Part One
Background

Queensland
Government
Department of
Main Roads

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1.0 Introduction

1.1 General

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A "Timber Bridge Strategy" is being pursued by BAM to examine all matters associated with timber bridge management and utilisation. Activities being undertaken include bridge and component testing, validation of analysis methods, development of alternative materials etc. An important component of this strategy is the release of the timber Bridge Maintenance Manual.

The purpose of this document is to give guidance on all matters associated with the maintenance of bridges containing timber components. Additionally, it is intended to provide a record of maintenance practices that have evolved over some 80 years of timber bridge use by Main Roads, Queensland.

With approximately 500 timber bridges remaining in service on DMR controlled roads, there will be a requirement to manage such structures for many years into the future. It is considered essential that the knowledge of experienced timber personnel be written down while still available, so it will be available to future generations of managers.

The Timber Bridge Maintenance Manual is being released as a stand-alone document, though it will be essentially a component of the Main Roads Bridge Maintenance Manual which has yet to be developed.

A standard specification for the supply of timber bridge materials, MRS11.87 Supply of Timber Bridge Materials and Components has also been released in order to control the supply of hardwood timber, steel trough decking, plywood decking and PSC decking.

The Timber Bridge Maintenance Manual is intended to be used in conjunction with the Bridge Inspection Manual and is cross-referenced to it for component designations, component deterioration mechanisms, defect types etc.

A "Timber Bridges" Manual was developed by Tony Platz in the 1980's, and this was distributed on a semi-official basis through regular Bridge Construction courses. This document has been the only published information generally available for guidance in maintaining Main Roads' timber bridges.

It is the intention that this Manual will give a more comprehensive coverage of all aspects of timber maintenance and structure management.

The various observations, recommendations and requirements made in relation to timber maintenance have come from a number of sources - structural analysis knowledge and site observations, and, in particular, information provided by experienced timber bridge practitioners. In addition, information has been sourced form Gympie District RMPC documents, the Vic Roads Maintenance Manual, and the draft RTA Timber Bridge Manual.

As a guide to understanding the behaviour of timber structures, an attempt has been made to describe the function of the various components and significance of failure of these members.

Maintenance activities have also been defined as "Routine Maintenance" or "Programmed Maintenance" to better describe the type of operation involved.

Maintenance Activity Numbers corresponding to the various works and operations required are given for use. Note that, these numbers do not generally correspond to the limited numbers of Activity Numbers listed in RMPC documents.

Recommended repair procedures for the various types and levels of deterioration are provided for the normal components associated with timber bridges. Warnings have also been included where certain repair procedures are deemed to be inappropriate.

A section on design methods for timber components is included in this Manual, in order to provide a design basis for replacement or additional members. This is considered necessary to enable consistent design to be carried out, as there is no timber design information in the current AUSTROADS Bridge Design Code.

1.2 Other Documents

The following lists other publications referred to or to be used in conjunction with this Manual.

AS1720.1 (1997) - Timber Structures, Part 1: Design Methods	[SA]
Bridge Design Code (1992)	[AUSTROADS]
Bridge Design Specification (1976)	[NAASRA]
Bridge Inspection Manual	[DMR]
MRS 11.87 - Supply of Timber Bridge Materials & Components	[DMR]

In addition, acknowledgement is made of the use of the following documents in the preparation of the Timber Bridge Maintenance Manual.

Draft RTA Timber Bridge Manual	[RTA, NSW]
Gympie District RMPC Supplementary Conditions & Specifications	[DMR]
Timber Bridges Manual - A. Platz	[DMR]
Vic Roads Maintenance Manual	[Vic Roads, VIC]

1.3 Abbreviations

A list of abbreviations used in this Manual is as follows:-

AUSTROADS-	Association of State, Territory & Federal Road & Traffic Authorities of Australia.	
ARRB-	Australian Road Research Board	
BAM -	Bridge Asset Management	[DMR]

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BAMS	Bridge Asset Management System	[DMR]
BIM -	Bridge Inspection Manual	[DMR]
BIS -	Bridge Information System	[DMR]
DMR -	Department of Main Roads, Queensland	

Also may be listed on various old records as Main Roads Board, Main Roads Commission or Department of Transport. During World War II, it also functioned as part of the Allied Works Council.

NAASRA -	National Association of Australian State Road Authorities	
PSC -	Prestressed Concrete	
RMPC -	Road Maintenance Performance Contract	[DMR]
RTA -	Roads & Traffic Authority, New South Wales	
SA -	Standards Australia	
TBMM-	Timber Bridge Maintenance Manual	[DMR]
Vic Roads -	State Road Authority, Victoria	

1.4 Use Of Manual

Figures 1.4(a) and 1.4(b) diagrammatically illustrate the intended process when using this Manual.

Once component related defects in timber bridges have been identified to the requirements of the BIM, the document process for maintenance repair using the TBMM requires reference to:-

Part 3 This Part lists the various component defects (generally only those defects resulting in Condition States 3 & 4) as identified by the BIM for the various bridge components.

For each defect a recommended (or alternative) repair activity is nominated.

- Identify applicable defect, and select repair activity.

Part 4 For each repair activity, this Part describes the activity, lists units, coverage, intervention levels and hold or approval points.

- Select Activity item for preparation of Bill of Quantities and Estimate.Complete Standard Form M1 (see Figure 1.5) for BIS entry.Note that for Mobile BIS, entry is direct.Further information on a particular component including a more detailed discussion of maintenance for major timber components is given in Part 2.

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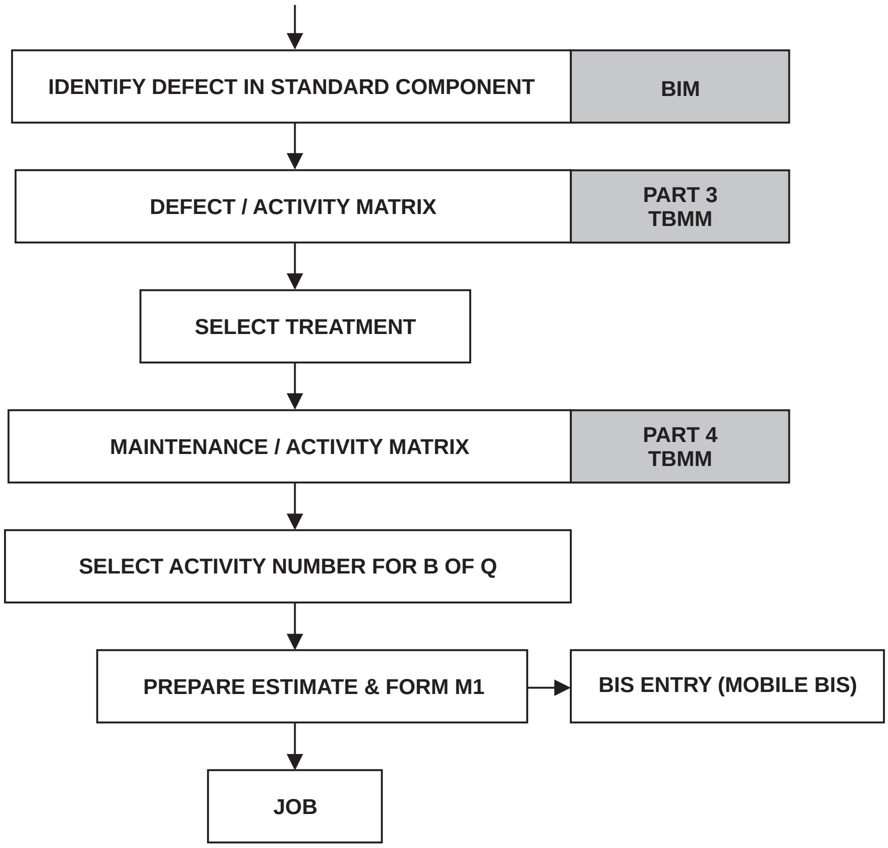

Figure 1.4(a) - Document Process

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Example:

A Level 2 or 3 inspections has determined a timber girder to be in Condition State 4, with a major rot hole which is significantly affecting strength.

Severe rot hole in girder side

Referring to Part 3, Section 2.2 Timber Girders, for "severe surface decay and holes", the recommended activity is 22T2 - replace timber girder.

Note that, at this time, the option of replacing in alternative materials would only be considered if full superstructure replacement was an option.

Referring to Part 4, Section 2.7, Item 22T2, a general summary of work operations, approvals, requirements and recommendations is given.

For further detailed discussions, refer to Part 2, Section 8.1.

Prepare the maintenance job as normal, complete Form M1 and enter data into BIS.

Figure 1.4(b) - Example

1.5 Maintenance Records

All proposed maintenance activities on bridge components, including estimates of cost, shall be entered into BIS. Form M1, shown in Figure 1.5 is to be used for this purpose. Appendix E provides guidance on maintenance activities relevant to each standard inspection component.

Recording of all maintenance activities in BIS will allow costs and effectiveness of various repairs to be determined across the State or Regions.

Structure Maintenance Schedule — **M1** — Sheet 1 Of

Structure ID.. Bridge Name ....
Crossing .... Road Number ....
Structure Type.. Owner..
Construction Type.. District ....
Construction Material.. Local Authority ....
Inspector .... Overall Condition Rating ....

Inspection Level 1 ☐ Level 2 ☐ Level 3 ☐ Underwater ☐

Date of Inspection.. Date of Next Inspection ....
Chainage…………(km) on the ……………………………………to……………………………………Road

Defect Location and Details (from B2 forms)

Component Location (Modification/Group/Component/Standard Number) ____/____/____/____
Component Description…………………………….. Significance 1☐ 2☐ 3☐ 4☐
Defect Details Condition State 1☐ 2☐ 3☐ 4☐

Maintenance Activity Schedule

Activity No.	Description	Unit	Quantity	Unit Rate	Amount	Priority	Completed

Sub-total $................

Inspector's Comments

Steward's Comments

Total Maintenance Backlog Amount for Structure $ ....

Figure 1.5 - Standard Form M1

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2.0 Maintenance

2.1 Overall Requirements

Though timber bridges have their own special requirements due to the nature of the material used, they must be treated as part of the overall maintenance requirement for all bridge structures. These requirements are laid out in the following sections.

2.1.1 Bridge Maintenance Strategy

All structures are to be maintained in a safe and serviceable condition at all times (in accordance with the original design specifications) which includes the following functional requirements:

- Load capacity
- Height clearance
- Width
- Suitability of safety barriers
- Alignment
- Flood immunity

Short term load or other functional restrictions may be implemented pending the implementation of approved remedial measures where the safety of the public is compromised and no other alternative crossing exists.

The development of bridge management strategies is predicted on:

- Road user safety;
- Strategic importance of link as identified in road network strategy;
- Condition of structure and rate of deterioration;
- Load capacity and heavy vehicle demands and strategies;
- Overload frequencies and quantum;
- Social and economic impact of bridge taken out of service;
- Environmental impacts; and
- Availability of alternative routes.

A consistent approach to the maintenance of bridges on a particular link should be adopted across district and regional boundaries. The "Whichbridge" methodology and software, covered in Section 2.5.2 has been developed to assist this endeavour.

2.1.2 Maintenance Definitions

Bridge maintenance is work performed during the service life of a structure to:

- Maintain its designed load capacity, functionality and serviceability;
- Ensure that the structure completes its designed service life; and
- Preserves the State's investment in its structural assets.

It includes both reactive and preventative activities that preserve or restore the condition of a structure or its constituent parts. Restorative works are generally termed rehabilitation activities.

The normal maintenance works carried out by Main Roads may be considered under:-

(1) Routine Maintenance
(2) Programmed Maintenance
(3) Rehabilitation

2.2 Routine Maintenance

2.2.1 General

This covers those activities, identified primarily by Level 1 and Level 2 inspections, that maintain the serviceability of the structure, and generally fall within the scope of Road Maintenance Performance Contract. These activities generally do not change structure condition and include clearing of drainage, localized repairs to the road surface, cleaning and adjusting deck joints and removal of debris.

2.2.2 Maintenance Activities

The main activities associated with routine maintenance of timber bridges are:-

(1) Tightening and replacing bolts in components.
(2) Applying preservative or waterproofing treatments on components, particular member ends.
(3) Temporary propping of defective components.
(4) Repairing bridge footway surfaces.
(5) Clearing scuppers
(6) Cleaning aggressive contamination on steel components.
(7) Removing deck vegetation or excessive vegetation which maybe a fire risk below the bridge.
(8) Spraying termicide poisons.

Refer to the relevant components in Part 2 for a further discussion of relevant routine maintenance activities.

2.3 Programmed Maintenance

2.3.1 General

This covers those activities, identified from the bridge inspection programme, that maintain the serviceability of the structure, and fall outside the scope of the Road Maintenance Performance Contract. While these activities generally do not change the structural condition, they may include the replacement of isolated timber bridge principal members and non-load bearing components in all structures. Programmed maintenance activities include painting of steelwork, repair or replacement of deck joints or seals, barrier repairs, timber member replacement and repair of scour damage to beds and batters.

2.3.2 Maintenance Activities

Programmed Maintenance includes both temporary repairs and longer term repairs of components, including both primary structural members and other non-critical members. Typical components replaced in timber bridges would be:-

(1) Girders and corbels
(2) Piles
(3) Headstocks
(4) Decking, kerbs

Emergency repairs of critical components may also be included in this group.

2.4 Rehabilitation

2.4.1 General

The objective of rehabilitation is to restore the structure to "as new" condition with respect to the original designed load capacity and level of service. This excludes the strengthening of bridges to provide a load capacity greater than the original design. Strengthening is considered to be part of the capital enhancement programme of works.

Rehabilitation activities includes deck replacement, splicing piles, installation of supplementary piles, plating corroded steel sections or barrier replacements.

2.4.2 Maintenance Activities

Rehabilitation implies a considerable amount of repair or replacement of components, and may often include complete replacement of superstructure spans as well as pile replacement or splicing. This work would be considered to be long term, with the bridge to remain in service for a considerable period of time.

2.5 Maintenance Management

2.5.1 General (BAMS)

The successful management of timber bridge assets requires the integration of inspections, design and maintenance strategies. This includes construction detailing to reduce the potential for deterioration due to in-built defects.

As part of the Bridge Asset Management System (BAMS) that has been recently developed, the Main Roads Bridge Inspection Manual (BIM) was released in July 1998 to introduce mandatory statewide and systematic procedures for inspection and condition rating. The objective of the policy and methodology was to provide network managers with consistent and reliable data to

- Determine the current load capacity of bridges;
- Identify maintenance needs;
- Assess the effectiveness of treatments;
- Model patterns of deterioration; and
- Forecast future maintenance, rehabilitation and replacement budget needs.

A visual inspection of every element above ground level is conducted by an accredited inspector. The inspection requires the compilation of a detailed inventory of standard components and the assignment of numerical condition ratings to each component and the structure as a whole based on standard state descriptions.

The Bridge information System (BIS) is the hub of the BAMS and provides the repository for, and a means of managing, all data pertaining to bridges. It has been designed to be readily accessible to all personnel involved in bridge management functions including planning, design, construction, maintenance and heavy load management. The BIS is a relational database constructed on the ARMIS platform which permits the linking of all data through a common reference system. As a consequence users have access to other ARMIS systems such as "Chart View" and "Map View" which permits a graphical representation of data queries. A limited number of standard reports are available which largely meet the requirements of the majority of users, however the "Data Browser" query tool permits the generation of customized reports across the entire range of stored road and bridge data. The BIS currently comprises the "Structural", "Design and Inspection", Maintenance" inventories, and "Bridge Capacity" and "Maintenance Prioritisation" modules.

2.5.2 Prioritisation

Prioritisation of maintenance activities is important for managing of bridges on a network basis. The safety of the public shall be paramount when prioritising bridge maintenance and it will be necessary to exercise informed engineering judgement when allocating scarce maintenance funds. It should be a guiding principle of asset managers that advice must be sought from the Executive Director (Structures) when structures are found to be in extremely poor condition and the safety of the public is at risk. The management options for this category of structures are itemised below.

Structure Condition State 5 (unsafe)

In this event, the structural integrity has been severely compromised and the structure must be taken out of service until a bridge engineer from Structures Division has reviewed the Level 2 Report on the structure and recommended the required remedial action. If no viable alternative crossing is available, immediate funding must be provided to restore the structure to an acceptable level of service. The following management options should be considered.

- Close structure and establish a side track; or
- Close structure, advertise the fact and direct traffic to an alternative crossing; and
- Contact Executive Director (Structures) or his delegate for advice which may include one or more of the following options;

  - Impose weight or width or speed restrictions or a combination thereof

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- Install height bars on each approach, and advertise the fact, to reinforce restrictions on vehicle weight.
- Raise an "Issues Alert" to the DDG;
- Install temporary propping or other strengthening;
- Carry out partial or full rehabilitation of structure; and
- Initiate a bridge replacement scheme

Structure Condition state 4 (Very poor)

The inspection programme has identified serious defects that affect the structure's performance and integrity. If principal, load bearing, components are affected then immediate intervention is required including a review of the Level 2 Report by a bridge engineer. Typically components rated as Condition State 4 will be showing advanced deterioration as evidenced by loss of section from the parent material, signs of overstressing or evidence that it is acting differently to its intended mode or function.

In this instance funding must be sought immediately to conduct the necessary investigatory works and make safe any grossly defective components. Other non-essential works may be deferred. With the exception of the "close structure" option, bridge management options to be considered are as listed for structures in Condition State 5.

Structure Condition State 3 (Poor)

Condition State 3 indicates that defects have been identified which are compromising the serviceability of the structure and require intervention within the next year to prevent the onset of structural damage if deterioration is unmitigated. If principal members are affected, a monitoring programme or Level 2 inspection may be required. Typically, components will be showing marked and advanced deterioration including the loss of protective coatings and minor loss of section from the parent material.

Given this scenario, funding should be sought and generally should be made available to conduct the investigatory works and effect remedial action to prevent the onset of structural damage in critical members. Deferral of maintenance when members have reached this stage of deterioration will generally not be cost effective and may compromise the long term integrity of the structure and safety of road users.

"Whichbridge" Prioritisation Methodology & Software

In order to provide a rational basis for distributing maintenance funds, a prioritisation module was added to the BIS in March 2003. The purpose of the module is to simplify decision making in a process which is complex because of the variability of numbers and level of defect in members of varying criticality to structure safety.

The prioritisation tool is a risk based methodology and software developed by BAM & ARRB, and is integrated with the BIS. It relies on Level 2 Inspection Reports and other data normally collected and contained within the BIS.

The software is effectively a management interface that filters data from the BIS and ranks the bridge stock in descending order of risk.

The ranking is compiled by a multi-criteria risk qualification process that determines the relative risk arising from various structural, road network, social and economic factors.

Examples of these are:- component load carrying importance, condition state, consequences of failure or detour distance.

A ranking above a nominated value of 1500 is deemed to represent a defective structure in need of urgent intervention.

The prioritisation tool is designed to assist network managers assign priorities and is not intended to be a "black box" solution that supplants sound engineering judgement.

For further information, refer to the "WHICHBRIDGE" Bridge and Culvert Maintenance Prioritisation Software User Guide (Reference 1).

An output extract and a sensitivity analysis on a timber bridge are included here for information.

Reference

1. Bridge Asset Management (2003). Whichbridge Bridge and culvert Maintenance Prioritisation Software - User Guide. Department of Main Road, Queensland.

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WhichBridge
Bridge and Culvert Maintenance Prioritisation Software

Structure Level Results - Bridges

ID	Name	District	Road ID	COF	PSF	MDF	DSR	Risk	No. Groups	Comps not included		Rank	Default
										No.	%		
7260	"Forgan Bridge"	8	'857	15	8470	69.1	2209	127044	71	96	23%	1	Yes
1054	"Scrubby Creek"	2	'40A	11.5	6676	41.0	5173	76775	16			2	Yes
950	"Vic Olsen Bridge"	2	'482	9.5	8059	36.7	10852	76563	15			3	Yes
1133	"Barcoo River"	7	'716	10	7062	50.8	4845	70621	35			4	Yes
845	"Kin Kin Creek No 3"	2	'141	7	9684	24.8	11242	67785	9			5	Yes
8178	"Burnett River"	12	'41C	13	4990	39.4	1997	64874	52			6	Yes
577	"Dumeresq River"	5	'241	9.5	6446	60.1	7380	61236	20	18	22%	7	Yes
7238	"Cattle Creek No 3"	8	'532	12	4938	49.6	4345	59250	27			8	Yes
7233	"Pioneer River"	8	'530	12.5	4368	18.1	529	54603	57			9	Yes
7322	"Alligator Creek"	8	'10G	16.5	2927	31.0	1045	48299	59	11	4%	10	Yes
16224	"Paroo River"	4	'94A	8	5211	76.0	10445	41690	18	10	21%	11	Yes
7237	"David Burgess Bridge"	8	'532	13	3118	40.3	2527	40529	33			12	Yes
7222	"Running Creek"	8	'517	9	4125	24.4	3463	37123	9			13	Yes
8378	"Un-Named Creek #1"	13	'305	6	5926	16.8	6640	35554	5			14	Yes
8171	"Fox Creek"	12	'41C	9.5	3428	14.2	1082	32571	14	4	8%	15	Yes
8379	"Western Creek"	13	'305	6	4668	14.9	2770	28008	9			16	Yes
539	"Accomodation Creek"	5	'22C	11	2494	33.5	1647	27437	15	15	54%	17	Yes
8279	"Clovernook Creek"	15	'46C	9.5	2854	26.6	3499	27108	15	1	2%	18	Yes
700	"South Kariboe Creek"	6	'41D	11.5	2345	15.2	1045	26966	26	1	1%	19	Yes
8042	"St Johns Creek"	12	'454	8.5	3010	29.3	2625	25583	18	1	2%	20	Yes
365	"Wallaby Creek"	3	'40B	11.5	2156	19.0	1031	24793	19	10	13%	21	Yes
7549	"Bohle River"	9	'83A	12.5	1904	54.9	6303	23797	11			22	Yes
8059	"Bin Bin Creek"	12	'475	8.5	2602	16.3	1783	22116	12			23	Yes
1349	"Sheepskin Creek"	8	'512	10.5	2079	13.9	752	21831	20			24	Yes
16221	"Bulloo River Bridge"	4	'94A	8.5	2513	38.5	3931	21362	9	2	6%	25	Yes
1004	"Eel Creek"	2	'4806	9.5	2176	8.4	823	20669	17			26	Yes
325	"Boyne River"	3	'4356	9	2252	10.6	462	20264	15			27	Yes
7211	"Stony Creek"	8	'512	10.5	1926	19.8	2287	20225	9			28	Yes
7201	"Plain Creek"	8	'512	10.5	1863	16.0	1626	19558	12			29	Yes
259	"Waterfall Gully"	3	'405	8	2306	24.4	4420	18447	9	1	3%	30	Yes

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WhichBridge
Bridge and Culvert Maintenance Prioritisation Software

CONDITION STATE 4 DEFECTS (ALL OTHERS CS1; COF = 13.5 Fs = 10)	MDF	DSR	PSF	RISK
None	1	10	50	678
Span 1:				
1 Girder	1.00	10	52	699*
2 Girders	1.53	17	82	1105
3 Girders	2.20	28	121	1632
4 Girders	3.13	48	177	2391
5 Girders	4.33	81	252	3399
Spans 1-4:				
1 Girder	1.00	11	57	763*
2 Girders	3.13	48	197	2658
3 Girders	5.80	122	401	5416
4 Girders	9.53	292	720	9716
5 Girders	14.33	641	1173	15832•
100% Deck plus				
Spans 1-4: 1 Girder	7.67	182	529	7139*
Span1: 5 Girders	11.00	371	776	10477
100% Deck and Steel Transoms plus				
Spans 1-4: 1 Girder	27.67	1855	2154	29083*
Span 1: 5 Girders	31.00	2483	2463	33248
100% Deck, Transoms, Spans 1-4: 1 Girder plus				
1 Headstock at Abutment	28.92	2073	2452	33097*
2 Headstocks at Abutment	32.67	2937	2996	40440*
1 Headstock at 2Abuts & 3 Piers	32.04	2759	3603	48642*
2 Headstocks at 2 Abuts & 3 Piers	45.17	6969	6641	89655*
100% Deck, Transoms, Headstocks Piles plus Span 1-4: 1 Girder	97.67	64936	23818	321547*
100% Deck, Transoms, Girders, Headstocks, Piles	111.00	82817	29878	403351•
100% Deck, Transoms, Corbels, Headstocks Piles, Bracing plus:				
Span 1-4: 1 Girder	112.67	115856	35893	484556*
Span 1-4: 5 Girders	126.00	139801	43328	584929•

Note: An Average Group Risk of 1500 represents threshold.

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2.5.3 Defective & Sub-standard Bridges

Defective Structures may be defined by any of the following criteria:-

1. More than 25% of principal components are rated in Condition State 4 within a single abutment, pier or span goup.
2. Greater than 25% of principal timber components are undersized in a single abutment, pier or span group.
3. Risk rating is in excess of 1500.
4. Overall condition rating of structure is 4 or 5.

Sub-standard Structures may be defined by any of the following criteria.

1. Timber bridges of less than 'A' Class design (Am, B & Bm)
2. Bridges of unknown design class.
3. Bridges assessed by Structures Division as being deficient in load carrying capacity.

Where a defective structure is detected, a "Structure Management Plan" shall be prepared and submitted for approval. For sub-standard bridges a management plan will generally be prepared in conjunction with Structures Division.

Refer to Appendix C for full requirements for management of such structures.

3.0 General

3.1 Historical Background

In order to determine where timber bridging fits in the overall context of all bridge types, a brief discussion follows:

Prior to the formation of the Main Roads Board in 1922, road bridges in Queensland were generally constructed through either the State Works Department or the various Local Authorities. A number of these very early bridges are still in service, the oldest dating from 1885. The majority of existing bridges found on State Controlled Roads were built by Main Roads, though a small proportion of Local Authority or privately constructed bridges have been added as a result of road acquisitions over the years.

Today there are some 2700 bridges to be maintained by Main Roads, with structure type roughly reflecting the era of construction and with each type having its own unique maintenance concerns. The major superstructure groups now existing consist of timber girders, reinforced concrete girders and slabs, composite steel girders and prestressed concrete girders and deck units. As well, a small number of steel truss bridges remain in service, though all timber truss bridges have been replaced, except for two cross-border facilities.

Because of early plentiful supplies of very strong and durable hardwoods, timber initially became the traditional form of bridging used in Queensland. Layouts and details were standardized by 1925 and no changes to basic structure and member sizes have been made to the present time. It should be noted, however, that there have been very significant increases in the mass of vehicles allowed on roads since these bridges were developed (approximately 4 tonnes per decade) contributing partly to the high maintenance costs for these structures.

In the past, timber has made a very significant contribution to our bridging needs but many of these structures have now been replaced. Figure 3.1 plots the number of remaining timber bridges compared to other types, based on construction year. Though timber bridges were built into the early 1960's, it can be seen that these now form an ageing population with a mean age of some 60 years.

Available records indicate that between 1925 and 1970, some 1300 timber bridges were built by Main Roads. At the present time, there are about 500 timber bridges still in service, with the largest population on the lesser class roads. At the current rate of replacement, it is apparent that there will be an ongoing need for timber bridge management for another 30 to 40 years.

Unfortunately, timber is susceptible to the most severe deterioration mechanisms such as rotting and insect attack and requires substantial maintenance outlays for the structure life. This problem is being compounded today because of increasing difficulties in obtaining large section logs and sawn timber as forest reserves dwindle. Considerable use of alternative decking such as steel trough, plywood or precast concrete slabs is now common.

Another early but more durable form of construction was that of reinforced concrete bridging which was only built in small numbers compared to the contemporary timber structures. These bridges took the form of girder (T-beam) or slab bridges, often made structurally continuous. By today's standards, concrete strengths were low, with the most common defects being shrinkage, cracking and corrosion induced cracking and spalling due to the lack of effective protection to the reinforcement.

The composite steel girder bridge in which the concrete deck slab is designed to act structurally with the longitudinal steel girders was the next important form of bridging to be developed. Main Roads constructed the first example in 1935,(believed to be the second in the world) but it was not until the early 1950's that substantial use was begun, subsequent to the preparation of a set of standard designs. A variety of spans were developed with 13.7m spans being the most common while longer spans were accommodated by making the girders continuous over 3 spans. This form of bridging began to replace timber, and by 1958 some 50% of bridging was in steel with timber still providing most of the remainder. With steel girder bridges, the main defect found has been metal corrosion in the girders and bearings in particular.

Today, essentially all our bridge construction is in prestressed concrete, either in the form of deck units or girders. The first such bridge was built in 1954 and from about 1962 onwards prestressed concrete construction began to rapidly replace both timber and steel as the preferred construction medium. Though initially thought of as having low maintenance concerns, experience has shown that considerable problems have occurred in PSC members. This problem is being addressed today by careful detailing practice and advancements in concrete technology understanding.

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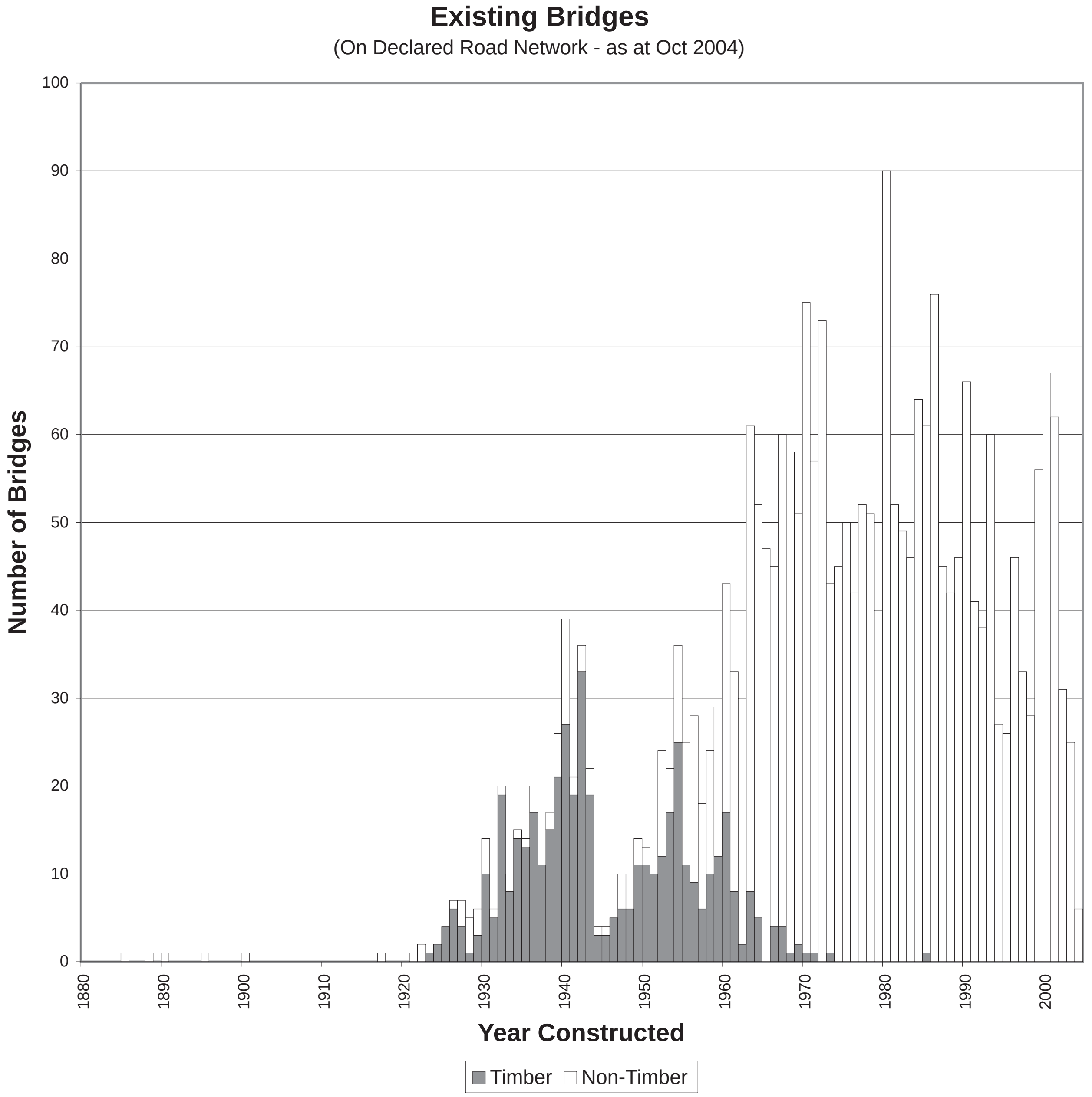

Figure 3.1 - Existing Bridges On Declared Road System

3.2 Definitions

The following lists the meanings of various terms, operations or components used with timber bridges. The names of components included here represent the common name used by Main Roads, Queensland and as shown on the original bridge drawings. A further description of function is given in Part 2. Further definitions pertaining to timber materials are given in MRS 11.87.

Backing Slabs:	Timber or concrete slabs placed horizontally at abutments or wings to retain embankment fill.
Ballast Board:	Timber planks placed horizontally at the ends of timber girders to retain fill.
Bracing:	Diagonal members placed across pier piles to transfer flood loads.
Cambering:	The jacking upwards of internal girders in order to bend decking to achieve tightness.
Check	Separation of timber fibres along the grain but not extending from one surface to another.
Composite:	"Composite action" is a structural analysis term indicating components acting together as a unit due to special connections between them.The term "Composite timber bridge" found on old drawings simply refers to the use of concrete, such as for a pier or deck in conjunction with timber.
Corbel:	A short timber member used at piers to help support the ends of timber girders. On concrete piers, an equivalent concrete (in-situ) member may be used.
Decking:	Originally consisted of hardwood planks placed transversely to girders and carried traffic loads. Various alternative forms of decking are also used now.
Distributor:	A member placed generally longitudinally below a deck (midway between girders) to improve wheel distribution.
Flexure:	Structural action referring to bending of a member.
Girders:	Main longitudinal member supporting the deck and spanning between piers.
Headstock:	Transverse member/s placed across the top of abutment or pier piles to transfer superstructure loads to the support piles.
Kerbs:	Longitudinal members at the edge of a deck used to support barriers and to provide edge restraint to vehicles.
Piles:	Driven members used to transfer all bridge loads into the foundations. Also includes column members above sill beams.
Shear:	Structural action which tries to split a loaded member longitudinally.
Sill beam:	Member used to transfer loads from short piles or columns into the foundation.
Snipe:	Cut taken out of the ends of girders or corbels in order to provide seating area on the corbel or headstock.
Spiking Plank:	Timber member on top of outer girder on to which timber decking is spiked.
Split:	Separation of timber fibres along the grain which extends from one surface to another.
Substructure:	The lower supporting members of a bridge comprising piles, headstock and corbels.
Superstructure:	The upper supporting members of a bridge comprising girders, deck and barriers.
Tingling:	Material placed over excessive width gaps between timber deck planks.

Figures 3.2(a) & (b) show photos of the various components, while Section 3.3 includes a typical bridge drawing.

Headstock - pier

Abut piles and backing slabs

Figure 3.2(a) - Timber Bridge Photos (1)

Girders and corbels

Deck and distributors

Kerb and barrier

Figure 3.2(b) - Timber Bridge Photos (2)

1

3.3 Timber Bridge Details

Figure 3.3(a) shows a typical standard drawing for a Main Roads timber bridge. These bridges were, in most cases, built as 4 girder (3.7m wide deck), 5 girder (5.5m deck) or 6 girder (6.1m deck). Larger numbers of girders or non-symmetrical girder spacings are generally the result of past bridge widenings.

Apart from a small number of remaining "all girder" bridges (timber girders touching side by side to form a slab), the timber bridge superstructure consisted of longitudinal round log girders with transverse sawn timber decking. Substructure consisted generally of driven timber piles with timber headstocks and bracing, though a number of bridges with concrete substructure were built. Figure 3.3(b) shows the main components of the traditional timber bridge.

For the purpose of supplying acceptable high strength and durable hardwood timber, Main Roads Specifications split the State into 5 regions for sourcing timber, each with specifically approved timber species for the various bridge components.

One design feature believed unique to Queensland bridges was the use of a spiking plank on top of the outer girders, with no connection of the decking to the inner girders. Cambering (jacking up of internal girders) was used to keep the deck/girder system tight and so reduce rattling. Because girder splitting initiated by deck spiking was absent, one source of girder deterioration was eliminated.

A feature, common in all States, was the use of timber corbels between girders and pier headstocks. Though there are potential structural benefits to girder capacity by introducing partial structural continuity at piers (provided the bolt system is tight), the main reason provided by early sources was that the time before girder replacement, necessitated by end deterioration of girders could be extended because of the longer support provided by the corbel.

In many cases, hardwood decking has been replaced by alternative forms such as plywood or steel troughing, while some use of prestressed concrete planking has been made in recent times.

Concrete was sometimes used for substructure components such as abutments or pier sill bases where there was inadequate depth for driven piles (generally 4.6m minimum penetration). Note that the term "composite" on the drawings for these bridges was not used in the context of the structural composite action which applies to steel and prestressed concrete girders where it defines interconnected action between deck and girder.

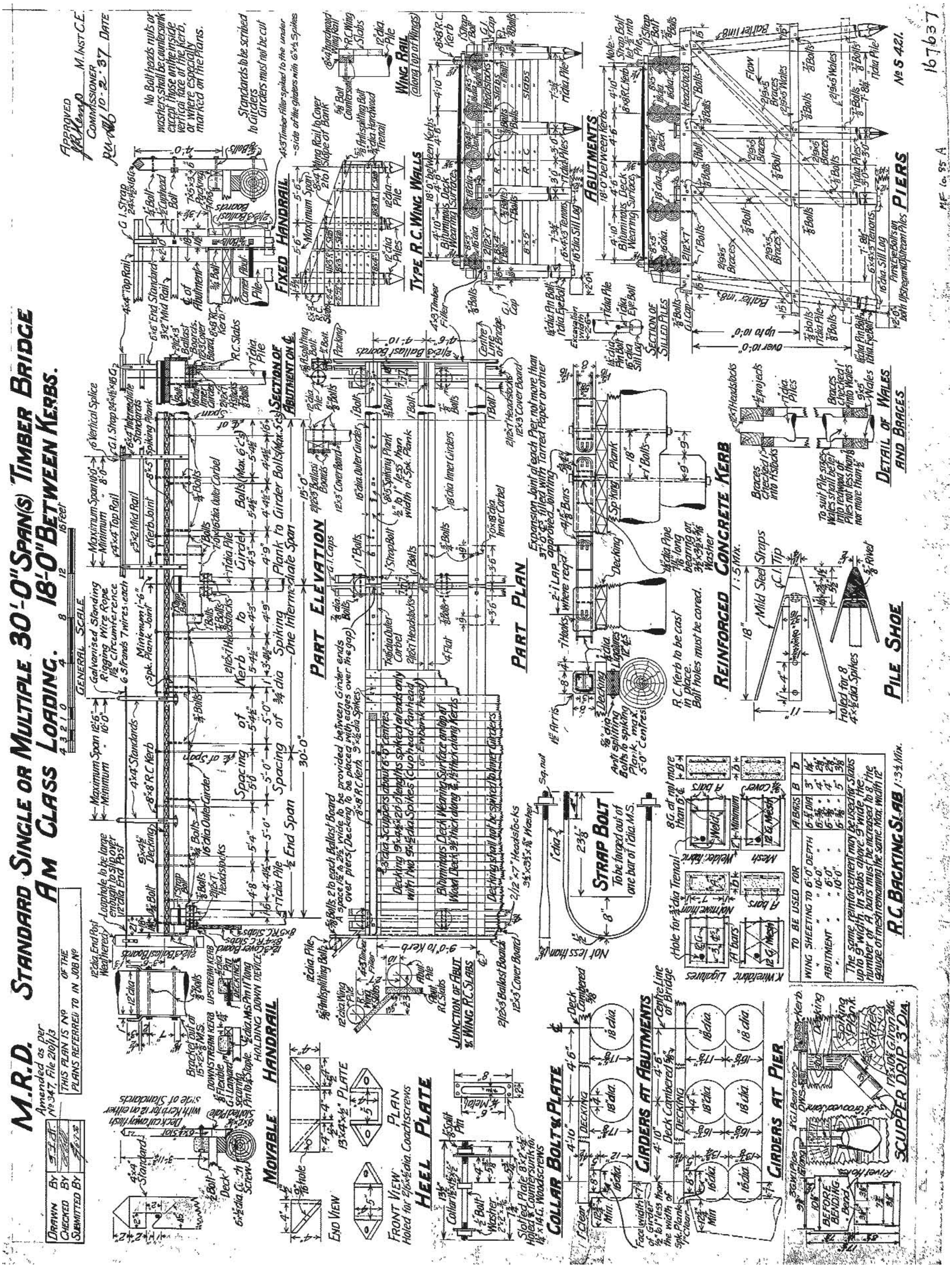

Figure 3.3(a) - Standard MRD Timber Bridge Drawing

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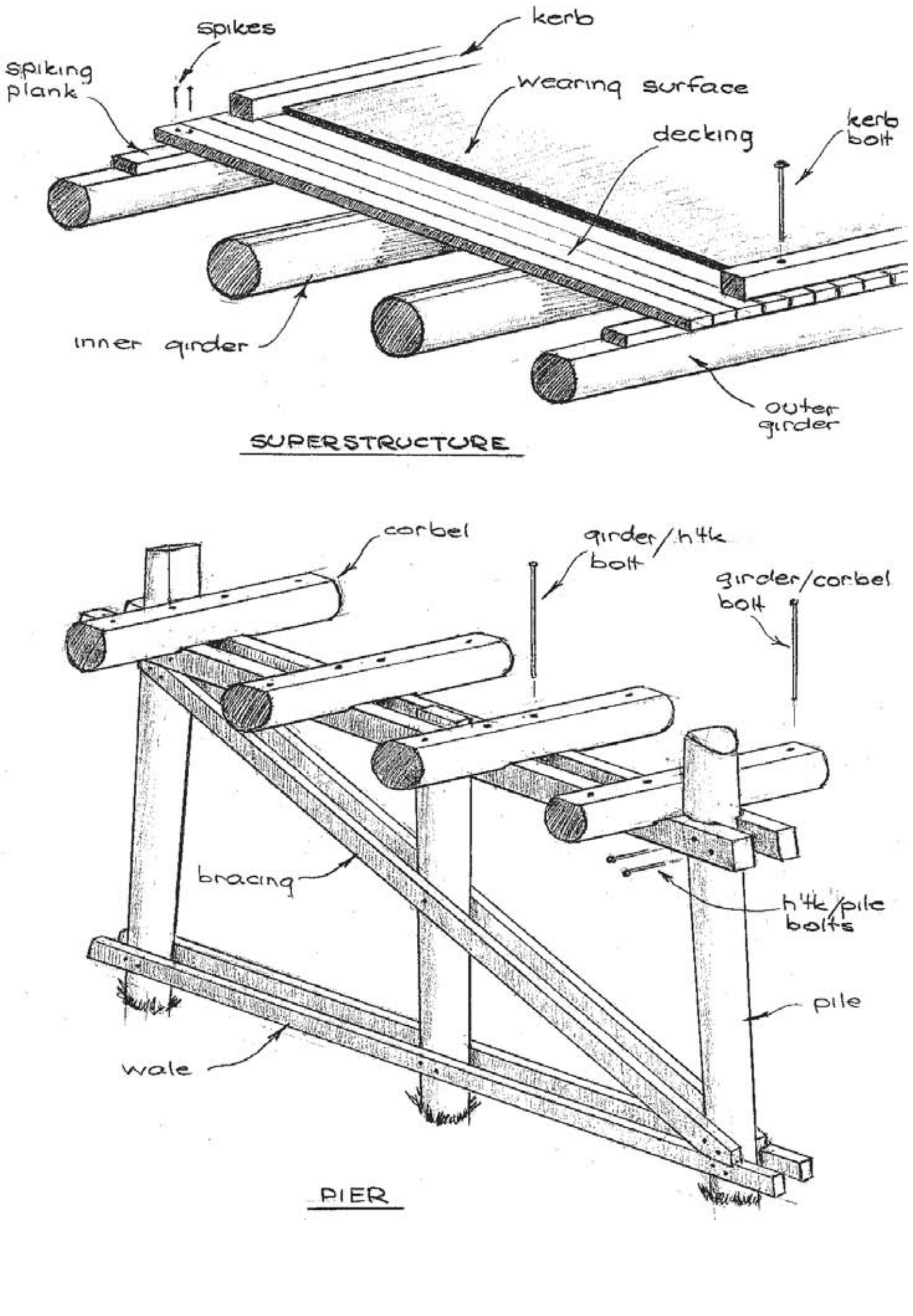


Figure 3.3(b) - Timber Bridge Details

3.4 Bridge Design Class

The bridge design class refers to the original designated design vehicle that was used to determine member sizes. It should be noted that, for timber bridges, design classes and member sizes that were determined in the 1920's have never changed, even though there has been a very substantial increase in mass of vehicles permitted on our roads (16 tonne trucks in 1922 to 42.5 tonne semi trailers today).

Up to the present time, bridge load limits have not generally been posted on the basis of design class. The necessity for load limits has been determined by BAM on an individual basis with consideration of bridge condition.

The design classes found on timber bridges are:

- 'A' class (also called "first class")..**Strongest**
- 'Am' class
- 'B' class (also called "second class")
- 'Bm' class -..**Weakest**

Figure 3.4(a) shows details of the 'A' & 'B' class vehicles respectively, each representing a tractor followed by 3 trailers. It should be noted these design classes are unique to Queensland and would be different to a Vic Roads 'A' class, for example.

The 'Am' (i.e. 'A' modified) and 'Bm' class bridges were a lower capacity version of the original with either

(a) 25 mm (1 inch) less for girder diameters, or
(b) stretched span for a certain girder size.

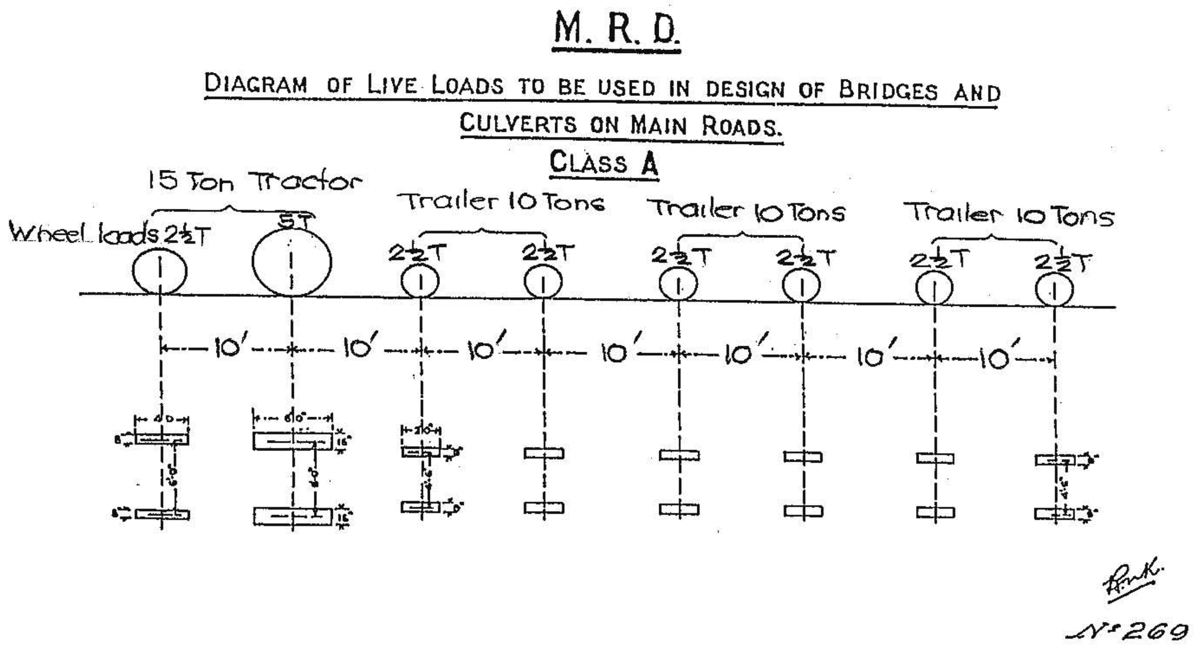
As such there is no actual design vehicle for these two classes.

In summary, design class had the following effect on girder sizes. Compared to an 'A' class bridge:

'Am' class had 25mm (1 inch) less for girder diameters
'B' class had 51mm (2 inch) less for girder diameters
'Bm' class had 75mm (3 inch) less for girder diameters

Figure 3.4(b) gives a summary of girder diameters for various spans and design classes for standard timber bridges.

For non-timber bridges, design class has periodically changed, with design vehicle mass progressively increasing. From about 1955, our bridges were designed for a H20S16 vehicle (also called HS20 and MS18 at various times). In 1976, the T44 design vehicles was introduced and remains the primary design vehicle to the present time, though a much heavier design vehicle is proposed for introduction in the near future.



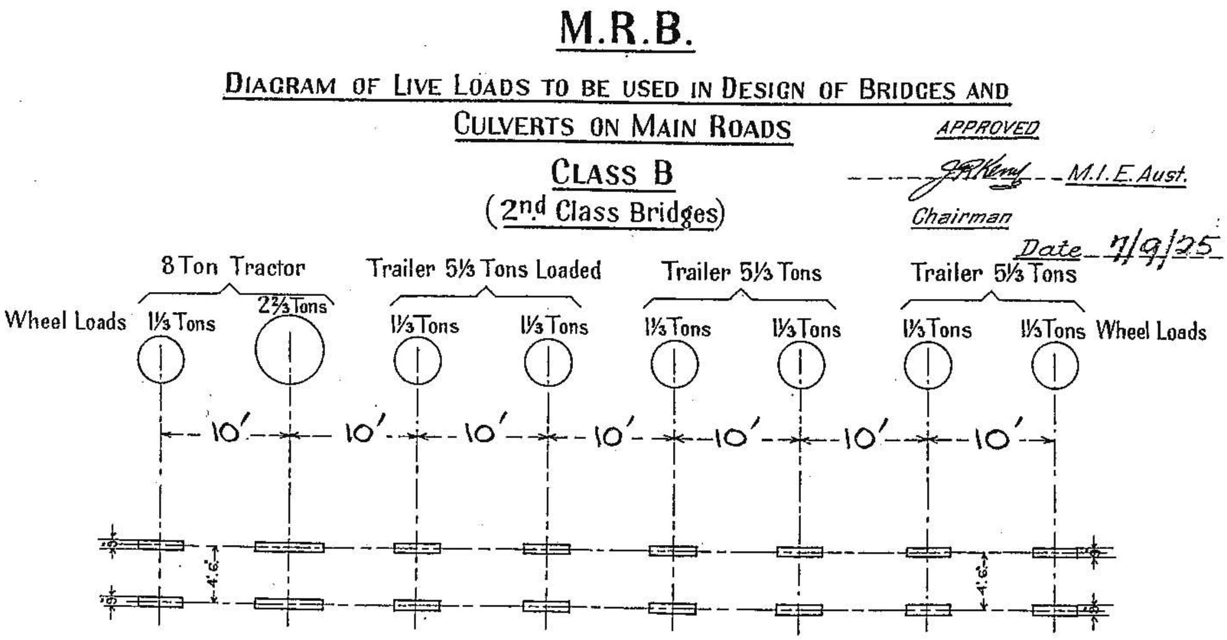


Figure 3.4(a) -'A' & 'B' Class Design Vehicles

'A' CLASS LOADING:-

SPAN m	ORIGINAL DECK WIDTH m	NO. OF GIRDERS	GIRDER DIAMETER Inches (mm)	
			OUTER	INNER
4.6 - 4.9	3.66	4	12 (304)	14 (355)
	5.5	5		
5.2 - 5.8	3.66	4	13 (330)	15 (381)
	5.5	5		
6.1 - 6.7	3.66	4	14 (355)	16 (406)
	5.5	5		
7.0	3.66	4	15 (381)	17 (431)
	5.5	5		
7.3 - 8.2	3.66	4	16 (406)	18 (457)
	5.5	5		
8.5 - 9.1	3.66	4	17 (431)	19 (482)
	5.5	5		
	6.1	6	16 (406)	18 (457)
10.7	3.66	4	19 (482)	21 (533)
	6.1	6	18 (457)	20 (508)

'B' CLASS LOADING:-

SPAN m	ORIGINAL DECK WIDTH m	NO. OF GIRDERS	GIRDER DIAMETER Inches (mm)	
			OUTER	INNER
3.0 - 4.9	3.66	4	10 (254)	12 (304)
	5.5	5		
5.2 - 5.8	3.66	4	11 (279)	13 (330)
	5.5	5		
6.1 - 6.7	3.66	4	12 (304)	14 (355)
	5.5	5		
7.0 - 7.6	3.66	4	13 (330)	15 (381)
	5.5	5		
7.9 - 8.5	3.66	4	14 (355)	16 (406)
	5.5	5		
8.8 - 9.1	3.66	4	15 (381)	17 (431)
	5.5	5		
10.7	3.66	4	16 (406)	18 (457)
	6.1	6		

Decking 'A' Class 9 in x 5 in (228 x 127)
'B' Class 8 in x 3 ½ in (200 x 88)

Figure 3.4(b) - Standard Girder Diameters For Various Spans

3.5 Component Designation

The general terminology used to label timber bridge components is shown in Figure 3.5(a), while Figure 3.5(b) shows specific terminology. Figure 3.5(c) shows the order of groupings & components, widenings & lengthening, which are used for general bridge designation. For further discussion, refer to Section 1.3 of Part 3 of the Bridge Inspection Manual.

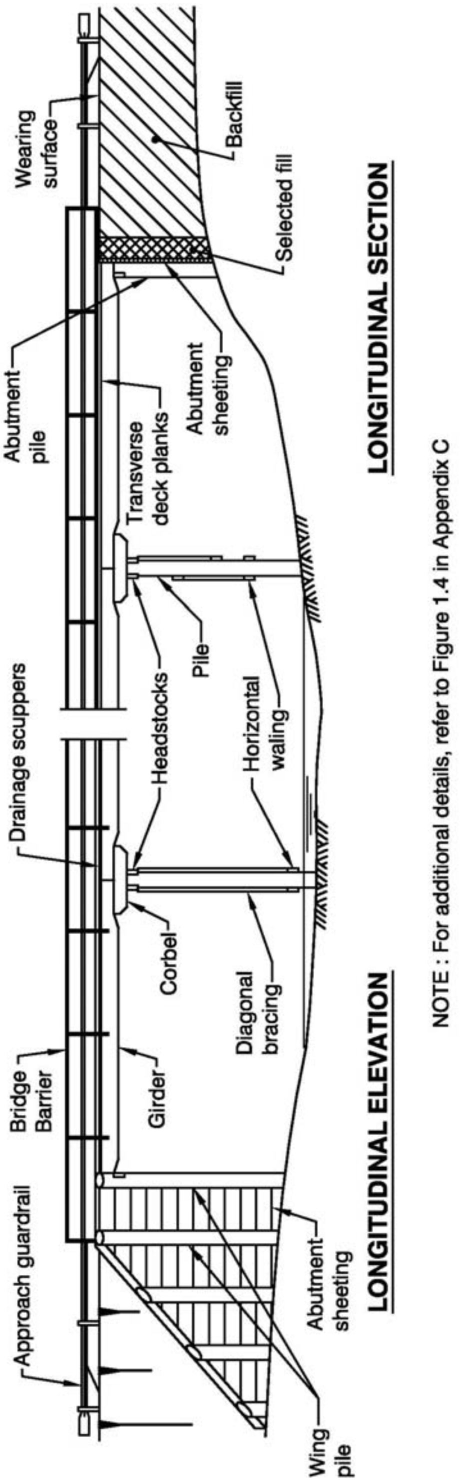
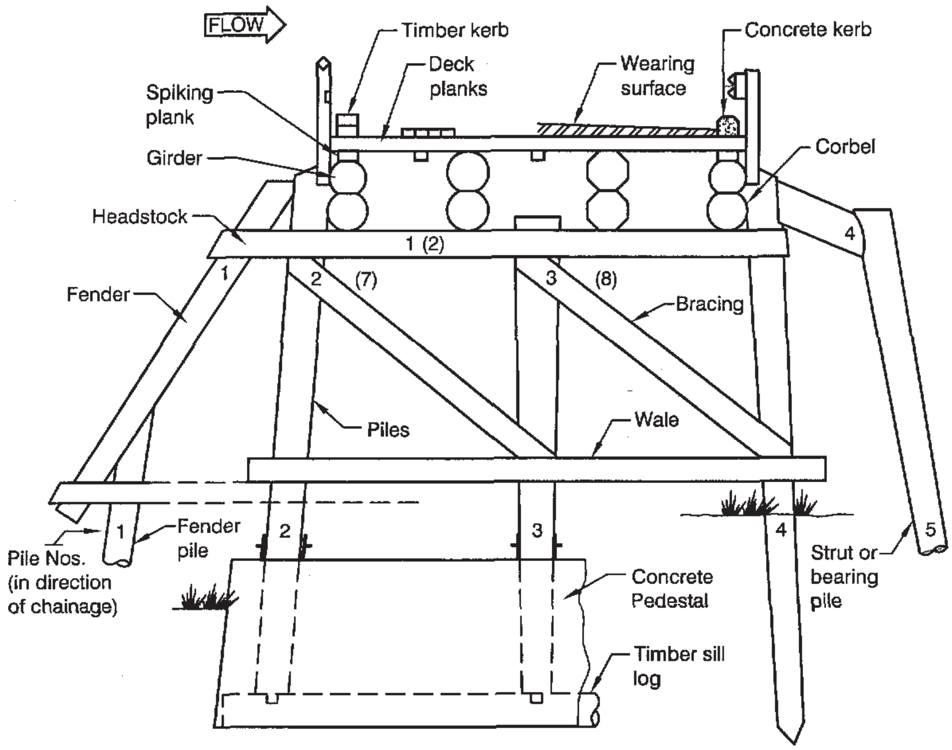
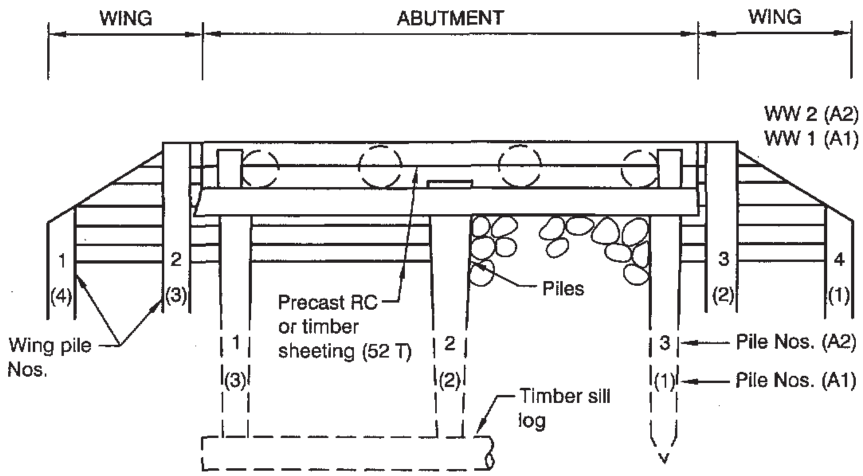


Figure 3.5(a) - Terminology for Timber Bridges



Bracing pair indicated 2 (7) with brackets denoting reverse side

PIER & DECK



ABUTMENT & WINGS

COMPONENT No.	
DECK	
1 C or O	WEARING SURFACE (if any)
2 T or S	BRIDGE BARRIERS
3 T or C	BRIDGE KERBS
SUPERSTRUCTURE	
29 T (or P)	DECK PLANKS
ALTERNATIVES	
20 T	PLY DECK OR STRESS LAMINATED
20 C	CONCRETE SLAB DECK
30 S	STEEL DECKING
22 T	GIRDERS
33 T	SPIKING PLANK
PIER	
27 T	CORBELS
54 T	PIER HEADSTOCKS (1 & 2)
56 T	PILES
57 T	BRACING & WALES
59 C, T	FOOTING& SILL LOG
ABUTMENT	
54 T	ABUTMENT (H'tks 1 & 2)
56 T	PILES
52 T, P or O	ABUT. SHEETING
53 O, P or C	PROTECTION
51 C, O or T	WING WALLS (discrete)
60 T, P, S	WING PILES

Figure 3.5(b)- Typical Timber Bridge

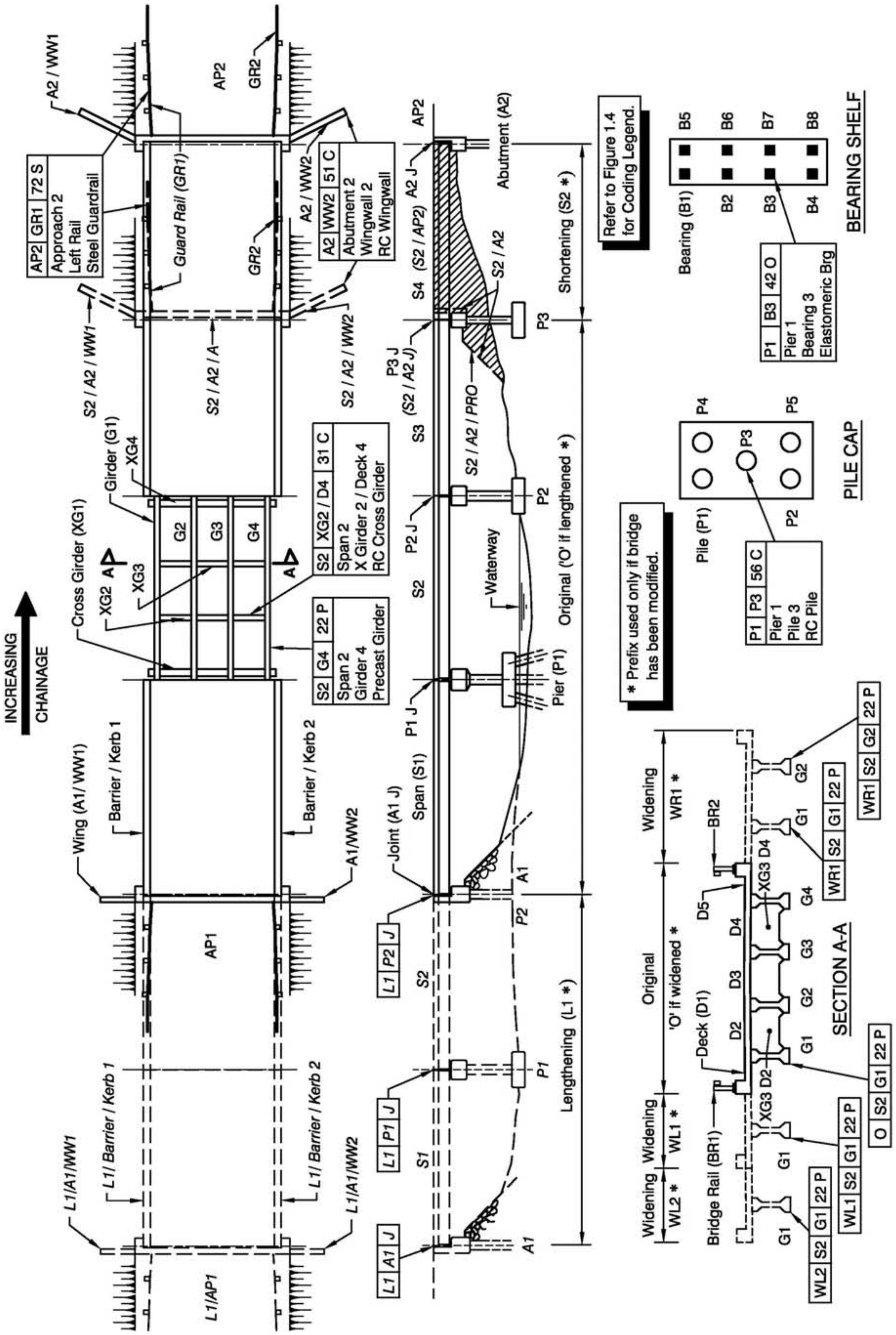


Figure3.5(c) - Bridge Component Designation

4.0 Timber Technology

4.1 Timber Structure

As a natural building material, timber has evolved unique material properties which dictate and influence use and also maintenance strategies. In the growing tree, the trunk acts as a structural member, anchored by the root system, to support the leaf and branch system. This ability to support both tree mass and wind induced loads makes timber a practical material for our structural component requirements such as for girders, piles and decking.

The structure of timber is essentially a collection of longitudinally orientated cellulose cells, cemented together by lignin, a complex polymer compound with also strengthens the cell walls. Figure 4.1(a) shows magnified structures for hardwood and softwood timbers and is included to show the general assemblage of wood cells. The structure of the hardwoods is more complex than those of softwoods.

Except for plywood decking, all timber used for conventional timber bridge maintenance is hardwood, and Figure 4.1(b) shows a section of a typical hardwood tree trunk, the main parts being:

Bark - this thin, external layer protects the trunk from fire and other injury and helps transport nutrients.

Cambium - this thin layer, immediately adjacent to the bark, is where new wood cells grow.

Sapwood - this layer, generally from 10 to 50mm thick and adjacent to the cambium, constitutes the living portion of the trunk, where water and nutrient flow occurs between roots and leaves. Because it is rich in nutrients such as starch, sapwood is very susceptible to fungal attack when it remains on structural timber.

Heartwood - this is the inner region of the trunk and is composed of the dead cells remaining after the sapwood growth front has moved further out. These cells become filled with wast products which result in heartwood having a darker colour and greater durability than sapwood.

Most current publications appear to define heartwood as all timber within the sapwood perimeter. It should be noted that older Main Roads Specification references to "heart" are referring to only the very central region of a log (also referred to as the pith) which is of lower durability because of inferior cell structure and density.

Because timber is essentially composed of longitudinal cells, its properties are anisotropic, i.e. strength and stiffness properties are much higher along the grain than across the grain. Another property that varies between tangential, radial and longitudinal directions in a log is shrinkage, which occurs as timber moisture content gradually reduces. Shrinkage is greatest in the tangential direction and results in the formation of longitudinal checks or cracks in the timber due to its weakness in tension across the grain.

4.2 Structural And Material Actions

The main structural actions that timber bridge members have to resist are bending and shear. In a bridge girder, the central region is subject mainly to bending effects from the weight of the structure and from traffic loads. The bending action results generally in tensile forces in the bottom section of the girder and compression forces on the opposite side. As indicated previously, timber fibres are strong in tension and compression due to alignment along the member. In a small timber section, free of imperfections, the result of beam bending is generally a ductile failure of the fibres in compression. However, in a normal beam, imperfections such as knots or sloping grain induce tension perpendicular to the grain and fracture is normally at the tension face, occurring in a brittle manner. Locations where bending failures are generally expected to occur are shown in Figure 4.2(a).

Bending failure in girders and headstocks is normally the result of traffic and dead loads exceeding the residual capacity of the member, whereas with pier piles this is the result of large flood debris loads. There is also the possibility of occasional failure in abutment piles due to earth pressure loads. If a large length of pile is exposed by scour and there is a considerable depth of backing slabs, the pile may not be able to carry the earth pressure loads due to the embankment fill and the surcharge loads produced by vehicles on the embankment.

The other major action in girders is shear which has the greatest effect near the ends of the member and is typically considered close to an abutment support or at the half length of a corbel outstand. Though shear forces along a beam are caused by vertical loads, the effect is transferred to an action trying to split the beam horizontally along its central region. When shear failure occurs, the beam is split into top and bottom sections, effectively weakening it for bending as well. Figure 4.2(b) shows the location of expected shear failures. In practice, a combination of bending and shear occurs as the normal structural mechanism along the member. Cracking and checks are often found in the ends of girders, typically as shown in Figure 4.2(c). Because of fibre interlock near the base of the cracks, the segments of timber act as cooperating pieces, giving adequate shear capacity. However, the base of cracks are also areas prone to decay, and if this occurs, the end of the girder effectively becomes a number of individual components, further reducing strength. The shear capacity of a cracked girder will also be further reduced if there is an associated pipe in the member. The section will have less timber area to resist shear forces while the resultant smaller independent timber sections will also be more prone to twisting instability. Timber corbels are generally subject mainly to shear loads and are prone to the same failure mechanisms as girders.

If sufficient timber has been removed from girder and corbel sections by internal decay, the high vertical loads transferred through the ends of the girders may cause bearing failure in the timber, because the loading is across the grain. When this occurs, crushing in the ends of girders and in the corbel at its support will be evident. This leads to settlements at deck level and possible dropping out of corbels, if a collapse occurs.

Refer also to Figure 4.2(d) for photographs of member failures.

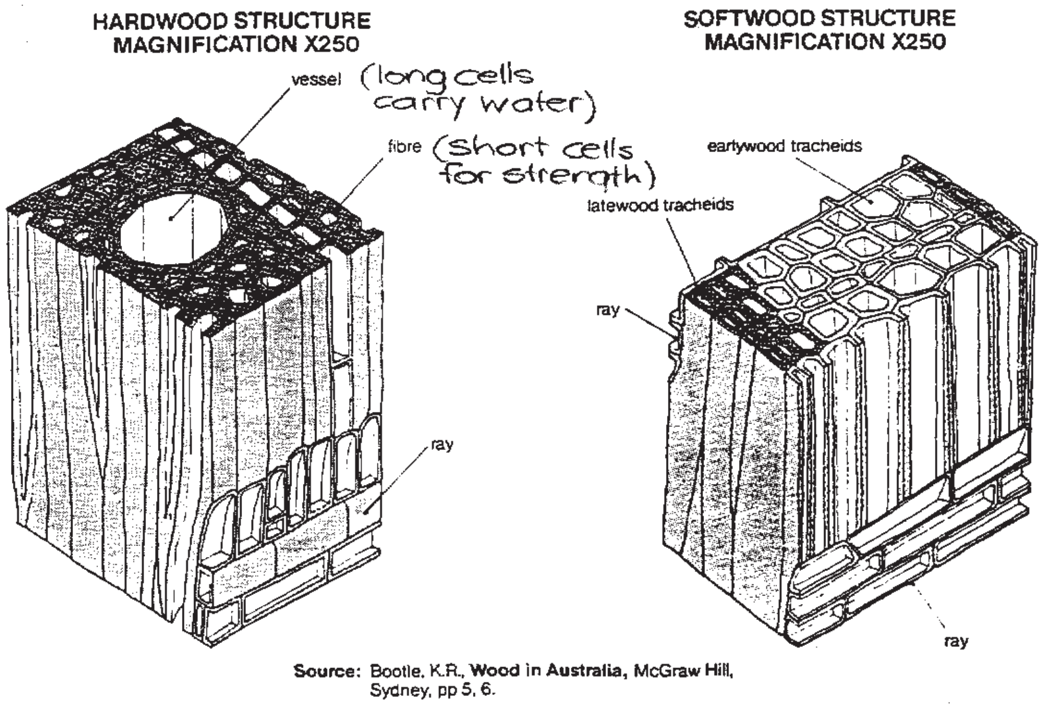


Figure 4.1(a) - Wood Structure

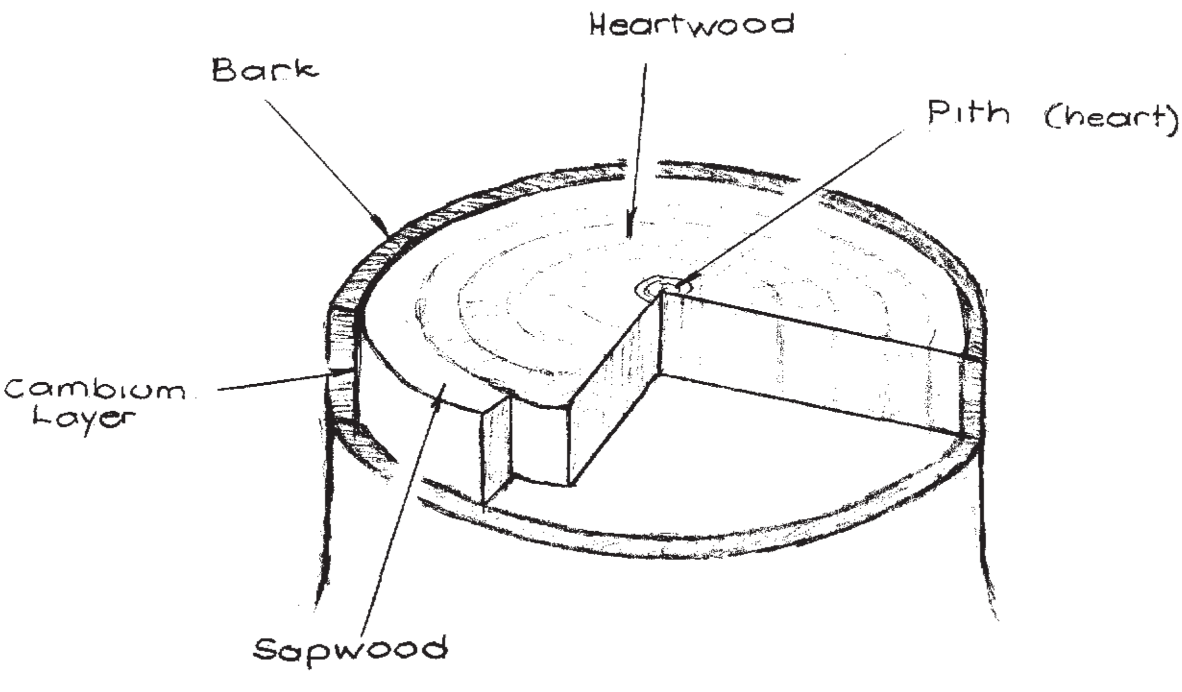


Figure 4.1(b) - Section - Tree Trunk

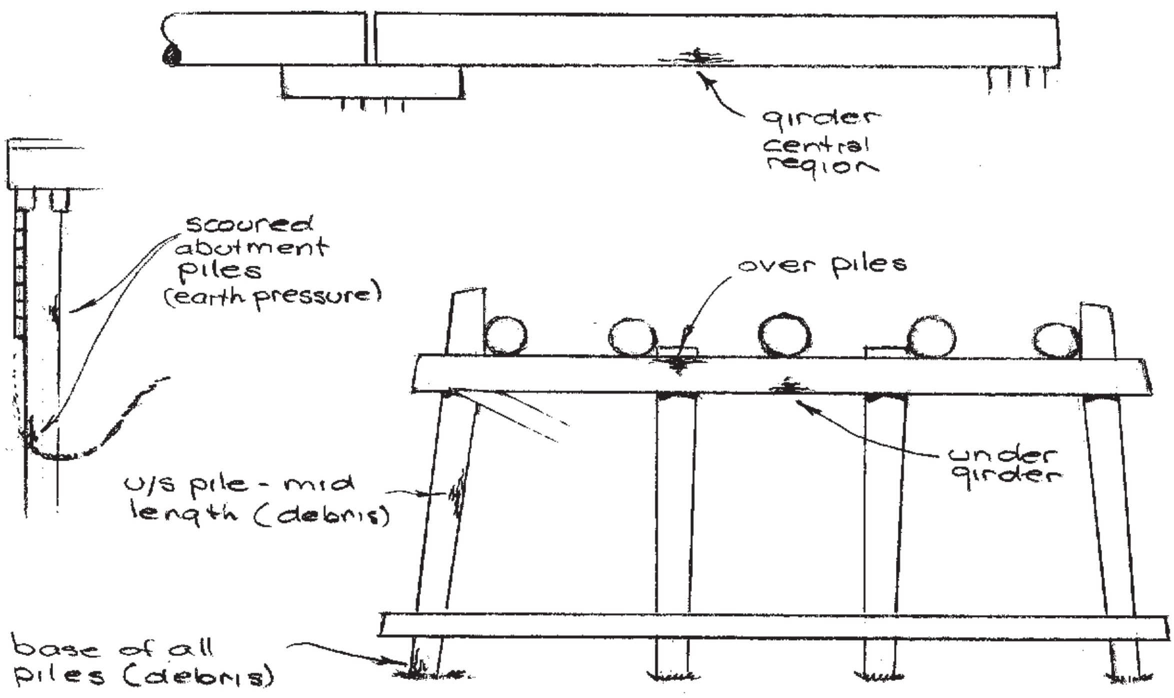


Figure 4.2(a) - Expected Bending Failure Locations

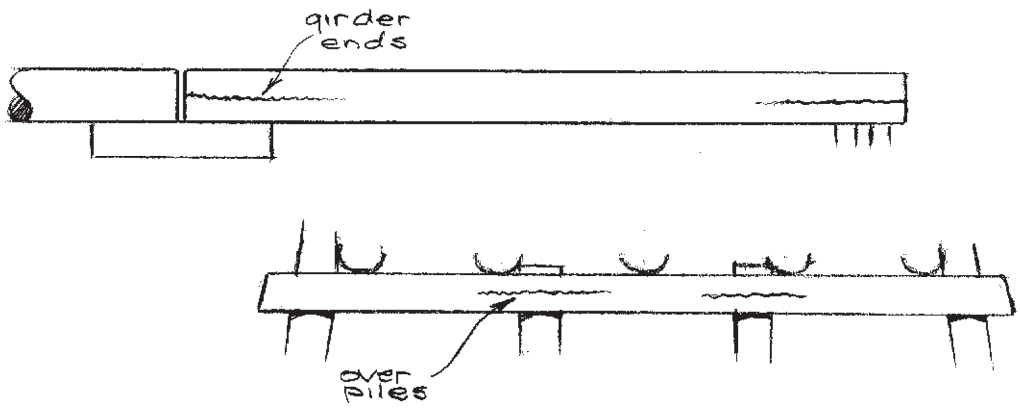


Figure 4.2(b) - Expected Shear Failure Locations

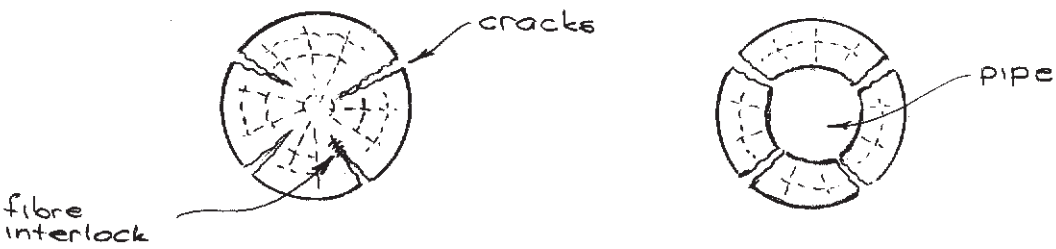


Figure 4.2(c) - Cracked Girder Sections

Headstock Failure

Girder Failure

Figure 4.2(d) - Component Failures

4.3 Timber Deterioration

Timber is subject to many processes of degradation, the most severe being:

(1) Fungal attack - rotting and decay
(2) Termite attack - loss of section
(3) Marine Organism attack - loss of section
(4) Corrosion of fasteners
(5) Shrinkage and Splitting
(6) Fire damage - loss of section
(7) Weathering

For a discussion of these deterioration mechanisms, refer to Article 1.4 of Part 2 of the Bridge Inspection Manual.

4.4 Timber Supplies

In the early days of timber construction, there appeared to be an inexhaustible supply of good quality, high strength hardwood timbers, and no thought was given to strategies to ensure everlasting supplies. Today, some 70 years later, with the depletion of forest reserves and the gradual closing down of forests to logging due to environmental and ecological reasons, there is now concern for the future. Even though the number of timber bridges continues to decline, it is very probable that such bridges will be in service for anther 30 to 40 years.

Main Roads typically requires girder logs which are 9m long and 480mm in diameter while piles could be 10 to 15m in length. Such timber sizes are becoming difficult to source, requiring much longer lead times for supply.

Discussions with the Department of Primary Industry on future forest management policies in Queensland indicate that timber for bridge girders and piles could come from either Crown Land native forests, private forests or hardwood plantations. For information, the expected situation is:

Crown Land Forests

Supply areas available are:

(a) South East Queensland (boundaries to Gladstone and Toowoomba)
(b) Western Area (Monto & west, Rockhampton & north)

South East Queensland Area

This area is covered by the South East Queensland Forests Agreement,(2001) which will phase out all forest logging over a 25 year period. During this time, restricted logging of timber will continue, with timber that is of girder quality initially being set aside for that purpose, with the remainder used for general saw log supplies.

For the first 5 years, DPI guaranteed a supply of girder sized logs at 2500 lineal metres per annum. However, only about 10-20% of these were suitable for Main Roads use. Beyond this period, no definite conclusions could be made, though it was thought that there is a high probability of being able to sustain this rate for the next 15 years.

Initially, girder timber was supplied to a restricted list of nine (9) purchasers with "Cut Timber" supplies supplied on an ad hoc basis to a larger set of purchasers. However, a new policy of non end purpose specified competitive sales is being introduced with all timber to be bid for on the open market, potentially increasing the difficulty in obtaining girder size material.

It should be noted that the SEQ Forests Agreement was driven by the existing timber industry and the environmental lobby, not by Government bodies. Consequently, Government Agencies are not stakeholders in the Agreement and, as such:

- Main Roads, Railways and Local Authorities will all be competing on the open market for the released girder supplies.
- Impact may be greater on Main Roads than Railways because of our need for generally longer girders.
- Any request for a firmer supply regime would only be possible through Ministerial involvement.

Western Area

DPI have started land use resource assessment for this area. It is noted that the scale of saw log reserves is similar to that of South East Queensland. Consequently, it is possible that 2500 linear metres per annum of girder materials may also be available.

There is the possibility of a future Forest Agreement for this area.

Private Forests

Under this process, local foremen from Main Roads (or a Local Authority), have arrangements with a local landowner to obtain suitable girder timber when this becomes available, for whatever reason.

Quantities harvested this way are not known, but DPI does not believe substantial quantities are involved.

Hardwood Plantations

Only selected species of hardwood will be grown (such as Spotted Gum - no Ironbark). However, in the nominated 25 year harvest cycle, no species will have grown to a size suitable for girder use.

Summary

In summary, total girder supply from Crown Land Forests for 0-10 years would be estimated at 5000 linear metres per annum (but with only the SEQ component considered to be reliable). Main Roads will be competing with Railways and Shires for these supplies. More ominously, all timber will soon be bid for on the open market, with no restrictions on end use.

Some on-going but more limited, supply from private forests will be available as well.

Guaranteed supplies of timber for headstocks, decking etc are also becoming more difficult. Because smaller section sizes are involved, the problem is not likely to be as severe, but the outcome will depend on yet to be completed forest management agreements.

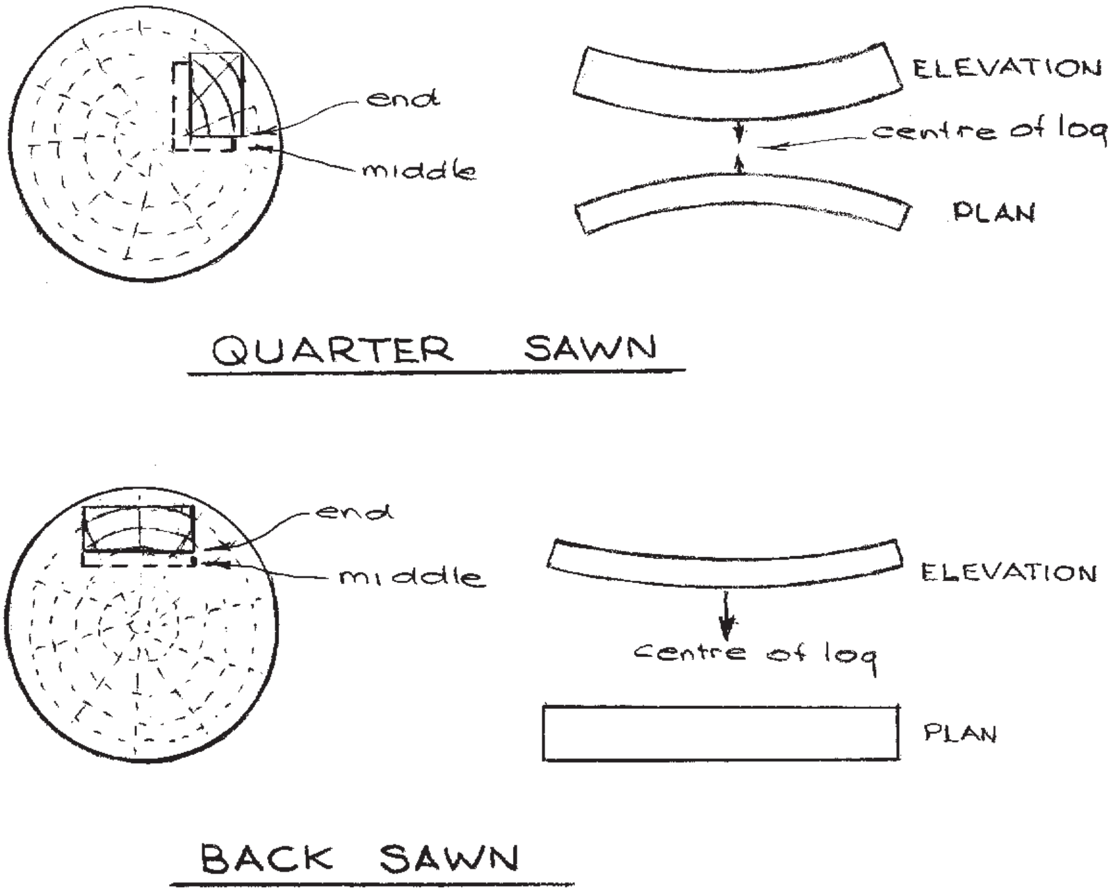
Refer to Article 6.0 for a discussion on various alternative materials being used or under investigation for future use to alleviate this problem.

4.5 Sawn Timber

Shrinkage occurs naturally in timber as a result of water loss from its constituent cells and this often produces unwanted side effects. Because of the non uniform structure of timber, the amount of shrinkage that occurs varies in different directions. The amount of shrinkage is highest in the tangential direction, then radially, with least shrinkage in the longitudinal direction of the log. Wood also exhibits an increasing rate of shrinkage the further it is from the heart. This results in various warping characteristics in sawn timber, depending on the location of the cut timber within the original log section. "Back sawn" and 'quarter sawn" are the normal types of cuts associated with sawn timber. Figure 4.5 illustrates these and the consequent warping distortions expected.

Warping, particularly in heavy section sawn timber such as headstocks can cause assembly and seating difficulties on site, and needs to be allowed for in the assembly process. Sawn decking timber is normally specified to be back sawn (no sideways bow) and laid with the heart side down. In this way the ends tend to curl upwards, and the act of spiking down the ends will keep the center region tight on the girders below.

Note that where the heart material is located at the center of a sawn member there will be no tendency to distort, though this heart material will likely reduce member durability. This situation will be most likely to arise with headstock members and will be discussed in Part 2.



Notes:

1. The ends of cut timber bend away from the centre of the log due to the rate of shrinkage increasing with distance from the heart.
2. Decking should be backsawn and placed heart side down (sapwood side up) i.e.

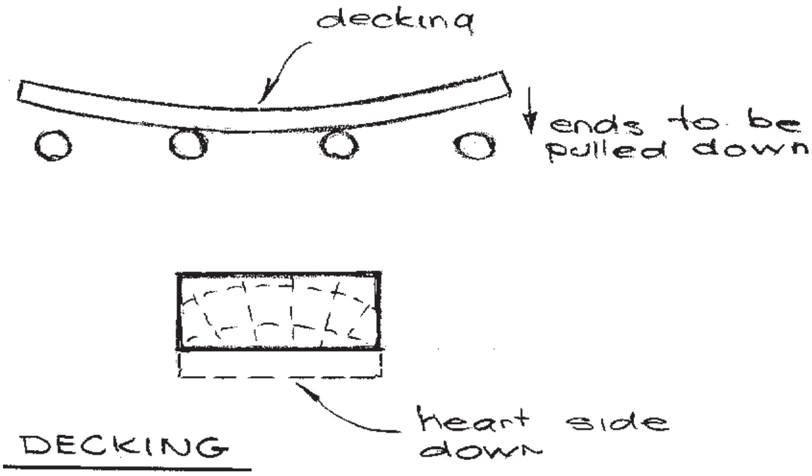


Figure 4.5 - Cuts For Sawn Timber

4.6 Girder Notching (Sniping)

One maintenance practice that has been identified as having the potential to adversely effect the strength of timber bridges is excessive depths of cuts made for girder and corbel seatings. Poor erection practice during maintenance replacement of these members has been suggested as one possible cause, where in the replacement of a single individual member was made easier by cutting out excessive quantities of timber to allow easy insertion and then packing added to restore line. This effect may also occur if a larger sized girder is substituted for an outer member but the spiking plank is retained.

Figure 4.6(a) shows examples of excessive member cuts.

Figure 4.6(b)(i) shows the seating cuts shown on the standard drawing for a 9.1m span, 5 girder bridge. Where the girder diameter remains the same, but the residual depth of timber is reduced, the shear capacity at the end of the girder will also reduce. Obviously bending capacity at the same location will also reduce, but this is not normally a problem near girder ends. Likewise, the bending capacity of reduced depth corbels will also be reduced. The shear and bending stresses based on actual measured depths are able to be calculated by the usual analysis methods

The main concern with large snipes in members is that there is a concentration of stresses occurring at the end of the notch, which increases the tendency for horizontal cracking to occur as shown in Figure 4.6(c). Notching not only reduces the area resisting bending and shear, but also, the resulting stress transfer around the notch involves forces perpendicular to the grain in the direction in which tensile strength of the timber is least. These forces, in conjunction with horizontal shear can cause splitting along the grain. Once a crack has formed at the notch, it is very likely to propagate along the member under heavily loaded situations, significantly reducing the ultimate capacity of the beam in bending because it is reduced into two separated sections, one of which is unsupported at the end - refer to Figure 4.6(b)(ii).

BAM has carried out a study of snipe effects in order to produce recommendations on acceptable depths of cuts at notches, but there are many uncertainties involved. Most well known publications including AS1720.1 warn against the effect of large snipe depths and recommend a flat slope at the end of the cut such as 1 in 4. In fact, using AS1720.1, the use of a 1:4 gradient theoretically increases the capacity of a girder in shear by approximately 3 times that of a "square" notch. However, most references are based on rectangular sections, and do not cover round or octagonal members as occurs in all our bridges.

As an example of the perceived critical requirement to keep notches as shallow as possible, previous common practice required the shear capacity of a section (based on actual residual depth) to be further reduced by the ratio (residual depth/unnotched depth). This would mean that a notch to half the depth of a member could reduce its shear capacity to ¼ of unnotched capacity.

Girder - Excessive Snipe

Corbel - Excessive Snipe

Figure 4.6(a) - Excessive Notching

1

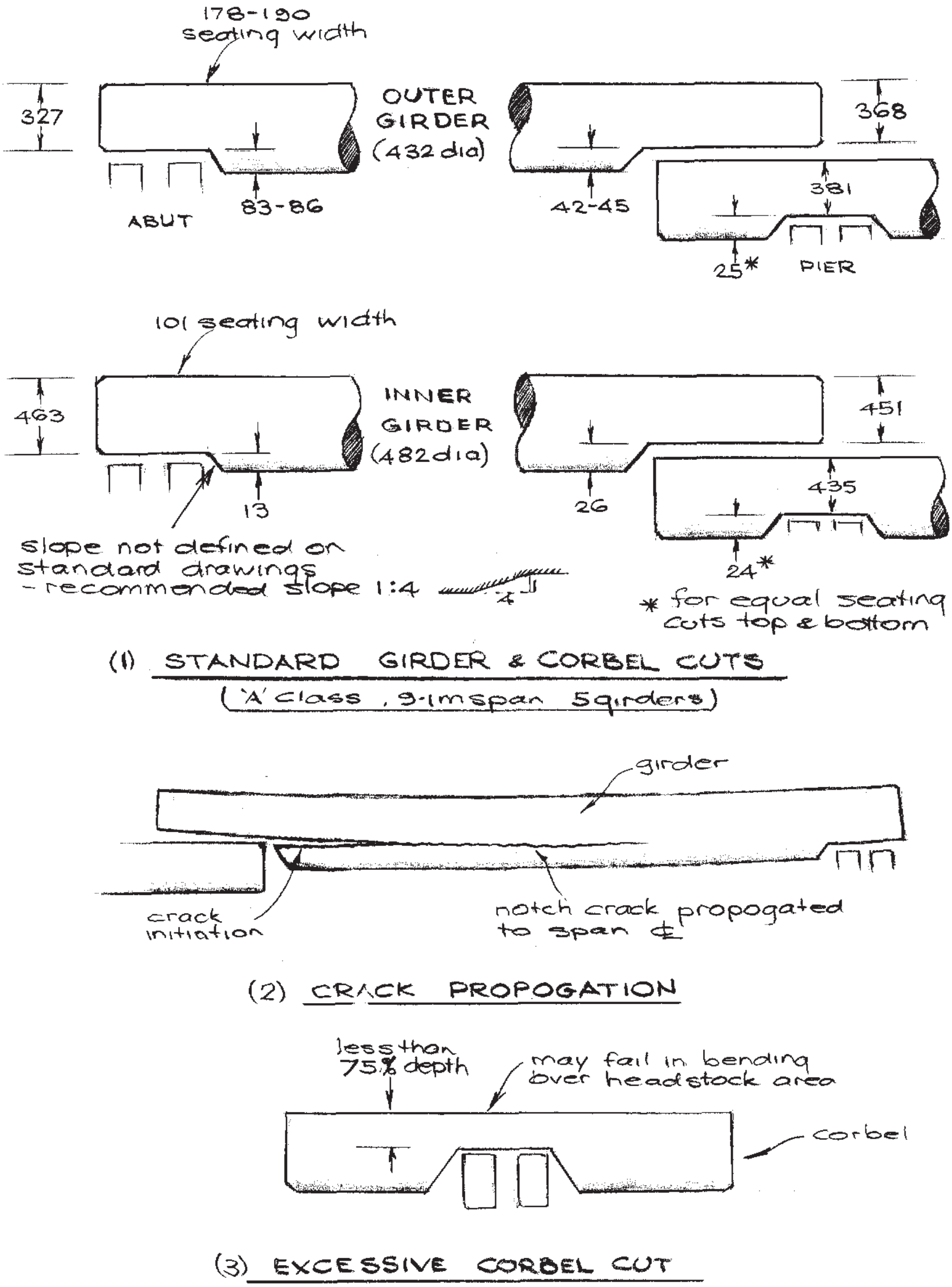


Figure 4.6(b) - Girder Notching

Figure 4.6(c) - Notch Cracking

To fully comprehend the situation, BAM is carrying out tests on full size notched beams to determine true effect on shear capacity. In the mean time, classification of girders during inspection into relevant condition states based on % consumption requirements is

required, as it brings this potential problem to attention. It should be noted that the BIM now requires pipe size to be added to snipe depth in determining condition state.

The situation is possibly a little less severe at corbelled girder ends if the system is tight, as continuity developed over the pier will have the effect of reducing the level of longitudinal bending stress in the girder soffit at the snipe location. However, this effect cannot be relied on because of inevitable loss of bolt tightness at corbels, while abutment ends of girders will always have the same level of longitudinal stress

The situation with corbels is less onerous because longitudinal stresses at the headstock support notch are in compression, reducing the potential for cracking compared to a tensile force situation. However excessive depths of cuts in the corbel may reduce its bending strength sufficiently that it is unable to support loads from the girders and it may fail in bending. Refer to Figure 4.6(b)(iii).

As an interim measure, BAM Advice Note No 23 has been issued with recommendations on treatment of snipe depths for girders. This is based on various findings given in a number published research papers. Refer to this Note for an extensive list of reference on this matter.

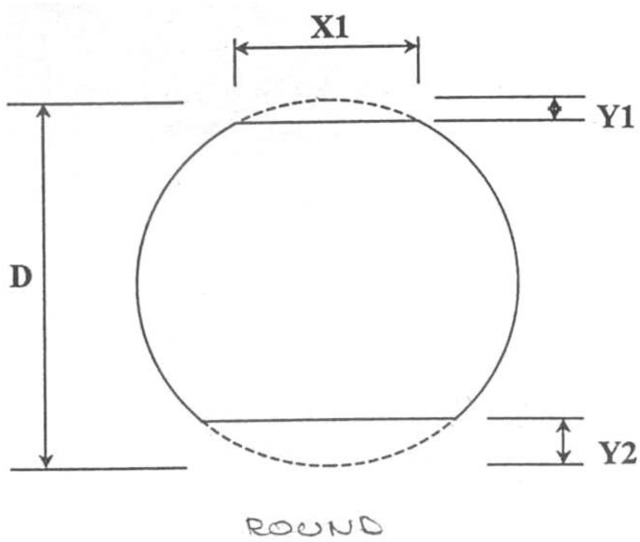
Essentially it is proposed that sniped members with a maximum loss of section area of 10% will be acceptable. For a greater reduction, up to 25% maximum, it is recommended that strengthening behind the sniped area be applied in order to reduce splitting potential. Above this loss, the sniped member should be replaced. As an approximation, the above limits are about 15% and 29% of girder depth after the top seating is removed.

Figure 4.6(d) & (e) shows recommended treatments for various girder diameters and snipe depths. Strengthening, if required, should be carried out using anti-splitting bolts as shown in Figure 8.1(f). (Part 2).

Further to the above, one anomalous situation may arise if a considerably larger size of girder is substituted for the original standard size.

If, after potential splitting as shown in Figure 4.6(b)(ii), the upper residual section of girder still satisfies the original girder section requirements, then the situation is not critical. Provided there is sufficient fastening between corbel and girder and the failure plane is not inclined, BAM will base girder assessment on the cracked section if notching depths are outside recommendations.

Refer also to Section 8.1 (Part 2) for a more detailed discussion of snipes to timber girders.



Inner Girder

X1 = 4" (101 mm)

D		Y1	Y2		
inch	mm	mm	mm No remedial work required	mm Anti splitting bolts	mm Replace
16	406	6	≤ 62	63 < Y2 ≤ 121	> 121
17	431	6	≤ 67	67 < Y2 ≤ 128	> 128
18	457	6	≤ 71	71 < 72 ≤ 136	> 136
19	482	5	≤ 75	75 < Y2 ≤ 144	> 144

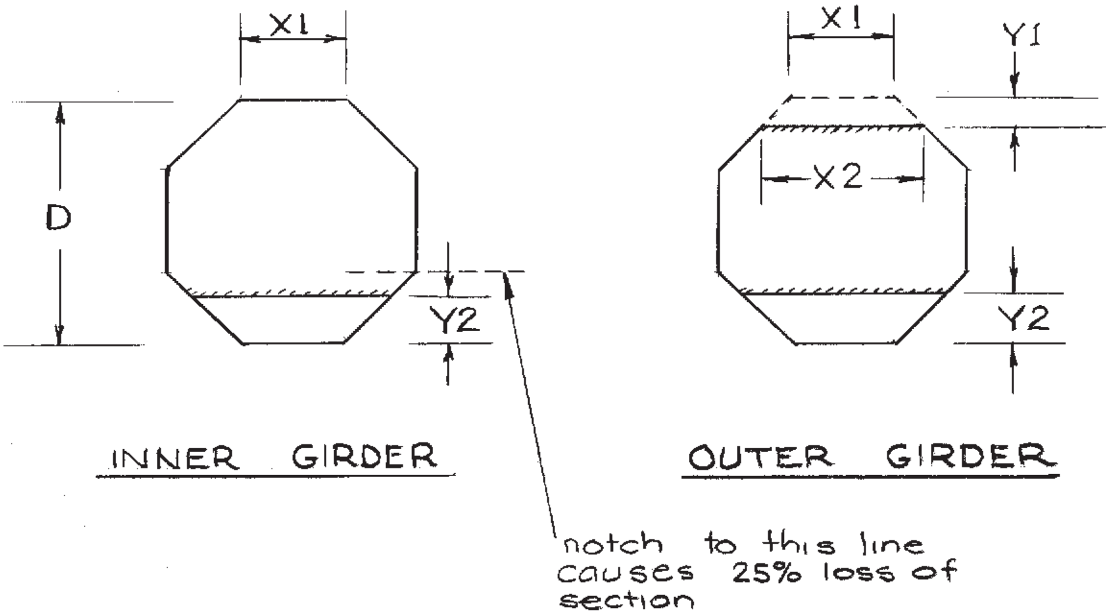
Outer Girder

X1 = 7 - 7 ½" (178 - 190 mm)

D		Y1	Y2		
inch	mm	mm	mm No remedial work required	mm Anti splitting bolts	mm Replace
14	355	31	≤ 54	54 < Y2 ≤ 103	> 103
15	381	29	≤ 58	58 < Y2 ≤ 111	> 111
16	406	27	≤ 62	62 < Y2 ≤ 119	> 119
17	431	25	≤ 66	65 < Y2 ≤ 126	> 126

Figure 4.6(d) - Notching Limits - Round Girders

1



Inner Girder

D		X1	Y2 Maximum Snipe		
Ins	mm	mm	mm Acceptable	mm Strengthen	mm Replace
16	406	168	Up to 60	>60 & <119	Over 119
17	431	179	64	>64 & <126	126
18	457	189	67	>67 & <134	134
19	482	482	71	>71 & <141	141

Outer Girder

D		Y1	Top Cut Y1	X2	Y2 Maximum Snipe		
inch	mm	mm	mm	mm	mm Acceptable	mm Strengthen	mm Replace
14	355	147	21	190	Up to 51	>51 & <102	Over 102
15	381	158	16	190	55	>55 & <110	110
16	406	168	11	190	60	>59 & <118	119
17	431	179	-	179	64	>64 & <126	126

Figure 4.6(e) - Notching Limits - Octagonal Girders

4.7 Girder Testing

In the assessment of girder adequacy for on-going service, there are a number of uncertainties caused by deterioration effects. For example, the determination of crushing loads at the end of piped girders, or the actual effects of pre-existing longitudinal cracks in the ends of girders on the shear capacity.

If adequate numbers of load tests could be carried out on members of varying size and level of deterioration, it may be possible to formulate procedures to ascertain capacities with confidence. This will provide improved credibility with assessments and help lower liability concerns.

No past records appear to be available of any load tests carried out by Main Roads on timber components such as bridge girders, in order to correlate design expectations with actual failure loads. In recent years, the Queensland University of Technology has carried out a number of tests on Main Roads sourced girders as part of thesis work. Some members were initially tested with simple supports at girder ends, while later tests modeled the effect of corbels at one end. Girder condition ranged from very poor due to end cracking and piping to sound, giving a large range of recorded capacities. Tests were also carried out to gauge the effect of bolt tightness on corbel/girder connections. Load testing of a small number of bridges in service was also carried out.

BAM has formulated a timber bridge strategy and as part of this initiative has participated in carrying out load tests on an in-service bridge (Bremer River), using a variety of test vehicles, vehicle speeds and locations in order to obtain an understanding of actual timber bridge actions.

As part of associated research work, laboratory testing to failure of a number of used girders both from the Bremer River bridge and other sites has commenced. Testing of girders to help formulate acceptable snipe treatments is also part of this research work.

Refer to References 2 to 10 for further information on results obtained from the QUT and BAM testing.

References

2. Kendall, M. (1997). Health Monitoring of Timber Bridges. Undergraduate Thesis, Queensland University of Technology.
3. Deane, S. (1997). Health Monitoring of Timber Bridges. Undergraduate Thesis, Queensland University of Technology.
4. Bowden, M. (1998). Health Monitoring of Timber Bridges. Undergraduate Thesis, Queensland University of Technology.
5. Leja, M.P. (1998). Health Monitoring of Timber Bridges. Undergraduate Thesis. Queensland University of Technology.
6. Rossow, S. (1998). Health Monitoring of Timber Bridges.
7. Dunne, T. (1998). An Investigation into the Corbel Behaviour and Failure Mechanisms of Timber Bridge Girders. Undergraduate Thesis, Queensland University of Technology.
8. Green, C. D. (1998). An Investigation into the Corbel Behaviour and Failure Mechanisms of Timber Bridge Girders. Under graduate Thesis, Queensland University of Technology.

9. Harris, A. D. (1998). An investigation into the Corbel Behaviour and Failure Mechanism of Timber Bridge Girders. Undergraduate Thesis, Queensland University of Technology.
10. Bridge Asset Management (2003). Capacity of Bremer River Timber Bridge. Department of Main Roads.

5.0 Timber Design

5.1 General

A knowledge of timber design methods will be of use when determining sizes of components during maintenance repairs, but is essential for those charged with timber bridge capacity determination and consequently structure and public safety.

As a guide to further understanding, this section provides a historical record of the various design processes used by DMR Bridge Branch over the period of timber bridge construction (1922 to 1960's) and up to the present time.

Since the 1960's, the main focus has been on determining the load capacity of existing bridges, both under general traffic access and for heavy load permit travel. At the present time, BAM is undertaking a testing programme both on individual girder elements and complete in-service structures (Refer Reference 4). This will certainly add to our knowledge of how timber bridges function and hopefully reinforce our analysis procedures. Design concepts disscussed here concentrate on primary structural members such as girders, headstocks and piles, the collapse of which may have catastrophic results.

It should be noted that building design in Australia is carried out to the requirements of various Standards published by Standards Australia. However, this is not generally the case for bridges under the control of State Road Authorities. Since 1953, all bridge design has been carried out to the requirements of Bridge Design Codes (BDC's) which are published by a national association body formed by the various State and Territory Road Authorities. Today, an anomalous situation occurs because:-

(a) The last BDC to provide requirements for timber design was dated 1976.

(b) The current BDC, dated 1992 provides no guidance for timber design.

Furthermore, in 1992, the fundamental design philosophy changed from "working stress" design to "limit-state design". In working stress design, a failure or yield stress of a material is reduced by a "factor of safety" (a number which may be 2 to 4) in order to obtain an allowable stress and this is compared to the effect caused by actual loads.

In limit-state design a failure or yield stress of a material is reduced by a "capacity reduction factor" to allow for material variability and consequence of failure and this is compared to loads which are increased by a "load factor" (a number 1.2 to 2.0).

Because there has been no guidance given by AUSTROADS, each State Road Authority has had to adopt its own procedures for timber design.

5.2 Historical

Details and member sizes for DMR timber bridges were determined in the 1920's and essentially no changes have been made to these in the subsequent 80 years of use. Our knowledge of early design methods is based on MRC form 633, dated 1942 (Figure 5.2) which gave the following parameters for design:-

(1) Allowable bending stress in timber = 3000 lbs / in2 (20.7 MPa)

(2) 25% impact to be added to live load

(3) One line of wheels distributed to one girder

Interestingly, this Form states that "only inferior timber fails at less the 14000 lb/in2" (96.5 MPa), which equates to a factor of safety of 4.6 on bending stresses. However, testing of old girders taken out of service indicates a loss of ultimate strength with age (not related to rotting defects etc.) Consequently, in practice, there will be a reduced factor of safety in old existing girders.

The vehicles used to design DMR timber bridges were 'A' & 'B' Class vehicles, and no conventional timber bridge has been designed for the subsequently heavier bridge design vehicles. A check of various timber designs also indicates that the distribution method given in (3) above is only applicable to inner girders of 4 & 5 girder bridges.

Pre 1976, the relevant BDC's and timber references were:-

Bridge Code	Year	Timber Design
COSRA	1953	Each State Authority to follow existing practices
COSRA	1958	Each State Authority to follow existing practices
NAASRA	1965	No reference
NAASRA	1970	Generally follow CSIRO Handbook"Timber Engineering Design Handbook"

WORKING STRESS ANALYSIS:

In 1976, the NAASRA Bridge Design code was released, and this referred directly to the then Australian Standard AS1720/1975 for timber design.

Section 2 of this BDC now excluded impact allowance on live load for timber structures, and some changes were made to the AS1720 requirements essentially achieving the same member sizes as previously obtained using earlier in-house methods. The changes made in Section 10 - Timber Structures were:-

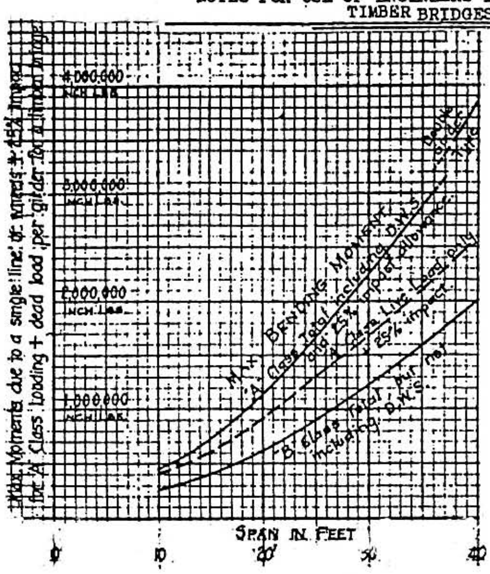
1. Time of duration constant to be 1.0 rather than 1.4 as required by Table 2.4.1.1 of AS1720 (using 5 months duration of loading)

2. Stress grades given in Table 6.2 of AS1720 for round timbers to be dropped one line, i.e. stress grades lowered for a particular strength group.

MAIN ROADS COMMISSION

NOTES FOR USE OF ENGINEERS IN THE FIELD WHEN EXAMINING TIMBER BRIDGES.

D.A. CRAWFORD
CHIEF ENGINEER
DATE: 8.5.42.



SECTION MODULUS ROUND BEAMS

DIAMETER INCHES	SECTION MODULUS
10	98
11	131
12	170
13	216
14	269
15	331
16	402
17	482
18	573
19	673

The Modulus of Section for a Rectangular beam is:

$$\frac{\text{Breadth x Depth}^2}{6}$$

Maximum fibre stress is:

B.M. (Max) $\div$ Section Modulus

"A" Class loading is a 15-ton Tractor followed by 5 ton Axles, all axles at 10 feet centres.

"B" Class is an 8 ton Tractor followed by trailers weighing $5\frac{1}{3}$ tons.

The above moments are those due to one line of wheels, and assume that the moments in the girders do not exceed those caused by one such line. Actually, if the decking is sound and well fitting, the moments on the girders may be less than those due to one line of wheels. It is possible however, in a wide bridge with few girders, for moments to occur in a girder exceeding those due to one line of wheels, for example, a 3-girder bridge more than 15' wide.

MAXIMUM FIBRE STRESS

A maximum fibre stress resisting bending of 3000 lbs. per sq.inch is used in design. For occasional loading, a fibre stress of 5000 or even 6000 lbs. per sq.inch would be safe, provided the timber was sound hardwood and not shewing excessive longitudinal cracking, and impact was minimised. Tests shew that only inferior hardwood fails at less than 14000 lbs. per sq.inch in bending.

For bearing, a fibre stress of 1000 lbs. per sq.inch across the fibres is used.

DECKING

The wheel load can be assumed to be distributed over the width of the wheel, which may be a dual pneumatic tyre. The moments due to such a load will be reduced if the girders are spaced to give adequate multi-point support. Example: Clear span between girders 52"
5 ton wheel 18" wide + 25% (Impact for pneumatic tyre) = 14000 lbs.

$$B.M. = \frac{14000}{2}\left\{\frac{52}{2} - \frac{18}{4}\right\}$$ (multiplied by say 0.7 or 0.8 according to continuity of deck plank over several supports.)

HEADSTOCKS

The strength of Headstocks may be calculated from the following table of GIRDER REACTIONS, due to Dead Load (incl.D.W.S.) and a train of "A" Class or "B" Class loading. Single 15-ton vehicles would reduce "A" Class Reactions by 10%, 20% and 27% for 20', 30' and 40' spans respectively, (Impact is allowed only on one wheel).

"A" CLASS REACTIONS			"B" CLASS REACTIONS		
SPAN	ABUTMENT	PIER	SPAN	ABUTMENT	PIER
40	31,000 lbs.	48,000 lbs.	40	19,500 lbs.	31,500 lbs.
35	28,000 lbs.	42,000 lbs.	35	17,700 lbs.	27,900 lbs.
30	25,500 lbs.	37,000 lbs.	30	15,700 lbs.	23,900 lbs.
25	23,000 lbs.	32,500 lbs.	25	14,100 lbs.	20,700 lbs.
20	20,500 lbs.	27,000 lbs.	20	12,200 lbs.	16,900 lbs.
15	18,500 lbs.	23,000 lbs.	15	10,800 lbs.	14,100 lbs.

DEAD LOAD

A timber bridge weighs from 200 lbs. to 250 lbs. per foot per girder, and 3"deck wearing surface 150 lbs.per foot ($4^1/2$ feet wide).

The moments depend on distance between piles, and are calculated as follows:-

$$\text{Moment} = \frac{ab}{a+b} \times \text{Reaction}$$

where a, b are the distances, in inches, to the centres of the nearest pile supports on each side of the Girder.

This moment may safely be reduced by 40% for interior sections of headstock where 4 or more piles are used, or by 20% where 3 piles are used, to allow for continuity, if the supports are all effective. Remember, however, the negative moment over the piles is as great as, or greater than the positive moment under the girders.

GENERAL

Piers of considerable height must be effectively braced. A pile of short length may safely carry an end load of 1 ton per sq.inch, but if the unsupported length exceeds 20 times the diameter this must be reduced. The safe end load for a length 30 times the diameter is only $\frac{1}{2}$ ton per sq.inch, depending on the straightness, and for greater lengths the safe end load is affected so much by eccentricity of loading that strength is problematical.

In examining old bridges, dimensions of all members with spacing of girders, piles and decking is necessary with description as to condition.

Figure 5.2 - MRC Form 633

Calibrations carried out by Bridge Branch indicated that for strength Group 1 timbers, at least, the previously used allowable stresses would be obtained by the new BDC requirements.

Consequently, a working stress allowable bending stress of 20.7 MPa was adopted for on-going use.

Section 11 of NAASRA 1976 BDC also increased allowable stresses by 50% for overload vehicle analysis, i.e. to 31 MPa.

The concept of AS1720 / 75 was to take a basic allowed bending stress for timber and modify this by a series of multiplying factors for various loading and environmental factors in order to obtain an allowable bending stress i.e.:-

Fb = k1 x k4 x k5 x k6 x k8 x k11 x k12 x F'b

Where	k1	=	Duration of load Factor
	k5	=	F' unseasoned / F' seasoned
	k6	=	Temperature Factor
	k8	=	Load Sharing Factor
	k11	=	Size Factor
	k12	=	Slenderness Factor
	F'b	=	Basic allowable stress based on F grade number of timber.

A further reduction factor also applied to round timber if it was shaved (to remove sapwood).

A number of changes where subsequently made to AS1720:-

(1) In 1988 a modified shaving factor was introduced

(2) In 1993, an amendment introduced a further modifying factor k2 which further reduced the allowable stress depending on the consequences of failure of a member.

As well, in 1992 the new limit State Bridge Design Code did not specifically exclude the application of impact to timber Structures.

However it is believed that the application of all these post 1976 changes cannot be applied to the NAASRA 76 BDC requirements without revisiting the associated original modifications made to AS1720 requirements at the time of development of the Bridge Design Code, i.e. the ignoring of the time of duration constant and the lowering of the stress grades for round timber.

5.3 Current DMR Approach (working stress)

Because of the lack of advice from AUSTROADS on timber design, DMR has basically retained the concept of using the NAASRA 76 code requirements. Using working stress analysis, allowable bending stresses of 20.7 MPa for normal loads are used while for overload permit analysis, 31 MPa is allowed.

Unless a very specialised and detailed analysis of a structure is required, a simplified model of a timber bridge is generally used. The complex interaction between corbels and girders which may lead to some structural continuity across piers is generally not considered and simply supported spans assumed. Distribution of wheel loads into the various lines of girders is generally carried out using earlier grillage analysis work.

Other considerations (common to all methods) are:-

PROPERTIES

(1) *Member Size*

Inspection often find girder diameters to be less than specified by the drawings, possibly the result of incorrect ordering, or in general due to shrinkage of the member in service. Actual measured sizes from inspections are to be used for analysis. This may also lead to a reclassification of Bridge Class, generally to a lower class.

(2) *Timber Strength*

Recent species testing of existing bridge girders has found all timber to be Strength Group 1 or 2, but with the majority to the lower strength S2. This is because of the increasing reliance on lower strength timbers such as spotted gum or narrow leaf red ironbark for girder replacements. The supply of new girder timber is now controlled by the requirements of MRS 11.87 which requires a minimum of F27 for girder timber. Though achievable with round timber, for some time now, replacement girders have been supplied as sawn octagonal members. Table 3 of MRS11.87 allows the use of nominal F22 timber with the proviso of no noticeable defects in the middle third of the girder length, effectively achieving F27 stress grade where Strength Group 2 sawn timber is used. As DMR timber supply specifications have always contained sufficiently onerous requirements to achieve the above result for sawn timber it is believed that F27 stress grade can be assumed for both round and sawn girders analysis. Where a definite determination of species or F grade is required, this service can be carried out by DPI, Queensland Forestry Research Institute, Indooroopilly.

5.4 Limit State Analysis

As an alternative to the above approach, timber analysis based on using the current Australian Standard AS1720-1997 is considered appropriate. Loads, load factor and dynamic load allowance shall be taken from the current AUSTROADS BDC. Both these documents are compatible as they are written in Limit State format.

BAM is currently investigating the most appropriate analysis method for timber in order to determine a DMR policy on the matter.

As previously indicated, the Limit State Design format of AS1720 requires member capacity to be based on use of a capacity factory (f), characteristic material strength (f'b), the section property and various multiplying constants for the timber material.

The effective "capacity" of a member so determined is then compared to the effects of applied loads which have been increased by multiplying load factors.

A typical bending capacity formula for a *round girder* takes the form :-

$\phi M = \phi$ k1 k4 k6 k9 k12 k20 k21 k22 [F'b Z]

Where k1, k4, k6, k12 are as for working stress design &

k9 = Strength Sharing Factor

k20 = Immaturity Factor

k21 = Shaving Factor

k22 = Processing Factor = 1

While for *sawn (octagonal) girders* becomes:-

$\phi M = \phi$ k1 k4 k6 k9 k12 [F'b Z]

Some guidance on the use of these various factors is given below:-

Capacity Reduction Factor (f)

This factor is related to both the variability of the material being used and to the significance of the member i.e. consequence of failure.

Timber has a high property variability and would generally only be visually stress graded. The appropriate f values given in AS1720 then are:-

- 0.65 for primary structural members. (girders, headstocks, piles)
- 0.80 for secondary structural members. (decking, bracing, corbels)

Note that for concrete structures, f may vary from 0.6 to 0.8, while for steel f is typically 0.9.

Duration of Load (k1)

Timber is known to exhibit a higher strength for short durations of loading compared to long term or permanent loads. In addition, the effects of short term loads are believed to be cumulative.

For the effect of live loads, a medium duration of loading of 5 months is specified, giving a k1 value of 0.80 for instantaneous loads. Dead loads are generally small compared to live loads, but being permanent require a k1 value of 0.57.

Engineering judgment and extrapolation of these values would be needed when considering other loads such as flood and wind.

Moisture Condition (k4)

Generally the timber sizes used in bridges are large (> 100mm) and so can be considered to remain in an unseasoned state. Seasoned timber will only be found in stress-laminated timber bridges.

For unseasoned timber, k4 is specified as 1.0.

Temperature (k6)

For unseasoned timber a value of 1.0 for k6 is appropriate to all areas of Queensland.

Strength Sharing (k9)

This factor is applicable to structural systems with multiple discrete components such as girders & decks, headstocks and piles which combine to support a load. Another requirement is that transverse members are able to transfer load to the other components in the event of failure of one of the components.

The principal behind this concept is that the characteristic design strengths calculated for members are at the low end of the strength spectrum (design values effectively assume 95% of members have a higher actual strength than that used). Consequently, if one sharing component is removed, it is expected that there will still be a global adequate strength in the remaining members.

Detailed methods of calculating k9, depending on actual component configurations are given in AS1720. - Section 2.4.5, defining kg as:-

$$k9 = g_{31} + (g_{32} - g_{31})[1 - \frac{2S}{L}] \text{ but } \ni 1.0$$

A conventional timber superstructure forms a "discrete parallel system and the following may be taken for girder consideration:-

- ^{n}com = 1 $\therefore$ g31 = 1
- ^{n}mem = number of girders
- From Table 2.11 of AS1720, g32 = 1.24 & 1.26 for 4 and 5 girder bridges respectively.
- S = girder spacing.
- L = effective span of girders.

k9 becomes 1.169 and 1.177 for 4 and 5 girder 9.1m span bridges respectively.

Stability Factor (k12)

This is related to slenderness effects in members. Detailed information is supplied in AS1720 for derivation of the factor k12.

Immaturity Factor (k20)

This effects only small diameter round timber and a factor of 1.0 is appropriate.

Shaving Factor (k21)

Where the outer layers of a round log are disturbed, i.e. where sapwood is trimmed off a round girder, a value of 0.85 is applicable for k21.

If only minimal trimming (bark etc. only) has occurred a value of 1.0 is appropriate.

Characteristic Strength (f'b)

In the design and evaluation of timber components, a characteristic strength which is related to an F grade of timber is used. As discussed in Section 5.3, all new timber is now ordered to this requirement using MRS 11.87. As well for existing girder timbers, the relevant F grade has generally been assumed based on specification strength group / species type. It is also possible for a visual stress grading to be carried out by appropriate experts, where a determination is essential. Contact DPI, Queensland Forestry Research Institute, Indooroopilly.

Table 3 of MRS 11.87 lists the minimum stress grades required for the various components in timber bridges - applicable to new components.

Table 4 of MRS 11.87 is listed here to illustrate the basis different approach between round timber design and sawn timber design (such as octagonal girders). Sawn timber strengths are based on a Structural Grade Number (related to the quantity and severity of defects). F numbers are lower than for round timber, but round timber requires a shaving factor if substantial trimming such as de-sapping occurs.

Relationship between Strength Groups and Stress Grades (unseasoned)

Strength Group	Stress Grade			
	No. 1 Structural	No. 2 Structural	No. 3 Structural	Round Timber (Piles, Girders)
S1	F27	F22	F17	F34
S2	F22	F17	F14	F27
S3	F17	F14	F11	F22

Note that for round timbers, Table 6.1 of AS1720 lists the correlation between strength group and stress grade.

For pile design, capacity formulae using k constraints similar to those for round girders apply.

For headstock design, a typical bending capacity formula takes the form:

$\phi M = \phi\ k1\ k4\ k6\ k9\ k11\ k12\ [F'6Z]$

Where k11 is a size factor which = 1 for our Standard headstock sizes.

Analysis

Timber bridges rely on bolted connections to maintain integrity. However, a varying degree of looseness will normally be present.

Analysis modeling may include continuity effects such as provided by tight corbels. However, the effect of loose connections should always be considered in the analysis, generally resulting in simple support conditions as one option.

As previously noted, the BAM approach is to generally assume loose corbel connections resulting in simply supported spans between adjacent headstocks. For distribution of wheel loads to girders, BAM normally uses influence lines which were originally developed from grillage analyses for various standard bridge widths and spans. Non-standard layouts will normally require a specific grillage analysis.

6.0 Alternative Materials / Systems

6.1 General

1

It is necessary that in the event that timber supply problems become critical, that viable alternative materials are available Materials with a guaranteed on-going availability such as steel, are being examined by Structures Division to develop acceptable alternatives. In past maintenance works, the most cost effective replacement material has been timber, but as supplies dwindle and supply costs consequently escalate, it may also be more appropriate to use other alternative forms of construction. The following describes the various alternative materials that have been used or are being considered.

6.2 Girders

The most promising alternative material for girders would appear to be steel. In the past, steel girders have had a very limited use in our timber structures both in new bridges (with timber headstocks) and as random replacement members in existing structures. Because of different stiffness characteristics between steel and timber, steel sections may not be structurally suitable for limited individual substitution because of possible unwanted changes to load distribution and load path. However, when combined with a suitable deck system, steel girders should be suitable for full span replacements. Because of their light weight, it is unlikely that significant changes to substructure would be required.

An engineered experimental re-decking for timber bridges by using steel girders with a plywood decking has been developed in order to determine suitability for maintenance replacement work.

BAM is also investigating the use of gluelam technology to produce glued hardwood girders as replacements for individual girders.

In general, alternative systems such as fibre composites, gluelam timber or stress -laminated decks may reduce superstructure depth, particularly if corbels are eliminated, requiring packers on top of substructure to retain deck levels. For a typical alternative steel girder detail refer to Figure 8.2 (Part 2).

6.3 Decking

There is already considerable use of alternative forms of decking such as steel trough or thick structural plywood, both of which allow widening of decks to be carried out by cantilevering the decking past the outer girders. It should be noted that where widening is provided, that the outer girders should be replaced with larger girders of the same diameter as the inners. It is important to check the capacity of girders and substructure where a deck is widened because of possible increased deck mass and potential for an extra lane of traffic.

6.3.1 Steel Trough

Steel trough decking has been used since 1970 and in most cases is placed transversely to the girders. If longitudinally placed troughing is used, additional transverse steel cross beams are required. Performance of steel decks has been found to depend on the type of infill material used. Where bituminous AC infill is placed, there is a high probability that significant defects will develop. The most common types of deterioration consist of rusting of the top of the troughing leading to perforation of the metal and possible buckling and collapse, while transverse troughing often develops cracking across troughing sheets both over and between girders. Where concrete infill has been used, however, there appears to be no reports of similar faults developing. It is recommended that only concrete infill, with reinforcing, be used for steel trough installations. Provision must be made to allow later replacement of girders by blocking out over and around attachment bolts.

6.3.2 Plywood

Plywood decking either 130 or 155mm nominal thickness has been used since 1980. AC surfacing is generally used to provide a drainage surface and to protect the soft timber surface from traffic abrasion. The main defect noted has been the development of cracks in the AC over the butt joint between ply sheets, often resulting in a wide crack as a result of ejection of a wedge of AC. This problem is caused by differential deflection of the edges of the adjacent ply sheets during wheel passage. It is recommended that only the thicker 155mm thick ply sheet be used and a longitudinal soffit distributor running across joints be used between each girder to try and alleviate this problem. Other methods of reducing joint differential movements are being examined.

6.3.3 Prestressed Concrete

A small number of bridges have also been redecked with precast, prestressed concrete transversely placed deck planks, which are commercially produced specifically for placement on timber girders. They are placed without deck wearing surface and with gaps to allow surface drainage. In addition, longitudinal steel distributors are bolted to the slab soffits between girders. The planks are also very rigid in comparison to timber systems and require good seatings to prevent shear and torsion problems being introduced. The system works well on square bridges, but in the absence of proven use on skewed structures are not recommended in this situation. The main problem with skewed structures and the transversely placed planks is twisting over the tops of piers due to opposite rotation directions in adjacent spans under traffic loading. For details of alternative deckings, refer to, Figures 5.2, 5.4 & 5.5. (Part 2)

6.4 Headstocks

Heavy steel channel sections have been used in a number of bridges as replacement members for sawn timber headstocks. Additional plates are added to the top flange to provide adequate bearing contact are for corbels or girders.

Plywood headstocks are known to have been placed on one structure, but long term performance is unknown due to road demaining. Ply usage is not recommended at this stage until further investigations are carried out. For typical details of steel headstocks refer to Figures 10.2(a), (b), (c) & (d). (Part 2)

6.5 Piles

Steel piles, generally of universal pile or beam sections, have been used successfully in several bridges for replacement of timber piles. Steel brackets or members are used to attach timber headstocks or to modify these to allow driving of the new pile section. For typical details refer to Figure 11.2. (Part 2)

6.6 Bracing

Again steel sections have been used in the past to replace bracing and wale members. They are particularly compatible with full pile replacement using steel piles. For typical details refer to Figure 12.2. (Part 2)

6.7 Stress - Laminated Timber (Deck & Girder replacement)

Another possible alternative is full superstructure replacement using stress-laminated timber technology where timber boards are stressed together by prestressing bars to form a slab. One experimental bridge has been constructed using imported treated Douglas fir to form a solid slab. On-going monitoring of bar forces and structure condition is being carried out to determine long term functionality. In this bridge, the original girders, corbels and decking have been replaced by a slab, made continuous over the 3 spans. No changes to substructure were made except for concrete packers on top of the headstocks because of the thinner overall depth. Refer to Part 2, Figures 5.9(a) & (b) for details.

Construction costs for the stress - laminated timber superstructure when built showed no economic advantage over conventional maintenance replacement works. However, as conventional hardwood supplies diminish, with consequent increases in supply cost, this system may become more attractive.

Though the RTA have built a number of stress-laminated bridges using hardwood boards, our future supplies will be limited generally to hardwood plantation timber such as spotted gum or western white gum which will limit board sizes. Consequently, any further stress-laminated bridges built in Queensland are likely to be from Australian plantation grown pine or hardwood material. Due to limitation in board sizes, a hollow box form of construction will be required as shown in Figure 5.9(a), (Part 2). One advantage with this system is that ½ width spans can be built elsewhere, transported to site and assembled in place. Because the timber is treated, a long useable life is claimed for stress-laminated construction, with suggestions of 30 to 40 years, though the oldest such construction in Australia is still only about 10 years old. This technology appears to be essentially limited to square geometry bridges, and is used for full span superstructure replacement.

Stress-laminated technology has also been used elsewhere to produce thin timber decks which are used to replace existing decks on steel girders. However, the use of plywood for this purpose would still be the cheapest option available, with the materials for ply manufacture having an assured supply availability.

6.8 Proprietary Products

There are a number of commercially produced concrete or concrete/timber composite girder systems which are marketed for timber superstructure replacement. No formal study of the performance of these types has been carried out as yet, but it would appear that some substructure strengthening and support detail changes would generally be necessary because of the resultant increase in superstructure mass.

6.9 Fibre Composites

Development work on fibre-composite (plastic) bridges is also being carried out by the University of Southern Queensland, in conjunction with Main Roads. In the long term, this technology may also provide a viable girder replacement system.

7.0 Timber Preservation / Practices

7.1 Historical Treatments

Because of timber susceptibility to degradation from biological attack by insects and fungi, an essential part of timber structure maintenance has always been the application of preservative control measures.

Until the late 1950's, all original and on-going anti-termite poisoning of timber bridges was carried out in accordance with the relevant Main Roads specification, which required an arsenic treatment of log components and sawn timber in contact with the ground. The insecticide used consisted of a liquid mix of arsenic trioxide, caustic soda (which acted as a solubiliser) and water. This liquid was specified to be poured into holes drilled into the heart at the ends of girders and corbels, tops and bottoms of piles etc. These holes were then plugged with timber plugs protruding sufficiently to allow their removal for further poisoning. In addition, any sawn timber in ground contact was also painted with the arsenic mix.

As an inhibitor to fungal attack, an envelope treatment using creosote oil was also required to be applied to all sawn and desapped surfaces of round logs. Particular attention was to be applied to treatment of member ends, bedding areas, joint areas and drilled holes. Creosote application also had a secondary benefit as it also acted as a repellant to termites.

Arsenic poisoning was always considered very effective, though tests indicate that arsenic compounds to not diffuse far from a hole in sound hardwood. However, termites would have had exposure where a poison hole intersected a crack, pipe or softened heartwood.

Use of the next generation of termicides began in the 1960's with the introduction of organo-chlorine poisons. These chemicals, such as aldrin and dieldrin were considered to be very effective, and were again applied by the same techniques used for arsenic. Eventually 50% of Districts were using organochlorines as the main poison. However, in 1987, as a result of the discovery of violatable levels of organochlorine pesticides in Australian export beef, the

Queensland Government passed Legislation to essentially ban these chemicals for most applications (including bridges).

By this time, the use of arsenic was being actively discouraged, so another chemical, chlorpyrifos, became the generally accepted termicide. This product, marketed under various trade names, is an organophosphorous compound, and was again applied by injection into drilled holes in timber members. Anecdotal evidence soon indicated that this product was not as effective as the earlier poisons and also required shorter time periods between poisonings. Chlorpyrifos remains the most commonly used poisoning material but has serious safety issues associated with its use as noted in Section 7.2.

Another form of termite protection that was promoted was the provison of poisoned soil barriers behind abutments and around piles. This was accomplished by puddling the liquid termicide into the soil to form the barrier. Use of this system by the DMR has not been documented but may have been carried out under poisoning contracts. Old Main Roads specifications always required girders to be desapped before installation, but in recent years, as a result of increasing supply difficulties, girder sizing and supply is accepted with sapwood on, provided the logs are preservative treated. The accepted process is CCA treatment which is a water borne, pressure impregnation with copper, chromium and arsenic salts, giving protection from both termite and fungal attack. It should be noted this process is ineffectual on heartwood only members.

Today, health concerns are being raised on the effect of public exposure to CCA treated timbers and consideration is being given to banning of use where access can be gained.

The use of liquid creosote effectively ended in the 1980's due to health and safety concerns. High temperature creosote oil which is a petroleum based product has the potential to sensitise skin to the suns radiation, causing burning if insufficient protective clothing is worn. Various proprietary products, such as copper napthenate have been subsequently used to provide surface protection.

Another product which has had limited exposure with Main Roads as an anti-fungal treatment is boron, in the form a salt, sodium octoborate. This product comes in the form of either a solid rod, or as a liquid, with both being placed in drilled holes where protection from rotting is most required. This is generally at the ends of girders and tops and bottom of timber piles. Where moisture content in a member is sufficiently high (the condition needed for fungal attack to occur) the product will diffuse into the timbers to provide protection against the rot fungus. This appears to be the only product which will actually diffuse through hardwood timber. The drilled holes for the product are plugged with removable plastic plugs, to enable replenishment to be made as required.

There is a substantial amount of technical literature to indicate that boron products are very effective in reducing rotting in timber.

Some protection against termite attack is also claimed for this product, though no controlled testing is known to have been carried out by Main Roads. Figure 7.1(b) shows details used with boron treatment.

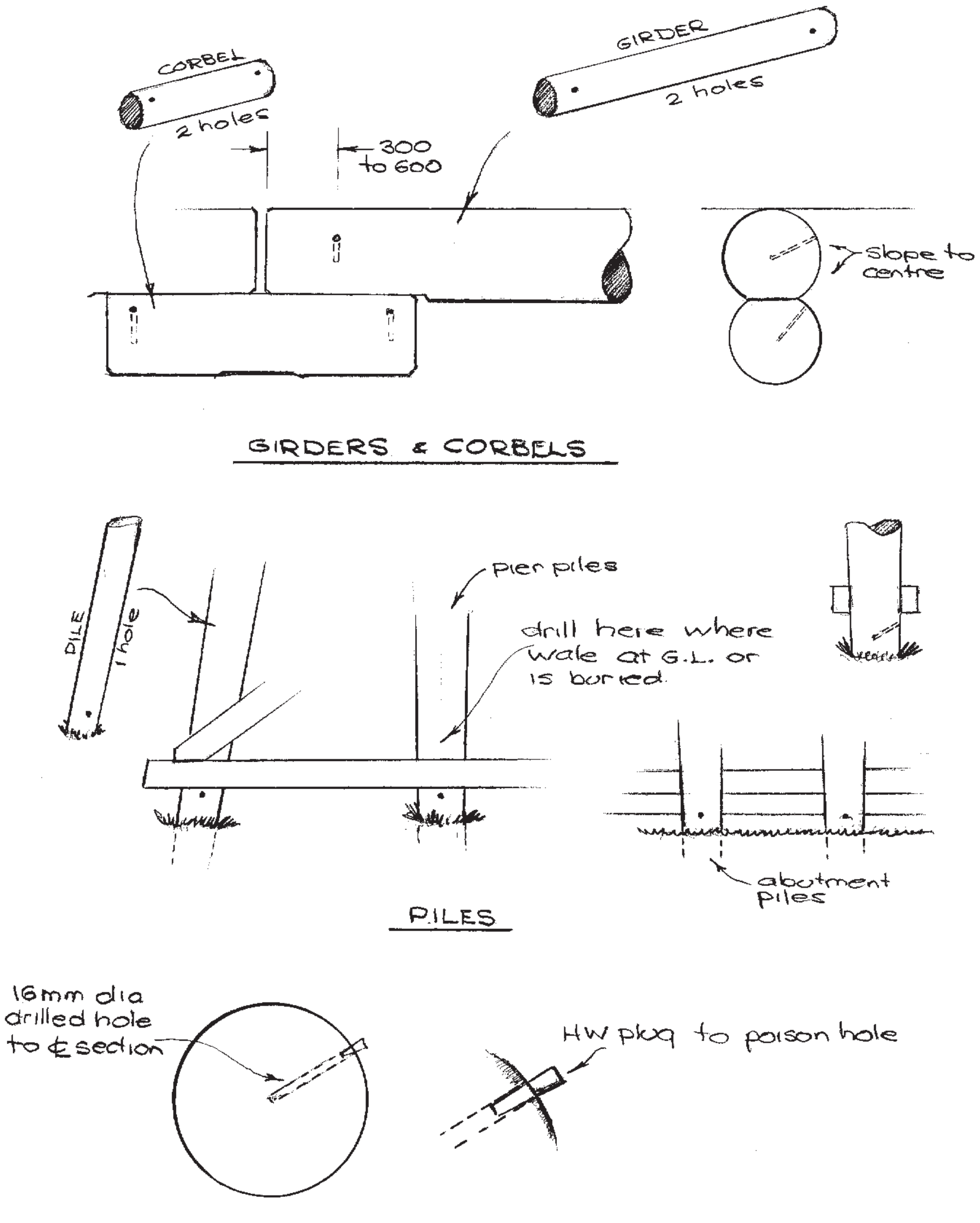


Figure 7.1(a) - Poisoning Details

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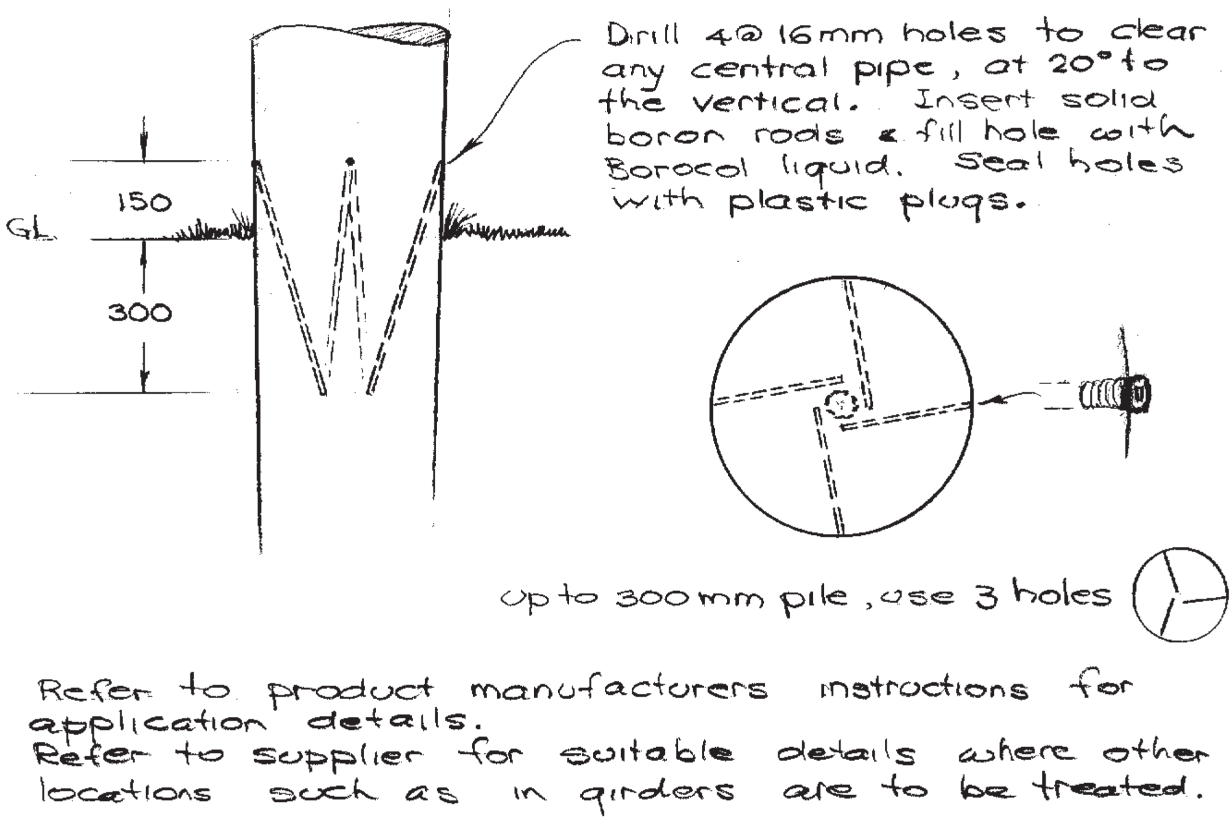

Figure 7.1(b) - Boron Treatment

7.2 Current Treatments & Recommendations

Table 7.2(a) lists some of the more common chemicals that are approved for use to control termites in Australia.

Table 7.2(a) - List Of Australian Approved Termicide Chemicals*

Alpha-cypermethrin	A member of the pyrethroid class of chemicals which are synthetic analogues of the naturally-occurring pyrethrums; it is used to form a barrier to repel or kill termites (see also deltamethrin, befenthrin and permethrin).
Deltamethrin	A synthetic pyrethroid similar to alpha-cypermethrin(see above); it is used in some termiticide products.
Bifenthrin	Another member of the pyrethroid class of chemicals; it is used to form a barrier to repel or kill termites.
Permethrin	Another synthetic pyrethroid, pyrethrin is commonly used as a barrier to repel or kill termites, and is also used for treatment of timber.
Chlorpyrifos	A member of the organophosphorus class of chemicals that is used as a barrier to repel/kill termites.
Hexaflumuron	A member of the benzoylurea class of chemicals that inhibit chitin formation in insects. It is used in strategically placed bait stations to attract foraging termites, which transfer the chemical throughout the colony.
Triflumuron	Another benzoylurea insecticide, triflumuron is applied directly to termite nests.
Imidacloprid	A member of the relatively new class of chemicals called cloronicotinyls. It is used to create a barrier or treated zone in the soil where it attracts termites, which die within the treated zone (partly from the effect of the chemical and partly from infection with fungi and other soil microorganisms).
Aresnic trioxide	A compound used to directly kill termites in active passages (this method has variable effectiveness).

*Registration of other chemicals to be checked before use.

Refer to Figure 17.8 (Part 2) for a list of various product names.

Some use of arsenic trioxide is being made by Main Roads today, in the form of dust puffed into termite passages with a hand blower. Where this method is used in conjunction with other control chemicals, it should be done two (2) weeks earlier as most other products repel termites.

Preservative treatments today may be listed under:-

(a) Use of pressure treated members supplied to site.

(b) Field applied preservatives.

Specification MRS 11.87 requires preservative treatment of piles, unsapped round girders or other components retaining sapwood, to the requirements of Australian Standard AS1604-2000 and the Queensland Timber Utilisation and Marketing Regulation 1998.

Treatments are defined in terms of Hazard Levels 1 to 6 each of which require defined minimum quantities of chemicals to be retained to give a protective envelope to the outer layers of timber. Note that for the timber types used in bridge building, only the sapwood can be penetrated by the preservative. Table 7.2(b) lists the higher level Hazard Classes (3 to 6) for various components and exposures. Bridge timbers generally requires treatment to H5 level.

Table 7.2(b) - Hazard Class Selection Guide

Hazard class	Exposure	Specific service conditions	Biological hazard	Typical uses
H3	Outside, above ground	Subject to periodic moderate wetting and leaching	Moderate decay, borers and termites	Kerbs, decking
H4	Outside, in ground	Subject to severe wetting and leaching	Severe decay, borers and termites	Girders, timber abutments, sheeting etc.
H5	Outside, in-ground contact with or in fresh water	Subject to extreme wetting and leaching and/or where the critical use requires a higher degree of protection	Very severe decay, borers and termites	Retaining walls,, wing piles, piers, piling
H6	Marine waters	Subject to prolonged immersion in sea water	Marine wood borers and decay	Marine piles, braces and wales.

The other factor effecting timber life is its natural durability which is the resistance of the outer heartwood to decay and insect attack. Timber is rated into durability classes 1 to 4 depending on performance in ground contact (equivalent to Hazard Level 5) and this durability class may be used as a guide for suitability of use.

Table 7.2(c) gives expected service life for these classes, but where timber is removed from ground contact and is correctly maintained, these times can be expected to greatly improve.

Table 7.2(c) - Durability Classes

Durability Class	Description	Expected Service Life(Years)
1	Highly durable	25+
2	Durable	15-20
3	Moderately Durable	8-15
4	Non Durable	<8

Only timbers with a durability class 1 or 2 are approved by MRS 11.87 for hardwood timbers.

Table 3 from this specification is given in below as a guide to minimum durability requirements.

STRENGTH AND DURABILITY REQUIREMENTS

Bridge Timber Type	Minimum Stress Grade	Minimum Structural Grade*	Minimum Durability Class
1. Driven Piles (tidal waters)	F22	Round only	2
2. Driven Piles	F22	Round only	2
3. Silled Piles	F22	Round only	2
4. Sill Logs (below ground) - Round - Rectangular	F22	- Round only - No. 2	2
5. Sill Logs (above ground)	F22	Round only	2
6. Wales & Braces	F22	No. 2	2
7. Struts & Fenders - Round - Rectangular	F22	- Round only - No. 2	2
8. Headstocks	F22	No.1	2
9. Girders - Round - Octagonal	F27	- Round only - No. 1	2
10. Corbels - Round - Octagonal	F22	- Round only - No. 2	2
11. Spiking Planks	F22	No. 2	2
12. Deck Planks (transverse)	F22	No. 2	2
13. Deck Planks (longitudinal)	F22	No. 2	2
14. Distributor Planks	F17	No. 2	2
15. Running Planks	F17	No. 2	2
16. Kerbs	F17	No. 2	2
17. Ballast & Cover Boards	F17	No. 3	2
18. Backing Boards	F17	No. 3	1
19. Handrails ∇	F17	No. 2	2

Where timber that has be pressure tested with preservative is supplied, in most cases this will be CCA treatment

For a description of field applied preservatives, which are applied to timber surfaces by brush, trowel etc., refer to Section 17.7 (Part 2).

7.3 Hazards

The current Australian Standard, AS3600.1/1995 for treatment of subterranean termites does not now specify acceptable chemicals for termite treatment, as it had previously. Today, all termicides must be approved and registered by the National Registration Authority for Agricultural and Vetinary Chemicals (the NRA). Before such chemicals can be used in Australia, they undergo a rigorous approval process, including an assessment for possible effects on human health (both of the public and operators who apply the chemical), and on the environment.

Issues related to toxicity and public health are assessed by the Commonwealth Department of Health, issues related to health of pesticide applicators by the National Occupational Health and Safety (NOHSC), and issues related to environmental Safety, by Environment Australia (EA). Together with the NRA, which assesses chemistry and efficacy issues, the combined work of these agencies form what is known as the National Registration Scheme (NRS).

Those chemicals known to have been used by Main Roads are pyrethroids (type unknown), chlorpyrifos and arsenic. Hazard profiles on the various active ingredients listed in Table 7.2(a) are available form BAM if required for information.

In general, pyrethroids are widely used in household insecticides and have a good safety record when used as directed, in that situation.

The organaphosphorus compounds such as chlorpyrifos need to be handled with caution because of their acute neurotoxicity in humans and animals. They are also exceedingly dangerous for marine life in the event of spillage. It should be noted that chlorpyrifos has been banned from sale for general public use in the USA. As an indication of the effect of chlorprifos poisoning, the following applies:-

Chlorpyrifos Toxicity

In mammals, the main signs of organophosphate poisoning are increased swallowing, excessive saliva, rapid breathing, pinpoint pupils, loss of coordination, excitement, twitching and rapid contractions of the neck and jowl muscles, coarse generalized body tremors, secretion of tears, urination, defecation, depression, prostration, convulsions, respiratory failure and death. The severity of the signs increases with the amount of exposure, but there is an effective antidotal treatment for chlorpyrifos poisoning. Regardless of the route of exposure (oral, dermal or inhalation), the toxic effects of chlorpyrifos are similar.

It must be stressed that these approved chemicals are safe for use by licensed operators provided all prescribed safety measures, such as using gloves, protective clothing and respirators are followed. All operators should also be familiar with resulting symptoms in the unlikely event of accidental poisoning.

Workers without suitable safety gear should avoid drilling sawdust from falling over themselves, due to the possibility of poison contaminants.

7.4 Contaminated Sites

In 1990, the Department of the Environment set up a register of contaminated bridge sites for Queensland on which timber bridges were included because of poisoned timbers and also possible soil contamination from spillages. Termite control from impregnated soil barriers was not normal practice, but may have been carried out under some pest extermination contracts.

This list is not complete because of the generally unknown location of earlier replaced timber structures. Another uncertainty is the actual extent of contamination and rate of loss of contaminant. It is known that arsenic binds well to soil particles, but no controlled tests are known to have been carried out on bridge sites to determine the extent of the problem.

7.5 Timber Disposal

Disposal of unusable and unwanted bridge timbers is now controlled by environmental requirements. Bridge timbers are likely to contain some remnants of the various poisons used over the life of the members and this has been confirmed by a recent limited test carried out by Main Roads. General burning on or off site is not permissible because of the toxic nature of some smoke borne contaminants. Arsenic and other heavy metals are particularly hazardous when inhaled.

Similar restrictions obviously also apply to CCA treated timbers because of the arsenic content.

All unwanted timber is to be sent to registered Local Authority dumps where the most likely disposal method will be by burying the timber, though it is possible that some high temperature furnace burning will take place.

7.6 Timber Recycling

Recycling of timber taken from demolished bridges has been practice for many years but is now assuming greater importance as timber supplies become more difficult to obtain.

Though most components are recyclable, it will often only be recently placed timbers which are usable. Because of size, the most critical members will be girders, to be reused as such or cut down to corbel lengths.

The following guidelines should be applied:-

(1) Only components in Condition State 1 or 2 should be reused.

Deteriorated ends are to be cut off and the remainder of the component considered for use.

(2) For timber from bridges built and maintained by Main Roads, it can be assumed that acceptable timber species were used.

If there is doubt, tests may be carried out by the DPI.

(3) Determine the locations where residual components may be used. For example, Figure 3.4(b) lists minimum girder sizes for various span lengths and bridge classes.

7.7 Detailing & Construction Considerations

7.7.1 General

Observation of Main Roads timber bridges will quickly establish the fact that much of the visible deterioration in timber components is generic from bridge to bridge, both in type and location, for example:-

1. Transverse hardwood deck planks will generally begin to rot from the exposed outer end and the kerb contact areas. This is because the outer ends of planks are continuously exposed to

the sun and weather, while moisture will be trapped between the kerb bottom face and the deck plank, with consequent greater potential for decay. Similar moisture entrapment at running and spiking planks often leads to decay of the top and bottom contact surfaces of the deck, and supporting members.

2. Rotting of timber headstocks generally is obvious in the ends of the member, again because of weather exposure. Rotting is also often found at the vertical contact surface with the pile because of moisture presence.

3. Rotting of spiking planks often results in a hollow member, as a result of decay in top and bottom contact surfaces, while decay often progresses down into the top of the supporting girder. Generally outer girders in a structure are more prone to internal decay than inner girders because of greater exposure to weather. As well, timber bridge decks in general are not particularly water proof because of the lack of water tightness of asphalt deck wearing surfaces.

A number of strategies appear to have been adopted at the development stages of DMR timber bridges.

1. Timber deck planks were spiked down only to outer girders through the outer spiking plank which was bolted down. Though normal checking occurs, by stopping splitting of the girders as a result of spike penetration, it was hoped that water ingress into the girder top would be reduced. Bolt holes in the outer girders would be the only entry points for water. This strategy was not entirely successful as water penetration of girders is often found.

2. Flashing, in the form of galvanised iron sheet caps were placed on the ends of headstocks and wales, and the tops of piles. The purpose of these caps would have been to remove direct exposure to sunlight and also to retain preservative such as creosote in the end grain regions of the members.

3. Galvanised iron drip spouts were detailed to be placed at internal scuppers as originally detailed.

Because it has never been possible to completely isolate timber bridge components from weather and water contact, these structures have always and will always decay. However, a number of good detailing and construction practices can help reduce the rate of deterioration.

7.7.2 Construction Detailing

The overall concept of improved detailing is to try and reduce water traps and stress concentrations in members and joints where they can be easily avoided. The main considerations are:-

- Avoiding moisture traps by sealing timber contact surfaces.
- Use of flashing to reduce direct exposure to weather and moisture.
- Protection of components where exposed to water access.
- Reducing the potential for end deterioration.
- Sealing top washers and bolt heads
- Avoiding unnecessary notches or abrupt changes of section.
- Avoiding rebates in upper surfaces.

Contact Surfaces

Where timber surfaces are nominally in contact, there is the potential for moisture to accumulate in any air gaps, leading to increased decaying areas which are often not readily visible. Of particular concern are those areas noted in Figure 7.7(a), such as headstock / pile or pile splice contact areas, though any contact surfaces at all have the potential to collect water. It is recommended that these areas be protected with a preservative and grease before assembly to provide enhanced protection from fungal attack. In order to remove any remaining air gaps, the placement of a bitumen impregnated felt (or equivalent) material is recommended at contact areas with major components.

Suitable products for these treatments are shown in Figure 17.7 (Part 2).

Flashing

DMR personnel appear to have mixed views on the effectiveness of metal caps on the ends of headstocks etc., with concerns the caps can hide rotting and termite infestation from view. It is certainly not recommended that flashing be used where water can gain access unless the flashing can be sealed against the timber. As well nail holes would need to be sealed to prevent any moisture ingress.

It is recommended that flashing be used on the ends of timber headstock members and the tops of exposed timber piles (including wing piles). However, after placing a preservative, a thick layer of grease should be applied before installing the cap tightly to remove any air gaps.

Figure 7.7(b) shows general details.

Flashing is also used on stress-laminated timber decks, in conjunction with a waterproof membrane to protect the board components.

Where concrete packers are placed on top of timber headstocks, use of flashing between the components is also recommended.

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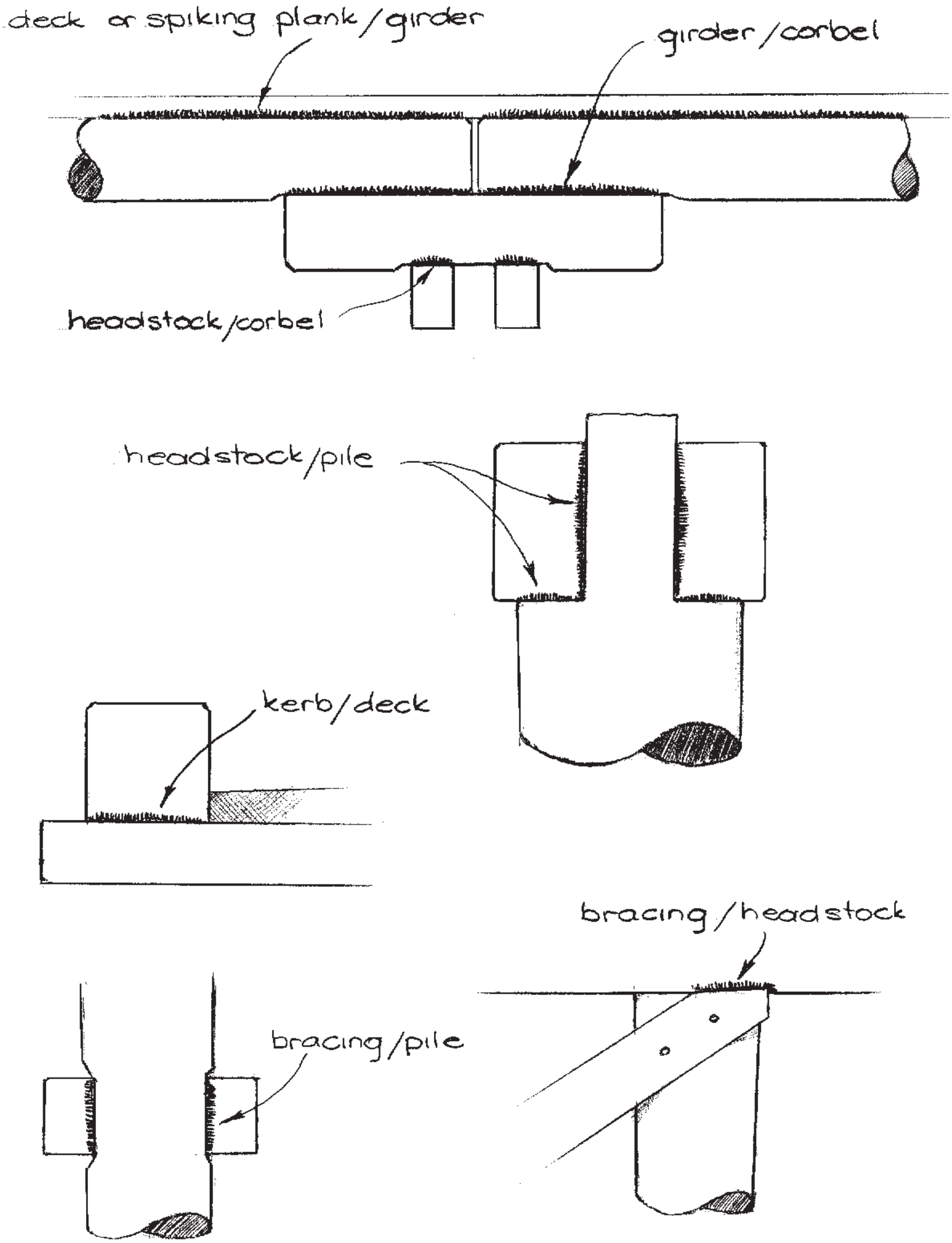

Refer to Figure 17.7 Part 2 for details of recommended preservatives and joint treatment materials.

Figure 7.7(a) - Contact Surfaces

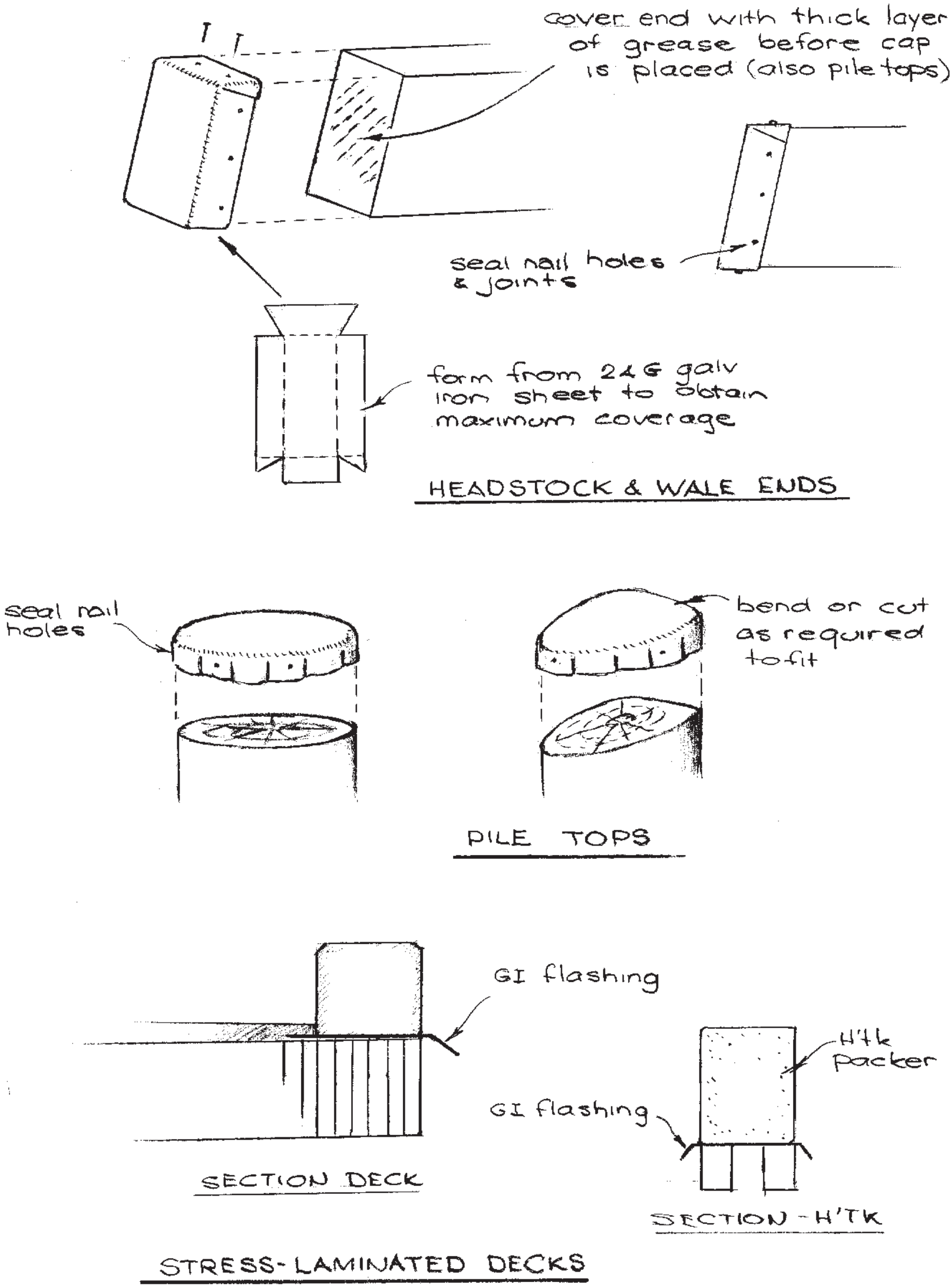

Figure 7.7(b) - Flashing

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Member Protection

Unless there is a significant deck cantilever such as with a ply deck, outer girders are generally exposed to weather and direct sunlight. This leads to reduced life, particularly with octagonal girders where the grain has been cut across during the sawing process. Consideration may be given to applying a protective preservative to the outer girders (at least). Section 8.1, Part 2 details a recommended envelope treatment using currently available materials which should provide some measure of protection.

Where PSC decking is used, open deck joints allow complete access of water to all main girders. Protection should be applied to the girders to waterproof the tops. Figure 7.5(c) shows various applicable details. Protection methods could vary from metal flashing to a grease impregnated tape.

Also noted in Figure 7.7(c) are various waterproofing measures applicable to plywood decking. Where new planks are being placed, a polyethylene elastomer joint filler should be used between joint sides to provide water tightness.

Where resurfacing of ply decks is to be carried out, and gaps are 6mm or wider, these should be sealed with a foam backing rod and Megaprene 40 or (equivalent) sealant. Where gaps are small, a stick-on bituminous tape such as Bi-tak or a thin metal cover strip tacked to the ply is recommended. In both cases, soffit distributors would need to be in place to prevent differential movement of the ply edges.

A drip strip should also be tacked to the outer ply soffit edge to prevent water running or being blown under the sheets.

End Protection

MRS 11.87 requires the application of an end sealant and recommends the placing of end nailing plates on log members soon after cutting in order to reduce the normal tendency for end splitting of such members. Wax emulsion sealers as specified, are specifically formulated to control the rate of moisture loss in green timber, by forming a durable wax membrane between the exposed end grain and the surrounding ambient atmosphere. The goal is not to prevent moisture from moving through the wax coating, but instead to retard the rate of moisture evaporation, thereby reducing drying defects such as end grain checking. Refer to Figure 17.7(b) (Part 2) for suitable proprietary products.

After trimming of new members such as corbels and headstocks, use of nail plates on the ends should be considered in order to reduce the potential for end splitting. Where sawn octagonal corbels are used, end nail plates shall be used. Refer to Figure 7.7(d) for details.

Exposed ends of members such as girders and corbels in service must have an end grain sealant applied to reduce the drying out and cracking associated with this area.

This may take the form of a thick toweled on application of copper naptherate, or preservative and grease applications. It may not be possible to reapply such coatings to girder ends, depending on the end gaps.

Plywood decking, in particular, requires a sealant such as bitumen to be applied to the exposed outer face, in order prevent drying out of the CCA treatment and possible delamination of laminates.

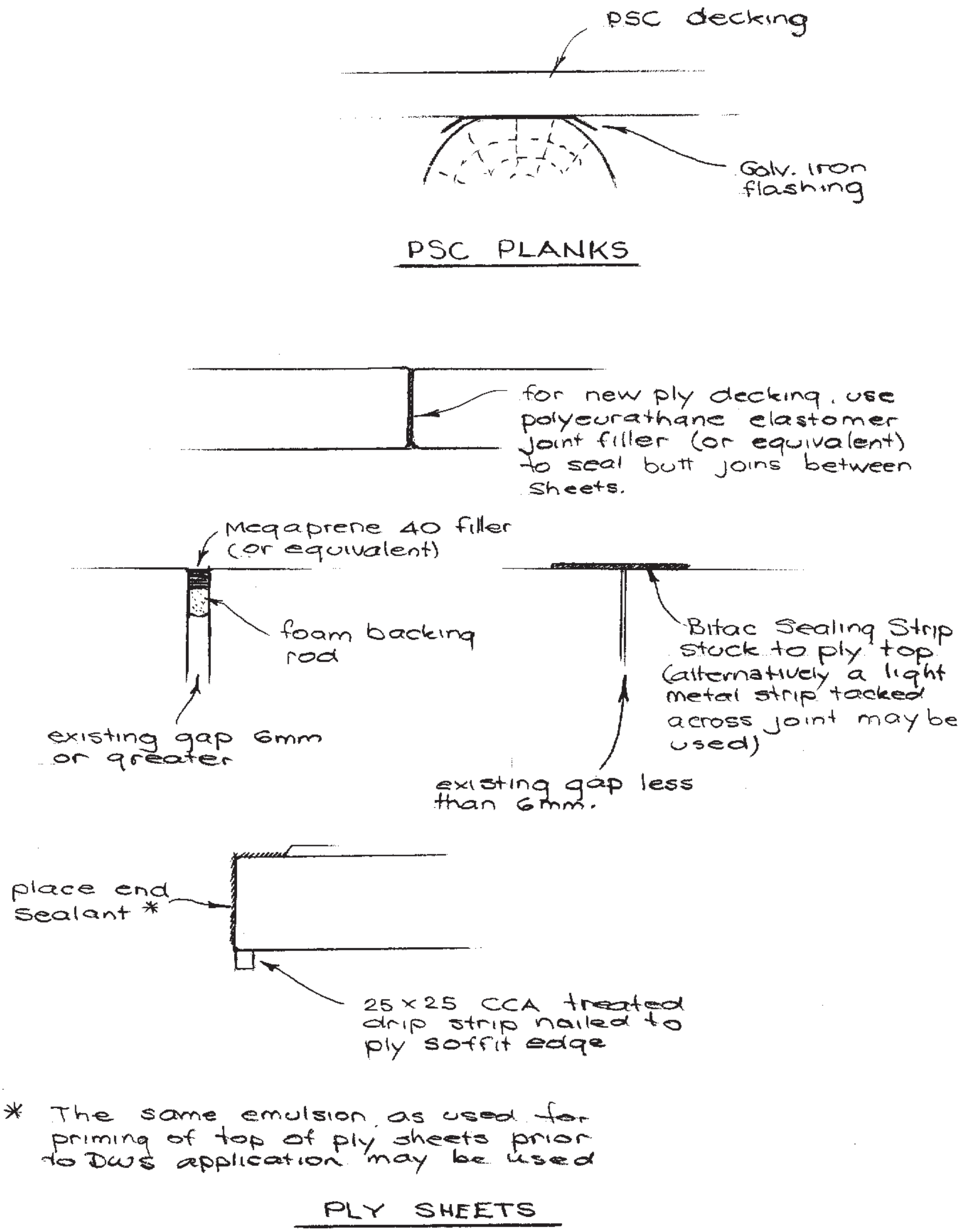

Figure 7.7(c) - General Protection

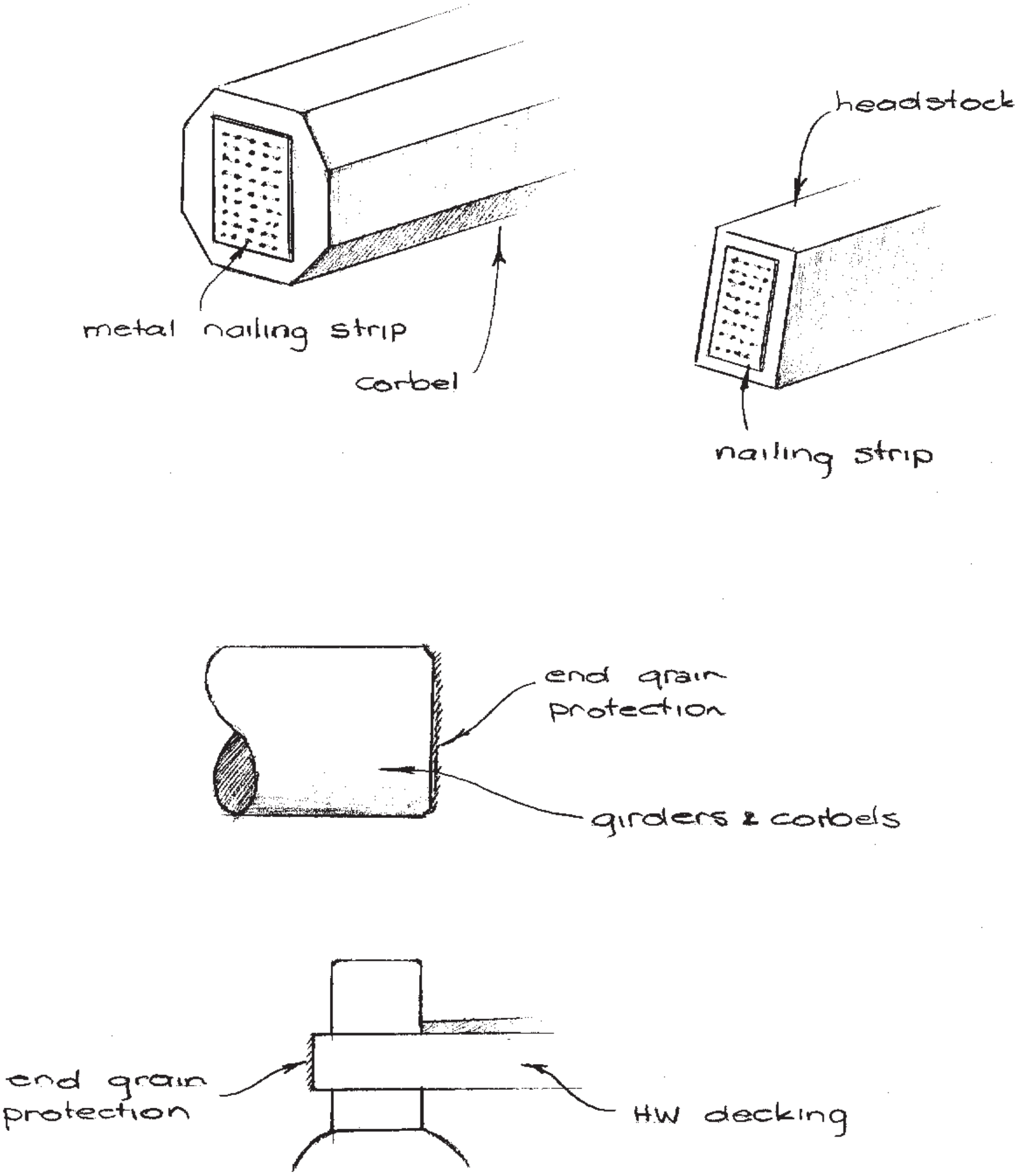

Figure 7.7(d) - End Protection

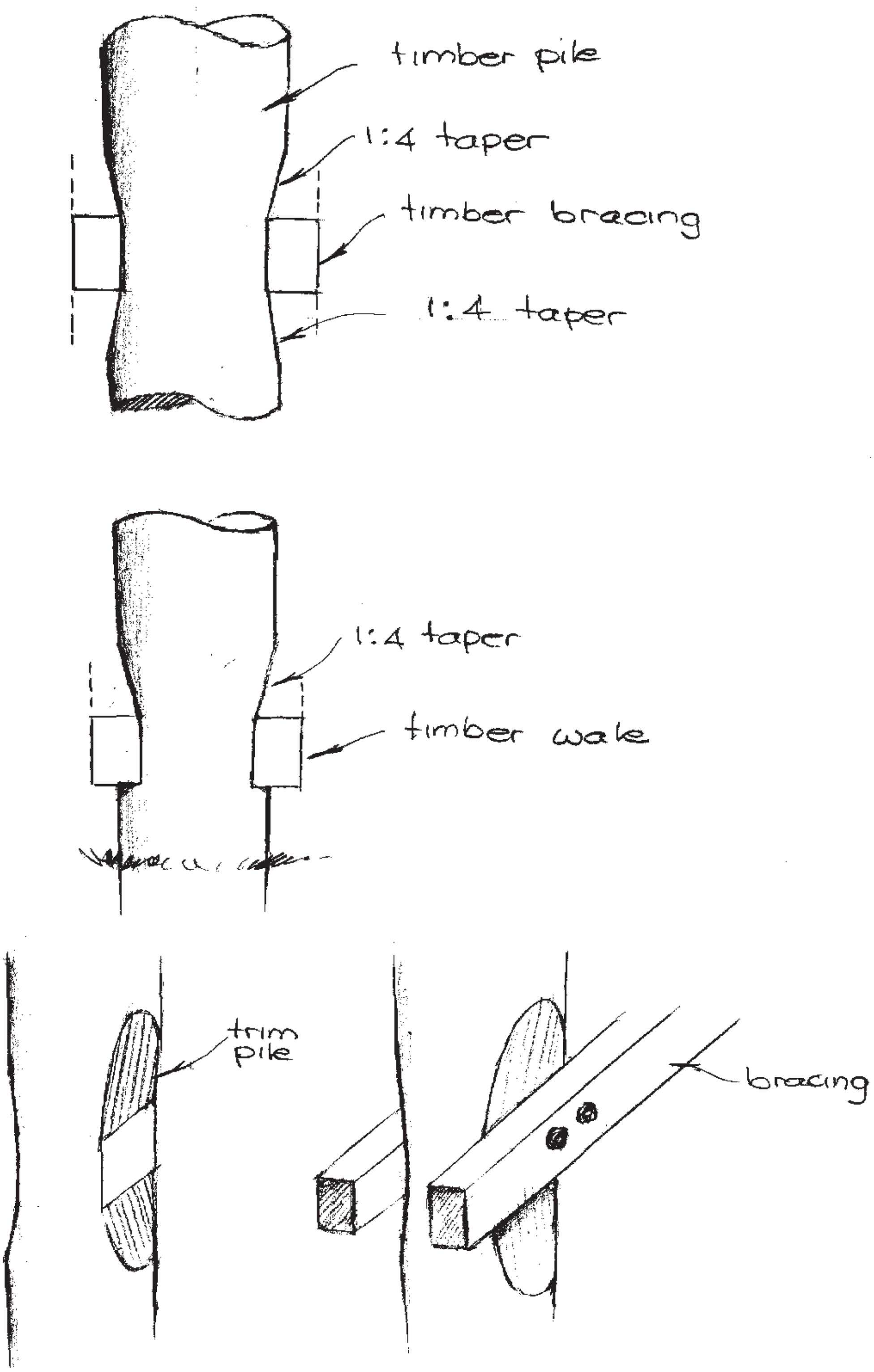

Figure 7.7(d) - End Protection

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Bolts

Because drilled holes for bolts provide access pathways for water ingress, these should be preservative treated, ideally with a thick material which will not run out of the hole. It is recommended that grease or petroleum jelly be applied to the bolt as well before installation, to try and fill the void around the bolt. There will also be benefits in sealing the bolt heads and washers to try and stop water penetration, but the lower washer should not be sealed in case water does access the hole.

Notching

As noted in Section 4.6, notching of girders causes stress concentrations and should be detailed as described in Section 8.1 (Part 2). Likewise, the minimum notching possible (such as for pile bracing) should be carried out to also reduce the potential for water entrapment and to improve access for inspection and treatments. In general, areas which require section changes or flat surfaces for connections should be tapered using a 1 in 4 slope as shown in Figure 7.7(e)

Rebating

In general, it is not recommended that top plates or washers be rebated into timber because of water trapping. If necessary, however, they should be sealed with a rubberised epoxy.

8.0 Inspection

8.1 General

An essential part of the management of timber bridges is a regular inspection regime because of bridge susceptibility to deterioration. Since release of the Bridge Inspection Manual, there is now a policy requirement on inspection frequencies and standardised methods of defect recording, allowing quantifiable estimates of structure condition to be made.

As well as for structure management requirements, this information is essential to BAM both in the assessment of overload permits for heavy vehicles and for general bridge load rating.

8.2 Defect Locations

Timber deterioration may take place at any location in a bridge, but tends to occur most frequently in specific areas. Figure 8.2 is included as a guide to expected locations.

8.3 BIM Requirements

Inspection requirements for bridge management purposes are covered in the Bridge Inspection Manual under Part 1, Article 1.5. Inspection procedures for the various levels of investigation are covered in Part 2 of that manual.

8.4 Additional Requirements

Further to the above requirements listed in the BIM, additional consideration should be given to the following:-

- Rotting of a headstock in the contact area with a pile (particularly the exposed outer pile)- refer to Figure 8.4(a)(1).

  A hole should be drilled into this area as shown and checked with a probe to determine if rotting is occurring.
- Lack of headstock section due to excessive cuts in the headstock. Standard drawings require braces to be notched 25mm into headstock soffits, and if carried across the full width would reduce bending strength by 16% in a 300mm deep member. Apart from stress concentrations at a notch, a reduction in depth of 50mm would reduce bending strength by 30%.
- Lack of headstock seating due to removal of pile edge for bracing - refer to Figure 8.4(a)(2).
- Adequacy of pile top for flood uplift - refer to Figure 11.1(g) (Part 2)
- Pile ground support. Even though a pile may drill satisfactorily, it should be observed for any sign of movement or pumping under a heavy load.
- Presence of the top pile strap bolt - this may have been removed during previous work - refer Figure 11.1(g) (Part 2).
- Presence of combustible material below a bridge. There is the possibility that in remote locations, there may be a build up of vegetation or flood debris, with increased risk of fire damage to piles and headstocks.

  Fire hazard maps are available for the State and could be used to determine the likelihood of bushfire in the bridge area.

  Figure 8.4(b) shows the end result of a bushfire on a timber bridge.

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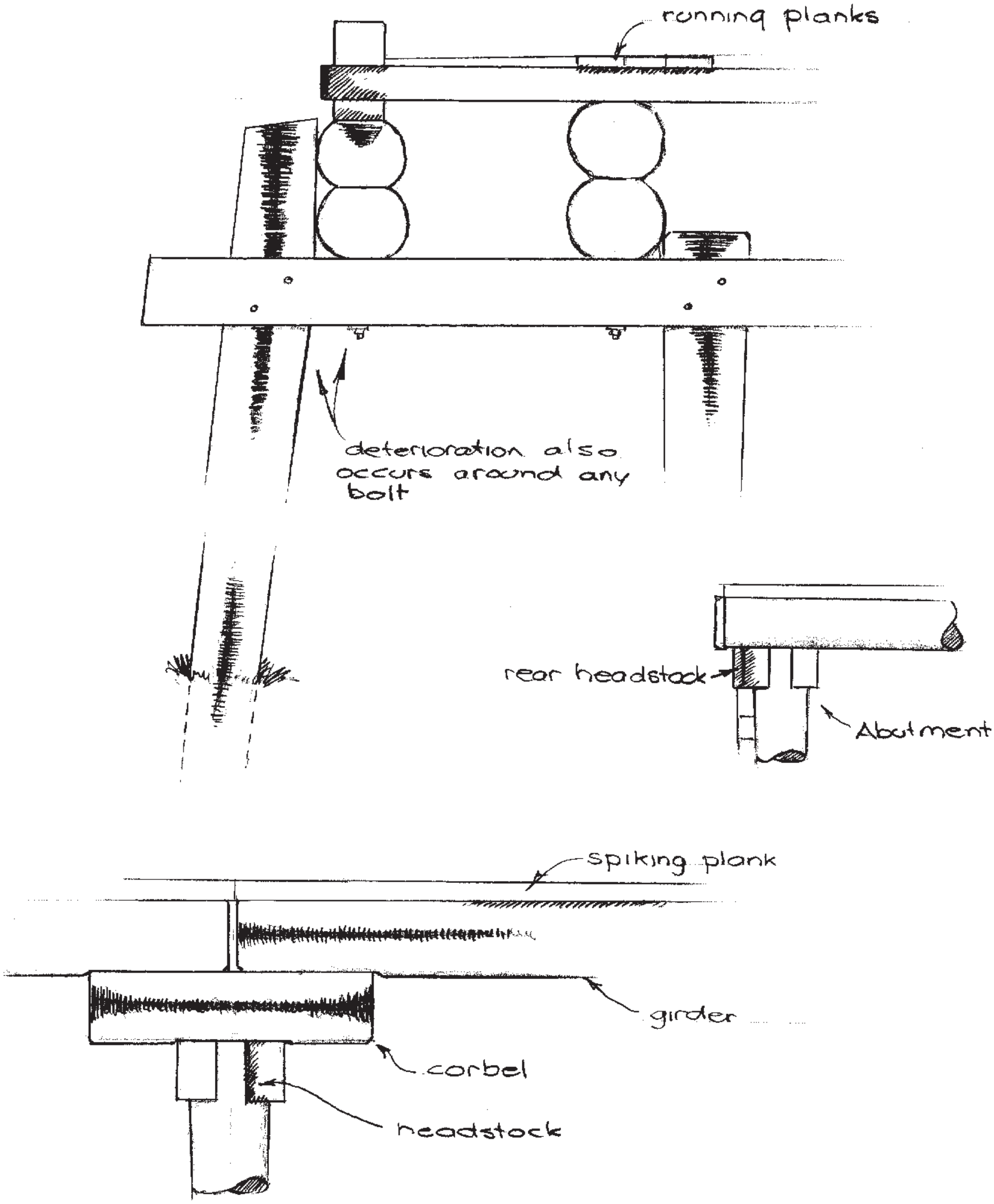

Figure 8.2 - Defect Locations

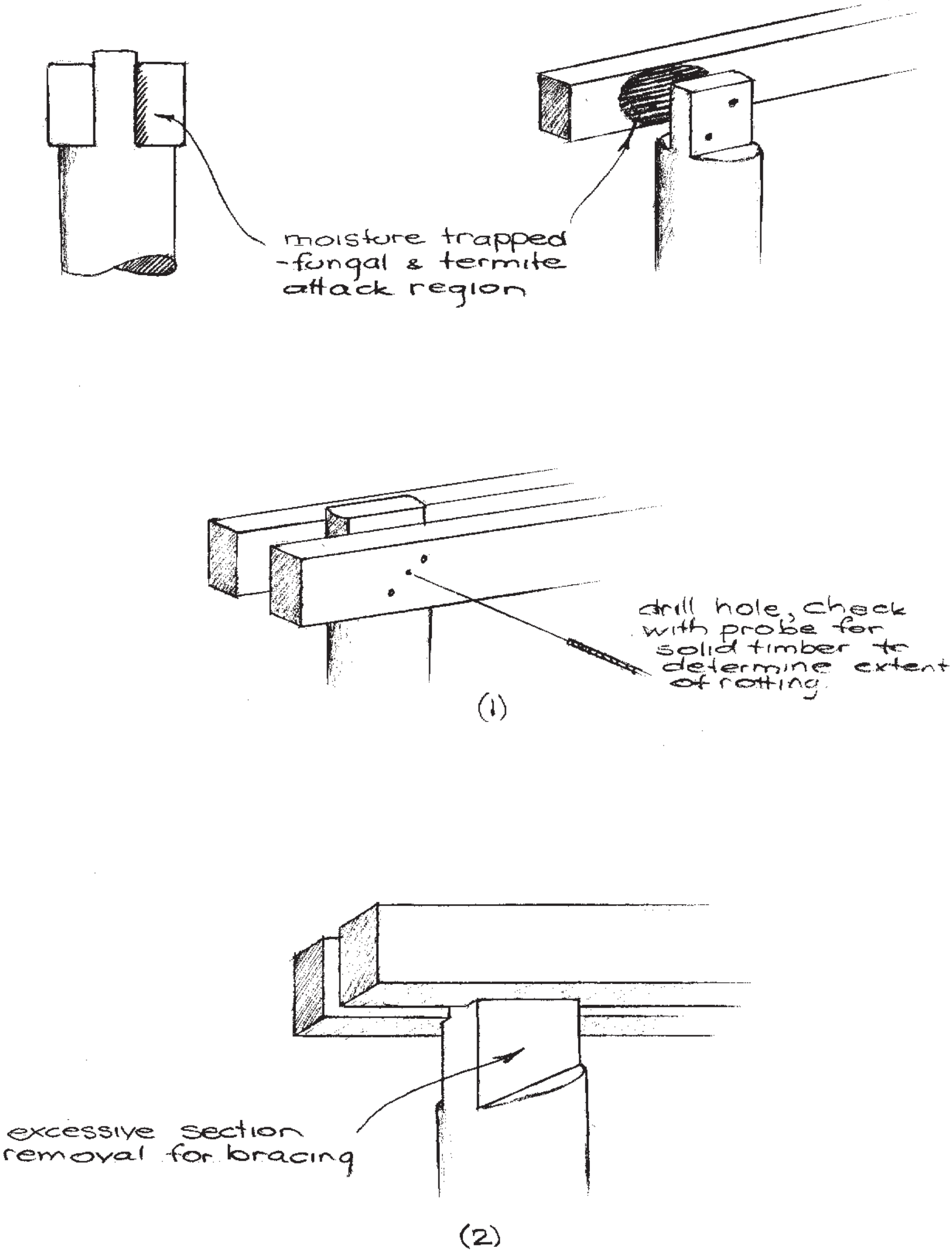

Figure 8.4(a) - Inspection Details

Bushfire Result

Figure 8.4(b) - Fire Destruction

8.5 Timber Drilling

The traditional method for the internal examination of timber members such as girders, piles and headstock has been to drill these members and record depths of sound timber to determine internal defect sizes. BIM Section 3.10 (Part 3) gives details.

However, in order to improve reliability and consistency of reporting and determination of condition states, additional Guidelines have been issued by BAM in Advice Note No. 24. Refer to Part 2 of Appendix D for these extra requirements.

8.6 New Testing Methods

Recently a number of alternative technologies have been trialled to determine if more accurate testing results can be obtained. These technologies are:

(1) ground penetrating radar (non destructive)
(2) nuclear densometer (non destructive)
(3) resistograph

The radar method records reflected radar signals to produce a continuous readout over the length of a member. Interpretation of the results is difficult, but it is claimed that calibrated tests on a number of ex-bridge girders has proven that an experienced operator can identify the extent and location of internal defects. However, this method is not considered suitable given the expert interpretation required.

The second method uses an isotope source and a detector which also gives a continuous readout along a member. This method gives a measurement of soundness by determining average density along a member. Site tests on ex-bridge girders have also indicated that this method can give encouraging results.

The resistograph uses a very fine drill (2mm) to drill a member, with the resistance graphically output to a paper trace to indicate timber soundness. Tests indicate the results of each drilling accurately portray internal soundness, but gives results at only the discreet points drilled.

For further information on nuclear densometer and resistograph testing refer to Appendix D. Recommendations have now been made that the preferred option is to scan with the densometer and then probe with the resistograph.

8.7 Load Testing

Non-destructive testing by observation of bridge response under traffic is available as a means of determining girder adequacy. Trials were recently carried out on two methods as part of a project to accurately determine timber girder capacities in the field. Reference 10 gives procedures and test results from both these methods which are:-

(1) Vehicle load test - test vehicles of known mass are driven at various speeds and lateral locations across a bridge and the resultant deflections and strains are recorded.

From these test results, assessments can be made of the load carrying ability of the girders in a particular span.

(2) Dynamic impact assessment - developed by the University of Technology Sydney, this method measures the accelerations induced in the bridge girders after exitation by a deck impact with a nodal hammer. From the dynamic responses of the girders, the stiffness and capacity of the bridge spans can be determined.

Reference

10. Bridge Asset Management. (2003). Capacity of Bremer River Timber Bridge. Department of Main Roads, Queensland.

9.0 Legal Liabilities

Timber bridges are known to be very forgiving of certain defects with loads often being redistributed when one member fails or weakens.

Except for cases of gross overload or extreme flood loadings, few catastrophic failures have occurred.

However, there are limits!

Over the years, many timber bridges have been found to have primary structural members in poor condition (now listed as Condition State 4), but because of members appearing to continue to function satisfactorily, there was often no immediate push to carry out repairs.

Because of legal ramifications of catastrophic failure to Main Roads, such a culture which grew up over the years needs to be modified.

Legal advice is being sought on these matters, with particular reference to management of permit overload vehicles on our timber bridges.

The current situation may be summed up as:-

Increasing magnitude of loads coupled with decreased maintenance expenditure exacerbated by increasing material and labour costs.

10.0 References

1. Bridge Asset Management. (2003). Whichbridge Bridge and Culvert Maintenance Prioritisation Software - User Guide. Department of Main Roads, Queensland.
2. Kendall, M. (1997). Health Monitoring of Timber Bridges. Undergraduate Thesis, Queensland University of Technology.
3. Deane, S. (1997). Health Monitoring of Timber Bridges. Undergraduate Thesis, Queensland University of Technology.
4. Bowden, M. (1998). Health Monitoring of Timber Bridges. Undergraduate Thesis, Queensland University of Technology.
5. Leja, M.P. (1998). Health Monitoring of Timber Bridges. Undergraduate Thesis, Queensland University of Technology.
6. Rossow, S. (1998). Health Monitoring of Timber Bridges.
7. Dunne, T. (1998). An Investigation into the Corbel Behaviour and Failure Mechanisms of Timber Bridge Girders. Undergraduate Thesis, Queensland University of Technoloby.
8. Green, C.D. (1998). An Investigation into the Corbel Behaviour and Failure Mechanisms of Timber Bridge Girders. Undergraduate Thesis, Queensland University of Technology.
9. Harris, A.D. (1998). An investigation into the Corbel Behaviour and Failure Mechanisms of Timber Bridge Girders. Undergraduate Thesis, Queensland University of Technology.
10. Bridge Asset Management. (2003). Capacity of Bremer River Timber Bridge. Department of Main Roads, Queensland.
11. Infratech Systems and Services. (1998). Performance of Steel Trough Decks on Timber Girder Bridges.
12. Infratech Systems and Services. (1998). Performance of a Deteriorated and Rehabilitated Timber bridge Deck.