be reduced to 'walking pace' or 5 km/h, under engineering supervision, to reduce the dynamic load allowance effects.

There are several aids used in assessing these Single Trip Permits, due to the volume of permit applications requiring assessment. These are described below:

a) Guideline Tables

Tables of values of permitted axle group loads for standard tri and 484-quad floats on the heavy haulage routes and numerous common transport routes are available (refer 05/4210). These values have been calculated for various vehicle configurations for each bridge, and the critical bridge values for each route entered into the Table. These Tables provide a quick, easy reference against which most tri and 484-quad vehicle combinations can be checked for most or part of the requested route. This reduces the number of bridges to be checked. Where the requested axle group loads exceed the permitted values in the Tables or when the vehicle is outside these tri and 484-quad guidelines, then the next stage is to find comparable permits previously assessed.

b) Heavy Loads Records

There are many Records of previously permitted loads for many vehicles covering most routes. Comparable loads, vehicles and routes can be quickly checked against these Records, and the assessment for suitability is then almost complete. Where the requested loads are very close to the comparable, previously permitted loads, then bridge condition may need to be considered if the previous case was done years before.

c) HLR Computer Program

The HLR computer program was developed to quickly analyse heavy load movements on a road network containing bridges. The program stores route information in the form of nodes and links and bridge information, including longitudinal bending capacity envelopes and reactions, if applicable. The analysis module of the program calculates shear and moment envelopes and reactions for moving axle loads on bridges. The user inputs the requested vehicle details such as axle loads, spacings and widths (or selects from a list of stored vehicles which can be modified), inputs the requested route (also available as a list of stored standard routes), and the HLR program will then run the requested vehicle across all bridges on the nominated route, and produce a summarised report.

This report will either permit or prohibit the heavy load move, and list out the bridges requiring supervision conditions and those that are overloaded. It is then up to the user to determine what, if any, additional conditions are to be applied to the permit if the move is to be allowed. Where the move is prohibited by HLR, the user should check the detailed report containing the calculated results at sections where supervision is required, or overloads occur, for the critical bridges. This is required, as the overload may be negligible, or some allowance may be justified, due to the generally conservative distribution method applied by the program (based on Section 3 of the 1976 NAASRA Bridge Design Code), effects such as theoretical 'knife edge' supports compared to finite width supports, and moment redistribution. The user must apply experienced engineering judgement when doing this, and each bridge, its condition, type of construction and location of overload all must be considered. Previously permitted loads may give some indication of any allowance which might be justified.

Experienced engineering judgement is also required by the user to be aware as to which cases HLR may provide non-conservative results and additional calculations by hand are required to obtain the correct assessment. Two examples are single lane bridges and beam bridges with widely spaced beams.

HLR is a quick and useful tool for analysing heavy loads travelling long distances (and crossing many bridges). It is not used to analyse every permit, as use of the guideline tables and previous records can be quicker and simpler.

It should be noted that HLR is based on the 1976 NAASRA Code, and is therefore 'working stress method' based.

19.4 LOAD RATING OF BRIDGES

19.4.1 General

Bridge rating is the process of determining the maximum vehicle load which a bridge can safely carry for a given vehicle configuration. Historically, for heavy load purposes, the practice was to rate bridges for a standard M1 vehicle, and was called the M-Rating. Other historical bridge design and load rating vehicles are shown in Section 16 of the Bridge Branch Design Information Manual.

Today, bridges are rated for the most common heavy load vehicle configurations, such as quad low-loaders and 8 and 12 axle platforms etc. The various load rating vehicles for different bridge types are shown in Section 4 of the Bridge Branch Design Information Manual and the TIMBAR Procedure Manual.

19.4.2 Reasons for Bridge Rating

Bridges need to be rated for one of the following reasons:

i) to check the strength of an existing structure, which has been subject to deterioration, and to assess whether it can safely carry the normal Regulation loading, or if it is necessary to restrict the loading by load limit posting;

ii) to determine whether a bridge can sustain Period Permit loading; and/or

iii) to determine whether a bridge can safely carry a particular heavy load which is the subject of a Single Trip Permit application.

19.4.3 Requirements for Bridge Rating

To carry out a bridge rating, the following information is required:

i) detailed bridge inspection report on the condition of the structure;

ii) sizes of all bridge members;

iii) strengths of all materials in structural members;

iv) details of structural connections;

v) details of foundations; and

vi) details of vehicle(s) for which bridge is being rated.

The condition of the structure is very important, as any rating carried out without proper knowledge of the bridge condition is then purely theoretical, and the strength of the bridge may be overestimated if account has not taken of defects such as heavily corroded steel members, buckled steel plates, decayed and rotten timber, and cracked or spalled concrete.

19.4.2 T44 Ratings of Bridges

Most of the existing heavy loads records have bridges with a T44 rating. This information can be found in the IRIS bridge inventory system. Details for load rating of existing bridges are found in Section 4 of the Bridge Branch Design Information Manual and the Load Rating and Refurbishment Design Manual for Existing Timber Bridges.

19.4.3 New Load Ratings

New load ratings are required for new or refurbished/strengthened bridges, and also for existing bridges previously not rated. The design engineer designing the new bridge is responsible for rating the structure, as described in Section 2 of the Bridge Branch Design Information Manual.

Rating of existing bridges is the responsibility of the Engineer Bridge Loading, and Limit State methods are used for concrete and steel bridges, and working stress methods for timber bridges. The methods of analysis for the various bridge types are described in

Chapters of this Manual and in the Load Rating and Refurbishment Design Manual for Existing Timber Bridges.

19.5 LOAD LIMIT POSTING

Bridges which cannot carry the legal limit or regulation vehicle (VSR) loads (refer Section 19.2.3), are required to be load limit posted.

To determine the limit required for posting, the following process needs to be followed:

- If M Truck < 10t Load Limit = M Truck
- If M Truck $\geq$ 10t, and Tandem < 16.5t Load Limit = Tandem / 0.8
- If Tandem $\geq$ 16.5t and Tri < 20t Load Limit = Tri / 0.8
- If Tri $\geq$ 20t, and Quad < 20t Load Limit = Quad / 0.8

The above 0.8 factor accounts for a prime mover in the total vehicle configuration as the load limit sign is expressed in tonnes gross vehicle mass and not the rated axle mass.

For the process of obtaining approval for load posting a bridge, reference is made to the Structures Engineering Management System Manual.

CHAPTER 20

DESIGNING FOR CONSTRUCTION AND MINIMUM MAINTENANCE

CONTENTS

20.1 INTRODUCTION

The objective of this Chapter is to provide information to ensure that designs are "constructible" and that the design, detailing and specification of materials etc are such as to ensure the structure is durable, i.e. will have a long life with minimum maintenance. These two aspects, constructability and minimum maintenance often go hand-in-hand. If a structure is hard to build then the chances of substandard workmanship and consequent future problems is increased.

These subjects are infrequently covered in text books or other publications, and if they are then comments are not specific to the structures designed and built by MRWA.

There are two AustRoads publications, "Guide to Construction Practice" and "Bridge Management Practice" which, while not directly addressing the topics of this Chapter, do contain useful information on construction techniques and common maintenance problems and so can be helpful. Federation Internationale de la Precontrainte (FIP) also has a publication, "Practical Design of Reinforced and Prestressed Concrete Structures (based on the CEB-FIP model code (MC78))", which contains useful information. Also, reference should be made to SAA HB64, “Guide to Concrete Construction” by Standard Australia. For maintenance works, a conference held in Melbourne has some useful information, "Rehabilitation of Concrete Structures - Proceedings of the International RILEM/CSRIO/ACRA Conference, Sept 1992".

In addition there are a number of general books on bridge aesthetics, which while being primarily architectural, do show good and bad details, and the bad ones are not just bad aesthetically, they usually also create a maintenance problem, e.g. cracked abutment wingwalls where insufficient allowance has been made for movement, extensive staining where inadequate provision has been made for drainage etc. Useful publications of this type are listed in Section 2.2.7 of this Manual.

The approach taken in this Chapter is to first look at the materials commonly used in bridge construction (Sections 20.2 to 20.5), and then at each of the structural elements making up a bridge (Sections 20.6 to 20.11). Aspects covered include:-

Materials
- Special properties and important things to look out for.
- Detailing and specification of the material in different situations.
- Maintenance problems and methods.

Elements
- Typical methods of construction and how this affects design and detailing.
- The main influences on choice of construction method.
- Specific maintenance problems and methods.

Obviously there will be a deal of overlap here, as many of the aspects covered in materials will apply to the elements made from them, eg concrete properties and reinforcement cover.

It must be stressed that this Chapter is far from being an exhaustive coverage of the subjects. It is intended to serve as a guide and draw attention to things which might otherwise get overlooked. There is no substitute for experience in these matters, plus talking to people involved in construction and maintenance at all levels, and visiting construction sites.

PART A – MATERIALS

20.2 CONCRETE

Concrete is the most used (and probably the most abused) of all modern construction materials. In the past it was considered that concrete was a low maintenance material with an indefinite life and no problems, as long as a few simple rules were followed. For these reasons it is used instead of steel in a lot of applications, as it was always considered that with steel continual maintenance was required.

However, a number of major problems have emerged with concrete structures. These range from the obvious ones caused by lack of cover, bad compaction, exposure to chlorides, (eg sea water and saline streams) etc, to those less obvious, but equally deleterious, such as carbonation, chloride penetration and Alkali-Aggregate Reaction (AAR), or Alkali-Silicate Reaction (ASR). Engineers are now thinking that perhaps steel is not such a bad material after all, because at least any problems are usually visible and you know you have to carry out regular maintenance, whereas concrete may just get forgotten until the problems are really serious.

It is therefore important that more thought is given to the specification and placing of concrete. A number of these issues are addressed in the concrete durability section of the CODE, (Part 5, Clause 4) and associated Commentary, which also contains a number of useful references. The CODE concentrates on requirements for curing and cover and the Commentary provides guidance on minimum cement contents in different situations.

The basic aim of all the requirements for specification, placing and curing is to produce a dense, durable concrete.

20.2.1 Specification

It was thought that, except in very special situations, if there was sufficient cement in the mix then the concrete would be alright. However, it is now realised that the type of cement and also the additives in it are also important, especially where the concrete is in an aggressive environment, eg exposed to salt water.

For bridges, MRWA only uses N, or normal grades of concrete from AS 1379, as blinding or mass concrete, all structural concretes are S, or special mixes, so that special requirements that will enhance durability, such as low water/cement (w/c) ratio and relatively high cement content can be specified, but with a restriction on total alkali content (see later). In addition, MRWA principally uses S40 as the standard structural concrete. The only exception to this rule is that N class concrete may be specified for insitu works on culverts at remote locations. In this case a minimum of N40 would be used in lieu of S35.

MRWA TDP Specification 820 contains mix details for the concretes commonly used in MRWA Bridges, ie. 35, 40 and 50MPa. Basically, the aim is not to have over-sanded mixes, as these are thought to have higher shrinkage, and to specify a minimum cement content and maximum w/c ratio. The mix design details specified in Specification 820 should be followed closely to avoid these problems. This is particularly important when using pump mixes, as these tend to have a higher sand and water content to assist pumpability.

The amount of cement may well be more than what is required for strength, but is considered necessary for durability, as a dense, high cement content mix reduces the rate of carbonation and chloride penetration and ensures that the reinforcing steel remains in a highly alkali environment for as long as possible.

Maintenance of this alkalinity is essential as it provides chemical protection (steel does not rust in an alkaline environment), in addition to the purely physical barrier provided by the cover concrete, therefore delaying the onset of corrosion.

Carbonation is the process by which carbon dioxide in the atmosphere, in conjunction with water vapour, also in the atmosphere, combine to produce a weak acid, which then slowly attacks and neutralises the concrete. This removes the protecting alkalinity so that any chlorides present can begin attacking and corroding the steel. Carbonation proceeds more quickly in "poor" concrete as this will be more porous and therefore more permeable to carbon dioxide and moisture.

Similarly, chloride penetration through the cover concrete occurs more readily in poor quality, porous concrete. This is obviously particularly important in areas close to the sea or large areas of salty water where chlorides can be carried by the wind for large distances.

The MRWA Specification also puts a maximum value on the mass of reactive alkali in the concrete. This is to reduce the possibility of alkali-aggregate reaction (AAR) in the future. AAR or ASR requires the coexistence of three things; a reactive aggregate, free water in the concrete, and free alkali. Alkali is normally present in the cement, but with the increasing use of additives it is important to check all components of the concrete, as these also may contribute a significant quantity of alkali.

In addition to these controls, there are a number of special cements and mixes which can be used, for instance in large elements where heat of hydration can cause problems, and in aggressive environments, such as exposure to salt water.

Heat of hydration problems can occur when pouring large "blocks" of concrete, eg pile caps that may be over 2 metres thick. It takes a considerable time for all the cement in a mix to react and for all that time the reaction is generating heat. However, the outside of the block cools off fairly rapidly, as it is in contact with the air, but the inside, which is insulated by the outer layers, does not. This can set up high thermal gradients, (the inside temperature may reach over 70^{o}C) which can result in cracking, leading to possible future problems. Two ways to avoid this are; all round insulation of the concrete, as this reduces the temperature gradient by preventing the outside cooling off too quickly; and/or the use of a low heat cement. This latter will be a type GB cement, a mixture of cement and granulated blast furnace slag. This will help with heat of hydration problems, as blended cements have a slower reaction rate and therefore generate heat at a slower rate. However, other problems can result as slag cements have a low rate of early strength gain and require greater attention to proper curing than does Ordinary Portland Cement (OPC).

Blended cements are also made with fly ash in place of slag. The properties are similar to slag blends, these are available in WA.

In aggressive environments, particularly where salt is present, it is imperative to have a dense concrete to resist penetration by chlorides. As well as using a high cement content concrete, with proper compaction and curing, another way of ensuring this is by the addition of silica fume to the concrete. This entails replacing a small proportion of the cement in the mix (5%-8%), with silica fume. Silica fume is a by-product of silicon smelters, being the condensate from the effluent stacks. It acts to densify the concrete, in particular by closing up the pores caused by the presence of minute amounts of free water in the mix. Silica fume is expensive and special precautions are needed in mix design and the mixing process, so it should only be used where really essential, for structural elements in very aggressive environments, eg pile caps and piers within the splash zone. It is often used in conjunction with blended cement, as these cements also convey benefits in situations where the use of silica fume is justified. Such concretes are called "triple blend" mixes. In all cases a 7% replacement of cementitious material with silica fume is typical.

There is a range of other products that claim to improve the long term durability of concrete, mainly waterproofing agents of one form or another. These include both concrete additives and coatings. MRWA allows for the use of waterproofing additives in precast concrete box culverts in highly aggressive environment for additional durability or as an alternative to greatly increased cover. External waterproofing membranes are a possibility however. The obvious ones are various forms of bituminous membranes, either a solid sheet product, such as 'Bituthene', or a liquid, such as 'Elastoseal'. These are useful for buried elements where there is concern about the nature of the ground or groundwater, however for aesthetic reasons they should not be used where the concrete will be visible. It is also important to ensure that the entire element is covered, including the underside.

Other possible coatings are Silanes, a range of clear coatings that have been shown to provide an effective degree of waterproofing. Even here though, care is needed, for instance, it is known that concrete "breathes" to some extent, exchanging water vapour with the atmosphere. Silane and other coatings seem to provide a one way barrier, they prevent water entering the concrete, but permit water vapour to escape. If an otherwise sealed element has its base not sealed and is immersed in salt water then this “breathing” action, combined with vapour movements within the concrete itself, may result in salt water being "sucked" up into the body of the concrete. Obviously this will be a very slow process, but may lead to long-term deterioration and should be considered where a column is on a pile cap and the pile cap cannot be completely sealed as it is in water.

20.2.2 Placing

The best specified mix in the world would be of little use unless it is properly placed. This is in fact one of the reasons why much concrete placed 30 or 40 years ago is still performing well. The mix design might not have been as good, but a reasonably dry mix was used and it was thoroughly rodded into position. Nowadays, with everybody looking for easy, quick, less labour intensive solutions, wet, high slump, pumped mixes are widely used, with mechanical vibration and it is essential that more attention is given to placement, both from the point of view of the specification and on-site supervision.

The important thing that the designer can do to increase the chances of getting well compacted concrete is to ensure that placement and compaction are as easy as possible. One way to help is to provide adequate spacing between reinforcing bars and around prestress ducts. Generally a minimum clear distance of 100mm is preferred, with 75mm the absolute minimum. This is to permit the use of a large, 65mm diameter, vibrating poker. In some instances the spacing will need to be increased, eg tall columns or walls, where access is required for a tremie pipe or pump hose. Also in large, tall columns it is often necessary to send someone down inside to satisfactorily vibrate the lower layers of concrete and reasonable space will be required between the stirrups for this.

Two critical areas, which usually have congested reinforcement, but where sound concrete is even more essential, are the bases of columns and walls, and prestress anchorage zones. Reinforcement detailing is especially important in these areas, eg use different length starter bars so that laps are staggered and are not all at the base of the column; and butt weld anchorage zone stirrups to minimise overlapping bars. Remember, it is just as important to have sound concrete as it is to have the correct amount of reinforcement.

Apart from detailing for easy placement there is little else the designer can do, it is then up to the site staff to ensure that all concrete is properly compacted and finished.

"Superflow" concrete is used in situations where it is not possible to guarantee adequate vibration. This is concrete with a special additive which gives a short-term "flowable" mix needing no, or very little vibration. While this can be useful in certain extreme situations, it is expensive and should be used with caution. Also, a completely self-compacting, self-levelling concrete has been introduced into Australia, having been in use overseas.

Evidence from that use is that the resulting concrete has a higher void ratio than normally compacted concrete, and so should not be used in a corrosive environment. MRWA has used high range super-plasticiser in concrete for bored piles to increase flow ability during pour and placing without any compaction. MRWA standard S40 mix is not self- compacting concrete.

20.2.3 Curing

Once again, for sound, durable concrete, in addition to the correct specification and proper compaction, it is essential that the concrete be adequately cured. The purpose of curing is basically to make certain that the cement in the concrete is fully hydrated, thereby ensuring the full design strength is reached and also resulting in a denser, less porous concrete. Therefore the main aim of curing is to prevent the escape of water from the hardening concrete.

This can be done in a number of ways:-

- Keeping the concrete permanently wet is an obvious one. This can be with sprinklers, either alone or in conjunction with hessian, sand or foam rubber sheeting. This is one of the best methods, as, provided the concrete is kept wet, it guarantees proper curing. However, it cannot always be done, as the bridge may not be close to a water supply. Also proper provision will need to be made for water draining off the structure.

- Leaving the forms in place is another good method, however not always possible as the contractor may want to reuse them elsewhere or perhaps adjoining concrete has to be poured. In some instances, eg where thermal curing of large sections is required, properly insulated forms may be specified.

- Wrapping in an impermeable membrane, eg black plastic, is possible, but it is essential that it be kept tight against the concrete at all times otherwise the wind will get underneath and dry out the concrete.

- Chemical spray-on membranes are also widely used. These form a reasonably impermeable membrane over the concrete. They have the advantage that once applied no further action is required. However they are not 100% impermeable; they can be easily damaged, especially on deck slabs; it is difficult to see if they have been applied properly; they can result in staining to the concrete surface; and they can inhibit the adhesion of future surfacing. For all their faults they are still the most common form of curing, because they are so convenient.

The time for which concrete must be cured varies with the type of cement. For Ordinary Portland Cement concretes a minimum of 7 days is normally specified. For blended cement concretes at least 14 days is required, as the rate of hydration is much slower, so unless prevented from drying out the concrete may not achieve its nominated strength. With Triple Blend mixes, water curing should be specified wherever possible to ensure full hydration of the concrete.

20.3 STEEL

The other principal material used in bridge construction is steel. It is used in various forms:-

- as main structural members, eg beams and boxes;
- as reinforcement for concrete, eg reinforcing steel and prestressing steel; and
- in a large number of miscellaneous items, eg railings, joints, bearings etc.

The main problem with steel, in any of its forms, is that it corrodes, with consequent loss of section and so strength and stiffness. Also, in reinforced concrete, corroding reinforcement causes expansive forces to be applied to the concrete resulting in cracking and spalling, and thus further exposure and increased corrosion. Therefore measures have to be taken to prevent this, either by protective encasement or protective coatings and this protection has to be maintained during the life of the structure.

20.3.1 Structural Steel

Steelwork used as principal load carrying members in bridges is usually in the form of beams or boxes. These can be in columns or piles, or as longitudinal and/or transverse structural elements in decks.

Beams can be either standard off-the-shelf items or purpose made. Standard items usually come from OneSteel and the current range of larger sections includes:-

- Beams rolled sections to 610UB, welded to 1200WB.
- Columns rolled sections to 310UC, welded to 500WC.
- Piles very limited and subject to special ordering (column sections are usually used instead, or sometimes steel tubes).

In addition to these, imported steel sections are sometimes available at competitive prices, otherwise special requirements can be fabricated. Specialised "beam lines" available in local fabrication shops mean that welded beams can be fabricated quite economically, but it is still best and cheapest to wherever possible utilise the standard OneSteel range.

Rolled sections are available in grade 300 Plus as standard. Also the different low temperature impact grades of L0 and L15 can be obtained, but enquiries should be made well in advance. The standard grade supplied by steel producer is L15 and L0 can be obtained as a special order. For important superstructure elements MRWA usually uses Grade L15.

Welded sections are supplied in Grade 300 as standard with Grade 400 also available, but at a significantly greater cost and so are not normally used.

Note: Bear in mind that for standard elements from OneSteel, a lot of the larger sizes are only produced infrequently and are not held in stock, so pre-ordering will be necessary.

Both rolled and welded sections are normally only available in lengths up to 18 metres. Longer lengths are available subject to special ordering, but use of these should be considered carefully, as factors such as transport and handling difficulties will have to be considered, especially if beams are sourced from the Eastern States. For welded sections, fabrication locally can avoid some of these problems.

Consideration must be given at the design stage to the likely construction method and sequence and therefore the size of fabricated members. Obviously for erection on site the least number of splices the better. However, the designer must bear in mind the transport and handling problems associated with long and/or large sections. Although it will depend on the shape of the element and the transport route, 30 metres is normally the maximum length that can be handled. Always check with the MRWA Heavy Vehicle Operations Section before making a final commitment.

Designs involving straightforward use of beams are likely to involve little fabrication and therefore have few construction problems, except perhaps joints between beams, which are discussed later.

Fabricated sections, usually in the form of box sections, are used for both bridge decks and columns. The main thing to consider whilst carrying out the design is how the section will be fabricated. This will normally mean, how it is to be welded. It is important to ensure that satisfactory access is available for welding, and for subsequent inspection. This is especially important around the supports in boxes, where there is likely to be extensive stiffening. Also try to avoid the specification of difficult or expensive welds, eg external corner welds that may have to be ground to achieve the correct shape. In addition, always bear in mind that folding plates is a much cheaper option than welding and can be used for columns or simple boxes, provided the shape and plate thickness are appropriate, ie. a section which is constant or changes linearly and a maximum plate thickness of 16mm.

Another problem with welding is that it can create distortions in the section. This occurs due to shrinkage of the steel following high heat input from the welding process and subsequent cooling. To a large extent distortion is the fabricator's problem but some things can be done to help, such as not having large welds with small plates. Also any large areas of plate which have stiffeners welded on, eg sides of a box girder, should have a minimum plate thickness of 16mm, otherwise distortion at each stiffener position will produce a wave affect in the plate, which is aesthetically undesirable.

With thick plates (>40mm), precautions have to be taken against lamellar tearing. This is caused by planes of weakness being rolled into the thickness of the plate during forming, which allows the possibility of the plate "peeling apart" during service. If the use of thick plates is unavoidable, the specification must call for full testing before fabrication to check for the presence of lamellar intrusions.

For further information on welding see:-

- The Weldability of Steels — WTIA
- The Procedure Handbook of Arc Welding — Lincoln Electric
- Bridge Welding Practice — NAASRA

Joints between members are another important area with steel bridges, especially bridge superstructures. A structure of any length will have to be handled in sections, which means that site jointing will be required. The alternatives are obviously welding and bolting. Welding is normally preferred structurally, but may be a problem where the joint is over traffic. It will be difficult to provide protection both from the affects of the welding, eg splatter, and subsequent X-ray inspection. Also welding will require temporary support or connection while it is being carried out and possibly extensive on-site repair works to the corrosion protection. Quality control can also be a problem, especially on remote sites.

Bolted joints are the other possibility, either bolted splice plates on the webs and flanges, or joints with heavy transverse diaphragms at the ends of abutting sections, joined with high strength friction grip bolts. In most cases a large number of bolts may be necessary, which can be expensive and requires provision of adequate clearance between bolts and at edges. Access for bolting can also be a problem, especially as the bolts are usually large (>M24) and therefore large, long spanners and/or pneumatic wrenches have to be used. These factors must be considered when nominating bolted joints.

A major concern with steelwork is of course the need for surface protection to prevent corrosion. This is covered in detail in Chapter 18 of this Manual.

20.3.2 Reinforcement

Some aspects of the use of reinforcement in bridges have been covered in the section on concrete, specifically the need for adequate cover, especially in potentially corrosive environments; and correct detailing to provide adequate space between bars to allow for proper compaction of the concrete.

Cover requirements for a particular situation can be calculated from Part 5, Clause 4.10 of the CODE. Note that, in some situations, especially for foundations, it may be necessary to test the soil and/or ground water to establish the correct exposure classification. Special measures may be necessary in extreme cases, or where the required cover cannot practically be achieved. One method, external waterproofing of the concrete has been covered previously. Another alternative is to provide a protective coating on the reinforcement. This is especially useful in thin structural elements (e.g. precast parapet panels), where the thickness of concrete would otherwise be inadequate for protection. Galvanising is the preferred treatment, although there are currently doubts about the degree of additional life it provides. It should be carried out after the reinforcement has been bent. Epoxy coating has also been used, but is now not recommended as any small holes or faults in the epoxy tend to lead to accelerated local corrosion. Also, as the epoxy is an insulator, epoxy coated bars preclude the future use of cathodic protection.

In the extreme, if there is serious concern about long term durability and corrosion of the reinforcement, then cathodic protection can be used. This is an expensive solution and only used as a last resort. One safety feature that should however be in all substructures, or at least those in aggressive environments is to provide an external connection to the reinforcement so that cathodic protection could be installed easily at a later date.

A further alternative, now receiving serious consideration and already in use in some parts of the world, is stainless steel reinforcement. Although too expensive for general application, a cost/benefit analysis may justify its use in selected areas, e.g. footings and columns in the splash zone. The selected use of stainless steel reinforcement may increase the initial construction cost, but it provides long term durability and thus a saving on future maintenance costs. The initial cost premium may also be less than the ongoing costs of cathodic protection.

Satisfactory detailing is hard to master as it depends mostly on experience. Some guidance is provided in the "Reinforcement Detailing Handbook" produced by the Concrete Institute of Australia.

Some general rules are:-

- always stagger laps in adjacent bars.

- be careful when using closed stirrups. It may be difficult to get the longitudinal steel into the enclosed cage and there is no tolerance adjustment. Two U-bars may be better.

- if things get really congested, e.g. anchorage zones, consider the use of welded stirrups.

- have reinforcement in all faces of an element, even where it does not seem to be needed, eg tops of walls, sides of footings. It will serve to control cracking in the event of any unforeseen distress, e.g. AAR.

- try and make sure the reinforcement forms a rigid cage, e.g. in footings and pile caps, as it will be much easier to work from when placing concrete.

- consider the possibility of pre-assembling the reinforcement cage, especially for footings and piers, the contractor may wish to do this to expedite construction.

20.3.3 Prestress Cables

Prestress cables are to some extent similar to large reinforcing bars and the above comments concerning cover and spacing apply equally to them.

The main design consideration with cables is to ensure they are not too long or too big to place, and that there is adequate room surrounding them for satisfactory compaction.

There are various methods of placing prestress cables. They can be fully preassembled and lifted in by crane, or even "walked" in manually if light enough. This obviously requires an open stirrup arrangement in the deck. Alternatively, the empty ducts can be positioned and then the cables pulled or pushed through before or after concreting. If the empty ducts are to be concreted then a temporary poly pipe liner should be installed for stiffness. With pulling, the whole cable is usually pulled through at once whereas pushing is strand-by-strand, (or wire-by-wire depending on the system). The latter is usually the preferred method.

If there are any doubts about the practicalities of the prestress system chosen for a particular bridge it is recommended to discuss matters with a specialist prestressing subcontractor at an early stage in the design. MRWA does not specify the use of any particular prestressing system, but because the systems are different from the point of view of cable sizes, friction, anchorage details etc, designs are usually carried out and drawings detailed based on a particular system, with the contractor being free to put forward an acceptable alternative.

MRWA almost exclusively use internal, bonded, fully grouted cables. There are both advantages and disadvantages to this. The main advantage is that the bonded cables make the analysis easier and significantly increase the ultimate capacity of the section. Disadvantages are that wider webs have to be used to cater for the cables and, although the grout should protect the cables, there is no way of inspecting and checking them. This means that if there is a problem, say due to corrosion, it will not become apparent until something goes wrong and then fixing the problem will be very difficult. On the other hand, external, unbonded cables can be inspected at any time, both visually and by checking the cable force, and if there are problems replacement is a simple matter.

There have been one or two failures of prestressed bridges overseas, without warning due to the internal, fully grouted cables corroding through. Because of this, external, unbonded cables, with facilities for future replacement (i.e. the bridge must be safe with one cable removed), are now being used. Admittedly the de-icing salts used in these countries contribute significantly to the problem, but even so perhaps consideration will have to be given to the use of unbonded cables in some high risk situations in WA.

Another new technique, currently being investigated in Europe, is the use of electrically isolated prestressing cables. These can subsequently be monitored to assess their condition.

20.3.4 Miscellaneous Items

Steel is used in various forms in a large number of miscellaneous items in bridges, e.g. hand and guardrailings, joints, bearings, gullies, cover plates etc. These are dealt with in detail in other Chapters in this Manual, plus the general comments above on fabrication, corrosion protection etc still apply.

20.4 TIMBER

Timber was used extensively in WA in the past for permanent bridge construction, and maintenance of this asset is an on-going task. Nowadays the major use of timber is in maintenance works and as falsework for concrete construction.

The use of timber in bridge construction and maintenance is covered in detail in Chapter 14and Load Rating and Refurbishment Manual for existing Timber Bridges (6706 – 02 – 2227), but it is worth noting here that one of the overriding considerations to lengthen the life of timber structures, is to detail so as to prevent water penetration and to provide positive drainage to all surfaces.

20.5 OTHER MATERIALS

Although concrete and steel are the major materials use by MRWA, there are a large number of other materials involved in bridge construction, such as:-

- Rubber and PTFE in bearings and seals.
- Glass, polycarbonate and acrylic in glazing, eg to footbridge canopies.
- Aluminium for railings, expansion joints and glazing bars.
- Sealants of various types in joints, eg silicone rubbers, polyurethanes etc.
- Bitumen and/or rubber/bitumen in waterproofing membranes.
- Geotextiles for soil reinforcement.
- Natural stones and various manufactured bricks for retaining walls and pitching.
- Glass Reinforced Plastic (GRP or fibreglass) and Glass Reinforced Cement (GRC) for cladding panels and permanent formwork.
- Stainless Steel for fittings in highly corrosive environments and sliding surfaces.
- Carbon fibre

There are too many to provide specific information here. Reference should be made to Australian, British or American standards and manufacturers' literature. One point that should be made though is to take care when using dissimilar metals in contact, to guard against bimetallic corrosion (see Chapter 18).

PART B – ELEMENTS

20.6 PILES

All aspects of piling, including optimum design, installation and durability/maintenance are dealt with in Chapter 7, which covers the various types of piling, timber, concrete and steel. To highlight some of the major points:-

Timber - In the right circumstances timber piles can still be used satisfactorily, however they are subject to both rot and insect attack, i.e. termites and marine borers. If they are permanently submerged in fresh water, or in the ground below water table then there are few problems, and in fact, in saline areas of the wheatbelt in WA, timber piles have proved more durable than either concrete or steel. However, unprotected piles must not be used in marine environments due to the prevalence of marine borers, and in other situations precautions must be taken against rot, especially in the alternate wet and dry region close to water table level, and termite attack. Refer Chapters 7 and 14 for details.

Steel - Precautions also have to be taken when using steel piles. Again if in the ground, permanently below the water table there will be few problems, but otherwise some form of corrosion protection is required. This is especially important for piles in salt water splash zones, where steel piles are not recommended, because of their high potential for corrosion. Corrosion treatments for piles are covered in Chapter 7, Appendix A and Chapter 18. Steel piles are also not recommended at locations where there is a very fast stream flow with abrasive material in the river channel.

Precast Concrete - Precast concrete piles are usually fairly straightforward to make and perform satisfactorily as long as the appropriate reinforcement cover and class of concrete are used. The main detailing problem is in the bursting reinforcement to prevent spalling of the head during driving. It is very difficult to design this area theoretically and past experience is vital. Again details are given in Chapter 7, Appendix A.

Cast Insitu Concrete - Cast insitu concrete piles can be more of a problem, especially uncased piles, as the achievement of satisfactory cover is very much dependent upon the construction method. For this reason the use of uncased piles is not recommended, see Chapter 7. Also, the actual placement of concrete in this type of pile has been a problem in the past and must be properly planned, with the concrete tremied or pumped and properly compacted to prevent segregation, voids and the like.

Installation of the different types of piles has also been previously addressed. A point to consider is adequate clearance at the site, both vertically and horizontally, for the pile driving rig. Overhead power lines and proximity to other buildings and structures can cause problems. Avoid underhand rake driving where possible or limit the rake to 1 in 10, as it is difficult to achieve safely. Proximity to housing or other sensitive buildings may also mean that the noise, vibration and dirt associated with driven piles is not acceptable and other forms of installation may have to be used, e.g. boring. Access is especially important when driving over water. If the flow is ephemeral, advantage can be taken of the dry season, otherwise a barge will have to be used, although if the water is shallow this may be difficult and a bund may have to be pushed out from the shore. The designer should first check however, that this is environmentally acceptable.

Piles which also act as columns (e.g. in most RC Flat Slab bridges) shall be oriented to follow any skew of the structure, for subsequent ease of assembly of deck falsework.

As pile driving is such a specialist business and one is never quite sure which contractors and types of piling are available, the use of a performance specification should be considered, ie. in the tender documents only provide the loads and other constraints, and let the contractor determine the most economical piling technique. Problems with this are that it will be necessary to check the design before installation can proceed, and substructure design cannot be completed until the type and number of piles is known. However on larger jobs it can produce substantial cost savings.

20.7 FOOTINGS

Both spread footings and pile caps have been covered previously, in Sections 6.4 and 7.10 respectively. The main design consideration is to note that these are usually large blocks of concrete, particularly pile caps, and it may be necessary to take special precautions to prevent thermal cracking. This may involve the use of special cements and/or insulated formwork (refer 20.2 above). Requirements for these must be detailed by the design engineer in the specification.

Construction of footings will invariably require excavation and careful note should be made of the proximity of things such as roads, railways, services, other foundations etc as these may have an influence on the location and size of the new footing. Constructing footings below the water table will also introduce difficulties for the contractor, as they will probably require an expensive dewatering system, and should be avoided if possible. Footings founded on rock can also lead to problems, especially if keying into the rock is required. It is often easier to use drilled dowels rather than have to cut into large areas of hard rock.

Footings are usually buried so formwork is fairly straightforward and proprietary "pan" systems are often used. It is the contractor's responsibility to design the formwork and any temporary works required, e.g. sheetpiling, but the design engineer must ensure there is sufficient room around the footing to facilitate construction.

20.8 COLUMNS

Columns are often one of the major architectural features of a bridge and so tend to come in an assortment of shapes and sizes. Both concrete and steel are used. Concrete columns are most popular because of the variety of shapes that can be obtained, however steel is also useful, especially as steel columns can be prefabricated and then positioned on site very quickly, although even these are usually concrete filled, for robustness under impact loads, or for structural reasons, e.g. stiffness to prevent buckling.

With concrete columns the designer should try and make all columns as similar as possible, so the contractor can obtain maximum reuse of formwork. It is normal to retain the same shape column and make any required length adjustment at the base.

Considerable care is required detailing reinforcement at the base of columns, as this is a congested area and it is especially important to get sound concrete in this critical zone. Kickers cast with the footing to provide a starter for the column are advantageous in separating the interface joint above the potential water penetration zone and for clamping formwork. However, particular care is required to get sound, well vibrated concrete in this area. With tall columns, reinforcement detailing should permit access inside if possible for vibrating the concrete during the pour.

Other aspects of concrete, reinforcement and steel have been covered previously.

20.9 ABUTMENTS AND RETAINING WALLS

There is not a lot to say about abutments and retaining walls, as most of the information has already been covered in the Sections on concrete and reinforcement. However there are some items that are peculiar to these elements, and these are covered below.

20.9.1 Joints

The spacing of joints is very important in abutment walls and retaining walls. The wall itself is usually poured onto a footing cast some time before. This means that there is considerable differential shrinkage between the two and unless control joints are placed at adequate centres vertical cracks will occur in the wall. Joints are required at a maximum of 3 metre centres. Most joints only need to be contraction joints, with expansion joints used at changes in wall thickness or where a construction joint is required.

20.9.2 Stepped Footings

With bridge abutment wingwalls, particularly curved or flared ones that go back into the rising ground of the bridge approaches, the footing level is often stepped to prevent excessive excavation and concrete. Stepping should be approached with caution as, although it does save on excavation, each step means another stage of construction, as the fill for the next level cannot be placed and compacted until construction of the preceding level is complete. This will slow down the job. Normally do not step less than 900 mm at a time, with a minimum soil cover on top of the footing of 300 mm.

20.9.3 Wingwalls

With bridge abutment wingwalls, it is normal to make them rectangular in elevation, ie. with a horizontal, not sloping, base. The base obviously could be sloped to match the abutment batter and so save some concrete, but it is usually not worth the trouble as it makes construction awkward since the base of the wall then has to be formed and subsequent compaction of the soil is difficult.

20.9.4 Limestone Walls

Limestone gravity walls can provide a fairly cheap and aesthetically satisfactory solution to many low to medium height retaining walls. They are usually economic to a height of around 3 to 4 metres. Although cut, natural limestone blocks have normally been used in the past, currently reconstituted limestone blocks are becoming more popular, due to their availability and consistent quality.

20.9.5 Mechanically Stabilised Earth Walls (MSE)

Mechanically Stabilised Earth, (also known as Reinforced Soil or Reinforced Earth) walls are sometimes used for abutment walls. These are proprietary items and design is normally done by the manufacturer, who should ensure that all details are satisfactory.

In the past, in conjunction with MSE walls, MRWA used sill beam type abutments, with the sill beam supported on top of the MSE "block". However, due to problems where fill has been washed out of the joints between the MSE wall panels it is now mandatory for the bridge abutment to be supported on piles or a spread footing below the "block", and the bridge must be designed accordingly.

If specifying an MSE wall check that the founding soil for the reinforced earth block is adequate and there is a source of acceptable fill material, (there are special restrictions on physical and chemical properties of fill for MSE).

20.9.6 Construction of Retaining Walls

Before designing a retaining wall give careful consideration as to how it will be built. If it is to be filled behind after construction then there will be no problems, but if the wall is in a cut situation then there could be, as it may be necessary to hold back the cut face temporarily while the wall is being built. This will involve the use of sheet piling, or perhaps specialist techniques, such as soil nailing. Often the cost of temporary works will be a large proportion of the cost of the wall and should be considered very carefully.

Drainage is another issue that must be considered at the design stage. A drainage layer should be included immediately behind the wall, with weepholes or a piped outlet, to prevent the build up of water pressure on the wall.

20.10 SUPERSTRUCTURES

There are many different forms of bridge superstructure. The principal ones used by MRWA, with their major advantages and disadvantages, have been covered in Chapter 2. Notes on the construction specific aspects of each type are provided here.

20.10.1 General

Construction Method - The construction method must be considered very early on, as it can have a significant impact on the design. This can vary from minor, such as making provision for attachment of formwork; to major, where the design has to be carried out around the construction method, e.g. stage-by-stage construction, incremental launching etc.

Construction Clearances - If a bridge is to be built over an existing road, railway or navigable waterway then consideration must be given to maintaining clearances, both vertical and horizontal, during construction. Some reduction to permanent clearance may be acceptable, but this should be checked carefully, especially vertical clearance over roadways. Although the legal height limit is 4.3 metres, over-height vehicles are around and the results of an impact with bridge falsework could be catastrophic. Therefore, a minimum clearance of 4.8 metres during construction is preferred, with 4.6 metres as an absolute minimum, and only then with the specific approval of SES. This low clearance would require the use of an advance warning over-height detector, and associated signing.

If there is insufficient room for falsework, or when working over a busy road or railway, especially if electrified, some form of prefabricated structure or incremental launching should be investigated.

20.10.2 Timber

Timber is rarely used nowadays, apart from widenings, overlays and other refurbishments. Standard details have been developed over many years of experience in the field and should only be changed for good reasons and after consultation with senior staff and construction personnel.

20.10.3 Reinforced Concrete Flat Slab

It is a fairly straightforward and standardised form of construction.

One should give some thought as to how the bridge will be built, especially when used over water. It is preferable to keep any falsework out of the riverbed. This is usually accomplished by providing support directly off the columns or piles, (where the piles extend into the deck as columns). This will involve welding to steel piles, or clamping, drilling and/or the use of hangers with concrete ones. Ensure that the detailing in this area permits this and specifically ensure that piles are oriented following any skew.

20.10.4 Prestressed Concrete Planks

The major variable will be the method of transverse connection of the planks, either transverse prestress or a concrete overlay. If transverse prestress is to be used, ensure the duct holes are of adequate size, otherwise it may be difficult to get the stressing bars through, as there are usually differing amounts of hog in the planks.

20.10.5 Prestressed Concrete I-Beams

Construction points to be noted are:-

- The positive moment connection at the bottom of the beam is important. The standard welded detail is awkward to perform on site, but can be done and should not be changed without very good reason and consultation with senior staff.

- Most bridges have the beams joining at a diaphragm at the pier with the diaphragm then supported on rubber bearings on a crosshead. With this form of construction the beams will have to be supported temporarily on timber blocks, or sand jacks, off the crosshead while the diaphragm is cast, so ensure the crosshead is wide enough for this.

For aesthetic reasons, some bridges have dispensed with a pier cross head and the pier diaphragm is supported directly on the columns. This type of construction is much more difficult, as the beams have to be supported independently while the diaphragm is cast, and it should only be used where really required.

- The type of formwork used for the deck is another variable. The easiest is some form of permanent steel formwork, e.g. Bondek or Condeck, which can be supported directly off the top surface of the beams. If specifying this make sure it is strong enough to carry the load of the wet concrete and other construction loads, and that there is sufficient room available on the top flange of the beam for proper, safe seating so there is no chance of the sheeting moving, (typically allow 40-50 mm). There must also be adequate clearance to reinforcement from the forms. Usually the main, transverse reinforcement goes in the trough of the sheeting, with the longitudinal steel above it at a minimum of 10 mm above the peak of the sheeting profile. Sometimes an off-form finish is required to the soffit between the beams and in this case holes should be left through the beams for the contractor to bolt on bearers for this formwork system.

20.10.6 Steel I-Beams

Comments similar to those for concrete I-beams apply here also. Continuity is achieved by bolting, welding and/or a concrete diaphragm. Deck soffit formwork can be permanent or conventional and adequate seating area is again required, although puddle welding is normally specified to ensure the permanent formwork does not move during construction.

20.10.7 Prestressed Concrete Insitu Tee-Beams

This type of construction usually has simple geometric shapes and detailing and pouring concrete is fairly straightforward. The deck is poured in one operation so stage-by-stage or some other form of incremental construction may be required on longer bridges. The normal freeway type overbridge however can normally be poured in one.

20.10.8 Prestressed Precast Concrete Tee-Beams (Teeroffs)

Teeroff type beams are now a standard form of construction. The major site problem is to make adequate provision for the beam hog when setting bearing heights and deck thickness. The hog of the beams varies depending on the age after casting. It is important that this be monitored and the bearing height adjusted accordingly so that the finished deck level is correct. Because of the hog, the deck thickness in midspan will be the minimum, with substantially greater depth over the supports.

20.10.9 Prestressed Concrete Multi-Cell Boxes

This form of construction is now not used much, being replaced by voided slab construction or precast concrete beams. Decks are usually cast in two pours, base slab and webs first, followed by the deck slab. Reinforcement should therefore be detailed accordingly. Consideration should also be given to using permanent formwork for the soffit of the top slab as removal of conventional formwork is difficult. Temporary manhole(s) need to be provided in the top slab if conventional formwork is to be used, so it can be removed later.

All cells should have drainage outlets in case water gets in during construction or even after completion.

20.10.10 Prestressed Concrete Voided Slab

These are used in similar circumstances to multi-cell boxes. Correct detailing of the void duct and associated reinforcement is very important. Ducts should be helical lock seam corrugated galvanised steel pipes of 1.6 mm minimum sheet thickness. Voids made of thinner sheeting should not be used as they have shown a tendency to deform under the pressure of wet concrete. The fixing of the duct is very important so that it does not float or become otherwise displaced while pouring concrete, but this is the contractor's responsibility. Minimum concrete thicknesses above and below the voids are usually 200 mm and 150 mm respectively. The minimum clear distance between ducts will depend on the size and number of prestressing cables used. There should be adequate room for a 65 mm diameter concrete vibrator.

Always provide drainage outlets to voids as a safety measure, even though they should be watertight.

20.10.11 Prestressed Concrete Boxes

These are used for larger and longer bridges and the construction method will usually have a significant influence on the design. Common construction methods are stage-by-stage construction and incremental launching and both will require the structure to be designed around the construction method. So would the use of cantilever construction or a launching girder, although these methods have not been used by MRWA.

Reinforcement detailing is important as areas are often congested making concrete placement difficult. Web thicknesses will depend on the prestress and shear requirements, but should be sufficient for concrete placement and access of a 65 mm diameter poker.

20.10.12 Steel Box Bridges

These are not often used by MRWA, except for footbridges. The main construction points are to have the prefabricated units as large as possible, so there are less field joints, but not so large that they cannot be transported and handled easily. Also, splices should be positioned away from roadways, as scaffold supports will be required and perhaps welding. The field splices themselves may be either welded or bolted. Bolted joints are easier to do on site, provided there is adequate room for the required number of bolts and access to tighten them. Also there is minimal damage to surface treatment. Welded joints are more flexible though and fabrication tolerances do not have to be so tight.

20.11 MISCELLANEOUS

This last section of this Chapter will deal with the numerous small details on bridges that have not been covered elsewhere, or if they have then with their construction related aspects.

20.11.1 Railings

These should be simple to fabricate, surface treat and install. Do not try and bend RHS rails around tight (<2.5 metre radius) curves, as the section will distort. If using hot dip galvanising ensure that hollow sections can be properly vented for safety, or otherwise use solid sections. Use bolted connections on site, welding damages surface treatment and cannot be properly inspected/tested in RHS and CHS sections.

20.11.2 Bearings & Joints

For most details refer to Chapter 16. Ensure there is access to bearings for inspection and possible future replacement. Also for replacement ensure there is adequate strength in the deck, (eg a solid diaphragm), near the bearing where jacking will take place.

For joints ensure there is provision for drainage underneath, (even if supposedly watertight), and for cleaning if using an open joint.

20.11.3 Drainage

Provision of drainage on bridge decks is to be avoided if at all possible, as it is an additional complication. The road profile should preferably be such that water can be collected at the abutments, before getting on the deck or after leaving it. The bridge should **not** be in a sag or on a level stretch of road. Make sure that the run-off cannot scour out the abutments.

On long bridges where drainage is unavoidable, the first choice is simple scuppers discharging straight through the deck. If this is not possible, e.g. a bridge over another road or railway, or due to environmental restrictions, some form of pipe collection system will be required, although this can be difficult on bridges with long cantilevers.

It is also important to ensure that all horizontal surfaces on a bridge, eg abutment bearing shelves, have positive drainage, as water lying about for substantial periods hastens deterioration of all materials, concrete, steel and timber.

20.11.4 Waterproofing

All bridge decks should have a waterproof membrane to protect the reinforcement and prestress against corrosion if the deck cracks. Spray-on rubber/bitumen is usually adequate, with preformed, self-adhesive sheeting, e.g. Bituthene or Bitac, used in awkward areas, eg under kerbs and medians, or where there is greater possibility of leakage, eg construction joints.

The above materials have been used successfully in the past and any proposed new materials should be carefully checked before use, as if subsequently found to be faulty they are very difficult to repair or replace.

20.11.5 Approach Slabs

Approach slabs are used on most bridges where there is any possibility of the approaches settling. A minimum length of 5 metres is recommended. A simple slab on ground is usually adequate, and as it settles it can be filled with bituminous concrete road surfacing (BC).

If large settlements are anticipated, >50 mm, then a much more complex jackable approach slab will be necessary (refer to Narrows Interchange bridges).

Ensure there is an adequate seal between the approach slab and the wingwalls, as otherwise water can penetrate and erode the embankment fill.

20.11.6 Services

Services can be carried on most bridges and the requirements of the various authorities should always be checked. Usually the provision of simple plastic pipes will suffice, but for large services, e.g. water, sewerage and gas, special fittings, expansion joints etc will be necessary. High pressure gas mains **must not** be carried in enclosed spaces, e.g. inside box girders, because of the danger of explosion. Also in enclosed boxes, drainage holes should be provided when water and sewerage pipes are carried, in case of rupture. The size of hole depending on the size of the service.

20.11.7 Maintenance Access

Always make provision for maintenance/inspection access to all parts of a bridge. For smaller structures, ladders or the underbridge inspection unit may be adequate. However, larger, higher structures will require more specific measures to be taken. Manholes may be needed through diaphragms, rails for travelling gantries, light and power in enclosed boxes etc. A structure is much more likely to receive regular inspection if it can be carried out safely and easily.

CHAPTER 21

CONTRACT DOCUMENTS AND CONSULTANTS' BRIEFS

CONTENTS

21.1 INTRODUCTION

The objective of this Chapter of the Manual is to provide an overview of basic information on construction contracts and consultants' briefs for those engineers with little contract experience. The emphasis is on construction contracts as these are the most complex. Material covered includes:

- Definition of a Contract
- Types of Contract
- Content of typical contract documents
- Tendering Procedures
- Contract Roles
- Contract Management
- Specifications
- Consultants' Briefs

21.1.1 Overview of MRWA WA Contract Documentation

MRWA has developed a comprehensive set of documentation for its contract management systems. The key Manuals are referenced below.

a) Tendering and Contract Administration Manual

This is perhaps the most important reference manual as it describes the procedures and processes involved in tendering and administration of MRWA contracts. It is quite comprehensive in covering all aspects such as the tendering process, e.g. planning, tendering, assessing and awarding contracts while administration involves post award activities associated with insurances, securities, contract management plans, time and cost, priced documents, variations, extensions of time, assignment, contract finalisation and document destruction. This Manual should be referenced, including the Appendices that summarise a great deal of this information in chart form and at the same time provide a good illustration of the involved nature of the process.

b) Procurement Management Manual

This manual describes the corporate management system and documents the corporate policies for the procurement of all goods, works and services within MRWA. It aligns MRWA policy with the Department of State Supply requirements.

c) Service Contracts Procedure Manual

The purpose of this manual is to describe the procedures for engaging and managing consultants, contract personnel, other service providers and for contracting out services for delivery by the private sector.

d) Purchasing Manual

The purpose of this manual is to describe the procedures for the purchase of all goods and services < $10,000 within MRWA.

Note that the above manuals address standard MRWA contracts and do not cover other styles of contracting such Design & Construct Contracts and Alliance Contracts.

e) Delegation of Authority

This is an important reference document as it provides guidance on the limits of personnel's delegated authority for a range of activities including purchasing.

f) Tender Document Preparation (TDP)

MRWA has established on-line through its website a complete set of standard forms required for contract documentation preparation including standard Specifications. This is discussed further in Section 21.4.

g) The Infrastructure Delivery Directorate

The Infrastructure Delivery Directorate has developed its own documentation to support such contracting mechanisms as Design & Construct contracts. The standard specifications are called the Scope of Works and Technical Criteria (SWTC) which is used in conjunction with a Deed of Agreement.

21.2 WHAT IS A CONTRACT?

Whole books have been written on the definition of a contract and general contract law. However the main point is that a Contract is an agreement, enforceable in law, between two parties, normally, but not necessarily, in writing, by which rights are acquired by one to act on behalf of the other.

The elements of a valid Contract are:

- An Offer by one party to do something for a consideration; and
- Acceptance of this Offer by the other party.

In civil engineering contracts the Contractor usually makes the Offer in the form of a tender to do the works specified, under the conditions of the contract and the Principal (in our case the Commissioner of MRWA) accepts the tender and agrees to pay the Contractor at the rates contained therein.

Contract documents comprise the General Conditions of Contract and any Special Conditions, Specifications (both Technical and Management Requirements), Schedules of Rates and Prices (Schedule of Rates Contract), Bills of Quantity (Lump Sum Contract), Drawings, the Form of Tender and the Form of Agreement.

A contract is usually executed under seal on the Form of Agreement to ensure it is legally enforceable, although it probably would be anyway if it could be proved that the basic elements of an Offer, the consideration and acceptance exist. It is important to be aware of this. A verbal agreement, freely entered into, will usually be looked upon by the courts as a binding contract.

21.3 TYPES OF CONTRACT

21.3.1 General

There are three basic types of contract used in civil engineering works, these are:

> ***Cost Plus*** - Under this type of contract the Contractor is paid the actual costs incurred in carrying out the work plus a fee, either a fixed sum or a percentage of the cost. Cost plus contracts are normally only used where it is essential that the work is started before planning, design and drawings are finalised. They have the obvious

danger that there is little opportunity for competitive tendering and little incentive for the contractor to do the work economically. They are rarely used by MRWA.

Lump Sum - Where it is possible to accurately define the works beforehand it is possible to get a Contractor to provide a fixed price for the contract. It is usual to still produce a Bill of Quantities and obtain unit rates from the Contractor to assist with interim payments. Changes to quantities can still be made, but become contract variations for which the price has to be agreed. This type of contract can be used for bridgeworks, where quantities are usually fixed, (except if piled foundations are involved), but rarely for roadworks.

Schedule of Rates - This is the usual type of contract used by MRWA. The documents include a schedule of approximate quantities covering the whole of the work and the Contractor enters a rate against each item at the time of tender. As the work progresses each item is remeasured and the Contractor is paid periodically for actual work done at the tendered rates. There are limits set in the documents on how much quantities may vary between actual and those in the schedule and rates can be renegotiated if there is a large variation.

21.3.2 MRWA Contracts

MRWA uses all of the types of contract mentioned above, but the actual method of forming the contract will vary depending on the size and complexity of the work.

Quotations - Direct quotations are used where the value of the contract is reasonably low (< $100,000), there are few local Contractors, the work is straightforward, the period is short and/or the requirement is urgent. The use of verbal or written quotes or public tender is in accordance with State Supply requirements. Refer to the Purchasing Management Manual for details and conditions. In Structures Engineering Branch this form of contract could be used for the supply of materials, purchase or hire of items of equipment.

Minor Works and Services Contracts - These are full contracts, as described above, but used for non-complex works where smaller Contractors are likely to be involved and so an abbreviated, simplified set of General Conditions is used. They can be either Bill of Quantities/Lump Sum or Schedule of Rates. In Structures Engineering Branch this form of contract is used for the supply of bridge bearings, precast concrete beams and piles and similar works.

Major Works Contract - This is the form of contract used for normal road and bridge works and is the one covered in the remainder of this Chapter.

Period Contracts - MRWA has also used period contracts where the supply is spread over a period of time. This type would be used for supply of culverts, standard reinforcing steel etc. The essential documentation is as for works contracts.

21.4 CONTRACT DOCUMENTATION

The basic contents of a typical contract document are shown below:

- Information for Tenderers (not always provided)
- Conditions of Tendering (inc. any Special Conditions)
- Form of Tender
- General Conditions of Contract (inc. any Special Conditions)
- Form of Agreement
- Technical Specification
- Quality System Specification

- Schedules of Rates (or Bills of Quantity)
- Drawings
- Addenda
- Letters/Minutes of Meetings

MRWA has established through its website a complete set of standard forms required for the compilation of a contract document. The Tendering and Contract Administration Manual considers all these components in detail. A brief overview is provided below.

21.4.1 Information for Tenderers

This is usually only used on larger Contracts, or those in remote locations and could include such things as geotechnical information, location of water bores or borrow pits etc. It should only contain pure information, e.g. bore logs, and **not** assumptions based on this information, this must be left to the tenderer. The Information for Tenderers is for tendering purposes only, must be clearly marked as such and does not form part of the final contract documents.

For Bridgeworks the geotechnical information should always go in as Information for Tenderers.

21.4.2 Conditions of Tendering

These give the rules for tendering and the actions Tenderers have to follow to submit a valid tender. There is a standard MRWA document, which must be used at all times. Special Conditions of Tendering would be used to cover any variations to the standard conditions, e.g. such things as alternative tenders.

21.4.3 Form of Tender

This is the standard form Tenderers must complete to confirm their acceptance of the terms and conditions of the Contract, i.e. to make their Offer valid.

21.4.4 General Conditions of Contract

These are the basic legal and administrative rules applying to the Contract, setting out the powers and responsibilities of all the parties involved. There are numerous General Conditions used in the construction industry, but the one used by MRWA for Category 2 construct only contracts is that issued by the Australian Standards AS 2124. On Minor Works and Service Contracts different, much simpler General Conditions are used and are included within the contract documenation.

An annexure to the General Conditions must be completed for each contract, giving essential specific information for that particular Contract, e.g. amounts of security and insurance, contract period etc.

The General Conditions have been developed over a number of years and should only be changed with extreme care and possibly after legal advice. If necessary this is done via the Special Conditions of Contract. These are amendments, additions or deletions to the General Conditions. Typical examples would be rise and fall, special conditions pertaining to working over railways or close to services etc.

Special Conditions take precedence over General Conditions.

21.4.5 Form of Agreement

This is only completed following acceptance of the Tender and is the formal record of the Offer and Acceptance. Once again a standard form is used.

21.4.6 Specifications

Technical Specifications - These are contract specific and give details of the work to be carried out. Each section of the specification defines the quality of materials and acceptance standards for the work. It is essential that the Specification is clear, complete and concise. More contract disputes can be traced to poor specifications than any other single cause.

All MRWA standard Technical Specifications are accessible through the MRWA website (Tender Preparation). The 800 series of standard Specifications is specifically allocated for bridges and major structures.

Management Requirements - This is a separate section within the Tender Document Preparation section and provides for management requirements such as Quality Assurance, Occupational Health and Safety, Traffic Management and Environmental considerations. These are standard documents, but information in the Appendices is contract specific and must be completed for each job.

21.4.7 Schedules of Rates/Bills of Quantity

In form these are fairly similar, although as mentioned previously they are used in different types of contracts. These provide a brief description and give the quantity of the different types of work to be carried out and are usually divided into three sections:

General Obligations - Covering general items such as insurances, supervision, site establishment etc;

Works - The major part, which gives details of the work to be carried out; and

Contingent Works - Day-works, contingency sums etc.

Methods of itemising work are usually in accordance with AS 1181 - "Method of Measurement of Civil Engineering Works and Associated Building Works". Schedules are prepared by a Quantity Surveyor to a standard format. Extensive use is made of preambles, to avoid having to go into great detail in the individual item descriptions.

21.4.8 Drawings

The drawings are arguably the most important part of the contract documents, as they define the work to be carried out. Drawings should be properly checked in accordance with Branch Procedures, amendments signed and a careful record kept of issued drawings.

21.4.9 Addenda

The Addenda to a contract are those changes, additions and corrections issued during the tender period. They form an essential part of the contract and must be included in the final signed document.

21.4.10 Letters

Similar to addenda, these are letters exchanged between the MRWA and the successful tenderer during the tender negotiation period. They will probably provide clarification on items in the contract and again form an essential part of the Contract and must be bound in the final document.

21.4.11 Site Meetings

If site meetings are held then minutes must be taken and these again are bound in and become part of the Contract.

21.5 TENDERING

21.5.1 Types of Tender

There are two common methods of inviting tenders:

> ***Open Tendering*** - Here advertisements are placed in the press and any organisation may submit a tender for the works. This should theoretically result in the lowest price, but has the drawback that inexperienced firms may win a contract based on an unrealistically low tender leading to claims and disputes later.
>
> ***Preselected/Prequalified Tenderers*** - With this method tenderers are preselected on their ability to carry out the work in question and only those Contractors are invited to tender.
>
> Preselection/Prequalification may be for individual projects, on a project by project basis, or for categories of work, (categorised by size/value and/or technical complexity), with lists of approved Contractors in each category. It has the advantage that the tender assessment period is reduced as only "approved" Contractors, for that class of Contract, are invited to tender, therefore price becomes the principal deciding factor. A list of prequalified Contractors is accessible through the MRWA website (Contracting to MainRoads/Prequalification).

21.5.2 Tendering Procedures

MRWA tendering procedures are currently covered in the Tendering and Contract Administration Manual. The Manual should be referred to for full details, however, the basic steps are:

> ***Preparation of Documents*** - The full set of contract documents as described above are put together. For an in-house bridge design, the Technical Specification will be done by the engineer in charge of the bridge design, with assistance as required from external specialists in specifications and quantity surveying.
>
> When considering timetables for tendering ensure that sufficient time is allowed for the printing of documents. There are usually many sets to run off and at least 1 week should be allowed for printing, collating and binding. There should also be time allowed for the designated Superintendent to review the Documents.
>
> ***Approval to Call Tenders*** - This has to be in accordance with the Delegation of Authority Manual and the approving authority will vary according to the value of the contract and source of funding.
>
> ***Advertising for Tenders*** - All tender advertising is handled by Supply Section. Adequate notice should be given to enable adverts to be placed in the local press. Supply Section will also issue all documents and any subsequent addenda.
>
> ***Tender Period*** - During the tender period the design engineer should be available to answer any queries from tenderers. If it is necessary for additional information, revised drawing etc. to be provided it is essential that the same information is provided to all tenderers, in the form of an official addendum.

Addenda are only issued through Supply Section and again approval is necessary as per the Delegation of Authority Manual.

Receipt of Tenders - Tenders are received by MRWA up to the time stated in the documents. Late tenders are ***NOT*** accepted. Tenders are usually placed in the tender box. Tenders are opened by the Contracts Officer and a quick check made at this time that all tenders are complete and unconditional. A list of tenders received, in ascending order of price, is published so that contractors know if they are in the running for the job or not.

Tender Assessment - It is recommended practice for the future Superintendent to carry out the tender assessment and prepare the recommendation for acceptance, with Structures Engineering Branch consulted on technical matters. Usually only the lowest two or three tenders are looked at in detail. The essence of the assessment is to ensure the tender is numerically correct; the tenderer fully understands the work; is technically and financially capable of carrying it out; any alternative tenders are assessed; the Management Requirements are checked; and that conditions and qualifications on the tender are removed or accepted by MRWA. All matters of consequence raised during tender discussions, by either party, must be confirmed in writing.

Award of Contract - Recommendation as to the preferred tenderer, with adequate supporting information is forwarded by the assessing officer for approval. This will be in accordance with the Delegation of Authority Manual. Again make sure time is allowed for this approval process as it can take up to 3-4 weeks.

Subsequent issuing of the Letter of Acceptance, receipt of insurances and bond, official signing of documents etc. is all handled by the Supply Section.

21.6 CONTRACT ROLES

The management of contracts involves both administration, dealing with the legal and contractual requirements of the contract and supervision, ensuring that the works are carried out in accordance with the drawings and specifications. To perform these functions there are a number of established roles, some formally defined in the General Conditions of Contract and others used as appropriate.

21.6.1 Principal

The Principal is one of the two parties to the contract, the client for whom the work is being done and provides payment for that work. On MRWA contracts the Commissioner is always the Principal.

21.6.2 Contractor

The other party to the contract, who is carrying out the specified works for the agreed consideration.

21.6.3 Superintendent

The Superintendent is responsible for the management of the contract, but *<u>is not a party to the contract</u>.* This point is most important. The Superintendent's duties and responsibilities are laid down in the General Conditions, but basically the role is to see that the conditions of the contract are adhered to, by both parties. The Superintendent has no authority to alter or waive any provision of the contract without the agreement of both parties, however the

General Conditions of Contract gives the Superintendent authority to issue directions such as to issue Variation Orders.

On MRWA contracts there is the additional complication that the Superintendent will be an employee of the Principal, but must still strive to be impartial and unbiased and the principle of fairness to both parties must be adhered to.

21.6.4 Superintendent's Representative

Usually the person who looks after the day-to-day running of the job on behalf of the Superintendent, especially the direct supervision on site. The limits of responsibilities will depend on the powers delegated by the Superintendent, which should be conveyed, in writing, to the Contractor at the commencement of the contract, in accordance with the General Conditions of Contract.

21.6.5 Contractor's Representative

Similar duties to the Superintendent's Representative, but on behalf of the Contractor.

21.6.6 Works Inspector

Assists the Superintendent's Representative in the supervision of the works. Name and powers must also be conveyed to the Contractor officially. The General Conditions of Contract recognises the role of Clerk of Works and Inspector but does not provide for the Superintendent to delegate authority to them under the terms of the contract. Hence their role can only be to observe and report to the Superintendent on the progress of the works.

21.6.7 Sub-Contractors

Any sub-contractors used by the main contractor are the Contractor’s responsibility. The MRWA Contract is with the Contractor and while parts of the work can be sub-let responsibility for any part of the contract cannot be passed to another. The exception to this is where MRWA nominates a sub-contractor to be used, but this is rarely done.

21.6.8 Principal's Representative

A position created when a consultant is the Superintendent for a MRWA contract. Has no contractual standing, but is a point of first contact and provider of advice to the consultant and ensures that the Principal's interests are properly represented.

21.7 CONTRACT MANAGEMENT

This is not really of direct concern to Structures Engineering Branch staff however it should be noted that MRWA has comprehensively detailed Contract Management guidelines. The main thing is that the designer should be available to provide quick answers to any queries from site and should also take an interest in activities on site and make site visits at appropriate times. Design and detailing can only improve if the designer has an appreciation of the problems associated with actually building what has been designed.

It is important to note that the designer has no role under the Contract and all dealings with the Contractor must be through the Superintendent or Superintendent's Representative.

Even with the best documented and managed contract, contract disputes do arise. The Superintendent will usually try and solve any problems with the Contractor directly and promptly; failing this the Contractor will appeal to the Principal for a decision and if still not satisfied can ultimately go to arbitration.

The designer is only likely to be involved in disputes to provide technical advice. With this in mind it is essential that written record is kept of any advice or instructions given to the Superintendent during the Contract or to any other party to the Contract. It is also worth noting, that in the event of a dispute going to arbitration, the Contractor's lawyers will have full right of access to all MRWA files, so care is needed when placing material on file, especially where one is expressing an opinion rather than stating facts.

21.8 SPECIFICATIONS

It is important that the specification clearly sets out the work to be done, including the materials to be used and the acceptance criteria for the finished product. Over-specification must be avoided, especially specification of potentially conflicting requirements, e.g. do not specify both the driving depth and the set at that depth for a pile. Also avoid method specification and concentrate on results and acceptance criteria, unless it is critical, e.g. part of the design assumptions and leave how the work is done to the Contractor.

For bridgeworks the Tender Documentation standard Specification is available and should be used on all major construct-only contracts, suitably modified to suit the particular job and site.

21.9 CONSULTANTS' BRIEF

Currently MRWA has three period service contracts which include the supply of bridge design services. These are titled Engineering and Technical Services and Bridge Design Services (ETS-BDS). The contracts provide MRWA a simplified method of accessing resources for a wide range of technical services. Procedure Document 15/20/12 is available on-line for guidance on using these contracts.

Structures Engineering personnel may need to utilise the services of consultants outside the current ETS-BDS contracts. This would then require the preparation of more detailed briefs, assessment of the submissions, engagement and on-going management of the contract. These will be mainly in the bridge design area, but could also include waterway analysis, research, software development etc. There is a standard Brief for Consulting Engineering Services in the MRWA website under Standards & Technical, Road and Traffic Engineering, Typical Design Processess, Standard Design Brief (Document No. 04/10064), which deals mainly with design services, but can also be applied to other forms of consultancy.

The general principles governing the engagement and management of consultants within MRWA is covered in the Services Contract Procedures Manual which includes both obligatory Procedures and general Guidelines.

Procedures for the writing of Briefs, calling of quotes, assessment, award and management of consultants are very similar to those for contractors already described, although they tend to be less formal and more flexible, in part because of the difficulty of defining in full detail the work to be done. Public tenders are rarely used, invited tenders being the norm, with firms known to be capable of carrying out the work to the required standard asked to put forward a submission.

21.10 CONCLUSION

This Chapter is intended only to provide a basic introduction to contract documentation and Consultants' Briefs in MRWA. For further information the detailed documents referred to in the text should be consulted.

CHAPTER 22

BRIDGE AESTHETICS

CONTENTS

22.1 INTRODUCTION

Main Roads Western Australia (MRWA) is an advocate for good design fitting of the environment and this Chapter aims to lead and encourage both engineers and owners to seek aesthetically pleasing bridges that reflect and enhance their environment through the adherence to selected aesthetic principles and guidelines that must be considered in the design of MRWA structures.

The subject of form and aesthetics for bridge structures is a complex one and cannot be covered comprehensively in a single Chapter. As a consequence the approach here has been to include selected aesthetic design principles that relate to the most common MRWA bridge types, with references that provide further guidance and information.

In addition to the principles, the Chapter also details a number of aesthetic design 'rules' in Appendix A. The requirements and formulas detailed therein are to exclude the application of the unattractive engineering design solutions and not intended to preclude design solutions that provide, in the view of MRWA, genuine aesthetic excellence. MRWA does not want design by a set of standard solutions or the automatic application of any particular formula, but to ensure that all aspects of visual excellence inferred from the principles and references are considered and achieved.

In preparing this Chapter, MRWA has relied not only on its own practical experience but acknowledges the extensive information and inspiration drawn from the references listed below and recommends their consultation:

1. Aesthetic Guidelines for Bridge Design (Minnesota Department of Transport, USA)
2. Bridgescape – The Art of Designing Bridges (Frederick Gottemoeller)
3. Bridge Aesthetics (RTA NSW)
4. The Design and Appearance of Bridges (Highways Agency, UK)

22.2 GENERAL CONSIDERATIONS

The following paragraphs have been drawn largely from Reference 1 which should be consulted for a more complete coverage of the topics.

22.2.1 Fundamentals of Aesthetic Design

The study of aesthetic design fundamentals includes the consideration of the visual relationship of a bridge and its site, as well as mass, shape and form of the structure.

The two visual concepts used to develop, describe and express visual ideas are: visual design elements and aesthetic qualities. The former defines visual perception and concepts. These elements include line, shape, form, colour and texture. Aesthetic qualities result from employment of visual design elements and are used to describe a visual composition. Aesthetic qualities include proportion, rhythm, order, harmony, balance, contrast and scale.

22.2.2 Aesthetic Design Objectives

There are a number of aesthetic design objectives that should be considered as a fundamental framework that designers can use to initiate the application of aesthetic design. Some key objectives are listed below.

Scale and Proportion: The primary structural components such as span lengths, girder depth, abutment height, should have good proportional relationships with each other. Generally, no single component should dominate the visual composition. The collective design of the structure should be in scale with the site and environmental considerations. The structural form should have an appearance of lightness.

When evaluating the scale and proportion of a structure, designers should ask such questions as: Are the substructure components proportional to the superstructure? Does the superstructure appear slender without appearing delicate, or is it ponderous? Does the proportion/size match the visual expectation required to carry the load?

Order and Balance: Order is achieved by limiting the direction of lines to a minimum. When evaluating order and balance of a structure, designers should ask: Does the arrangement of components work together as a unit or promote visual confusion? Are the lines limited to a few directions? Do the visual weight, texture and mass of the members promote visual balance?

Simplicity and Continuity: The bridge form should appear straightforward and uncomplicated. Simplicity of form and clean line are considered attributes of attractive structures. Shapes used to form components should be consistent. When evaluating the simplicity and continuity of a structure, designers should ask: Does the visual composition present a consistent design theme? Are any side profile joints covered with appropriate parapets providing a neat finish?

22.2.3 Aesthetic Design Factors

Principal aesthetic design factors are:

- Superstructure type and shape
- Vertical and horizontal geometry and their relationship to the surrounding environment
- Pier placement and shape
- Abutment placement and shape
- Interaction between the bridge and its surroundings/environment
    - Colours
    - Surface texture and ornamentation
    - Signing, lighting and landscaping
    - Material type

The designer must give careful consideration to the materials used and the form(s) of structures to ensure they are appropriate to and in sympathy with the surrounding environment and any adjacent structures.

22.2.4 Aesthetic Design Process

The level of aesthetic attention determines the participants in the aesthetic design process, the depth and to some extent the resources contributing to the aesthetic considerations. MRWA shall specify the Category for each project.

Three Categories of aesthetic design have been established for aesthetic requirements of bridges as follows:

Category A – A project of major aesthetic importance where aesthetics may be a significant factor in determining the structure type. Characteristics of structures in this category are highly visible bridges, bridge projects that generate significant public interest and/or bridges located in historic locations.

For this category of project, the designers and builders must incorporate public involvement in the process and achieve endorsement from a wide range of stakeholders. The public involvement may include, as appropriate but is not limited to Design Review Committee, public meetings, opinion surveys, and/or other public involvement.

The design process shall include a stage for aesthetic review and also the preparation of an aesthetic plan at the preliminary design stage. For Category A bridges it is mandatory to consider more than one aesthetic option for public involvement.

Category B – Bridges that require a moderate level of aesthetics, but not to the extent that it controls the superstructure design. Examples include bridges over freeways, main roads, roads in urban areas, bridges near recreational areas, parks, etc.

A common current example of this type of bridge would be one utilising teeroff beams as a grade separation in the Metro area.

For this category of project, a design process similar to that of Category A is required, although with a reduced emphasis on public involvement. The inclusion of stages for aesthetic review and the preparation of an aesthetic plan at the preliminary design stage remain unchanged. Architectural involvement is essential.

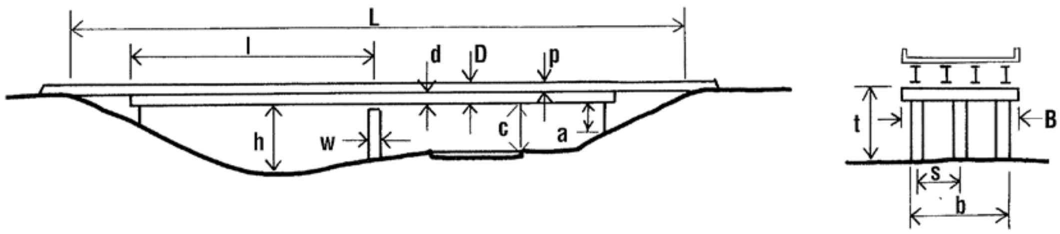
Category C – Bridges in locations where aesthetics is not as important as for Category B bridges. Examples include most rural road bridges, structures over minor creeks or railways subject to freight.

Notwithstanding the reduced aesthetic value of the lower visibility bridges, the basic precepts of sound aesthetic design shall be incorporated into the design as appropriate. Aesthetic review and an aesthetic plan are not required.

The category for aesthetic design will be set by MRWA in the design brief template and/or contract documents.

22.2.5 Aesthetic Guideline Abbreviations

Throughout this section specific letter abbreviations will be used to denote physical aspects of bridge components.



Guideline Element Abbreviations:

a – dist. bottom of girder to bottom of abut. face
B – pier length at cap or top
c – vertical clearance at pavement edge
D – total superstructure depth
h – vertical clearance to the ground
L – total bridge length
p – rail ht., top rail/bot. of deck

t – pier height
b – pier length at base
d – girder depth
l – span length
s – spacing of columns
w – pier width

Figure 22.1 - GUIDELINE COMPONENT ABBREVIATIONS

22.3 SUPERSTRUCTURE DESIGN GUIDELINES

22.3.1 Structure Layout

The designer should strive to develop a layout that embraces the fundamental aesthetic objectives outlined in the previous section. The designer should be considering scale and proportion along with order and balance. Examples of this include the abutment heights, the vertical and horizontal geometry, the location of piers and the predominant vantage point from which the bridge will be seen.

Within limitations the structure layout can be adapted by the designer to accommodate aesthetic and other considerations. This could include adding or removing a pier, moving abutments, changing the number of beam lines, using different materials, modifying the depth of structure etc. Bridge designers should make a conscious effort to control the structure layout to the benefit of the visual appeal of the structure as well as other aspects of design. Note that MRWA will often specify minimum abutment setbacks or restrictions on pier locations to assist in producing this result, and for other reasons such as creating safer zones for errant vehicles.

For shorter spans, harmony of proportion depends upon the relation of the structural mass to the size and shape of the openings. When the structure is on a slope, such that one end of the bridge is higher than the other, the abutment heights should be proportional to the clearance of the roadway edge.

22.3.2 Structure Depth and Proportions

In general, the primary aesthetic goal is to achieve a slender superstructure while maintaining continuity and proportions. The aim generally is to make the structure appear as a thin horizontal ribbon running from abutment to abutment, resting lightly on the piers.

The structure depth can be controlled to some degree, through adjustments in the span length, material selection, superstructure shape and detail.

The appearance of single span bridges is sensitive to the proportioning between the bridge components. Fitting proportions are required between the (suspended) superstructure and the height and width of the opening. The slenderness ratio for single-span structures may vary widely. Depending on the slenderness and proportions, the structure may appear heavy and clumsy or conversely a very slender superstructure could be over-powered visually by large abutments that appear disproportionate in scale. For a single-span bridge, the designer should consider the relations between the opening beneath the bridge, the mass of the abutments and the slenderness of the structure’s depth. This can be illustrated as follows:

- At one extreme is a bridge with deep abutments and a short span length and hence the opening beneath the bridge will approximate a square. In this instance the abutments provide a large mass and the proportions have more impact than does the slenderness. A lower slenderness ratio is appropriate for this situation. (Refer Figure 22.2, comparing the top two sketches.)
- At the other extreme is a bridge with shallow abutments and a long span length. The opening beneath the bridge will form a flat rectangle. As the span length begins to exceed the height, slenderness begins to become more significant. In this instance a higher slenderness ratio is appropriate.

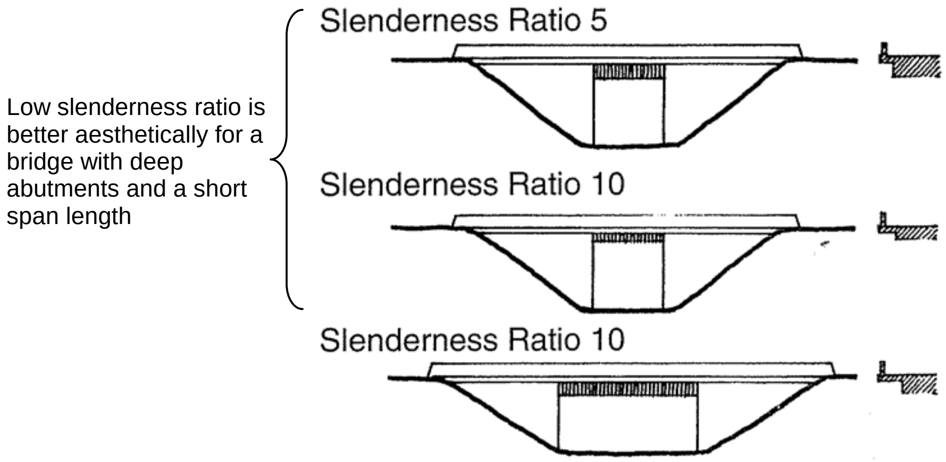


Figure 22.2 - NEED FOR PROPORTION CONTROL OVER NEED FOR SLENDERNESS ON SHORT SPANS

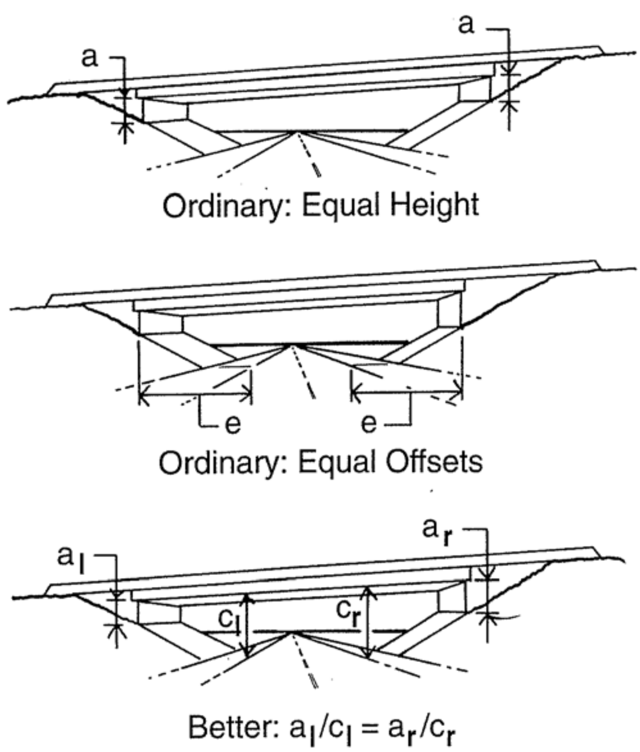


Figure 22.3 - SUBSTRUCTURE SHOULD BE PROPORTIONAL TO VERTICAL CLEARANCE

The total bridge length of two or more span bridges is typically much greater than the vertical clearance. Hence, the opening beneath the bridge is that of a flat rectangle. The principles of proportion are the same as for the single-span structure with flat rectangular openings. For a two span bridge with shallow abutments, the designer should strive for a very slender superstructure. For a two span bridge with deep abutments the proportion of the structure depth to the mass at each end of the bridge becomes more important and the designer should consider a slightly reduced slenderness ratio.

The pier should support the superstructure as unobtrusively as possible, allowing the horizontal lines of the girder and railing to dominate the view.

22.3.3 Girder Bridges

With the exception of Category C bridges, MRWA does not allow the use of small, multiple girders in bridge designs. Larger and fewer beams provide for a smoother and more attractive soffit line.

Along with economy, durability and other design aspects, aesthetic considerations shall be included in the design of girder bridges to achieve slender appearance.

22.3.4 Box Girder Bridges

Box girder bridges are inherently elegant because of their simplicity and structural efficiency. The form and shape of box girder bridges present clean structure lines, whether viewed in elevation or some other vantage point. The solid bottom soffit can provide excellent reflective properties that tend to naturally illuminate the area beneath the structure.

22.3.5 Other Superstructure Types

Further information on superstructure presentation, haunched girders and other bridge types is available from Reference 1 and other references.

22.4 SUBSTRUCTURE DESIGN GUIDELINES

22.4.1 Piers

Generally piers should not be the visual focal point of a bridge composition. The main visual emphasis of the formation should remain on the horizontal lines of the superstructure.

MRWA practice and preference is to minimise the column numbers and to maximise the visibility through the bridge.

The pier must look logical in relation to the superstructure shape, complementing its shape, slopes and general configuration. There must be consideration of architectural development and enhancement of the pier.

Piers with columns and integral collision walls as required for rail impact, shall only be used if the wall height is less than 30% of the total height of the pier (t in Figure 22.1).

There should be a consistency or compatibility between adjacent bridges and other bridges along the route.

The appearance of piers is primarily influenced by their proportion, e.g. their width relative to their height and the configuration of the pier cap with the pier column. Tall piers benefit from simplicity, fewer lines and slender proportions. Traditional short piers are more difficult to design from an aesthetic viewpoint because the pier cap is often large and visually clumsy in relation to the total pier. This is particularly the case for common Category B bridge types such as teeroff beam designs where the same architectural solutions keep recurring. The designer must deal with the challenge of not repeating the same design everywhere, but at the same time should not be jarringly different and needs to be economically practical while managing the practical constraints of the teeroffs.

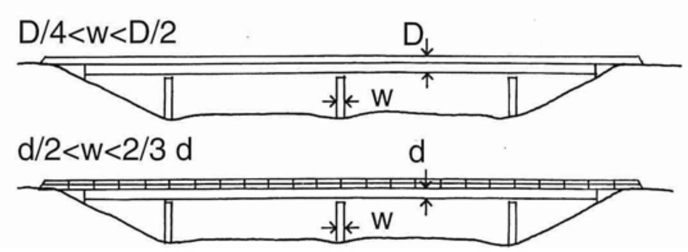


Figure 22.4 - PIER PROPORTIONS

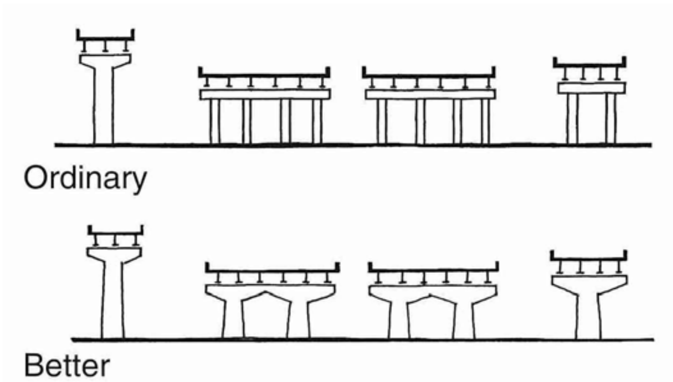


Figure 22.5 - FAMILY OF PIERS THAT VARY BY HEIGHT

22.4.1.1 Pier Columns

The width of columns perceived by the viewer is normally influenced by light reflecting from the column surfaces and edges. A square or rectangular column with strongly bevelled edges will appear more slender than a circular column due to the edge lines and varying shades of reflective light. An octagonal column will appear even slimmer because of the greater number of surfaces. The designer can use this principle of light reflection to ‘slim down’ a massive column or increase the apparent size of a column to offset a massive superstructure.

Columns do not always have to be vertical shafts of constant section. Square and round columns are easy to form, but they lack imagination. The use of other shapes can add variety and interest.

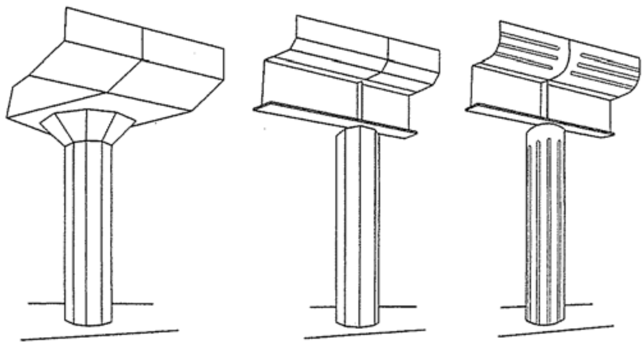


Figure 22.6 - BEVELLED EDGES AND SURFACE TREATMENT MAKE COLUMNS APPEAR THINNER

22.4.1.2 Pier Caps

The pier cap can pose an aesthetic problem to the designer. When viewed from a position approaching the bridge, the pier cap is clearly separate and distinct from the horizontal lines of the superstructure, but it does not quite relate to the predominantly vertical lines of the columns. The more dissimilar the shape of the pier cap to the other components of the bridge, the more of a distraction or disruption it becomes.

The end of a pier cap further aggravates the visual disorder. Pier cap ends are often relatively large, flat surfaces extended to the same plane as the deck fascia. As the ends of the pier cap protrude from beneath the structure, they typically reflect the full intensity of whatever light is available, thereby creating a visual 'hot spot' to the detriment of the horizontal flow of the structure.

The most effective method is to eliminate the pier cap entirely, or eliminate the cantilevered ends of the pier cap. If this is not possible or practicable in a particular situation, designers shall attempt to diminish the prominence of the pier cap and pier cap ends.

Pier caps can be eliminated or minimised by:

- Use of inverted T-shaped pier caps (refer Figure 22.7)
- Eliminating the cantilevered portion of the pier cap
- Continuity of the superstructure soffit over the pier (refer Figure 22.8)
- Incorporation of the pier cap into the superstructure (integral pier cap)
- Incorporation of the pier cap into the columns (integrated pier cap)

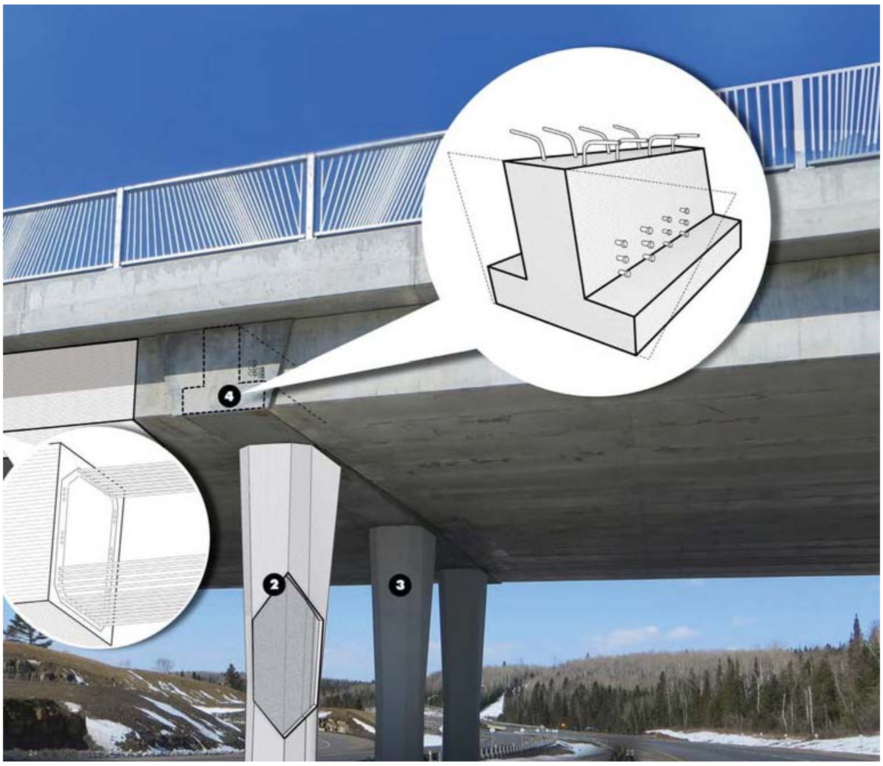


Figure 22.7 - T-SHAPED PIER CAP EXAMPLE

Figure 22.8 – TEEROFF BEAMS CONTINUING OVER THE PIER

The end of the pier cap can be minimised by:

- Reducing its mass and reflective surface
- Bevelling or tapering the surfaces of the pier cap end (refer Figures 22.9 to 22.11)
- Minimising the height of the pier cap (refer Figure 22.12)

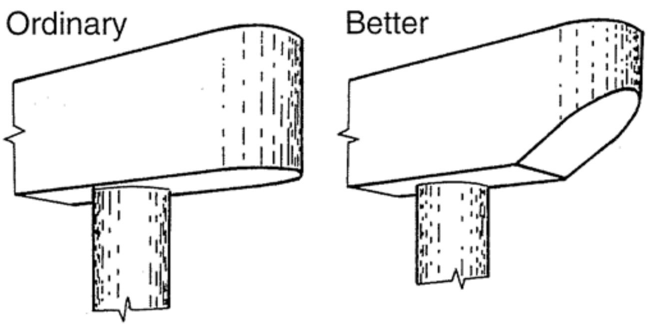


Figure 22.9 - ROUNDING THE PIER CAP END REDUCES ITS PROMINENCE

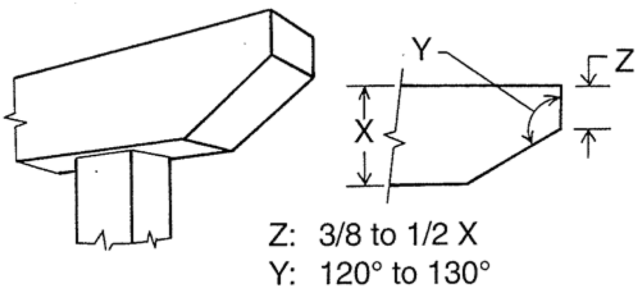


Figure 22.10 - BEVELLED PIER CAP END GUIDELINES

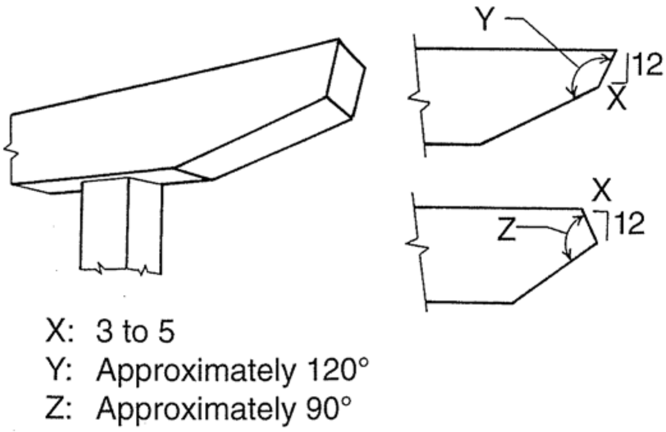


Figure 22.11 - DOUBLE BEVELLED PIER CAP END GUIDELINES

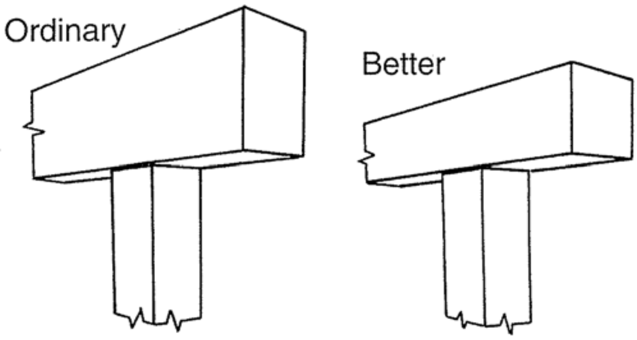


Figure 22.12 - REDUCING PIER CAP HEIGHT DIMINISHES VISUAL 'HOT SPOT'

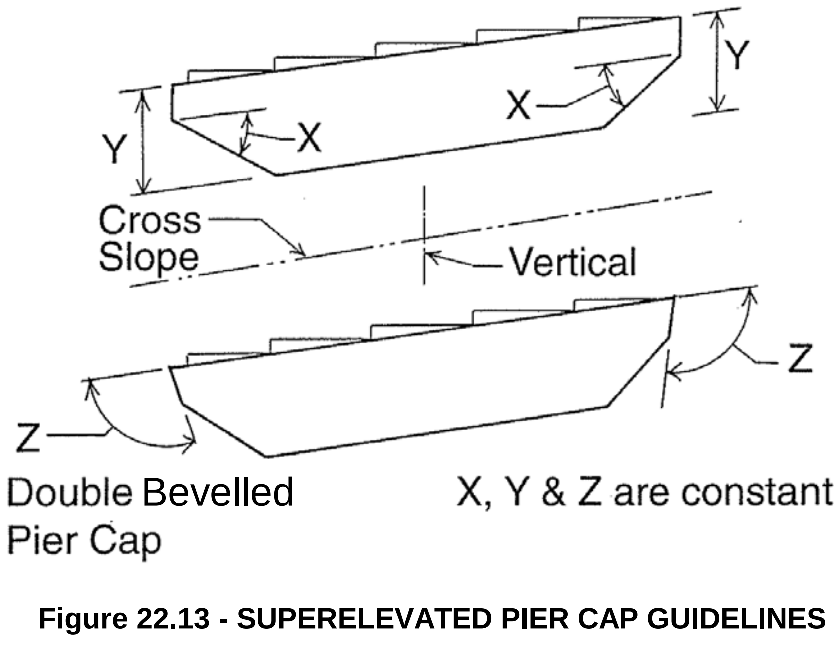


Figure 22.13 - SUPERELEVATED PIER CAP GUIDELINES

22.4.1.3 Short Piers

Short piers are considered to be those with base lengths that exceed their height (B > t), refer Figure 22.1 for abbreviations and Figure 22.14 below for a short pier example. Short piers can be constructed in several shapes but most commonly hammerhead piers and the traditional multi-column bents.

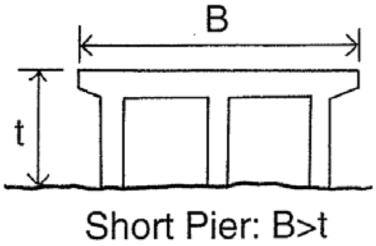


Figure 22.14 - SHORT PIER

A design issue that is common among all types of short piers involves the geometrics of the column shafts and pier caps that are used to construct the pier. The geometry of these individual components should be selected from the same shape 'family', circular, rectangular etc. That is, rounded pier cap ends and circular columns would appear more visually correct than square pier cap ends and circular columns. Ideally, a consistent shape will be carried to the abutments, railings and other components of the bridge.

a) Multi-Column Bents

Traditional multi-column bent piers need careful aesthetic consideration so that the visual effect is not that of a 'forest of columns'.

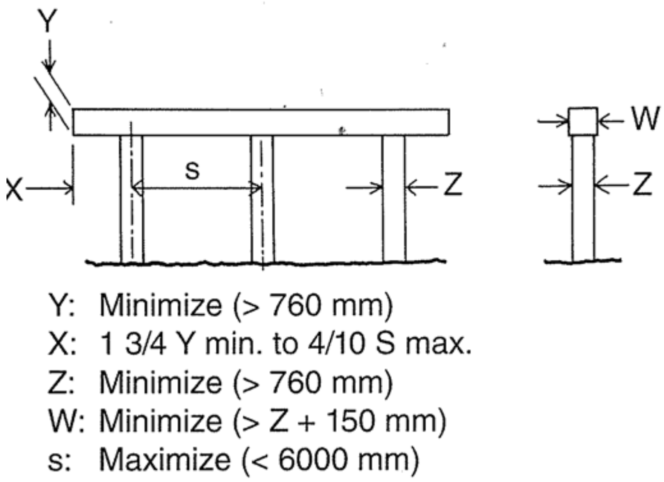


Figure 22.15 - MULTI-COLUMN BENT WITH CANTILEVERED PIER CAP GUIDELINES

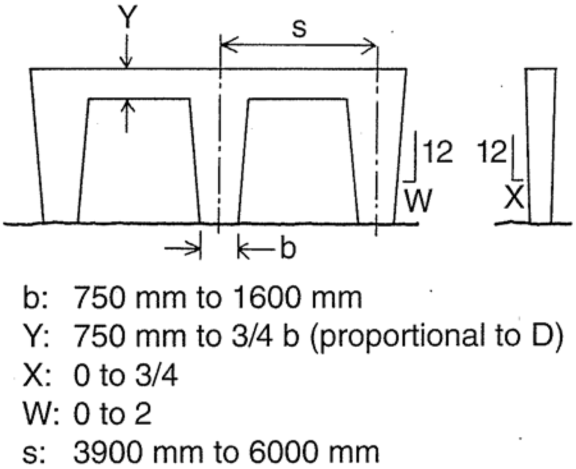


Figure 22.16 - MULTI-COLUMN BENT WITH INTEGRATED PIER CAP GUIDELINES

b) Hammerhead or T-Piers

The appearance of a hammerhead pier is sensitive to its relative proportions.

Considerations for hammerhead piers should include the following:

- Hammerhead piers in series should be consistent in appearance, e.g. same size, shape, proportion and details.
- Hammerhead pier shafts should not be shorter than the maximum capbeam depth plus 2 metres.
- Short hammerhead piers may use either vertical or battered sides, depending on the desired aesthetic effect.

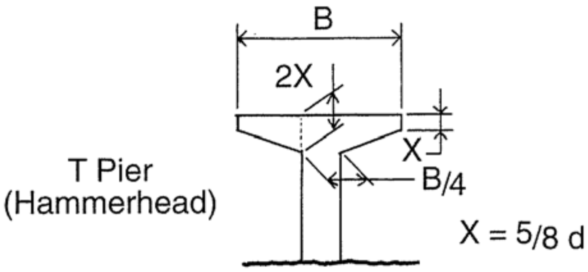


Figure 22.17 - HAMMERHEAD PIER PROPORTION GUIDELINES

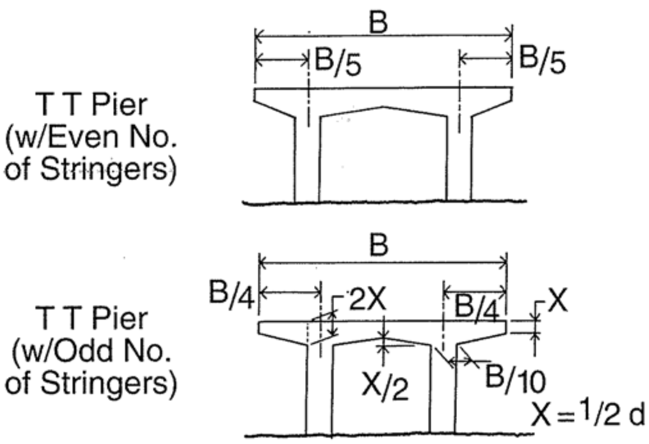


Figure 22.18 - DOUBLE HAMMERHEAD PIER PROPORTIONS

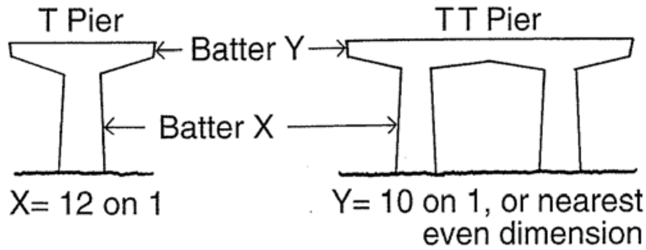


Figure 22.19 - BATTERED HAMMERHEAD GUIDELINES

22.4.1.4 Tall Piers

Tall piers are those piers whose height exceeds their base width (B < t, refer Figure 2.19). Straight pier shafts are appropriate for most tall piers. Tall piers can be constructed in several shapes but most commonly V-shaped and Y-shaped piers.

When pier heights begin to approach the span length, then a small taper can maintain the structural capacity at the base and preserve the slenderness of the pier. This is seldom applicable in WA and therefore inherently less of a problem.

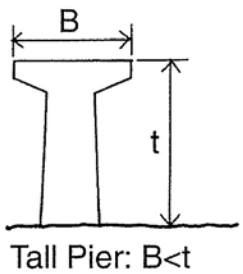


Figure 22.20 - TALL PIER

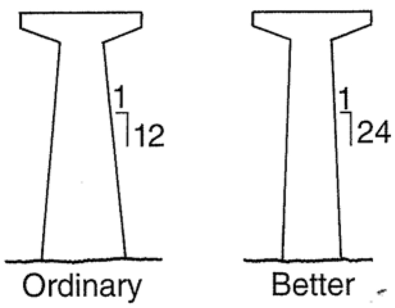


Figure 22.21 - TAPERING TALL PIER

a) V-Shaped Piers

Columns and pier caps can be eliminated entirely by using a wall pier that has a narrower lateral width at the base than at the top. While V-shaped piers eliminate the pier cap they create other visual problems effectively blocking the observer's sight when viewed from an oblique angle.

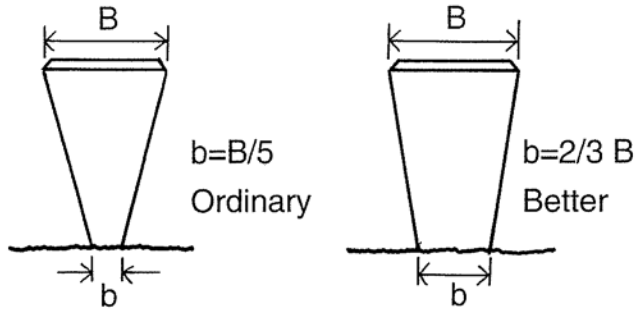


Figure 22.22 - BASE LENGTH VERSUS PIER LENGTH

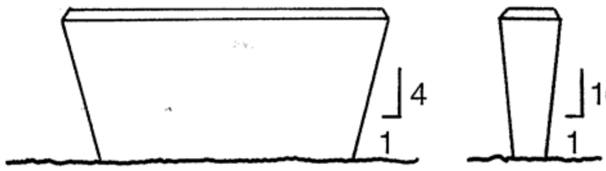
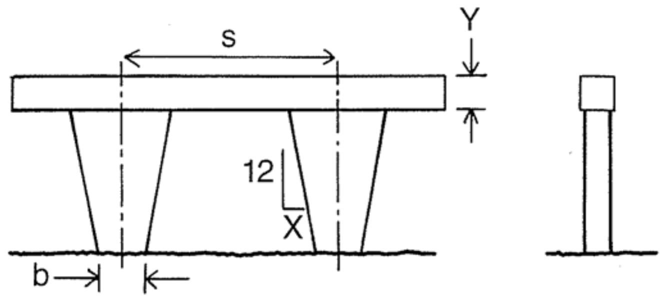


Figure 22.23 - SUGGESTED BATTER OF V-SHAPED PIERS



b: 1300 mm to 1900 mm
Y: 1/2 b to 3/4 b (proportional to D)
X: 2 1/2 to 3 1/2
s: 4800 mm to 6700 mm

Figure 22.24 - V-SHAPED COLUMNS WITH CANTILEVERED PIER CAP GUIDELINES

22.4.2 Abutments

The shorter the bridge, the more influence the abutment plays in creation of the overall visual image. As such, the proportions of the abutment are crucial to the mass, scale and proportion of short to medium-span bridges. The aesthetic objective for these bridges is to provide good proportion between the mass of the abutment and superstructure and to provide balance in the structure.

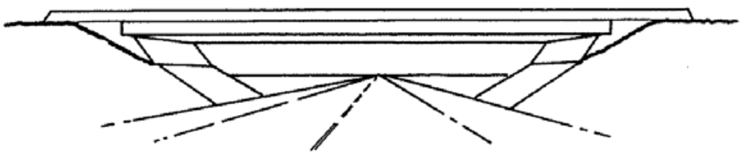


Figure 22.25 - PREDOMINANT ABUTMENT LINES CONTRAST HORIZONTAL FLOW

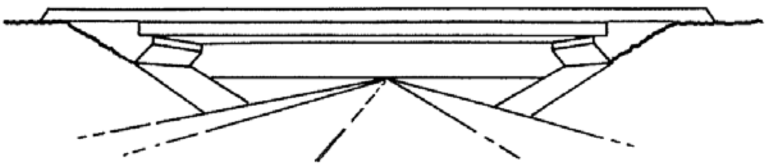


Figure 22.26 - PREDOMINANT ABUTMENT LINES COMPLEMENT HORIZONTAL FLOW

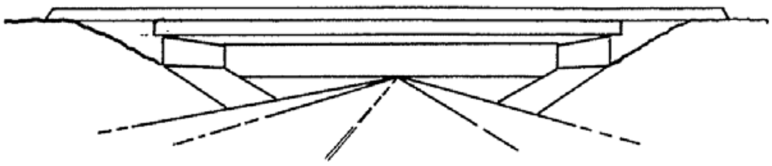


Figure 22.27 - VERTICAL ABUTMENT FACE PRESENTS A STATIC VISUAL IMAGE

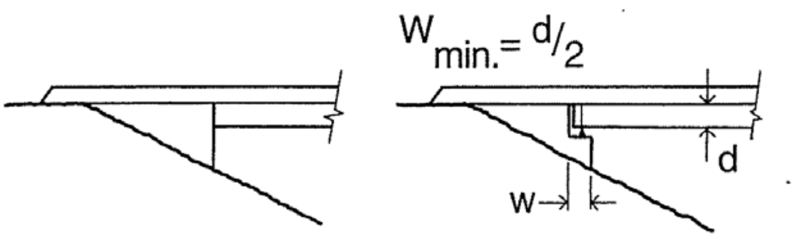


Figure 22.28 - MASK WALL/EXPOSED BEAM SEATS

Most abutments may be categorised as one of three types, shallow, semi-deep and deep, based on the visual height of the abutment. Each type has its own visual bias and the designer must create the proportional relationships to achieve the best visual balance. Refer to Reference 1 for guidance on proportions and balance.

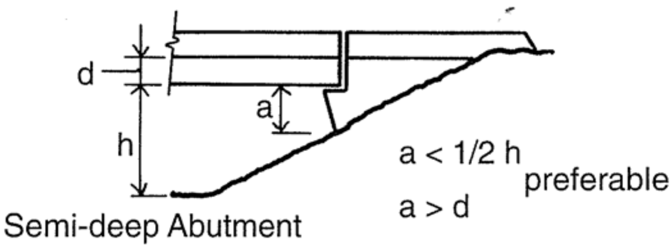


Figure 22.29 - SEMI-DEEP ABUTMENTS

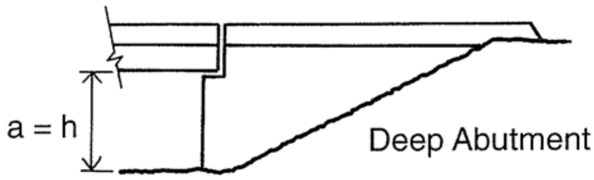


Figure 22.30 - DEEP ABUTMENTS

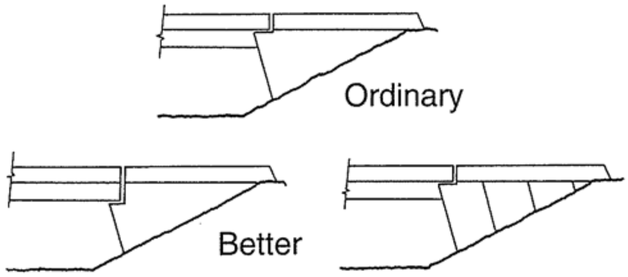


Figure 22.31 - METHODS OF REDUCING THE APPARENT HEIGHT OF AN ABUTMENT

Wing walls for abutments for road bridges over other roads in Category A or B locations are aesthetically better when curved in plan and profiled in elevation.

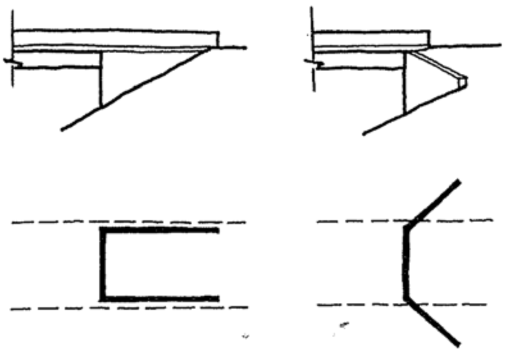


Figure 22.32 – PARALLEL AND ANGLED WING WALLS

If MSE abutments are used they should generally be constructed using full height panels. A standard off-form precast capping should be provided for bridges greater than 6 m clearance. For shorter abutments, a recessed groove is permitted in lieu of the capping. Elsewhere, horizontal feature lines in the front face should be avoided.

22.5 OTHER FACTORS FOR CONSIDERATION

The determination of the principal aesthetic design factors does not preclude the designers' opportunity to affect the overall aesthetic appearance.

There are a number of other factors that are linked to aesthetic decisions. They include:

- Colours: the colours of the uncoated structural materials as well as the coated components and the details.
- Surface texture and ornamentation: techniques that can add interest and emphasis.
- Railing, signing, lighting and landscaping.

22.5.1 Railings

The vehicle and pedestrian barriers and deck fascia are often the most visually prominent parts of the bridge.

Typically the designer will be constrained in part by the requirement to incorporate a barrier that will provide the nominated performance criteria for retention and redirection of errant vehicles. Within that limitation, MRWA's preference is to use metal post and railing barrier to allow travellers to see scenic areas and to provide an open appearance.

The faces of the rail system are prominent features of the structural presentation and may be discoloured or stained through corrosion or other factors. Designers should develop details that are less prone to corrosion and other stains.

22.5.2 Shared Path Bridges

Shared path bridges are generally lighter in structural form given the smaller loads compared with vehicular bridges. There is also a bigger opportunity to make them an aesthetic feature.

Superstructures must be either haunched in a smooth curve, symmetrical about midspan or of a constant depth.

Substructures should consist of no more than one column per pier.

22.5.3 Shared Path Underpasses

All underpasses must be single span structures with a continuous flat or curved soffit. Care must be given to providing light and openness in long (> 40 m) underpasses with additional clear width and height.

22.5.4 Railway Protection Screens and Screen Walls for Bridges

Railway protection screens and screen walls for bridges shall be curved structures comprising 'Expamet' JE1116 material for the full height. All steelwork must be hot dip galvanised and painted. The attachment of the screen to the deck must be concealed from elevation view.

22.5.5 Attachment of Services

Services, ducting and drainage pipes must be concealed from public view. Services are not permitted on shared path bridges.

22.6 VARIOUS REQUIRED OUTCOMES

The structure will present smooth, clean continuous lines, and have a minimum structural depth consistent with its spans and method of construction. This means that each bridge will have a consistent form and depth between adjacent spans and across the width of the pier.

Road bridges over the Highway will have similar appearances and there must be consistency in the design of their major components. Whilst acknowledging there will be special circumstances such as ramp bridges, adjacent bridges must be identical in structural form, span lengths and skew angle, and abutment and pier centrelines must align.

Future widening considerations for the ultimate stage need to be considered at the first stage of design and construction. It must be possible to widen the bridge and maintain the aesthetic form of the bridge.

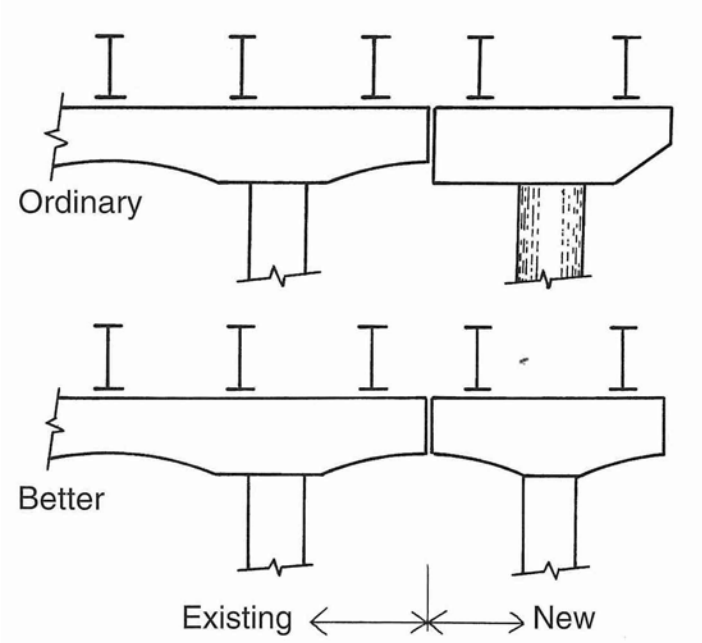


Figure 22.33 - MATCH NEW CONSTRUCTION CHARACTERISTICS WHEN WIDENING BRIDGES

APPENDIX A – AESTHETIC DESIGN RULES

All the Aesthetic Design Rules apply to Category B structures. A number of the Aesthetic Design Rules also apply to Category C structures where identified by a ‘C’ in the right hand side column.

These aesthetic design rules should also be used as a starting point for Category A structures but public involvement and architectural sign off will ultimately help to form the final aesthetic solution.

Any deviations to these rules for Category A and B structures need to be approved by MRWA’s project manager.

A.1 Superstructure

Except where otherwise stated the following must also apply for shared path bridges and underpasses:

i	Vertical profiles of adjacent structures must be such that profile variations are minimised.	
ii	Road bridges over the highway must have no more than two spans.	
iii	For multiple span structures, the length of a span must not be less than 70% of the length of either adjacent span.	C
iv	The superstructure depth must be constant for the full length of the bridge, except for haunched bridges in special structures.	C
v	All bridges must be of concrete or steel or a composite of these materials. External cladding with materials other than concrete or steel will not be permitted. The material composition (i.e. concrete or steel or concrete/steel composite) and structural form must be the same for the full length of the deck.	C
vi	Steel beams must be of box girder construction with a closed soffit.	
vii	Visible faces of steel beam surfaces must have a smooth finish at all locations including at spliced connections.	
viii	The finish to superstructure concrete surfaces, except for deck edge parapets, must be smooth off-form concrete.	C
ix	Faces of the deck or fascia panels visible below the deck slab cantilever must extend from the underside of the cantilever to the soffit in a single vertical or downward facing plane constructed of homogenous material without horizontal lines visible in elevation. Deck slab cantilevers must extend a minimum of 1.0 m beyond the top of this plane.	
x	Deck edge parapets must be precast concrete with galvanised steel fittings and must have an exposed aggregate finish on all surfaces visible in elevation view. The exposed aggregate finish must result in a texture depth of no more than 4 mm.	
xi	The parapet must extend a minimum of 50 mm below the edge of deck slab cantilever soffit. Concrete traffic barriers, where used, must incorporate a vertical component that extends 50 mm below the edge of the deck slab cantilever soffit.	
xii	Deck edge precast concrete parapets must continue off the deck along the full length of approach retaining walls.	
xiii	Recessed drip grooves must be included in the soffit and near the edge of deck slab cantilevers, or near the edge of flat slab type decks.	C
xiv	Cross girders/external diaphragms may only be located at the ends of each span and not within any span, and must not extend beyond the outermost beam web face into the edge cantilevers. At piers, the depth of cross girders/external diaphragms must not exceed one half of the depth of the longitudinal beams.	
xv	If beams are used for the superstructure, their number per span must be constant and they must be placed “head-to-toe” at each line of internal support.	C

xvi	Drop-in type beams that are not directly supported by piers or abutments will not be permitted.	C
xvii	For a bridge less than 30 m long with a mid-ordinate of D ≤ 50 mm, the horizontal alignment of the bridge may be straight, i.e. the chord of the arc between the ends of the deck. (Refer Figure A.1).	C
xviii	For a bridge of length ≥ 30 m with a horizontal mid-ordinate of D ≤ 75 mm, the horizontal alignment of the bridge may be straight, (e.g. a bridge with a 30 m long base centreline on a 1500 m radius has D = 75 mm, therefore the bridge may be straight.) No increase in road width is necessary in such cases other than due to road design requirements.	C
xix	For spans up to and including 12 m, where the mid-ordinate of an individual span is 10 mm or less and the angular deviation from one span to the next is less than one degree (1°), it will be acceptable for individual spans to be straight. (Refer Figure A.2).	C
xx	In all other cases the horizontal alignment of the deck slab must be set out on the curve or in chords of such length that the mid-ordinate between the chord and the curve does not exceed D = 5 mm. The maximum length of this chord can be found with the formula: $C = (40R)^{0.5}$ where C is the chord length in mm and R is the horizontal radius of curvature of the road centreline in mm.	C

The mid-ordinate of the bridge (D) can be found with the formula:

$$D = C^2 / 8R$$

where C is the chord length of the whole bridge and R is the horizontal radius of curvature of the road centreline, as illustrated in Figure A.1.

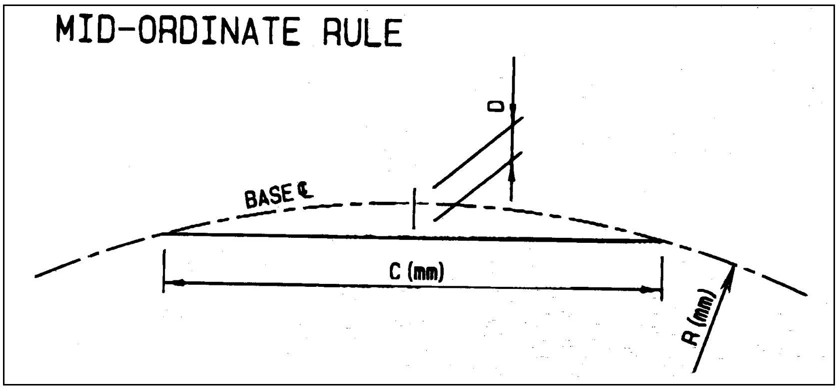


Figure A.1 - MID-ORDINATE

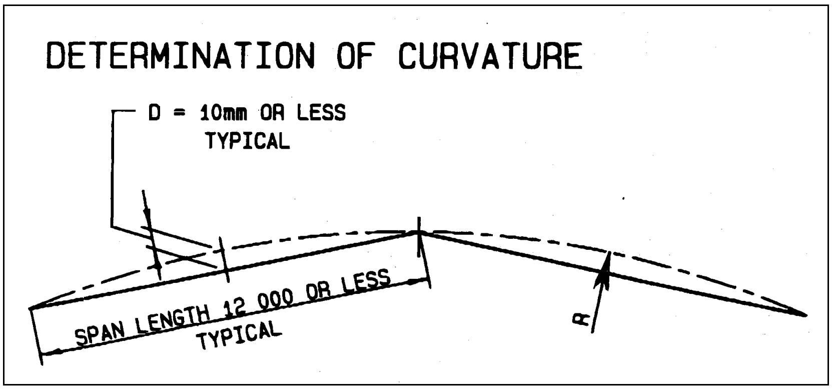


Figure A.2 - DETERMINATION OF HORIZONTAL CURVATURE

A.2 Piers

Except where otherwise stated the following must also apply for shared path bridges and underpasses:

i	Piers must be designed to maximise the visual openness between columns.	C
ii	Piers must be of concrete construction.	
iii	All discrete beams must be directly supported by columns and for each bridge the columns must support the same number of beams.	C
iv	All piers of each bridge must contain the same number of columns and each pier must appear similar.	C
v	The edges of the column must be straight from the top of the column to the base of the column.	C
vi	Each column of a pier must have the same dimensions at the top and the same angle of inclination of each face.	C
vii	The footings and pilecaps of pier columns must be buried a minimum of 300 mm below the finished ground level.	
viii	Columns which divide into separate upright components may comprise no more than two such components.	C
ix	Crash resistant walls for bridges over railways must not be independent of the pier.	
x	Where possible, piers in the median, including those of existing bridges, must be located centrally between ultimate carriageways.	

A.3 Abutments

Except where otherwise stated the following must also apply for shared path bridges and underpasses:

i	Abutments must be concrete. Limestone or reconstructed stone panels will not be permitted.	C
ii	MSE abutments will not be permitted on structures over a water course.	C
iii	If MSE abutments are used they shall be constructed using full height panels. A standard off-form precast capping should be provided for bridges greater than 6 m clearance. For shorter abutments, a recessed groove is permitted in lieu of the capping. Elsewhere, horizontal feature lines in the front face should be avoided.	
iv	Wing walls for abutments for road bridges over other roads must be curved in plan and profile. The exposed surface of the curved sections must not be formed by a series of chords.	
v	If precast prestressed concrete trough beams (e.g. teeroffs) are used for the superstructure, the exposed front face of abutments must not extend above soffit level and the profile of the top must be linear except for changes in structure crossfall and at junctions with wing walls.	C
vi	Unless precluded by the location of paths or other constraints, spill-through slopes must be provided at the foot of abutment walls facing the roadway to reduce the overall visual height of abutment walls.	
vii	The footings and pilecaps of abutment columns must be buried a minimum of 300 mm below the finished ground level.	

A.4 Shared Path Bridges

Additional requirements that are specific to shared path bridges include:

i	Abrupt angle changes in plan will not be permitted. No section of alignment may have a radius less than 4.0 m.	
ii	Loop type ramps must have in plan circular, elliptical helix or compound circular curves which approximate an ellipse.	
iii	Superstructures must be either haunched type or of a constant depth. Through, trough or truss girder type shared path bridges will not be permitted.	C
iv	The variation in deck depth must be achieved by varying the profile of the deck soffit in a smooth curve, symmetrical about midspan for the main spans.	C
v	The deck haunch must be an identical mirrored profile on either side of the main span pier supports.	C
vi	The deck soffit must have a radius of curvature of not less than 50 m, except at piers where abrupt angles will be permitted; and the deck depth must be equal on each side of a pier.	C
vii	For the constant depth profiled type structure the minimum deck span to depth ratio must be 40.	
viii	The deck cross-section of shared path bridges must have a closed form shape, that is shapes such as “I” sections, channels and other shapes featuring upward inclined surfaces on the underside of the deck will not be permitted.	C
ix	If cantilevers are included in the cross-section they must have a minimum length of 300 mm.	C
x	Where landings are provided, the shared path must be profiled independently from the other components of the deck, and must be hidden from elevation view by deck edge upstands.	
xi	The deck soffit must be smooth in profile and must not reflect landings/ramps.	C
xii	Cross girders/external diaphragms are not permitted on shared path bridges.	
xiii	Arch shared path bridges with a suspended deck will only be permitted following architectural advice and approval by MRWA.	
xiv	Piers must comprise no more than one column per pier.	
xv	MSE abutments will not be permitted.	

A.5 Shared Path Underpasses

Additional requirements that are specific to shared path underpasses include:

i	All underpasses must be single span concrete structures. External cladding with materials other than concrete will not be permitted.	
ii	The deck cross-section of shared path underpasses must have a continuous flat or curved soffit, that is, discrete beams will not be permitted.	
iii	Cross girders/external diaphragms are not permitted.	
iv	MSE abutments are not permitted.	C
v	Underpasses which are more than 40 m long or have an invert level at one or both entrances lower than 1.0 m below existing surface must have the following internal dimensions (measured perpendicular to the shared path): • A clear width at the base of the underpass of not less than 4.0 m; • A height of not less than 2.5 m; • A width at the widest part of not less than 7 m; and • A cross sectional area of the underpass of not less than 17 m^2.	C
vi	All other underpasses must have a clear width of not less than 4.0 m to a height of 2.5 m for the full width.	C
vii	The spill through slopes must be no steeper than 1.5 (horizontal) to 1 (vertical).	

viii	Where there is a median, each underpass must include a trafficable skylight in the median. The skylight width must be the same width as the underpass width and the total length must be at least two-thirds of the ultimate residual median width. The top of the skylight must be flush with the median.	C