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RECORDING TIMBER BRIDGE GIRDER DEFLECTIONS USING A LASER REFERENCE SOURCE AND A HIGH SPEED CAMERA

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ABSTRACT: This paper identifies a method of measuring the mid-span deflections of timber bridge girders when loaded by traffic. There are many short span timber beam bridges of unknown reliability in regional Australia that have high traffic loadings and many of these bridges were designed according to older codes. In order to identify the current safety index and probability of failure of these girders while in service, it is necessary to measure their deflections under normal and actual loadings. Because of the large numbers of girders that need to be measured, it is important to use a low cost method that is quick and easy to set up in the field.
The method proposed here involves a laser source which is adjusted to produce an image of the laser on a graduated chart mounted at the mid-span of the bridge girder. The source is mounted on a stable support. Traffic loading deflects the girder and the chart moves up and down synchronously. A high speed camera is used to record the movements of the chart relative to the image of the laser. The chart was inscribed so that any movement of the image could be easily read from the graduated scale. A video recording was made of the chart movements relative to the laser source and the recording was analysed to identify the peak movements. The results show that, when the girder is loaded by moving traffic loads, the peak deflection, the dynamic resonant behaviour of girder deflection and the recovery can be readily identified.

1. INTRODUCTION

1.1 Societal Implications

There are over 2000 timber bridges in NSW and it is not clear which bridges are structurally sound and which are not (Howard 2009; Roorda 2006). Many where built in the early part of the 19th Century with some extant bridges being built towards the end of the 18th Century (Glencross-Grant 2009). In this early era of bridge construction, probabilistic failure analysis techniques had not been established. Even the knowledge of the strength of Australian hardwoods was not well recorded.

To overcome this concern the first systematic evaluation of Australian hardwoods for bridges was carried out by Professor Warren in 1886 (Warren 1890). A more recent evaluation was carried out by the CSIRO in 1963 (Bolza & Kloot). Probably because of the wider use of concrete and steel, the use of timber for use in new bridges was in decline. Little attention has been paid, in more recent times, to the need for the characterisation of the temporal degradation of Australian hardwood bridge timbers.

In recent years, although theoretical techniques have advanced with papers such as those by Hasover (1974) and reviews by Elishakoff (2004) little has changed in relation to timber bridges. The definition of the safety of a typical timber bridge still does not include the probabilistic determination of the likelihood of failure. The failure of a timber bridge in Singleton NSW, in 2001, led to arguments that were taken to the High Court of Australia. The Court records that there was uncertainty in the exact load-bearing capacity of the bridge at the time of the accident and expert evidence included values of 9.3 tonnes and 13.5 tonnes. These figures were specified to a high level of precision, about 1%, yet they differ by about 45%. What was missing from the Court records was the probability of failure that was related to these proposed load bearing capacities.

Structural Health Monitoring can detect degradation and allow components to be replaced before they become excessively degraded. The main degradation processes that affect timber in NSW are: termite attack; chemical rot in the presence of high moisture content; and mechanical impact damage. These factors can all occur within short periods and between standard structural inspections. To identify any change in structural integrity a low cost continuous monitoring system is required. The system should be low cost because large numbers are required; continuous operation is required so that the bridge can be quantified safe after the passage of every vehicle. The objective of Structural Health Monitoring is to ensure the safety and reliability of an engineered system (Mahini et al., 2011). Monitoring movements, displacements and deflections has been reported by Mahini et al. (2011) as a practical method for assessing older bridges in Australia and Worldwide. The important parameter to monitor, in relation to timber bridges, is the mid-span deflection. Mid-span deflection data enables the vehicle mass and number of vehicles per day (vpd) to be determined which then allows the determination of parameters such as mid-span bending stress, girder shear stress at the supports and pile loading. Combining these data with material strength data enables the calculation of a probability of failure and a structural safety index.

1.2 Type of Bridge

The type of bridge examined as part of this research project was a short span timber beam bridge and one example bridge, Powers Creek Bridge, Armidale, NSW, shown in Figure 1, was used as a test bridge. It comprised four round un-sawn girders of span 8.6 m with timber deck planks of length about 5 m laid across the girders. The density of traffic crossing the bridge was low at about 300 vpd and characterised by morning and afternoon travel patterns as local residents moved from their homes to town for work and back again.

Figure 1: Side view of Powers Creek Bridge NSW; a single span timber beam bridge

2 THE STRUCTURAL HEALTH MONITORING PROCESS

2.1 Background

Historically, as Allan (1895) indicated, single value loads were compared with single value material resistance values. Providing the resistance values were a factor of 4-7 times the load values structures were regarded safe; that is the load would not cause the structure to fail. However, as timber components degrade temporally there was no convincing quantifiable method of defining how the safety factor might vary. The process became increasingly difficult

to quantify as some degraded components were replaced with some less degraded ones. Components were sometimes recycled; an inner girder might be moved to an outer kerb position and a new girder placed in the inner position. In the absence of any quantification of the material parameters of these recycled components any attempt at calculating a single valued safety factor could only be an estimate with an unknown level of confidence.
Methods of calculation that involve the distribution of the material parameters and provide a probability of failure for all the applied loads have been developed to overcome this concern. The method used here is the Monte Carlo method (Moore *et al.* 2011; Wolfram 2004). This method, with modern high speed computers, allows the possibility of failure of timber beam bridges to be quickly calculated. Since only a few components are involved and the governing equations involve a low number of parameters the computing time might only be minutes.

2.1.1 Log-normal Traffic distributions

The type of traffic flow across the test bridge was not initially known. As an initial indicator a histogram was created for all the registered vehicles in NSW in 2008; refer Figure 2. The data were taken from the Australian Bureau of Statistics (ABS) and included all manufacturers with both more than 50 000 vehicles and vehicle models with more than 5 000 vehicles registered in NSW. Also shown in Figure 2 as a continuous curve is the log-normal distribution (Limpert *et al.* 2001) that fits the histogram with: mean value μ^* of 2.57; deviation σ^* of 0.24; and population of 3.4 million vehicles.

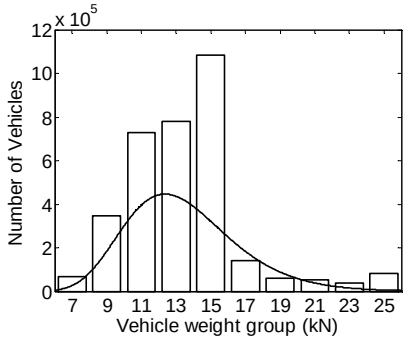


Figure 2: Histogram of registered vehicles in 2008 (ABS 2008)

2.1.2 Modulus of Rupture distribution

When first installed, the girder Modulus of Rupture (MOR) may have been within a distribution that has a mean value of about 184 MPa with a variation of about 10% and within its working life the variation might stay below 20% (Bolza & Kloot 1963:Ironbark (*Euc.spp*)). As the girder temporally degrades towards the end of its working life the distribution variation can increase to about 30% (Crews *et al.* 2004; Moore *et al.* 2011) .

2.2 Measuring Bridge Parameters

2.2.1 Type 1 recording apparatus

The experimental recording apparatus took two forms. The first, Type 1, is shown in Figure 3. It comprised a fixed laser stably mounted and directed at a graduated chart. The graduated chart was rigidly attached to the girder at mid-span. With the bridge in an unloaded state the position of the laser image on the chart provided a quiescent zero position. As static load was applied to the bridge the mid-span of the girder and the graduated chart moved downwards. The position of the laser projection movement was easily read on the chart. However, dynamic vehicle movements caused the graduated chart to move too quickly to visually monitor with the human eye. A high speed camera recording at 240 frames/second was, therefore, used to record the relative movements of the laser projection and the chart. The data recorded using the Type 1

apparatus were related to a small number of vehicle bridge crossings and an operator was continually present.

2.2.2 Type 2 recording apparatus

The second form, Type 2, comprised a purpose built laser source and detector. The source was mounted stably to the headstock and the detector to the girder mid span as shown in Figure 4. A PVC pipe was used to stop insects and debris obscuring the laser beam. The detector took the form of a minimalist high speed camera attached to the graduated chart (Moore 2008). It had only 8 photo sensors (pixels) together with an electronic system that recorded the peak deflection of each vehicle as it crossed the bridge. The sensors were not linearly spaced but were positioned as shown in Table 1 (refer section 2.3.3). As the graduated chart moved vertically the sensors would be activated; a 6 mm vertical movement, for example, activating sensors 1 and 2.

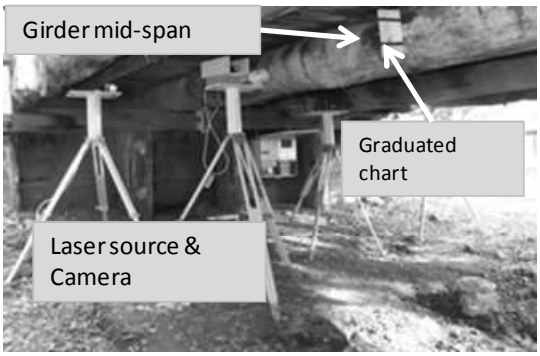


Figure 3: Experimental apparatus, Type 1

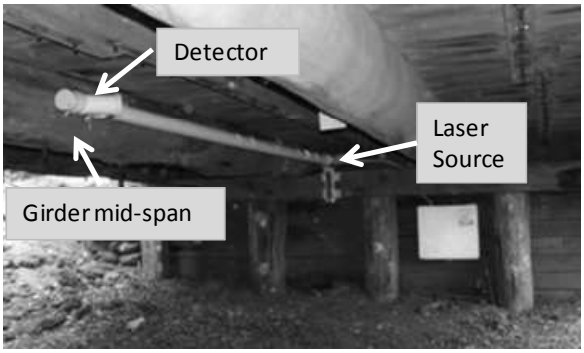


Figure 4: Experimental apparatus, Type 2

This Type 2 system was installed onto a bridge girder and left to function for three weeks. Power was supplied by battery and the data were automatically transferred to a USB memory. The data were then downloaded from the memory as required. Long term recording using the Type 2 apparatus, which was automatic, only required the site to be visited to download data and recharge batteries; with a solar panel providing power and a telemetry data communications system the site need be visited only infrequently.

3 RESULTS

3.1 Case 1: low speed dynamic loading

A small loaded truck of Gross Vehicle Mass (GVM) 4.4 tonne, wheelbase 2.6 m, was driven across the bridge at about 10 km/h and the variation of the position of the laser projection on the graduated chart recorded using the Type 1 apparatus. The video recording was then viewed frame by frame and the measured deflection data of the girder mid-span interpolated and plotted. In Figure 5: the measured data are the points labelled ‘measured’; the data calculated for individual axles labelled ‘Axle 1’ and ‘Axle 2’; and the combined dual axle vehicle labelled ‘calculated’.

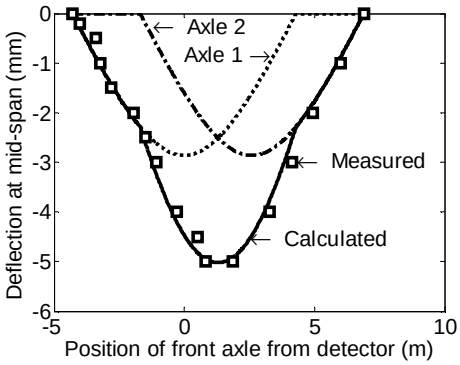


Figure 5: Mid-span girder deflection produced by a 4.4 tonne vehicle crossing the bridge at about 10 km/hr

3.2 Case 2: High speed dynamic loading

A heavy vehicle crossed the bridge at about 60 km/h and the deflection transient was recorded as for Case 1. The data obtained are shown in Figure 6; the measured data as a dotted curve and a simulated data set as a solid curve. The simulated data are only a simplified sinusoid construction not derived from a rigorous analysis of the measured data. The simulated signal comprises natural frequencies at 1.3 Hz and about 7 Hz. A half cycle transient at 1.3 Hz is initiated as the vehicle first impacts the bridge, after about 0.5 second a second impact overrides the first transient and produces a second positive 1.3 Hz transient. Both of these transients are coupled with a further transient at about 7 Hz. The complete transient lasts about 3 seconds.

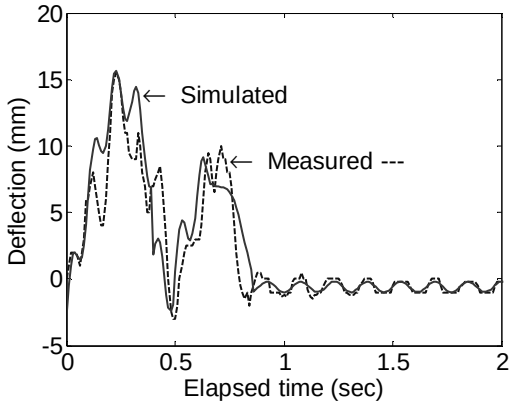


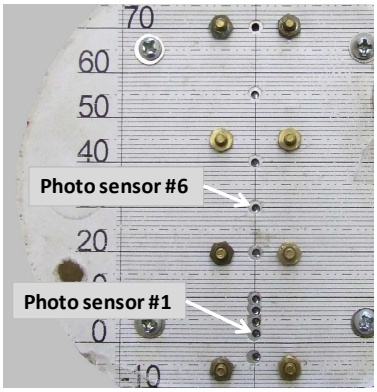
Figure 6: Deflection transient caused by heavy vehicle

3.3 Case 3: Long term recording

The data obtained with the Type 2 apparatus are shown in Table 1. A vehicle was classed as being in a particular threshold group if the lower threshold was exceeded but not the upper threshold. The applied load to cause each threshold to be exceeded was determined by a static load calibration using the Type 1 apparatus. The data in Table 2 are also shown in Figure 7 together with a log-normal distribution fitted to the data, the line of which obscures the heavier load data. The log-normal traffic load distribution, shown in Figure 7, was fitted to the data of Table 1 using the MatLab® 'dfittool' (MathWorks 2011). The parameters obtained were: mean value μ^* of 2.62; deviation σ^* of 0.41; for the population of 5585 vehicles. This distribution, although apparently a reasonable fit to the data, does not represent well the tail of the measured data. If traffic conformed to the log-normal distribution then a period of more than three weeks would be required for at least one vehicle in the weight range 60-250 kN to cross the bridge. Yet there were about 50 vehicles within that range in the measured data.

Table 1: Number of vehicles crossing the test bridge-v-threshold and image of detector face

Photo sensor # (pixel)	Position of Sensor relative to reference line (mm)	Load to exceed threshold (kN)	Number of Vehicles
1	-2.0	12	4938
2	-4.5	27	363
3	-7.0	42	107
4	-9.5	57	128
5	-19.7	118	24
6	-29.9	180	25
7	-40.0	240	0
8	-55.0	330	0



The mid-span deflection produced by applying a load selected from the log-normal demand load distribution, as shown in Figure 7, was calculated using Monte Carlo techniques and compared with the 1:600, limit state deflection to span ratio. Representing the demand distribution by *f(D)* and the capability distribution by *f(C)*, the probability of failure, p_f, is estimated from:

p_f = probability that *f(D)* is less than or equal to *f(C)* (1)

Generally the probability of failure is:

$$p_f = \int_{G(x)\leq 0} f_C\big(G(x)\big)dx \tag{2}$$

where the limit state function is:

$$G(x) = f(C) - f(D) \tag{3}$$

which when solved numerically by the Monte Carlo method, p_f is then given by:

$$p_f = \frac{n^*}{n} = \frac{Number\ of\ times\ G(x)\leq 0}{Total\ number\ of\ cycles,n} \tag{4}$$

The mid-span deflection limit state function *G(x)* was calculated from:

$$G(x) = \delta_{crit} - \delta_{MS}(x) \tag{5}$$

where the deflection δ_{MS} was given by:

$$\delta_{MS}(x) = \frac{4P(x)L^3}{3\pi k_E E k_I(x) D^4} \tag{6}$$

and where *P(n)* was selected randomly from the applied log-normal traffic load distribution, *L* was the span (8.6 m), k_E the temporal elasticity factor (1), *E* the modulus of Elasticity (24 GPa), *D* the initial girder diameter (0.5 m) and k_I the temporal second moment of area factor was selected from a normal distribution defined as having a mean value of 0.92 and a standard deviation of 8%. This was equivalent to the girder mid-span diameter being degraded to 0.49 ± 0.01 m which could occur at a surface erosion rate of 0.2 mm/year.

This calculation was repeated 10^7 times and the number of times *G(x)* was less than, or equal to, zero was 4. The failure rate was thus 4 in 10^7 and the safety index β, 4.9; the safety index β is related to the probability of failure by the inverse of the normal cumulative distribution function (Microsoft Excel® function NORMSINV(4E-7) = -4.9). The limit value for the safety index β, in relation to a servicibility failure is 3.1 (Elishakoff 2004). The histogram outline shown in

Figure 8 comprises the total 10^7 data set and shows the demand deflection distribution and the single valued deflection limit capability distribution which is only exceeded 4 times.

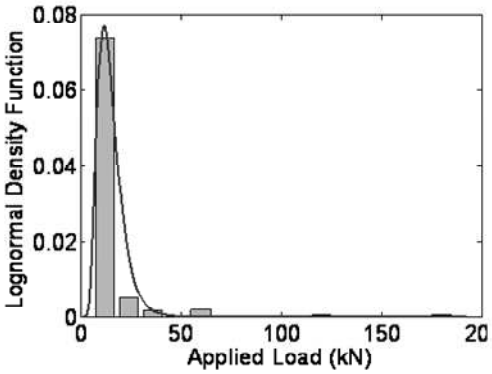


Figure 7: Traffic Histogram

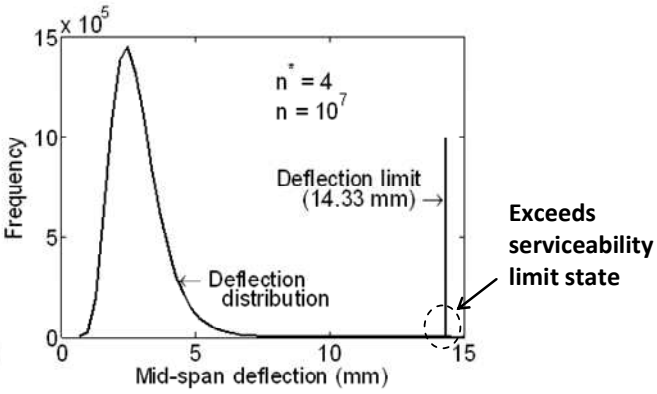


Figure 8: Frequency-v-Deflection curves

4 DISCUSSION

In order to accurately quantify the current probability of failure of regional timber bridges, both load data and timber strength data need to be known to a higher level of accuracy than is currently the case. Since each bridge is different and degrades temporally to a different extent it will be necessary to measure these parameters for each bridge. Examination of the numbers and types of vehicles registered in NSW provided a log-normal distribution with a similar mean to that measured for the test bridge. The measured distribution had a higher deviation but this was to be expected because the vehicles were carrying a variety of loads. However, there was a range of heavy loads measured that needs to be more effectively included in the traffic distribution description. The measured distribution appears to be bi-modal consisting of the log-normal distribution, of mean 2.62 and deviation 0.41 plus a uniform distribution in the range 80-200 kN. More research needs to be carried out to determine the long term details of this distribution.

The temporal variations in both the MoR and the effective second moment of area of the girders can confound effective load capacity estimates. Changes in effective diameter can be identified using the Type 2 apparatus either by calibrating the deflections obtained against known loads or by observing the changes in the numbers of vehicles exceeding particular thresholds. More research is required to determine how effective this approach can be.

The Type 1 apparatus provided a simple method of determining the dynamic deflections that occur with high speed heavy vehicles. The data presented in Figure 10 contained transients that interfered with each other; a positive cycle being over-ridden by the arrival of a second axle. It also contained evidence of insufficient damping in the structure with oscillations occurring several seconds after the vehicle had passed. Similar data from specific bridge measurements could aid in improved bridge designs.

The calculation of useful probability of failure and safety index values will depend on the generation of better knowledge of the parameter distributions that pertain to each bridge. It is not viable to just use design data without taking into account the temporal degradation that occurs. A better understanding of these distributions and what causes them to vary will also allow methods of prolonging component lifetime to be devised.

5 CONCLUSION

The use of a high speed camera in conjunction with a stable laser reference image can provide data that can be interpreted to identify details of traffic flow, maximum bridge loading, dynamic behaviour of bridge girders and enable the probability of structural failure to be computed. However, to improve the confidence level in such probability calculations further research is required to quantify the temporal distributions that pertain to degraded timber bridge components.

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