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AN OVERVIEW OF THE DEVELOPMENT OF ON-SITE ASSESSMENT FOR TIMBER STRUCTURES IN AUSTRALIA

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Abstract

The use of non destructive assessment techniques for evaluating the "health" and safety of existing timber structures has become a key focus of interest over the past 20 years for Engineers who act as the asset managers of public infrastructure - such as utility poles, buildings and bridges. Over this period considerable research has been taken throughout the world using a variety of technologies to accurately identify the effects of aging and degradation of structural elements. There are usually two considerations that must be held in tension in quantifying these effects; the first seeks to keep the structure in-service for as long as possible in order to maximise the economic life of the asset, whilst the second focuses on the imperative of maintaining public safety.

This paper presents an overview of several research and development projects undertaken in Australia over the past 15 years that have contributed to the current "state of the art" of non destructive on-site assessment of timber structures, and discusses some of the issues associated with their implementation and acceptance.

1. INTRODUCTION

As a nation, Australia has historically had the great fortune to be blessed with plentiful supplies of native hardwood timbers, many species of which are both strong and durable. As a consequence of this, the expansion of European settlement since 1788 relied heavily on utilisation of these timbers for developing transport links such as wharf and bridge structures and later timber utility poles used for transmission of electricity and communication lines.

As a consequence of this widespread use of timber, particularly in rural regions, Australia is now faced with an extensive network of small span bridges and utility poles that are aging and in need of careful management to ensure both functionality and safety. In order to meet these criteria, it has been essential to develop appropriate techniques for assessing the condition of the timber elements in these structures, and quantify the condition in terms of remaining life and satisfactory (safe) performance. The remainder of this paper will discuss development of these techniques through a series of research projects, with specific application to timber poles and bridges.

2. NON DESTRUCTIVE ASSESSMENT OF TIMBER POLES

2.1 Background - Poles in Australia

Timber Utility poles represent a significant component of Australia's infrastructure, which has often been taken for granted. There are an estimated 5 million timber poles in Australia's electricity distribution network – with a current net worth of about $11 billion. At present, the average usage of power poles in the eastern states of Australia, is approximately 60,000 per annum, with a total annual delivered to depot cost of approximately $20 million. Most of these poles are timber.

Investigations of asset management systems for the power distribution industry indicate that the design and assessment methods for utility poles, which form traditional industry practices in Australia, are imprecise and often unreliable. Lack of reliable assessment techniques creates problems for poles in service, since the power supply industry needs to maintain a high level of reliability against pole failure, whilst extracting the maximum value from these assets. In order to achieve this asset management "balance", an effective assessment system should ideally be capable of:

- correctly identifying any pole which is likely to fail, due to inadequate strength.
- only infrequently condemning poles which have adequate strength.

Since the 1980's most network managers have relied upon assessment systems which presume that the remaining strength of a pole is proportional to the modulus of the cross section of the sound wood surviving in the critical plane (which in Australia is assumed to be located at or just below the ground line).

The current industry "best practice" is to determine the section modulus of a pole by periodic drilling, to determine the extent of degradation and remaining sound wood. The annulus of sound wood is then assumed to be an average thickness and the section modulus "Z" is calculated, based on this assumption. The bending moment capacity of a pole at the ground line is the "Z" value multiplied by the characteristic strength of the wood in bending "f'_b" - commonly assumed to be a constant value of 80 MPa.

Unfortunately, both the variability of wood strength and the inherent errors in determining the section properties contribute towards a high level of uncertainty in predicting the bending capacity of a particular pole. Thus whilst the "section modulus" method was unquestionably a significant step forward from previous practices, it still has deficiencies. Recognition of these deficiencies and limitations has been the catalyst to identify alternative non destructive technologies.

2.2 Investigations and Assessment of NDE Technologies

It was within this context, that an R & D project was undertaken by the University of Technology, Sydney (UTS) between 1997 and 2001, with the specific intention of:

- Evaluating and comparing as many NDE devices/systems as possible, using the benchmark of current industry best practice as a reference.
- Establishing a database of physical properties (specifically bending strength and section modulus) from full scale testing of poles removed from service at the apparent end of their design life.

– Providing the NDE system developers with the unique opportunity of calibrating and developing their systems with the database for eucalypt hardwood poles.

The principal objectives of the project focused on assessing alternative devices / systems by comparing the reliability of their performance with that of the current "industry best practice" *Section Modulus* method, undertaken by industry inspectors. Specifically, the objectives sought to quantify the accuracy / reliability of each NDE system with respect to correctly classifying a pole, in terms of one or both of the following:

1. remaining bending strength
2. extent and location of loss of wood in a critical plane/zone (usually the ground line).

A third objective was also recognised. The absence of data to correlate NDE equipment outputs with real pole strength or condition information has hampered its acceptance and use. Thus the third objective of the project was to provide an opportunity for any of the NDE systems to correlate their systems with a database of known properties, derived from testing a significant population of eucalypt hardwood poles typical of those in service in NSW.

In order to provide an accurate and objective "yard stick" for assessing the efficacy of any NDE system for meeting either of the first two objectives, it was considered necessary to undertake full scale destructive testing and detailed post-mortem analysis of each pole in the sections where the NDE readings were undertaken. Details and results of the physical testing undertaken on 350 poles are presented elsewhere [1].

It is essential to understand the difference between what is being "measured" by any NDE system and what is being inferred or assumed from that measurement. This is a similar principle to that used for grading timber, whereby strength properties are assigned to timber on the basis of a correlation between grading parameters (eg: minor axis stiffness, or visual characteristics such as knot size and location, slope of grain etc) and a particular property (eg: bending strength).

Sawn timber in an "as new" condition has the advantage that the "Z" value can be directly measured - the uncertainty associated with its determination is negligible and so the main area of uncertainty is its strength. The design strength properties are derived using statistical methods that consider the variability and uncertainty associated with a particular product and the level of confidence in the basis of derivation. For strength, these "characteristic" properties are generally based upon the 5^{th} percentile strength for the entire population of the particular grade (and size) of material.

When it comes to assessing the remaining strength of existing timber structures such as poles, which have been in service for many years, the level of uncertainty is considerably greater and the problem of determining both geometric and strength properties is much more complex. The section itself is not regular, and in many situations may be affected by splits, decay pockets and internal piping. Most of these effects will not be visible or easily quantified by visual inspection. The strength properties will also be effected - not only are they influenced by the inherent variability of timber described above, but strength degradation will have occurred due to loading history as well as biodegradation effects.

2.3 Types of NDE Systems

The various NDE systems / techniques assessed in the project [2] are described below in Table 1. The salient feature of most, is that that they focus on determination of the section geometry at the ground line – which is generally deemed to be the critical section. In other words, the principal correlation parameter for the NDE "measurement" is the geometric properties of the pole at a particular location - usually the section modulus "Z".

Table 1: List of NDE Systems Participating in the R&D Project

NDE system	Principal Measurement Technique	Pole Assessment Parameter
AcoustoSCAN	Ultrasonic signal	section modulus Z
AMAC (see note)	Sonic Resonance / frequency analysis	section modulus Z
Andec / CLUE (see note)	Sonic Resonance / frequency analysis	section modulus Z
Attar	Acoustic emission / Load application	pole strength – GL bending capacity
Curtin Rake - section modulus	physical measurement of "sound" wood in drill hole	section modulus Z
Deuar 1 (see note)	Load application - proof loading for strength	pole strength – GL bending capacity
Deuar 2 (see note)	Load application - deflection proof load	pole strength – GL bending capacity
DK Tector	Sonic Resonance / frequency analysis	section modulus Z
Foley	Ultrasonic Tomography	section modulus Z
Integrated Imaging (see note)	Radiographic Imaging	section modulus Z
Integrity 1	Sonic Resonance / frequency analysis	EI & section modulus Z - at GL
Integrity 2	Sonic Resonance / frequency analysis	EI & section modulus Z - locations other than GL
Metriguard	Sonic transmission / Stress wave timer at selected location	pole strength – GL bending capacity
Polux	Load application (low intensity) via probes	pole strength - fibre bending strength at GL
PortaCAT 1	Radiographic Imaging – fine resolution	section modulus Z
PortaCAT 2	Radiographic Imaging – medium resolution	section modulus Z
PortaCAT 3	Radiographic Imaging – coarse resolution	section modulus Z
PURL	Ultrasonic Tomography	section modulus Z
Resistograph 1 (inexperienced operator)	physical measurement of "sound" wood in drill hole	section modulus Z
Resistograph 2 (experienced operator)	physical measurement of "sound" wood in drill hole	section modulus Z
Resotest (see note)	Sonic Resonance / frequency analysis	section modulus Z
Section Modulus - Inspector	physical measurement of "sound" wood in drill hole	section modulus Z
Section Modulus - LOGIN	physical measurement of "sound" wood in drill hole	section modulus Z
Shigometer	measurement of electrical resistance in drill hole	section modulus Z
Sibert	physical measurement of "sound" wood in drill hole	section modulus Z
Sounding - modulus	Sonic Resonance (subjective)	section modulus Z
Tracerco ver1	radiographic densitometer	section modulus Z
TRU-TEC (Tracerco ver2)	radiographic densitometer	section modulus Z
XP-1 (see note)	radiographic densitometer	section modulus Z

Note: These systems were not assessed in the final report, either due to inconclusive results or inadequate data

A few systems actually focus on prediction of the bending capacity, but none actually measure it directly – since to do this, one would have to break the pole! Thus the principal reference point and calibration parameter for assessing the efficacy of most of the NDE systems used in this project is an accurate determination of the section modulus at critical sections, where the NDE readings are being taken. For those systems where prediction of pole strength is the principal parameter, actual bending strength and / or stiffness at the ground line (determined from full scale destructive testing) was used as the assessment benchmark.

2.4 Basis of Rating the NDE Systems

The method for determining the "actual" section modulus for each of the poles involved cutting the ground-line segment from each pole and creating a digital image of each face of the segment – which nominally is the location where NDE assessment was also undertaken. This digital image was then analysed using a specifically developed finite element program for determination of the centroid with respect to reference axes, enabling the second moment of inertia to be calculated. The two values of the section modulus about each principal axis were then derived and the average for each Z_{xx} and Z_{yy} determined. The experimental error involved in this process is estimated to be approximately ±2.5% i.e. a 5% error band.

For the reasons previously noted, the primary calibration or reference parameter for assessing the efficacy of most of the NDE systems was the section modulus in the ground line region. However, since prediction of remaining strength is of vital interest to Asset Managers, fibre strength (f_b) models were also developed to predict the GL bending strength of the poles using the NDE determined values of "Z", and these predicted values were compared to the actual bending capacities at failure determined from full scale testing.

Analysis of the final data submitted by the NDE practitioners was undertaken as follows:

1. The ratio of the NDE value / the digital value for each relevant section property (geometric) was determined. In the few cases (Metriguard, Polux and ATTAR) where the data presented is the predicted bending capacity, the actual bending capacity derived from the full scale pole testing was used as the reference parameter for comparison.

2. The ratios of predicted / actual, were grouped into five categories:

- ratio < 70%
- ratio 70% to 90%
- ratio 90% to 110%
- ratio 110% to 140%
- ratio > 140%

3. The frequency of each group was determined.

4. The ratio groupings were analysed using basic descriptive statistical processes to quantify averages, variability and confidence intervals.

5. The statistical data was then used to rate the respective systems according to criteria such as: average accuracy, degree of variability, least squares regression for data "fit", reliability (and extent of conservativeness), ability to detect very weak or rogue poles where failure is probable, and cost implications of either early removal of a sound pole or failure to detect a rogue pole.

6. A sensitivity analysis of the rating system was completed, comparing a "weighted" with and "unweighted" assessment; with the weightings selected to highlight variability and uncertainty influences in the assessment process.

The ratio categories represent "bands", which are indicative of the efficacy of a system to accurately and reliably predict the section properties of a particular pole. If a particular method is very conservative, it will have a high frequency of results in the lowest band, if unconservative (i.e.: it over predicts "Z") it will have a high frequency of results in the upper band and if "accurate" a high frequency of results in the central band.

2.5 Results, Conclusions and Future Developments

A summary the comparative assessment is presented below in Table 2. The results indicate that systems utilising fine resolution tomography or radiographic imagery techniques provide a significant increase in accuracy and reliability for determination of the section modulus, when compared to traditional drilling methods.

Table 2:Summary of Comparative Assessment of NDE Pole Systems

System Provider:	Zave (below)	Rank Zave	Rank Zxx	CATEGORY
PortaCAT 1	> 80%	1	2	A
TRU-TEC	> 80%	2	3	A
Foley	> 80%	3	1	A
LOGIN	Approx 80%	4	4	B
Resistograph 2	Approx 80%	5	5	B
Resistograph 1	Approx 80%	6	5	B
Inspector	Approx 80%	7	5	B
PortaCAT 2	< 80%	8	12	C
Sibert	< 80%	9	8	C
Sounding	< 80%	10	10	C
DK Tector	< 80%	11	9	C
Shigometer	< 70%	12	11	C
Curtin	< 70%	13	13	C
PURL	< 70%	14	14	C
AcoustoSCAN	< 60%	15	15	C
Integrity 2	< 60%	16	17	D
PortaCAT 3	< 60%	17	16	D
Tracerco 1	< 60%	18	18	D
Integrity 1	< 60%	19	19	D
Attar (inconclusive)	N/a	20	20	D

Since the project was completed in 2002, several of these systems have been developed further to consider the relationship between the section modulus and the strength of the timber in the pole being assessed. These developments have primarily focused on utilising results of surface hardness or density, such as can be estimated using Pylodin.

However, one drawback of all of these systems is the reliance on assessment at a localized section (usually the ground line). In order to address this limitation, a new research project for determination of both the location and size of defects in a timber pole and has recently commenced at UTS, utilising a combination of stress wave analysis, model strain energy, time-frequency analysis and artificial neural networks. Whilst considerable obstacles still need to be addressed, initial results confirm the potential of this "hybrid" system to provide a reliable means for whole of pole assessment, rather than localised analysis at a particular section.

Details of research undertaken to date that contributes to the development of these techniques are presented elsewhere in these proceedings [3, 4]. The proposed strategy for use of this hybrid system is to develop it as a screening tool that will quickly confirm all sound poles, whilst identifying any poles that appear to contain defects for more detailed assessment using a combination of imaging and strength prediction technologies noted above.

3 NON DESTRUCTIVE ASSESSMENT OF TIMBER BRIDGES

3.1 Background – Timber Bridges in Australia

Recent surveys indicate that there are in excess of 40,000 bridges in Australia. An estimated 27,000 of these are timber bridges, with approximately one third of these in excess of 50 years old [5]. These are located predominately on regional roads, owned and maintained by local government agencies, which are often under resourced and lack the engineering expertise to undertake strength assessment of their bridge assets. Consequently a significant proportion of these bridges are functionally obsolete and structurally deficient to varying degrees. The most common form of construction for these regional timber bridges uses round girders fabricated from native hardwood spaced at 1000mm to 1500mm centres, transversely overlaid with decking planks. Typical spans are between 7m and 12m.

Currently, reliable determination of the structural condition of a short to medium span bridge requires undertaking a load test involving the application of a heavy vehicle, representative of the design load, and the measurement of the deformations caused by this applied load. A number of state Road Authorities have undertaken various forms of load testing, ranging from the application of representative service loads, to quantifying responses to proof loads that are 2 to 3 times the magnitude of normal design vehicles. Such testing is costly to conduct, requires specialist instrumentation, acquisition equipment and where significant degradation has occurred, can potentially cause damage the bridge being assessed. Furthermore, the expertise required to undertake this type of testing is unavailable to most local government authorities.

In order to address this problem and develop cost – effective risk management systems for assessing safe load capacities of bridges (particularly timber bridges), a program of research and development was undertaken between 2000 and 2003, through the Centre for Built Infrastructure Research (CBIR) at UTS, details of which are discussed in Section 3.3.

3.2 Techniques for Condition Assessment of Timber Bridges

Traditional load testing involves placing a vehicle (usually a legally loaded truck) with known axle loads and fixed geometry, onto a bridge deck and measuring the deflections and / or strains of the girders – usually at mid span. Deflection or strain data is interpreted to indicate the stiffness of the deck system – with low stiffness indicating that the girders may be degraded and thus weak. In recent years, a number of alternative techniques that do not involve the application of heavy vehicles to assess structural adequacy have also been trialled. The relative benefits and limitations of various test methods are summarised in Table 3. All of these methods require converting "stiffness" information into "strength" in order to determine the actual safe load capacity of a bridge. It is therefore important to note that the effectiveness of all methods is limited by the reliability of the strength model used.

Predicting the strength of bridge elements and thus the safe load capacity is generally not a significant problem for concrete or steel bridges (unless the steel or concrete has undergone significant degradation), but for timber bridges the level of reliability of such a load test will be directly dependent upon the accuracy of the strength model being used. The inherently variable nature of timber (both new and old) necessitates the use of a probabilistic strength model, usually based on 5th percentile strengths for a given grade or population of timber elements. The most reliable strength models are probabilistic (statistical) that have been developed by extensive full scale testing of old timbers and are based upon empirically derived relationships between the actual strength of girders and the parameters measured during the load test.

Table 3: Summary of Load Testing Approaches for Timber Bridges

Method	Application and Comments
Static Deflection methods	Provide good indication of "global" serviceability response and load distribution between girders. Has the benefit that it can be undertaken using survey equipment. Can be time consuming.
Dynamic Deflection methods	Provide good indication of "global" serviceability response and load distribution between girders. Can also indicate dynamic load amplification effects. Requires specialised equipment.
Strain methods	Provide good indication of relative load distribution between girders. Dynamic strain measurements have the advantage that peak loads and distributions can be identified, provided the instrumentation is located correctly. Has the limitation that the results are highly localised and may not indicate global response accurately. Requires specialised equipment.
Ground Penetrating Radar (GPR)	Can provide a longitudinal profile of each girder to identify localised degradation. Must be coupled with analytical modelling or another test method to understand structural response of a bridge. Requires specialised equipment which is very expensive and is still regarded as a research tool.
Dynamic Frequency methods	Provide good indication of "global" serviceability response of superstructure. Limitations are dependent on assumptions made by some methods about mass of the deck, input force and identification of frequencies. Can be quick to perform and are normally cost effective.
Dynamic Frequency methods with damage detection	Provide good indication of "global" serviceability response of superstructure and can identify both location and size of defects in all girders. When used with known impact force and applied modal mass, provides the most accurate information for any non destructive method currently available. However further development is required for field application.
Proof Loading	Requires extensive instrumentation and special vehicles to a level usually 2 to 2.5 times the legal axle load. Totally unsuited to timber bridges and can result in undetected damage occurring that could subsequently have catastrophic results. Very expensive.

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3.3 Development of Frequency Based NDE Techniques

The aim of the UTS project was to research and develop an accurate and reliable method for assessing the structural condition and predicting the safe load capacity of timber bridges, based on analysis of the fundamental frequency of the superstructure to determine the global stiffness of a bridge deck. It should be noted that whilst the idea of using dynamic frequencies to determine stiffness of a structural element is not new, the scope of this research project contained a number of unique features, specifically designed to address uncertainties and a lack of reliability inherent in previously developed systems.

Historically, use of Frequency Response Function (FRF) methods for the assessment of defects is a well established and documented approach across aeronautical, mechanical and civil engineering disciplines. However, it is only in last three decades, especially after the 1980's that this method has evolved and matured; in recent times quite rapidly. The pile testing technique proposed by Davis & Dunn [6] is claimed to be the theoretical "backbone" of a number of assessment systems used in Australia and elsewhere. This technique was innovative at the time it was developed in 1970's when vibration based damage detection approaches first began to emerge. However, when compared with the latest developments in the area, these early developments were greatly simplified and applicable only to idealized situations or very simple cases.

Haritos presents a further specific development of dynamic frequency methods for determining the stiffness of a bridge deck system [7]. The technique used vibratory response modes produced by traffic on a number of different bridge superstructures to estimate the effective gross stiffness (EI), by matching the observed lower mode frequencies with theoretical values obtained from a simple beam model constructed using finite element techniques. Haritos recognised the importance of providing a "controlled" dynamic force excitation for determination of frequencies, mode shapes and damping from an analysis of the modal responses. Although it was recognised as both costly and bulky, a linear hydraulic shaker system was considered to provide the greatest flexibility in achieving this controlled excitation.

Subsequently, a simpler form of excitation was sought as an alternative to the shaker system, involving the measurement of vibrations caused by traffic to determine resonant frequencies; the idea being that one uses the output signal to calculate the input force and signal characteristics. A variation of this method using velocity transducers is advocated by at least one provider of bridge assessment services in Australia, and is purported to provide accurate stiffness information on concrete, steel and timber bridges [8].

3.3.1 Recognition and Addressing Limitations

However, use of an output signal to calculate the input signal, leading to calculation of Frequency Response Function (FRF), can be erroneous and unreliable - based upon the following observations. The input (impact force) is not merely a function of mass and acceleration. This is an over-simplification and only under idealized conditions (e.g. rigid body assumptions) can this simplification be considered valid, which is not the case with most bridges, particularly those comprised of damaged or degraded timber. The reliability of analysing any vibration based excitation lies in being able to measure the input force accurately. Therefore, it is doubtful that an accurate estimate of the input force can be achieved without this force being carefully measured, such as by the use of a modal hammer with in-built force sensor [9].

For a given response (output) signal, one cannot practically separate the impact force and free vibration part of the signal without some pre-assumed knowledge. Unfortunately, in order to calculate the input force accurately, the impact contribution has to be separated from the free vibration component of the response signal, and this creates significant potential problems for instrumentation and the type of sensor used to capture the vibration signals. The modal analysis inherent in the vibration based excitation previously noted uses what is known as a "peak-picking" method to identify the modal frequencies. Whilst in theory this approach has merit, it can present significant uncertainties and sources of error in practice – particularly when applied to a timber bridge. Higgs and Tongue [8] note that some of the peaks occur at frequencies which correspond with natural frequencies of the superstructure whereas others are not so readily identified and may be caused by vehicle suspension dynamics, but they do not elaborate on how to distinguish between the two.

Furthermore, using a peak-picking method to identify the correct frequencies induced by vibration is unlikely to be reliable because of a lack of control over the modal magnitude where many mode frequencies with possible coupling are present. Simply using a peak-picking based method without proper modal identification may result in incorrect selection of modal frequencies. Putting it simply, vibrations from vehicular traffic will generate a broad based spectrum, with all frequencies being excited, but none being emphasised. The impact sensitivity will also be influenced by the type of vehicle and suspension as well as the type of structure being assessed.

It is acknowledged that the use of output-only signals for modal analysis has advanced in recent years, such as using ambient vibrations as the source of excitation or Natural Input Modal Analysis [10]. However, it is the experience of the UTS research team that using a peak-picking based method to correctly identify modal frequencies, especially when broadband excitation is applied, is likely to be highly erroneous and will exhibit a lack of repeatability. In such cases the FRF will be complicated and in fact the potential for error is significantly increased when damage and unknown or unquantifiable boundary conditions are present. All of these uncertainties contribute to peak-picking methodologies having the potential to be unreliable for determining modal frequencies for many types of bridges – especially for degraded timber bridges.

3.4 Development of a Reliable Frequency Response Function Method for Timber

It needs to be stressed that the quality of FRF is critically important to any assessment method purporting to predict stiffness and subsequently the strength of a bridge deck. Unless high quality FRF can be obtained consistently and accurately, the whole method will lack practical usefulness. This is particularly true of timber bridges which also often have higher levels of inherent noise as a result of "looseness" in the deck and movement between decking elements and the super-structure girders, making it much harder to distinguish critical frequency modes.

Awareness of the limitations of not accurately knowing either the input force or the mass used to calculate modal frequencies, were two specific areas targeted by the author's team at the commencement of the UTS research project. This approach has also been adopted by similar research undertaken independently at the Pullman campus of Washington State University in the USA [9].

The UTS assessment procedure known as Dynamic Frequency Analysis (DFA) involves the attachment of accelerometers to each of the bridge deck girders. The vibration response and natural frequency of the bridge superstructure is measured when a "calibrated sledgehammer" (modal hammer) is used to hit the unloaded deck, and then again with a relatively small (known) mass applied at mid-span. The difference in response is used to determine the stiffness of the deck superstructure, from which the load carrying capacity of the bridge is then calculated. Vibrations are measured with uniaxial accelerometers that are robust and simple to attach for the unloaded deck and with one or more loads equivalent to approximately 10% of the mass of the deck applied at midspan.

The data is logged and the bridge deck properties evaluated, using a dynamic signal analyser. Two sets of bending frequencies are measured for the bridge, 'as is' (ω_1), and 'when loaded' (ω_2) by the extra weight. By loading the bridge, the bending frequency of the bridge decreases. The resulting frequency shift due to added weight (ΔM), enables the flexural stiffness of the bridge can be calculated, using equation 1.

$$k = \frac{48EI}{\alpha L^3} = \frac{\omega_1^2 \omega_2^2}{\omega_1^2 - \omega_2^2} \Delta M \qquad (1)$$

For timber bridges the safe capacity predictions are made probabilistically using strength models derived from extensive testing of aged timber girders and laboratory investigations of corbel / girder interactions. This test data has been used to derive a valid statistical relationship between actual measured stiffness (EI) and actual bending capacity. Using a probabilistic approach, this relationship can be used in reliability-based models to predict the load capacity of a deck from the stiffness data obtained from the dynamic frequency method, with acceptable and transparent degrees of uncertainty. This research and analysis has been used to develop some proprietary algorithms that are now incorporated into the DFA analysis software.

During the R&D phase up to July 2003, over $0.5m (AUD) had been expended and a total of 69 bridges / 127 spans had been successfully assessed using the new methodology. At the time of writing some 300 bridges and 700 spans have now been assessed using this method.

3.5 Further Developments

The DFA method is essentially based on determining global response parameters that relate to the system behaviour of a timber bridge deck. However, one of the limitations of the DFA method is the fact that whilst it provides an accurate estimation of the global stiffness of a bridge superstructure, it is not reliable for identifying the specific location and / or size of a defect in a particularly bridge beam or girder. In order to address this limitation a number of subsequent projects have been commenced, with the aim of developing and implementing dynamic methods for evaluation of localised damage in timber bridges. Details of one recently completed project that has made significant advances in this regard are presented by Choi et al [3], elsewhere in these proceedings.

4 CONCLUSIONS

This paper has presented an overview of two key developments in the use of non destructive evaluation technologies to assess the structural integrity of timber poles and bridge superstructures. These research projects have demonstrated that NDE technologies such as accurate imaging for assessing critical ground line sections in poles or piles and frequency analysis methods for bridge decks have been established as the "new generation" of assessment techniques for quantifying safety and adequacy of these important infrastructure assets. Implementation of these two technologies is already realising a significant improvement in cost efficient management of these assets – particularly in regard to risk management and public safety.

On going research projects are refining these techniques and improving their reliability. NDE technologies such as these are providing a new generation of essential tools for asset management of timber structures that enable public utilities to maintain aging infrastructure in a cost effective, whilst at the same time ensuring public safety.

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OVERVIEW OF DY
TIMBER BRIDGES

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Abstract

Asset management of
of aging infrastructure an
nor sustainable in many
and utilise state-of-the-a
bridges and to be able to
safety. This paper presen
implementing several dy
technique of detecting da
in the literature as damag
single and multiple dama
laboratory conditions. It
was well suited to detect
For the laboratory bridge
successful in detecting sin

Keywords
Damage evaluation, moda

1. INTRODUCTIO

Timber bridges have
Besides serving as means
heritage values. Accordin
(DOTARS) [1], there ar
estimated that a third of t
are functionally obsolete