1 INTRODUCTION

1.1 Background

There are a large number of aged timber-bridges in New South Wales (NSW) that require assessment of their structural strength. This issue was highlighted at a recent meeting of Mid North Coast councils, NSW, where it was reported that “One of the main issues confronting local Councils is the number of timber-bridges that need major maintenance or total replacement” (Troy 2011). A decade ago, in 2001, the failure of a timber-bridge in Singleton NSW led to arguments that were taken to the High Court of Australia. The Court records that “There was a dispute, and some uncertainty, as to the exact load-bearing capacity of the bridge in its condition at the time of the accident. There was expert evidence as to what its load limit should have been. It is not a simple matter to calculate, but figures between 9.3 tonnes and 13.5 tonnes on various assumptions were given [by expert witnesses]. Vehicles of the same weight as, or greater weight than, the applicants' truck had safely crossed the bridge right up until the time of the collapse.” (Brodie v Singleton Shire Council 2001). The applicant’s truck was identified as being about 22 tonnes which was the mass that eventually caused the bridge to fail. The two figures, 9.3 and 13.5 tonnes, and which are identified in the Court records, are indications of the safe load limit that the bridge should have been able to withstand. These safe load limits are specified to a high level of precision, about 1%, yet they differ by about 45%. What has been filtered from the expert witness’ submissions, and what identifies why there can be two load limits, is the probability of failure that is related to these limits; it has not been quoted. The trucks that crossed the bridge prior to its failure, did so several times on the day in question and also crossed a second timber-bridge on the same road that had a signified load limit of 18 tonnes. The same 22 tonne truck did not cause the second bridge to fail. It can be inferred from this case that the public perception is that load limit signage, where present, does not accurately identify bridge carrying capacity and may be ignored.

One way to overcome this concern is to apply Structural Health Monitoring (SHM) techniques and continually monitor structures, such as bridges, and then provide alarms when the integrity of a structure has changed. However, there are still no commercial methods that can be used to implement SHM on timber-bridges and in recent decades the level of research related to timber-bridges in NSW has been low. In the last thirty years significant advances have been made in the design of electronic measurement equipment but such advances have

not been commercially applied to fully evaluate timber-bridges. One reason for this was probably the change of maintenance responsibility for local roads resulting from changes to the Local Government Act 1993. Many timber-bridges became the responsibility of Local Government which had previously been the responsibility of State Government. This inference is plausible since engineers working for Roads and Traffic Authority (RTA) reported at two conferences (Law & Morris 1992; Law *et al.* 1991) the state of timber-bridge girder research and it can be seen from the outcomes of the recent Road Asset Benchmarking project (Howard 2009; Roorda 2006) that little has changed over the last two decades.

Assessments for maintenance procedures of structural integrity of timber-bridges currently rely upon both regular visual inspections and regular engineering inspections. Regular visual inspections, for a particular bridge, are implemented about every two years and engineering inspections, in the absence of structural concern, about every ten years. If concerns arise from a visual inspection, about the integrity of a structure, then an engineering inspection can be initiated at any time. A visual inspection can provide an indication of the condition state of a girder. The condition state is simply the definition of decay and deterioration of a girder on sliding scale (refer Chapter 2). If a girder is in Condition State One (CS-1) or Two (CS-2) then it is probably structurally sound. If in Condition State three or four it may not be structurally sound. Careful proof loading, as part of an engineering inspection, can then indicate the load that a structure in Condition State Three (CS-3) or Condition State Four (CS-4) can withstand. In this thesis it is shown that the analysis of historical girder strength and elasticity datasets leads to the conclusion that these maintenance procedures are a valid method of assessing structural strength. An opportunity arises for new measurement procedures to provide more detailed, lower cost and quicker assessments.

Better economic outcomes are likely to be obtained by using bridge structural components to their full potential, by using components for the whole of their structural life and replacing them prior to the start of measurable deterioration. Although this concept is common in many other industries it is not applied to aged timber-bridges in any quantified way and requires detailed knowledge of component lifetimes. Timber-bridges tend to exist in remote locations and to minimise maintenance costs it is important to only visit them when they need maintenance. For these bridges to be maintained as reliable structures they need to be constructed from components of known reliability and longevity. Only then can structural reliability be predictable and allow maintenance to be planned with safety and economics in mind. A reliable structure should not fail in an unpredictable manner. The difficulty that

currently exists is that a particular bridge girder may have a Modulus of Rupture (MoR) value, from new, that is much higher than the minimum prescribed by a relevant design code. After a period it may not be necessarily obvious, unless tested in detail, what value of MoR that the particular girder currently will have. The girder might be replaced because of a lack of reliable structural information well before its end of life; an uneconomic outcome. Another girder might, because of significant and non-quantified degradation, fail in service well before its expected end of life and allow excessive damage to be caused to other components; another uneconomic outcome. One method for overcoming these difficulties is to provide more predictable and improved economic outcomes by monitoring temporal changes in girder performance.

Early work in identifying changes in timber-bridge girder performance involved the use of a theodolite to take static measurements as confirmation of design criteria. This method could have been used to monitor temporal change, but there appears to be no evidence of that outcome. More recent work, aimed at providing an instrumental quantification of girder performance, has examined the use of vibration analysis and deflection analysis. Vibration analysis techniques can be appropriate for stress laminated bridges that act as a coherent element but are less appropriate for timber beam bridges with loose connections between components that may vibrate independently. Neither these earlier measurement practices, nor current standard procedures, take temporal changes into account.

This research is aimed at testing the hypothesis that instrumental measurement of girder mid-span deflection could be used to identify girder condition by monitoring temporal variations. Because the experimental equipment used is not commercially available and the owners of timber-bridges are not currently using the proposed approaches, it is not part of this research to evaluate the costs involved; this evaluation will be the subject of future research.

1.2 Aim

In the light of this background, the aim of this thesis is to test the hypothesis that:

Continuous deflection monitoring can be used to assess the probability of timber-bridge girder failure

Three procedural steps are required to determine the probability of Timber-bridge girder failure. The first is to identify a parameter that can be used to predict failure. It then has to be shown that that parameter can be measured to an adequate level of accuracy for in-service structures. The third step is to utilise that parameter as part of a SHM analysis technique to determine structural health and the probability of failure.

Two types of failure are considered. The first case is span to deflection limit state failure and the second case is girder structural failure. In the first case, two parameter distributions need to be determined. These distributions represent the applied traffic loading and the girder stiffness. It needs to be demonstrated that in-service loading and stiffness data can be temporally recorded. In the second case a method of identifying girder strength also needs to be determined.

In order to achieve the aim, the following objectives are examined:

1. The determination of the strength and elastic behaviour of a single girder;
2. The determination of a measurable parameter indicative of bridge girder strength;
3. The identification of the load-v-deflection curve for an in-service girder;
4. The deflection distribution caused by traffic loading;
5. The effect of dynamic activity on the measurements;
6. The determination of girder in-service MoE;
7. The identification of the percentage of applied load supported by an in-service girder;
8. The determination of girder MoR; and
9. The use of these data to identify a bridge limit state safety index.

1.3 Thesis Outline

In Chapter 2 a literature review is presented that examines some of the theoretical concepts that relate to timber beam bridges together with a historical review. Timber beam bridges were first constructed in the 19th century and some bridges from the early 20th century are still in use. This review, therefore, covers concepts developed over a long period of time and some of them are now only used by technical staff that maintain or design timber-bridges. Some more modern concepts are only recently being applied to timber structures and these newer technologies are also reviewed.

Data derived from the testing of over 300 in-service timber-bridge girders are theoretically analysed in Chapter 3. Four independent datasets are examined and compared. These data are

also compared with excised sample species characterisation data. By statistically analysing and comparing these data relationships between girder MoE and MoR are developed.

In Chapter 4 in-service measurements are described. These are measurements that can be performed on an in-service bridge to determine the mid-span deflection produced by vehicles traversing the bridge. New measurement methods are developed that can be used that do not interfere with traffic flow. Then in Chapter 5 new SHM strategies are developed to transform the in-service measurements into probabilities of failure. From these probabilities, safety indices are calculated. It is also shown how temporal changes of deflection that occur over the life of a girder can be used to predict the probability that a limit state will be exceeded. In Chapter 6 the theoretical and experimental work is discussed and conclusions and recommendations are made in Chapter 7.

2 LITERATURE REVIEW OF THEORETICAL AND HISTORICAL BACKGROUND

2.1 Introduction

There are many thousands of timber beam bridges in Australia with about 2000 extant in NSW (Howard, 2009; Roorda, 2006). Timber-bridges are constructed in a variety of forms, so to ensure that these bridges are known to be structurally sound, it is important to quantify the strength of the major structural elements. A general outline of a timber-bridge is shown in Figure 2-1 (RTA, 2007). Some bridges have: multiple spans; some are only single span; others are high level spans that are above normal flooding as suggested by the outline in Figure 2-1; while some are low level as in Figure 2-2. However, one major structural element that is common to all timber beam bridges is the girder.

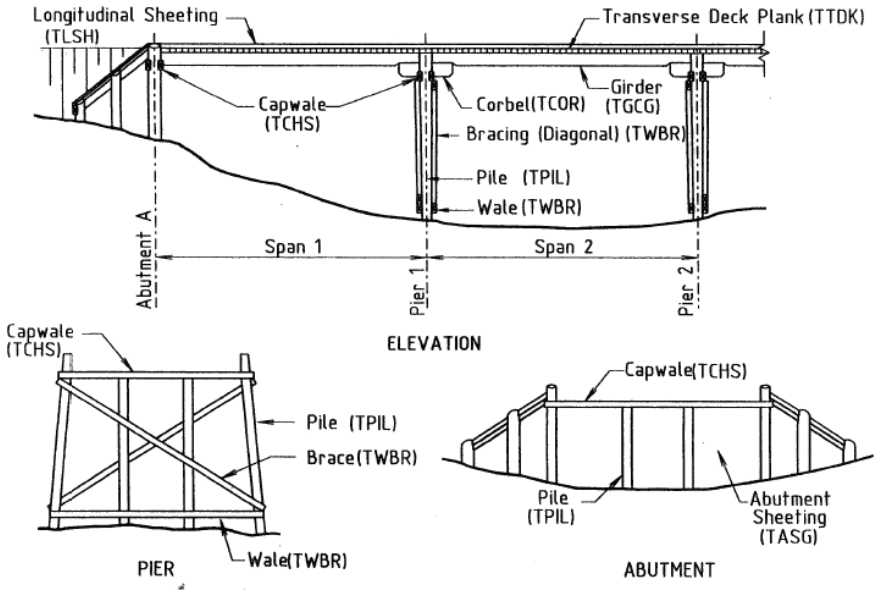

Figure 2-1: Schematic of multiple span timber beam bridge (RTA, 2007: Sketch No. 11)

Figure 2-2: Gwydir River Bridge, Bundarra, NSW. This is a low-level timber beam bridge, but typical of many throughout NSW on local roads

Currently, it is difficult to accurately define the individual strengths of aged hardwood girders for timber-bridges in NSW. The reasons for such are partly because there is high variation of strength in the total set of girders throughout NSW and partly because of availability of a suitable method to measure girder strength. Historically, girder strength has been evaluated from a design perspective (refer glossary) and girders have been specified to fulfil the requirements of appropriate Australian Standards such as AS 1720.1 and AS 5100. However, in order to accurately define the strength of a particular aged in-service girder an alternative approach is required.

There has been little identified research to establish such an alternative approach. Some work occurred in the early 1990s. The RTA examined, from the design perspective, firstly the maintenance of timber-bridges and secondly investigating the use of alternative materials to full-size old growth hardwood logs (Carter, Crews, & Keith, 1991; Carter, Crews, & Taylor, 1992; Law & Morris, 1992; McTackett, 1991; Oldfield & McTackett, 1991; Yttrup, Law, & Audova, 1991; Yttrup, Law, & Subramaniam, 1991). Tyson (1992) reported a method of hydrostatically measuring mid-span static deflection. Putt et al. (1992) investigated the use of an extensometer to measure longitudinal strain for the purpose of structural evaluation. This work did not determine a new method to quantify or predict girder strength. As a result, current bridge maintenance literature (QDMR, 2004;2005; RTA, 2007; Standards Australia, 2004a; VICROADS, 2011) cites only proof-loading as a method of determining current girder strength. Several Non destructive evaluation (NDE) measurement techniques are available to identify deterioration, but these techniques are not correlated with girder strength. Two such NDE techniques are identified by Queensland Department of Main Roads (QDMR). The first involves resistance drilling with a small diameter very long bit, and the second real time density measurement using a radioactive isotope (QDMR, 2005, Appendix D). Presence of deterioration is detected, in the first case by low resistance to the drill and in the second case by the low absorption of X-rays. These are both indirect methods, since any material deterioration has to be related to areas of high stress. Significant deterioration can occur in low stress regions of a girder and not affect its strength.

Prior to 1993, significant research was performed by the RTA because they significantly controlled thousands of timber-bridges in NSW. But in 1993, the NSW Roads Act No. 33 (1993:Section 7.4) shifted the responsibility of timber-bridges on many classified roads to local government. Therefore, the timber-bridges for which RTA was responsible declined to a few hundred. Since then, the new managers of such timber-bridges, the local councils, have not had

the resources (financially or technically) to identify timber-bridge girder strength. Instead, supported by the NSW Timber-bridge Partnership (Roads and Maritime Services, 2012), many councils have replaced their timber-bridges with concrete structures.

More recently research related to timber-bridges has been conducted by third parties and not by the managers of such bridges. Lake (2001) examined the use of strain gauges to dynamically measure the mid-span deflection of both timber and concrete bridges. Samali et al. (2007) applied dynamic measurement techniques to assessing flexural stiffness of bridges. Then Wilkinson (2008) examined the concept of using laminated pine replacement girders and Moore (2009) reported a laser based method of measuring girder mid-span deflection. However, an alternative approach to the measurement of in-service girder strength still needs to be commercialised. This review will examine the extent to which available research and data can be used to identify individual girder strength.

2.2 Theoretical Background: technical terms

In later chapters technical terms are used without detailed explanation. Some of these terms may not be in general use and are, therefore, introduced here. The type of term varies, some relate to the description of specialist parts such as clears. Other terms refer to theoretical terms such as prediction band and still others refer to concepts such as design perspective.

2.2.1 Clears

Small clears are samples that have been cut from larger timber sections. They are carefully selected so that they do not contain strength reducing characteristics such as knots, gum veins, splits or checks. These samples comprise wood fibres that are aligned with the long axis of the sample and have no sloping grain. They are used to determine the upper boundary of timber properties for the species from which they are cut. These samples are very strong in tension and when subjected to bending they tend to fail in the compression zone (Bootle, 2004, p. 23).

2.2.2 Condition State

The lack of low cost methods of instrumentation that can be used to determine the structural integrity of timber-bridges has necessitated the extensive use of visual inspection methods (RTA, 2007). In NSW, the RTA published the *Bridge Inspection Procedure Manual* (RTA, 2007, Definitions 17), which specified the procedure for determining each condition state, as shown in Table 2-1. A similar set of conditions is specified for Queensland (QDMR, 2004;2005).

Table 2-1: Condition state identified as part of visual inspection procedure

Condition state	Description
CS-1	The timber is in good condition with no evidence of decay. There may be cracks, splits and checks having no effect on strength or serviceability.
CS-2	Minor decay, insect infestation, splitting, cracking, checking or crushing may exist but none is sufficiently advanced to affect serviceability.
CS-3	Medium decay, insect infestation, splitting, cracking or crushing has produced loss of strength of the element but not of a sufficient magnitude to affect the serviceability of the bridge.
CS-4	Advanced deterioration. Heavy decay, insect infestation, splits, cracks or crushing has produced loss of strength that affects the serviceability of the bridge.

(RTA, 2007, Definitions 17)

2.2.3 Component lifetime

It is critical that a reliable structure such as a bridge be kept in states CS-1 or CS-2 (refer Section 2.2.2) and not decay into a states CS-3 or CS-4 (Kadar, Zavitski, Manamperi, & Mann, 2011). As an example Kadar et al. (2011) discuss the case of painting the Sydney Harbour Bridge and identify that it is more cost effective to keep paint damage to a minimum and repair small faults than allow damage to reach level CS-3 and then attempt repair. If damage reaches level CS-3, major renovation is required. However, to ensure that level CS-3 is not reached the bridge must be instrumented so that any small incident of damage can be detected. A rating of CS-1 is equivalent to ensuring that major components, such as bridge girders, are used for the portion of their life during which they have a low probability of failure (Bazovsky, 1961; Kumamoto & Henley, 1996). As shown in Figure 2-3 the lifetime of a typical component can be broken into three regions. The first is the early failure region; the second the random, or chance, failure period; and the third the 'wear out' period. Early failure tends to be related to manufacturing faults and can be minimized by appropriate quality control during manufacture and commissioning. The useful life of a component is described by the random failure period and is minimized by careful design, selection and manufacture. Components are readily damaged if they are operated under stress conditions that are close to their maximum operating conditions. Towards the end of life components will start to wear out. In relation to many mechanical components, such as bearings; and many electrical components, such as fluorescent tubes; the lifetime is very well known (Bazovsky, 1961; US Department of Defense, 1991). However, in relation to timber-bridge girders the lifetime is not well known. Not only are these lifetimes not well known they can also vary according to local situations. Girders that are in-service in a benign (or stable) environment can last for over 100 years, one example being the roof members of Durham Cathedral, England (Bull, 1993); yet some girders and poles in adverse environments might not last 20 years (Law, Matheson, & Yttrup, 1992a). A major difficulty in determining

lifetime is that as checks and pipes develop in a girder, they can combine within a short period and reduce component lifetime below expectations (refer Sections 2.4.1 and 2.5.3).

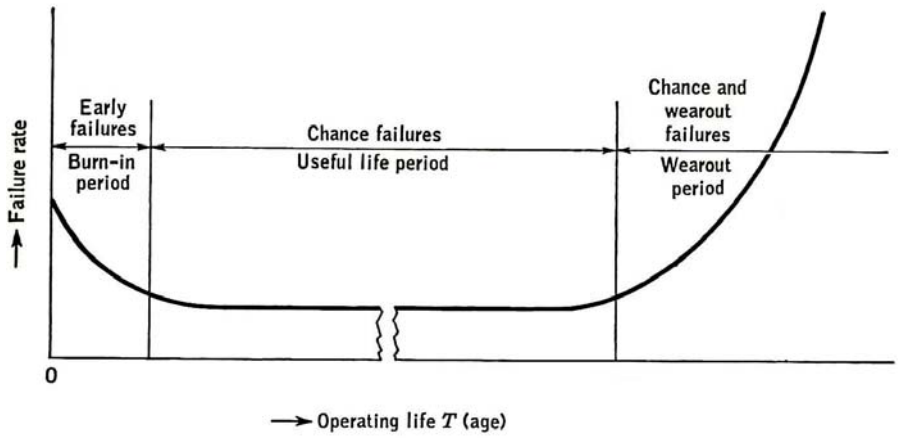

Figure 2-3: Component failure rate-v-time, adapted from Bazovsky (1961, Figure 5.1)

2.2.4 Dynamic deflection

Moving vehicles impose increased forces and stresses on a bridge girder. These increased forces cause increased deflections (Biggs, Suer, Louw, Engineering, & Works, 1956; Fuller, Eitzen, & Kelly, 1931; Mufti, 2001). Mufti (2001) recently summarised the usage of terms related to dynamic deflection using a diagram by Biggs et al. (1956). A simplified form of that diagram is repeated here as Figure 2-4. A reference point placed at the mid-span of a bridge will deflect in response to a load that is moved across the bridge. A static load placed on the bridge and displaced incrementally across the bridge will produce the deflections shown as the curve labelled *Static deflections* in Figure 2-4. If a load is moved continuously across the bridge, any unevenness in the surface will cause the load to move up and down. This is reflected in the measured response and an example is shown in Figure 2-4 labelled *Dynamic deflections*. By averaging the measured response data an intermediate response is obtained, identified as the curve annotated *Median deflections* in Figure 2-4.

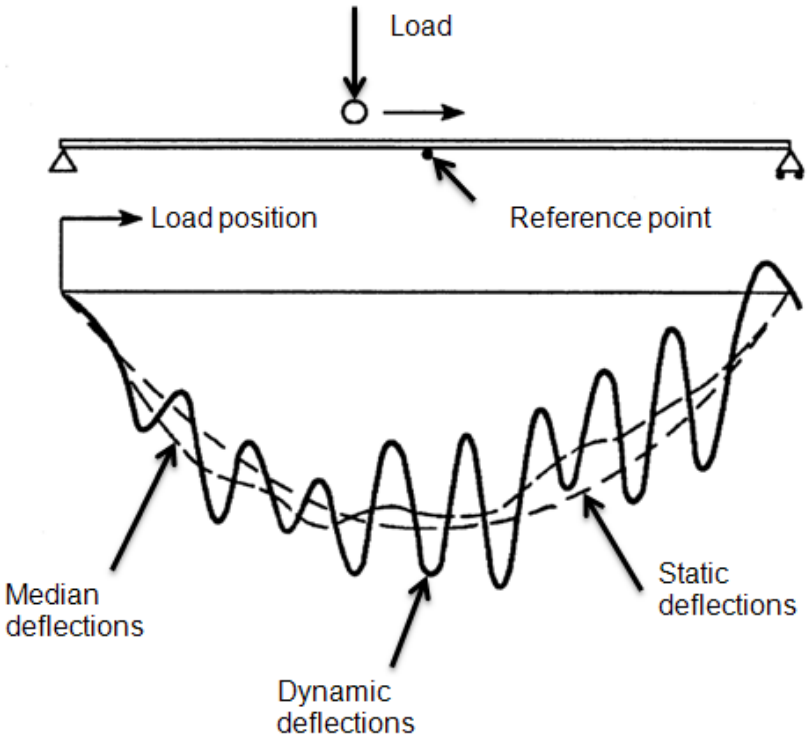

Figure 2-4: Mid-span girder deflections under a moving load, adapted from (Bakht & Pinjarkar, 1989: Figure 1)

2.2.5 Axle load regulations

The peak deflection that occurs on bridge girder is influenced by the maximum load that is applied. This maximum load limit is legally specified in terms of the maximum mass that is to be supported by an axle group. The definition of an axle group is as shown in Table 2-2 (Commonwealth of Australia, 1986). The maximum mass is set at 20 tonnes as shown in Table 2-3 and is limited according to the number of axles. This mass is distributed between the axle group such that the equivalent mid-span point load will be slightly less than 20 tonnes. However, for the purposes of the calculations in this research a mid-span 20 tonnes is specified as the maximum mid-span girder load.

Table 2-2: Axle group definitions, source Commonwealth of Australia (1986)

INTERSTATE ROAD TRANSPORT REGULATIONS 1986 - REG 2 **Interpretation (1) In these Regulations, unless the contrary intention appears:** ***close coupled axle group* means a group of:**	
(a)	2 axles whose centres are not more than 1 metre apart; or
(b)	3 axles whose centres are not more than 2 metres apart; or
(c)	4 or more axles whose centres are not more than 3.2 metres apart.

Table 2-3: Axle load limits, source Commonwealth of Australia (1986)

Commonwealth of Australia Consolidated Regulations **INTERSTATE ROAD TRANSPORT REGULATIONS 1986 - SCHEDULE 4** **Axle load limits (subregulation 12B (1) and 12BA (1))**		
Column 1	**Column 2**	**Column 3**
Item	**Axle group**	**Axle load limit (tonnes)**
1	Single steer axle on a motor vehicle	6.0
2	Single axle or single axle group fitted with:	
	(ii) on a bus licensed to carry standing passengers	10.0
	(iii) on any other vehicle	9.0
5	Triaxle groups:	
	(a) on a vehicle fitted with single tyres with a section width of less than 375mm on all axles, or single tyres on 1 or 2 axles and dual tyres on the other axle or axles	15.0
	(b) on a pig trailer fitted with single tyres with a section width of at least 375mm, dual tyres on all axles, or a combination of those tyres	18.0
	(c) on any other vehicle fitted with single tyres with a section width of at least 375mm, dual tyres on all axles, or a combination of those tyres	20.0
	(d) dual tyres on all axles	20.0

2.2.6 Design perspective of timber beam girders

Structural design relates to the design of civil engineering structures such as buildings and bridges. Such structures are designed by engineers and constructed by builders according to national standards and codes. This design approach requires that any new structural component must have a Modulus of Rupture (MoR) and a Modulus of Elasticity (MoE) above certain defined minimum values. The final owner/manager of a particular structure can then be assured that it will reliably support the design loads with a low probability of failure. This predictability, of a safe structure occurs despite the design engineer, the bridge builder and the material supplier being totally independent organisations. It is a reliable system because each party works to the same specifications and standards.

Building by the design perspective works because everybody involved in creating a structure uses the same standard specifications. Such specifications are used to ensure that all components are known within specific limits. As an example, a set of specifications might specify timber strength as F27 according to AS 1720.1 (Standards Australia, 2010). Timber strength is graded such that users can be 95% confident that a timber member is stronger than the specification. In this example, timber of grade F27 has a bending strength(f_b') of 67 MPa.

2.2.7 Maintenance perspective

This term is not in common use and was not readily identified in the literature. It is introduced here to identify that the currently used design perspective is not the only approach. Structural design of bridges is normally made using a design perspective (refer technical term: design perspective) and not from the perspective of an individual girder. Current structural assessment of aged structures is from a similar viewpoint. The aged structure is compared with its original design specification and a determination made of areas of performance that have deviated from the specification (Standards Australia, 2004c). Typically, aged structures will be visually assessed every two years. A detailed structural assessment might only be made if there is cause for concern, or during a nominated period, say every 10 years. Yet an individual girder may degrade between inspections or not even be visible during such inspections. In order to determine the presence of degradation, or mechanical impact damage, individual girders need to be continually assessed and a parameter, indicative of the degradation of an individual girder, needs to be monitored. There is thus a need for a maintenance perspective.

In this perspective a girder is examined individually. Its current strength is determined in relation to both its current condition and its past history. Individual girder strength is identified as a distribution of values. This distribution is only related to a specific girder and not necessarily related to other girders. When a girder is new its strength can be related to a number of quantifiable prediction parameters such as MoE, moisture content (MC), density, species, incidence of checking and the onset of non-linearity in a load-v-defection test. Each in-service girder will individually age and any temporal variation in these prediction parameters can be used to identify possible changes in the girder strength distribution. If an explicit relationship between the prediction parameter and strength is known then the mean value of the distribution can be modified. One example of this would be a change in the second moment of area (SMA) of the girder, since it is inversely related to MoR. Currently the relationship between any of these prediction parameters and the strength of a timber girder is poorly understood. If the parameter distribution is known it can be used. For example the variation in strength might be known to be 5%. If the distribution is not well specified a much wider distribution might have to be used. As an example although the variation for an individual part may not be known, the variation of all parts might be known. The strength variation of all samples of all ages might be known to be less than 30%. By using the 30% figure, instead of 5%, reliability can be calculated but the design may not be optimal. The essence of a maintenance perspective is a reliability approach that requires the application of SHM techniques.

2.2.8 Prediction band

The calculation of a linear regression line through a dataset provides a model of the mean performance. A mean value of the dependent variable can be predicted from the independent variable. However, for data with significant variation the confidence limit of the predicted value also needs to be known. Two separate cases can be identified. The first case is the prediction of the average response of the dependent variable, x_0, and is shown in Figure 2-5 as a solid black line; the regression line. This prediction is subject to sampling variation and is quantified by the dashed black line close to the regression line. It is 95% certain that the regression line is contained within and between these two dashed lines that is, two standard errors from the mean. In this case the standard error is given by (Krzanowski, 1998, p. 65):

$$\boldsymbol{S_M = s\sqrt{\left\{\frac{1}{n} + \frac{(x_0 - \bar{x})^2}{S_{xx}}\right\}}}$$

..... Equation 2-1

The second case is the prediction of an individual value of the dependent variable for a specified value, x_0, of the independent variable. In this case the standard error is given by (Krzanowski, 1998, p. 66)

$$\boldsymbol{S_M = s\sqrt{\left\{1 + \frac{1}{n} + \frac{(x_0 - \bar{x})^2}{S_{xx}}\right\}}}$$

..... Equation 2-2

The next predicted value of the dependent variable is contained within and between the outer dotted lines of Figure 2-5 to a confidence level of 95%; two standard errors from the mean. The important features to note from the curves in Figure 2-5 are that both standard errors depend on the value of the independent variable and that the variances increase quadratically as the independent variable moves away from the mean value, $\bar{x}$. The regression line prediction band is thus limited by the dashed lines to a 95% confidence level and the next predicted individual value is limited by the dotted line prediction band to a 95% confidence level.

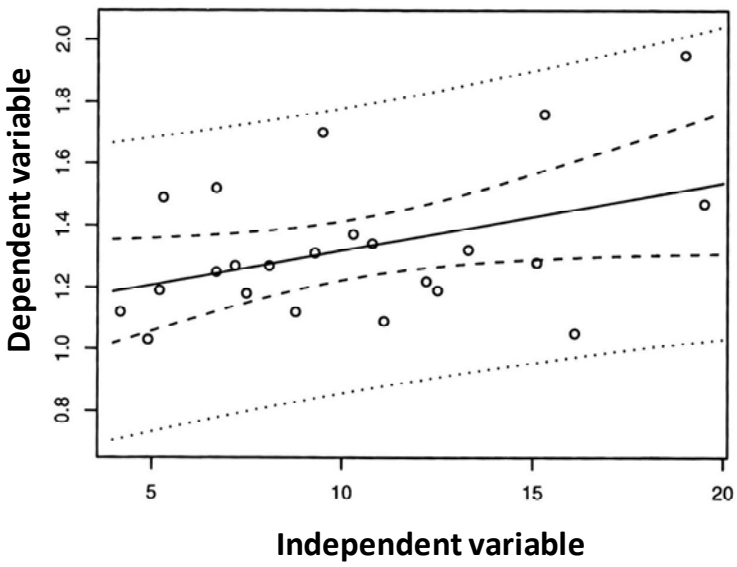

Figure 2-5: Sample linear regression with prediction confidence bands, adapted from Dalgaard (2008, p. 121)

2.2.9 Proof Test

A proof test is required to identify a bridge that is structurally performing to the design criteria. Such a test might be made after commissioning to confirm, to the bridge owner/manager, that the structure performs to its design. A test might also be made later in the life of a bridge to ensure that it maintains performance to the design criteria. Particularly in the case of a bridge that is of unknown structural condition it is unwise to fully load it to capacity in one step. Therefore, "in order to protect the bridge and the testing personnel, proof test loadings shall be applied incrementally from a base load of 50% of the theoretical rated ultimate capacity … Testing shall be terminated when non-elastic behaviour is observed" (Standards Australia, 2004b, AS 5100.7, Section 6.1).

2.2.10 Proof Load

The proof load is the maximum load that does not produce non-elastic, or non-linear, behaviour. However, to determine the presence of non-linearity the structure under test must be loaded to the non-linear region. A theoretical target proof load is estimated to minimise the degree of overloading that might occur. The bridge is then carefully loaded up to this target limit, but no additional loads are applied once non-linear behaviour is identified (Standards Australia, 2004b, AS 5100.7 Section 6.3.3).

2.2.11 S2 limit state

The S2 limit state is a specification of the required strength and elasticity for a replacement girder (refer Section 2.5.5). A sound timber-bridge girder should have MoE above 14 (GPa) and

MoR above 86 MPa. This term is specified here as the lower limit for in-service girders. Girders below this limit should be removed from service.

2.2.12 Ultimate strength

The ultimate strength of any material occurs when the strain energy equals the surface energy of the cracks that exist after failure (Gordon, 1991, p. 71). The stress required to produce this level of strain energy is approximately equal to the MoE value; refer Gordon (1991) and Figure 2-6. It is not normally possible to achieve this level of stress in most materials because of inherent defects. One example of a material with significant defects is normal window glass. If these defects are removed then the strength of normal glass is considerably increased. One way of removing these defects is to heat the glass and pull thin strands from it. The level of increase in strength that can occur in thin glass fibres is shown in Figure 2-6. This level of increase is not limited to glass and recent research has shown that nanocrystalline cellulose, or purified wood, has eight times the tensile strength of stainless steel (Ferguson, 2012).

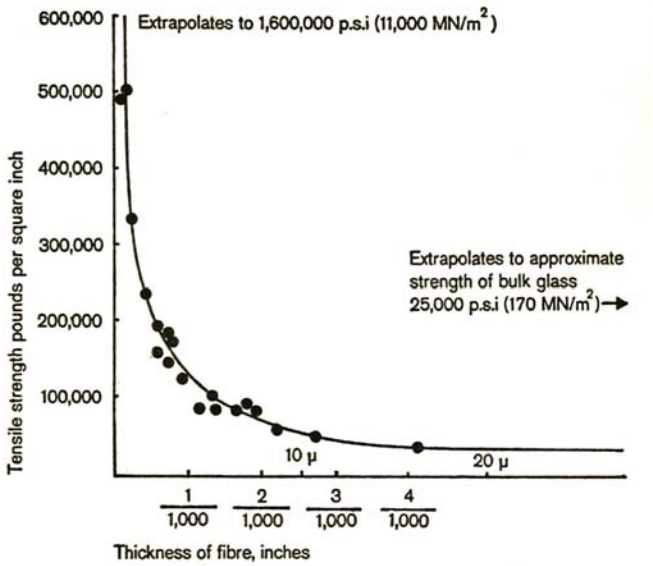

Figure 2-6: Tensile strength-v-glass fiber thickness (Gordon, 1991, p. 75)

2.2.13 Vector trajectory

All of the MoE and MoR data presented in current standards is in tabular form. Some researchers such as Yttrup & Nolan (1996) have presented their data in a graphical form with MoE as abscissa and MoR as ordinate. However, they have only presented these data in a static form with no temporal information. In Chapter 4, data available with effective temporal information are presented as a vector. That is, at one moment in time, a girder may have a first MoR_1-v-MoE_1 value and at another moment in time it may have a second MoR_2-v-MoE_2 value. The MoR-v-

MoE vector trajectory is specified here as the path in time and direction between the first and second MoR-v-MoE values.

2.3 Historical Background of the use of Timber for Bridge Girders

Australia, and particularly NSW, has had a long association with timber-bridges. Because of the availability of suitable hardwoods in the 19th Century and the lack of suitable economic alternative materials, many rural bridges were constructed from local timber. Hardwood timber was seen to be a very sustainable material, locally grown, whereas alternative materials (such as iron) were imported and expensive due to high transportation costs. The engineers in the late 19th and early 20th centuries improved on traditional European and contemporary American designs to suit local Australian conditions. Some of these early designers were: Allan (1861 – 1930); Dare (1867- 1949); de Burgh (1863 – 1929); and McDonald (c. 1850 – 1933). William Henry Warren (1852 – 1926), as Foundation Professor of Engineering at Sydney University, was instrumental during this period in instigating pivotal work characterising potential Australian hardwoods for use as structural members, and particularly as bridge girders. These improved designs were discussed by Glencross-Grant (2009, Figure 3; 2011).

A characteristic timber beam bridge design of this period is shown in Figure 2-7. What is surprising about this design is firstly that there are many thousands of timber-bridges that are of similar basic design that remain in service carrying modern vehicle loads and traffic volumes (Howard, 2009; Roorda, 2006); and secondly, that the design has not varied significantly from the original. Indeed, Newell (1938) reported that this basic design had been around since the early 1860s.

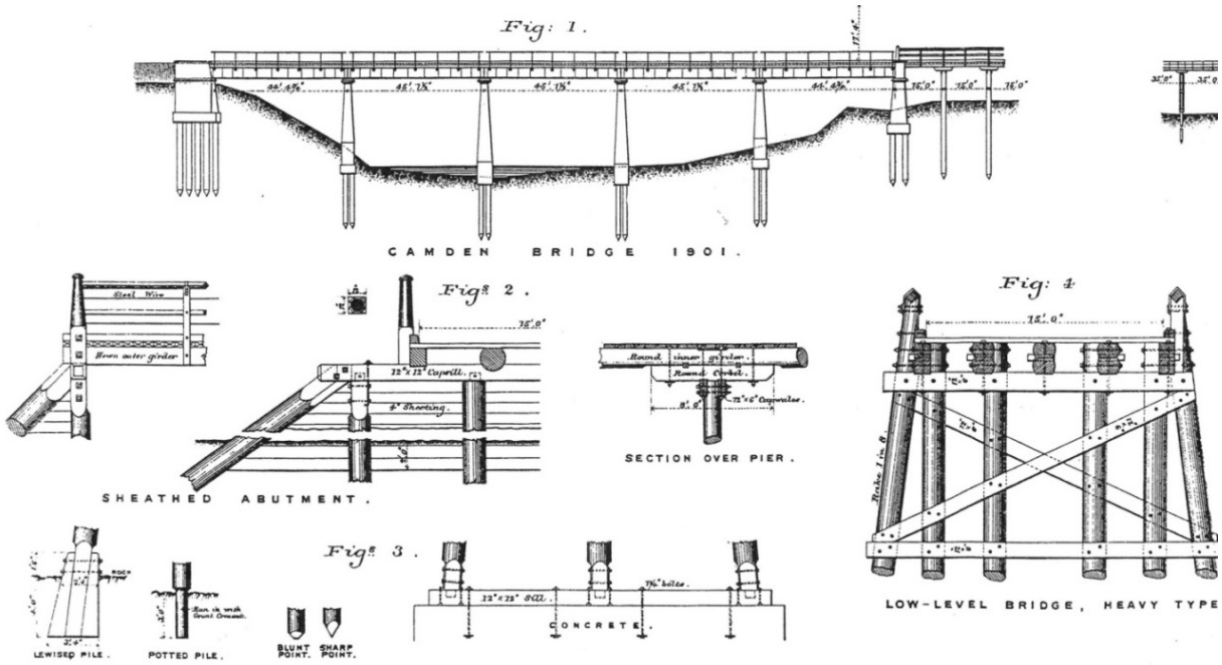

Figure 2-7: Plans for Camden Bridge, ca. 1901 (Dare, 1903)

The challenges that designers in the 19th Century were grappling with related to the expected lifetime of materials (durability) and increasing volume and mass of loads that were travelling on roads and crossing bridges, which of course remain topical. One example of a heavy load of this period is shown in Figure 2-8. Allan (1895) identified the typical design criteria for such a load that the bridge should support "16 tons and a distributed live load of 18.8 cwts per foot run" (Allan, 1895). In SI units (Chiswell & Grigg, 1971) this is equivalent to a distributed live load of c. 5 kN m^{-2}.

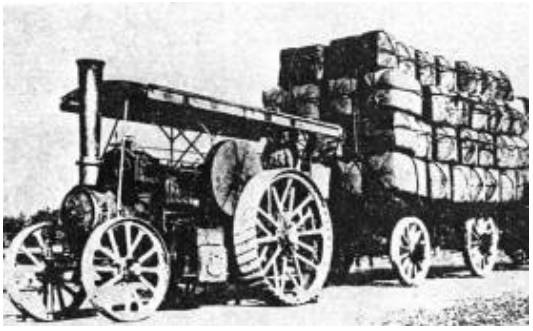

Figure 2-8: 19th Century traction engine towing a wagon load of wool bales (Coltheart & Fraser, 1987, 51)

The fact that bridges constructed to these early designs are still carrying modern traffic loads of 30 tonne gross vehicle mass (GVM) and greater is an indication of the conservative safety factor that was inherent in the design. The deflection of a local single span timber-bridge in the Armidale area, built c.1930, to the 1860s design (refer Figure 2-9), was measured in 2008 (Moore, 2009). It was found to be capable of supporting 16 tonne with a mid-span deflection of

about 25 mm, which equated to a span:deflection ratio of about 340:1. The original designers would probably have accepted this ratio as a reasonable figure. However, current design standards are more demanding because of the expectation of the use of more easily fractured decking materials such as concrete. In such standards the span to deflection ratio is specified as 600:1 (Standards Australia, 2004c, AS 5100.2, Section 6.11), although some lower figures, such as 380:1 and 450:1, have also been suggested for flexible decks (Crews & Ritter, 1996).

Other features of recent design standards are also more demanding. AS 5100.2 also specifies the design life as, "... the period assumed in design for which a structure or structural element is required to perform its intended purpose without replacement or major structural repairs" (Standards Australia, 2004a, AS 5100.1, Section 4.2). It is also specified that the design life of structures to be 100 years (Standards Australia, 2004a, AS 5100.1, Section 6.2). Such a period without significant structural repair is a very demanding criterion. The bridge shown in Figure 2-9 has lasted about 80 years, but it has been structurally repaired many times during its life. These repairs have involved the replacement of several of the major components. This bridge across Powers Creek on Dangersleigh Road, 10 km south of Armidale NSW, comprised four supporting girders and had a timber deck comprising cross-decking at least 100 mm thick. A longitudinal view of this bridge is shown in Figure 2-10.

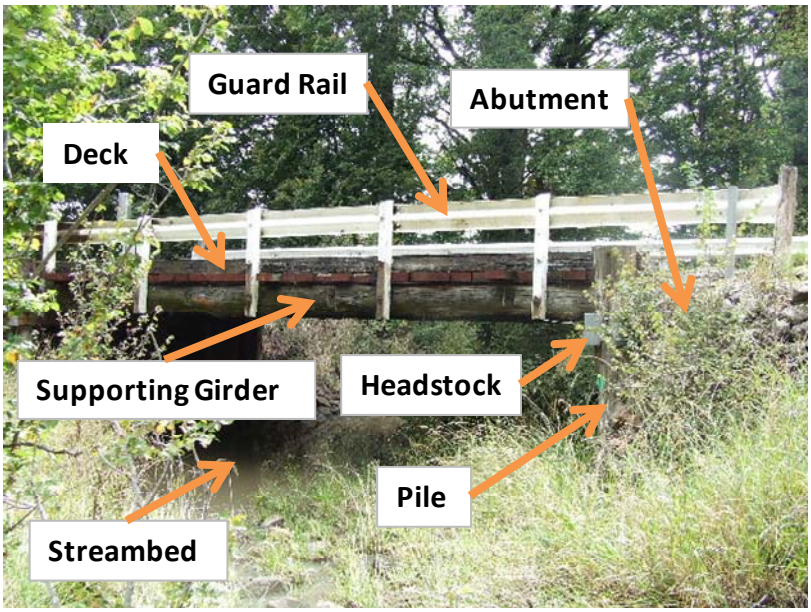

Figure 2-9: Single span timber-bridge (Powers Creek Bridge, 1930 – 2011)

Figure 2-10: Longitudinal view of Powers Creek Bridge

Although the majority of Australian timber-bridges are similar to those designed by 'pioneering' engineers such as McDonald, Allan and Dare, there have been some later designs, such as the composite timber and concrete structures built in Tasmania (Nolan, 2006) and more recently in Gloucester Shire, NSW. An example of this latter structure is a bridge constructed in 2010 to replace Leslie's Bridge across the Little Manning River near Bretti on Thunderbolts Way, between Gloucester and Walcha (Roads and Maritime Services, 2012). The original bridge was a single lane low-level timber-bridge of two spans and susceptible to flooding (Figure 2-11). The replacement structure comprised a monolithic concrete deck supported on round timber girders and corbels, which are in turn supported by a steel substructure fixed to reinforced concrete footings (Figure 2-12). The former bridge was below the normal flood level and occasionally inundated, whereas the new structure has a much lower probability of overtopping by flood waters. Thus, the replacement structure has improved the level of service to traffic in the area by providing a greater level of access and an improved approach alignment.

Figure 2-11: The former low-level Leslie's Bridge, over Little Manning River, Bretti, NSW

Figure 2-12: The replacement high Level composite steel/timber/concrete bridge (Leslie's bridge)

The timber used in these designs has been governed by the strength of available hardwoods and, historically in NSW, has been limited by specification to a select range of timber species that was colloquially called the 'Royal Species'. In an ornamental planting in Grafton, Northern NSW there are examples of five hardwood species characterised as 'Royal Species' (*pers. obs*.). These species are shown in Table 2-4 together with their characteristic values of MoE and MoR.

Table 2-4: Names of timber species identified as the 'Royal Species'

Common Name	Scientific Name[1]	MoE (GPa)[2]	MoR (MPa)[2]
Grey Box	Eucalyptus moluccana	20	163
Tallowwood	Eucalyptus microcorys	18	138
Coast Ironbark	Eucalyptus siderophloia	24	181
White Mahogany	Eucalyptus acmenoides	17	130
Grey Gum	Eucalyptus biturbinata	NA	NA

Note 1: (Boland et al., 1984)

Note 2: (Bolza & Kloot, 1963; Bootle, 2004)

Other species were also used, with the choice dependent on availability of local species. Potential species for structural use, as identified by Boland et al. (1984), are shown in Table 2-5, with their currently accepted values of MoE and MoR. These timbers were chosen primarily because of their strength. A common plantation species timber currently used for commercial construction is *Pinus radiata.* It has typical MoE and MoR of 10 GPa and 81 MPa respectively. Grey Box (*Eucalyptus moluccana)* typically has values of both MoE and MoR that are twice those of *Pinus radiata.* These differences together with durability are the reasons that species such as *E. moluccana* were preferred species for bridge construction

Table 2-5: Some other possible species for use as timber-bridge girders (Boland et al., 1984)

Common Name	Scientific Name[1]	MoE (GPa)[2]	MoR (MPa)[2]
Grey gum	*Eucalyptus punctata*	18	140
Ironbark grey Queensland	*Eucalyptus drepanophylla*[3]	24	183
Narrow leaved red Ironbark	*Eucalyptus crebra*	16	118
Spotted gum	*Eucalyptus maculate*	19	142
Gum-top ironbark	*Eucalyptus decorticans*	17	158
Gum grey mountain	*Eucalyptus goniocalyx*	18	141
Grey Ironbark	*Eucalyptus paniculata*	23	185
Woollybutt	*Eucalyptus longifolia*	16	128
Ironbark red	*Eucalyptus sideroxylon*	17	149
Tasmanian blue gum	*Eucalyptus globulus*	20	147
Silvertop ash	*Eucalyptus sieberiana*	17	137
Brown top stringy bark	*Eucalyptus baxteri*	17	126

Note 3: Six sigma range is MoE 22 – 26 GPa and MoR 169 – 197 MPa (Bolza & Kloot, 1963)

Individual girders are not usually proof-tested for individual strength in the construction of contemporary timber-bridges. New designs are based on the general fifth percentile strength recommendations provided in current standards rather than the documented mean strength of particular species (Boughton & Crews, 1998; Law, Matheson, & Yttrup, 1992b; Standards Australia, 2010). This is an example of the design perspective being enacted. Typically, a design will include factors that account for possible future variations in girder material. Parameters such as strength, elasticity, moisture content and size factor are considered together with factors that account for the variation in basic working stresses caused by loading. The final design will contain a factor of safety (FoS) that may vary from a value of 10 in a new structure to a value of about 3.5 in an older structure (Law et al., 1992b). The short-coming with this approach is that such calculations fail to identify FoS in aged girders. Because of the variability in girder strength the calculations are based on mean strength of a specified population of girders. One example of the difficulties that can be encountered in applying this method to old structures was mentioned in Section 1.1 with respect to a bridge in Singleton NSW that failed catastrophically (HCA, 2001).

Most bridge owners and managers do not have detailed historical records of individual girder strength. Even a post-service analysis is unlikely to accurately identify how the effects of checking, splitting and piping affected performance. It is important, therefore, in the case of aged timber-bridges to quantify individual girder material parameters to more effectively quantify in-service working stresses. Current methods of girder strength analysis do not take into account in-service measurements in real time, nor the correlation of girder behaviour over time. This deficiency provides more than appropriate justification for this present research.

2.4 Historical loading concerns affecting timber-bridges

2.4.1 Failure under load

In NSW vehicles are driven across timber-bridges, that is part of the public road network, with little concern. This is providing that such vehicles are within load limits of particular bridges. The difficulty for the authority owning/managing the bridge is how to define the load limit. The authority needs to identify whether specific signage is required to instruct users about what is safe compared to the approach road. As has been already discussed, the specified design strength of girders is one factor that needs to be considered when defining a load limit. But there are other factors that also contribute to the safe operation of girders. The first is piping, which is unseen and only significant in degraded girders. Another factor is that kerb girders on some bridges may be of smaller diameter than the main central girders. Kerb girders can be significant if a heavy vehicle is not driven centrally across a bridge. Speed is of concern in terms of its effect on impact loading and how girder fastening systems are affected by vibration. A further factor is the distribution of traffic that is likely to use a particular bridge. These factors are briefly discussed with regard to how they can affect bridge safe use.

2.4.2 The effect of piping

The level of piping in new girders is low and, if less than about 20% of the cross-sectional area, should not significantly reduce girder strength unless severe checking is present (as an example refer Figure 2-22, page 36). It is not until the piping percentage reaches around 60% of the cross-sectional area, or more, that the mid-span deflection is significantly affected. At 60% the mid-span deflection can increase by several millimetres; a measureable amount. The nominal deflection that might occur in a girder for a constant outer diameter pipe is shown in Figure 2-13 (Moore, Glencross-Grant, & Patterson, 2009).

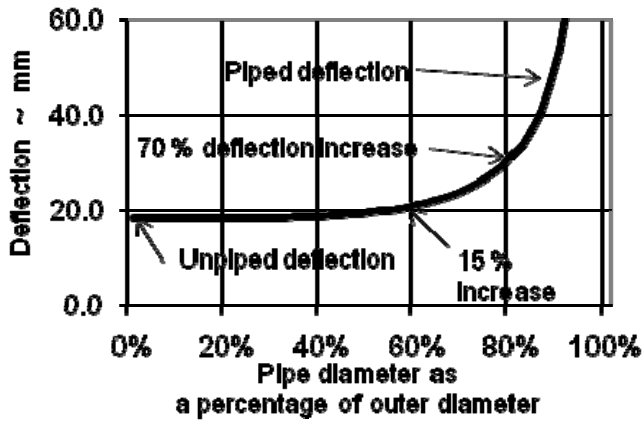

Figure 2-13: Effect of piping on deflection while under constant loading conditions (Moore et al., 2009, Figure 1)

There are three prime reasons why piping can be of significant structural concern. The first is that the pipe diameter exceeds 60% of the external diameter. The second is if the pipe occurs in combination with other defects, as shown in Figure 2-22. The third is when the pipe becomes a habitat for termites and fungus. If the termite activity causes sound wood in the highly stressed regions to be removed, then such activity is of more concern than if sound wood in lowly stressed regions is removed. There is no apparent evidence in available literature when this concern has been addressed. As already discussed, Curling et al. (2001) examined fungal activity on hemi-cellulose removal, but work examining the activity of termites in stressed and un-stressed timber was not able to be located. The importance of hemi-cellulose to girder strength is also scantly addressed. In regard to timber-bridge girders, it is important to monitor the level of piping because it can vary temporally, and quite quickly at times, especially if termites are active.

2.4.3 Non-central load distribution

Early timber beam bridge designs, c. 1900, comprised:

> Longitudinal beams that were usually 12 inch x 12 inch dressed or 15 inch (380 mm) logs dressed only at their ends to sit flat on the corbels. Both 12 inch x 12 inch beams and 15 inch round beams were used in timber beam road bridges. The most common arrangement was for the dressed or squared timbers to be on the outside for appearance sake and the round logs with their bark attached "hidden" in the interior.
>
> (RTA, 2006, 19)

There are many timber beam bridges still in service that have followed this concept and have been designed with kerb girders that are of smaller diameter than the central main girders. However, with today's heavy vehicles this type of design can lead to unexpected overloads. The structural failure shown in Figure 2-14 was initially caused by the failure of timber decking. This failure caused a shift in the centre of gravity, thus transferring excessive load to the kerb girder, which then failed. The failure may also have been exacerbated by the driver of the vehicle not driving centrally across the bridge. It is interesting to note that only the kerb girder failed, the vehicle was still supported by the central girders. If the load had been better supported by the two central girders, by virtue of a narrower wheelbase or a multiple wheel low loader, the failure would probably not have occurred.

Figure 2-14: Example of structural overload on a timber beam bridge, with catastrophic failure of a kerb girder, c. 1975 (Photo: R. Glencross-Grant)

2.4.4 Multiple span bridges and the effect of corbels

A timber-bridge can be a single span structure or it can consist of many spans. An example of a single span structure was Powers Creek Bridge, shown in Figure 2-9. An example of a multiple span structure, Munsie Bridge, is shown in Figure 2-15. Munsie Bridge is situated about 10 km east of Uralla, NSW, over Salisbury Waters at Gostwyck. Each span consists of four 10 m long girders mounted on and bolted to the corbels. The piers are concrete with a timber headstock and as with the Powers Creek Bridge the decking is cross-planking. In contrast to the Powers Creek Bridge deck, which was relatively smooth and allowed vehicles to cross at 100 km h^{-1} the Munsie Bridge deck is uneven (varying thickness and varying degrees of fastening) and traffic crosses at low speed.

Figure 2-15: Munsie Bridge, Gostwyck - an example of a multiple span bridge

Figure 2-16: Munsie Bridge- uneven timber decking

The mid-span deflection of a particular girder in a multi-span bridge will be affected by the loadings and deflections that occur in adjacent spans (RTA, 2008, Section 4.6.3.3; Yttrup, Law, & Audova, 1991). How a particular span is affected will depend on: the axle loads; the distance between axles; the corbel lengths; the degree of tightness between the corbels and the girders; and the stiffness of the girders and corbels. Thus, each corbel may behave differently depending on the level of continuity between spans. Yttrup, Law, & Audova (1991) report that on a particular bridge upward deflections of about 20 mm occurred on the two spans adjacent to a loaded span that deflected downwards about 60 mm when that span was loaded to about 80% of the specified load limit (Yttrup, Law, & Audova, 1991). The inference from these data is that each bridge must be independently reviewed to determine the level of interconnectedness, or continuity, between spans and how it might affect temporal deflection recording. The fastenings between the girders and other components are likely to be more stressed on a multiple span bridge in comparison with a single span, but no literature can be cited that quantified such vibrational effects.

2.4.5 Traffic loading

In NSW the number and the type of all registered vehicles are recorded by the Australian Bureau of Statistics (ABS). An example the number and type of vehicles on NSW roads distributed by the Ford Motor Company of Australia are shown in Table 2-6 (ABS, 2008). The related kerb masses, consistent with each type, are inferred from manufacturers' data. The number of light vehicles (less than 3 tonne) on NSW roads is determined by correlating: vehicle types; vehicle kerb masses; and number of vehicles of each type. The result, about 80%, is shown in Table 2-7.

The inference from these data is that the mass of a light vehicle will be below about 3 tonne GVM. Previous research (Lake, 2001) addressed the concept of identifying bridge health by examining the structural response to loading by heavy traffic. While this is appropriate for large highway bridges, with significant heavy vehicle traffic, it is not appropriate for small timber-bridges with less than 20% heavy traffic. Heavy vehicles will cause the most stress to a deteriorating timber-bridge, but the deflection caused by light vehicles can be measured without stressing the bridge. Light vehicles can also provide a temporal record with about ten times more data in the same measurement period. Thus, with much more data, trends can be more readily determined.

Table 2-6: Number of vehicles in NSW distributed by the Ford Motor company of Australia and registered in 2008 (ABS, 2008)

Model	No. manufactured	Kerb mass[1] (kg)
Courier	24794	1560
Econovan	9035	1493
EL Falcon	14849	1600
Escape	6070	1527
Fairlane	8554	1651
Fairlane/LTD	6927	1676
Fairmont	6600	1575
Falcon	80188	1690
Falcon Cab chassis	6294	1694
Falcon Fairmont	72500	1560
Falcon GLi/S	86508	1510
Falcon ute	31733	1795
Festiva	29625	952
Fiesta	6587	1062
Focus	20742	1350
Laser	67133	1125
Telstar	9538	1260
Territory	21509	2085
Transit van/cab chassis	9179	1410

Note 1: Kerb mass is the total mass of a vehicle with standard equipment and all operating consumables, such as a full tank of fuel, but without passengers or cargo.

Table 2-7: Percentage of light vehicles on NSW roads (ABS, 2008)

Parameter	Approximate value
Number of vehicles on NSW roads	4.4 million
Number of light vehicles below 3000 kg	3.4 million
Percentage of light vehicles	80%

2.4.6 Summary of historical use of timber for bridges

The way timber has been used to construct, inspect and repair timber beam bridges has not changed significantly in Australia since the early part of the 20th century. Most of these bridges are still evaluated by visual inspection procedures and can have faults that are not visible. Conversely, they can appear to be degraded, but in fact have adequate strength for their task.

Bridge assessment and analysis is carried out from a design perspective and there is little literature reporting any alternative approach. It is because of concerns about girder strength that some bridges are being replaced, yet in most cases there is no attempt to adequately quantify that strength. Again it is the absence of literature that is notable. In 1969 it was reported that in Central Queensland bridges required significant maintenance (B.J.P., 1969). The article details the mechanical work that was carried out and the tools that were used but there is no mention of any measurement being made. Nor were more recent reports detailing regular quantification, or temporal change, of timber bridge girder performance found for this review. It is, therefore, unknown whether the strength of any particular girder over time is stable or declining. Because of this lack of quantification, bridges are possibly being replaced that seemingly do not need to be replaced.

2.5 Variation of timber girder MoE and MoR

There does not appear to be publicly available data that contain the temporal records of bridge girders showing how they degrade temporally under service conditions. Nor are there readily apparent reviews of the variation that might be expected in individual aged timber girders. The reason for this is that solid round timber-bridge girders are typically evaluated from a design perspective. This section addresses the current literature that contains data from which an indication of the aged timber properties of individual girders can be inferred.

2.5.1 Tasmanian girders

Yttrup & Nolan (1996) have reported that timber beam bridges have been extensively used in Tasmania, with about 3500 timber-bridges on the state road network being constructed from native hardwoods. They do not report the species utilised, but describe the timbers as being "...not as hard, strong nor durable as those found on mainland Australia" (Yttrup & Nolan, 1996). As these Tasmanian bridges reached the end of their life, or were replaced because a road was re-aligned, they were first load tested and then the girders recovered and tested as part of the University of Tasmania (UTas) timber-bridge research project. Some 50 timber-bridges were field tested and about 200 girders tested to failure in the UTas laboratory. These data from testing 95 girders were presented in a graph that has been adapted by adding clearer axes (see Figure 2-17). The complete measurement procedure has not been publicly recorded, but it was stated that for the 95 girders:

- The beam sectional properties were estimated by measuring the gross external dimensions, with no reductions for decayed wood areas.
- Each girder was placed in a purpose built, 100 tonne three point loading machine with the girder ends supported on sound sections of the girder and the load applied at mid-span.
- The nominal MoE was determined from the linear part of the load-v-deflection curve (an example curve was published) and the nominal MoR from the point of failure.

(Yttrup & Nolan, 1996).

Statistical data provided by UTas included a regression equation (MoR = 3.36 + 2.79 x MoE) and a coefficient of determination of 0.52. Yttrup and Nolan did not provide a prediction band for these data. What is of immediate interest in these data is that there are no values of MoR above 86 MPa; that is the S2 limit state. Although it is reported that these data are calculated without any reductions for decayed areas they are still unusually low. The species that are reported by Boland et al. (1984) as suitable for structural use in Tasmania, such as Tasmanian blue gum (*Eucalyptus globulus*), brown top stringy bark (*Eucalyptus baxteri*) and silver top ash (*Eucalyptus sieberiana*) all have values of MoR and MoE similar to other suitable species referred to in Table 2-5. Yttrup and Nolan (1996) discuss the strength of girders from the design perspective but do not explain how the MoR-v-MoE vector trajectory of these timbers moves from acceptable values to unacceptably low values. There is a temporal movement from new MoR-v-MoE values to the published aged MoR-v-MoE values but no attempt to quantify this movement. Also, from what is published about this dataset it is not clear if similar results should be expected from extant girders in NSW.

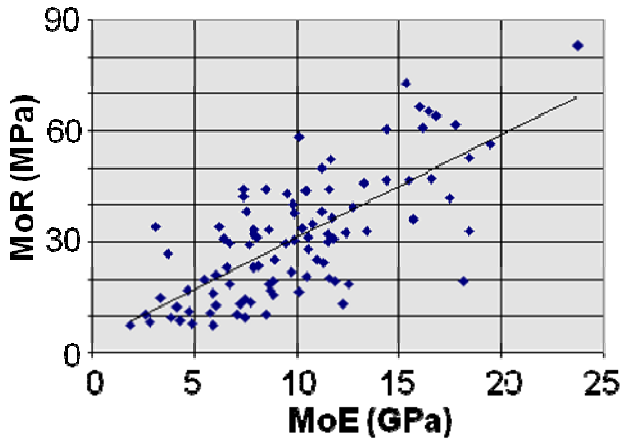

Figure 2-17: MoR-v-MoE data for Tasmanian timber-bridge girder, adapted from Yttrup & Nolan (1996, Figure 4)

2.5.2 NSW electrical poles dataset

Data were gathered as part of a project examining power poles in NSW (Crews, 1998; Decosterd, 1998; Horrigan, Crews, & Boughton, 2000). That project examined the strength of poles at the ground line and not at mid-span. It involved poles of a wide variety of ages and quality and involved a wider range of timber species than the Royal Species. The critical step in identifying the strength of timber-bridge girders is to identify whether there is a non-destructive parameter that equates to girder strength. An initial candidate parameter is MoE, which can be readily measured for an in-service girder with appropriate apparatus. Samali, Li and Crews (2007, Figure 3) present MoR-v-MoE data, which is reproduced here as Figure 2-18. The significant variation demonstrates that there is no apparent relationship between MoR and MoE (often referred to as a 'cloud diagram'). However, what can be estimated from this dataset is that the variation of the values of MoR for degraded timber can be about 30% CV.

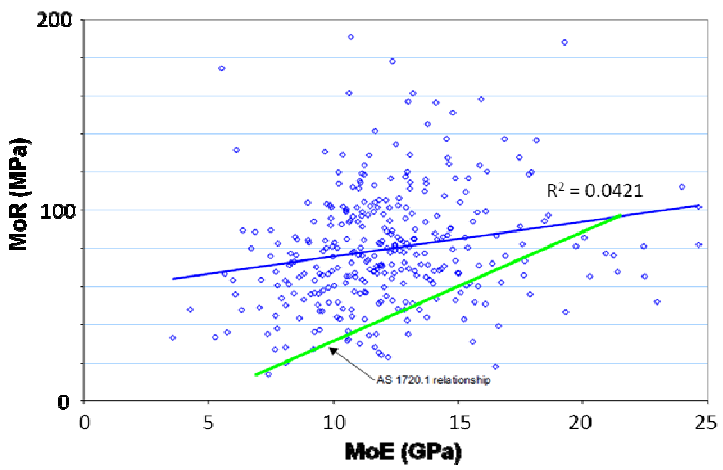

Figure 2-18: MoR-v-MoE for round timber poles, adapted from Samali et al. (2007, Figure 3)

These data need to be examined according to individual ages, condition state, species and other specifically relevant sub-grouping in order to determine any relationships between these parameters. It is currently not possible to conduct such an independent analysis since the detailed data cannot be found in readily sourced literature. Horrigan (2000) has published some summary data and these are shown in Table 2-8.

Table 2-8: Comparison of MoR for a population of new and ex-service electrical utility power poles (Horrigan et al., 2000, Table 1)

NEW POLES		EX-SERVICE POLES (5 to 35 years)	
No. of Samples	60	No. of Samples	103
Coefficient of Variation	35%	Coefficient of Variation	36%
Fifth Percentile	103.5 MPa	Fifth Percentile	56.6 MPa
Characteristic Strength	99.2 MPa	Characteristic Strength	55.5 MPa

This summary is presented from a design perspective and a conservative fifth percentile characteristic strength calculated. There are no data presented that relate to individual poles. However, it is apparent that there is significant physical degradation within a relatively short period- 5-35 years. Unfortunately, too much information has been averaged to provide the meaningful trajectory that the MoR-v-MoE vector of a particular pole could follow during its life.

2.5.3 NSW girder datasets

RTA N150

In 1990 the RTA began a girder testing program which involved testing girders from regional bridges. Full scale testing was performed in a portable test rig similar to the type shown in Figure 2-19. This testing was conducted at 20 different locations throughout NSW. Amongst the 169 girders tested there were ones of different age, species and degree of deterioration. The girders were bent in three point bending with the mid-span load and deflection being incrementally recorded. The resulting data were published in an internal RTA *Girder Rig Testing Report No. N150-GRT* (Law et al., 1992a) and some aspects were reported publically by Law and Morris (1992). From these data gained a linear regression line through each load-v-deflection curve was used to estimate the girder stiffness. The SMA was estimated from the external vertical and horizontal diameters of the girders, with the cross-section assumed to be elliptical in shape. In estimating these sections, defects such as piping, checking and rot were specifically not taken into account, neither were they recorded in the available parts of the N150-GRT report (Law et al., 1992a). The age of the girders was estimated by RTA personnel.

Figure 2-19: RTA portable girder test rig showing three point loading of full-size specimens, c.1990 (RTA photo, unknown source)

A further 21 girders were subjected to full-scale bending tests in the laboratory, clear samples were excised from regions of low stress and also tested. These data were published in an internal RTA *Report No. 150/06* (Law et al., 1992b). Additional information about this work was also published (Law, 1989; Law, Yttrup, McTackett, & McNaught, 1991; Yttrup, Law, & Audova, 1991; Yttrup, Law, & Subramaniam, 1991).

Law and Morris examined these data from a design perspective and reviewed whether the AS 1720 and NAASRA Bridge Design Specification codes of the time correctly represented the required reduction factors for the MoR and MoE of aged girders. They concluded that the codes were not conservative. The Law and Morris data (Law & Morris, 1992, Figure 5) are reproduced here as Figure 2-20. The MoR data appear to be normalised according to the mean values obtained from new girders and the measured values of the aged girders cover a range of about 34% to 74%; representing a mean reduction of about 46%. Since there is no evidence that the normalisation levels are specified, the published MoR values cannot be interpreted in comparison with the MoE values.

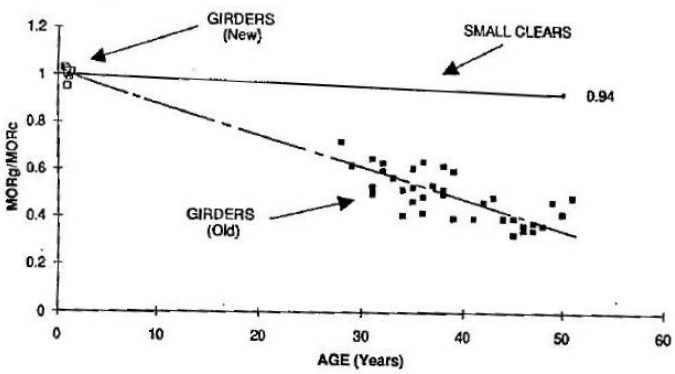

Figure 2-20: Normalised MoR-v-Age,

Source: Law and Morris (1992, Figure 5)

Yttrup, Law, & Subramaniam (1991) discuss the difficulty and uncertainty of defining the effective SMA for a particular girder, with two distinct areas of concern identified. The first concern was the contribution of sapwood to both MoE and MoR. The second concern was the combined contribution made of checks and pipes. In the relevant bridge code of that time (NAASRA Section 10) it was recommended that a minimum of 15 mm of sapwood should be ignored for the purpose of structural calculations. In some current Australian Standards dealing with timber (Standards Australia, 2000;2004b;2010) 'sapwood' is not mentioned. In the RTA *Timber-bridge Manual* (RTA, 2008) it is mentioned once in Section 1.3.5.2 where it is identified that it should be inspected in all components. In the QDMR *Bridge Inspection Manual* it is specified that "Hewn or sawn girders will generally not have any outer sapwood except in the case where copper chrome arsenate (CCA) preservative treatment has been applied" (QDMR, 2004). The approach taken in the QDMR *Timber-bridge Maintenance Manual* is somewhat more instructive. Reference is made to factors such as the 'shaving factor' (k_{21}), which is explained in AS 1720 (Standards Australia, 2010) but the approach is from a design perspective and the outcome is intended to be conservative such that strength is not over estimated (QDMR, 2005; Standards Australia, 2010). The concerns of Yttrup, Law, & Subramaniam (1991) are thus still valid in relation to aged timber beam bridges, since the current design standards, codes and manuals refer more to new designs than existing structures.

The concerns of Yttrup, Law, & Subramaniam (1991) in relation to sapwood are best re-iterated:

> Sapwood is not as durable as hardwood unless correctly treated. It is, therefore, normal to neglect the sapwood in structural calculations. NAASRA, Section 10, suggests that a minimum of 15 mm of sap should be ignored. Because of low durability of the sapwood ignoring it in strength calculations is probably prudent. For deflection calculations the stiffness contribution of the sapwood is very substantial and completely ignoring the sapwood is unrealistic.
>
> The engineer is confronted not only with a tapered section, but also trimming causing large changes to section. To compound these difficulties there is the uncertain depth of sapwood and whether to ignore all of it or part of it. In a 400 mm diameter log ignoring 15 mm of sapwood changes the SMA, 'I', by 26%, and 20 mm of sapwood by 34%.
>
> For first order investigations of timber-bridges the section properties of the beam at mid-span can be used as the equivalent uniform section. The overall

> depth and width of the beam, plus sapwood should be measured. The assumption that the beam [cross section is a] trimmed circle allows a convenient method of computing section properties. For strength calculations ignore all untreated sapwood. For serviceability checks, include the sapwood as part of the section. This is partly justified by the observation that real bridges are invariably stiffer than predicted using heartwood section only
>
> (Yttrup, Law, & Subramaniam, 1991)

The implications of these comments are quite clear. The first implication is that sapwood is a girder component that contributes to girder performance, but not in any well quantified manner. The second implication is that the concern has been minimised in modern designs by either removing it or treating it. An example of sapwood in a newly sawn hardwood log is shown in Figure 2-21.

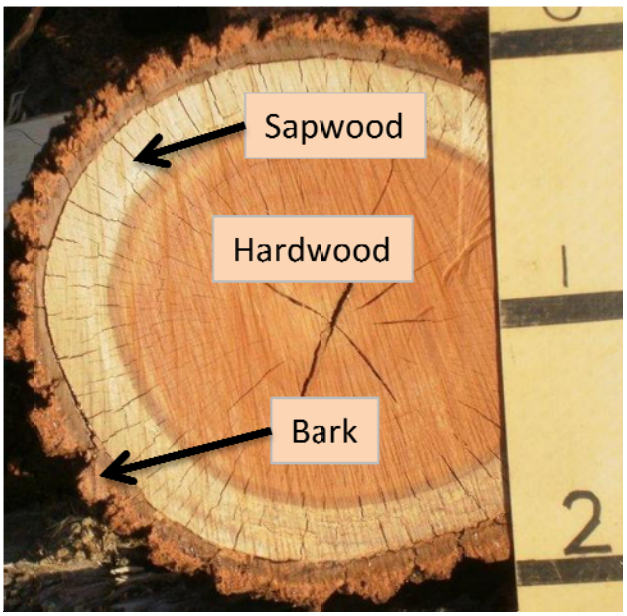

Figure 2-21: Cross-section of a typical sawn hardwood log; scale units 100 mm (Photo courtesy R. Patterson)

In relation to the second concern, that is the combined contribution of checks and pipes to the SMA, Yttrup, Law, & Subramaniam (1991) cite that:

> In addition to defining the external sectional dimensions of a beam section the role of internal defects must also be considered. 'Pipe rot' does not in itself have a great impact on sectional 'I' and 'Z' but does reduce the shear area at mid depth of the section. The combined effects of checks, a natural feature of timber, with pipe rot can be to destroy shear transfer across the section…

> When timber shrinkage causes 'discretization' of the section by the formation of checks, shear transfer occurs by relatively few fibres across the root of these checks, and the section behaves as a single unit. However, checks often trap water leading to accelerated biodegradation at the root of these checks attacking the small but important 'fibre interlock' that exists across the checks. The section then changes from a single unit to a series of cooperating pieces. The loss of strength and stiffness of old bridge beams is due to these effects, that is, degradation at discreet locations such as checks, and not a general degradation of the whole of the section, which is the far more advanced state of deterioration.
>
> The natural tendency for a log to check and have a pipe can have significant structural consequences. Unfortunately, this is very difficult to see during an inspection, not visible by boring examination etc., invisible to ultrasound and x-ray examination. Behaviour of the beam under load is probably the most effective method to judge how the beam is working, as a competent whole or bundle of partial sections…
>
> A well planned load testing of a bridge should recognize 'lazy' beams, see Figure 9 [reproduced here in modified form as Figure 2-22]. No amount of non destructive ultrasonic or x-ray methods will detect defects of this nature.
>
> (Yttrup, Law, & Subramaniam, 1991, p. 710)

An example of the combined contribution of checks and pipes is shown in Figure 2-22. In this figure is shown how a central pipe can combine with two checks to cause the girder to separate into virtually two independent elements. The effect of this separation is significant as indicated by Moore (2009, A-103). In the case of a simplified equivalent rectangular beam, separation into two elements of equal cross-section reduces the MoE by a factor of the square of the number of elements, which is 2^2 or four. In a load test the load would have to be reduced by four so as to not exceed the deflection at which rupture occurs. It would thus appear that the girder MoR would be reduced by four when compared with an apparently similar looking, but intact, girder.

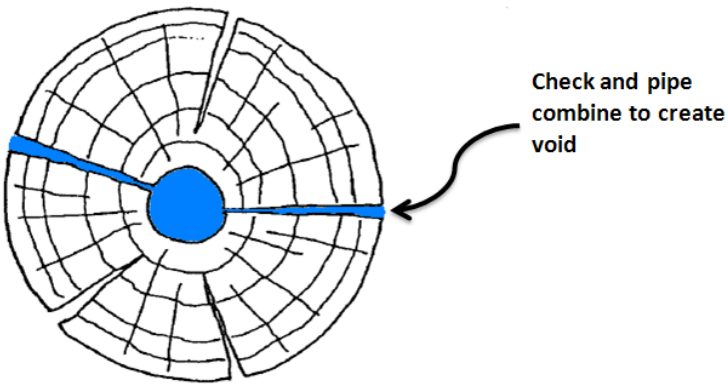

Figure 2-22: A simplification of the combination of a check and a pipe, adapted from Yttrup, Law, & Subramaniam (1991, Figure 9)

In Figure 2-23 an example network of two fine checks connecting an incipient pipe is highlighted. Because of the fine nature of this network it is not possible to visually assess their structural significance. There may well be significant connectivity across the checks, as shown in Figure 2-23, and if similar checks were present in the mid-span of a girder then they would be continually stressed and the connectivity would temporally vary.

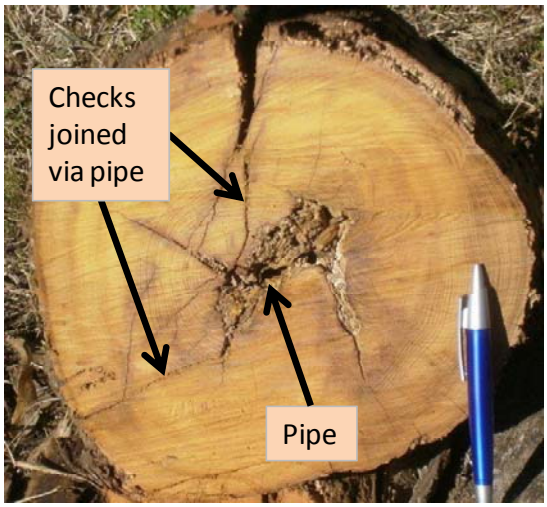

Figure 2-23: Cross-section of timber with checks joined by an internal pipe (Photo courtesy R. Patterson)

Taylor (1985), in Yttrup, Law, & Subramaniam (1991), reports that the timber between the defects is usually similar to the 'as new' condition. They also report that "the changing strength and stiffness of a bridge beam is due more to section property change than material property changes"(Yttrup, Law, & Subramaniam, 1991). Yttrup, Law, & Subramaniam (1991) also identify that the best method to detect such changes is as part of well-planned load tests. These

data represented in Figure 2-20, adapted from Law and Morris (1992, Figure 5) show a 6% drop in the normalised mean values of the clear samples A calculated variation for the MoR hardwood data shown in Figure 2-20, is about 13% CV. It is, therefore, not clear if the cited 6% drop is statistically significant, because it is difficult to draw conclusions when the normalisation is not explained and samples are not species specific.

RTA N150/06

The additional 21 condemned and redundant girders (already cited in Section 2.5.3, *RTA N150*) were tested by the Queensland University of Technology (QUT), School of Engineering, Brisbane. The test procedures, equipment used, photographs of testing and results were presented in Report No CET 2830 (ref 9), but this report is no longer publicly available. Surviving records indicate that each complete girder was tested and its MoE estimated from the load-v-deflection plots. The SMA was based on the girder cross-section at mid-span but ignoring the effects of the marked surface checking, splits and other effects. The MoR was considered as the measure of the ultimate short-term load-carrying capacity based on the same SMA used to calculate the MoE. The results of these girder tests were also presented in the RTA N150/06 report (Law et al., 1992b), only portions of which are available.

Once the girder tests were complete small clear samples were excised from areas A and B at locations 1 to 4, as shown in Figure 2-24, and each independently tested to determine the MoE, MoR and moisture content of each sample. It is cited in the RTA N150/06 report (Law et al., 1992b) that this work was carried out by Queensland Forest Service but their report was not available. Although some available documents identify that moisture content was recorded, there was insufficient work carried out to determine its effects. These data from these tests have survived, but detailed discussion was not able to be located.

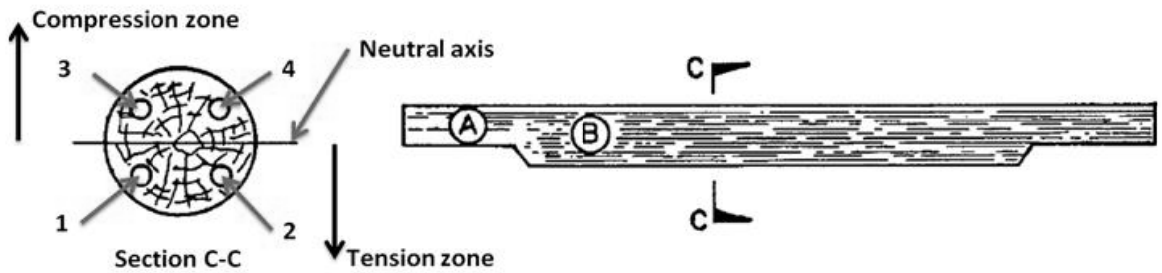

Figure 2-24: Diagrammatic simplification of girder and position of samples, adapted from RTA Report N150/06 (Law et al., 1992b, Table 3.1)

2.5.4 Queensland girder dataset

A more rigorous testing regime was undertaken by Wilkinson (2008), which involved testing to bending failure 53 round timber-bridge girders supplied by the QDMR. This girder set contained examples of each of the four condition states, CS-1 to CS-4, and of four species. The Condition State (QDMR, 2004) of each girder was determined by experienced QDMR personnel and samples were taken that were used to identify the timber species of each girder. The girders were loaded in four point bending with the supports spaced at 6.6 m and two equal loads (P_{jack}) applied 1.1 m either side of centre (Wilkinson, 2008, p. 69). The girder diameters were averaged from three measurements taken at three unspecified locations. Wilkinson (2008) calculated the MoE for each girder using Equation 2-3 and an estimate of P_{jack}/δ from the linear section of the load-v-deflection curve of each girder (refer Figure 2-34).

$$E = \frac{P_{jack}\, a}{24\delta I} \times (3L^2 - 4a^2)$$

..... **Equation 2-3**

where:

E	is the MoE (Pa)
P_{jack}	is the applied load at each of two jacks (N)
a	is the distance between support and jack
L	is the span between supports (m)
δ	is the mid-span deflection (m)
I	is the SMA (m^4)

Wilkinson's interpretation of these data was from the design perspective using the results to guide the design of a laminated girder that was to be used to retrofit an existing solid round timber girder. The results of the bending tests of the round girder set were presented in two diagrams. The first was an MoR-v-MoE scatter diagram, as shown in Figure 2-25 (Wilkinson, 2008, Figure 4.7). The second was an MoR-v-Condition State scatter diagram, as shown in Figure 2-26 (Wilkinson, 2008, Figure 4.9). The slope of the MoR-v-MoE regression line is reported as 3.8 but no significance is given to this value by Wilkinson. It will be shown in Chapter 3 that a slope of about this magnitude is typical for timber girders. The coefficient of determination for the sub-set in Figure 2-25 was quoted as 0.6951 and a fifth percentile curve identified (Wilkinson, 2008, Figure 4.7). Wilkinson did not discuss the MoR-v-MoE vector trajectory of an individual girder and only provided the observation that MoR generally declines with condition state as shown in Figure 2-26 (Wilkinson, 2008, Figure 4-9).

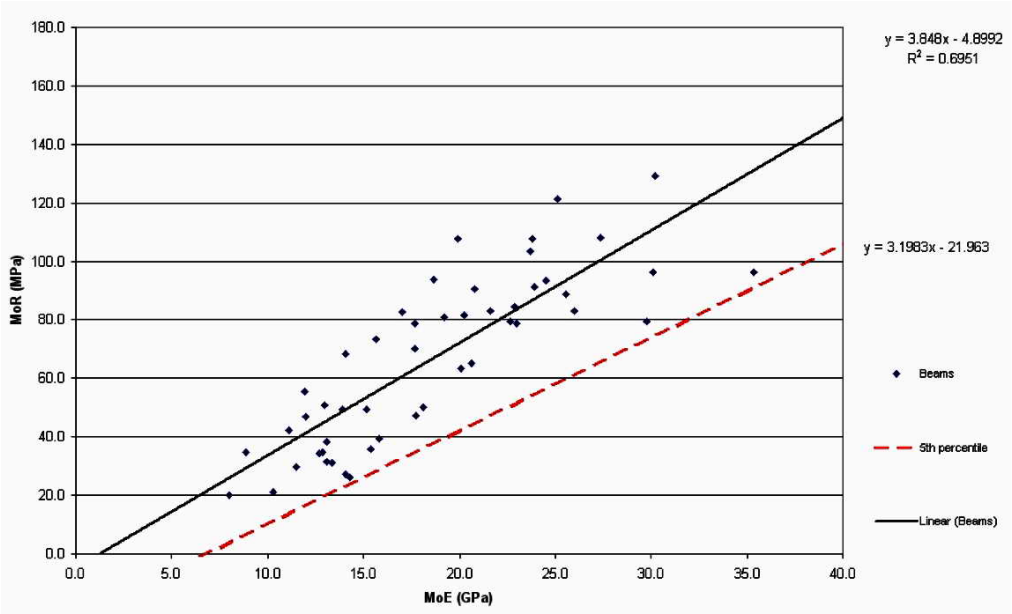

Figure 2-25: MoR-v-MoE scatter plot, Source Wilkinson (2008, Figure 4.7)

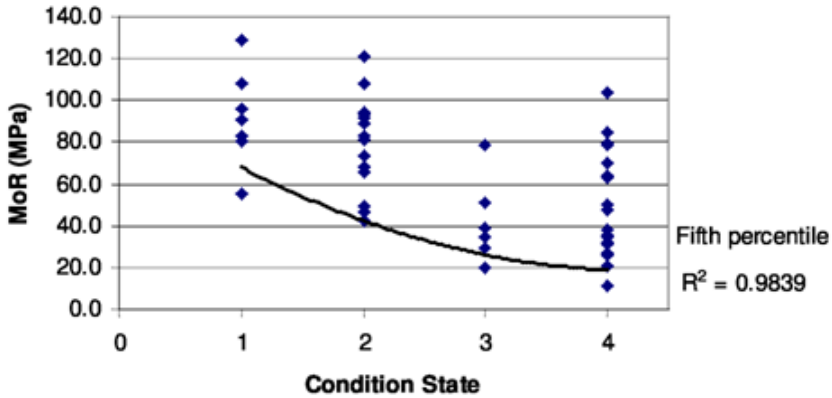

Figure 2-26: MoR-v-Condition status, Source Wilkinson (2008, Figure 4.9)

2.5.5 Australian reference data: Relationship between MoR and MoE

Australian Standard data

The minimum requirement for the replacement of round timber-bridge girders is specified by the RTA as F27 Hardwood, Group S2, Durability class 2 to AS 1720 with a minimum diameter of 450 mm (RTA, 2008, Section 1.9.1.2). This specification is made from the design perspective and does not enable the mean value of an in-service girder to be identified. All that is known is that a girder should have the strength of F27 hardwood and be of Group S2. However, there is ambiguity between these specifications. The MoR and MoE data for F27 are specified in AS 1720.1: 2010 as 67 MPa and 18.5 GPa respectively (refer Table 2-9). The F27 stress grade is also identified as being equivalent to the S2 strength group (refer Table 2-10) but the S2 bending specifications are not cited in AS 1720.1 (Standards Australia, 2010). These S2 specifications are cited in AS/NZS 2878 (Standards Australia, 2000) as 86 MPa and 14.2 GPa respectively (refer

Table 2-11). That is MoR is 28% higher and MoE is 23% lower. While this level of inconsistency may not be of concern from a design perspective it is a critical concern from a maintenance perspective.

Addressing the concern that the standard data did not reflect measured pole data Samali et al. commented that, "The relationship between strength and stiffness used in the current load assessment methods is based on an assumed relationship between MoR and MoE defined in AS 1720.1. However, it is not commonly understood that this relationship is both idealized and theoretical" (Samali et al., 2007). The thought that these data are theoretical is supported by the following statement in AS 2878:

> A strength group may be visualised as a nominal species with established clear-wood strength properties representing a collection of actual timber species that have similar or slightly higher mean strength values. The group limits have been chosen so that the ratio between representative strength values of groups is constant at approximately 1.2.
>
> (Standards Australia, 2000, AS/ANZ 2878)

These data quoted in AS/ANZ 2878 have, it appears, been chosen to represent a collection of species and are linearly related. It is not clear how these theoretical data reflect the measureable MoR and MoE of in-service timber girders. Samali et al. further identify that "the actual 5th percentile strengths for strength groups 1 and 2 timbers range between 30 and 55 MPa, not 80 to 100 MPa" (Samali et al., 2007). The range of 30 to 55 MPa is certainly below the figures of 67 MPa and 84 MPa for strength groups 1 and 2 cited in Australian Standards (2010, AS 1720.1). These data do not allow an engineer to identify the strength of an individual in-service girder, particularly when that strength has declined below the strength that it had when it was originally commissioned because of aging and deterioration.

Table 2-9: Characteristic design values for MoR (f'_b) and MoE (E) by stress grades (Standards Australia, 2010: AS 1720.1, Table H2.1)

Stress grade	Characteristic values, MPa						
	Bending	Tension parallel to grain		Shear in beam	Compression parallel to grain	Short duration average modulus of elasticity* parallel to the grain, MPa	Short duration average modulus of rigidity, MPa
		Hardwood	Softwood				
	(f'_b)	(f'_t)		(f'_s)	(f'_c)	(E)	(G)
F34	84	51	42	6.1	63	21 500	1 430
F27	67	42	34	5.1	51	18 500	1 230
F22	55	34	29	4.2	42	16 000	1 070
F17	42	25	22	3.6	34	14 000	930
F14	36	22	19	3.3	27	12 000	800
F11	31	18	15	2.8	22	10 500	700
F8	22	13	12	2.2	18	9 100	610
F7	18	11	8.9	1.9	13	7 900	530
F5	14	9	7.3	1.6	11	6 900	460
F4	12	7	5.8	1.3	8.6	6 100	410

NOTES:

1 The characteristic values for bending listed in this Table for F-grades apply for naturally round timbers and for rectangular beam sections not greater than 300 mm in depth. For rectangular beams greater than 300 mm in depth, the characteristic values are obtained by multiplying the value in this Table by $(300/d)^{0.167}$, where d is the depth of the section.

2 The characteristic values for tension listed in this Table for F-grades apply for tension members with width not greater than 150 mm. For tension members with a cross-sectional dimension greater than 150 mm, the characteristic values are obtained by multiplying the value in this Table by $(150/d)^{0.167}$, where d is the width or largest dimension of the cross-section.

3 The modulus of elasticity (E) is an average value and includes an allowance of about 5 percent for shear deformation. For estimating a fifth percentile value, expressions are given in Appendix B.

Table 2-10: Relationship between round timber strength groups and Stress grades (Standards Australia, 2010, AS 1720.1)

Strength group	Stress grade
S1	F34
S2	F27
S3	F22
S4	F17
S5	F14
S6	F11
S7	F8

NOTE: The equivalence expressed is based on the assumption that all poles or logs are cut from mature trees. Factors for immaturity are given in Clause 6.4.1.

Table 2-11: Unseasoned timber classification values (Standards Australia, 2000, AS/NZS 2878, Table 2.1)

Minimum species mean	Strength group						
	S1	S2	S3	S4	S5	S6	S7
Bending strength, MPa	103	86	73	62	52	43	36
Modulus of elasticity, MPa	16 300	14 200	12 400	10 700	9 100	7 900	6 900

* As measured or estimated at a moisture content above fibre-saturation point.

Commonwealth Scientific and Industrial Research Organization (CSIRO) timber species classification data

The Division of Forest Products, CSIRO, over a period of about 30 years conducted tests on Australian Timbers (Bolza & Kloot, 1963). These tests were conducted to provide a tabulation of the mechanical properties of 174 species of timber grown in Australia. The particular MoE and MoR data that are of interest here were obtained from static bending tests that were made according to British and American standards. The number of samples for each evaluation varied and ranged from 1 to 74 with a median of 5. An example of the tabulated data provided by CSIRO is shown in Table 2-12. Data from this tabulation are quoted, in SI units, in Table 2-4 and Table 2-5 in Section 2.3.

Table 2-12: Tabulated data of MoR and MoE (Bolza & Kloot, 1963, p. 54)

Species	Origin of Timber Tested	Moisture Condition	No. of Trees Tested	Density		Static Bending (Centre-point)		
				Basic (lb/cu. ft.)	Air-dry (lb/cu. ft.)	Stress at L.P. (lb/sq. in.)	Modulus of Rupture (lb/sq. in.)	Modulus of Elasticity (10^3 lb/sq. in.)
IRONBARK, GREY *Eucalyptus paniculata*	N.S.W.	Green	5	5 56·3 1 2¼		5 9860 4 12	5 17600 1 4¼	5 2750 2¾ 6
		12%	5		5 70·2 1 2¼	4 17000 – 10½	4 26800 – 4	4 3320 – 5
IRONBARK, GREY, QUEENSLAND *Eucalyptus drepanophylla*	Q.	Green	15	15 56·4 ¾ 3		15 10500 2½ 10½	15 17300 1½ 6¼	15 2840 3 11¾
		12%	21		21 68·9 ¾ 3¼	13 15400 1¾ 7½	13 26500 2½ 9¼	13 3460 3 10½

Although these data are provided to indicate the strength of timber derived from specific species it is not clear how they relate to the strength and stiffness of in-service timber-bridge girders. As an example, the mean value of MoR for Ironbark (*E. drepanophylla*) is, from Table 2-12, 183 MPa. The six sigma range is from 170 MPa to 197 MPa, and the variation of the distribution

2.5% CV. The upper limit of 55 MPa quoted by Samali et al. is only 32% of the Ironbark minimum. The fifth percentile value of 67 MPa that is cited in AS 1720.1 is 40% of the Ironbark minimum. While these data can be usefully utilized in construction from a design perspective, they are inadequate to provide a guide to the MoR of an in-service girder.

2.5.6 North American timber power poles

Bodig (1985) cites the variation of the strength of timber power poles and compares this variation with other construction materials. These materials and their variations are summarised in Table 2-13. The level of variation is typically about 10% to 20%; construction materials will measurably vary from sample to sample. Of particular interest is that poles prior to installation might vary up to about 20% and after being in-service the variation could be 33%. The mean MoR for the in-service poles was about 31 MPa with a standard deviation of about 10 MPa. Thus one pole MoR could be 20 MPa and another 40 MPa; the loading capacity of the former being half that of the latter. It should be noted that these data are for softwood poles and not directly comparable to these data previously quoted by Samali et al. (2007). Douglas Fir is a common name for about six species of *Pseudotsuga* evergreen timber trees (Britannica, 1975). Since the specific species are not cited by Bodig (1985) the variation cited in Table 2-13 cannot be directly compared with species data. It is, therefore, not possible to compare the in-service data with data from clears of the species.

Table 2-13: Material strength variation (Bodig, 1985)

Material type	Variation
Mechanically [machine] rated lumber	11%
Reinforced concrete	8 – 13%
Wood poles [prior to being placed in-service]	13 – 21%
Masonry walls	19 – 21%
Glued laminated timber	7 – 25%
Visually graded heavy timber	11 – 33%
Douglas fir in-service transmission poles	33%

2.5.7 Summary of research into timber variation

Because of the variation in timber elasticity and strength values, standardised fifth percentile grades have been created for use in structural design. Such grades are too conservative to be used to identify the mean values of in-service timber-bridge girders and the S2 grading specified by the RTA (2008) is the more appropriate minimum for new designs. Measurements of MoR related to strength groups of in-service timber are inconsistent with standardised data. In-service girders are degraded from their commissioned values and the Tasmanian, NSW, QDMR and North American datasets of timber removed from service provide an indication of this additional level of variation.

Visual grading techniques that identify the girder condition status have been used to grade aged girders. However, there appears to have been no instrumented attempt to identify either the temporal variation in strength or lifetime of NSW bridge girders. There is, therefore, a need to instrumentally quantify significant changes in the strength and elasticity of in-service girders. However, the Tasmanian, NSW and QDMR data as presented in this section contain more useful information in relation to parameters that indicate how MoR might vary and are analysed further in Chapter 3.

2.6 Structural Health Monitoring

2.6.1 Background

The discussion to this point has been aimed at understanding the use of round timber girders in extant timber beam bridges and the information published about them. Timber-bridges tend to be aged structures and both the organisations financially responsible for such bridges and those responsible for their maintenance are very concerned about how to maintain them in the future (Howard, 2009; Roorda, 2006; Troy, 2011). One path to coping with this maintenance problem has been to replace aged timber structures with new concrete ones. However, this replacement does not solve the maintenance issues; it only puts it off until the new concrete structures require significant maintenance, albeit delayed. Such replacement can also be a high cost solution. Whether the optimal path is to replace or repair, is currently obscured by the seemingly high maintenance cost of timber structures. It is also not clear what construction materials should be used for the replacement structure (Moore, Mahini, Glencross-Grant, & Patterson, 2011). In Europe and North America, engineers are designing and building new timber-bridges, large sports halls and arenas, swimming complexes and motorway maintenance centres with spans in the range 50 to 100 metres; timber structures that are very large and exposed to the vagaries of the weather and atmospheres that are likely to advance degradation. In regional Australia, engineers are much more tentative as exemplified by the composite material bridge shown in Figure 2-12.

One step on the path to improving confidence in the use of materials, new or old, is the implementation of Structural Health Monitoring (SHM), the purpose and axioms of which are defined by Worden et al. (2007). The application of probability based techniques is discussed by Cremona (2011).). Such techniques can be used to compare a base-line measurement with a later measurement, to determine if a significant structural change has occurred. Earlier work in relation to system reliability was related to defining component lifetimes and creating reliable

systems (Bazovsky, 1961; Kumamoto & Henley, 1996) and improving quality (Austenfeld, 2001). These aims were achieved in, for example, the motor vehicle and aerospace industries by improved manufacturing tolerances and quality control and in the space industry by creating equipment that contained redundant components that were only switched into operation when another component failed or reached the end of its life. Such techniques were especially required early in the life of the electronics industry because some parts had lifetimes that were less than that required for the equipment in which they were installed. As discussed by Worden et al. (2007) SHM is about taking the next step for major structures, including bridges and important buildings.

Following their experience monitoring the Tsing Ma bridge, Chan, Wong, Li, & Ni (2011) identified the need to expand the definition of SHM to include two components: Structural Performance Monitoring (SPM) and Structural Safety Evaluation (SSE). The reason for this is that bridges are currently designed to serviceability limit states (SLS) and when these states are reached there may be no damage present. One examples of this may be caused by a combination of excessive wind and traffic loading. In the case of excessive wind loading the structure may still be structurally sound, because of its design, but may not be regarded as safe to use. In such a situation it is of interest that the SSE system provides an indication to the bridge maintenance staff that a SLS has been reached and also when the bridge is again safe to use. In the case of slow but progressive deterioration an SSE system can be used to identify progressive structural change which will allow the prediction, within limits, of an impending SLS failure. One example of how this might be achieved for a timber bridge girder is proposed in Section 5.6.3.

Particularly in the maintenance of the timber beam bridges used in NSW one of the next steps is the determination of girder lifetimes. It is not yet possible to do this adequately in the design process. The dataset shown in Table 2-5 illustrates some of the variation that can occur in new girders and the dataset in Figure 2-17 what is found in aged girders removed from service. But it is not obvious how to relate the aged girder data to the new girder data to determine age or lifetime. It is also not currently possible to predict the likelihood of an aged girder failing a service limit state test. Nor is it possible to predict the strength of aged in-service girders from standard data. The approaches of both Deming (Austenfeld, 2001) and SHM (Chan et al., 2011; Worden et al., 2007) need to be applied. Deming advised that the process should be allowed to run and to be monitored for variation before an action is taken. The long lifetime of timber girders means that “allowed to run” can be many decades and involve data from a significant number of lifetimes. The principles of SHM are similar to Deming’s: abnormal variations in a

process need to be detected; an intelligent review made of those variations; followed by action to improve the process if warranted. The Deming mantra was simplified as a cycle, so: Plan, Do, Act, Change. In summary this is a process of continuous quality improvement.

The point of SHM is to ensure that structures perform over a long period in a manner similar to that specified in the design (Chan & Thambiratnam, 2011, p. vii). Some examples of SHM in older bridges are reported by Mahini, Glencross-Grant, & Moore (2011). Another bridge, the Tsing Ma bridge (cited above) is in an area that can suffer severe typhoons and it is only practical to assess this structure in a SHM manner; to achieve this 282 sensors are deployed throughout the structure (Chan, Wong, Li, & Ni, 2011). In Brooklyn, New York, a SHM system was used to identify base-line performance criteria prior to scheduled maintenance (Talebinejad, Fischer, Ansari, & Bojidar, 2010). Fibre optic, crack, tilt, thermal and acceleration sensors were installed to identify the severity of masonry cracks such that economical and efficient repair systems could be created. In Taiwan a bridge with three 30 m prestressed concrete bridge spans was exhibiting spalling of the concrete cover and visible corrosion of the steel tendons (Chen, Wang, Chen, & Wang, 2008). The bridge was measured under load using fibre optic bending and strain gauges.

In order to improve the quality of timber-bridges, and their level of service, it is important to remove degraded girders before their end of life and to be warned of end of life by detecting abnormal variations. A single overload is an abnormal variation, but may not cause any reason to take immediate action. Subsequent monitoring of the structure can lead either to the opinion that no damage has occurred, or that damage has occurred. The assessment must then be made as to whether the damage is minor or significant. Such an assessment can be made by calculating the probability of failure and the related safety index, β (Elishakoff, 2004; Melchers, 1999; Wolfram, 2004).

2.6.2 Probability of failure (p_f)

The probability of failure of a structure is determined by the difference between the two distributions: the demand; and the capability. This fundamental theory is covered by Elishakoff (2004), Wolfram (2004) and Melchers (1999) who each use different nomenclature. A brief review of this theory, as it applies to determining significant abnormal variation in bridge girder performance, has been published as part of this current research (Moore, Glencross-Grant, Mahini, & Patterson, 2012). If the demand distribution is represented by *f(S)* and the capability

distribution is represented by *f(R)* (Melchers, 1999) then the probability of failure, p_f, is estimated from:

$\boldsymbol{p_f}$ = probability that *R* is less than or equal to *S*

$$\boldsymbol{p_f = P(R \leq S)}$$

..... Equation 2-3

where:

p_f Is the probability of failure
R Represents the capability distribution
S Represents the demand distribution

The probability of failure is an indication of the number of times a limit state is exceeded. As an example, to calculate the probability of failure of a random load causing the mid-span girder stress to exceed its MoR the calculation is repeated 10^7 times. The number of failures is then the number of times the calculated stress exceeds the MoR. If the number of failures is five then the probability of failure is 5 in 10^7 (0.00005%). The location of the Failure Region in which the demand (load) can exceed the capability (material strength) is shown diagrammatically in Figure 2-27.

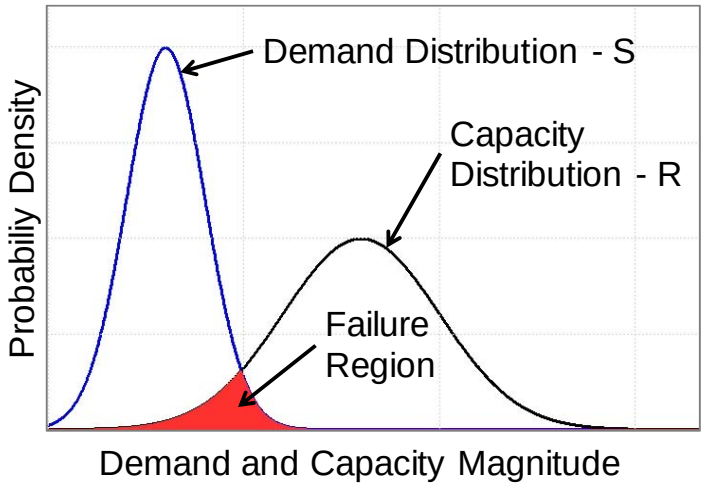

Figure 2-27: Schematic of typical demand and capacity distributions

2.6.3 Safety Index, β

The safety index, β, represents the number of standard deviations below the mean value of the limit state function and is related to the probability of failure of a system (Elishakoff, 2004; Wolfram, 2004). That probability can be translated to a safety index, β, when the probability density is a normal distribution. The safety index provides an indication of the position of the measured rate on the normal distribution curve and indicates the number of standard deviations below the mean. If the probability of failure is 5 in 10^7 then the safety index is -4.89.

Some sample values are:

if $p_f = 0.001348$ (1348 failures in 10^6 operations) then $\beta = -3.00$

if $p_f = 0.000000286$ (286 failures in 10^9 operations) then $\beta = -5.00$

Significant research has been conducted by (Elishakoff, 2004; Kumamoto & Henley, 1996)to determine socially acceptable levels of failure, and two commonly accepted levels are:

1 in 10^3 ($\beta = -3.1$) for situations not involving risk of life; and

1 in 10^6 ($\beta = -4.75$) for situations involving life threatening risks.

These levels of failure can be used as limit values when determining whether any detected abnormal variation in timber-bridge girder deflection is of concern or not. In the experiments and determinations that follow three bridge spans are measured, under static and dynamic traffic loading and distributions of deflection and stress created. These distributions are then compared to identify a probability of failure and limit conditions for each span. As an example determination the applied load can be modelled as a distribution using the formula:

$$\boldsymbol{P(n) = \mu_P + \sigma_P \sqrt{-2 \times \ln(u_1)} \times \cos(2\pi u_2)}$$

..... Equation 2-4

Where the value $P(n)$ is the applied load evaluated at each value of n which typically has a range of:

$$\boldsymbol{0 < n < 10^7}$$

..... Equation 2-5

and where: the value μ_P and σ_P are the mean and standard deviations, respectively, of the distribution $P(n)$; the values u_1 and u_2 are random numbers and the mid-span deflection of a simply supported girder, δ, is given by:

$$\boldsymbol{\delta = \frac{P(n)L^3}{48EI}}$$

..... Equation 2-6

where:

δ is the mid-span deflection (m)
P is the applied mid-span load (N)
n is the calculation repetition number
L is the span between supports (m)
E is the MoE (Pa)
I is the SMA (m^4)

One example use of this determination is to calculate the mid-span deflection. This deflection can then be compared with the span to deflection limit state and the number of times it is exceeded, recorded. The safety index, β, can then be determined.

2.6.4 Monte Carlo analysis

A Monte Carlo analysis is one that involves creating a single structural loading condition and randomly repeating the condition a large number of times with a distribution of input values. One simple example is the calculation of the mid-span stress of a girder. Firstly the girder load is defined as a distribution of values and the girder MoR is also defined as a distribution of values. Then a random load is selected from the load distribution and a random MoR is selected from the MoR distribution. The mid-span stress caused by the load is calculated and compared with the MoR. This process is then repeated many thousands of times and the number of times that the calculated stress exceeds the MoR is determined. The number of failures divided by the number repetitions is the failure rate which is then converted to a safety index. Program scripts are written in high level MatLab® and R computer languages to carry out these calculations and present the results.

2.6.5 Application: Evaluation of timber power poles

Bodig (1985) presents a reliability assessment procedure to evaluate in-service poles or a shipment of new poles (refer Figure 2-28). This procedure involves firstly creating an MoR strength distribution for a batch of poles. A typical strength distribution will have a variation of about 20% for new poles and about 30% for in-service poles. The strength distribution is then compared with a design load distribution to determine what in-service loads can be applied.

This procedure is well suited to support a design process, but not a maintenance requirement. The design loads can be varied according to the actual distribution of pole strengths. Also, if it is identified, after in-service measurements that the distribution has varied then the design loads can also be changed. However, this procedure does not lead to the identification of the strength of an individual pole. The process relies on the measurement of the mean pole strength. If the strength of an individual pole is inaccurately measured then the statistics of the pole batch will

minimise the effect of that error. Effectively the procedure allows the use of a less accurate measurement process than is required to identify the strength of an individual pole. The critical difference in the maintenance situation is that the strength of an individual pole, or girder, needs to be considered. Each girder needs to support the load applied to it and the maintenance procedure must identify any lack of structural capacity in individual girders. There is, therefore, a need to adequately quantify girder material parameters and the need for appropriate measurement systems.

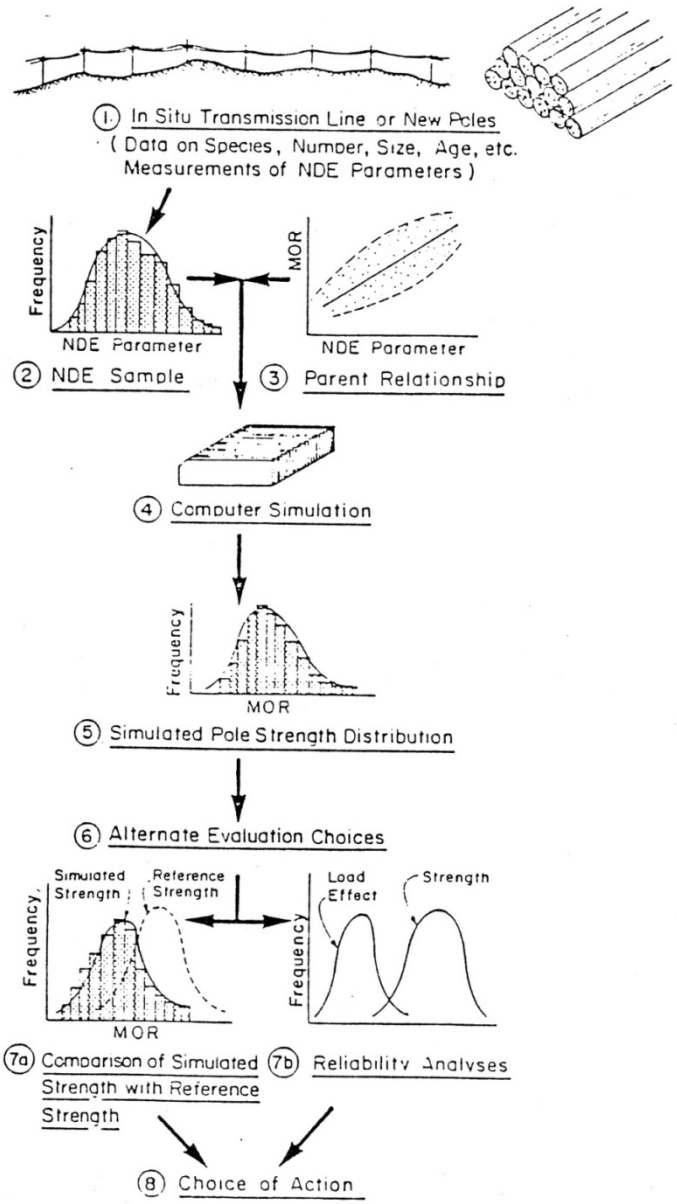

Figure 2-28: Reliability assessment procedure to evaluate timber poles (Bodig, 1985)

2.6.6 Measurement techniques for timber-bridges

Strain gauges have been commonly used for the measurement of strain in laboratory situations for over 70 years (Heywood, Karagania, & Baade, 2003; Lake, 2001; Moore, 2009). As a bridge girder bends under vertical load the underside of the girder is stretched. A strain gauge is a small electrical resistor that must be adequately attached to the girder and must also be stretched by the deflection of the girder. The technician applying the strain gauge must confirm accuracy by ensuring that the amount of stretching that occurs in the gauge is consistent with the amount of stretching in the girder. It is possible to achieve consistency for laboratory situations and for short term measurements of in-service girders, as shown in Figure 2-29. But to maintain unattended operation for many years on aged girders can be difficult to achieve and has not been demonstrated by the available literature. A strain gauge installed on an in-service girder will require regular recalibration. As a timber girder ages and checks occur the gauge attachment quality will decline. This decline will be further exacerbated by continual movement of the girder under load. Therefore, the gauge attachment will require regular maintenance followed by a load test to re-calibrate the gauge after any change. Thus, the practicalities of using traditional strain gauges for continuous measurements preclude more generalised use.

Figure 2-29: Strain gauge and LVDT in experimental use on a timber-bridge girder (Heywood et al., 2003)

More recently, Bragg grating interferometers have been installed into fibre optic cables and used in situations where strain gauges might have historically been used (Talebinejad et al., 2010). One example sensor for measuring the movement of cracks in masonry is shown in Figure 2-30 and some advantages and disadvantages of fibre optic Bragg sensors are shown in Table 2-10.

Figure 2-30: Crack sensor on masonry vault arch of Brooklyn Bridge, New York, adapted from Talebinejad et al. (2010)

Table 2-14: Some example Fibre optic sensor advantages and disadvantages (*pers. obs.*)

Advantage	Disadvantage
Many sensors in one fibre cable	Small sensor must monitor tensile fibre: attachment difficulty
Monitoring equipment remote from sensor location	High cost if only one sensor
Sensor not sensitive to nearby lightning surge	Temperature sensitive and may require correction
Can be designed to be used in harsh environments	Interrogation speed about 100 Hz

For the purpose of monitoring the mid-span deflection of timber-bridge girders two disadvantages of the fibre optic sensor are of concern. The first is the quality of attachment and the second interrogation speed. The difficulty of attachment is a similar concern to that for strain gauges. Quality data will only be obtained while the sensor is firmly attached. A good quality attachment can be achieved in clean dry masonary as shown in Figure 2-30 but long term attachment in aged timber girders can be problematical. Sensor interrogation speed (Doyle, 2012) is not a disadvantage when monitoring slow moving cracks in buildings, but may be too slow to accurately record peak girder deflection caused by traffic moving at highway speeds. Therefore, alternative forms and methods of routine measurement are needed for use on timber beam bridges.

Lake (2001) discussed how "... the health of a bridge could be inferred by measuring the response of the structure to the passage of heavy vehicles" - together with how the response might be measured with strain gauges. The difficulty with such an approach for small rural bridges is that there may be very few heavy vehicles crossing a particular bridge. Also, it will not be known whether an isolated heavy vehicle has produced non-linear behaviour in the bridge. There are thus two problems that need to be overcome. The first is the attachment quality of the sensing system and the second is the concern related to using heavy vehicles.

An alternative approach, which overcomes the attachment difficulty, is to measure the bulk vertical movement of the girder. Historically this has been achieved by the use of linear variable

differential transformers (LVDTs), as shown in Figure 2-29. Although the attachment method shown does not differ from that used for the strain gauge this need not be the case. The LVDT only has to follow the bulk movement of the girder and reliable attachments can be conceived. In contrast a strain gauge has to follow the fine movements of the longitudinal fibres of the girder as it bends. As the surface of a girder degrades the fibres carrying the most stress will not remain on the surface. The most highly stressed fibres might be more than 10 mm below the surface. An LVDT, however, has the added difficulty of requiring an anchor point above or below the mid-span of the girder.

This difficulty can be overcome by using an alternative system to measure the bulk vertical movement of the girder. One approach is to use a laser based system and another approach is to use a camera and photogrammetric methods. Several good/bad, or go/no go, laser based monitoring systems are reported. One example is the use of a simple laser beam source and a simple detector. A common application of this type of system is to identify the entry of a customer into a shop. Someone interrupting the beam triggers an alarm and identifies their presence to sales personnel. Savino (1988) described a system that involved plates mounted on a bridge structure that would pass a laser beam when the structure was in good condition and obstruct the laser beam if a bridge structure deflected excessively. This measurement system as proposed was intended to be used to investigate the movement of bridge piles and the sway of a structure. Tomiolo (1989) further adapted the use of laser beams to detect building movement. Several detectors are used and placed on buildings of interest. A laser beam is controlled by a rotating optical system which scans a number of detectors. If the detectors move significantly they will not detect the laser beam and an alarm raised. A third variation was claimed by Canty (1995) who hung solid target blocks from the centre of the main girders and positioned laser beams to run horizontally just below them. Excessive deflection of any girder breaks the beam and initiates an alarm.

There are several more complicated sensing options. One is to use a vibrometer (Rasgsdale, 2004) which is a Doppler based instrument. A laser beam is modulated and reflected off a moving object. Analysis of the reflections enables an object's movements to be determined. Another option is to use a Lidar type system. Lidar is commonly used for assessing agricultural systems to an accuracy of 100 mm and more by measuring time of flight (CSIRO, 2011). More recently it has become commonly used for vehicle speed assessment replacing microwave radar systems. The advantage, over microwave systems, is that because it is based on an optical laser it can be directed at a single object. However, one of the disadvantages cited is the need to average

these data over a period of 0.3 sec, which is too long for bridge deflection measurement (Wikipedia, 2012). These more complicated sensing systems all have the same disadvantage as do LVDTs in that they require to be mounted normal to the girder mid-span. This is difficult to achieve for many bridges because of access problems, confinement, etc.

The systems proposed by Savino (1988) and Canty (1995) do not suffer this disadvantage, but they do not provide a measurement proportional to deflection. A prototype system that overcomes this disadvantage and provides a signal proportional to deflection is shown in Figure 2-31 and Figure 2-32 (Moore, 2009). A fixed laser was mounted on, or near, the headstock and a detector rigidly mounted to the girder mid-span. The detector output was designed to be dependent on the relative vertical position between the girder mid-span and the fixed laser. A data logger was used to record the peak deflection caused by individual vehicles.

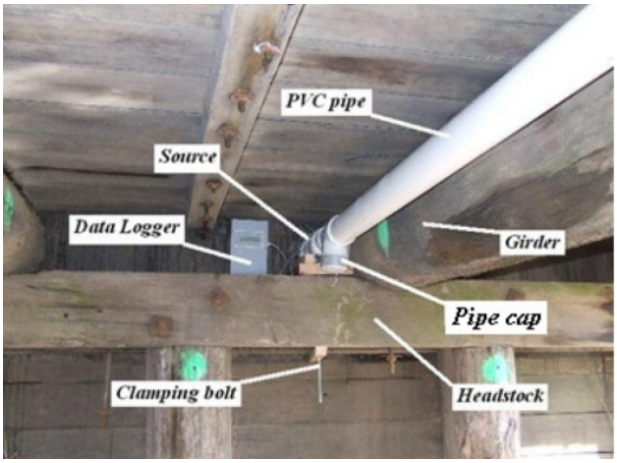

Figure 2-31: Typical installation of laser source and logger under a timber beam bridge (Moore, 2009)

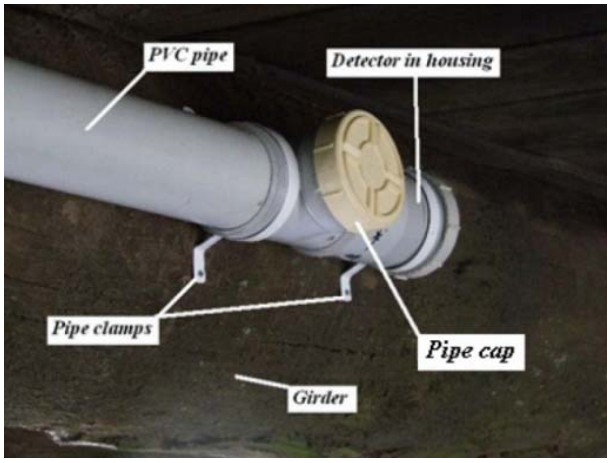

Figure 2-32: Detector mounted at girder mid-span (Moore, 2009)

To reduce the volume of data recorded the peak deflection data was quantised into eight levels as shown in Table 2-15. A vehicle was recorded as a Threshold 1 if the deflection it caused

exceeded the threshold level for Threshold 1 and similarly for all the other levels. Because of the narrow span of the bridge, a load that would cause a deflection greater than 55 mm was unlikely since this would equate to a span to deflection ratio of about 150 or less. Since many bridges of this type were designed to achieve a span to deflection ratio of about 300 or more (refer Section 2.3) a Level 8 was expected to be an upper limit of deflection, a value unlikely to be exceeded.

Table 2-15: Pre-defined positions of measurement thresholds

Threshold	Threshold position relative to reference line (mm)
Beam presence	-3.2
Reference line	0
1	1.9
2	4.5
3	7.0
4	9.5
5	19.7
6	30.0
7	40.0
8	55.3

Laser based measurement systems provide data indicating the relative movement between a specific source and target. The relationship between the source and target is typically constrained within a structure. Photogrammetric methods have more freedom. They still require a specific target to be measured, but there is freedom to place the recording camera remote from the measured structure. This freedom allows large and complex structures to be evaluated with one camera. However, the need for deflection measurement of large and complex structures is quite different to that required for small single lane timber-bridges. On a small timber-bridge typical traffic volumes are low and each vehicle passage is an independent event. The average time between vehicles might be several minutes. A typical small timber-bridge will commonly have only four main girders and need only two different deflection measurement scenarios. The first scenario, representative of the vehicle loading, is the measurement of the combined main girder deflection caused by each vehicle. This scenario allows the traffic load distribution to be determined. The second scenario is the measurement of the deflection of each girder that is caused by a vehicle of known mass. This second scenario allows the calculation of the fraction of the vehicle load that is applied to each girder by all similar vehicles. There is no need for a large number of measurement stations but there is a need for the accurate measurement of sub-millimetre deflections occurring within periods of a few milliseconds.

Significant research has been applied to the case of measuring the deflection of beams in complicated situations using photogrammetric methods. Some examples involve the measurement of highway bridges and others involve the measurement of cracks and movement

in buildings (Jauregui, White, Woodward, & Leitch, 2003; Jiang & Jauregui, 2007; Maas & Hampel, 2006; Piot, 2010; Tsakiri, Ioannidis, Papanikos, & Kattis, 2004; Yoneyama, Kitagawa, Iwata, Tani, & Kikuta, 2007). In these cases photogrammetric methods are of benefit and can be shown to be of lower cost than other methods. They provide solutions that enable evaluation of movements over long periods of time. As an example, Jauregui et al. (2003) reports one survey that took 10 h of field time and three weeks of image processing. Suitable low cost high speed measurement systems for use with small timber-bridges have not yet been demonstrated. The use of a high speed camera, to enable the measurement of mid-span girder deflection caused by vehicles travelling at highway speeds, is part of this current research.

Mufti (2001) reports the use of SHM sensors being applied to bridges in Canada starting in 1993. However, the difficulty of measuring the mid-span deflection of timber girders for long periods remains. Mufti (2001) reports how an SHM approach can be applied to several small bridges with non-timber girders and the related instrumentation that can be employed. Modern fibre optic techniques are described together with older strain gauge approaches and the use of earlier strain counters. Also identified is the need to "avoid excessive data files" and it is suggested that data size can be limited by an appropriate choice of sampling rate. Several SHM monitoring systems are identified that utilise sensors to acquire data and communication systems to transmit data for processing remote from the bridge, refer Figure 2-33 (Mufti, 2001: Figure 2.1 and 6.11). This is an example of the type of SHM system used on concrete bridges in Canada. The option of intelligent sensing systems that process and filter data prior to transmission and that need only transmit warning signals, is not covered.

In NSW it is accepted that bridges are rated for dynamic loading according to Australian Standard AS5100.7. A structure is first proof load tested. The results of such testing are then used to identify factors such as the live load factor, δ_l, or a modification ratio r_m. Factors, such as the live load factor, are then used in an equation to derive a rated load. However, the use of dynamic load testing depends on many vehicle configuration factors and it is currently recommended by Standards Australia that the outcomes of such tests require the approval of a relevant authority (Standards Australia, 2004b: AS 5100.7). However, different researchers do not calculate the "dynamic increment" resulting from field testing in a uniform manner (Mufti, 2001). There is, therefore, a necessity to both measure and interpret the data from the testing of bridge structures in a uniform manner. One reason for this lack of uniformity in NSW is because each Local Government Shire determines what measurement to make and the method used. SHM is not currently used in NSW to monitor timber-bridges. By implementing continuous

SHM on a large number of bridges throughout NSW, regular bridge performance data will be obtained that can be compared in a consistent manner. It is one of the objectives of this research to identify measurement techniques that can be used to achieve this for the case of timber beam bridges.

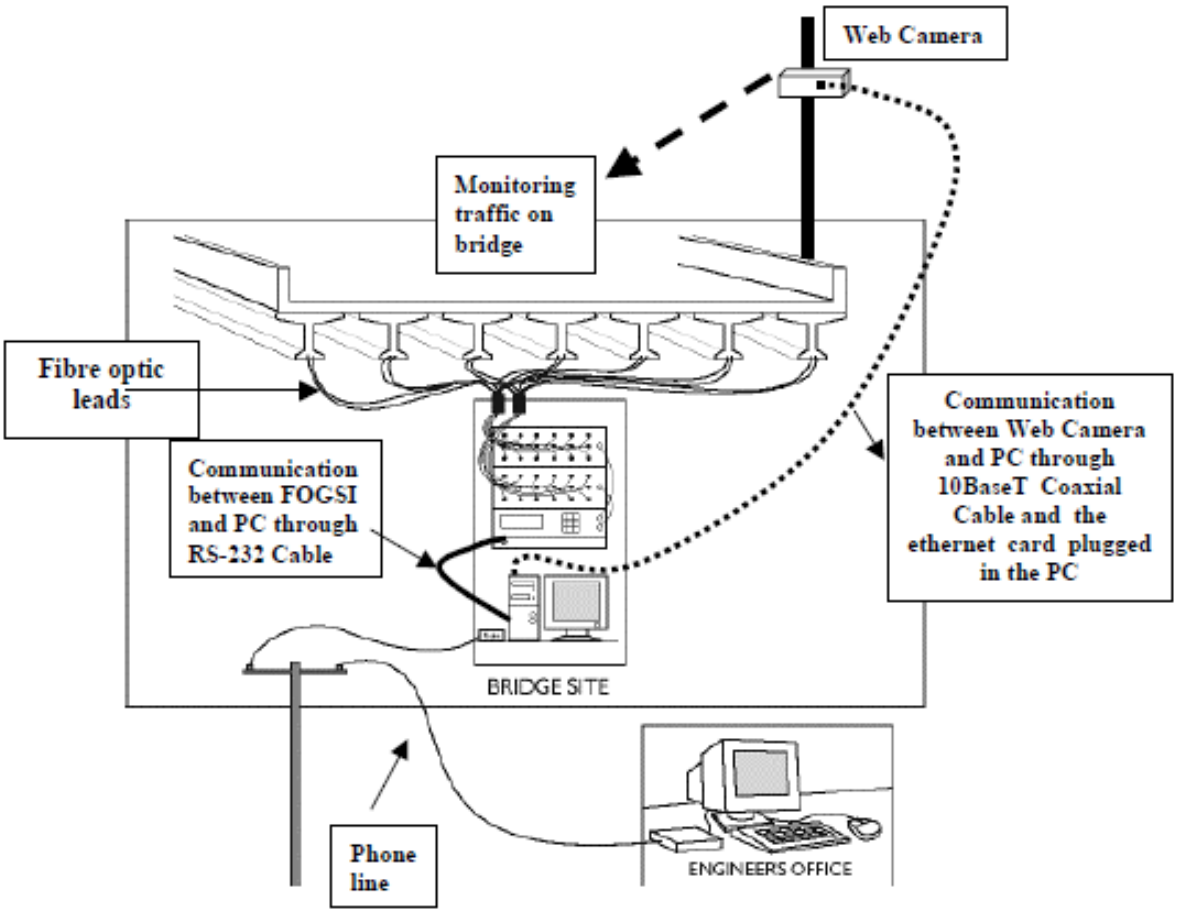

Figure 2-33: Layout of monitoring systems in Taylor Bridge, Provincial Road No. 334, Manitoba, Canada (Mufti, 2001: Section 6.6.1)

2.6.7 Summary

Timber girders have inherent flaws and defects and the assessment of degradation requires comparison between two system states. Sensors can be used to create signals representative of girder deflection and a traffic loading distribution determined. The variation in girder elasticity can also be sensed and a capability distribution determined. Comparison of these two distributions allows a probability of failure to be identified. Then by reviewing the temporal probability of failure any significant changes can be used to identify system degradation. Such reviews allow degraded girders to be removed before end of life and un-degraded girders to be retained. How this can be achieved, using a laser based bridge deflection meter (BDM), a high speed camera and Monte Carlo analysis techniques, is described in this thesis. Practical

measurement systems are beginning to be trialled but how the outputs of such systems can be related to probabilistic analysis techniques still needs to be demonstrated.

2.7 The measurement of MoE and MoR

In order to predict the probability of failure, as discussed in Section 2.5, temporal bridge girder performance needs to be measured. This requires the measurement of parameters that are indicative of that performance. Typical parameters that need to be considered are: MoE; MoR; SMA; moisture content; and density. These parameters are material constants of the particular girder. If any of them vary then it is probable that the timber has been modified in some way; the material has changed. However, it is important to separate random measurement error from changes to the distribution of values. Significant change can only be established if the mean value, or the standard deviation, of these data varies according to statistical criteria. In this section factors affecting measurement accuracy are discussed.

2.7.1 Determination of MoE from load and deflection measurements

The tolerance of measured girder material parameters has not been well recorded. As an example, girder MoE is typically determined, whether in the field or in the laboratory, by three or four point bending tests (Bolza & Kloot, 1963; Law et al., 1992a; Record, 1914; Wilkinson, 2008). However, in evidenced literature there is scant (if any) discussion of measurement accuracy. Published literature is seemingly by those interested in how to design new structures and not in the subsequent maintenance of individual girders. There was also negligible reference to the performance of individual girders; instead each girder was treated as part of a set in order to derive minimum performance data. It is not to be doubted that these girder bending tests were made to an appropriate level of accuracy, but just that the accuracy was not discussed. The level of accuracy that is achievable is discussed in Section 2.7.4.

Figure 2-34 and Figure 2-35 show two typical load deflection curves of girders removed from service. The first (Figure 2-34) from Wilkinson (Wilkinson, 2008) represents girder number four and the second (Figure 2-35) by Law and Morris (1992, Figure 3) represents a typical plot for an RTA bridge girder. What is apparent in both of these figures is that the bottom half of the curve is linear, but at higher loadings it becomes non-linear. This change was specifically identified by Wilkinson (Wilkinson, 2008), who calculated the girder MoE from the linear part of the curve (refer Figure 2-34) for loadings less than 140 kN. Neither Wilkinson (2008) nor Law & Morris (1992) attributed any significance to these curves except to identify that MoE was determined from the linear region.

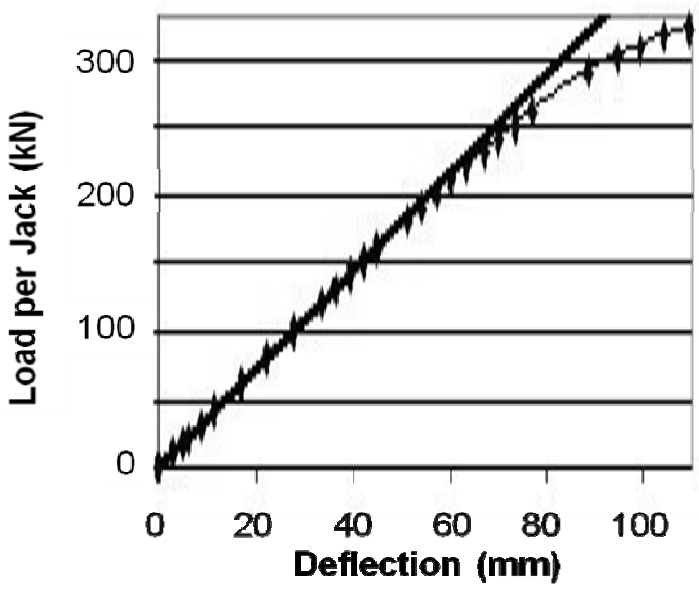

Figure 2-34: Load-v-deflection response for girder #4, adapted from Wilkinson (2008, Figure 4.6)

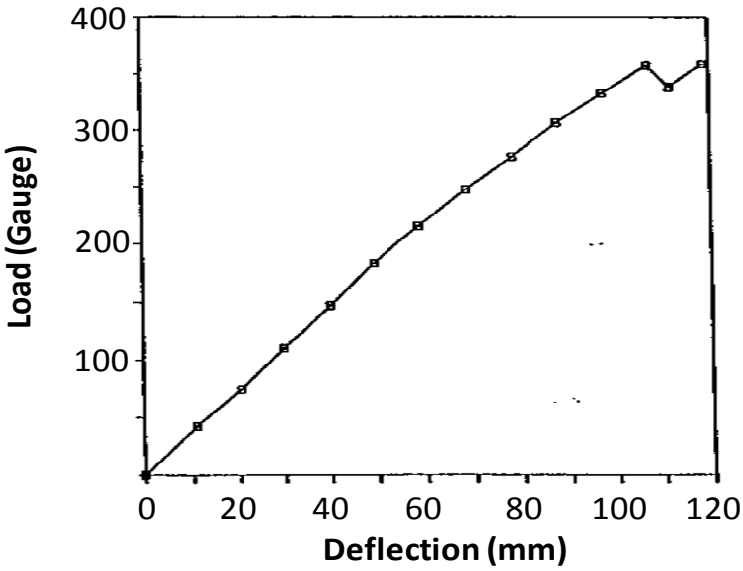

Figure 2-35: Load-v-deflection response for a DS-2 sample girder, adapted from Law et al. (1992a)

2.7.2 Parameters affecting the Determination MoE and MoR

The SMA of a girder is a major parameter that affects the mid-span girder deflection under load but it is difficult to exactly quantify for an in-service girder. As an example the use of Euler-Bernoulli theory allows the following equations to be derived for three point bending of a round girder of constant cross-section:

$$\boldsymbol{MoE = \frac{PL^3}{48\delta I}}$$

..... Equation 2-7

where:

MoE	is the modulus of elasticity (Pa)
P	is the applied mid-span load (N)
L	is the span between supports (m)
δ	is the mid-span deflection (m)
I	is the SMA (m^4)

and

$$\boldsymbol{MoR = \sigma_{MoR} = \frac{M_{max}}{\frac{I}{y}}}$$

..... Equation 2-8

where
:

σ_{MoR}	is the maximum mid-span stress (Pa)
M_{max}	is the maximum mid-span bending moment (N.m)
I	is the effective SMA at mid-span (m^4)
y	is the distance of the surface from the neutral axis at the mid-span

More generalised results can be derived for other girder shapes and loading conditions but the significance of the SMA, *I*, remains; both MoE and MoR are inversely proportional to SMA. For a girder of circular cross-section, *I* is given by:

$$\boldsymbol{I = \frac{\pi D^4}{64}}$$

..... Equation 2-9

where:

I	is the effective SMA at mid-span (m^4)
D	is the effective girder diameter at mid-span (m)

In the case of an elliptical shape a similar relationship applies:

$$\boldsymbol{I = \frac{\pi D_w D_d^3}{64}}$$

..... Equation 2-10

where:

I	is the effective SMA at mid-span (m^4)
D_w	is the effective girder width at mid-span (m)
D_d	is the effective girder depth at mid-span (m)

Although moisture can increase the outside diameter through swelling, it is more likely that degradation and atmospheric effects will reduce the effective diameter. SMA is thus nominally proportional to the fourth power of the diameter. A reduction of the outside diameter of a circular cross-section girder of about 2% will reduce the SMA by about 8% ($1-(1-2/100)^4 \times 100$). If the girder is solid, homogenous, of circular cross-section and simply supported then:

$$\boldsymbol{MoE = \frac{4PL^3}{3\delta\pi D^4}}$$

..... Equation 2-11

where:

MoE	is the modulus of elasticity (Pa)
P	is the applied mid-span load (N)
L	is the span between supports (m)
δ	is the mid-span deflection (m)
D	is the effective girder diameter at mid-span (m)

and

$$\boldsymbol{MoR = \sigma_{MoR} = \frac{32M_{max}}{\pi D^3}}$$

..... Equation 2-12

where:

σ_{MoR}	is the maximum mid-span stress (Pa)
M_{max}	is the maximum mid-span bending moment (N.m)
D	is the effective girder diameter at mid-span (m)

Thus, girder MoE is inversely proportional to the fourth power of the diameter (D^4) and MoR inversely proportional to the third power (D^3). However, determining the effective outside diameter or the section modulus is not a simple matter, as has already been discussed in Section 2.5.3. One attempt on how to improve this measurement was reported in relation to the NSW utility poles project discussed in Section 2.5.2 (Crews, 1998; Decosterd, 1998). In that project, because of the high variation in these data, a significant confounding variable was considered to be the section modulus. Many of the poles examined had significant checks, pipes, splits and other deformations and the sections of poles were examined and each pole effective section modulus was estimated from the cut sections. One example section is shown in Figure 2-36.

However, as discussed by Yttrup, Law, & Subramaniam (1991) (refer Section 2.5.3) whether a girder operates as one composite section or two (or more) sections is not easily identified except by load tests (Yttrup, Law, & Subramaniam, 1991, 771). Figure 2-36 shows the section of a possible example of this concern. It is delineated by a red line into two sections, but whether the

two sections are effectively independent will depend on the quantity and quality of interconnecting material in other nearby sections. It is not obvious just from Figure 2-36 why the section is only divided into two because there appear to be six checks that join the pipe to the surface and what cannot be seen directly in Figure 2-36 is the interconnecting material. It is inferred that the only method of determining the level of separation of these sections is by load testing a particular girder and measuring both the mid-span deflection and the degree of movement between sections.

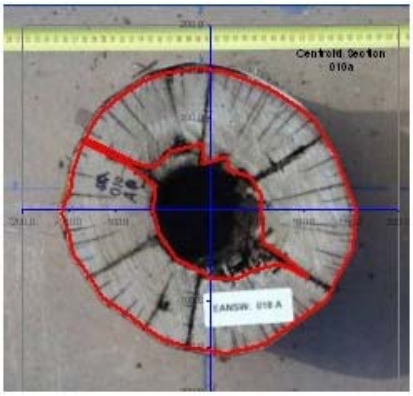

Figure 2-36: Cross-section of a power pole showing extensive piping and checking (Crews, 2005)

2.7.3 Effect of moisture content on MoE and MoR

It is well known that the moisture content (MC) of timber affects its mechanical properties (Bootle, 2004; Faherty & Williamson, 1995; Record, 1914). It is also accepted that the nominal effect of MC on strength and elasticity of small clear specimens is as shown in Table 2-16. Bootle (2004) has not cited any background to these data and Desch (1968, p. 168) cites the same data in the same tabular form. However, Desch (1968) also cites Garratt (Garratt, 1931, p. 133) as the source of these data and a copy of which was not found. Although these data appear to be commonly accepted, no literature was found, as part of this review, that related MoE, MoR and MC, neither in more detail nor in relation to hardwood species used for timber-bridges.

Table 2-16: Change in MoE and MoR produced by change in moisture content

Small clear properties	Percentage change per 1 per cent property change in moisture content
Modulus of rupture	4
Modulus of elasticity	2

(Bootle, 2004, p. 25)

How the MC might be distributed in an in-service girder is not well understood (Law et al., 1991). Law et al. report that the centres of some of the measured girders had MC as high as 42%; and that some girders with high internal MC levels had "relatively dry wood around the surface perimeter (MC less than 15%)" (Law et al., 1991). More recently Tannert et al. (2011) measured the MC of a laminated timber-bridge in Obermatt, Switzerland. They inferred from their measurements, shown in Figure 2-37, that the "… MC at all measuring points remained well below 25% with the MC near the surface being slightly lower than in the deeper regions" (Tannert et al., 2011). Measurements named 'OF' were taken at the surface and those named 'T' were at a depth of about 200 mm. The abscissa is time and the ordinate per cent relative humidity (%rH). This bridge was early in its structural life and would be graded as CS-1. Therefore, it can be inferred that in CS-1 timber the MC can be expected to remain both constant and at low levels.

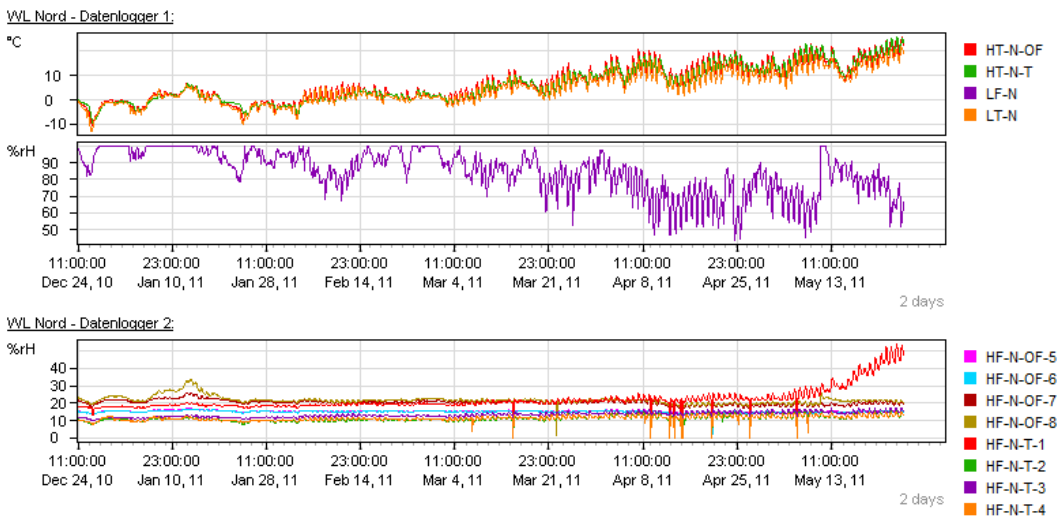

Figure 2-37: Typical MC of timber-bridge components (Tannert et al., 2011)

How the relative humidity will typically vary near a rural timber-bridge in NSW can be estimated from Figure 2-38 and Figure 2-39 by comparing relative humidity between locations. The long term variation in the morning is about 20%, with a mean of about 75%. In the afternoon the long term variation is again about 20%, with a mean of about 65%. The daily mean is then about 70%, which is also the daily mean value shown in Figure 2-40. The humidity might, therefore, vary daily from about 60% to 80% but the mean level is effectively constant (refer Figure 2-40). How this might affect an aged girder in comparison to a relatively new girder, as measured by Tannert et al. (2011), has not been published.

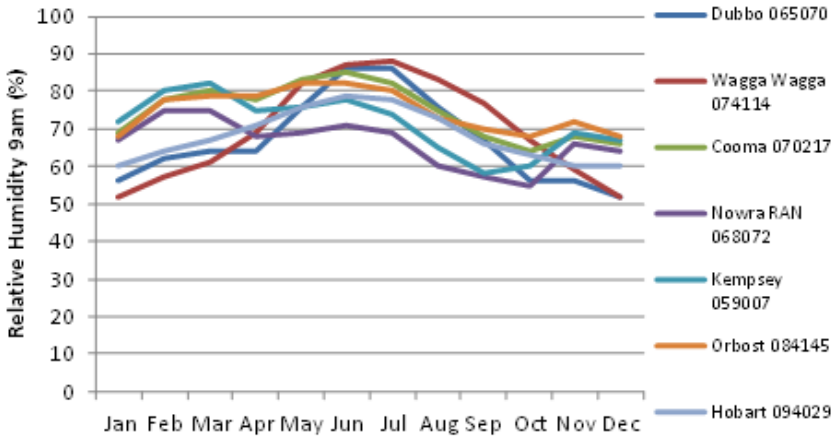

Figure 2-38: Morning relative humidity at selected Australian weather stations (courtesy R.Patterson, *pers. comm.*)

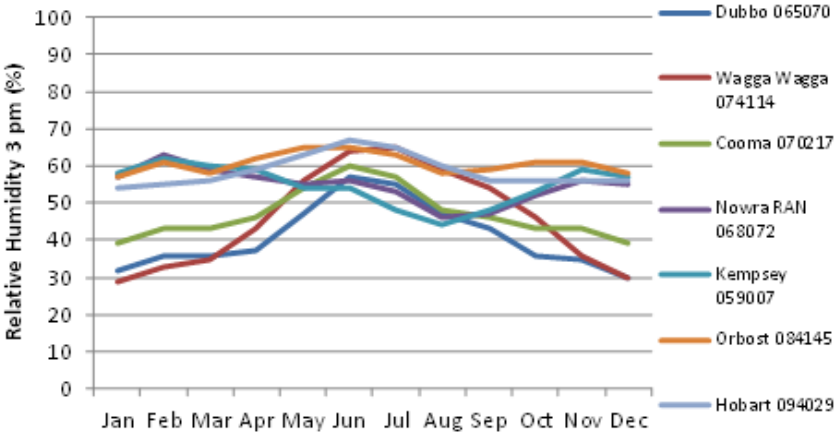

Figure 2-39: Afternoon relative humidity at selected Australian weather stations (courtesy R. Patterson, *pers. comm.*)

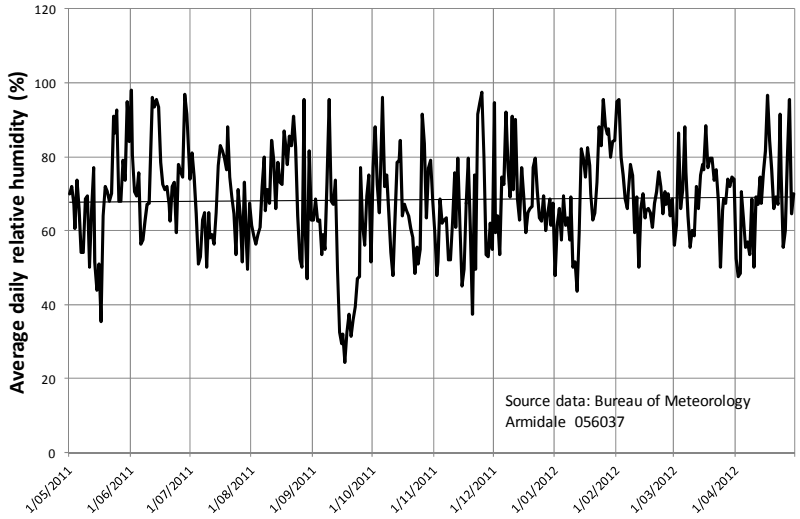

Figure 2-40: Daily humidity range for Armidale weather station (courtesy R Patterson, *pers. comm.*)

The percentage change identified in Table 2-16 is the percentage change in property value for each percent change in moisture content. As an example if the initial MC of a timber sample is 25% and it is dried to a MC of 12% then, for a change in MC of 13% and using the relationship as shown in Equation 2-13 and data defined in Table 2-16 the increase of MoE and MoR is given in Table 2-17:

$$\boldsymbol{Increase\ in\ MoE} = \left(\frac{(\%\ \boldsymbol{change\ in\ property} + \mathbf{100})}{\mathbf{100}}\right)^{\boldsymbol{change\ in\ moisture\ content}\ \%}$$

..... Equation 2-13

Table 2-17: Calculation of change in MoE and MoR for a 13% change in MC

Parameter	Percentage change	Calculation (Eqn 2-13)	Increase
MoE	2	$(1.02)^{13}$	1.29
MoR	4	$(1.04)^{13}$	1.67

2.7.4 Measurement accuracy

The ability to measure, in a field environment, to a level of accuracy of more than 95% requires the use of calibrated equipment for which any systematic errors (equipment and personnel) are well understood. However, it is not necessary to measure individual deflections to an accuracy of one per cent or less; it is only preferable that such a level of accuracy is achieved on the average. Thus, by application of the central limit theorem, adequate accuracy can be achieved by measuring deflections caused by 20 or 30 passes of vehicles of accurately known mass or a higher number of vehicles of mass known to a lower accuracy. By recording the deflections caused by thousands of vehicles over a period of a few months a suitable accuracy in the measurement of deflection can be achieved; especially when some of these vehicles are of known mass. No evidence was found in the literature researched for this thesis indicating how such long period measurements would be made on timber-bridge girders, nor how such measurements could be used to identify structural health.

Further, whether timber, a natural material, is repeatable to a level of few percent is not obvious from the literature. No records, or evidence of the accuracy of measuring timber girder deflections, were able to be located. There is also a paucity of literature indicating the level of repeatability that can be achieved. In reporting on the material properties of 174 species, Bolza and Kloot (1963) determined the standard deviations that were achieved for a number of samples. They do not report the repeatability of measuring one sample, nor do other standard textbooks (Bootle, 2004; Boughton & Crews, 1998; Bowyer, Shmulsky, & Haygreen, 2007; CSIRO, 1974; Desch, 1968; Record, 1914). The reason for this absence of discussion of repeatability is that, from the perspective of the structural designer, it is important to ensure that

any standard timber member will be suitable for use in a construction project. Usually there is little interest as to whether one particular beam will deflect 5 mm and another 8 mm; only that in general, grades of timber exceed a fifth percentile value (Boughton & Juniper, 2010). However, from a maintenance perspective it is important for an engineer to understand why an individual beam changes from deflecting 5 mm to one deflecting 8 mm, an increase of 60% without any change of loading. As indicated in Section 2.6.3, the material properties of timber vary according to the moisture content. The variation in MoE is quoted as being 2% for each 1% variation in moisture content (Bootle, 2004). It can be inferred, therefore, that to achieve such accuracy (refer Table 2-16) the variations in the measured MoE of the samples were less than 1%; although it is more likely that such accuracy was achieved by use of the central limit theorem. No recent validation of this early research (Garratt, 1931) was found, nor was any research found that identified the accuracy to which MoE could be measured.

2.7.5 Density

The measurement of MoE and MoR on in-service girders is discussed in this thesis and how an outcome of a probability of failure can be realised from such measurements. The quality of this outcome depends on the relationship between deflection; other measured values; and the estimated values of MoE and MoR. Unfortunately the relationships between these parameters appear to be not well understood. One empirical relationship, between girder material density and MoE, was cited by Law et al. (1992a) but the degree of scatter in these data was high and not explained. Desch (1966, p.149) cites that "... the density of wood is of practical interest because it is the best single criterion of strength" But he then reports that "Density is of limited value in determining the strength properties of individual pieces of wood" (Desch, 1968, p. 149). The reason he gives for density being of limited value is that defects are of greater influence. However, density can be useful for "indicating the lower limit for a species" (Desch, 1968, p. 149).

The density of a girder can vary according to the level of degradation (Curling et al., 2001), which can itself vary along the length of a girder. Some sections can degrade and have little effect on the structural strength of a girder; such as in the vicinity of the neutral axis in a low stressed region away from the mid-span and away from the supports. It is important to know the density in highly stressed regions of the girder and to know whether the measured densities are normal or abnormal. A normal density being typical of good quality timber and an abnormal density being typical of degraded, or decayed, timber.

One technique for estimating the presence of decayed wood and piping, that is the presence of decreased density, is to drill the girder and record the torque required to rotate the drill (QDMR, 2005, Appendix D; Wilkinson, 2008, Chapter 3). This technique has also been used to evaluate aged timber structures (Tsakanika-Theohari & Mouzakis, 2010; Tseng, 2010), but in both areas, aged beams in buildings and timber-bridge girders, the technique is only used as a method of comparative appraisal and not an absolute value. There has been no significant attempt to relate timber-bridge girder material density to the effective MoE or MoR of the girder. Such an investigation will necessarily involve examination of the microscopic cellular structure of the timber in regions of high and low stress.

Some authors have reported work where they have examined timber at the cellular level to determine its structural properties. However, as yet, there is no apparent work examining timber-bridge girders at this level (Ashby, 1983; Bergander & Salmén, 2002; Farruggia & Perré, 2000; Hepworth, Vincent, Stringer, & Jeronimidis, 2002; Kuo & Arganbright, 1978; Simon, Maigre, Bigorgne, Eyheramendy, & Jullien, 2010). Some of the difficulties of this work are that it requires levels of magnification of at least 100 X. To prepare hardwood samples for examination they are firstly softened by heating in water and then thinly sectioned using a specialised machine. Such procedures may change the configuration of the cells being examined (Kuo & Arganbright, 1978). Alternative approaches, using sufficiently high resolution computer-aided tomography (CAT scans), are not commonly available. Kuisch et al. recently reported how small compression failures could be identified in tropical hardwood using an environmental scanning electron microscope (ESEM) with magnifications of 100 X (Kuisch, Gard, Botter, & van de Kuilen, 2012). Such measurement equipment is currently only available for laboratory use and not readily available for the evaluation of in-service girders.

2.7.6 Summary

Quantification of girder mechanical parameters is not part of normal timber-bridge inspection practice. As a first example load-v-deflection curves are commonly created when testing girders to destruction in the laboratory and enable both MoR and MoE data to be determined. However, such curves are only used as part of proof testing of in-service bridges to identify the onset of non-linearity, rather than identify values of MoE.

As a second example, the measurement of SMA or section modulus has been considered for power poles, but not attempted to any degree of accuracy for in-service girders. A typical measurement by a bridge inspection crew will only provide a nominal assessment of external

width and depth of girders and visual appearance. This in itself can be subjective, depending on comparative experience of those inspecting.

It was demonstrated that girder moisture content is not of prime concern, since in many cases it is relatively stable, particularly in most inland areas in NSW. This is providing that the girders dry out through the day and do not remain in a high humidity environment. If they do remain in an environment of high humidity then MC will become a concern. Although both density and moisture content are parameters that can be used to identify deteriorated timber they are not commonly measured. While it would appear possible to measure parameters such as MoE, SMA, moisture content, and density, on in-service bridge girders, few timber-bridge owners, if any, are doing so. Nor are these owners or managers recording any temporal data. Therefore, bridges can change in subtle ways and the change will not be appreciated until a gross and in most instances, damaging change has occurred.

2.8 Conclusion

We are living and using, in the 21st century, timber-bridges designed in the 19th century and built through into the 20th. Ownership and management of these structures have changed. Local government authorities are now responsible for their maintenance yet do not have the resources to create new maintenance techniques. These techniques remain those of the early 20th century. Because of the age of many of these bridges, and the changes in ownership, detailed structural records are often not continuous or worse still, not available. The original designs catered for span to deflection ratios of about 380:1, but new designs are expected to achieve 600:1 with increased gross vehicle masses.

In NSW there is scant evidence of research being directed at regional timber beam bridge girders for the last two decades. There has been little attempt to upgrade them structurally to meet increasing vehicle masses. The more common approach has been to remove them and replace them with concrete structures. One notable exception is the combined work by the RTA and Gloucester Shire Council in constructing new timber composite bridges.

These data from the destructive testing of girders removed from service c. 1990 were used to identify the girder MoR fifth percentile. The reason for this was that this research was made from the perspective of the designer and not from a maintenance point of view. More recent researchers, c. 2005, have also followed this path by identifying the fifth percentile strength values of girders removed from service to enable them to design replacements. Others have devised instrumental methods of proof testing bridges using heavy vehicles. In the literature

there is no indication of a strong relationship between MoR and MoE being identified nor any direct way to identify the strength of in-service girders.

SHM is being applied in other industries and to large structures. The theory is well developed and it has been identified that by comparing distributions, instead of single valued functions, this enables determination of the probability of failure of a structure. Such techniques have not been applied commercially to timber beam bridges and there appears to be little attempt to measure the component lifetime of timber-bridge girders. While there are measurements made of girders removed from service, there is no clear indication of the measurement accuracy that is achievable. Thus, although there is a theoretical understanding of the possible variation of parameters such as MoE and MoR, there is no record of what the variation of MoE might be for a particular in-service girder, nor how its distribution might temporally vary. To be able to identify any clear relationship between parameters, such as MoR and MoE, they first have to be measured with a known accuracy. Variations in confounding variables such as effective diameter need to be minimised.

Because instrumented methods of measuring timber-bridge performance have been expensive and required technical staff they have only been used when absolutely necessary. The fall back has been the regular use of visual inspection. When methods using instrumentation have been used they have been part of an engineering evaluation commonly implemented after structural failure and the types of instrumentation used have mainly been limited to the use of strain-gauges and LVDTs. In NSW more modern techniques examining structural vibration with accelerometers have been applied. However, vibration techniques are more difficult to interpret for timber beam girder bridges, especially when the girders and decks are loosely related. These types of assessments are in contrast to the measurement of large concrete structures, where more modern fibre optic, laser scanning and photogrammetric techniques have been used.

One current disadvantage of these more modern techniques is that they involve the creation of large datasets often many Gigabytes in size. This is not of concern when used to scan buildings and slowly moving components or when recording the long term mean deflection of a bridge. A further disadvantage is that the recording of mid-span deflection using a photogrammetric or laser scanning technique requires that a measurement station has to be placed remote from the structure. This then necessitates expensive high quality camera optics to ensure that the deflections can be recorded to sub-millimetre accuracy at distances of many metres. It also requires that the field of view is not restricted. The use of such techniques for the recording of

mid-span deflections of timber bridges caused by highway speed traffic has not been commercially demonstrated.

Many timber-bridge owners in NSW do not know how to economically assess bridge strength. Current methods of girder strength analysis do not facilitate field measurements in real time nor correlation of girder behaviour over time. As part of this thesis new methods are identified that can be used to overcome these shortfalls.

3 THEORETICAL ANALYSIS: PREDICTION OF GIRDER STRENGTH FROM HISTORICAL DATASETS

3.1 Introduction

The literature review (Chapter 2) illustrated that prior research into the strength of girders was based upon design criteria derived from standards and guidelines. While checks, piping and moisture, impinge upon girder strength, any actual quantification with existing in-service girders is absent from that literature. Independent research by Yttrup and Nolan (1996), Wilkinson (2008) and RTA (Law et al. 1992a;1992b) provide girder data but based on the literature review performed in this research no combined analysis appears to have been published. The CSIRO quantified 174 timber species (Bolza & Kloot, 1963) and the RTA (Law et al. 1992b) excised and tested to failure samples from 21 in-service girders. However, no current comparison was found relating MoE_c and MoR_c of excised clears to MoE_g and MoR_g of in-service girders. In this research these six datasets are pooled together and analysed here for what is believed to be the first time. These data have been introduced in Chapter 2 and are identified, for the purposes of analysis, as:

1. DS-1: 95 Tasmanian Girders (Section 2.5.1)
2. DS-2: 169 NSW RTA N150-GRT girders (Section 2.5.3)
3. DS-3: 21 NSW RTA N150/06 girders (Section 2.5.3)
4. DS-4: 53 QDMR girders (Section 2.5.4)
5. DS-5: 21 NSW RTA N150/06 excised clear samples (Section 2.5.3)
6. DS-6: CSIRO characterisation of 174 Australian timbers (Section 2.5.5)

Historically these datasets could not be directly compared because there is no common representative statistical data. Firstly, to overcome this limitation, DS-1 to DS-6 are analysed as independent datasets but with a common statistical approach (Sections 3.2 to 3.7). Techniques such as statistical box plot summaries, quantile comparisons with a normal distribution and regression analysis are employed. MoR is related to prediction parameters such as MoE, age and condition state. These data are particularly compared in MoR-v-MoE scatter plots and in each case the same axes are used, as in Figure 3-1. The abscissa and ordinate are chosen to accommodate the range of values that occur in the data and the S2 MoE and MoR limit states, of 14 GPa and 86 MPa respectively, are plotted. In some instances the data are clustered into a subset that represents data with common characteristics. One example of a cluster is the data representing all of the tested Queensland Ironbark girders for which there are MoE, MoR and

bending mode failure data (Refer Section 3.5, DS-4-4). Prediction bands, as introduced in Section 2.2.8, are also plotted (refer Figure 3-1).

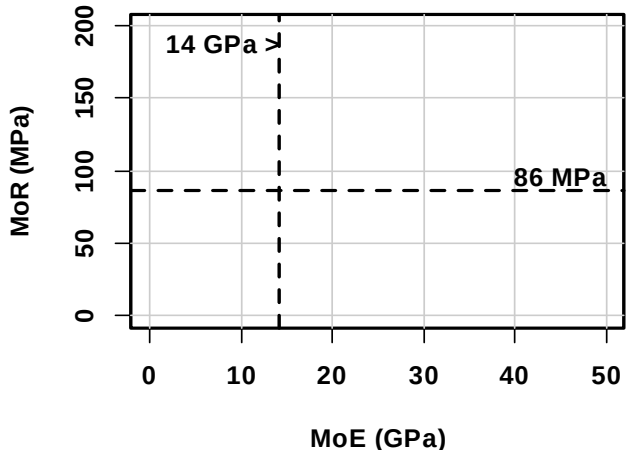

Figure 3-1: Scatter plot axes

Secondly these data are pooled and analysed (Sections 3.8 to 3.13), both in combination and as clustered subsets. By pooling these datasets, the independent sample number is increased and the statistical mean representation of an in-service girder obtained with a higher level of confidence. The confidence in this mean representation is achieved because it is calculated from data obtained from the destructive testing of a wide range of in-service girders from Tasmania, NSW and Queensland. This representation is, however, still not complete because there is a lack of data representing girders early in their service life. By pooling the available data it is possible to infer MoR-v-MoE relationships that have not been previously identified. Relationships that can be used in subsequent experiments and applications that identify how SHM techniques can be applied to a timber bridge girder.

3.2 DS-1: Tasmanian girders

DS-1 has been replotted in a MoR-v-MoE scatter plot in Figure 3-2 (refer Figure 2-17 for original author's version). Additional statistics in Figure 3-2 show the regression line (solid purple line) and the 95% confidence bound of the regression (dotted purple lines). The 95% confidence bound of the next predicted value of MoR for a given new value of MoE is also shown (dotted red lines). The data have been inferred by measuring the MoE and MoR co-ordinate points in the published graph (refer Figure 2-17). Each point in Figure 3-2 represents a two valued MoR-v-MoE vector of the tested girder. As an example girder #68 has values, MoE 10 GPa, MoR 30 MPa and is plotted as a black circle. The values for girder #68 are close to the mean value of the set, both for MoR and MoE.

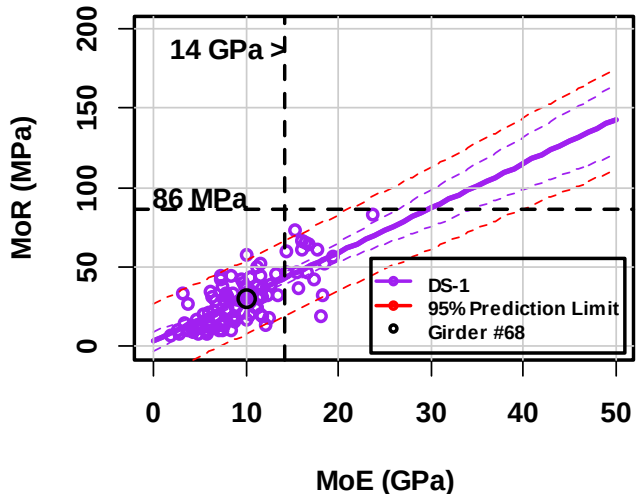

Figure 3-2: MoR-v-MoE for Tasmanian girders (DS-1), adapted from Figure 2-17

A statistical summary of these data is shown in Table 3-1. Comparisons of both the MoE and the MoR distributions of these data with a normal distribution are made in Figure 3-3 and 3-4. Also included in Figure 3-3 and 3-4 are the 95% confidence limits of those comparisons.

Table 3-1: Statistical summary of DS-1

Parameter	Value	Yttrup and Nolan (1996)
Mean MoE	9.9 GPa	NA
CV of MoE	44%	NA
Mean MoR	31.1 MPa	NA
CV of MoR	54%	NA
Regression Model, 95% confidence (Eqn 3.1)	MoR = 3.4 + 2.8 MoE	MoR = 3.4 + 2.8 MoE
Tolerance of slope, 95% confidence	0.55	NA
Coefficient of Determination, r^2, MoR-v-MoE	0.52	0.52
Number of girders	95	95

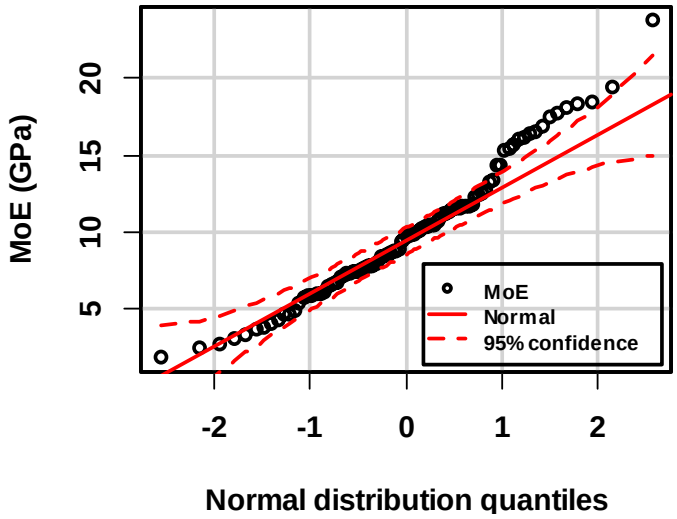

Figure 3-3: Comparison of DS-1 MoE distribution with a normal distribution

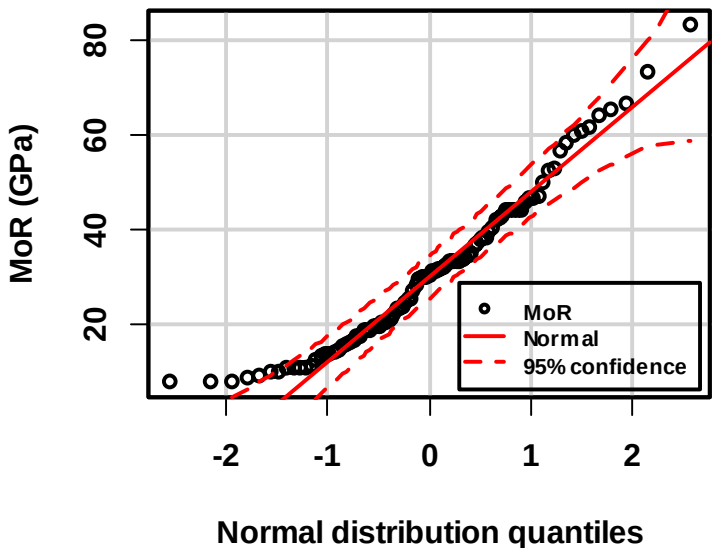

Figure 3-4: Comparison of DS-1 MoR distribution with a normal distribution

3.3 DS-2: NSW RTA N150-GRT girders

3.3.1 DS-2 MoE and MoR values

DS-2 was obtained from the RTA N150-GRT dataset that included the results of bending tests on 169 bridge girders from various locations within NSW. These data were reviewed here in order to characterise the strength of in-service timber bridge girders in NSW. A statistical summary is shown in Table 3-2. These data contain clusters that represent girders with similar features and have been clustered into subsets; DS-2-1 to DS-2-8. The number of girders, n, in each subset varied, with the smallest subset (DS-2-5) containing only five girders and the largest (DS-2-1) 169.

Age was one parameter used to cluster girders into subsets. Some of the girders were aged less than two years, DS-2-2, whereas the remaining girders were aged over 15 years, DS-2-1. These two subsets, DS-2-1 and DS-2-2, allow the MoE and MoR values to be compared between newly commissioned and aged in-service girders. A second parameter, MoE, was used to cluster those girders below and above 30 GPa; DS-2-3 and DS-2-4 respectively. DS-2-3 represents the girders with data that fits a normal distribution to a confidence level of 95% and DS-2-4 represents the outliers. The data of DS-2-4 were also clustered into a further subset, DS-2-5. DS-2-4 included both new girders of age less than two years and in-service girders of age greater than 2 years. DS-2-5 represents only those older girders with an age range of 15 to 45 years. Because they are outliers, each of the girders in subset DS-2-5 is also identified by its RTA N150 test number (refer Table 3-2, Note 1).

Whereas all 169 girders had a recorded value of MoE, they did not all have a recorded value of MoR. Specifically those of age less than two years were not tested to destruction because the owners wanted to use them in future structures. This feature was used to cluster data for DS-2-6 which only included those girders with both MoE and MoR. There was only one MoR outlier, girder #24 with an MoR of 120 MPa, and by excluding this DS-2-7 was obtained. To obtain model regression relationships between parameters a set was required that contained the complete set of cases of each parameter considered. This final cluster, DS-2-8, included all the girders that had all three parameters: MoE less than 30 GPa; measured MoR; and age data. Thus DS-2-8 only included 95 of the total 169 girder set; the other 74 girders were eliminated for lacking one or more of the three parameters.

Table 3-2: Summary statistics for DS-2

Parameter	Units	Mean	CoV%	Max	Min	n	Subset
MoE for all girders with measured MoE	GPa	15.9	51	48.0	0.4	169	DS-2-1
MoE for Girders aged less than two years	GPa	24.0	28	38.3	13.3	22	DS-2-2
MoE for Girders with MoE less than 30 GPa	GPa	14.9	44	29.9	0.4	148	DS-2-3
MoE for Girders with MoE greater than 30 GPa	GPa	38.8	14	48.0	31.0	8	DS-2-4
MoE for Girders[1] aged > 2 yrs and MoE > 30 GPa	GPa	40.4	15	48.0	31.0	5	DS-2-5
MoR for all girders with measured MoR	MPa	63.5	41	161	9	111	DS-2-6
MoR for all girders with measured MoR < 120 MPa	MPa	62.64	39	112	9	110	DS-2-7
MoR for all girders with MoE < 30 GPa and AGE data	MPa	62.60	38	112	9	95	DS-2-8

Note 1: Girders of age > 2 yrs and with MoE above 30 GPa, subset 2-5, were numbered: 24, 85, 92, 157, 164

Box plot summaries of the MoE data for DS-2-1 and DS-2-3 are shown in Figure 3-5. The box plot representing DS-2-1 shows a summary of all the 169 girders and that representing DS-2-3 shows a summary with the outliers above 30 GPa excluded. The DS-2-1 MoE data distribution is compared with a normal distribution in Figure 3-6. When the data from the girders that appear as outliers in Figure 3-5 and 3-6 are removed, the quantile comparisons are then as in Figure 3-7. The DS-2-3 MoE data then compares well to a normal distribution with a 95% level of confidence.

Box plot summaries of the DS-2 MoR data are shown in Figure 3-8. The box plot representing DS-2-6 shows a summary of the 111 girders that had recorded values of MoR and the box plot representing DS-2-7 shows a summary with the single outlier, at 161 MPa, excluded. The DS-2-6 MoR data distribution is compared with a normal distribution in Figure 3-9 and with the outlier removed in Figure 3-10. Without the outlier, the DS-2-7 MoR data, compares well to a normal distribution with a 95% level of confidence.

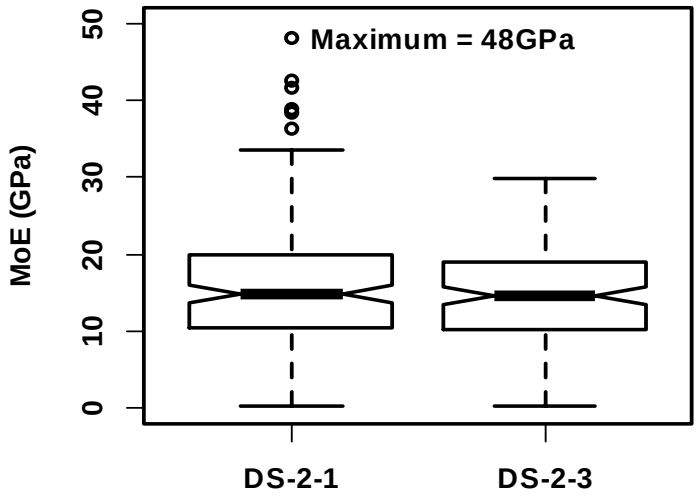

Figure 3-5: MoE box plot summaries; DS-2-1 (All MoE) and DS-2-3 (MoE<30GPa)

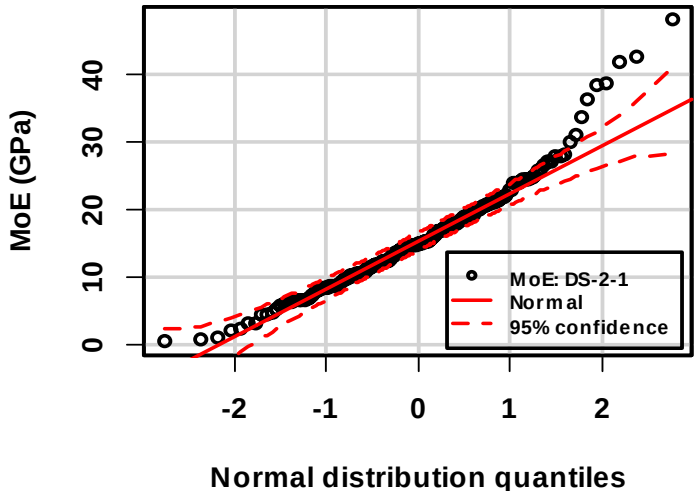

Figure 3-6: DS-2-1 MoE distribution compared to a normal distribution

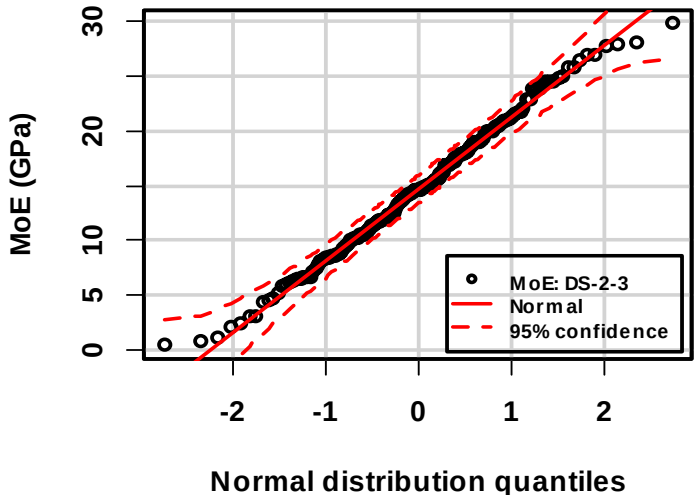

Figure 3-7: DS-2-3 MoE distribution (< 30GPa) compared to a normal distribution

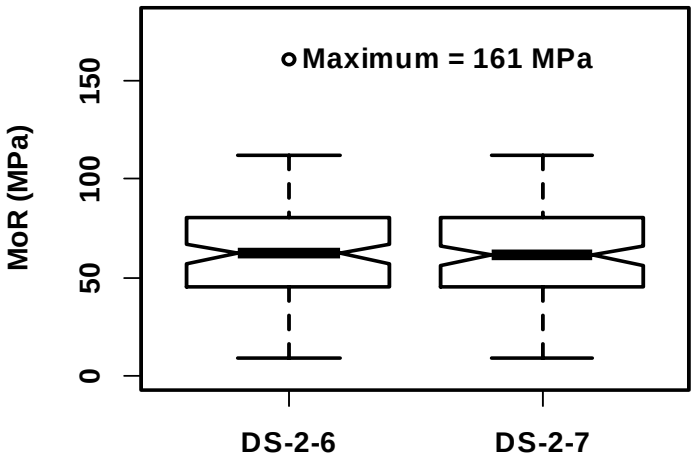

Figure 3-8: MoR box plot summaries of DS-2-6 (All MoR) and DS-2-7 (< 120 MPa)

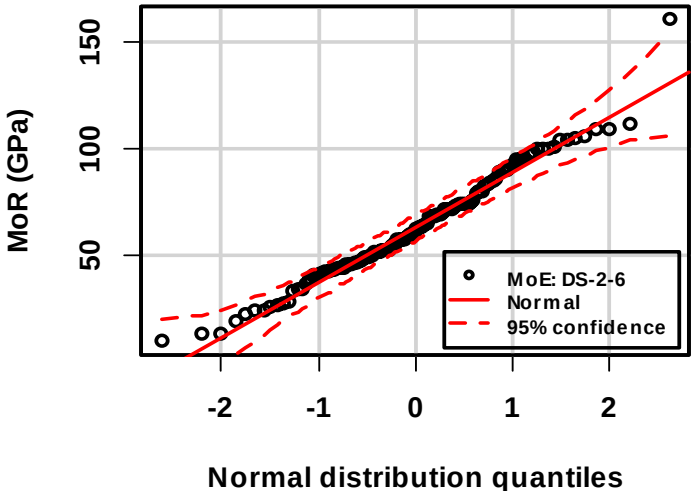

Figure 3-9: DS-2-6 MoR distribution compared to a normal distribution

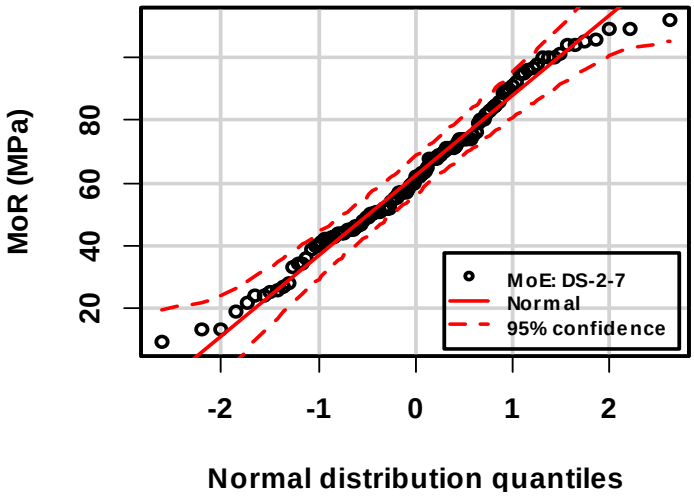

Figure 3-10: DS-2-7 MoR distribution (< 120MPa) compared to a normal distribution

In DS-2-6 there is only one significant outlier with an MoR of 161 MPa (refer Figure 3-8). This data, related to girder #24, is about six (5.93) standard deviations higher than the set mean. The next strongest girder, #96, had MoR of 112 MPa and is within two standard deviations of the mean. Girder #24 also had a high value of MoE, 41.7 GPa, as shown in Table 3-3. Both of these girders, #24 and #96, had depths below the NSW minimum diameter requirement of 450 mm. Sorting DS-2 to find any girders that were over 450 mm in depth and that also had measured values of MoE and MoR above the mean values of MoE and MoR produced only one girder, #32. The available details of these three girders are shown in Table 3-3.

Table 3-3: Summary statistics of girders #24, #96 and #32

Parameter	**Value**			
Girder Number	**#24**	**#96**	**#32**	**DS-2 means**
Bridge site	Cutty Creek	Skinners Swamp	Barrangnayatti	NA
MoE (GPa)	41.7	23.8	19.1	15.9
MoR (MPa)	161	112	92	63.5
Depth	320	340	450	403
Width	300	379	450	427
Span	8900	8700	10400	8300
Age (Years)	15	15	30	27

3.3.2 DS-2 girder diameters

The girders in the RTA N150 dataset were described as being both of elliptical cross-section and sawn rectangular cross-section but whether a girder was elliptical or round was not directly identified in the data. However, all calculation in the source data of the girder SMA was on the basis of an elliptical cross-section; there was, therefore, no evidence of any rectangular section girders in the available data. Both axes, vertical and horizontal, of each girder were measured and recorded. They are described here as the depth diameter and the width diameter respectively. Although the depth and width were recorded, there was no estimate of the condition of the surface and, therefore, no estimation of the effective diameters. The effective diameter of a girder will be less than the actual outside diameter because there will be some material in the surface layers that does not contribute to the load bearing capacity of the girder. Since the only diameter recorded was the outside diameter it defines the maximum diameter with the effective diameter being either the same or less. Therefore, in the following calculations the measured diameter is considered the effective diameter. The girder depth diameters (blue points) and width diameters (green points) are shown in Figure 3-11. Also highlighted in red are the five girders in sub-set DS-2-5 (> 2 yrs, > 30 GPa). Although the DS-2-5 girders had high MoE they were in the bottom half of the diameter distribution, with diameters below about 400 mm. The mean depth

diameter, of DS-2, was about 400 mm and an upper limit including 95% of all girders was about 520 mm with about 80% of all the girders having a depth diameter of less than 450 mm.

Figure 3-12 shows the girder depth diameter to girder width diameter ratio (blue points), the mean value of which is 0.96. There were about half of the girders (55%) wider than they were deep. Their major axis (width diameter) was horizontal and their minor axis (depth diameter) was vertical. There was no reason recorded in the data identifying any orientation preference. Also highlighted in Figure 3-12 are DS-2-5 data (red points).

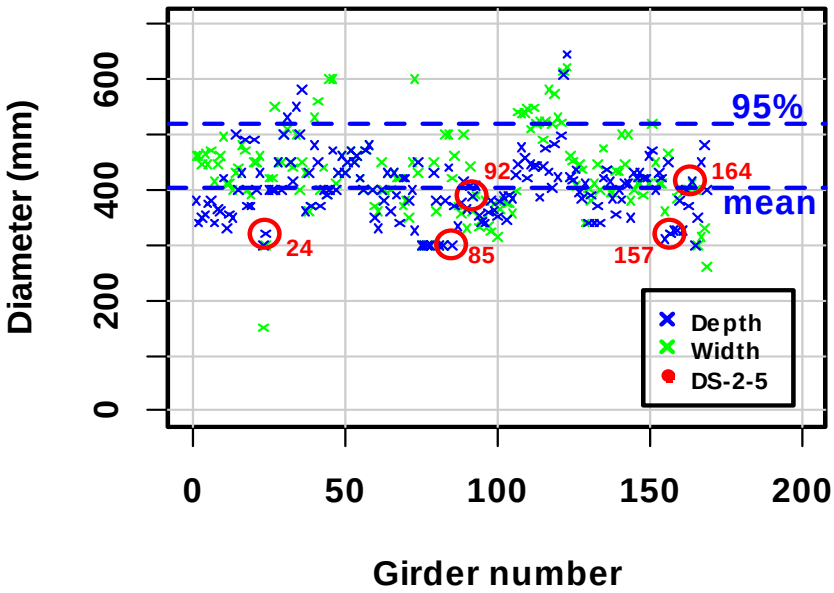

Figure 3-11: Girder diameter (both width and depth) versus girder number for DS-2; DS-2-5 girders highlighted

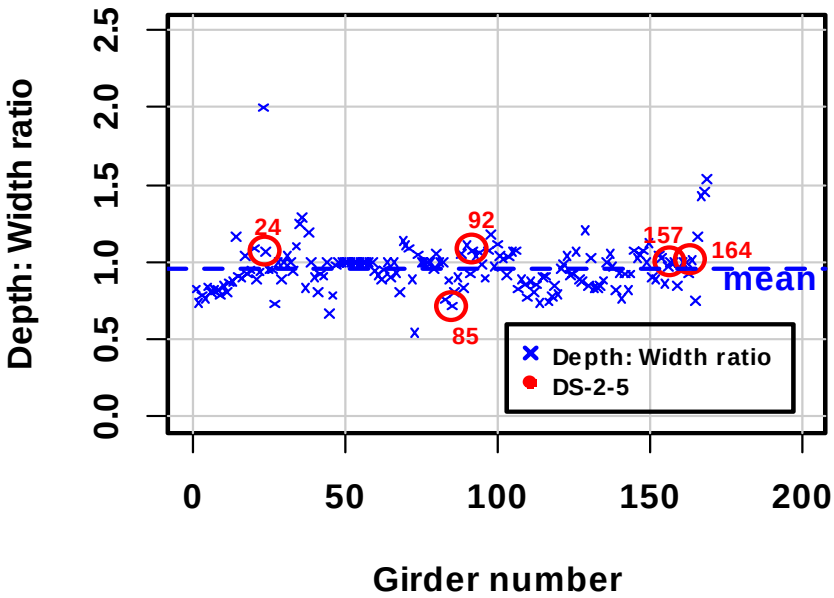

Figure 3-12: Ratio of DS-2 depth diameter: width diameter versus girder number; DS-2-5 highlighted

3.3.3 DS-2 MoE and MoR with girder age

The girders clustered as DS-2-4 are those girders which all have MoE above 30 GPa. They are from a wide range of age (six months to 45 years) and include three girders aged less than two years. They are the upper 5% of DS-2, with MoE above the 95% upper limit, for set DS-2-3, of 27.5 GPa. In Figure 3-13 DS-2-4 MoE is plotted against age. Also shown in that figure is a regression line, the slope depicting the change of MoE per year. The regression line has a slightly negligible slope of −0.03 ± 0.28 GPa to a confidence level of 95%. Most (95%) of the DS-2-3 girders have MoE below 27.5 GPa and this confidence bound is shown as a dotted green line (Figure 3-13). These summary regression data for Figure 3-13 are given in Table 3-4. The characteristic details of each of the eight DS-2-4 girders are as in Table 3-5. Also included in Table 3-5 is data for girder #32 from Table 3-3.

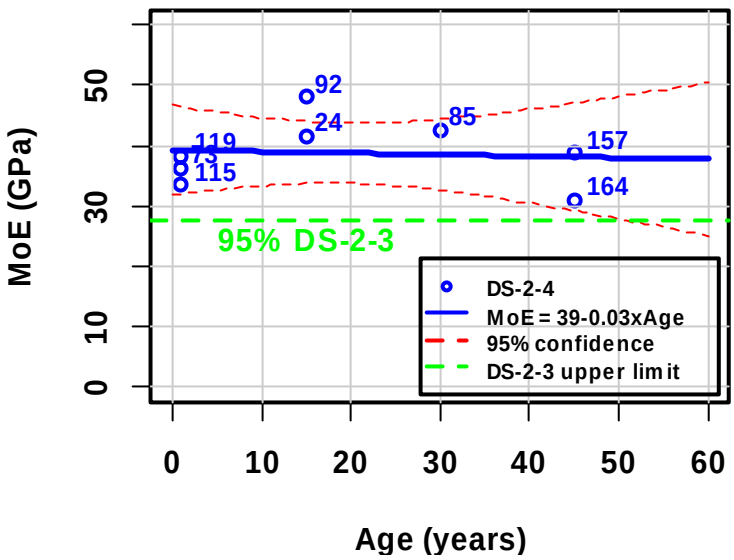

Figure 3-13: DS-2-4 MoE-v-Age (MoE > 30 GPa, all ages)

Table 3-4: Rate of change of MoE-v-Age for girders in DS-2-3 (MoE > 30 GPa)

Parameter	Value
Intercept, MoE	39.3 (GPa)
Mean Value of slope	-0.03 (GPa per year)
Tolerance	0.28 (GPa)
Lower limit of slope at 95% confidence	-0.31 (GPa per year)
Upper limit of slope at 95% confidence	0.25 (GPa per year)
Upper 95% limit of DS-2-3, MoE	27.5 (GPa)
Regression model, 95% confidence (Eqn 3.2)	MoE = 39.3 – 0.03 ± 0.28 x Age

Table 3-5: Characteristics of DS-2-4 girders (MoE > 30 GPa) and girder #32

Girder	Site	Number of girders at site	Length	Width	Depth	Age (Years)	MoE (GPa)	MoR (MPa)	Girder subset
24	3	4	8900	300	320	15	41.7	161	DS-2-4
73	10	6	10500	600	325	1	36.3	NA	DS-2-4
85	11	8	8500	420	300	30	42.6	74	DS-2-4
92	12	35	8500	360	387	15	48.0	NA	DS-2-4
115	12	35	8330	490	441	1	33.5	NA	DS-2-4
119	12	35	8350	572	483	1	38.3	NA	DS-2-4
157	19	10	8580	320	320	45	38.7	109	DS-2-4
164	19	10	10730	410	415	45	31.0	109	DS-2-4
32	5	4	10400	450	450	30	19.1	92	DS-2-3

All those DS-2 girders with MoE that fit a normal distribution are represented by DS-2-3. These MoE data are plotted, for each girder, against girder age in Figure 3-14 (blue points). The mean rate of change of MoE with age is shown by the regression line (solid blue line) with a negative slope of −137 ± 56 MPa per year. This MoE-v-Age data contains a high level of variation and a coefficient of determination of 0.14. Thus, only 14% of the variation is explained by variation in Age. The S2 limit (14 GPa) divides the DS-2-3 set into about two with 56% having MoE above the limit. If only those girders of age greater than two years are considered then only 42% have MoE greater than the S2 limit and girder #32 is one of these. The prediction limits (95% confidence level) are within 12 GPa of the regression line (dotted red lines, Figure 3-14). The DS-2-4 data (green points) are above these prediction limits and also over eight standard deviations higher than the CSIRO published data (refer Table 2-3) for Ironbark. Any historical reason for a separation between DS-2-3 and DS-2-4 was absent from the source data (refer Table 3-5 for available data).

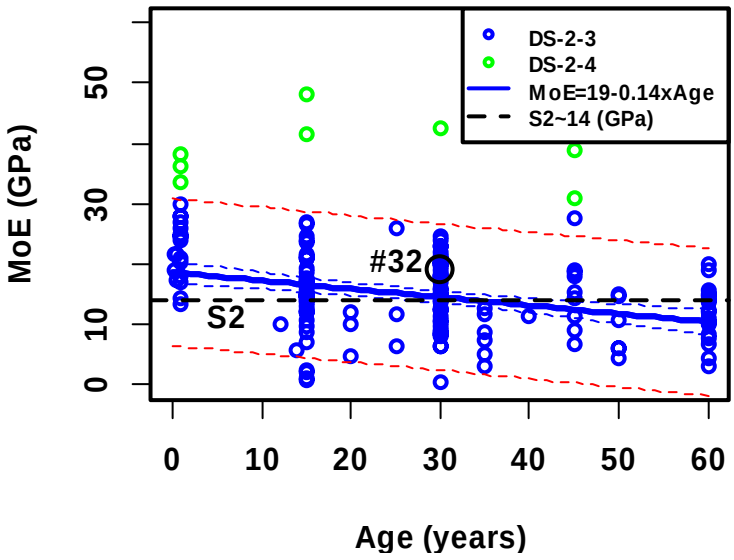

Figure 3-14: DS-2-3 regression of MoE-v-Age (MoE < 30 GPa) together with DS-2-4 outliers (green) and girder #32 (black point);

The DS-2-7 MoR data is shown in Figure 3-15. The mean rate of change of MoR with age is shown by the regression line (solid blue line) with a negative slope of −0.43 ± 0.28 MPa per year; the regression limits (95% confidence level) are indicated by the dotted blue lines. Only 22% of girder MoR is above the S2 limit of 86 MPa and this includes girder #32. There is a high level of variation in this data with Age alone being a poor predictor of MoR; the explained variation is about 11% (coefficient of determination = 0.11). The range of the prediction limits (92 MPa, 95% confidence level shown as dotted red lines) are similar to the full data range (105 MPa).

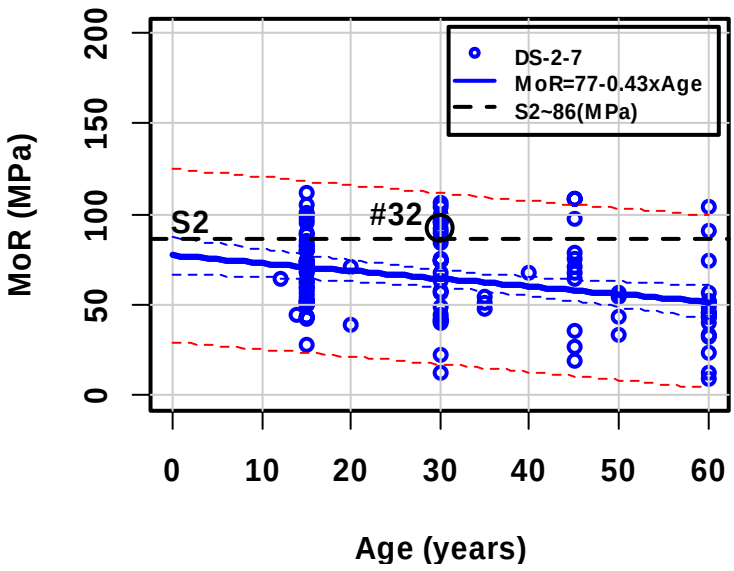

Figure 3-15: DS-2-7 regression of MoR-v-Age; also girder #32 (black point) and S2 limit (dotted black line)

In Figure 3-16 the regression of MoR is examined with MoE as the prediction parameter. The data utilised were DS-2-8 with consistent MoR, MoE and Age data but minus outliers. By utilising MoE as a predictor the unexplained variation was decreased by about 58% with a coefficient of determination of 0.48 being achieved. The slope of the regression equation was about 3 ± 0.6 (MPa/GPa) (95% confidence, p-value 0.003). Most of the girders, about 82%, were below the S2 limit states. Thus only 18% were serviceable girders, that is with MoR above 86 MPa and MoE above 14 GPa.

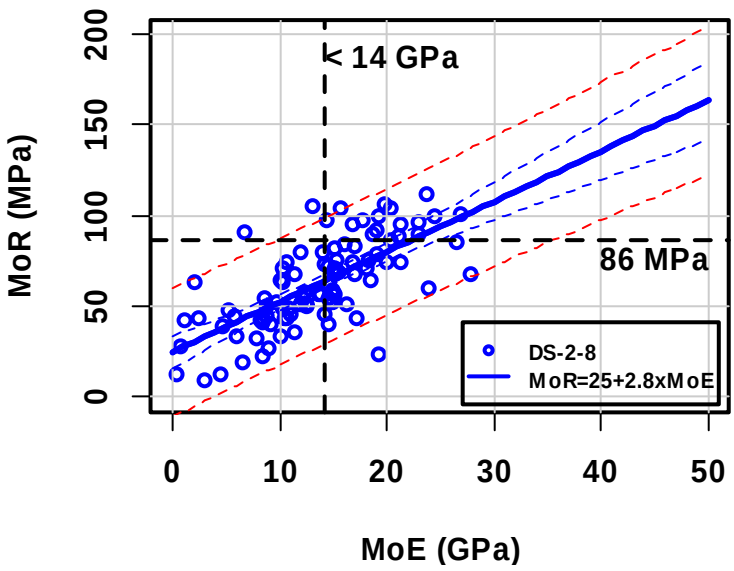

Figure 3-16: DS-2-8 regression of MoR-v-MoE

3.3.4 Summary of DS-2 regression models

The rate of change per year of MoE for DS-2-3 and DS-2-4 are shown in Table 3-6 for a confidence limit of 95% (Note: units are MPa per year).

Table 3-6:Rate of change of MoE-v-Age for girders in DS-2-3 and DS-2-4

Subset	Description	Change per year; slope of regression lines	Tolerance	p-value	r^2	Figure
DS-2-4	MoE > 30 GPa	-25 MPa per year	±280 MPa	0.84	0.007	4-12
DS-2-3	MoE < 30 GPa	-137 MPa per year	± 56 MPa	< 0.001	0.14	4-13

In addition to the regressions of MoR on Age and MoR on MoE a further regression was performed for the DS-2-8 data. This was the regression of MoR on both MoE and Age. All three regression models are shown in Table 3-7. These models include the slope estimates (Est 1 and Est 2) and the 95% confidence intervals shown as a tolerance (Tol 1 and Tol 2).

Table 3-7: Estimates of slope and slope tolerance for MoR regressed on MoE, Age, and MoE with Age for DS-2-8

Eqn	Regression model (MoR ~ Est1 + Est2)	Est 1	Tol 1	Est 2	Tol 2	Intercept	r^2	p-value
4.3	MoR (MPa) ~ MoE (GPa)	2.77	0.59			24.8	0.484	< 0.001
4.4	MoR (MPa) ~ Age (Years)[1]	-0.48	0.27			77.7	0.115	< 0.001
4.5	MoR (MPa) ~ MoE (GPa) + Age (Years)	2.64	0.56	-0.35	0.20	37.7	0.546	< 0.001

Note 1: On average the nominal reduction of MoR = -0.48 ± 60% MPa per year

3.4 DS-3 and DS-5: NSW RTA N150/06 girders and clears

DS-3 and DS-5 data, both from the same girders, are shown as box plot summaries in Figure 3-17 and 3-18. In Figure 3-17 the full range of MoE from the sections A, B and C are shown and in Figure 3-18 the full range of MoR. The physical positions of these girder sections are shown in Figure 2-24.

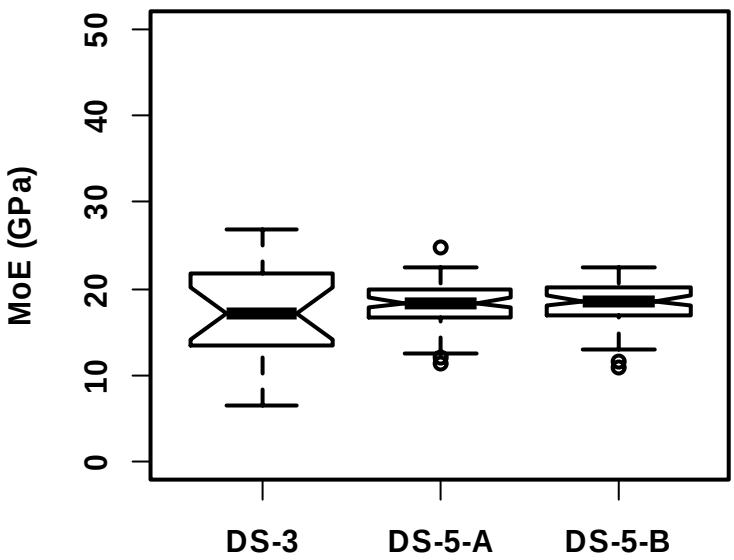

Figure 3-17: Box plot summaries of DS-3 AND DS-5 MoE

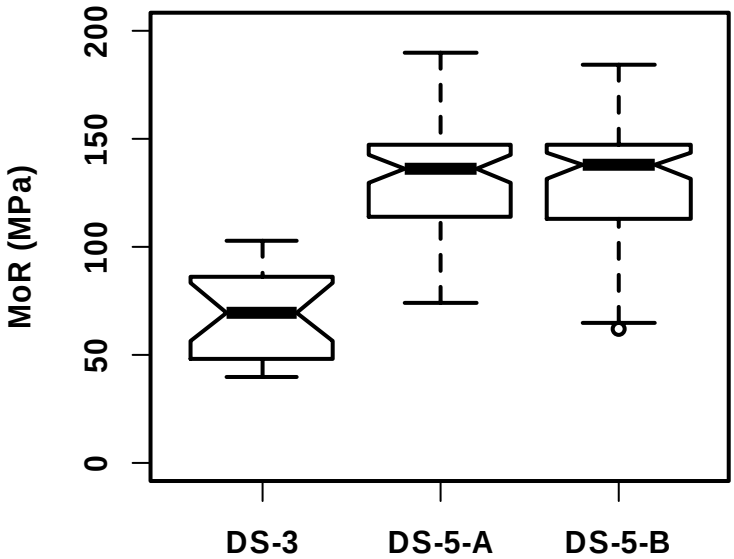

Figure 3-18: Box plot summaries of DS-3 and DS-5 MoR

In Figure 3-19 the DS-3 and DS-5 MoE data are compared according to each girder. The samples from region B, between the supports and the mid-span, are from the region that is nominally the least stressed. In region A, a girder will have been subjected to compression between the load and the support while at the edge of the support the girder will have been subjected to shear

stress. Those samples from region C will only have been subjected to bending stress. It is notable that the mid-span MoE (region C, round brown dots) is, for some girders, above the excised clear MoE data (region A and B, green and blue dots) and for other girders it is below. The DS-3 and DS-5 MoR data are compared in Figure 3-20. In this case it is notable that the mid-span MoR (region C, brown dots) are equal to, or below, the excised clear MoR data (region A and B, green and blue dots)

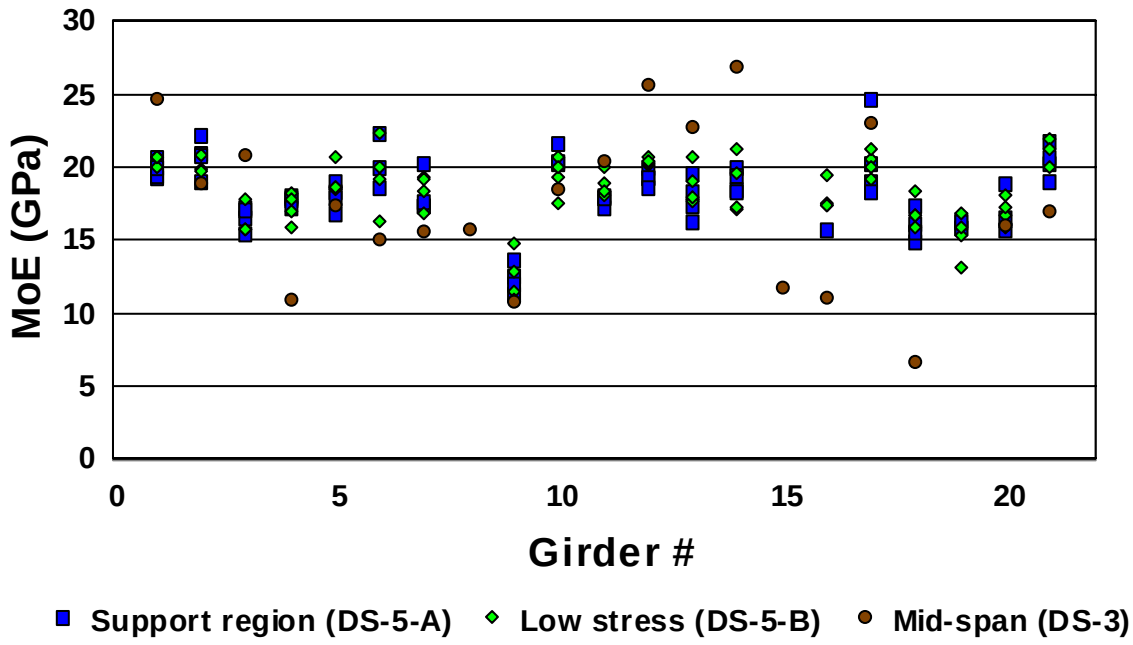

Figure 3-19: MoE-v-Girder number for all sample regions, DS-3 and DS-5

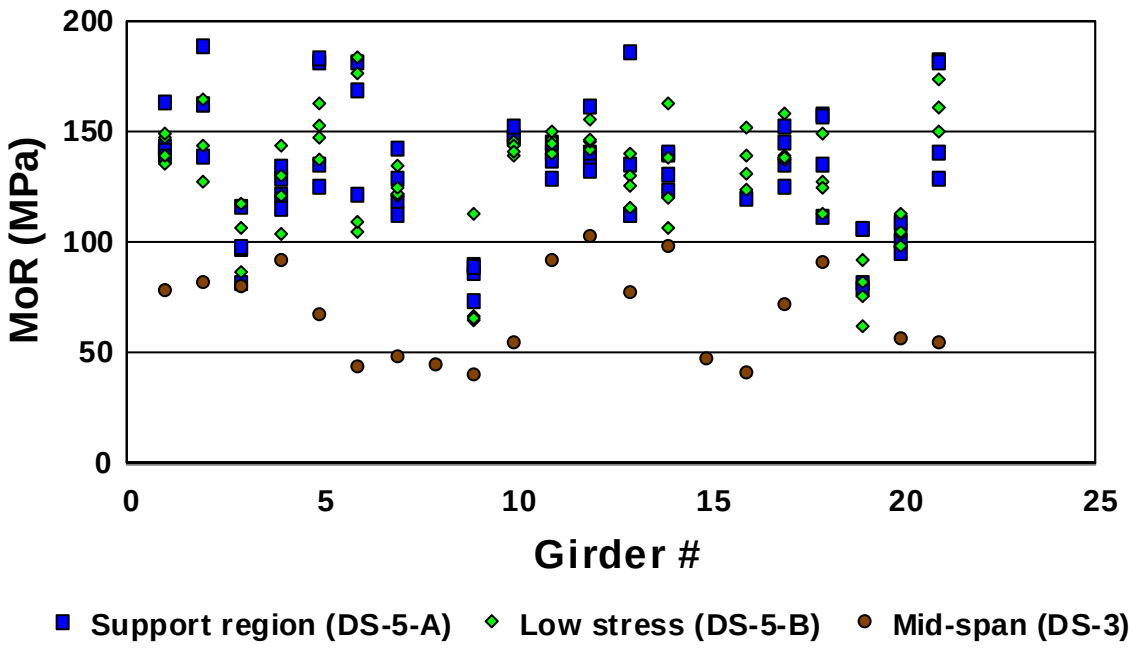

Figure 3-20: MoR-v-Girder number for all sample regions, DS-3 and DS-5

The combined MoE data (from Figure 3-19) and the combined MoR data (from Figure 3-20) are shown in Figure 3-21 but with any outliers excluded. These data are shown as MoR-v-MoE points that represent the MoR-v-MoE vector that the girders reached when they were removed from service. They represent girders at the end of life and only 17% of the girder vectors (brown dots) are above the two S2 limit states. In contrast the early service life is represented by the excised low stress clear sample data (green dots). Only one of these early life vectors is below the S2 limit state values, the rest are all above. These DS-3 and DS-5 data do not include girder age nor condition state. A numerical comparison of DS-3 and DS-5 MoR is provided in Table 3-8.

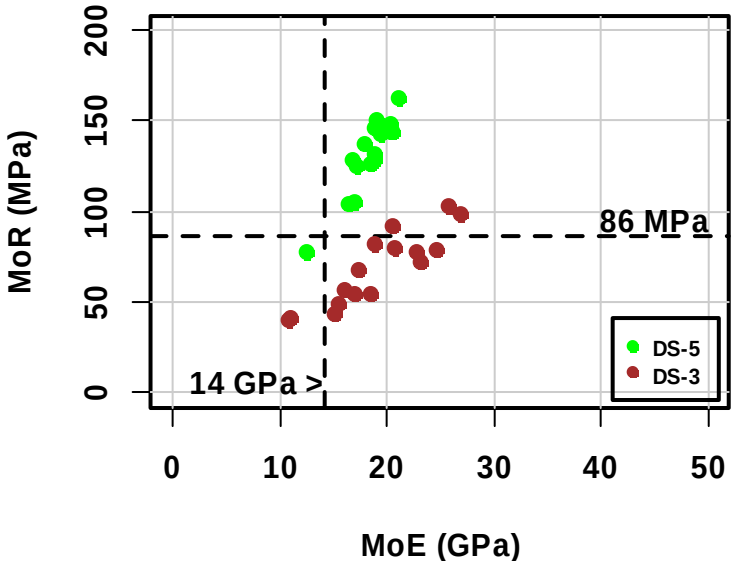

Figure 3-21: DS-3 and mean DS-5 MoR-v-MoE scatter plot

Table 3-8: Numerical summary of DS-3 and DS-5 MoR features

Parameter	Value
Mean girder ratio of DS-5 MoR to DS-3 MoR (green dot to brown dot for each girder)	2.0
Minimum girder ratio of DS-5 MoR to DS-3 MoR (green dot to brown dot for each girder)	1.3

DS-3 Girder #14 is an example of a girder with MoE that is higher than that of any of its excised clear sample MoE (refer Figure 3-19 and 3-20). Since the species of girder #14 is identified as Ironbark, the MoR-v-MoE data for this girder is shown in Figure 3-22 together with the six sigma Ironbark mean range. A statistical summary of these data are shown in Table 3-9. The excised clears are exptected to be selected for an absence of defects. Since defects can reduce MoR it is anticipated that clear MoR will be higher than girder MoR, which it is. The mean clear MoR is 35% higher than the girder MoR and all the individual clear samples have MoR above

the girder MoR. This same girder has a variation in the clear MoR data of 16% CV which appears a high level of variation for clears that are both low in defects and from the same girder.

While the girder MoR is less than the clear MoR the girder MoE does not follow this pattern. The clear MoE is only about 70% of the girder MoE. It is not obvious that the same mechanism that caused a reduction of girder MoR can also cause an increase in girder MoE. Any reason for this increase is absent from the DS-3 and DS-5 data.

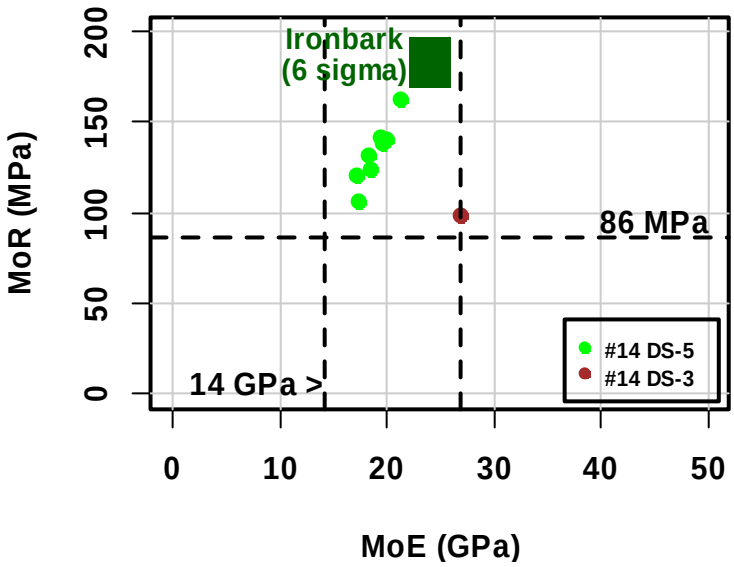

Figure 3-22: DS-3 and DS-5 MoR-v-MoE for girder #14

Table 3-9: Statistical summary of Girder #14 data

Parameter	Value
Excised sample Mean MoE	19.0 (GPa)
Girder MoE	26.9 (GPa)
MoE Coefficient of variation, CoV	7%
Ratio of MoE clears to Girder MoE	0.71
Excised sample Mean MoR	133.3 (MPa)
Girder MoR	99 (MPa)
MoR Coefficient of variation, CoV	16%
Ratio of MoR clears to Girder MoR	1.35

3.5 DS-4: QDMR girders

3.5.1 DS-4 MoE and MoR values all species

DS-4 consists of the QDMR dataset that included the results of bending tests on 53 bridge girders from various locations within QLD and carried out c. 2005 by Wilkinson (2008) (refer Section 2.4.4). An overall statistical summary is given in Table 3-10. These data contain clusters that represent girders with similar features and have been clustered into subsets; DS-4-1 to

DS-4-11 (refer also Table 3-11). Data from girders #1 and #39 are omitted as outliers; otherwise DS-4-1 is the complete set. The girder species, condition state and failure mode were all reported. DS-4-2 represents all those that failed in mid-span bending and DS-4-3 all those that failed in modes other than mid-span bending. Other modes include crushing, shear, compression and other non-mid-span failures. DS-4-4 represented just the Ironbark girders that failed in mid-span bending and DS-4-5 those Ironbark girders that failed in other modes. DS-4-6 represents all the Ironbark girders no matter what their failure mode. Summary box plot statistics of data in Table 3-10 are presented in Figure 3-23 and 3-24 of MoE and MoR respectively for DS-4-1 through to DS-4-5.

Table 3-10: Summary statistics for DS-4

Parameter	Units	Mean	CoV%	Max	Min	N	Sub-set
MoE for all girders with measured MoE	GPa	18.2	36	35.3	4.1	51	DS-4-1
MoR for all girders with measured MoR	MPa	65.7	44	129.1	11.5	51	DS-4-1
MoE for girders failed in mid-span bending	GPa	19.4	37	35.3	8.8	22	DS-4-2
MoR for girders failed in mid-span bending	MPa	80.1	35	129.1	29.5	22	DS-4-2
MoE for girders with other failure modes	GPa	17.3	33	29.8	4.1	29	DS-4-3
MoR for girders with other failure modes	MPa	54.8	45	91.3	11.5	29	DS-4-3
MoE for Ironbark girders failed in mid-span bending	GPa	25.5	21	35.3	18.6	9	DS-4-4
MoR for Ironbark girders failed in mid-span bending	MPa	101.4	13	129.1	80.7	9	DS-4-4
MoE for Ironbark girders with other failure modes	GPa	15.9	36	29.8	4.2	18	DS-4-5
MoR for Ironbark girders with other failure modes	MPa	47.4	47	83	11.5	18	DS-4-5
MoE for Ironbark girders all failure modes[1]	GPa	19.6	34	35.3	8.0	26	DS-4-6
MoR for Ironbark girders all failure modes[1]	MPa	65.5	50	129.1	11.5	26	DS-4-6

Note 1: Girders numbered 1 and 2 omitted as outliers

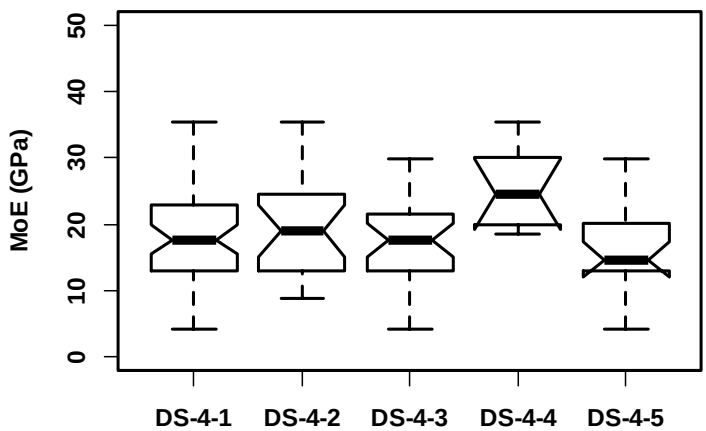

Figure 3-23: DS-4-1 to DS-4-5 MoE Box plot summary statistics

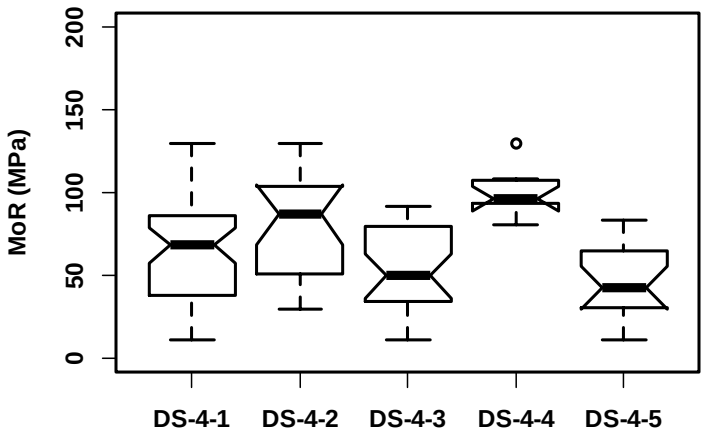

Figure 3-24: DS-4-1 to DS-4-5 MoR Box plot summary statistics

Each girder had a condition state rating, based on visual assessment and identified prior to testing. In Figure 3-25 a box plot summary is given of the DS-4 Ironbark girders MoE in each condition state and in Figure 3-26 a summary of DS-4 Ironbark girder MoR. The mean values of both MoE and MoR nominally decrease with condition state but it cannot be identified from these summaries how MoR interacts with MoE. To represent that interaction a scatter plot must be drawn.

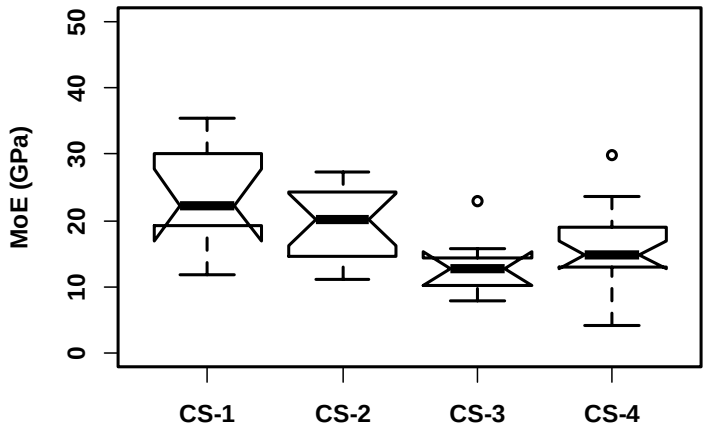

Figure 3-25: Box plot summaries of DS-4 MoE as ranked by condition state CS-1 to CS-4

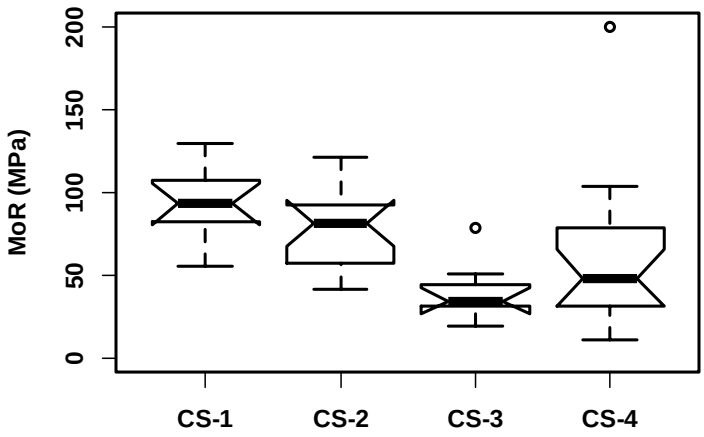

Figure 3-26: Box plots summaries of DS-4 MoR as ranked by condition state CS-1 TO CS-4

A scatter plot of the DS-4-1 MoR-v-MoE vector points are shown in Figure 3-27. Only 25% of these girders have MoR-v-MoE vector points in the upper right quadrant above the S2 limits. The regression model, to a 95% level of confidence is: MoR = -7.3 MPa+ (3.9 GPa± 19%) (Eqn 4.6). The explained variation is 70% with a coefficient of determination, of 0.6997 and the girder MoR is nominally proportional to MoE.

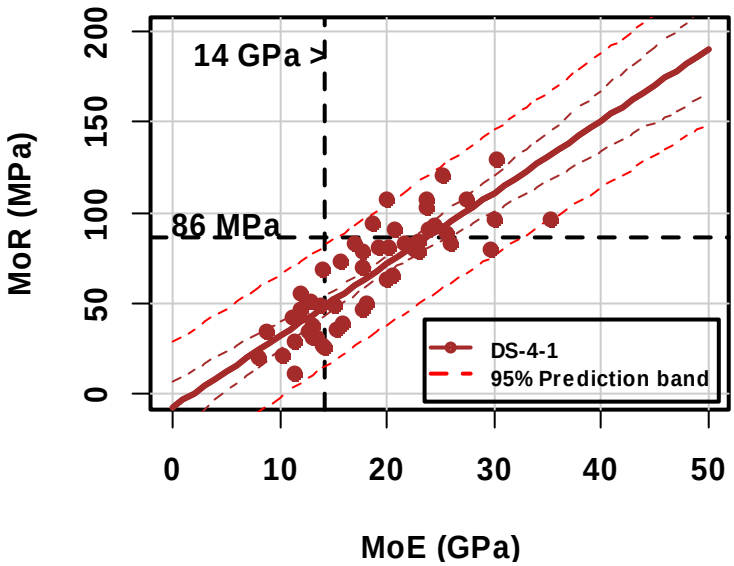

Figure 3-27: DS-4-1 MoR-V-MoE data

3.5.2 DS-4 MoE and MoR values for Ironbark

The DS-4-6 data represents only the 26 Ironbark girders. Both their in-service end-of-life condition status and their mode of failure were recorded. It is, therefore, possible to cluster the data into subsets that represent five levels of degradation. These subsets are described in Table 3-11 and the MoR-v-MoE vector for each girder is plotted in Figure 3-28. The DS-4-7 and DS-4-8 MoR-v-MoE vector data points are all (except one) in the top right quadrant above both

the S2 limit states. DS-4-9 to DS-4-11 are all below the S2 MoR limit state of 86 MPa. However, scatter in DS-4-9 and DS-4-10 is significant with girder #48 and girder #49 having MoR values of about 79 MPa, 92% of the S2 limit. This is despite these two girders being visually assessed as being CS-3 and CS-4 respectively. If the data for these two girders are eliminated the remainder of the DS-4-10 and DS-4-11 data is then less than 60% of the S2 limit.

Table 3-11: Description of the breakup of sub-set 4-6 into further subsets

Description of subset	Number of girders, N	Subset
Bending failure, CS-1	6	DS-4-7
Bending failure, CS-2	3	DS-4-8
Non bending failure, CS-2	4	DS-4-9
Non bending failure, CS-3	3	DS-4-10
Non bending failure, CS-4	10	DS-4-11

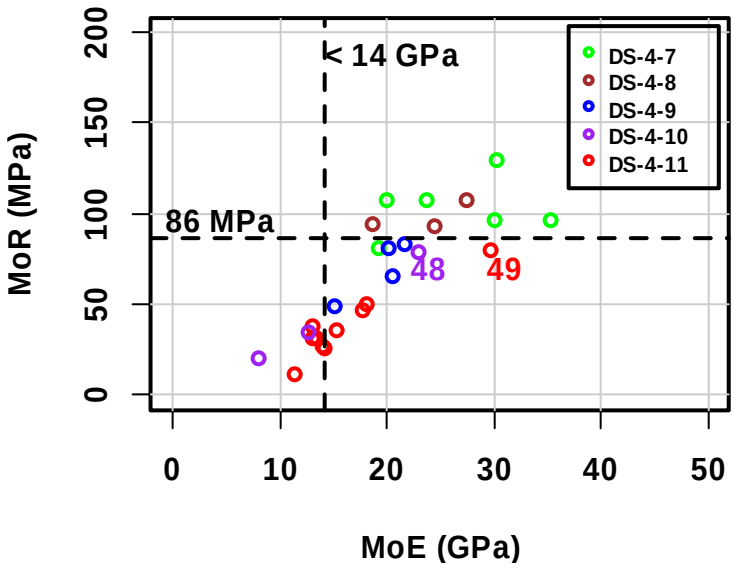

Figure 3-28: MoR-v-MoE values for the Ironbark girder sub-sets DS-4-7 to DS-4-11

Each DS-4-6 MoR-v-MoE vector point in Figure 3-28 is an independent pair of values representing a particular girder. By plotting the path of a vector representing the mean values of these data an MoR-v-MoE vector path is obtained that represents girder degradation. To identify the level of degradation the girders were firstly selected for failure mode and secondly for condition state as already identified in Table 3-10. Then the mean values for Ironbark were used to define a starting point. The two groups DS-4-10 and DS-4-11 were combined into one subset to define the end point. The results of this clustering are as shown in Table 3-12 and Figure 3-29.

Table 3-12: Description of DS-4 Ironbark MoR-v-MoE

Point number	Description of MoR-v-MoE point	Number of girders	Sub-set
1	Mean value for Ironbark	13	
2	Mean value CS-1, Bending failure	6	DS-4-7
3	Mean value CS-2, Bending failure	3	DS-4-8
4	Mean value CS-2, non Bending failure	4	DS-4-9
5	Mean value CS-3 and CS-4, non Bending failure	13	DS-4-10 and DS-4-11

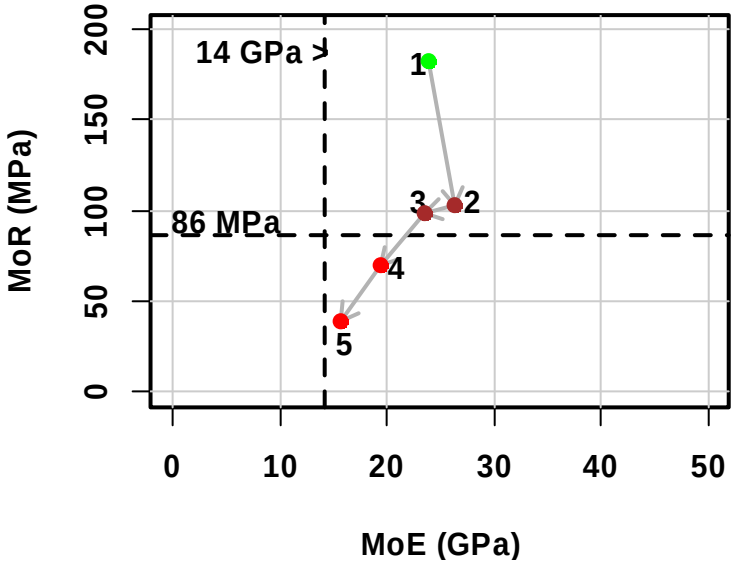

Figure 3-29: DS-4-6 Ironbark degradation vector path (points 1-5, refer Table 3-12)

Point number 1 is specified as the mean MoR and MoE values for un-degraded Ironbark, representing the point where a new girder might start its in-service life; particularly its MoE. Point 2 represents the first measureable girder MoR-v-MoE value, identified by the DS-4-7 girders that were graded CS-1 and that failed in bending failure. These girders were assessed, by visual inspection, as not having suffered any significant degradation. Those represented by point 3 are the DS-4-8 girders that were graded by visual assessment as CS-2. They had identifiable surface erosion and checking but still failed in mid-span bending. Point 4 represents the DS-4-9 girders that were identified as CS-2 but failed in a mode other than mid-span bending because of unidentified degradation in the region of the girder away from the mid span. Such faults include splitting, rot and piping. The last point, number 5, represents those girders (DS-4-10 and DS-4-11) that were identified as CS-3 and CS-4 and also did not fail in mid-span bending but by some other failure mode.

The DS-4-6 data was also clustered according to condition state as shown in Table 3-13 and plotted in Figure 3-30. A statistical summary of the regression model is given in Table 3-14. Parameters that can be used to estimate DS-4-6 girder MoR are MoE and Condition State. The regression models of MoR on both MoE and Condition State are shown in Table 3-15

Table 3-13: Description of Ironbark MoR-v-MoE points in Figure 3-30

Point number	Description of MoR-v-MoE point	Number of girders	Sub-set
CS-1	Mean value Condition State 1	6	DS-4-7
CS-2	Mean value Condition State 2	7	DS-4-8 and DS-4-9
CS-3	Mean value Condition State 3	3	DS-4-10
CS-4	Mean value Condition State 4	10	DS-4-11

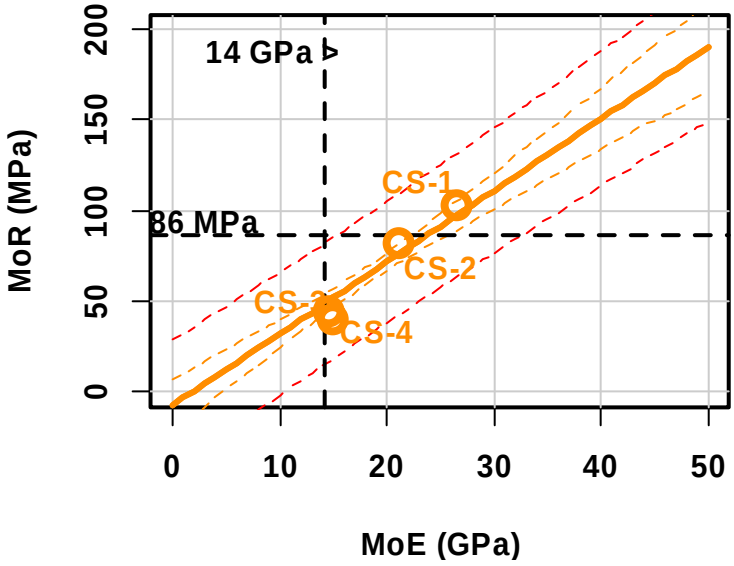

Figure 3-30: DS-4-6 Ironbark mean MoR-v-mean MoE

Table 3-14: Statistical summary of the regression shown in Figure 3-30

Parameter	Value
Regression Model, 95% confidence	MoR (MPa) = -35.0 + 5.3 x MoE (GPa)
Coefficient of Determination, r^2, MoR-v-MoE	0.98
Tolerance of slope, 95% confidence limit	± 2.2 (GPa)

Table 3-15: Estimates of slope and slope tolerance for MoR regressed on MoE, Age and Condition State for DS-4-6 Ironbark girders

Regression model	Est 1	Tol 1	Tol 1%	Est 2	Tol 2	Intercept	Coefficient of determination, r^2
MoR ~ MoE	4.11	1.10	27%			-15.3	0.712
MoR ~ Condition State	-22.2	6.53	29%			124	0.674
MoR ~ MoE + Condition State	2.66	0.986	37%	-13.2	5.49	48.2	0.861

3.5.3 Comparison of DS-4-6 (Ironbark) with DS-4-1 (all species)

In this section the DS-4-1 Ironbark data is compared graphically with the complete DS-4-1 data. Firstly the regression model for DS-4-1, already shown in Figure 3-27, is repeated (grey points and lines) in Figure 3-31 and the DS-4-6 data for Ironbark overlaid. The DS-4-6 data is a subset of the DS-4-1 data and so is expected to be nominally within similar prediction band limits, as it is. The inference from Figure 3-31 is that the DS-4-6 data also nominally has a similar range and mean as DS-4-1. The lifetime in-service MoR-v-MoE vector for an Ironbark girder does, therefore, follow a similar path to that of the other species and vice-versa.

Secondly the regression model for DS-4-1 is repeated in Figure 3-32 and the data from Figure 3-30 overlaid. The mean MoR-v-MoE vector values for the clusters that represent the different

condition states are positioned within one standard error of the DS-4-1 regression line. It is inferred, therefore, that the mean CS-1 to CS-4 data is representative of a mean in-service girder. These mean data, CS-1 to CS-4, represents the combined experience of trained visual inspectors. It is also inferred, therefore, that on the average trained visual inspectors are correctly estimating the condition status of bridge girders. This does not mean that every girder is correctly assessed. Girder #48 and girder #49 might be examples of incorrect assessment (refer Figure 3-28). However, the reason for such an incorrect assessment may be caused by an inability to quantify the girder strength by visual assessment techniques and not by any error on behalf of the inspector.

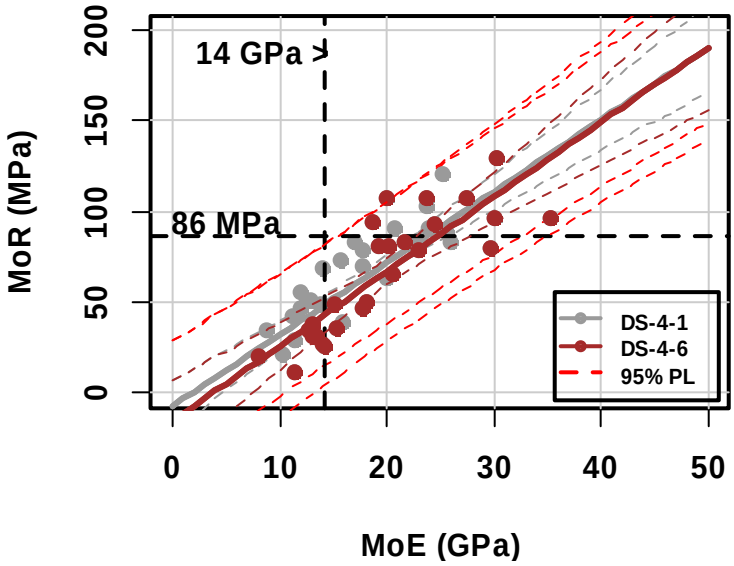

Figure 3-31: DS-4-1 MoR-V-MoE data and regression model overlaid with DS-4-6 data and regression model

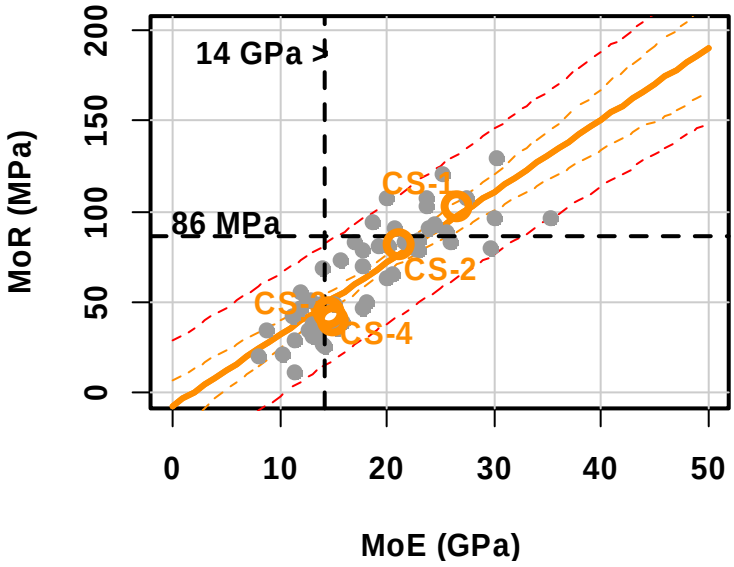

Figure 3-32: DS-4-1 overlaid with data as Figure 3-30

3.6 DS-6: CSIRO evaluation of 174 Australian Timbers

In Section 2.5.5 data was tabulated for two of the 174 species examined by CSIRO. DS-6 consists of data for 118 of those species that had both MoE and MoR data. A scatter plot, and regression, of DS-6 is shown in Figure 3-33 and a statistical summary in Table 3-16. Although this data have been public for over four decades no evidence was found of it being published in both graphical form and in comparison with girder MoR-v-MoE data. These data are representative of all species both hardwoods and softwoods. There is no apparent discontinuity between the softwood data and the hardwood data. The variations in both MoE and MoR for the complete dataset are about 25% and 29% respectively. In addition the coefficient of determination is about 0.74 and the slope, of about 7.1 MPa/GPa, has a tolerance of about 11%. No discussion was found that identified any reason for an apparent relationship between MoR and MoE of excised clear samples as might be inferred from Figure 3-33.

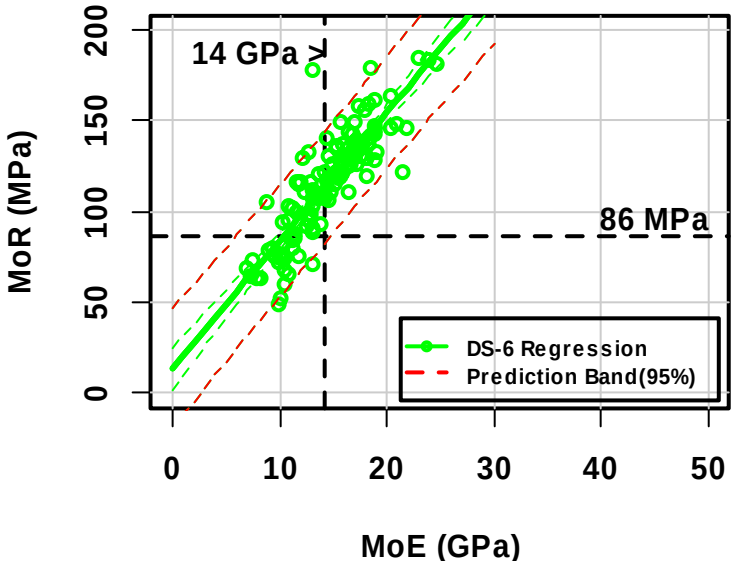

Figure 3-33: MoR-v-MoE regression analysis of DS-6

Table 3-16: Statistical summary of DS-6, CSIRO evaluation of 174 Australian timbers

Parameter	Value
Mean MoE	14.5 GPa
CV of MoE	25%
Mean MoR	111.8 MPa
CV of MoR	29%
Regression Model, 95% confidence	MoR = 13.6 + (7.1 ± 0.77) x MoE
Coefficient of Determination, r^2, MoR-v-MoE	0.74
Number of Species sampled	118

The DS-6 prediction bands (95% confidence level) but without the data points, are shown in Figure 3-34. The regression line through the data (solid green line) has a slope of about 7 $MPaGPa^{-1}$; about 0.007. The tolerance of this slope is about 11% which means that it is well defined. The 95% confidence level prediction band limits (dotted red lines) are spaced about 25 MPa either side of the regression line. Within this band are found 95% of species MoR-v-MoE vectors. Therefore if the MoE is measured for a clear of any species its MoR is expected to be found in the range ± 25 MPa of the regression line.

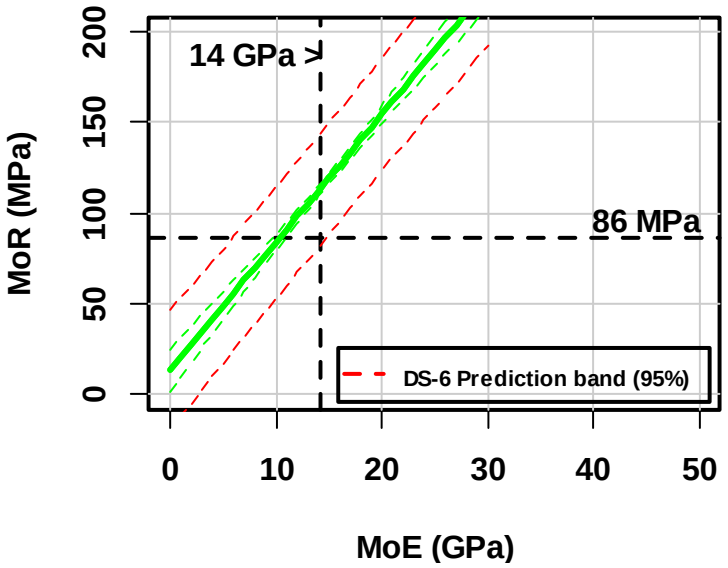

Figure 3-34: DS-A regression and prediction limits

3.7 Summary of MoR Regression Models

In sections 3.1 to 3.6, data has been analysed that relates MoR to MoE, age and condition status. These model relationships are summarised in the following two tables. The regression models for full sized girders are summarized in **Error! Reference source not found.** and those models for excised clear samples in Table 3-18.

Table 3-17: Summary of regression models used to predict girder MoR

Equation[3]	Table/Figure	Dataset	Regression model[1]	Coefficient of determination, r^2
3.1	3-1	DS-1	MoR = 3.4 + (2.8 ± 0.55) x MoE	0.52
3.3	3-7	DS-2-8	MoR = 24.8 + (2.8 ± 0.59) x MoE	0.48
3.4	3-7	DS-2-8	MoR = 77.7 - (0.48 ± 0.27) x Age	0.11
3.5	3-7	DS-2-8	MoR = 2.6 + (2.8 ± 0.56) x MoE – (0.35 ± 0.2) x Age	0.54
3.6	Fig 3-27	DS-4-1	MoR = -7.3 + (3.9 ± 0.75) x MoE	0.70
3.7	3-15	DS-4-6	MoR = -15.3 + (4.1 ± 1.1) x MoE	0.71
3.8	3-15	DS-4-6	MoR = 124.5 - (22.2 ± 6.5) x CS	0.67
3.9	3-15	DS-4-6	MoR = 48.2 - (2.7 ± 1.0) x MoE - (13.2 ± 5.5) x CS	0.86
3.10	Fig 3-45	DS-1 to DS-4 incl.	MoR = 10.2 + (3.1 ± 0.32) x MoE	0.58

Note 1: Tolerance specified for the coefficient of the prediction parameter is at a 95% confidence level
Note 2: Girder numbers 1 and 2 omitted as outliers
Note 3: Equation 3.2 is omitted from this table because it is a regression of MoE-v-Age, refer Table 3-4

Table 3-18: Summary of regression models used to predict MoR of excised clears

Equation	Table	Dataset	Regression model[1]	Coefficient of determination, r^2
3.11	3-16	DS-6	MoR = 13.6 + (7.1 ± 0.77) x MoE	0.74

Note 1: Tolerance specified for the coefficient of the prediction parameter is at a 95% confidence level

3.8 Pooled Data sets

The determination of a bridge safety index derives from the probability of failure as discussed in Section 2.5.2. It is set by the probability of failure of an individual girder. Previous research related to timber bridges has been restricted to the examination of girder performance from the design perspective (refer Section 2.5). Such historical research has established the minimum fifth percentile strengths for specified sets of all graded timbers. In contrast, in this current research, the temporal performance of individual timber bridge girders is considered. Because historically there have been no temporal data available representing individual girders it has been necessary to infer individual girder performance from the performance of many girders. Therefore, the objective of this section is to identify individual girder performance.

DS-1 to DS-6 data has been pooled to create a combined dataset that represents a wide range of in-service girders. By statistically grouping clusters of girder data, the behaviour of individual girders at different points in their life cycle has been inferred. Statistical techniques have also been used to identify both the mean performance of these clusters and the prediction bands in which new values are found. These prediction bands represent the performance of the measured girders for which data is available and of girders for a wide range of conditions. These statistically inferred prediction bands provide upper and lower limits that might be encountered for an individual girder.

In relation to timber bridge girders, two particular limit state failures are commonly considered. The first is excess deflection and the second is excess loading causing structural failure. To determine the first limit state an understanding of in-service girder MoE is required, while the second requires an understanding of MoR. The determinations set out, in this section, seek to enable identification of both the girders MoE and MoR behaviour. In following sections the determinations presented are:

1. The effect of MC on in-service girder MoR-v-MoE;
2. The range of in-service girder MoR prediction bands;
3. Prediction of in-service girder MoR from standardised data;
4. The range of in-service girder MoE; and
5. The MoR-v-MoE vector path of in-service Ironbark girder.

3.8.1 Determination 1: The effect of MC on in-service girder MoR-v-MoE vector

It was identified in Section 2.6.3 that the MC of a timber bridge girder can be relatively constant at about 10% early in the girder's life. In contrast, some aged girders had central MC as high as 42% (DS-2 girders, refer Section 2.7.3). However, it could not be identified in the available records whether these MC levels had any impact on the MoR-v-MoE relationship. The effect of MC was only treated independently; its effect firstly on MoE and secondly on MoR. In this section the effect of moisture on the combined MoR-v-MoE vector is examined.

The data for both Ironbark girders excised clears, from DS-3 and DS-5, are plotted in a MoR-v-MoE scatter plot in Figure 3-35. The Ironbark girder data (brown dots) represents the MoR-v-MoE mid-span relationship of in-service girders. The Ironbark excised clear data (green dots) represents samples removed from the low stressed regions of each girder. Although the data representing each clear is correlated to the data for a particular girder this correlation is not shown in Figure 3-35 but is set out in Determination 4 and 5.

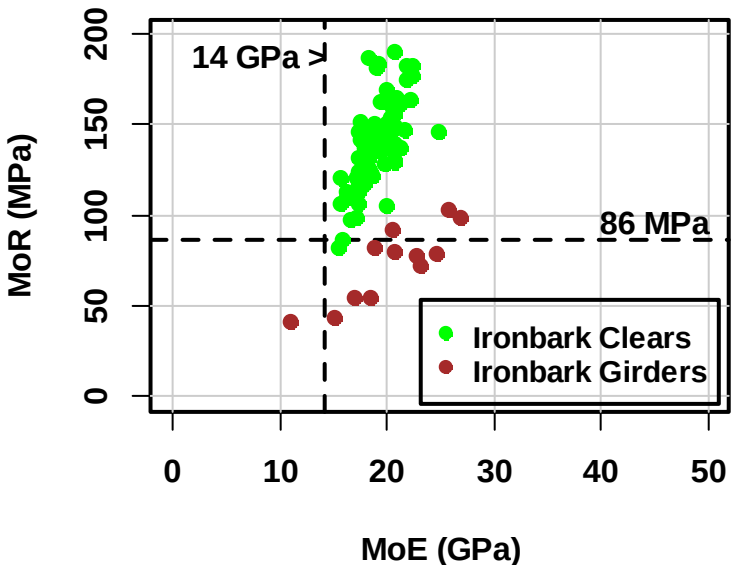

Figure 3-35: MoR-v-MoE scatter plot for Ironbark clears and Ironbark girders

The Ironbark clear and the Ironbark girder data shown in Figure 3-35 are repeated in Figure 3-36. Overlaid with that data are the MoR and MoE values of the same Ironbark girders but modified by Equation 2-13 and 2-14 respectively. To determine the effect of a decrease in MC of 13% the MoR values are increased by 1.67 and the MoE values by 1.29. The average girder moisture content was reported as being about 25%. It is unclear, from the available data, where on the girder this MC was measured and how it varied across the section. The data for clears are commonly specified for a MC of 12%. The data presented in Figure 3-36 represents how girder

MoR-v-MoE would vary if MoR and MoE varied only according to Equation 2-13 and 2-14 respectively. The girder MoR-v-MoE vectors are modified as if they initially represent girders at 25% MC (brown dots) but which are then dried to 12% MC (blue dots).

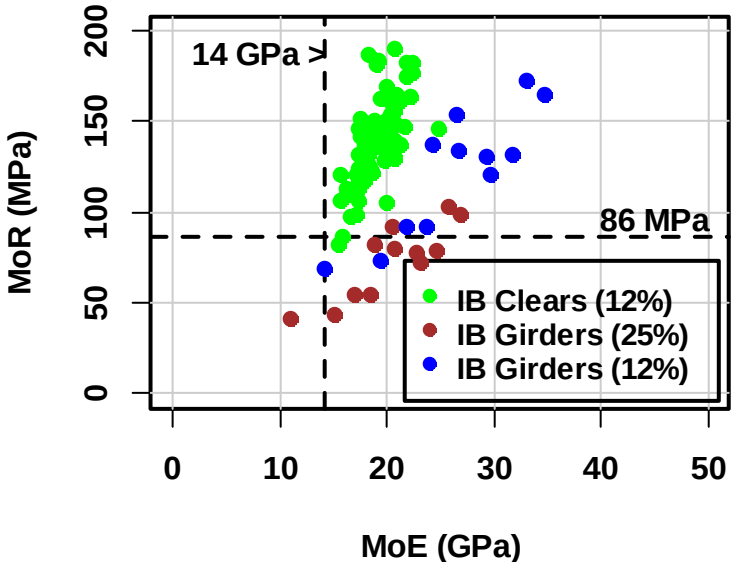

Figure 3-36: MoR-v-MoE scatter plot for Ironbark clears (12%) and Ironbark girders (25%) together with Ironbark girders at 12% MC

The same data from Figure 3-35 are again repeated in Figure 3-37 (green and brown dots). This time they are overlaid with the Ironbark girder values with unchanged MoE but MoR values multiplied by a factor of 2.0 (blue dots). The multiplying factor is chosen to align the girder MoR-v-MoE with the girder excised clear MoR-v-MoE. The mean species value for Ironbark is also shown (dark green dot). While multiplying factors of 2.0 and 1.0 respectively transform the girder data to the general vicinity of the clear data, the two sets (green dots and blue dots) do not have the same range. The actual transformation will have to be a more complex function than a simple multiplier, it will have to also contract the range. Further research is required to determine this transformation and a physical reason for it.

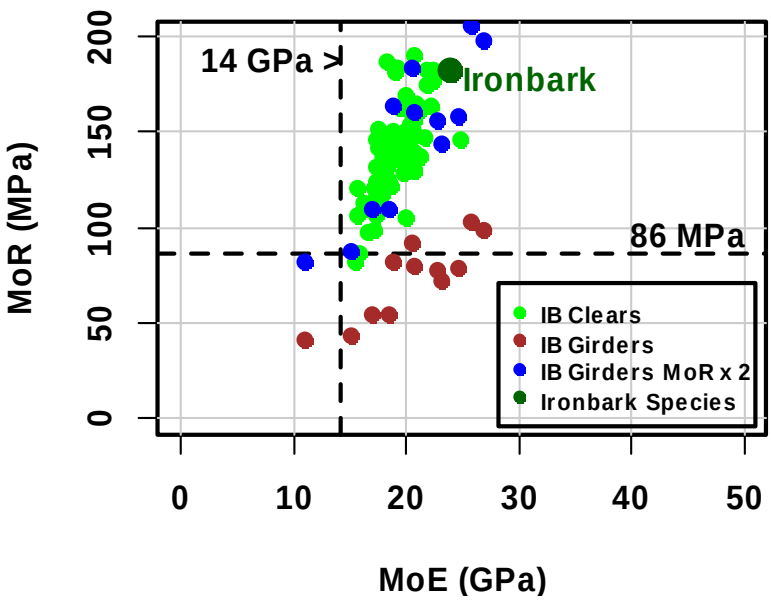

Figure 3-37: MoR-v-MoE scatter plot for Ironbark clears and Ironbark girders together with Ironbark girders of MoR x 2 and Ironbark species mean

Figure 3-38 shows the data for just one of the Ironbark girders, girder #14. The girder data (brown dot) and the excised clear data (green dots) are exactly as shown for girder #14 in Figure 3-35. All the data for the other Ironbark girders, shown in Figure 3-35 has been omitted. Overlaid (blue dot) are the MoR and MoE values for girder #14 increased according to a 13% decrease in MC as shown in Figure 3-36. Two other features are also shown. The first feature is the six sigma range of the mean values for the species of Ironbark. The mean MoR and the mean MoE, for several newly cut, 12% moisture content, Ironbark excised clears are expected to be within the dark green rectangle. The second feature is a scaled value of the girder MoR-v-MoE vector point. The girder has the measured MoR-v-MoE value of 99 MPa, 26.9 GPa (brown dot). This measured value is scaled by MoR x 1.3 and MoE x 0.7 to place it near the mean value for the excised clears (green dots) for girder #14. This resultant scaled value is represented by the orange dot. None of the plotted points lie within the six sigma range for new 12% Ironbark, neither for the girder nor for the excised clears. Nor could these plotted points be averaged to provide a mean value within the six sigma range.

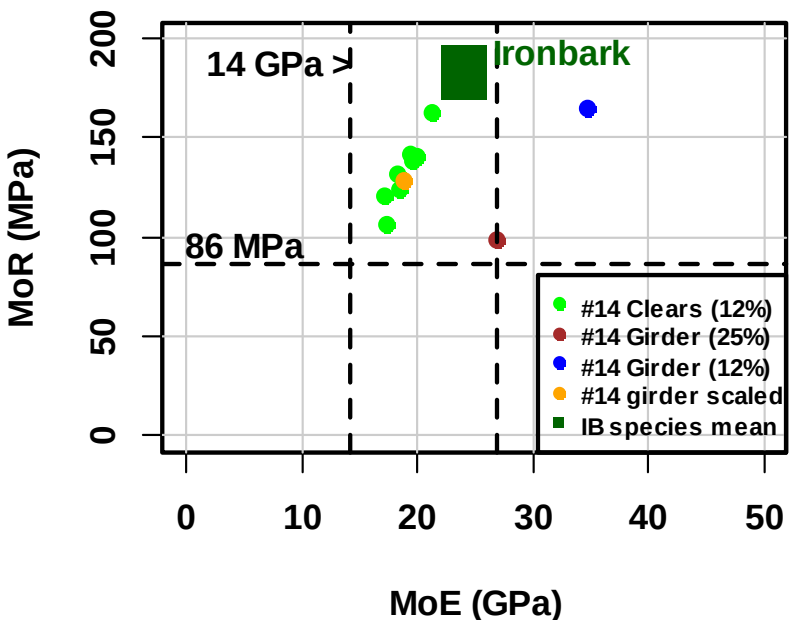

Figure 3-38: MoR-v-MoE scatter plots for girder #14

3.8.2 Determination 2: Range of in-service girder MoR prediction bands

In Section 3.1 to Section 3.7 the individual datasets DS-1 to DS-6 were examined. By pooling them, the range of in-service girder MoR-v-MoE is established. This total pool of data is shown in Figure 3-39. The data are comprised of two major clusters of data, the first representing the excised clears and the second the in-service girders. These data are used to create background prediction bands (refer Figure 3-46) in which any in-service girder or excised clear is expected to be found at a 95% level of confidence. The steps to arriving at this combined background are shown in the preceding figures as detailed in Table 3-19. In these preceding figures the data points, the regression lines and the prediction bands are presented. In Figure 3-46 only the derived regression lines and prediction bands are presented.

Table 3-19: Data comparisons to identify MoR prediction bands

Figure	Data	Description
3-39	DS-1 to DS-6	The complete set of data as a scatter plot
3-40	DS-5 cp DS-6	In service girder excised clears compared to clears representing new timber from multiple species
3-41	DS-5 AND DS-6	The combination of the two datasets to produce the prediction band for excised clears
3-42	DS-1 cp DS-2	Two independent sets of girders compared
3-43	DS-3 cp DS-2	Two independent sets of girders compared
3-44	DS-4 cp DS-2	Two independent sets of girders compared
3-45	DS-1 TO DS-4	The combination of the four girder datasets to produce the girder prediction bands
3-46	DS-1 to DS-6	The derived prediction bands for the complete set of data (95% confidence level)
3-47	#14 DS-3 & DS-5	Girder #14 data overlaid on prediction bands (Figure 3-46)

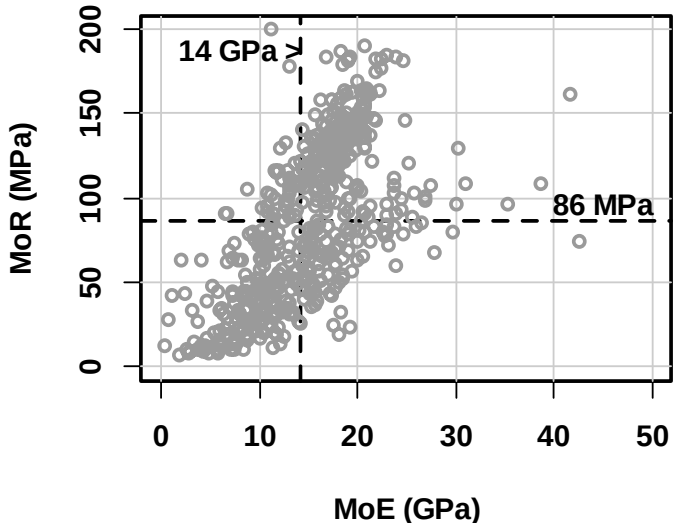

Figure 3-39: DS-1 to DS-6 complete set of MoR-v-MoE data

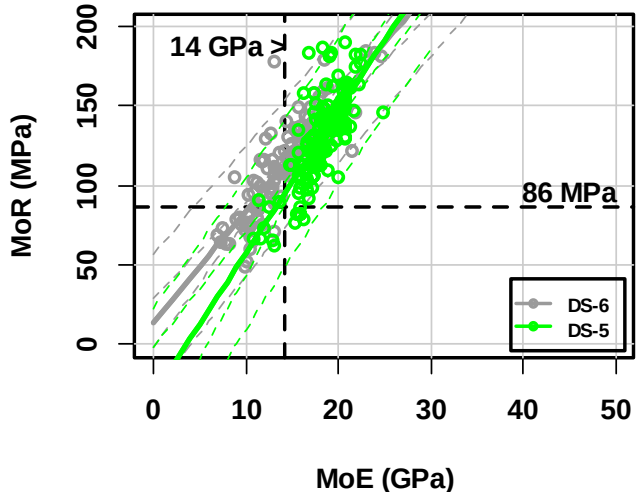

Figure 3-40: DS-5 MoR-v-MoE comparison to DS-6

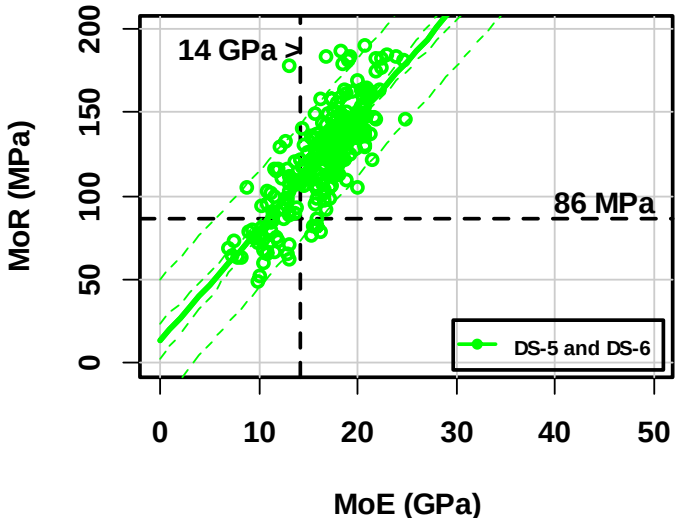

Figure 3-41: DS-5 and DS-6 combined MoR-v-MoE (excised clears)

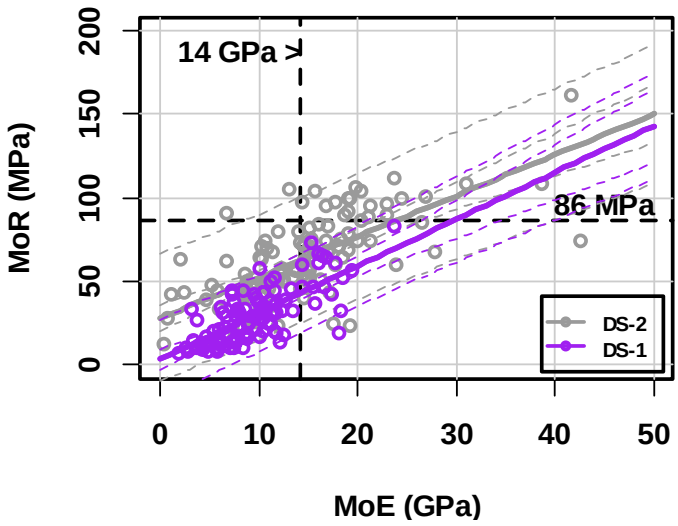

Figure 3-42: DS-1 MoR-v-MoE comparison to DS-2

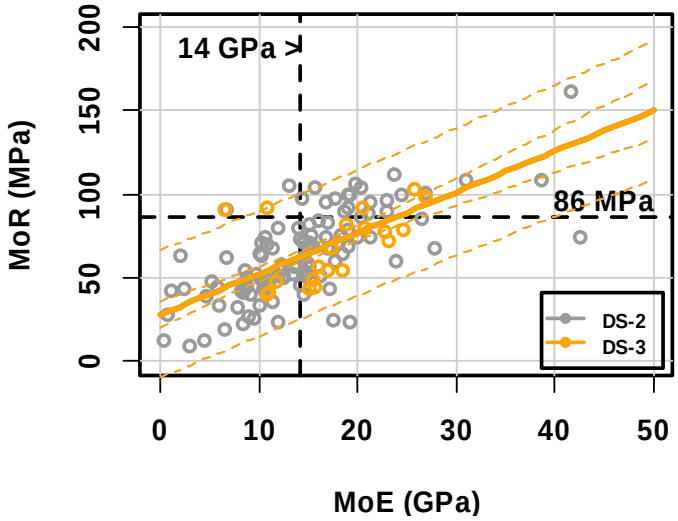

Figure 3-43: DS-3 MoR-v-MoE comparison to DS-2

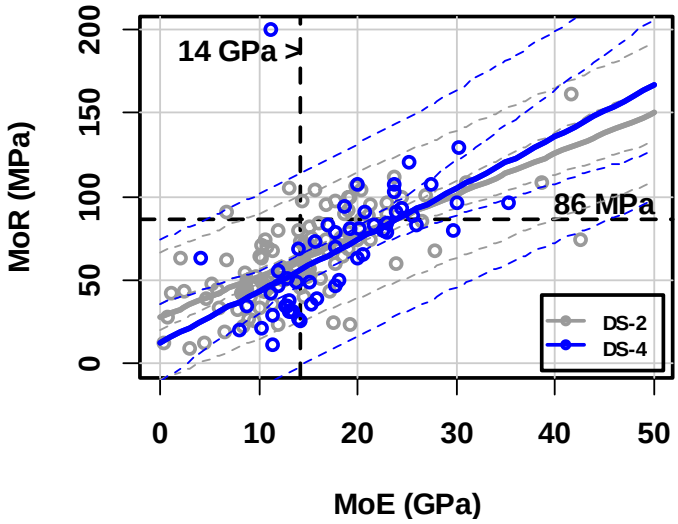

Figure 3-44: DS-4 MoR-v-MoE comparison to DS-2

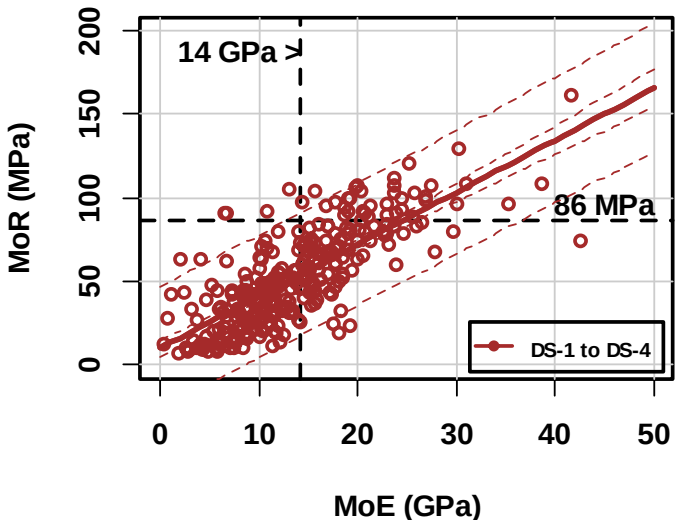

Figure 3-45: DS-1 to DS-4 combined MoR-v-MoE (girders)

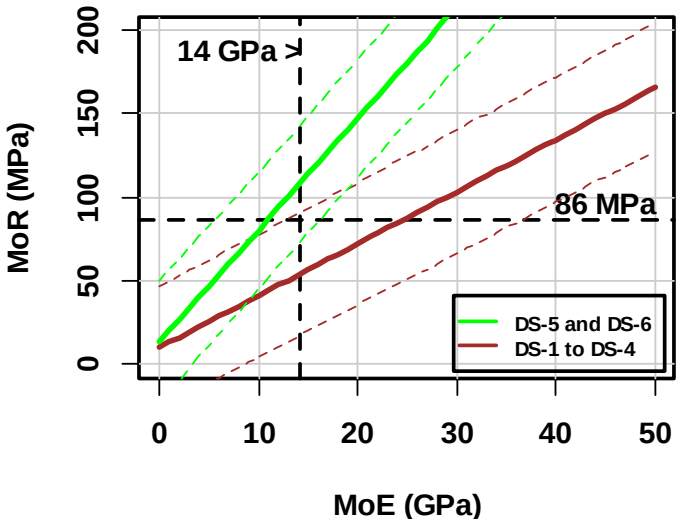

Figure 3-46: MoR-v-MoE in-service prediction bands

As one example, showing how girder data fit within these derived prediction bands, the data for girder #14 are shown in Figure 3-47. The girder MoR-v-MoE vector is at the centre of the girder prediction band close to the regression line (brown lines). The data representing the excised clears are within the clear prediction band and apart from one sample are all close to the regression line (green lines). All of these data representing girder #14 are in the correct prediction bands. Also shown is the six sigma range for the mean values of Ironbark. This range is also situated in the correct position within the clear prediction band.

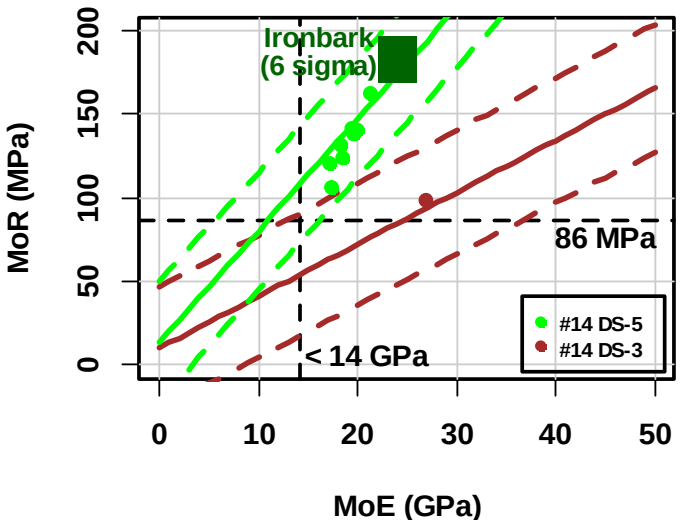

Figure 3-47: Girder #14 DS-3 and DS-5 overlaid on MoR-v-MoE in-service prediction bands

3.8.3 Determination 3: Prediction of in-service girder MoR from standardised data

Comparison with Design standards

In Section 3.10 the limits of the in-service girder MoR-v-MoE were determined using available experimental data. In this determination, available Standard data are compared with in-service data, refer Table 3-20. Three determinations are made, the first determination 3-A (Figure 3-48) shows that the timber species historically used for timber bridge construction are characterised both above the S2 limit states (MoE 14 GPa, MoR 86 MPa) and within the derived prediction bands for excised samples. In the second determination, 3-B (Figure 3-49), the standardised AS 2878 data (Table 2-7) is within the excised clear prediction bands and not within the in-service girder prediction bands. In the third determination, 3-C, there is some agreement between the AS 1720 data and the in-service girder data. However, this is only true over a limited range of MoE between about 10 GPa and 20 GPa. In particular the AS 1720 mean data provide a reasonable approximation to the girder regression and the AS 1720 fifth percentiles a reasonable fit to the lower girder prediction band. The agreement is not of a quality that it should be extended beyond the given data, because the slopes of the curves appear different.

Table 3-20: List of figures comparing Standard data with MoR-v-MoE in service prediction bands

Determination	Figure	Standard data source
3-A	3-48	CSIRO Hardwood species characterization, refer Table 2.2 and 2.3
3-B	3-49	AS 2878
3-C	3-50	AS 1720

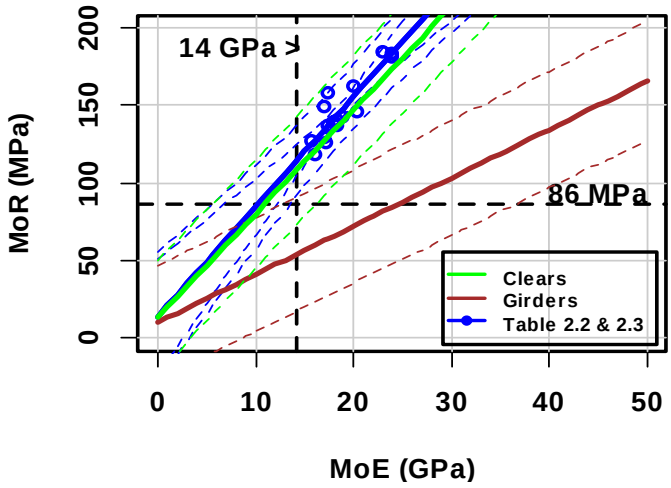

Figure 3-48: Hardwood species (refer Table 2.2 and Table 2.3) cp MoR-v-MoE in-service prediction bands

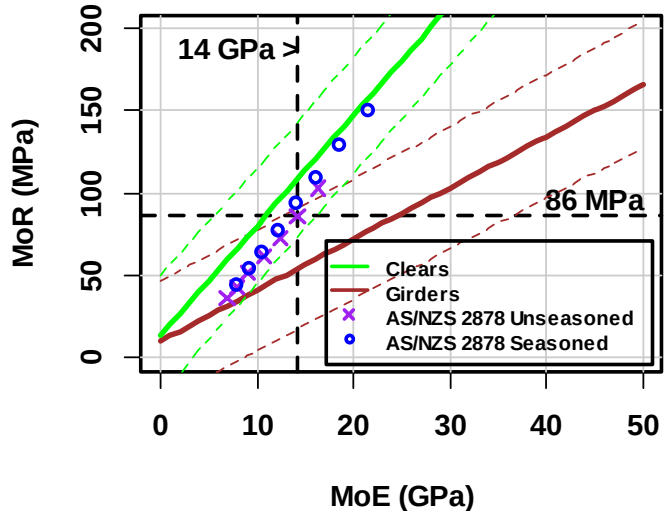

Figure 3-49: AS/NZS 2878 compared with MoR-v-MoE in-service prediction bands

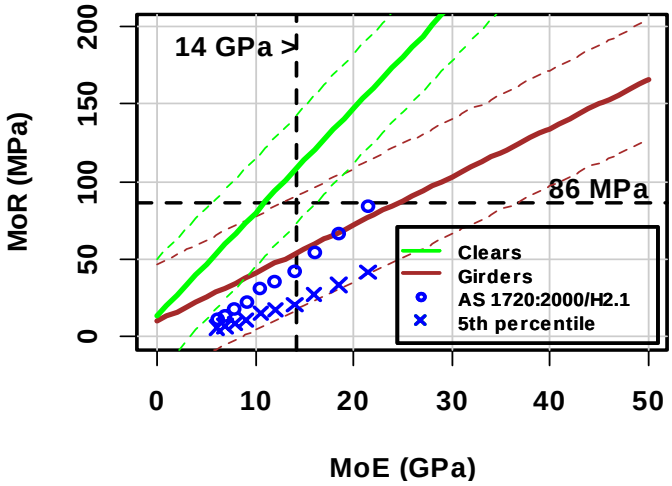

Figure 3-50: AS 1720.1:2000 Table H2.1 compared with MoR-v-MoE in-service prediction bands

Case 3-D: Comparison of DS-2-2 with prediction bands

The girders represented by the clustered subset DS-2-2 were not tested to destruction because the owners expected to use them in bridge repairs or new constructions. Both the species and MoR are unknown for this cluster of girders. The DS-2-2 measured MoE is compared with the in-service prediction bands (refer Figure 3-46) in Figure 3-51. The DS-2-2 range of MoE is from 13 GPa to 38 GPa and is shown as an orange polygon drawn symmetrically around the girder regression line. The mean MoE for Ironbark, 24 GPa, is also shown (green dot). Ironbark has the highest value of MoE for any species as characterised by CSIRO (refer Table 2-3) yet the in-service girder MoE maxima are 1.6 times the mean for Ironbark and 1.5 times the upper three sigma limit of the mean.

The point plotted in Figure 3-51, representing Ironbark, is a consistent MoR-v-MoE value. This is because the MoR data represents the same set of girders as does the MoE. Similar consistent MoR-v-MoE data cannot be plotted for individual excised clear specimens. The only available data are statistical means. In the literature review no data were found that related MoR to MoE for an individual new unstressed excised sample used for species characterisation. However, as shown in Figure 3-47 the clears excised from an aged Ironbark girder were all within the prediction band for excised clears (green lines).

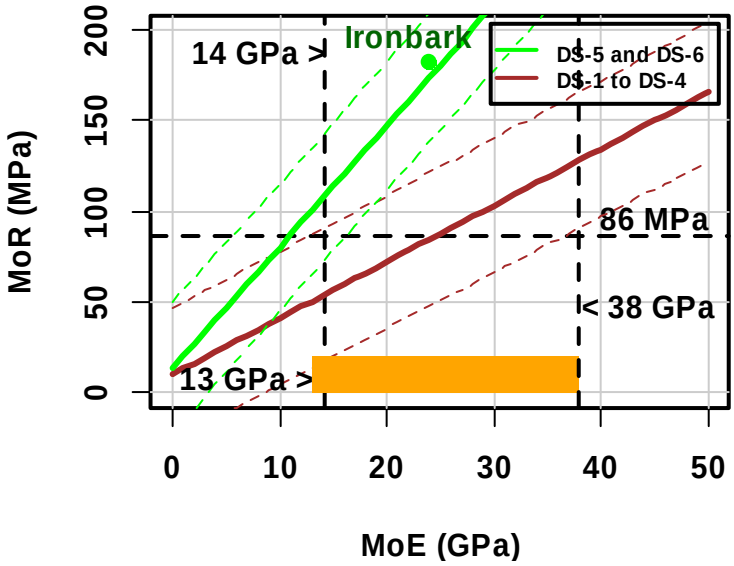

Figure 3-51: DS-2-2 range of MoE for girders aged less than two years, 13 GPa to 38 GPa (No MoR data available for these girders)

Case 3-E: Comparison of Ironbark six sigma range with prediction bands

The MoR and MoE Ironbark species data that is available are shown in Figure 3-52. These data are plotted onto the prediction bands (refer Figure 3-46) and the ordinate of this graph is

increased to accommodate the range of data that lie within the excised clears prediction band. However, much of the single sample MoR and MoE data are shown outside the same prediction band. This data can only be shown this way because no data was found that could be used that included information correlating MoR with MoE for individual samples. As an example from the characterised standard data, it is not known if any excised clear sample has both an MoR of about 130 MPa and an MoE of about 31 GPa. Nor is it known, from current characterised data, if the MoE of all samples with MoR of about 130 MPa is limited to the prediction range (light green dotted lines).

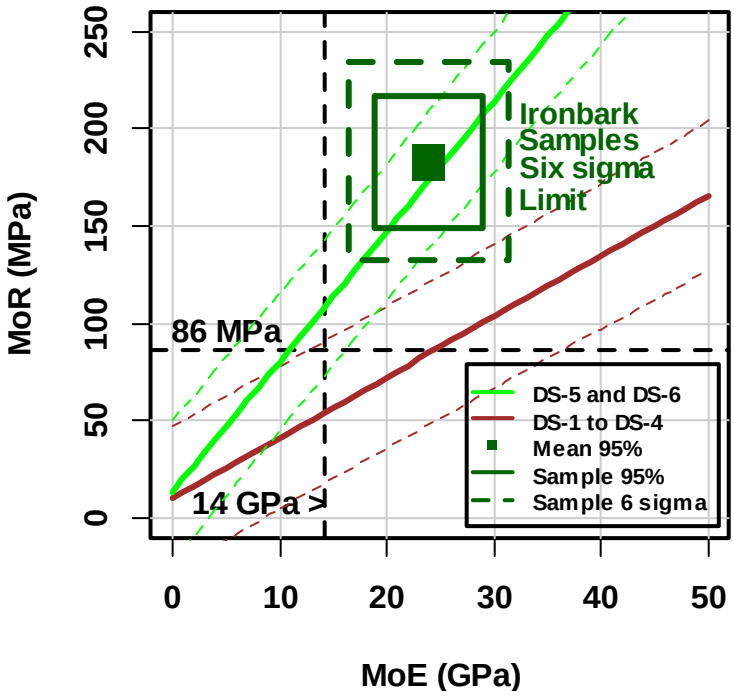

Figure 3-52: Range of characterised MoR and MoE for individual excised samples of Ironbark (*E. drepanophylla*)

3.8.4 Determination 4: The range of in-service girder MoE

In determination 1, it was identified that the DS-5 data is correlated with the DS-3 data. The DS-5 data are excised clear samples from low stressed regions of the girders represented by DS-3. The MoR-v-MoE vector point for each girder with MoE above 14 GPa is plotted in Figure 3-53 while the data for girders with MoE less than 14 GPa are omitted. Also plotted are the mean MoR-v-MoE of the excised clear for each girder (DS-5) and a trajectory arrow indicating the relationship between each girder and its related excised clears. These data represent only two points in the life of each girder. DS-5 represents the excised clear characterisation values and

DS-3 represents the girder MoR-v-MoE when removed from service. There are examples of girders that have MoE above their related excised clears, as well as girders that have MoE below their excised clears. In Figure 3-54 only those data are plotted for girders that have MoE at the end of life that is greater than their related excised clears. In the available DS-3 and DS-5 data (Section 2.5.3) there is no recorded reason, which can be used to explain, why girder MoE should be expected to be higher, or lower, than its related excised clear MoE .

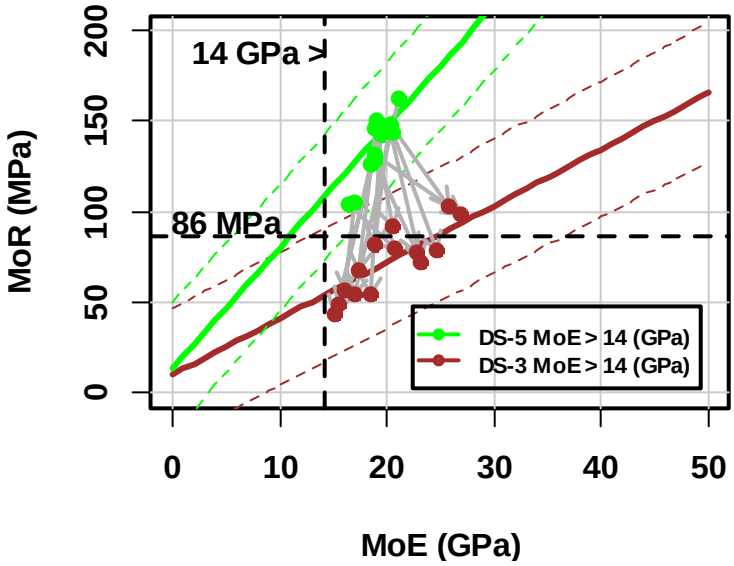

Figure 3-53: MoR-v-MoE trajectories for DS-3 girders with MoE > 14 GPa

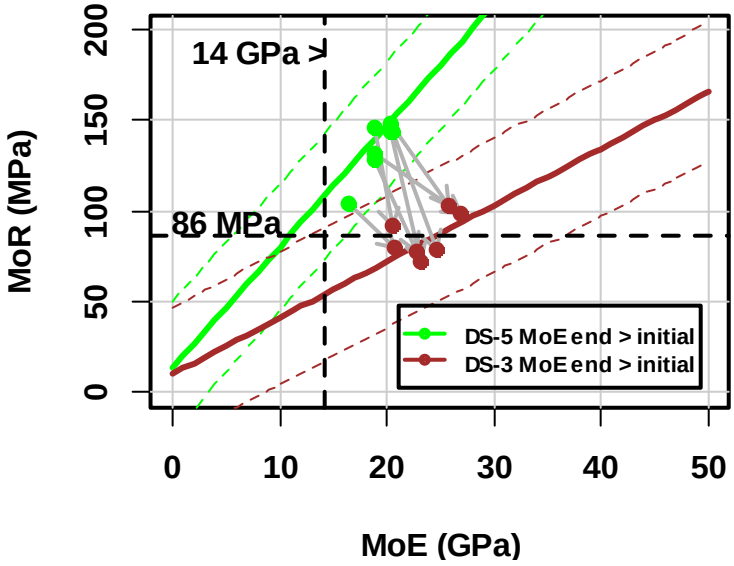

Figure 3-54: MoR-v-MoE trajectories for MoE final > MoE initial

The effective MoE of a girder is determined from the slope of the load-v-deflection curve. If either the MoE or the cross-section of the girder is varied then the slope of the curve responds. The girders, represented by the data shown in Figure 3-53, have either varied in MoE or in cross-section. Although it is not known whether these girders changed in cross-section or MoE, all of

the effective change can be attributed to a change in average cross-section. Such a change in cross-section can then be determined as an equivalent radial loss, as shown in Table 3-21. In both Case 1 and Case 2 the girder is specified as having an initial MoE of 20 GPa and a diameter of 0.45 m. In Case 1 the final measured end of life MoE is specified as 15 GPa. For a girder with MoE constant at 20 GPa, the radius would have to decrease by 17 mm to achieve the measured deflection achieved by a girder with 15GPa MoE and no change in diameter. In Case 2 the final measured end of life MoE is specified as 25 GPa. For a girder with MoE constant at 20 GPa, the radius would have to increase by 12 mm to achieve the measured deflection of a girder with 25 GPa MoE and no change in diameter. In Case 1 the effective radial loss could be identified as surface damage, although there is no available record of the level of damage for DS-3 girders. A comparison cannot, therefore, be made. In Case 2 there is a need for an increase in surface material and no evidence was found of this occurring. Since there is no evidence of additional surface material and the diameter does not increase it must be inferred that MoE must increase.

Table 3-21: Change in girder effective diameter for constant MoE

Case	Initial MoE	Measured final MoE	Effective Radial loss
1	20 GPa	15 GPa	-17 mm
2	20 GPa	25 GPa	+12 mm

3.8.5 Determination 5: The MoR-v-MoE vector path of in-service Ironbark girder

As identified in Section 3-5-2 DS-4 contained information about girder condition state, species and mode of bending failure. A summary of girder data clustered for common characteristics was presented in Table 3-12 and Figure 3-29. This data is now superimposed onto the prediction band limits (refer Figure 3-46) as shown in Figure 3-55. Each common cluster, or point, in Figure 3-55 represents an in-service girder at a different point in its service life. The particular features of the data shown in Figure 3-55 are:

- Excised clears
  This regression (green lines), introduced in Figure 3-41, represents the combined DS-5 samples of excised clears from 21 girders removed from service together with DS-6 samples of 174 timber species. The regression of excised clears represents the data obtained from un-degraded timber samples used for strength classification.
- In-service girders
  The in-service girder regression (brown lines), introduced in Figure 3-45, represents the combined DS-1 to DS-4 samples of 338 girders removed from service. This regression represents the data obtained by bending to failure aged timber bridge girders.

- Point 1

  This point represents the mean MoR-v-MoE characterisation vector of un-degraded Ironbark.

- Points 2,3,4 and 5

  These points represent in-service Ironbark girders (DS-4) of increasing degradation (refer Table 3-12).

- Point 2A

  This represents the MoR-v-MoE vector of girder DS-2 #24. This girder was aged 15 years with an MoR-v-MoE vector of 161 GPa, 42 MPa. The strengths and elasticities for this girder were over 3 standard deviations above the mean for the complete range of DS-1 to DS-4 data.

- Point 2B

  This point represented the mean of eight girders. They were the eight girders with MoE above 30 GPa from DS-2 and DS-4. This set of eight included the two girders #24 and #164.

- Point 2C

  This represents the MoR-v-MoE vector of girder DS-2 #164. This girder was aged 45 years with an MoR-v-MoE vector of 109 GPa, 31 MPa; these elasticities and strengths were about 2 standard deviations above the mean for the complete DS-1 to DS-4 data.

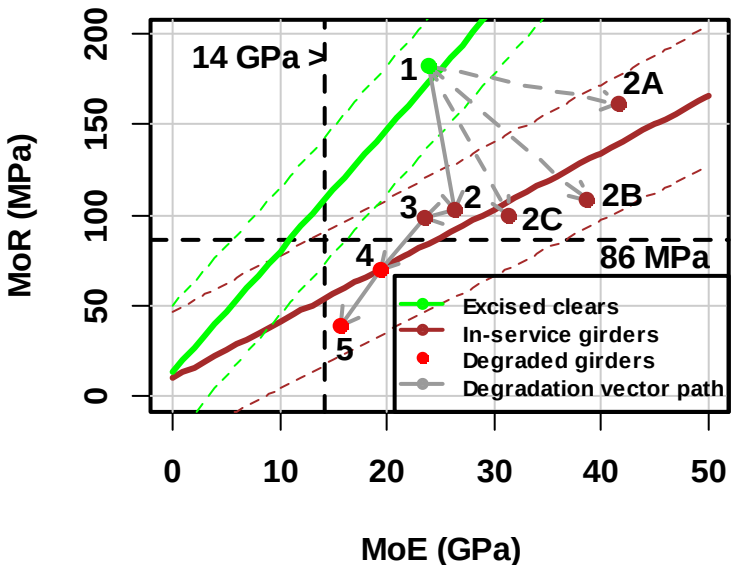

Figure 3-55: Degradation vector path overlaid on MoR-v-MoE regressions for DS-1 to DS-4 and DS-5 & DS-6

3.8.6 Proposed design approaches

The objective of this analysis was to establish MoR-v-MoE relationships that can be used to enable SHM techniques to be applied to timber bridge girders. The outcome of this determination is that a girder MoR-v-MoE vector will vary in a characteristic manner throughout the service life of a girder. One example of such variance is shown in Figure 3-56 where three zones are identified; a red zone, an orange zone and a green zone. The MoR-v-MoE vector will always be, to a 95% level of confidence, within the prediction bands (brown dotted lines). The red zone represents girders with MoE and MoR that to a 95% confidence limit are below the S2 limit states. The green zone represents girders that to a 95% confidence limit are above the S2 limit states. In between is the orange zone where some girders are above the S2 limit state and some below.

Girders that have MoEs in the red zone are most likely degraded for more than one reason and should be replaced or strengthened. Girders with MoEs in the green zone are most likely to be stronger than their design specification and in which case they can be left in service without reservation. In the orange zone each girder needs to be assessed to determine whether it is stronger than its related design specification. As one example if a girder is measured to have a mean MoE of 30GPa then its mean MoR will be about 100 MPa (refer Figure 3-56). The standard error of its distribution will be about 20 MPa which is equivalent to a 20% CV. As will be shown later in the applications developed in Chapter 5, a girder stress distribution can be computed from a measured load distribution and compared with the MoR distribution just determined. The outcomes of this comparison are firstly the probability of girder failure and secondly the girder safety index.

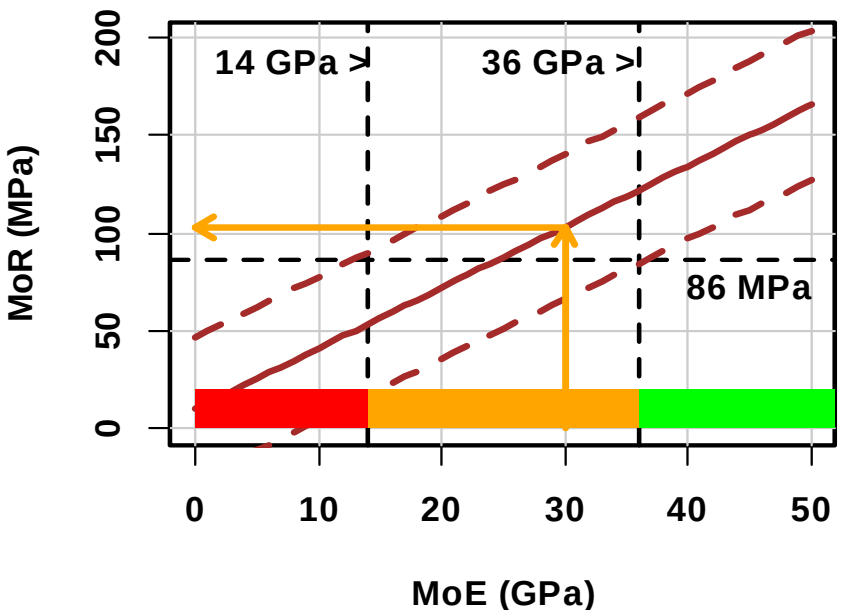

Figure 3-56: Example of using MoE to predict MoR

The example data shown in Figure 3-56 is derived from test data representing over 300 girders removed from service. The prediction bands shown in this example, therefore, represent the variation in over 300 girders. It is likely that the strength variation of one girder can be predicted to a higher level of accuracy but such an improved prediction capacity will require further research to establish the level of accuracy that can be achieved. The design approaches developed in this research, as proof of principle, will utilise the data as shown in Figure 3-56 and are described in Chapter 5.

3.8.7 Summary

In order to apply SHM techniques to the measurement of timber bridges a parameter needs to be identified that can be measured and used to establish a probability of limit state failure. It has been established in this chapter that the measurement of parameters such as MoE and condition state are indicative of girder structural health. In particular two characteristic relationships are determined that relate MoR to MoE. The first relates the MoR-v-MoE of excised samples and the second the MoR-v-MoE of in-service girders. It is not apparent from the available data why these two relationships are different, the first with a proportional constant between MoR and MoE of about 7 and the second with a proportional constant of about 3. In Determination 1 it is established that MC cannot alone account for the different nature of these two relationships.

Then in Determination 2 the relationship is established for over 300 in-service girders. The limits of this relationship are established by statistically derived prediction bands and model regression equations. A comparison is then made, in Determination 3, of these relationships with standard

data to establish if current Australian Standard data could be used to predict girder failure using SHM techniques. The outcome is that Standard data with a sufficiently high level of confidence is not currently available. It is also established that there is a need to determine if characterised sample MoR-v-MoE can exist outside the derived prediction bands. If not then the range of characterised sample MoR-v-MoE should be limited to be within those bands.

Finally in Determination 4 and 5, MoE is identified as a parameter that can be used to predict how close an in-service girder might be to the end of its service life. In particular three zones are identified. One zone with MoE that is below 14 GPa, a second zone between 14 GPa and 36 GPa and a third zone above 36 GPa. The maintenance requirement for each zone is different. A girder found to be in the first zone should be strengthened or replaced. A girder in the second zone analysed to ensure its safety factor is acceptable and girders in the third zone will normally be above standard design requirements.