Review of Condition Assessment and Retrofitting Techniques for Timber Bridge Assets in Australia

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Abstract: This paper provides an overview of the types of timber bridges in Australia and identifies the shortcoming associated with the ageing stock in order to preserve these assets and render them safe and useable under new loading standards. Methods available to assess the condition and residual capacity of timber bridges by non-destructive techniques and retrofitting techniques to enhance the strength are discussed, with the objective of increasing the design life and satisfying strength criterion.

Key words: timber bridges, rehabilitation, ageing, structural upgrade.

1. INTRODUCTION

In Australia, timber was the dominant construction material for bridges in the late 19th and early 20th centuries because of the abundance of hard wood timber and the relative low cost compared to steel or concrete. There are around 27,000 timber road and rail bridges in Australia with most at least 80 years old. In the Victorian municipal road network there are about 1100 timber bridges (Engineers Australia Victorian Infrastructure Report Card 2005). In contrast, Tasmania has about 3500 timber beam bridges; whilst New South Wales has approximately 4000 timber bridges including 82 timber truss bridges built between 1800 and 1920.

The most common form of construction of these bridges uses round girders fabricated from native hardwood, spaced 1m to 1.5 m apart, with a typical span of 7 m to 12 m and transversely overlaid with decking planks, as shown in Figure 1.

Timber degrades when left exposed to the environment and therefore has a high maintenance demand. Timber used in bridge construction needs to be sourced from large diameter hardwood trees, which are becoming increasingly scarce. The main shortcomings of timber bridges include:

- compromised serviceability and strength for heavier and faster modern traffic;
- high maintenance costs;
- construction details which allow the penetration of water and hence the deterioration of structural members and connections over time;
- low durability due to the declining quality of the hardwoods used for replacement of members.

As more than 70% of these bridges are ageing and in a high maintenance phase of their useful life, many bridge asset owners are confronted with:

- increasing shortage of large section hardwood suitable for repair and rehabilitation of these structures;
- insufficient funds to construct replacement structures;
- ongoing need for the assets to remain in safe and useable conditions under increasing design axle loads associated with the higher mass limits permitted by 1999 legislation.

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Clearly there is a need for condition monitoring and cost effective retrofitting means for the effective asset management of timber bridges which is the objective of this paper.

2. LOADING CONDITIONS
The design load $E_d$ for ultimate strength limit state design is determined from the Australian Bridge Standard, AS 5100.2 (2004) as given by:

$$E_d = (\gamma_g G + \gamma_s S + \gamma_q Q)(1 + \alpha)$$

where, $G$ is the dead load, $S$ the superimposed dead load (surfacing material, utility services etc.), $Q$ the traffic load and $\alpha$ is a dynamic load factor. The load factors $\gamma_g$, $\gamma_s$, and $\gamma_q$ for ultimate and serviceability limit states are presented in Table 1.

Design traffic loads consist of W80 wheel loads (80 kN), A160 axle loads (160 kN), M1600 moving loads (12 axle and uniformly distributed loads totalling 1600 kN), and S1600 stationary loads (12 axle and uniformly distributed loads totalling 1600 kN) as described in AS 5100.2 (2004). In addition, the relevant transport authority may specify the heavy platform load HLP320 or HLP400 as described in AS 5100.7 (2004) (HLPs are prescribed as having 16 rows of axles with an axle load of either 200 kN for HLP320 or 250 kN for HLP400).

The dynamic load multiplying allowance $\alpha$ allows for the increase in the traffic load resulting from the interaction of moving vehicles and the bridge structure. The dynamic load allowance applies to both the ultimate and serviceability limit states and the values are presented in Table 2. However, as timber bridges possess excellent inherent damping, a 30% reduction on the dynamic load allowance is permitted for the serviceability limit state (Crews and Ritter 1996). The deflection under serviceability limit state is limited to span divided by 600. While some bridges may be designed for this limit, serviceability limits of span divided by 400 is common for timber bridges. In many cases where timber members have degraded resulting in significant loss of section, the deflections will be higher than these limits.

3. FAILURE MODES IN TIMBER BRIDGES
The possible failure modes of timber bridges consist of shear, compression or tension failure of the timber member or failure at the connections:

- Shear failure is rarely the controlling parameter for member capacity, except in beams with high loads and short spans;

<p>| Table 1. Load factors for bridge design |</p>
<table>
<thead>
<tr>
<th>Load factor</th>
<th>$\gamma_g$</th>
<th>$\gamma_s$</th>
<th>$\gamma_q$</th>
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<tr>
<td>Ultimate limit state</td>
<td>1.2</td>
<td>2.0</td>
<td>1.8 (1.5 for heavy load platform)</td>
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<tr>
<td>Serviceability limit state</td>
<td>1.0</td>
<td>1.3</td>
<td>1.0</td>
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<p>| Table 2. Dynamic load allowance for bridge design |</p>
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<th>Loading</th>
<th>Dynamic load allowance, $\alpha$</th>
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<tr>
<td>W80 wheel load</td>
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<tr>
<td>A160 axle load</td>
<td>0.4</td>
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<tr>
<td>M1600 load</td>
<td>0.3</td>
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<tr>
<td>S1600 load</td>
<td>0</td>
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<td>HLP loading</td>
<td>0.1</td>
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Compression failure is of less concern since this failure mode is ductile in nature and associated with crushing and local buckling of individual cellulose fibres;

Tension failure is a more common failure mode, since timber is a natural material with local defects such as knots. Typical defects in the tension zone are shown in Figure 2. Local defects cause stress concentration and initiate propagation of cracks leading to failure. Timber is stronger by an order of magnitude in the parallel direction to the grain as compared with perpendicular direction, and defects cause the timber fibres to be no longer aligned with the principal bending stresses, thereby weakening the timber specimen;

Connection failure is presumably the most common form of failure in timber trusses and headstocks due to the lower shear and bearing stress capacity of timber and the concentration of stresses in the vicinity of the fasteners. In addition, the placement of fasteners can create long term durability issues since moisture can ingress and cause corrosion of the fittings and rotting of the timber.

4. DESIGNING FOR DURABILITY
Timber bridges are often exposed to harsh environmental conditions causing deterioration resulting from decay, insect attack, weathering, and structural damage. Biological degradation of timber is exacerbated if the timber is damp. The trapped water, as a consequence of poor detailing, can accelerate biological degradation particularly where the end grain of timber is exposed. Durability could be improved through: (1) structural detailing with chemically treated members; (2) surface protective coating, and (3) regular maintenance. MacKenzie (1997) provides some guidance to improve detailing of members and connections including:

- shedding of water and avoidance of trapped moisture/water;
- ease of application of protective coating;
- ease of inspection and maintenance;
- resistance to corrosion of fasteners;
- provision of extra member thickness to compensate for loss of material with ageing and deterioration.

Chemical treatment can significantly increase the design life of timber. Chemicals can impregnate the wood cells and make them resistant to decay, insects, weather and fire. A range of different chemical treatments are available for enhancing the durability of timber. The most commonly used treatments in Australia are Copper Chrome Arsenate, Creosote and Pigment Emulsified Creosote (Crews 2003). Unprotected end grain surfaces are at risk as they absorb water through capillary action and hence the end grains must be sealed and maintained with a high quality coating.

5. CONDITION ASSESSMENT
A non-destructive evaluation (NDE) of the bridge structure is considered warranted after routine physical inspections and condition assessments have identified areas of deterioration or concern. The proper application of NDE techniques enable the material properties and in turn the structural integrity and residual capacity of the bridge to be assessed without impairing or compromising the performance. In the condition assessment procedure, the bridge needs to be initially assessed as a global system using a NDE technique in order to locate the possible areas of damage. The areas identified could then be investigated further using a more localized form of NDE to confirm the damage and to assess the magnitude of the damage.

5.1. Condition Assessment of Overall Bridge System
The two common methods employed for global condition assessment are (a) dynamic system identification, and (b) diagnostic load testing.

5.1.1. Dynamic system identification
Experimental modal analysis can be used to identify natural frequencies and corresponding mode shapes of the bridge. This involves placing sensors (typically accelerometers) at various locations on the bridge and exciting the bridge by a mechanical shaker, a single vehicle, or ambient vehicular traffic. The modal analysis examines each mode shape of the vibrating structure and compares it either to previous experimental data or

Figure 2. Failure modes associated with tension zone in bending (John and Lacroix 2000)
predicted data obtained through finite element modelling (Haritos et al. 1997). Ideally, field data of the structure would be measured and analysed on a regular basis so that any changes in the modal properties could be used to identify damage and deterioration. However, the field measurements on the undamaged structure are not usually available and consequently the modal properties have to be interpreted from a finite element analysis based on the as-built structure. Differences in dynamic characteristics are used to diagnose the damage in the structure (Peterson et al. 2003), while the damage localization algorithm proposed by Stubbs et al. (1995), may be used to locate the possible areas of damage. In this algorithm, the differences in modal strain energy between the undamaged (or previous measurement) and damaged structure provides the basis for identification of localized damage as described below:

The modal strain energy stored in the beam for the i-th mode of vibration $U_i$ is given by:

$$U_i = \frac{1}{2} \int_0^L E(x) I(x) \phi_i''(x)^2 \, dx$$  \hspace{2cm} (2)$$

where, $E(x)$ is the modulus of elasticity of the girder, $I$ the second moment of the area of the cross section, and $\phi_i''(x)$ is the second derivative of the i-th mode shape with respect to the position along the girder. If the girder is divided into $j$ elements, the modal strain energy in the j-th element for the i-th mode is given by:

$$U_{ij} = \frac{(EI)}{2} \int_j \phi_{ij}''(x)^2 \, dx$$  \hspace{2cm} (3)$$

where, the integral is applied over the limits of the j-th element only and it is assumed that $E$ is constant over the length of the element $j$. The fraction of the modal strain energy for the i-th mode, concentrated in the j-th element, can be expressed as:

$$F_{ij} = \frac{U_{ij}}{U_i}$$  \hspace{2cm} (4)$$

where, $0 < F_{ij} < 1.0$. Similar expressions can be written for the damaged girder, denoting the damaged state with a superscript asterisk. The damage indicator for the j-th element and the i-th mode $\beta_{ij}$ is then defined as:

$$\beta_{ij} = \frac{1 + F_{ij}^*}{1 + F_{ij}}$$  \hspace{2cm} (5)$$

To account for all available modes, $NM$, the damage indicator for a single element $j$ is given as:

$$\beta_j = \frac{\sum_{i=1}^{NM} \text{Num}_{ij}}{\sum_{i=1}^{NM} \text{Denom}_{ij}}$$  \hspace{2cm} (6)$$

where, $\text{Num}_{ij}$ is the numerator of $\beta_{ij}$ and $\text{Denom}_{ij}$ is the denominator of $\beta_{ij}$. Assuming that the damaged indicator values $\beta_j$ are random variables, they are transformed into a standard normal space to scale the values. The standard normal damaged indicator is expressed as:

$$Z_j = \frac{\beta_j - \mu_{\beta_j}}{\sigma_{\beta_j}}$$  \hspace{2cm} (7)$$

where, $\mu_{\beta_j}$ is the mean of $\beta_j$ for all $j$ elements, and $\sigma_{\beta_j}$ the standard deviation of $\beta_j$ for all $j$ elements. Now the hypothesis testing is used to classify the elements into one of two categories, that is element $j$ is undamaged or element $j$ is damaged. A threshold value must be judgmentally selected and used to determine which of the $j$ elements are possibly damaged along the span of the girder.

5.1.2. Diagnostic static load tests
Diagnostic load tests typically involve relatively small static loads applied to a bridge in order to determine its true elastic response. The test results are used to determine the elastic modulus, whilst the maximum load carrying capacity of the structure is determined through statistical correlation between modulus and strength. In contrast, larger proof load test (generally 2 to 3 times the design vehicular load) can determine the structure’s safe load carrying capacity, but has the danger of permanently damaging the ageing structure (Crews et al. 2005).

5.2. Local Condition Assessment of Individual Members
The three NDE techniques commonly employed to locate and assess the magnitude of damage or decay in timber members are: (a) visual inspection, (b) impact test, and (c) microwave/ground penetrating radar technique.

5.2.1. Visual inspection (qualitative assessment only)
Using visual inspection one can quickly develop a qualitative assessment of the structural integrity of individual members, relative to each other. Observations
that are possible with visual inspections include the following (Ross et al. 1999):

- Sunken faces or localised surface depressions implying underlying decay whilst crushed wood can also be an indicator of decay;
- Staining or discoloration indicates that timber has been subjected to a higher moisture content suitable for decay. Rust stains from connections are also a good indication of wetting;
- Plant or moss growth in splits and cracks indicates that timber has been at a higher moisture content for a prolonged period and may sustain decay;
- Insect activity, characterised by holes, frass, powder posting, may also indicate the presence of decay;
- Localised damage such as crushed or fractured fibres can indicate excessive stresses from an overload condition.

Visual inspection is very useful but has definite limitations. Components with limited access may be susceptible to error, unexposed components cannot be inspected and the information is limited to the exterior surface of the timber.

5.2.2. Impact test

Stress waves generated from an impact on the surface of the member, propagate through the material and reflect from external surfaces, internal flaws, and boundaries between adjacent materials. The time taken for a stress wave to travel a specified distance can be used to assess the condition of the material. Since stress waves travel more slowly through decayed wood than sound wood, the localized condition of a member can be determined by measuring stress wave time at incremental locations along the member. Locations that exhibit longer stress wave times are locations of potential decay. Incipient decay may be identified by observing that sound wood transmits higher frequency components while decayed wood transmits only low frequency components (Bozhang and Pellerin 1996).

A common application of impact testing is to determine the modulus of elasticity for structural members. Using time-of-flight measurements over a predetermined length, the velocity of the stress wave can be calculated and hence the dynamic elastic modulus by multiplying the square of the velocity and the density of the material. The strength can be estimated using statistical correlations between modulus and strength (Ross et al. 1997). In practice, commercial equipment is used for time-of-flight measurement, where a stress wave is induced in a member by a hammer and is detected with accelerometers at two points along the propagation path. The timer starts when the wave front arrives at the first accelerometer and stops when the wave front arrives at the second accelerometer from which the propagation time between the accelerometers is displayed.

5.2.3. Microwave/Ground penetrating radar technique

Microwave and millimetre wave inspection techniques involve the propagation of electromagnetic waves from antennas at frequencies ranging from 300 MHz to 300 GHz. In electrically insulated materials, separate transmitting and receiving antennas may be employed (through transmission technique), or a single antenna may be used for transmitting and receiving reflected wave energy (ground penetrating radar technique).

Since electromagnetic waves are sensitive to the presence of moisture, this technique has significant potential for detecting decay in members of ageing timber structures (Zoughi 1990). As timber decays certain electrolytes are released from the timber structure and the electrical properties of the material are altered. By measuring the attenuation of microwave radiation, the dielectric properties and their corresponding directions are identified. The dielectric properties can be related to moisture content by suitable calibration and hence the state of timber is deduced (Schajer and Orhan 2005).

5.3. Assessment of Ultimate Strength of the Damaged Structure

Using a reliable statistical correlation between bending strength or modulus of rupture (MOR) and modulus of elasticity (MOE), similar to the one shown in Figure 3 obtained experimentally for Tasmanian hardwood, the strength of the damaged members can be estimated from stiffness measurements. A finite element model could then be updated to predict the ultimate global strength of the deteriorated structure. Noting that the relationship between MOR and MOE given in the Australian Timber Structures Code, AS1720.1 may overestimate the rupture strength of degraded timbers, Crews et al. (2005), developed a statistical relationship between the girder bending strength and gross section stiffness (MOE × MOI). This relationship is based on test data of full scale round timbers for native Australian hardwood, taking into account the reduction in the section properties due to timber decay. This relationship was then used in a reliability-based model to predict the load capacity of a deck bridge from the stiffness data obtained from the dynamic frequency method, where frequencies of the deck bridge were determined when unloaded and with approximately 10% of the mass of the deck attached at the mid span (Samali et al. 2003).
6. POSSIBLE METHODS OF ENHANCING THE STRENGTH

6.1. Strengthening of Reinforced Timber Beams by FRP

Strengthening of timber beams by means of bonding fibre reinforced plastics strips (FRP) to the tension side of the timber beams enhances the strength and stiffness performance of the beam (Buell and Saadatmanesh 2005; Svecova and Eden 2004) and importantly changes the failure mode from a brittle failure on the tension side of the beam to a ductile failure on the compression side thereby increasing the safety of the structure. The FRP prevents tensile cracks from opening, confine the local rupture, and strengthen the defect resulting in overall higher system strength.

Figure 4 schematically depicts a strengthening scheme using U-shaped FRP to prevent shear failure and encourage compression flexural failure in retrofitted timber beams. The surface of the timber is cleaned, rough-sanded and then primed with a coat of resin to improve the bond between the timber and fibre sheets. Retrofitting using either carbon fibre reinforced plastic strips (CFRP) along the bottom tension layer or half wrapping U-shaped glass fibre reinforced plastic (GFRP) around the bottom of the beam (to eliminate shear failure in stronger beams) were found to increase the strength by as much as 70% to 100% of the existing capacity (Johns and Lacroix 2000). One of the benefits of carbon fibre composite material is that its modulus is higher (200–300 GPa) than that of timber (5–20 GPa) and timber is able to tolerate the larger strains needed for the composite action (Humphreys and Francey 2004).

The success of fibre composites to retrofit timber bridges relies heavily on the condition of the timber, and the ability of the fibre composite material to remain bonded to the timber long term under fluctuating humidity and temperature conditions. Further, the work done by Richie (2003) on glue laminated (Glulam) beam reinforced with GFRP indicates that the stiffness or strength would not decrease significantly under repeated fatigue loading. Due to negligible dimensional changes in the FRP and more measurable changes in the timber under the same environmental conditions, higher stresses would occur at the edges of the bond which could encourage delamination at the FRP-timber interface over time (Barbero et al. 1994).
6.2. Hybrid Composite-Timber as Replacement of Hardwood

An example of a hybrid composite-timber member is shown in Figure 5. This member is made of softwood, in the form of plywood or laminated veneer lumber (LVL) sections, with composite reinforcement infill modules to increase the strength and stiffness to a level equivalent to a high quality hardwood. The timber sections provide the shear capacity and control the overall beam dimensions. The composite reinforcement infill modules use a patented combination of composite materials which have a high modulus of elasticity (around 60 GPa) and a cross section sufficient to satisfy the bending strength requirements. The infill composite modules are bonded to the timber using a high strength epoxy adhesive.

Figure 6 compares the results of some relatively small-scale testing of hybrid composite beams with the behaviour of a solid F14 ply beam (constructed from the same size timber sections) and typical design values for similar beams made of high-quality Australian F27 and F34 hardwood (AS 1720.1, 1974). The composite beam demonstrates increased stiffness and strength, and the addition of the reinforcing modules results in a pseudo-ductile failure mode with significant warning of failure through cracking of the ply and associated large deflections.

In Australia, the hybrid composite-timber girders have been installed by the Road Traffic Authority (RTA) of NSW and the Rail Infrastructure Corporation of NSW either as replacement members for existing hardwood girders in old bridges, or as girders in new lightweight road and pedestrian bridges.

Figure 5. Schematic cross-section of hybrid beam (Heldt et al. 2005)

Figure 6. Hybrid composite and Australian hardwood beam tests (Heldt et al. 2005)

6.3. Composite Concrete-Timber Bridges

An alternative method for retrofitting old timber bridges is to pour a concrete slab over the timber deck to create a concrete-timber composite. Full composite action is possible if non-slip joints are used. A shear connection which does not allow any slip can be constructed by creating a slot in the timber and inserting and gluing a steel plate before the concrete is poured. Figure 7 shows the concrete-timber composite system with a reinforced concrete slab layer stressed in compression and timber ribs stressed in tension.

Various types of shear connection have been grouped in Figure 8 according to their relative stiffness. Connections in group (a) are the most flexible and provide approximately 50% of the bending stiffness of group (d) which provide full composite action. Connections in groups (a), (b) and (c) are arranged in ascending order of stiffness and permit some relative slip between the concrete and timber.

Composite systems using engineered wood products such as LVL or glued laminated timber members, when well connected to concrete, can provide up to three times the load capacity and a six fold increase in flexural rigidity compared to traditional timber deck-beam system (Makippuro et al. 1996). The deflection under dynamic loads will also be less in the composite bridge due to higher damping of the composite system (about 2% damping compared to 1% damping of timber bridges). Some disadvantages of the concrete overlay include, an increase in the dead load has long term creep and deflection implications, timber can deteriorate at the concrete-timber interface without adequate protection and visual inspection is more difficult, particularly for the...
Figure 7. Concrete-timber composite bridges (Ceccotti 2002)

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| (d) | 1 | 2 |

Figure 8. Concrete-timber connection types (Ceccotti 2002)

- (a1) nails; (a2) glued reinforced concrete steel bars; (a3) & (a4) screws.
- (b1) & (b2) split rings and toothed plates; (b3) steel tubes; (b4) steel punched metal plates.
- (c1) round indentations; (c2) square indentations; (c3) cup indentations and prestressed steel bars; (c4) nailed timber and steel plates.
- (d1) steel lattice glued to timber; (d2) steel plate glued to timber.
timber members supporting the concrete slab. This type of composite system has been used for over 30 years with no collapse or severe serviceability problem (RILEM 1992). A famous application of this system is the 85 m span, precast concrete-timber deck over the Mur River in Austria (Pischl and Schickhofer 1993).

7. SUMMARY
Timber due to its abundance and low cost was the dominant construction material for bridges in the late 19th and early 20th centuries. However, most of the timber bridges are now ageing and more than 80 years old, and consequently there is a need for an effective strategy to improve the durability and performance under existing and increased traffic load conditions. This strategy will involve on-going visual inspections, condition monitoring, analysis and maybe structural retrofitting. Non-destructive evaluation techniques applied first globally and then locally are an attractive procedure to assess the structural integrity and residual capacity of a bridge without compromising the overall condition and strength. The residual capacity of the timber bridges could be enhanced by placing a concrete slab over the timber deck and developing composite action between the two materials through appropriate shear connections. Alternatively, one could retrofit using advanced composite materials such as carbon fibre reinforced plastic or glass fibre reinforced plastics to enhance the strength capacity.

REFERENCES


Engineers Australia (2005). Victorian Infrastructure Report Card, Engineers Australia, Victoria Division, Australia.

