University of Southern Queensland
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Using Renewable Plantation Timber as a Replacement Option for Unrenewable Hardwood Girders in Bridges

A dissertation submitted by

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towards the degree of

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Abstract

This project investigates the possibility of integrating reinforcement with softwood glulam (glue laminated) timber beams to provide an economic alternative to hardwood timber girders. The resources of hardwood timbers are dwindling due to a reduction in tree felling, and hardwood timber plantations are unable to keep up with demand. Due to this growing shortage, there is an increased need to find a suitable replacement for deteriorating hardwood timber girders on rural bridges throughout Queensland. This project analyses a possible alternative. From a design stand point, the beams should cater for the requirements specified by the Department of Main Roads.

An experimental approach has been taken to study the effect of the integration of steel reinforcing within the laminates of a glulam timber beam. This design will be tested for strength using a four-point bending arrangement with two hydraulic jacks providing force on the member.

The results of the test showed the beam catered for the requirements and specifications outlined by the Department of Main Roads up to 660kN. However, while the beam supported the load, the tested Modulus of Elasticity (MoE) fell outside the range of these specifications. It is considered that this could be easily rectified following minor changes to the design.

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ENG4111 Research Project Part 1 &
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Limitations of Use

Certification

I certify that the ideas, designs and experimental work, results, analyses and conclusions set out in this dissertation are entirely my own effort, except where otherwise indicated and acknowledged.

I further certify that the work is original and has not been previously submitted for assessment in any other course or institution, except where specifically stated.

Matthew Lorne Figg

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Date

Acknowledgements

I would like to thank John Muller of Tasbeam for his persistence and management during the overall process involved in undertaking this project. I would also like to thank Kevin Covey for his experience and guidance in designing and analysing the beams.

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INTRODUCTION

1.1Background Information

Hardwood timber is a material that has been used in the field of bridge construction in Australia for many years, predating the first half of the 20th century (Walter 1996). A natural resource, hardwood timber is an all-purpose construction material which is able to hold sufficient capacity without fabrication or altering of the member. For this reason, it is commonly used as a bridge girder, particularly in small span rural bridges. Large diameter Australian hardwood timber has typically been used as components in bridge structures, however, this has become increasingly difficult, as supply cannot meet demand. Due to public pressure to preserve natural forests and the resulting decline of physical resources, several asset owners from federal through to local governments are now sourcing an alternative to hardwood timber girders.

While concrete and steel are the most commonly used structural elements for the construction of bridges throughout the world, beams made of these materials are generally not suited as an interchange material for rural bridge girders as they have markedly different properties than that of timber members (Cowan 1988). Timber is typically elected as a material for rural bridge construction due to its cost and transport benefits. There have, however, been some successful attempts in the development of alternative hardwood timber girders incorporating materials such as fibre composites (Buell & Saadatmanesh 2005; Heldt, et al. 2004; Ritter, Williamson & Moody 1994). Fibre composites consist of polymers (plastics) reinforced with carbon, glass, and/or amarid (Kevlar) fibres, and are only a fraction of the weight of steel, concrete and timber, as well as being stronger in terms of bending capacity. However, fibre composites are inferior to these materials with respect to deflection and cost (Department of Main Roads 2006). This project aims to investigate the appropriateness of a further substitute. With many Queensland timber bridges requiring upgrading or replacement within the foreseeable future, and in light of the public's negative perception associated with forestry logging, it is thought that further alternative options would be embraced.

1.2 Aims of this Project

The project seeks to provide renewable timber girders in the field of bridge construction by researching the relationship between reinforced steel and softwood plantation timber, in the form of glue-laminated (glulam) timber. It is proposed that the addition of reinforcing steel will increase the flexural capacity of glulam girders and achieve minimum requirements as provided by the Department of Main Roads in Queensland, or the equivalent of the presiding hardwood. This project is considered to be valuable due to the lack of hardwood timber stocks currently available for this specific use in Australia.

1.3 Overview of this Dissertation

Chapter 1: Provides some background information and insight on the requirements of timber bridge girders and the necessity to find an alternative of the hardwood girder so commonly used. This chapter also includes the main objectives of the project.

Chapter 2: Reviews available literature on the use of hardwood girders and glulam timber construction.

Chapter 3: Discusses the adopted experimental procedures used in order to gain the necessary data.

Chapter 4: Analyses the results obtained from gathered data.

Chapter 5: Analyses the cost and discusses the economics of whole life construction.

Chapter 6: Contains conclusions obtained from tests carried out, and any comments on any further work, which may need to be done to further satisfy asset owners for the use of glulam timber girders.

CHAPTER 2

LITERATURE REVIEW

2.1 History of Timber Bridges

The earliest recorded timber bridge in Australia was the Bridge Street bridge constructed in 1788 in Sydney (Botany Bay) but was washed away soon after construction (O'Connor 1985). As many as ten timber bridges were recorded in the Sydney area by 1805, none of which remain today (O'Connor 1985). The first recorded laminated bow arch bridge in Australia was constructed between 1870 and 1900, but developed many problems as a result of separation of laminates due to the large amount of shrinkage in Australian hardwoods (O'Connor 1985). The shrinkage left gaps that enabled fungal and termites into the structures. These bridges failed because replacement of individual laminates was not possible. The more common timber bridges, some of which are still standing today, evolved early in the 20th century. By this stage many settlements had spread all over the country, with road and rail network evolving to cater for the transportation of goods, hence a large number of bridge structures were required. The timber for these structures was sourced and milled locally and bridges constructed by the local communities (O'Connor 1985). Hardwood timber girders were available, cost efficient and were able to carry high loads.

2.2 Necessity for alternative hardwood girders

Prior to the 20th Century, hardwood timber was the major material used for both highway and railroad bridges. The development of steel and reinforced concrete provided other options, and these have become major bridge building materials (Walter 1996). However, there are still a significant amount of small span rural bridges throughout Queensland, which are currently in operation that utilise hardwood timber girders, as shown in Figure 2.1.

Figure 2.1*: An existing rural bridge in use in Queensland with a typical hardwood girder.*

The Department of Main Roads currently owns and maintains approximately 475 timber bridges across rural Queensland, which vary in condition. These bridges are inspected in accordance with DMR guidelines which assess several components of each bridge using a 4-point rating scale, where 1 is representative of “good condition” and 4 signifies “very poor" condition. At present there are approximately 750 girders that are in condition state 4, and therefore require replacement. Additionally, there are 1100 girders in condition state 3 which will also require replacement in the near future (Department of Main Roads 2003).

The spans of the girders currently used generally range from 5.2 metres to 9.1 metres. However, there are a small number of outliers down to 3.0 metres and up to 10.7 metres in length, which would also require servicing (Department of Main Roads 2003). Hardwood girders of the required diameter and quality are rapidly becoming difficult to obtain due to cost, availability and poorer general quality than has previously been the case, and thus the need to investigate other methods of replacing girders. This study will explore renewable plantation timber as a potential alternative. Plantation timber can be fabricated into an engineered dimension member known as glue-laminated (glulam) timber.

2.3 Background of glulam timber

Glue-laminated (glulam) timber refers to large, structural members, which are made by gluing pieces of dimension timber together (Boughton & Crews 1998). It is an engineered, stress rated product of a timber laminating plant (Ritter, Williams, & Moody 1994). Glulam can be formed into many curved shapes, and the sizes are limited only by transport restrictions. It is generally used for columns and beams, and frequently for curved members loaded in combined bending and compression (Boughton & Crews 1998). Glulam timber is also used as an exposed architectural product, or it can be hidden or left unfinished to only serve in a structural role.

The materials used for the manufacture of glulam timber are a special grade. It is dried to maximum moisture content of 15 percent, and planed to a closer tolerance than generally required for dimension timber (Australian Standard 1998). The pieces are end jointed and then arranged in horizontal layers or laminations with the location of the pieces determined by the strength requirements. For example, a beam made for a single span will have the highest quality timber situated at the bottom where tension is greatest.

Glulam timber is a structural product used for headers, beams, girders, columns and heavy trusses. Glulam timber has a number of advantages over solid-sawn hardwood timber. One such advantage is that glulam timber can produce deeper, wider and longer members. It can also easily fabricate cambered, curved, and tapered configurations, and can use lower-grade timber in lower-stress zones of the member, resulting in minimal waste and subsequent conservation of the timber resource. Additionally, seasoning the laminates lead to less member deformation resulting in the structure experiencing less distress (National Association of Forest Industries 1989). These advantages have paved the way for glulam timber to replace hardwood trusses and beams.

A defined deficiency with glulam timber members is their long-term creep deflections due to sustained load, which lowers their flexural stiffness (Tasbeam Pty. Ltd. 2004). Creep is defined as 'a gradual change in shape under stress' (Kent 1998, p.117). This limitation can be compensated by using larger members or by engineering the member to more efficiently carry the applied loads. One form of engineering timber to do this is the application of reinforcement or composite.

2.4 Reinforced/composite beam construction

One method of composite that has been extensively researched is the use of Fibre Reinforced Polymers (FRP). Dolan, Galloway, and Tsunemori (1997) investigated the addition of high-performance fibres between the laminations of glulam timber beams, and increased flexural capacity of the member. These results indicate that using small volumes of pretension Kevlar yarns increases strength and stiffness to a lesser degree of the beam. However, Stephens and Criner (2000) carried out the economic analysis of Fibre Reinforced timber and found FRP to be an expensive alternative of reinforcing, taking into account the market value and current demand of the product. Due to budget restraints of many agencies responsible for the maintenance and replacement of bridge girders, it is not feasible to replace every deficient bridge in this way. Various polymers researched to increase the performance of glulam beams were not feasible to utilise for many asset owners due to the associated cost.

Another option to be considered is the use of steel reinforcement. Steel reinforcement is currently used widely with concrete to form a very efficient structural member. Steel reinforcement in concrete enhances its bending capabilities, as concrete is very weak in tension which will cause cracking and failure under load (Warner, et al. 1998; Cowan 1988). Steel is predominately used for its stiffness, price and its ability to work as a composite material when added to concrete. Following the success of reinforced steel in concrete in the past, and the current widespread acceptance of glulam timber, the two combined as a composite material should make a cost-effective and structurally-efficient member which may come to serve as an alternative to hardwoods.

Steel reinforcement of the member is a proven technique for engineering timber currently used in the industry. A study by Bulleit, Sandberg, and Woods (1989) found that the use of structural steel in glulam timber enhances the stiffness, which aids deflection, as the composite material reduces the effects of relaxation, also eliminating long-term creep.

The proposed study seeks to investigate the effectiveness of steel-reinforced glulam timber beams and the potential for such beams to be used in the future to replace hardwood girders in bridges throughout rural Queensland. It is hypothesized that with the addition of reinforced steel creating a composite member with glulam timber, the flexural capacity and deflection of the member will be enhanced. By using reinforced steel in the bottom laminates, where tension is at its greatest, combined with the natural characteristics of timber, it is proposed that an efficient member should be produced providing the materials act as a composite.

CHAPTER 3

METHODOLOGY

3.1 Scope

This thesis will address the performance criteria as defined by the Department of Main Roads refer to Appendix D. Table 3.1 shows a brief summation of the required parameters to be met as defined by the Department of Main Roads.

Table 3.1: *Overview of the Department of Main Roads Specifications*

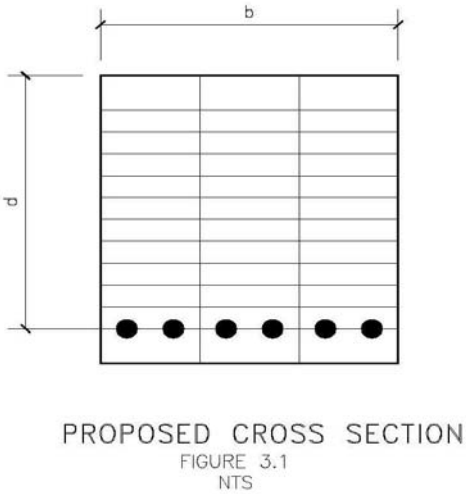
Maximum Dimensions	Width (mm)	350
	Depth (mm)	375
Mmin at Failure	kNm	518
Negative BM Capacity	kNm	30% of +ve Capacity
Max. Shear	kN	350
Max. Deflection at Failure	mm	120
EI of girder	Nmm^2	2.34×10^{13}

3.2 Beam design

Calculating the bending of members made of several materials is the initial starting point when carrying out the design of the beam. If the member subjected to pure bending is made of two or more materials with different moduli of elasticity, the approach to the determination of the stresses in the member should be modified from typical analysis. This beam, when subjected to a positive or negative bending moment, is supported by steel reinforcement placed in the bottom laminate (see Figure 3.1). While timber typically is not considered weak in tension, steel is significantly stronger in its ability to sustain its shape under high loads, thereby increasing the overall load capacity.

In order to obtain the transformed section of the test beam, the cross sectional area of the steel needs to be replaced by an equivalent area of timber (n*A_s refer Fig 3.2) (Beer &

Johnston, 2002). In the figure 3.2 *n* is the ratio of the steel modulus against the glulam timber modulus ($n=E_s/E_g$). It is assumed the steel will be acting independently of the timber when under pure bending. For this reason the steel will be the only material calculated below the neutral axis (N.A).



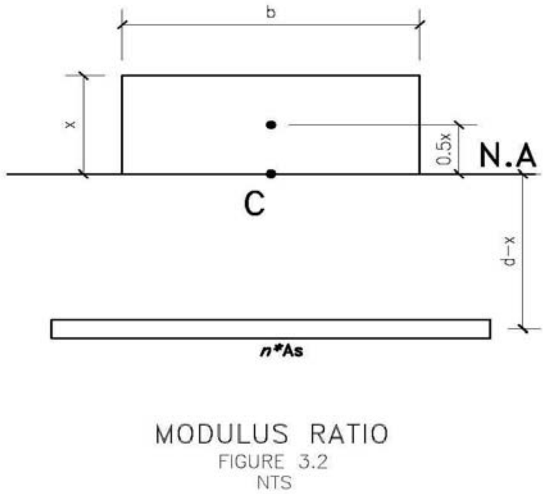
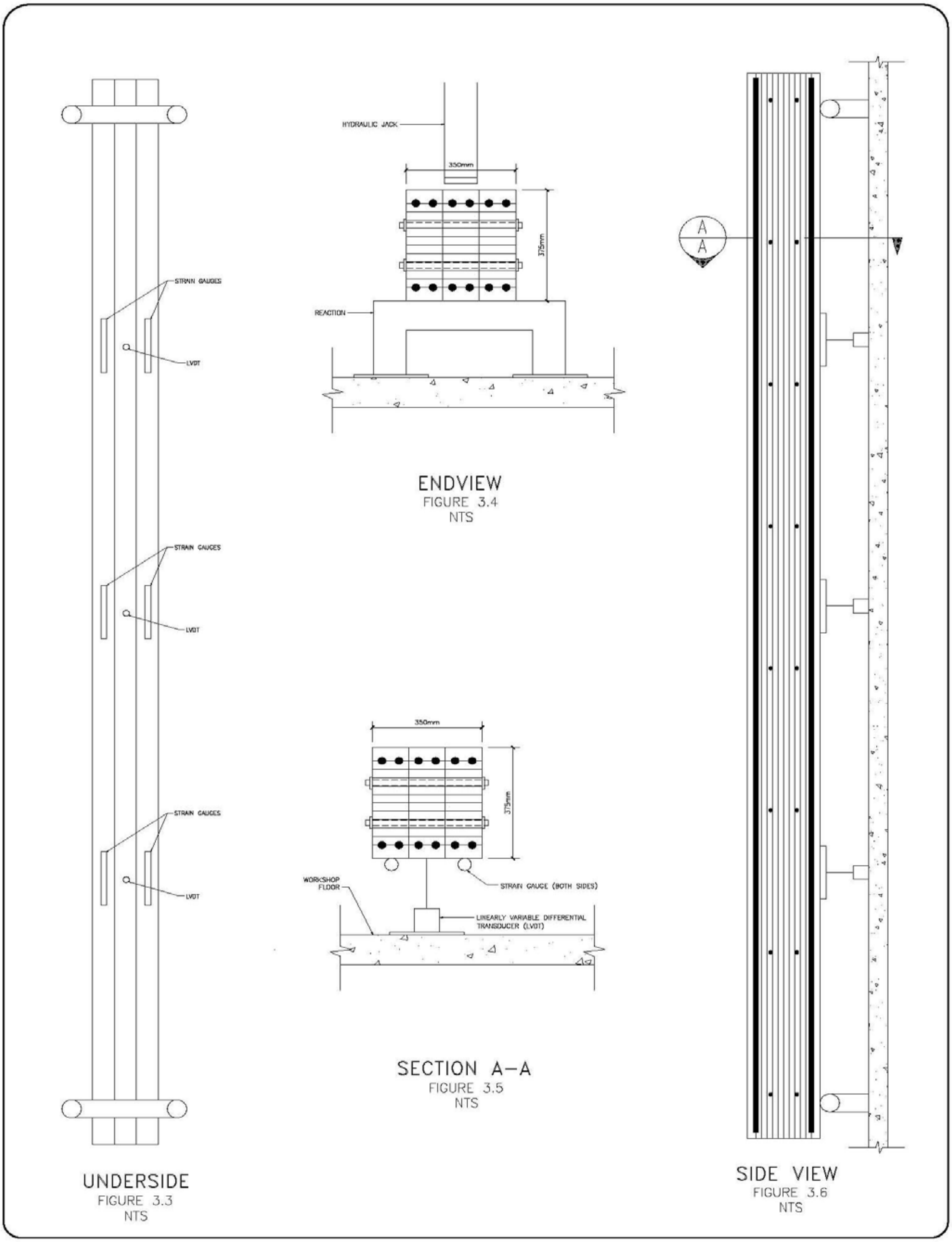


***Figure 3.1**: Proposed Cross Section* ***Figure 3.2**: Modulus Ratio*

The location of the neutral axis is calculated by determining the distance (x) from the upper face of the beam to the centroid (C) of the composite section (see Figure 3.2). This location is found by solving the following quadratic equation for (x) using first moment principles.

$$1/2bx^2 + nA_sx - xA_sd = 0 \text{ (Beer \& Johnston, 2002)}$$

Solving for (x) will obtain both the position of the neutral axis in the beam and the portion of the cross section of the timber beam that is effectively used. Following this, the modulus of elasticity and the respective timber and steel stresses can be calculated. Following the calculations and analysis, the beam displayed in Figures 3.3 through 3.6 was designed and fabricated.



Figures 3.3 to 3.6*: Beam Sections and Elevations*

3.3 Testing Configuration and Instrumentation

A four-point bending test configuration was adopted (see Figure 3.9) to ensure pure bending was acting at mid-span, while maximum shear was developed at the supports. It is considered important to ensure the beam fails due to bending force and not due to shear, as this is closest to the real life scenario of girders of this type in rural bridges. A three-point bending and shear diagram (see figure 3.7) shows that when under load there is still shear present at the centre of the beam. In comparison to this the four point bending diagram (see figure 3.8) shows bending is very similar in the mid-section for each scenario. However, the shear diagram in figure 3.8 shows no shear at the mid-section of the member, thereby using a four point testing configuration it is ensured pure bending acts at the mid-span of the beam. By ensuring pure bending it should give an accurate result of the bending moment capacity of the reinforced glulam timber beam.

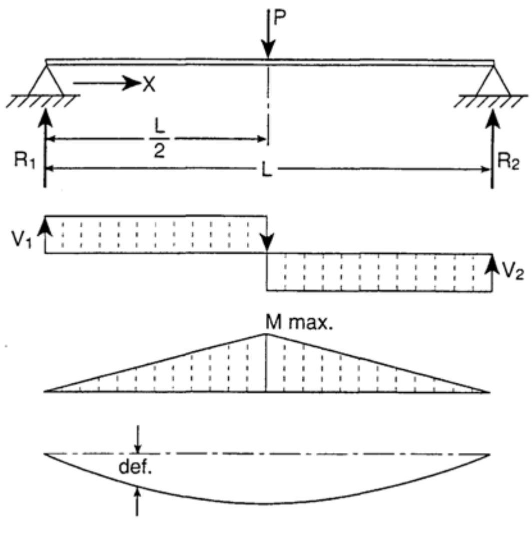


Figure 3.7 *Shear and Bending Diagrams (Three-Point Bending) (Australian Standard 1999)*

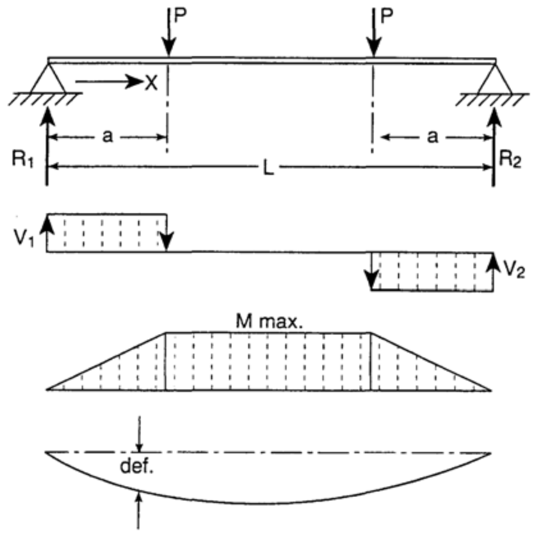


Figure 3.8 *Shear and Bending Diagrams (Four-point Bending) (Australian Standard 1999)*

The two hydraulic jacks were positioned 0.6m either side of the centre of the test span and equal loads were applied by each jack to the girder. The test configuration is shown in Figure 3.9.

Figure 3.9: *Four point bending configuration*

Nine testing implements were installed and used to capture data during testing (see Figure 3.3). Three Linearly Variable Differential Transducers (LVDT) were used to measure vertical deflection of the girder. Of the three, two measured deflection 1.4 metres from the mid-span in each direction to measure deflection and another at the mid-span of the girder. In addition, six strain gauges were installed to measure the elongation of the girder. These were installed at the same locations as the LVDT's in figure 3.3 and are shown in figure 3.10. These locations ensured a definitive reading for the maximum deflection at the mid-span, and the offset gauges and transducers indicated if the whole member was deflecting uniformly.

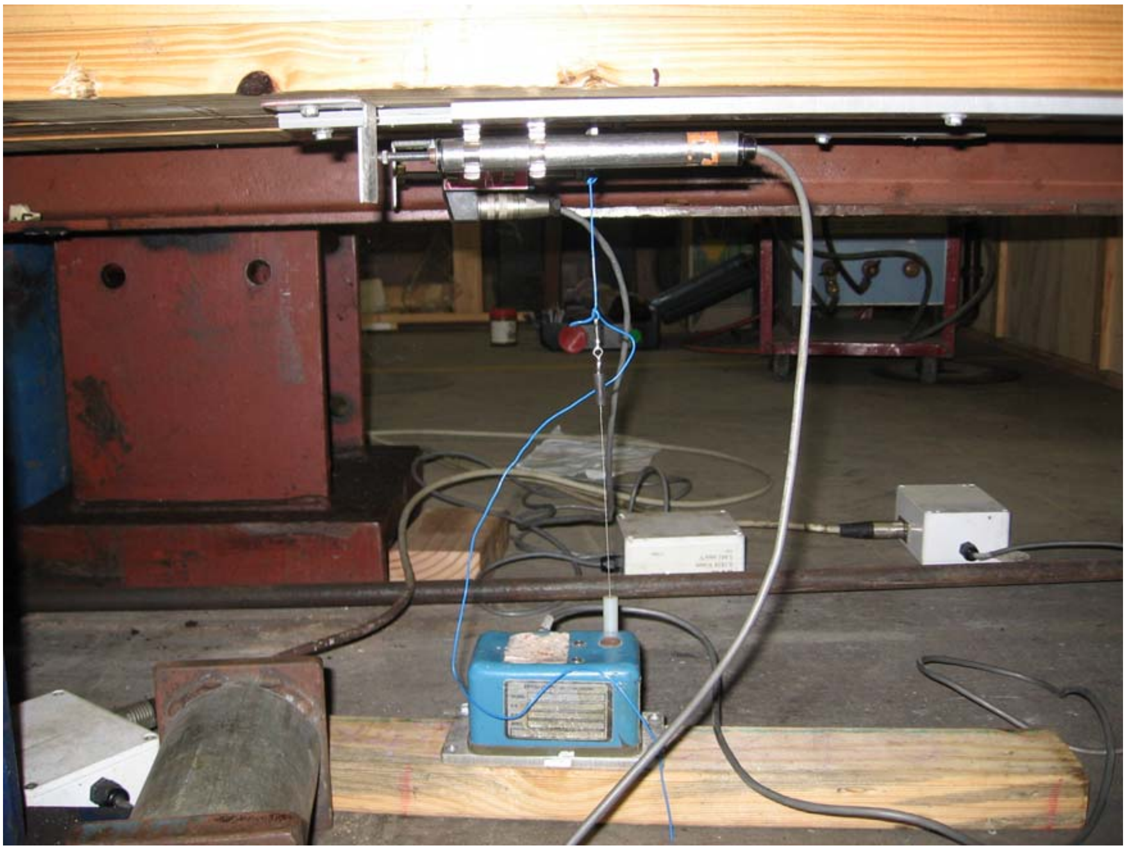


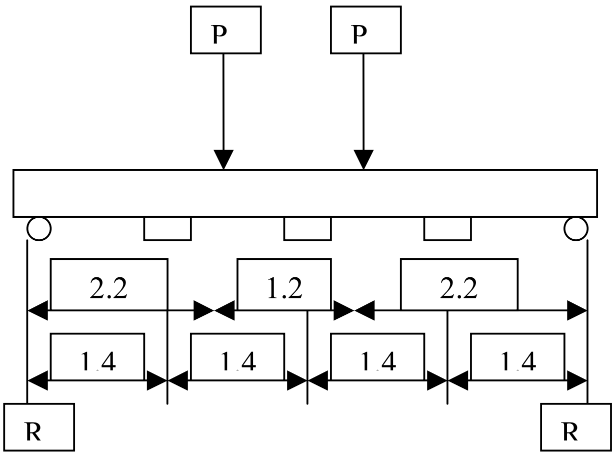
Figure 3.10: *LVDT & Strain gauge installed beside each other.*

The LVDT, as shown by the blue box in Figure 3.10 was tethered to the beam by a piece of string. As the beam deflected the string reduced and the information was sent to a computer for data collection. The strain gauge, shown as the aluminium cylinder running longitudinal to the beam in Figure 3.10 measured the elongation of the beam under load. As the load increased the spring loaded component of the strain gauge extends out against the L shaped bracket, measuring the elongation. This information is also sent to a computer for data collection.

3.4 Procedures

The following procedure was adopted for the testing of the girder.

- The clear span test length of 5.5m was marked on the strong floor using a string line.
- Hydraulic jacks were placed above the beam in the location as shown in Figure 3.11. The Jack was hung centrally over the string line. Figure 3.11 shows the approximate location of the jacks, which are referenced as point loads in relation to the supports.



LEGEND
○ Reaction
▭ Strain Gauge & Deflection Transducer

Figure 3.11 *Four point Bending test layout*

- Before commencing the first test, both jacks were calibrated.
- The girder into the testing rig using a gantry crane.

- String pot deflection gauges were attached so that the string was vertical and not being obstructed by surrounding equipment. It was ensured that the devices were calibrated before use.
- The LVDT's were attached to the underside of the girder at mid-span and directly below each jack using a battery drill. A 300mm gauge was used to mark the locations where the screw was attached. The LVDT's were calibrated prior to use.
- The girder was loaded with 25 kN from each of the hydraulic jacks (approximately 5t) and pressure released immediately. Ensure all equipment is working properly.
- The girder was reloaded. The load from the hydraulic jacks is to be increased steadily until failure occurs.
- Document failure mechanism and remove instrumentation.

3.5 Girder Description

The table below summarises the details of the girder tested.

***Table 3.2**: Girder Summary*

Timber species	Bolts Epoxied	Width (mm)	Depth (mm)	Length (m)
Slash pine	Yes	350	375	5.5

3.5.1 Stiffness

As discussed in Section 3.1, the effective stiffness of a reinforced wood section may be computed using a transformed section. In a transformed section the steel is replaced by an equivalent amount of wood. The equivalency is derived by transforming the width of the steel by the modular ratio of the steel and timber. The modular ratio is the ratio of the modulus of elasticity (E) of the steel divided by the modulus of elasticity of the wood.

The modular ratio for steel in wood is approximately 14 (Assuming N bar with yield strength of 500MPa is to be used).

3.5.2 Girder Modulus

The girder stiffness (EI) can be determined from a derivation of a calculation for the maximum deflection for a simply supported beam with two equally positioned point loads as shown in figure 3.8(Δ = Pa/24EI* ($3L^2$-$4a^2$)) (Australian Standard 1999). The following equation is the formula derived for the calculation of the girder stiffness (Australian Standards 1999).

$$\therefore \quad EI = \frac{Pa}{\Delta 24}(3L^2 - 4a^2)$$

Where P = the applied load

a = the distance between the reaction and the applied load

L = the clear span

E = Modulus of Elasticity

I = Moment of Inertia

Δ= Difference (Deflection)

The average of the girder stiffness was calculated using the results obtained over the linear section of the load versus deflection plot (see Figure 4.1). The overall stiffness of the test girder is calculated and shown in Table 4.1.

CHAPTER 4

RESULT ANALYSIS

4.1 Overview of Results

The full results can be located in Appendices B and C of this thesis however, an overview of the results easily referenced in Table 4.1.

***Table 4.1**: Overview of Results*

Point Load Per Jack (kN)	Moment Capacity (kNm)	Maximum Shear (kN)	Maximum Deflection (mm)	Stiffness EI (Nmm^2)
440	329	440	51	$5.71x10^{13}$

4.2 Stiffness

From comparing the results obtained in Table 4.1 which were calculated from the formula explained in section 3.5.2, to that of the specifications in Table 3.1 it can be seen that the required stiffness of $2.34x10^{13}$ is exceeded by $3.37x10^{13}$ to a value of $5.71 x10^{13}$. Concluding a decrease in the overall stiffness the test girder of 59 percent is required to meet specification.

4.3 Maximum Dimensions

Allowing for variation and tolerances (assuming +5mm) the girder falls within the maximum dimensions set by the Department of Main Roads (Refer appendix D). The maximum dimensions of the specified beam are 350mm x 425mm and the beam tested falls within the range set by the Department of Main Roads with overall dimensions of 350mm x 375mm (see Figure 3.4).

4.4 Moment Capacity

To reach the minimum required bending capacity on the test span, the beam is required to have a bending moment capacity of 518 kNm which equates to a point load of 241 kN per jack. Comparing this to the results actually achieved in Table 4.1 it can be seen the composite girder exceeded this limit by 199 kN per jack. The test beam passed with a total force of 880 kN load while only 482 kN load was required. By using the results in table 4.1 and the equation set out in Australian Standard 1999 and as shown below:

M = Pa

M = Bending Moment (kNm)

P = Point load (kN)

a = the distance between the reaction and the applied load (m)

M = 440 kN * 2.15 m = 946 kNm

It can be seen that the test beam exceeded the specifications set out in Table 3.1 by approximately 82 percent and the beam had not yet failed. However, the beam exceeded the maximum bending capacity by 286 kN (refer specifications – Appendix D).

4.5 Shear Capacity

For this testing scenario the shear is calculated from Australian Standard 1999, shear is directly equal to the point load placed on it ($V_1 = P_1$ & $V_2 = P_2$(see Figure 3.8)). From examining the location under the reaction it was seen that little to no compression indentations occurred at the maximum test loads. The test beams total shear capacity was 440 kN. The test specimen exceeded the minimum shear requirement of 350 kN as set by the department of Main Roads specification shown in table 3.1 (Refer Appendix D).

4.6 Deflections

Steel reinforcement assists the deflection performance of the glulam beam in a number of ways. The increased stiffness results in a lower deflection than the non-reinforced section. Also, the creep properties of the steel are superior to wood. Therefore, reinforcing should reduce the long-term deflections.

4.7 Deflection of Maximum Load

The deflection of the girders at failure is below the limits set by the specification supplied by the Department of Main Roads. Calculation for the test span (based on the specification supplied in appendix D) indicates that a maximum deflection of 66mm would be permitted at the minimum failure moment of 518kNm (load per jack of 241kN). At a load of 241kN per jack the deflections of the girder given from the instruments attached was 27.5mm.

Figure 4.1 shows the deflection versus load graph for the girder tested. In addition, the plot based on the specification is also shown. Based on the results shown on this graph the test beam is outside the limits set by the Department of Main Roads for the deflection criteria.

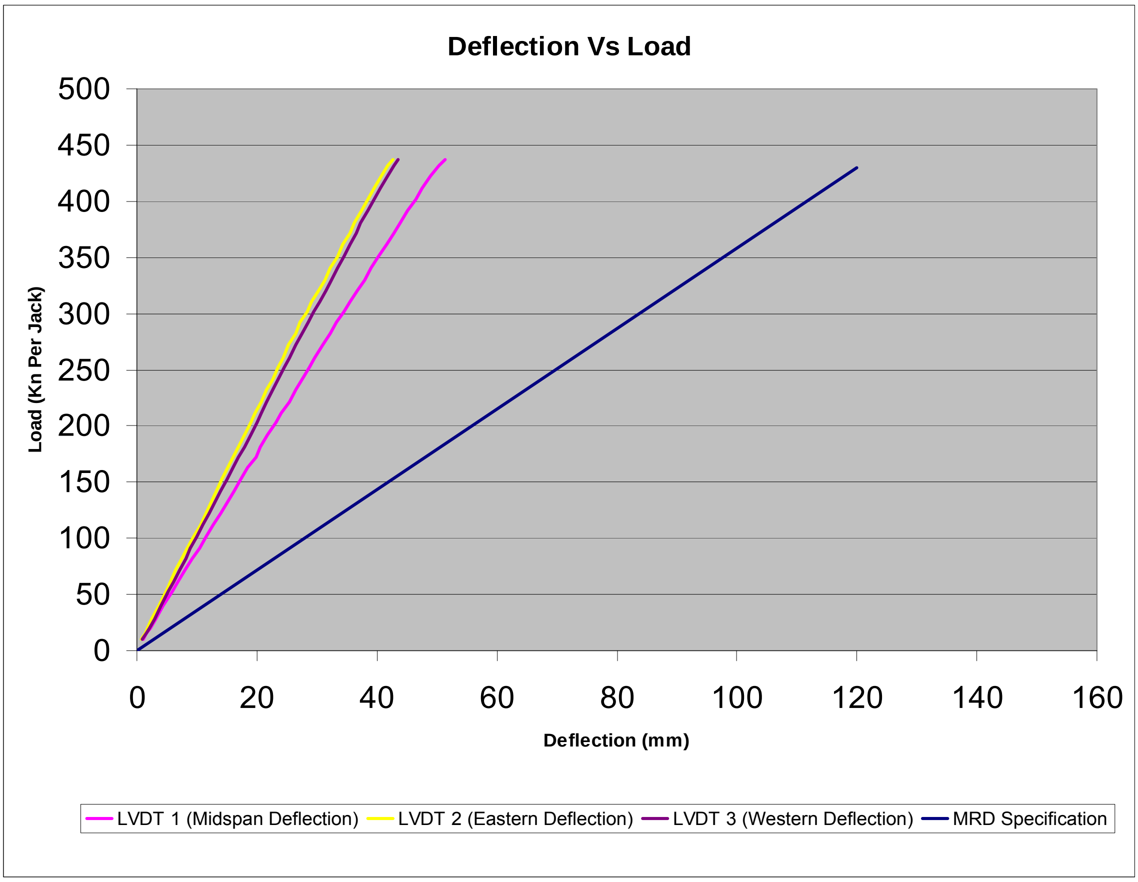


Figure 4.1*: Load versus deflection Graph*

4.8 Treatment

The protective treatment of the girders must not be compromised, as the degradation of the timber will be increased, shortening the lifespan of the girder. Where glulam timber is to be used in exposed situations, it is vital that the correct species/adhesive combination is specified and that appropriate protective measures are taken to preclude the adverse effects of light and moisture to the timber. Generally in these circumstances, where glulam is being used for bridge girders, light is not a major issue, however, the moisture increase that may occur in a flooding situation can cause rot in the timber and is potentially a large safety concern.

All glue laminated timber in exposed situations must be inspected and pass all Australian Standards as identified in AS 1328 as “service class 3”. The adhesive used for gluing the laminates is also stated in AS1328 and must be one of the following:

- Resorcinol formaldehyde (R)
- Phenol/Resorcinol formaldehyde (P/R)
- Casein

All aspects of the construction, proposed maintenance and execution of this project with regards to glulam timber beams are to be carried out in accordance with AS1328 – Glue Laminated Timber.

4.9 Damage at Maximum Load

During and after testing little damage was observed. All of the finger and scarf joints were still intact and showed no evidence of weakness. The only cracking was observed in the surface layers of the timber at the maximum failure load.

From examining Figure 4.2, it can be seen that the test girder acted in a linear manner until approximately 900kNm. This confirms that the member acted as a composite beam and the adhesive did not fail between the timber and reinforcing at any time.

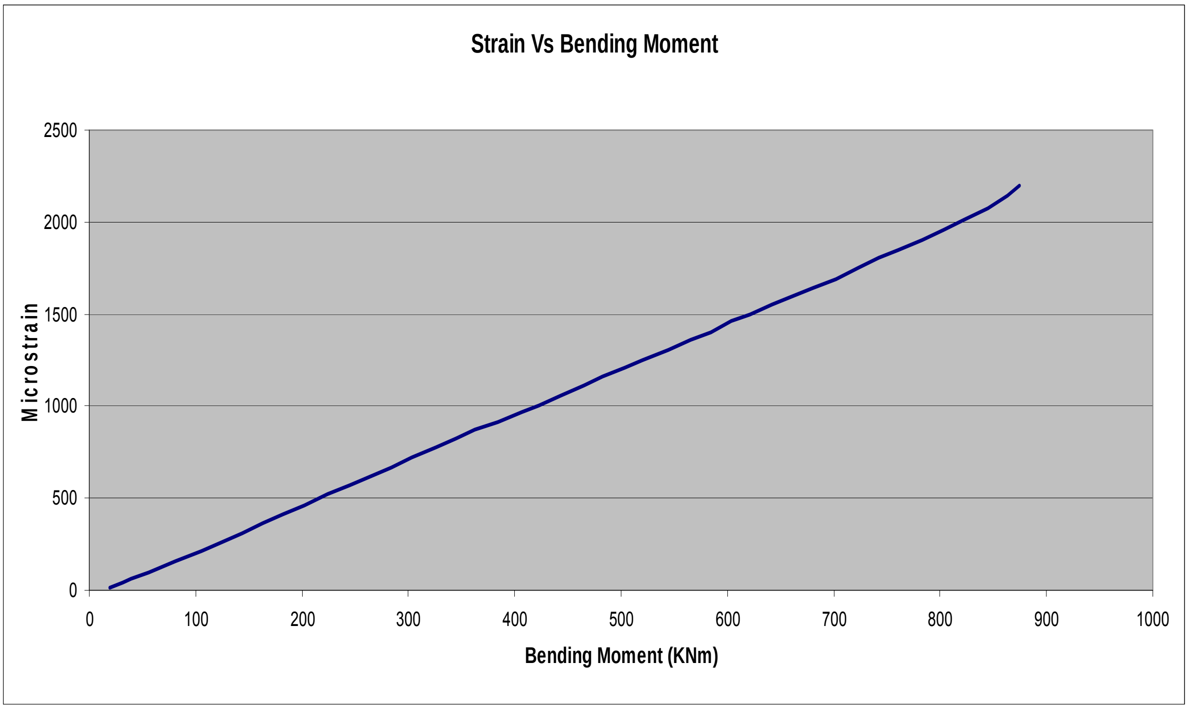


Figure 4.2: *Strain versus Bending Moment Graph*

4.10 Modulus of Elasticity (MoE), Modules of Rupture (MoR) and Steel Stress

As the steel and timber are acting as a composite section, it is necessary to perform a transformed section analysis to determine the moment of inertia of the composite section. It is assumed that the elastic and cross sectional properties of the section are constant throughout the entire girder. The additional material used for strengthening the end of the girder and the discontinuous steel was disregarded.

The girder stiffness calculated as described in Section 3.5.2 is divided by the transformed moment of inertia to produce the girders Modules of Elasticity. In performing the transformed analysis, the following assumptions were used:

- Es = 200,000 MPa
- Eglulam = 18,500 MPa

MoR has been calculated using two different methods. In both of these methods sections are assumed that plane sections remain plane. As such, a linear relationship between stress and strain can be expressed by:

$$\sigma = E\varepsilon \quad \text{(Hookes Law: Beer \& Johnston, 2002)}$$

and

$$\sigma = My/I$$

As strain (ε) at mid-span has been measured, the maximum moment, moment of inertia and MoE has been calculated based on known values and therefore the stress at maximum load (represented in this thesis as MoR) can also be calculated. From examining Table 4.2, in can be seen that two values have been represented for MoR. As there is a variation of approximately 8 percent of these values, it shows that plane section does not remain plane.

***Table 4.2**: MoE and MoR values*

MoE (GPa)	MoR1 (MPa)	MoR2 (MPa)	% variation
17.0	53.57	49.80	-7.57

The final stress to be considered is the stress developed in the steel. By using the transformed section analysis, this stress can be determined by calculating the stress at the level of the steel based on the transformed section and then multiplying it by transformation ratio. Table 4.3 shows the steel stress at failure.

***Table 4.3**: Steel Stress*

	Steel Stress (MPa)
Test Girder	44.27

As N-grade bar (*fsy = 500MPa*) was used in the section, it can be seen that approximately only one tenth of the available capacity of the steel was used.

CHAPTER 5

ECONOMIC ANALYSIS AND RECOMMENDATIONS

5.1 Economical Analysis

Glulam timber is readily available in the current construction market. Reinforced glulam beams are already in use in the industry frequently as an alternative to hardwood or steel typically for house and low-rise unit construction. This essentially minimises cost and effect on the environment. Due to this, the process to fabricate reinforced glulam timber is already in use and relevant specifications are outlined in AS 1328. Should this form of girder be commissioned in the industry it is required to achieve both AS 1328 standards and DMR minimum specifications.

While the design resulted in the layout shown in figure 5.1 it is not anticipated that by including or excluding additional reinforcing it will further economise the girder. It is considered any change in the reinforcing size would not increase or decrease the overall cost to fabricate this type of member dramatically. However, by changing the way in which the reinforcing is orientated the cost to fabricate the member could potentially change dramatically when the process changes. The technology and fabrication workshops are already in place to supply members of this description. Figure 5.2 below shows an existing product (17C reinforced Hyne glulam timber) which is manufactured for the use as structural beams for building construction and has an approximate cost of $70.00 per metre (2006) while the girder as shown is figure 5.1 is estimated to cost approximately $550.00 (2005) per metre.

Figure 5.1 *Test beam cross section.*

Figure 5.2 *Existing structural glulam member.*

5.2 Economical Recommendations

The economical value of the structure fabricated is not considered to be expensive in terms of its value as a structural member or in comparison to other equivalent steel or concrete members. Should further studies or analysis be carried out using the Department of Main Roads specifications the beam would be required to deflect more (less stiff). In order to increase the deflection of the beam less stiffness needs to be applied. To fabricate a beam with less stiffness less steel reinforcing will need to be inserted and therefore the cost to fabricate the girder will be lowered. In any case the overall cost to fabricate supply & install a girder similar to the properties shown in this thesis would be more economical and proactive in ensuring future sustainability of the environment. For this reason it is not considered beneficial to make the girder any more cost economical than it is already.

CHAPTER 6

CONCLUSION

6.1 Conclusions from analysis

At the beginning of this project a number of goals were set out to achieve, these goals included:

- Detailed calculations of steel required achieving strength required by Main Roads specifications.
- Consider several different reinforcing layouts with respect to strength versus economics.
- Investigate various performances of adhesives and assess the best in order to keep the steel and timber flexible.
- Preliminary testing on the best timber joints for this purpose.

After the completion of the testing and analysis, there are a number of conclusions which are able to be made.

- The beam that was tested reacted and deflected much in the way it was expected and calculated to. The initial concerns of how the timber in conjunction with steel would react in one composite member were alleviated when the member with stood the required load without destruction. The beam reacted well and passed the Department of Main Roads specifications including minimum bending moment capacity, minimum shear capacity, maximum dimensions. However, the beam did not pass the specifications in terms of deflection, stiffness and exceeded the maximum bending moment and shear capacities. As stated in section 4.6 the deflection at the minim moment at failure was 27.5mm instead of approximately 66mm as required by the Department of Main Roads. The beam is too stiff or rigid to be used as an alternative to hardwood timber girders. It is proposed that by minimising the size of the reinforcement hence lowering the overall strength of the member to that as specified by the department of main roads than the specified deflection and stiffness could be applied.

- The overall economics of the beam is discussed in chapter five and it is thought that economically the proposal of a softwood timber laminate in place of hardwood timber girders is an excellent alternative. However due to time constraints and financial aid only one beam was able to be tested and a thorough economic analysis of different beams was not able to be achieved.
- The adhesive used was limited to AS 1328-1998 requirements. All timber for service class 3 (external classification) *AS1328-1998* are manufactured using a phenol/resorcinol adhesive in accordance with the relevant Australian standards. It was therefore not considered worthwhile to explore different avenues of adhesives to defy the Australian Code.
- Unfortunately due to the testing facility and time constraints the preliminary testing of which kinds of joins would better service the design was unable to be carried out. It was proposed that testing be carried out for both finger and scarf joints to determine which join was more durable under load.

The test girder met the Main Road Specification for replacement girders in relation to maximum dimension's, minimum failure bending moment and minimum shear capacity. However, the overall stiffness of the girders (EI) was in excess of the stipulated value. Main Roads has indicated that a tolerance of $\pm$10 percent would be acceptable. The test girder also exceeded the maximum values of bending and shear capacities set out by the Department of Main Roads shown in Appendix D.

6.2 Difficulties with the Project

- The first and most important issue with testing a girder of this size is not only the cost to fabricate the member but also a facility to test it in. Having commenced the detailed design of the girder in early 2005 it was anticipated that the testing would occur in June to July of the same year. However due to the constraints of the testing facility the girder was not tested until March of the following year. This delayed the project by an entire year slowing any progress and was entirely out of anyone's control to do anything about.
- Secondly it was found that the importance of making the girder work as a composite beam was integral to the overall success of the design. If the adhesive failed between the steel and timber and the steel started to work independently of the timber the immediate destruction of the beam would almost be inevitable due to the timber also working independently and cracking away from the reinforcing under high loads.
- The last equally important issue with undertaking a project of this description is the financial requirements. The cost to fabricate the member and test it is something that a student alone would find too difficult without support. The national forestry association or timber manufacturers may be an avenue to alleviating some of the financial burdens

however by doing this the schedule for destruction of the beams is again subject to the supply of the beam and availability of the testing facility.

6.3 Further research

Any additional research to follow this project should keep one main objective. It is imperative that the reinforcing and timber act as a composite member under load. As shown in the results of this thesis the stiffness (EI) which is an integral parameter fell outside the range of Main Roads requirements and therefore the girder must be made to be less rigid. With this additional deflection the timber must stay bonded to the steel or the girder will fail as the timber cracks away from the reinforcing.

The reinforcing steel should be able to cater for any design loads however the timber also needs to work in composite as the steel deflects.

APPENDIX A

Research Project Specification

University of Southern Queensland

Faculty of Engineering and Surveying

ENG 4111/4112 Research Project
PROJECT SPECIFICATION

FOR:	Matthew Lorne Figg
TOPIC:	Using renewable plantation timber (Laminated Veneer Lumber LVL) as a replacement option for unrenewable hardwood timber girders in bridges.
EXAMINER:	Dr Nigel Hancock
SUPERVISOR:	Dr Santhi S Santhikumar Faculty of Engineering and Surveying
ASSOCIATES SUPERVISOR:	Kevin Covey, Covey & Associates Pty Ltd
PROJECT AIM:	The project seeks to provide renewable timber girders in the field of bridge construction by researching the relationship between reinforced steel and softwood.

1. Detailed calculations of steel required to achieve strength required by Main Roads specifications.

2. Consider several different reinforcing layouts with respect to strength versus economics.

3. Investigate various performances of adhesives and assess the best in order to keep the steel and timber flexible.

4. Preliminary testing on the best timber joints for this purpose.

As time permits

5. Design an improved beam based on testing carried out in order for a stronger or more economic beam.

AGREED: ___________________ (student) ______________________ (supervisor)

DATED: ____/____/____

APPENDIX B

Table B.1 - Deflection data

Jack 1 Load (kN)	Jack 2 Load (kN)	LVDT #1 (mm)	LVDT #2 (mm)	LVDT #3 (mm)
9.63	9.63	1.05	0.77	0.92
15.49	15.49	1.59	1.34	1.59
19.69	19.69	2.19	1.73	2.09
27.95	27.95	3.14	2.50	2.91
40.34	40.34	4.44	3.64	3.96
52.89	52.89	5.87	4.79	5.20
60.77	60.77	6.77	5.56	6.04
71.71	71.71	7.96	6.51	7.09
81.60	81.60	9.17	7.46	8.13
91.23	91.23	10.45	8.42	8.96
101.12	101.12	11.41	9.39	9.99
112.10	112.10	12.72	10.53	11.03
122.40	122.40	13.95	11.48	12.08
142.02	142.02	16.13	13.21	13.96
151.68	151.68	17.30	14.16	15.00
162.77	162.77	18.45	15.12	16.04
172.28	172.28	19.76	16.08	16.88
181.33	181.33	20.61	16.84	17.93
192.04	192.04	21.79	17.93	18.98
203.37	203.37	23.11	18.96	20.01
211.37	211.37	24.05	19.72	20.76
221.50	221.50	25.41	20.70	21.67
232.49	232.49	26.47	21.66	22.71
241.23	241.23	27.49	22.61	23.55
251.87	251.87	28.65	23.58	24.59
260.06	260.06	29.61	24.35	25.43
272.24	272.24	30.98	25.32	26.54
282.53	282.53	32.28	26.46	27.53
292.41	292.41	33.19	27.21	28.58
301.82	301.82	34.46	28.39	29.41
310.46	310.46	35.48	29.13	30.41
320.88	320.88	36.74	30.29	31.52
329.90	329.90	37.94	31.25	32.36
340.28	340.28	39.05	32.22	33.39
350.87	350.87	40.30	33.44	34.43
361.37	361.37	41.60	34.33	35.47
371.28	371.28	42.86	35.53	36.52
380.43	380.43	43.93	36.26	37.34
391.37	391.37	45.13	37.41	38.49
401.56	401.56	46.45	38.55	39.59
411.29	411.29	47.57	39.51	40.48
422.42	422.42	48.98	40.68	41.72
431.77	431.77	50.29	41.82	42.79
437.43	437.43	51.35	42.60	43.49

APPENDIX C

Table C.1 – Strain data

Jack 1 Load (kN)	Jack 2 Load (kN)	Strain Gauge #1 (mm)	Strain Gauge #2 (mm)	Strain Gauge #3 (mm)
9.63	9.63	-0.01	-0.02	-0.01
15.49	15.49	-0.02	-0.03	-0.02
19.69	19.69	-0.03	-0.04	-0.03
27.95	27.95	-0.04	-0.05	-0.04
40.34	40.34	-0.06	-0.08	-0.06
52.89	52.89	-0.09	-0.10	-0.08
60.77	60.77	-0.10	-0.11	-0.10
71.71	71.71	-0.12	-0.14	-0.11
81.60	81.60	-0.14	-0.16	-0.13
91.23	91.23	-0.16	-0.18	-0.15
101.12	101.12	-0.19	-0.19	-0.16
112.10	112.10	-0.21	-0.22	-0.18
122.40	122.40	-0.23	-0.24	-0.20
142.02	142.02	-0.27	-0.27	-0.23
151.68	151.68	-0.29	-0.29	-0.25
162.77	162.77	-0.31	-0.32	-0.27
172.28	172.28	-0.33	-0.33	-0.28
181.33	181.33	-0.35	-0.35	-0.30
192.04	192.04	-0.37	-0.37	-0.31
203.37	203.37	-0.39	-0.39	-0.33
211.37	211.37	-0.40	-0.41	-0.34
221.50	221.50	-0.42	-0.43	-0.36
232.49	232.49	-0.44	-0.45	-0.38
241.23	241.23	-0.46	-0.46	-0.39
251.87	251.87	-0.48	-0.48	-0.41
260.06	260.06	-0.50	-0.50	-0.42
272.24	272.24	-0.52	-0.53	-0.44
282.53	282.53	-0.54	-0.55	-0.46
292.41	292.41	-0.56	-0.57	-0.47
301.82	301.82	-0.58	-0.59	-0.49
310.46	310.46	-0.60	-0.61	-0.50
320.88	320.88	-0.62	-0.63	-0.52
329.90	329.90	-0.64	-0.65	-0.54
340.28	340.28	-0.66	-0.67	-0.56
350.87	350.87	-0.68	-0.69	-0.57
361.37	361.37	-0.70	-0.72	-0.59
371.28	371.28	-0.72	-0.73	-0.61
380.43	380.43	-0.74	-0.75	-0.62
391.37	391.37	-0.76	-0.77	-0.64
401.56	401.56	-0.78	-0.79	-0.66
411.29	411.29	-0.80	-0.81	-0.68
422.42	422.42	-0.83	-0.84	-0.70
431.77	431.77	-0.86	-0.86	-0.71
437.43	437.43	-0.88	-0.87	-0.72

Jack 1 Load (Kn)	Jack 2 Load (kN)	Strain Gauge #4 (mm)	Strain Gauge #5 (mm)	Strain Gauge #6 (mm)
9.63	9.63	-0.01	-0.01	-0.01
15.49	15.49	-0.02	-0.02	-0.02
19.69	19.69	-0.03	-0.03	-0.03
27.95	27.95	-0.04	-0.05	-0.04
40.34	40.34	-0.06	-0.06	-0.06
52.89	52.89	-0.08	-0.08	-0.08
60.77	60.77	-0.10	-0.10	-0.09
71.71	71.71	-0.11	-0.11	-0.11
81.60	81.60	-0.13	-0.13	-0.13
91.23	91.23	-0.15	-0.14	-0.15
101.12	101.12	-0.17	-0.16	-0.16
112.10	112.10	-0.19	-0.18	-0.18
122.40	122.40	-0.20	-0.19	-0.20
142.02	142.02	-0.24	-0.22	-0.23
151.68	151.68	-0.25	-0.24	-0.25
162.77	162.77	-0.27	-0.26	-0.27
172.28	172.28	-0.29	-0.27	-0.28
181.33	181.33	-0.30	-0.29	-0.30
192.04	192.04	-0.32	-0.30	-0.31
203.37	203.37	-0.34	-0.32	-0.33
211.37	211.37	-0.35	-0.33	-0.35
221.50	221.50	-0.37	-0.35	-0.36
232.49	232.49	-0.39	-0.36	-0.38
241.23	241.23	-0.41	-0.38	-0.39
251.87	251.87	-0.42	-0.39	-0.41
260.06	260.06	-0.44	-0.41	-0.43
272.24	272.24	-0.46	-0.42	-0.44
282.53	282.53	-0.48	-0.44	-0.46
292.41	292.41	-0.49	-0.45	-0.48
301.82	301.82	-0.51	-0.47	-0.49
310.46	310.46	-0.53	-0.48	-0.51
320.88	320.88	-0.55	-0.50	-0.53
329.90	329.90	-0.56	-0.51	-0.54
340.28	340.28	-0.58	-0.53	-0.56
350.87	350.87	-0.60	-0.54	-0.57
361.37	361.37	-0.62	-0.56	-0.59
371.28	371.28	-0.64	-0.58	-0.61
380.43	380.43	-0.65	-0.59	-0.62
391.37	391.37	-0.67	-0.61	-0.64
401.56	401.56	-0.69	-0.62	-0.66
411.29	411.29	-0.71	-0.65	-0.67
422.42	422.42	-0.73	-0.67	-0.69
431.77	431.77	-0.75	-0.68	-0.71
437.43	437.43	-0.76	-0.70	-0.72

APPENDIX D

Table D.1 – Department of Main Roads timber girder specifications

Assessment Criteria for Replacement Girder Sections				
Performance Criteria	Unit	Minimum	Maximum	Comment
Minimum Dimensions	Width (mm)	350	350	
	Depth (mm)	375	425	
M_{min} at failure	KNm	518	660	Proof test
Negative BM Capacity	KNm	30% +ve BM	30% +ve BM	Proof test
V_{max} at failure	KNm	320	350	Proof test
Max deflection at failure	mm	120	170	Proof test
EI girder	Nmm^2	2.34×10^{13}	2.96×10^{13}	Proof test
Specific Function Requirements				
General Durability Requirements				Members must be suitable for external use but not in contact with the ground.
				Member must be resistant to termite and fungal attack.
				Member will be subject to regular submergence (AATOS 72 Hrs) and exposure to sunlight and/or rain.
				Typical working temperature range: -5 to 45 degrees C
Minimum Treatment Level (timber)		H3	H3	Demonstrated compliance with AS1604
Design Life	Years	30	30	Minimum
Weight	Kg	1500	2000	Upper Limit
Appearance-timber		B Grade	B grade	As defined in AS 1328.1: A machine planed finish is acceptable. Occasional skips in the surface are permissible and minor blemishes, voids and manufacturing want shall be acceptable. The outermost laminations shall be free of loose knots and voids.
				Suitable for application of costings

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