Miles Canyon Bridge
Level II Inspection Report
Whitehorse, Yukon

Presented to

Nick Rodger, Peng
Senior Project Manager
Infrastructure Development
Department of Community Services
Yukon Government

Prepared by

Dan Tingley Ph.D., P. Eng. (Yukon)

Wood Research and Development

PO Box 70 • 10476 Sunnyside Rd SE • Jefferson, OR 97532 • USA
Tel: 541-752-0188 • Fax: 503-385-8493 • www.woodrandd.com

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EXECUTIVE SUMMARY

Wood Research and Development conducted a Level II Inspection of the Miles Canyon Bridge between 28 October and 31 October 2015. A partial inspection was conducted in mid-October which revealed significant areas of concern. The bridge was closed at that time and this complete inspection was scheduled. The inspection included a visual survey of all of the main structural elements as well as the use of EPHOD™ stress wave time (SWT) analysis and other non-destructive testing methods to identify areas of internal decay in the timber elements. The non-destructive test results were supplemented by several targeted assay samples which were tested in the laboratory to identify the species, moisture content, and specific gravity. The number of assay samples recovered was increased while inspection work on site was being completed to further verify the results of the non-destructive testing. The assay sample testing correlated well with the results of the non-destructive testing.

Miles Canyon Bridge is a pedestrian bridge crossing the Yukon River south of Whitehorse. It is a suspension bridge supported on two steel cables with a main span of approximately 130 feet. The cables are supported by a timber-framed tower on each bank and are anchored into the soil at each end. The main suspension cables support vertical suspender cables and 6 by 6 inch timber crossbeams at approximately six feet on center, which in turn support 2 by 6 inch longitudinal stringers and a transverse 2x10 deck.

The suspension cables were inspected visually and found to be in fair condition, with minimal rust and no section loss. The towers at each abutment were found to be in fair condition with only isolated areas of moderate decay which are not currently threatening the structural integrity of the bridge. The deck and railing are in fair condition overall, with only isolated instances of broken or decayed elements. The crossbeams and stringers were found to have significant areas of advanced decay throughout the bridge. The most problematic area is the connection between the crossbeams and the vertical tension hanger cables. At these connections, the cable loops through a U-bolt which passes vertically through the beam; these vertical U-bolts have provided a path for water to enter the center of the beam and have caused accelerated decay. This is a primary support point in the structure, and failure of one of these connections could potentially cause the failure of the adjacent crossbeams and the stringers above. The application of heavy solids paint (over 29%) to the bridge timber elements has accelerated the decay in the larger members by trapping the moisture in the element and preventing evaporation of the water from the wood. This has resulted in higher average moisture contents in the structural timber elements and subsequently led to accelerated decay. The Stress Wave Time results and assay samples show high levels of decay around this connection in 11 of the 21 crossbeams. It is recommended that the decayed crossbeams and stringers be replaced in the near term to prevent a failure, which would add to the cost of repairs. Until these repairs are completed, the bridge should remain closed.

If the repairs recommended in this report are completed, the bridge can be restored to safely carry pedestrian traffic. This report recommends a strategy for completing these repairs in a timely, cost-effective fashion. In addition, several upgrades are recommended which will limit future decay and greatly improve the longevity of the structure.

1.0 INTRODUCTION

The inspection of Miles Canyon Bridge was completed by Wood Research and Development (WRD) Level II Certified Inspection Technicians on 28-31 October 2015. The objectives of the investigation were to establish the general condition of the primary structural elements, and to assess what upgrades and refurbishments may be required to achieve a pedestrian load rating 5kPa live load. The investigation included visual inspection, nondestructive testing, and assay sampling. An assortment of instruments were utilized to complete the nondestructive tests, including; EPHOD™ Stress Wave Technology, distameter, psychrometer, moisture meter and digital camera. Assay sampling included the use of an increment borer to collect core samples, which were lab-tested to determine specific gravity and moisture content.

The inspection was performed by three Level II Certified Inspection Technicians from WRD with assistance from two employees of the Transportation Maintenance branch of the Yukon Government, one of whom recently completed a Level I Timber Bridge Inspection course.

This inspection report has been prepared by Dan Tingley Ph.D., P.Eng. (Canada), P.Eng., MIEAust, CPEng, RPEQ, senior engineer and wood technologist for WRD and Robert Keller, P.E., (Oregon) Project Engineer for WRD.

2.0 WOOD DETERIORATION AND INSPECTION TECHNIQUES

2.1 Wood Deterioration

Wood deteriorates for numerous reasons and as deterioration implies, adversely affects the wood properties. The two primary causes of deterioration in wood are biotic (living) agents and physical (nonliving) agents. In many cases, the agents that first alter the wood also provide the conditions for other agents to attack (e.g. insects bring woodpeckers). The effectiveness of an inspection of deteriorated wood depends upon the inspector's knowledge of the agents of deterioration. A timber bridge inspector must be well-trained in all aspects of wood technology. A solid understanding of the way wood transfers stresses through different directions (it is anisotropic) and its subsequent response to degradation, both biotic and physical, is essential for accurately assessing wood deterioration. Deterioration is most commonly caused by decay causing fungi, and so decay causing fungi will be the focus of this discussion. For further information on other forms of degradation such as ferric embrittlement which leads to loss of connector capacity and moisture retention induced degradation due to application on heavy dimension timbers (over 50 mm minimum dimension) of heavy solids content paints and coatings (greater than 30% solids) see Appendix H. Also for information about ultraviolet degradation of section contact the undersigned and see articles in the Appendix.

For further information and background materials on these topics see the following articles/publications authored or co-authored by Dan Tingley: Appendix E (Segment of text published by McGraw Hill, written by Tingley on Restoration of Structures), Appendix F for paper written and presented by Dan Tingley called Advanced Inspection, Non Destructive Testing, Remote Monitoring and Refurbishment Techniques for Timber Bridges by D. Tingley (3rd Australian Small Bridges Conference 2009) and Appendix G for paper co-authored by Dan Tingley and Stephen Richards (then Assets Manager Mitchell Shire) called Investigation of Australian Short and Medium Span Timber Bridges by Stephen Richards (3rd Australian Small Bridges Conference 2009)

2.1.1 Wood Deterioration Due to Biotic Agents

Biotic organisms that attack wood include bacteria, fungi, insects, and marine borers. As living organisms, they require certain conditions for survival such as moisture, oxygen, temperature, and food, the latter usually being the wood. When the basic living conditions are provided, biotic agents of wood deterioration will freely proliferate. But if any one condition is removed, the wood is safe from further biotic attack.

Fungi are the most common form of wood deterioration. When exposed to favorable conditions, most types of wood become an attractive food source for a variety of decay-producing fungi. The fungi require moderate temperature, oxygen, and a moisture content of approximately 20% or greater (oven dry basis) to become active. Decay in wood caused by fungal growth progresses most rapidly at temperatures between 5C (40F) and 50C (120F). Outside this range, fungal activity slows considerably and ceases when the temperature drops to 2C (35F) or below or rises to 38C (100F) and above. Wood can be too wet for decay also. If the wood is water-soaked (saturated), the supply of oxygen may be inadequate to support development of typical decay fungi[1]. Thus, wood will not decay, and decay already present from prior infection will not progress if appropriate conditions are not met.

Decay fungi may be generally classified into two categories by the appearance on the wood surface:

1. Brown rot: Appears darker and can crack across the grain. Brown rot fungi attack the cellulose in the wood fibers. The brown color is due to the remaining lignin (the binder which holds the cellulose structure together), which is not consumed by the fungi. The decayed wood tends to form into small cubic shaped sections, which is a sign of advanced decay.

[1] Forest Products Laboratory. 1999. Wood Handbook: Wood as an Engineering Material. U.S. Government Printing Office. Agric. Handbook. 72. Washington DC: U.S. Department of Agriculture; rev. 1999.

2. White rot: Appears lighter in color and does not crack across the grain until severely degraded. In contrast to brown rot, white rot fungi consume both the lignin and cellulose and leave the surface appearing generally intact, but with little or no significant mechanical strength. The surface of the decayed wood tends to have a "white" appearance. White rot impacts longitudinal shear resistance and is very common in cross heads in Tasmania which are often governed by applied longitudinal shear. The wood often appears cubed and cracked across ray or longitudinal cell lines.

Dry rot is a common type of decay fungi in which the wood becomes brown and crumbly in an apparent dry condition. However, dry rot is a misnomer because the wood must have some moisture in it to decay, although it may become dry later. A few fungi have water-conducting strands (hyphae) which are capable of carrying water, usually from the soil, into buildings or wood piles where they moisten and rot wood that would otherwise be dry.

Interior decay damage can occur even when some precaution has been taken. Surface-treated wood material can form cracks, which extend beyond the treated surface into untreated core material. Water can also get into the core of "protected" wood by the fungi hyphae. In either case, water enters the core material and provides adequate conditions for decay fungi to live.

Surface decay can be identified by both visual and probing techniques. Decayed wood tends to be very rough in texture with closely spaced cracks and grooves. With a pocketknife or flat-head screwdriver, decayed wood can easily be penetrated and partially removed. These techniques are only suitable for identifying possible surface decay. The depth of the damage may be determined by taking core samples.

2.1.2 Effects of Fungal Decay of the Properties of Wood

1. The primary effects of fungi attack on wood can be characterized by the following points[2]:
2. Change of color
3. Change of odor
4. Decreased weight
5. Decreased strength
6. Decreased stiffness
7. Increased hygroscopicity (easier absorption of water)
8. Increased combustibility
9. Increased susceptibility to insect attack

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[2] Bodig, J., Jayne, B.A. Mechanics of Wood and Wood Composites. Krieger Publishing Co. Florida, 1993. pp. 586-589.

The incipient stages of fungi attack are characterized by a change of color and perhaps a change in the odor and may not be detected by changes in hardness or by surface tests. This stage may be very difficult to detect visually. Decay may reduce the mechanical properties by 10 percent before any significant weight reduction is noticed. When weight loss is between 5 and 10 percent, the reduction in mechanical properties may be reduced 20 to 80 percent[3]. Usually when decay is discovered by visual inspection, the damage has already been done.

Advanced stages of fungi attack reduce the specific gravity (weight) which decreases nearly every other mechanical property, including strength and is indicated by soft, punky, or crumbly wood. This factor is one of the primary misunderstandings by engineers that have not been trained in wood technology practices. A very common method of checking the quality of a timber pile is to core with a drill bit, to establish the amount of piping or cavities. No attention is paid to the loss of Specific Gravity (SG) of the outer ring of apparently sound wood (annulus). The test involves assessing the amount of piping or coring. Without a clear understanding of the quality of the outer ring of wood which can be obtained utilizing Stress Wave Timing and core recovery and testing, there is no way to properly assess the ability of the timber pile to continue to resist vertical axial loads and vertical axial loads combined with lateral forces (e.g. water flow, wind or impact (vehicular traffic). Simple piping estimates gained by drilling a hole and inserting a feeler to measure the thickness of the annulus, to access section loss and not annulus quality, often leave the bridge substructure open to excessive deflections and lateral deformations. See Figure 2-1 for photograph of annulus, where a log is cavitated with an annulus that is apparently sound. In this figure the decay is shown in area at 2 o'clock in the annulus and again in an area around a non-galvanized steel spike, which has allowed ferric degradation (Appendix H) and condensation hydration for decay to propagate. This area would not support axial compression loads and could initiate buckling failure or bending movement failures, if it were a pile in service.

[3] Forest Products Laboratory. Wood Handbook: Wood as an Engineering Material. U.S. Government

Figure 2-1: Cavitated/piped timber pile section where the area at about 2 o'clock is decayed in the annulus and SG is significantly reduced. Also note in this area an old spike where there is ferric degradation (see Section 2 for explanation of ferric degradation) and decay propagated by moisture content (MC) in the wood at the fastener caused by condensation off the spike.

Without a proper assessment of the outer ring of remaining timber pile, in a piped or cavitated timber pile, excessive super structure movement and deck movement and constant maintenance can occur. In addition, eventually greater localized failures in the piles will occur such as brooming/feathering of the pile. Other related failures are feathered tops, loose; cross brace, sash brace, waler and cross head connections from elongation of the connector holes. Finally, cracked and spread piles occur, laterally buckled piles and skewed piles. All of these characteristics will usually be associated with more pronounced lateral and vertical movement in bridge decks under lower and lower vehicle loads and speeds.

The typical approach to fix this problem is to band with heavy steel bands (hopefully galvanized) which do not protect against lateral inward movement of the outer annulus. When inward motion occurs, the bands become loose and slip downwards or out of place. Another problem with bands on piles that have very little piping or cavities is that the wood develops extremely high tensile stresses in the band due to outward moisture related expansion of the timber pile. Bands are simply not effective in providing continuous collective action against compression parallel to grain in the timber pile. In fact they are in many cases a detriment, as they hold moisture against the timber pile, allowing ferric degradation to occur in non-galvanized or poorly galvanized bands. Infilling and epoxy welding are generally accepted current state-of-the-art techniques used to replace section loss and reduced mechanical properties in timber piles. This remedial work should be followed by diffuser treatment to prevent further decay. See Appendix I for more details on how typical steel banding of timber piles is no longer a recommended practice for stabilization of timber piles that are degraded by decay, splits, cracking, broomed/feathered tops. See Figure 2-2 below for examples of ineffective steel banding in the Shackells Folly Bridge Moira Shire, Victoria, Australia.

(a)

(b)

(c)

Figure 2-2: Banded piles. See (a, b and c) above for band in Shackell's Folly bridge pile. If the wood expands due to the moisture, it will develop 1000 mpa in the band, well over its maximum stress capability. The thin gage banding intended to conform to the surface better still makes no contact and is loose! Totally ineffective for the intended purpose! Shackell's Folly Bridge is particularly interesting for the steel banding as the steel deck ballast tray is leaking and during the inspection there was ample water flowing onto the log piles causing them to swell, note the water on the cross head and log pile in (c) above. Further, this bridge has a steel ballast tray with vertical through connectors into the log girders similar to the vertical connectors into the cross bearers used in many timber bridges throughout Australia, which has led to decay in the centers of the cross bearers in many timber bridges.

In addition to timber piles, the effect of SG reduction in the annulus can have very detrimental effects in round log girder performance in bridges. See Figure 2-3 below where a round log girder in a log girder/log corbel bridge in Mitchell Shire (Costello's), Victoria, Australia had received a clean bill of health in a Level III report and was found to have a very high Stress Wave Time across the diameter (8-9,000 ms) by WRD inspectors. When the round log girder was prepared for application of the retrofit lamination by removing a slab from the bottom face, a branch butt end was removed with a chain saw. When the branch butt end was removed a very large cavity that ran 2/3 of the length of the girder was exposed with an annulus that had a SG reduced by nearly 35%. In addition the annulus thickness at the bottom in the high tensile bending stress zone was thinned to 15 mm due to the cavity growth. This girder barely held its own dead weight and fortunately was a side girder or it would have collapsed under low traffic loading of 1T or less. Other such girders were found in interior positions in the bridge. It is actual testimony to the need for utilization of advanced inspection methods when inspecting old timber bridges. Simple sounding bores at the end of the log girder in a single location will not properly allow assessment of the girder condition.

(a)

(b)

(c)

(d)

(e)

(f)

(g)

Figure 2-3: Cavitated timber bridge girder (a, b and c) in place at Costello's Bridge in Mitchell Shire. In addition to the large cavity running 2/3 the length of the log, the annulus SG is significantly reduced. Decay propagated by constant elevated MC in the wood around the metal vertical through bolts was caused by condensation on the through bolt shank and shelf water following the bolt channel into the core of the log. This constant hydration source provided fertile ground for the decay fungal colony to grow. Such excessively decayed, cavitated, reduced SG annulus log girders can be retrofitted utilizing keyways and new treated hardwood keys (d and e) and interior injection with fire proof polymers that slowly polymerize as they work their way into all the open cavities in the log. In addition, high strength fiber retrofits (f and g) are applied and diffused to prevent further decay from occurring in the annulus wood. Note the oak bungs in the side of the log plugging the hole where the diffusers are placed in the log annulus (f and g)

2.2 Detecting Deterioration

Methods for detecting wood deterioration can be divided into two categories: interior detection and exterior detection. In each case, specific methods or tools are appropriate for different types of damage and structures. There is no certain method that will accurately determine the condition of a given structure save sectioning and destructive testing which is not practical, but a combination of methods, tools, and a well-trained inspector can provide a reasonably accurate assessment of any deterioration.

2.2.1 Exterior Detection Methods

Exterior detection methods are easy to employ, because of easy access to exterior wood. The methods most commonly used include visual inspection, probing, and the pick test. These methods provide a basis for further interior detection methods to define the extent of damage.

2.2.1.1 *Visual Inspection*

Visual Inspection is the simplest method for locating wood decay on the outside (exterior) of the member and is suitable for detecting decay in more advanced stages. Visual inspection may not be an effective method to find early stages of decay when control is most effective. Some common indicators of decay, which can be found by a visual inspection, are listed below[4]:

1. Fruiting bodies: Some types of fungi produce fruiting bodies, which appear on the surface during the decay process. These types of indicators can easily be partially cleaned off by weathering. If fruiting bodies are observed on exterior wood members, the decay is most likely extensive. See Figure 2-4 below for a photograph of white fruiting bodies on log girders in Hamilton’s Road Bridge and Bridge 10 on the range road in Cassowary Coast Regional Council, Northern Queensland.

[4] Forest Products Laboratory. 1999. Wood Handbook: Wood as an Engineering Material. U.S. Government Printing Office. Agric. Handbook. 72. Washington DC: U.S. Department of Agriculture; rev. 1999.

(a)

(b)

(c)

Figure 2-4: Photograph of fruiting bodies (brown) on timber element in the tenth timber bridge found in the Kirrama range in CCRC (a). Note the shelf water from the leaking deck and outside the deck rain shadow, falling on the outer timber log girder. This coupled with the girder being on the south side of the bridge, the uphill side and closer to moisture all led to increased levels of moisture and elevated decay conditions. Proper drainage techniques for decks and protection for the elements are important. Proper steps should be taken to allow water to move quickly away from timber bridge structural elements. See photograph in (b). (particularly interesting hyphae at 9 o'clock on left side of photograph coming out of pore and into another adjacent pore) above, taken with a microscope, of a wood core taken from a timber bridge in Murrindindi Shire in Victoria, where fungal hyphae can be seen growing through the large pores in the core cross section. These hyphae tips secrete enzymes on the wood that break the cellular structure down as discussed earlier. When fruiting bodies are evident, hyphae are present in the wood at work breaking down the cellular structure and causing loss of structural capacity. Photograph in c) above shows a log girder in Hamilton's Road Bridge with a fruiting body due to similar conditions discussed above.

2. Sunken faces: Localized surface depressions are often a sign of decay near the surface. The wood may be intact or partially intact at the surface. See Figure 2-5 below for sunken faces found in typical timber bridge cross bearers which looked to be in great condition from the outside and had been installed just years early at great expense to the owner.

(a)

(b)

Figure 2-5: Photograph of sunken face on a cross bearer in a timber bridge (a). This is strong evidence of decay from excessive water migration into the cross bearer vertically along deck spike traces. Even though hot petroleum jelly (see black stain beneath the square edge) was used, it is obvious how ineffective this method is in preventing decay caused by such improper timber construction practices. Proper deck clips fastened to the cross bearers with horizontal through bolts are required (b). Advanced decay of the type witnessed in the timber cross bearers causes significant loss of structural capacity.

3. Staining or discoloration: A surface blemish can indicate if the wood member has been subjected to surface water contact.
4. Bulging of wood over the bearing points in beams. The decrease in specific gravity caused by fungi attack greatly diminishes the perpendicular-to-the-grain bearing capacity of wood. See Figure 2-6 below for photographs of bulging in cross bearers in a timber bridge.

Figure 2-6: A bulging face on a cross bearer in a timber bridge. This is strong evidence of decay from excessive water migration into the cross bearer along deck spike traces. Even though hot petroleum jelly (see black stain beneath the square edge) was used, it is obvious how ineffective this method is in preventing decay caused by such improper timber construction practices. Advanced decay of the type witnessed in the timber cross bearers causes significant loss of structural capacity. These cross bearers were only a few years in service but had their useful lifetime shortened by 80% or better because of poor connector installation practices.

5. Insect activity can be identified by holes, piles of wood powder, or frass.
6. Plant or moss growth indicates that a relatively high moisture level is present, a condition suitable for decay.

2.2.2 Interior Decay Detection

Due to lack of visible indicators, interior deterioration is difficult to detect. Several methods and tools exist for assessing interior damage they include moisture meters, core sampling, and stress wave timing.

2.2.2.1 *Moisture Meters*

Moisture meters can help identify wood at high moisture content internally. Typically up to 50 mm deep with a face MC meter and deeper up to 100 mm with a prong MC meter. High moisture content wood is a suspected area of potential decay. Untreated wood with moisture content higher than 20-25% indicates conditions suitable for decay.

2.2.2.2 *Core Samples*

Core samples, a type of assay sample, can be recovered from bridge structural timbers by using an increment borer, widely used by the forestry industry on living trees, can be used to obtain a core sample of a wood structural member. Core samples are a solid wood core that can be examined for evidence of decay, or void pockets. Core samples can show the limit and extent of deterioration and provide lab samples. Lab samples can be cultured to indicate the presence of decay fungi to provide an assessment of future risk, and also to analyze the specific gravity of the wood. Suspected decay areas, determined by moisture meters, visual inspection, or other methods, can be confirmed by coring.

2.2.2.3 *Stress Wave Propagation*

The use of stress wave measurement techniques to locate internal decay, have recently become popular because of their non-destructive nature. Stress wave analysis consists of sending a “compression” wave through a medium (wood) and measuring its velocity. The compression wave is introduced into the material by striking it with a hammer or blunt object. When the compression wave is initiated by the hammer, an accurate timer is started; when the sound reaches a second accelerometer, the timer is stopped. The distance between the “start” and “stop” accelerometer is measured. By measuring the distance (gage length) and time, the average velocity of the stress wave (compression wave) can be measured. The Modulus of Elasticity (MOE) and strength of the material is theoretically related to the velocity of the stress wave and the density. It is the measured velocity of the compression wave that indicates if decay is present or not.

If the sample has been subject to fungi decay, the specific gravity (weight) of the wood will decrease. The decrease in specific gravity causes a decrease in the velocity of the stress wave. Therefore, if decay is present the stress wave times are greater over a fixed distance (i.e. velocity decreases). The EPHOD™, Electronic Pulse Highlight and Outline Diagnostic, is a type of stress wave analysis procedure that was developed by WRD and is used in the inspection of timber bridges. See Appendix E and F for articles written by Tingley that contain more information on stress wave time analysis procedures.

2.3 Preventing Decay

There are many types of man-made chemical preservatives, which are used to prevent fungi attack. The best known is creosote, which is often used to preserve wood utility structures. Pentachlorophenol (Penta) and Copper Naphthenate (CN) are also used to treat bridge girders and other wood members where human exposure is limited. Problems such as the leaching of creosote into the water in rivers and its’ toxicity have caused its’ use to be slowly limited. Chromate copper arsenate

(CCA) is an effective wood preservative which is safe to handle for humans. The CCA treatment has been changed in recent years due to carcinogenic concerns over its use. A less carcinogenic substitute called *ACQ* (Alkaline Copper Quaternary) has taken its place around the world. The ACQ option adversely affects galvanized steel and much higher coatings of galvanizing on steel must be utilized to protect the steel connector from accelerated degradation from the ACQ.

Unfortunately, the treatment process for CCA and ACQ uses water as the transport mechanism which can cause splits and checks, especially for larger wood members. The effectiveness of CCA in the heartwood is in question due to generally poor penetration (often caused by tyloses, a naturally occurring occlusion of the cell cavities which prevents preservative travel through the wood cellular structure).

In summary, Penta and CN are the most commonly utilized bridge treatments methods. Both treatments should be applied with petroleum based solvent to prevent water related degradation that can occur during and after treatment. In addition, the CN should be borne in the solvents at high concentrations of at least 1%, not like typical hardware store diluted solutions such as .05%. The Penta can be borne in light or heavy solvents and should be treated to at least and uptake of 5 kg/mm (3) (or refusal). Finally, all bridge timbers should be treated after all holes are drilled and other forms of machining completed. This is one of the important reasons that oil based treatments are preferred and recommended by such agencies as the American Railway Engineering and Right of Way (AREMA) association. Further, the water based treatments cause the reactive agents to rest in the cell lumens and don't fix to the cell walls like Penta. Once exposed to ambient moisture in service the reactive agent mobilizes again and leaches back out and the wood loses it resistance to decay.

It is important that minimal machining occurs after the pressure treatment on the site. Also incising of the elements should be completed prior to treatment. Incising exposes more end grain and deeper side grain and thus improves uptake of preservative and better distribution of same. See photographs in the following sections of new bridge decks installed in timber/steel bridges with incised pressure treated glulam decks. If machining is required after treatment it should be followed by preservative with at least 1% CN field treatment, followed by end sealing with paraffin wax in solvent solution e.g. anchor seal to prevent end grain feathering.

Most chemical treatments require special pressure tanks to obtain the necessary penetration depth for effective decay resistance. Surface treating is not nearly as effective as pressure treatment because once the protective coating is broken by localized splits, checks, and moisture cracks an avenue for fungi attack is created. This creates a problem for post treating of treated wood elements in existing wood structures or components in-situ.

There are other forms of preservation of timber bridge elements such as fumigants and diffusers. Fumigants were developed to provide chemical protection without the requirement for pressure treatment and moisture content in-situ in the timber elements. This allowed structures already in the field to be treated. The first use of the technology was applied to wood utility poles and has developed from there to use in beams and columns in bridges. Diffusers act similarly to fumigants except that they begin to diffuse or deplete and vaporize through the wood when moisture contents exceed 20% whereas fumigants deplete and vaporize through the wood at all moisture contents. Fumigants are toxic to fungi as the vapour kills the fungi, whereas diffusers are naturally occurring basalts that neutralize the PH wave that is created by fungi hyphae secreting acidic enzymes that break down the wood. When the wood is not at or above 20% moisture content diffusers don't deplete and stay intact until needed when the MC again exceeds 20%. This means that they travel more effectively and are utilized with the wood reaches decay causing levels of moisture This moisture content (MC) triggered dissipation reduces the maintenance cost for maintaining diffusers versus fumigants which dissipate continuing and need constant recharging. Further, fumigants are often very toxic to humans whereas diffusers are not. This is an excellent feature of diffusers versus fumigants which deplete continuously regardless of the moisture content in the wood.

Boron is a type of fumigant and is very effective in controlling wood decay but is not as toxic to humans as the chemical preservatives noted above. Boron can be processed into rods, gels, and liquids, and inserted into predrilled holes in a structural wood member. The boron preservatives slowly dissolve over time and the natural moisture in the wood facilitates the migration of the boron through the pores.

A type of diffuser is a basalt diffuser with a borate compound that is fused into the basalt. These rods are sold under the trade name Decaystop™. This type of diffuser combines the positive decay toxicity with the PH wave neutralizing effects of the basalt (decay hypae secrete an acidic enzyme on the wood to break it down and become edible thus reducing the strength of the wood by reduced SG). Since the borate/basalt diffuser preservatives depend on moisture to transport the preservative, treatment with rods may not be appropriate in areas where construction detailing, flashing, or roof repair has been performed which eliminated the moisture supply for the fungi. Research has indicated that the moisture content of the wood needs to be greater than 40% for adequate boron transport through Douglas-fir heartwood[5]. Basalt/borate diffusers operate well at MC's over 22%. For exposed beams or structural members in contact with the ground, water or in close proximity to these

[5] Morrel, J. J. Sexton, C. M., Preston, A. F.1990. Effect of Moisture Content of Douglas-fir Heartwood on Longitudinal Diffusion of Boron from Fused Borate Rods. Forest Products Journal. 40(4):37-40.

conditions, Decaystop™ diffusers are ideal. Typical high quality Basalt/borate rod treatments are excellent ways to stop further decay by diffusion.

3.0 INSPECTION FINDINGS

3.1 Visual Inspection

Miles Canyon Bridge is a clear-span suspension bridge over Miles Canyon on the Yukon River. The bridge was originally constructed in the 1920s and has gone through several major repairs since then, most recently in approximately 2009 following an inspection that was conducted in late 2008.

The bridge has a 1-1/2 inch thick transverse timber plank deck supported on 2x6 inch timber stringers and 6x6 inch timber crossbeams. The crossbeams are supported by vertical suspender cables which hang off of the main suspension cables. The main suspension cables pass over timber towers at each end of the bridge and are anchored into concreted blocks buried below ground. See supplemental drawings of the bridge prepared with this report.

3.1.1 Deck

The deck is made up of nominal 2x10 timber planks. The planks are secured with vertical nails into the stringers below, and are coated with heavy, white paint. The deck planks are in fair condition; however, the paint has worn off along the center of the walking surface and many of the nails have worked loose over time. It is recommended that un-painted timbers be used for future deck replacements, and that the planks be secured using an alternative fastener system from below the deck; see Figure 3-2 below for an example.

Figure 3-1: Deck from above, looking toward Abutment 2. Note the worn paint down the center of the walking surface.

Figure 3-2: An example of a pedestrian walkway deck secured with angle clips and screws below the deck. This system avoids the use of vertical fasteners that penetrate the top surface of the deck or stringers.

3.1.2 Longitudinal Stringers

The deck is supported on longitudinal 2x6 inch stringers. There is one stringer running lengthwise under each edge of the deck, and a pair of stringers fastener laminated together running underneath the center of the deck. The typical stringers are continuous over four spans, and the joints are staggered (i.e. at each crossbeam, three of the four stringers are continuous and one is broken). The stringers are secured with a vertical carriage bolt though the stringer and the crossbeam below; the bolt heads are countersunk into the tops of the stringers to allow clearance for the deck planks above. The deck planks are nailed into the stringers from above. The stringers are coated with heavy white paint. The use of heavy solids paint is not recommended, because when water enters the timber (through fastener penetrations, shrinkage cracks, etc.) the paint prevents the timber from breathing and drying out. This elevated moisture content leads to accelerated decay.

The outside stringers were inspected visually and through use of non-destructive testing. The center pair of stringers could not be completely tested, because it could not be accessed for inspection over the water, and the stress wave timer cannot be used horizontally through the pair of sistered stringers. Deck planks were removed over Crossbeam 11 and Crossbeam 5, allowing visual inspection and non-destructive testing off the center stringers in those locations. SWT readings were taken vertically through each stringer at these locations. Additionally, core samples were taken from all four stringers in Span 3. These samples provide a comparison of the condition of the center stringers relative to the outer stringers.

The stringers show some visible signs of decay, but the paint prevents a thorough visual inspection. The non-destructive testing revealed further internal decay, especially near the top surface. This is a common problem were the deck is secured with vertical nails; the nails allow moisture to enter the stringer and cause accelerated decay.

Figure 3-3: Stringers passing over Crossbeam 1, seen from below. Note the vertical bolts through Stringers 1 (right) 3 and 4.

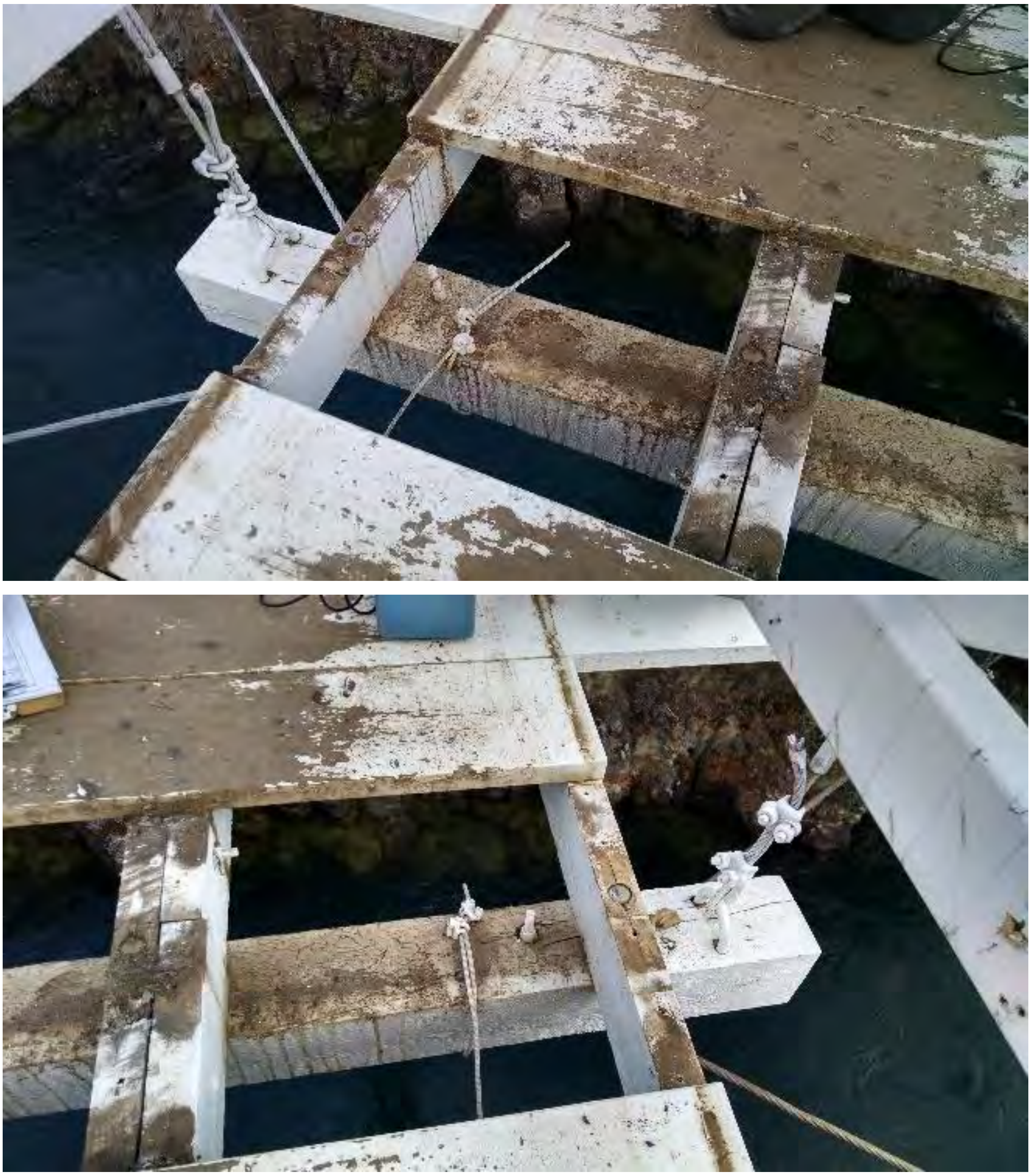

Figure 3-4: Deck planks were removed over Crossbeam 11, allowing visual inspection and non-destructive testing of the stringers in the ends of Spans 11 and 12. Further visible evidence of decay was seen when the top surfaces were exposed, especially near the vertical bolt in Stringer 2; this was confirmed by high SWT results in the same area.

Stringer 1 in Span 22 was found to have significant decay in and partial failure near the support at Crossbeam 21. This area also shows excessive deflection under load, which has led to the failure of the railing cross brace connections in Span 22.

Figure 3-5: Stringer 1 has failed over Crossbeam 21. This has caused excessive deflection in Span 22, and the railing cross braces have failed there.

In Span 3, the core samples show that the center stringers have higher moisture content and similar levels of decay compared to the outer stringers. SWT results of the stringers near Crossbeams 5 and 11 show somewhat elevated times in the center stringers compared to the outer stringers. Based on these results and the limited visual inspection, it is expected that the center stringers will have similar or slightly higher levels of decay compared to the outer stringers overall, but specific areas of decay cannot be identified.

3.1.3 Crossbeams

The deck and stringers are supported on transverse carry beams, which hang from the vertical tension hanger cables. These crossbeams are 6x6 inch solid sawn timbers, and are painted with heavy white paint.

Although most of the crossbeams appeared to be in fair condition based on visual inspection, the non-destructive testing revealed advanced decay in many of the beams, especially around the connection to the suspender cables. The beam-to-cable connection consists of an inverted U-bolt, with each leg passing vertically through the beam; the U-bolt is secured with a steel plate and two nuts below the beam. This is a very problematic connection, as it is directly exposed to weather from above, and the vertical penetration provides an easy path for moisture to enter the center of the member. Vertical connections such as this are even more detrimental when they are combined with the use of heavy solids paint, which prevents the timber from breathing and drying. Moisture that enters the beam along the fasteners is trapped, and the continually elevated moisture content causes accelerated decay. The SWT results and assay samples clearly show advanced decay directly around the U-bolt in many of the crossbeams.

These connections are primary load-bearing connections in the main structure. Failure of one of these connections would effectively double the span of the stringers above and significantly increase the loads on the adjacent crossbeams. If the adjacent crossbeams or stringers in the area are also weakened by decay, the increased stresses could cause them to fail as well. This situation creates the potential for a rapid progressive failure of a large portion of the primary structure.

Figure 3-6: Typical connection between the crossbeam and the suspender cable (at Crossbeam 1, Side 1). There are external signs of decay around the connection, including discoloration; peeling paint; and soft, punky wood. SWT results confirmed the severity of the decay. This beam had red zone readings (1-2000 microseconds per foot) within the first 16 inches, and yellow zone readings throughout.

The crossbeams also have vertical bolts connecting the stringers. These connections are at least sheltered from the weather, but they still lead to increased moisture content and accelerated decay in the timber.

All of the crossbeams were tested using non-destructive testing near the ends, where they could be accessed from the deck. The beams that could be reached from shore were also tested near the middle of their span; deck planks were removed above Crossbeams 5 and 11 to allow further testing of those beams as well. SWT results showed that these beams are in better condition near the center, where they are more sheltered from the weather and have fewer penetrations.

Figure 3-7: Crossbeam 20 was found to be severely decayed at End 2, and a portion broke off during SWT testing. This is a result of the combination of shrinkage cracks in the timber and severe decay around the vertical fastener.

3.1.4 Abutment Towers

The abutment towers are constructed from 12x12 inch solid-sawn timbers. Following an inspection in 2008, the towers were rebuilt. All of the posts; the caps; and the longitudinal sill beams at Abutment 2 were replaced with new, pressure-treated, unpainted timbers. The remainder of the timber elements, including the crosspieces on the A-frames and the cribbing at Abutment 1 were rebuilt using salvaged timber. When the towers were rebuilt, new concrete footings were also poured. These footings are in good condition. The timber elements in the towers were inspected both visually and using non-destructive testing, and were found to be in good condition overall.

The caps are covered with metal flashing on top and on the ends. Flashings like this are not recommended for timber construction, as the prevent air-flow around the timbers. Moisture that enters the timber due to condensation or through leaks in the flashing (such as where the cable saddles are screwed to the cap through the flashing) is trapped and causes accelerated decay. The cap at Abutment 2 showed signs of moderate decay and contained Yellow Zone SWT readings throughout.

The older, salvaged timbers are coated with heavy solids paint, and have a number of holes from old fasteners. As discussed above use of heavy paint is not recommended on large timber members, as it prevents the timber from breathing. In spite of this, most of the elements are in fair condition, with only isolated cases of moderate decay. This is partially in thanks to the minimal use of vertical fasteners used in the tower connections. In both towers, the element with the most decay is the subcap; in addition to being painted, the subcaps contain several vertical spikes penetrating the top surface.

At Abutment 1, there appears to be a vertical drift pin through the cribbing elements directly below each pile. It was anticipated that there would be more severe decay around these pins, so a denser grid of SWT readings were taken in these areas. While the times were slightly elevated around the pins, the results did not reveal severe decay at any of these connections.

Figure 3-8: Towers at Abutment 1 (left) and Abutment 2. The towers are in good condition overall, with moderate decay in some of the elements, especially the elements which were made from salvaged timber when the towers were rebuilt in approximately 2009.

Figure 3-9: The salvaged timbers are coated with heavy solids paint. This is not recommended for large timbers, as water will inevitably enter the timbers through, shrinkage cracks, fastener holes, peeling paint, etc; the paint then prevents the timber from breathing and keeps the moisture content elevated, accelerating the rate of decay. Also note that several of these members have holes and cutoff fasteners left from their previous use before the towers were rebuilt.

Figure 3-10: The subcap has signs visible of decay in both abutments, including soft, punky wood along the centerline on the bottom face. This is likely a result of the vertical spikes securing the crossbeam on top (the end of one can be seen about 6 inches from the end of the white crossbeam in the left photo). These fasteners penetrate through the preservative treated layer of timber and create a path for moisture to enter the core of the member, causing accelerated decay.

3.1.5 Cables

The suspension cables were visually inspected, and appear to be in good condition. The main suspension cables and vertical suspender cables are galvanized steel and show minimal signs of corrosion and no signs of fraying or physical damage. The suspension cables are anchored at each end with a steel turnbuckle and a large chain which is embedded in a concrete block below ground. Additionally, a steel cable sling is connected in parallel with the turnbuckle and chain and is embedded in the same concreted block providing a redundant connection to the concrete anchor. Two of the four cable anchors were dug out for visual inspection to the point where they enter the concrete block, approximately 3 feet below ground. The steel elements are in fair condition, with surface rust but no signs of significant section loss. The size of the concrete anchor blocks, depth of embedment, and the extent of reinforcing steel within the concrete are unknown.

Figure 3-11: Cable anchor at Cable 2, End 1. The chain and cable sling are embedded in concrete, approximately 3 feet below ground. The steel components show surface rust, but no signs of section loss. It is unknown how far the cable extends into the concrete, or how large the concrete block is.

Figure 3-12: Cable anchor at Cable 2, End 2. At approximately 2 feet below ground, this anchor is not as deep as the anchor at End 1; however, it is in similar condition, with minimal surface rust, and no signs of section loss.

The bridge is braced by a pair of lateral stabilizing cables – one on the upstream side and one on the downstream side of the bridge. Each cable is anchored into the rock on each bank, and passes through a steel strap below Crossbeam 11, at mid span. Both cables have splices in them, which appear to be repairs from previous damage, but otherwise the cables and anchors to the rock appear to be in fair condition with minimal signs of surface corrosion. The connection to the crossbeam shows significant signs of deterioration. The steel strap is connected with a pair of vertical bolts through the crossbeam, these bolts are sinking into the surface of the beam and the timber and shows signs of decay around the fasteners.

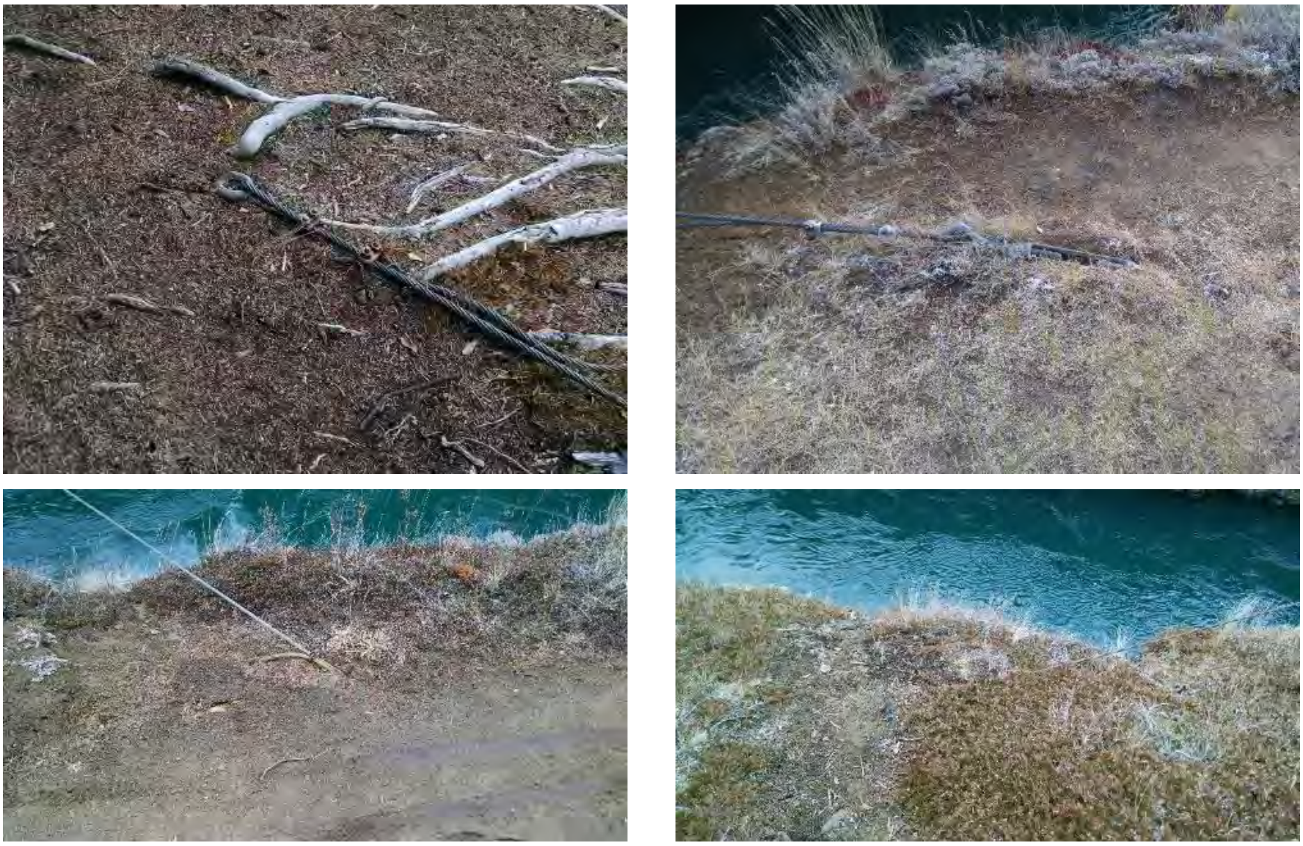

Figure 3-13: Lateral stabilizing cable anchors (clockwise from top left) Side 1 End 1; Side 2 End 1; Side 2 End 2; Side 1 End 2. The cables and their connections to the rock appear to be in fair condition.

Figure 3-14: The Side 1 cable (left) has a splice near End 1 and the Side 2 cable has a splice near mid-span on End 2. These splices appear to be repairs to previous damage and are in fair condition.

Figure 3-15: Lateral cable connection to Crossbeam 11 at mid-span. The vertical through-bolt which connects to the steel strap is sinking into the beam, and the timber shows signs of decay.

3.2 Stress Wave Time Testing Results

This inspection included the use of non-destructive test equipment identified as EPHOD™ (Electronic Pulse Highlight and Outline Diagnostic) compression wave technology. The EPHOD™ equipment was utilized to complete stress wave measurements along with other WRD techniques to locate internal decay in a non-destructive nature. Stress wave analysis consists of sending a "compression" wave through a medium (wood) and measuring its velocity. The compression wave is introduced into the material by striking it with a hammer or blunt object. When the hammer (start) strikes the wood, an accurate timer is started; when the compression wave reaches a second accelerometer (the stop) on the opposite side of the member, the timer is stopped. The distance between the "start" and "stop" accelerometer is measured. Knowing distance and time, the average velocity of the stress wave (sound wave) can be measured.

The Modulus of Elasticity of the material is theoretically related to the velocity of the stress wave and the density according to Equation 1.

Equation 1:

$$\mathbf{E = c^2/\rho}$$

Where,

E = Modulus of elasticity

c = Velocity of the stress wave

ρ = Density of the material

It is the Modulus of Elasticity (MOE) calculated in Equation 1 which indicates if decay is present or not. Typically, the MOE for sound Douglas-fir ranges from 1.5x106 psi to 2.2x106 psi. The range can be tightened if the exact grade is known. If the sample of Douglas-fir has been subject to fungi decay, the specific gravity (weight relative to water) of the wood will decrease. The decrease in specific gravity causes a decrease in the Modulus of Elasticity, which decreases the velocity of the stress wave, increasing the time required for the stress wave to travel through the material. If decay is present, the measured Modulus of Elasticity using the stress wave timer will be significantly lower than the expected range.

Stress wave times were taken on all of the accessible timber main structural elements, including crossbeams, stringers, and timbers in the abutment towers. Due to limited access, portions of the bridge could not be tested. These areas included the centers of the crossbeams over the water; and Stringers lines 2 and 3. Deck planks were removed at two locations (over Crossbeams 5 and 11) to allow partial testing of these inaccessible areas; based on those results, it is believed that the worst-case areas were tested, and the inaccessible portions, which are more sheltered, will have less decay. Therefore, it is believed that the inspection gives a complete picture of the condition of the timber elements in the bridge.

Table 2 in Appendix B shows the stress wave time results. Results are also shown graphically on the drawings provided with this report. When the through wave time values (adjusted for a 300mm gauge length) exceed 700 microseconds (ms) but are below 1000 (shown in yellow) the area measured is capable of carrying its own dead weight and an unknown live load. When the times exceed 1000 ms (shown in red) the element is not capable of carrying its own dead weight at that localized area. When the values reach numbers over 3000 ms, the element could collapse at any time in that area.

Advanced decay in a number of the main structural elements, as described in Section 3.2 above. The worst examples are in the crossbeams, specifically around the connections to the suspender cables.

See articles provided by Tingley on Australian Hardwood Timber Bridge degradation. Also see article written by Tingley on the difference between bore sounding, global stiffness inspection methods and an elemental strength NDT equipment system like the EPHOD™ system used in this inspection.

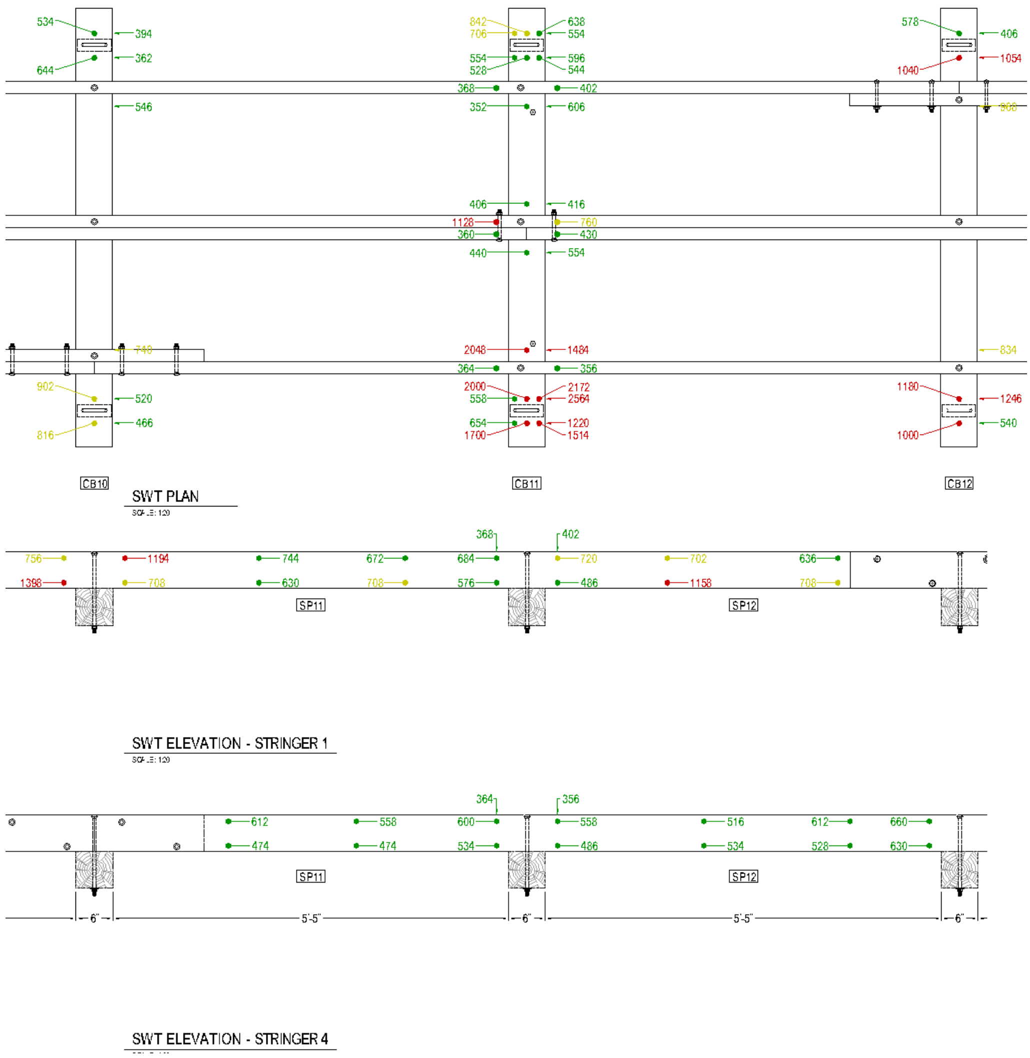

Figure 3-16: The drawing above shows the SWT results for Crossbeams 10 through 12 and stringers in Spans 11 and 12. Values in green show areas where the timber is in sound condition; values in yellow indicate that the area is weakened decay and capable of carrying and unknown live load; values in red indicate that the area is no longer capable of carrying even its own dead weight. Note the concentration of red-zones around the vertical U-bolt connections in Beams 11 and 12. Deck planks were removed above Crossbeam 11 allowing for additional test of this beam and of Stringers 2 and 3. See Appendix B and the provided drawings for complete results.

3.3 Assay Sample Results

Core samples from fifteen locations throughout the bridge were collected and returned to WRD's facility in Jefferson, Oregon for laboratory analysis. The locations were selected to give a representative sampling primary structural elements. Where possible, samples were taken in locations that had corresponding SWT test results to provide a secondary confirmation of the SWT results. Samples were selected to represent the full range of SWT values that were found in the bridge, from sound timber to high Red Zones. Nine samples were taken from crossbeams, five were taken from stringers, and one was taken from a sill beam in an abutment tower.

Samples were taken using an increment borer. When possible the cores were taken either horizontally or vertically from the bottom face of the element stopping approximately ½ inch from the top face. This was to avoid creating vertical penetrations through the tops of members, which would lead to accelerated decay. Each sample was divided into 4 segments (e.g. Bottom Outside, Bottom Inside, Top Inside, and Top Outside) for testing to show changes throughout the cross-section. Samples were tested for Moisture Content and Specific Gravity. Additionally, they were examined under a microscope to identify the species.

Findings are summarized below, and complete laboratory results are provided in Appendix C.

3.3.1 Crossbeams

One sample was taken from each end of Crossbeams 2, 3, 19, and 20, and one sample taken from Crossbeam 11. The samples were visually examined under a microscope and the species was identified as Spruce. This is believed to be the case for all of the crossbeams and stringers.

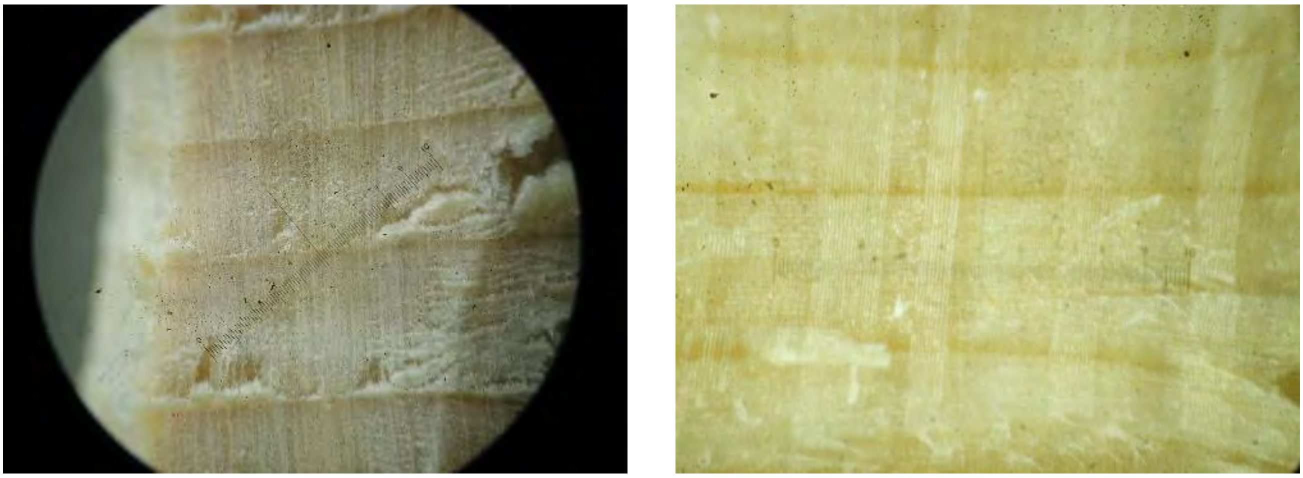

Figure 3-17: Samples from Crossbeam 3, photographed under high magnification. Based on the grain patterns and cell structure, the species was identified as Spruce.

Beam 2	Length: 72.0"	Depth: 6.0"		Width: 6.0"				
Location:	4"	8"	16"	32"	40"	56"	64"	68"
Horiz. raw	269	268	265	256	172	142	550	387
Horiz. adj	538	536	530	512	344	284	1100	774
Vert. raw	168	239	189	284	264	235	233	276
Vert. adj	336	478	378	568	528	470	466	552
Vert. Side 1 raw	207	223					3100	394
Vert Side 1 adj	414	446					6200	788
Vert. Side 2 raw	256	293					591	888
Vert Side 2 adj	512	586					1182	1776
Core Sampe Data	4"							68"
Side 1 Out SG / MC	0.45 27.4							0.39 18.7
Side 1 In SG / MC	0.38 33.7							33.3
Side 2 In SG / MC	0.44 26.0							21.5
Side 2 Out SG / MC	0.45 27.9							0.57 34.1

Figure 3-18: Crossbeam 2, 4 inches from End 1. Horizontal core, Side 1 at left.

Figure 3-19: Crossbeam 2, 68 inches from End 1. Horizontal core, Side 1 at left. The middle section of this sample was crumbled into fine powder, indicating severe decay and cavitation at this location. Specific gravity calculations cannot be made on the crumbled section, but results show very high moisture content. The sample was taken approximately 2 inches from the suspender cable U-bolt connection.

Beam 3	Length: 72.0" Depth: 6.0" Width: 6.0"							
Location:	4"	8"		16"	32"	40"	56"	64"		68"
Horiz. raw	336	347		266	258	185	438	540		380
Horiz. adj	672	694		532	516	370	876	1080		760
Vert. raw	233	250		191	194	289	234	577		374
Vert. adj	466	500		382	388	578	468	1154		748
Vert. Side 1 raw	203	156						215		321
Vert Side 1 adj	406	312						430		642
Vert. Side 2 raw	325	213						308		390
Vert Side 2 adj	650	426						616		780
Core Sampe Data		8"						64"		
Top Out SG / MC		0.37	18.7					0.40	14.1	
Top In SG / MC		0.46	31.7					0.40	19.5	
Bottom In SG / MC		0.35	33.3					0.45	15.8	
Bottom Out SG / MC		0.40	18.6					0.38	19.5	

Figure 3-20: Crossbeam 3, 8 inches from End 1. Vertical core, bottom at left. This sample shows somewhat reduced specific gravity in the Top-Outside and Bottom-Inside sections. Note the knot in the Top-Inside section which may account for the higher specific gravity in that section

Figure 3-21: Crossbeam 3, 64 inches from End 1. Vertical core, bottom at left.

Beam 11	Length: 72.0"	Depth: 6.0"	Width: 6.0"							
Location:	4"	8"	16"	32"	40"	56"		64"	68"	
Horiz. raw	277	298	303	208	277	742		1282	610	
Horiz. adj	554	596	606	416	554	1484		2564	1220	
Vert. raw	421	264	176	203	220	1024		1000	850	
Vert. adj	842	528	352	406	440	2048		2000	1700	
Vert. Side 1 raw	353	277						279	327	
Vert Side 1 adj	706	554						558	654	
Vert. Side 2 raw	319	272						1086	757	
Vert Side 2 adj	638	544						2172	1514	
Core Sampe Data						56"				
Top Out SG / MC							20.4			
Top In SG / MC							20.8			
Bottom In SG / MC							4.4			
Bottom Out SG / MC						0.52	20.4			

Figure 3-22: Crossbeam 11, 56 inches from End 1. Vertical core, bottom at left. This core shows severe decay and crumbling in the top three quarters; specific gravity readings could only be made in the Bottom-Outside section. These findings support the SWT results which show high Red Zone readings throughout this end of the beam.

Beam 19	Length: 72.0"	Depth: 6.0"	Width: 6.0"							
Location:	4"	8"		16"	32"	40"	56"	64"		68"
Horiz. raw	229	258		231	149	121	173	150		170
Horiz. adj	458	516		462	298	242	346	300		340
Vert. raw	200	181		173	163	126	181	254		164
Vert. adj	400	362		346	326	252	362	508		328
Vert. Side 1 raw	187	196						179		184
Vert Side 1 adj	374	392						358		368
Vert. Side 2 raw	200	170						198		212
Vert Side 2 adj	400	340						396		424
Core Sampe Data										
Top Out SG / MC		0.38	20.8					0.44	25.5	
Top In SG / MC		0.36	24.3					0.39	34.1	
Bottom In SG / MC		0.45	20.0					0.40	19.8	
Bottom Out SG / MC		0.39	12.8					0.36	27.0	

Figure 3-23: Crossbeam 19, 8 inches from End 1. Vertical core, bottom at left. This core came out cleanly and is in fairly good condition. Specific gravity readings are fairly consistent across the section. These results are consistent with the good SWT results in this beam.

Figure 3-24: Crossbeam 19, 64 inches from End 1. Vertical core, bottom at left.

Beam 20	Length: 72.0" Depth: 6.0" Width: 6.0"							
Location:	4"	8"	16"	32"	40"	56"	64"	68"
Horiz. raw	250	244	204	134	155	236	389	320
Horiz. adj	500	488	408	268	310	472	778	640
Vert. raw	250	225	180	128	151	146	460	879
Vert. adj	500	450	360	256	302	292	920	1758
Vert. Side 1 raw	227	216					266	298
Vert Side 1 adj	454	432					532	596
Vert. Side 2 raw	238	192					636	
Vert Side 2 adj	476	384					1272	
Core Sampe Data		8"					64"	
Top Out SG / MC		0.39 218.5					115.7	
Top In SG / MC		0.37 134.5					126.3	
Bottom In SG / MC		0.36 106.8					187.5	
Bottom Out SG / MC		0.50 25.0					153.5	

Figure 3-25: Crossbeam 20, 8 inches from End 1, 1 inch from Side 2 of beam. Vertical core, bottom at left.
This sample has very high moisture content and reduced specific gravity in the top three quarters.

Figure 3-26: Crossbeam 20, 64 inches from End 1, 1 inch from Side 2 of beam. Vertical core, bottom at left.
This sample crumbled into fine pieces and specific gravity calculations could not be made. Moisture content was very high throughout. The condition of this sample is consistent with the Red Zone SWT reading at the same location.

3.3.2 Stringers

Samples were taken from all for stringers in Span 3, 24 inches from the face of Crossbeam 2. One sample was taken from Stringer 1 in Span 19, 48 inches from the face of Crossbeam 18. The cores were taken vertically from the bottom.

The samples in Span 3 show that the center stringers have higher moisture contents than the outside stringers. This is probably because the sistered stringers have reduced ability to breath and because the deck limits air flow around them. Stringers 1 through 3 all show similar levels of reduced specific gravity. Based on these results it is predicted that the center stringers will have similar or slightly higher levels of decay than the outside stringers overall, but without the ability to conduct complete SWT analysis, specific area of decay cannot be pinpointed.

Span 3 Stringer 1	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans:	1-4		
SWT Data	2"	24"		32"	48"	63"				
Horiz. Top raw	117	136		85	94	78				
Horiz. Top adj	702	816		510	564	468				
Horiz. Bottom raw	63	131		79	58	46				
Horiz. Bottom adj	378	786		474	348	276				
Core Sampe Data		24"								
Top Out SG / MC		0.56	22.2							
Top In SG / MC		0.40	24.4							
Bottom In SG / MC		0.37	22.2							
Bottom Out SG / MC		0.49	21.8							

Figure 3-27: Span 3 Stringer 1. Vertical Core, bottom to left.

Span 3 Stringer 2	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans:	2-5		
Core Sampe Data		24"								
Top Out SG / MC		0.63	32.3							
Top In SG / MC		0.41	24.8							
Bottom In SG / MC		0.44	23.8							
Bottom Out SG / MC		0.43	22.3							

Figure 3-28: Span 3 Stringer 2. Vertical Core, bottom to left.

Span 3 Stringer 3	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans: 1-3			
Core Sampe Data		24"								
Top Out SG / MC		0.40	32.5							
Top In SG / MC		0.40	34.4							
Bottom In SG / MC		0.47	20.2							
Bottom Out SG / MC		0.51	25.7							

Figure 3-29: Span 3 Stringer 3. Vertical Core, bottom to left.

Span 3 Stringer 4	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans: 3-6			
SWT Data	16"			48"	63"					
Horiz. Top raw	59			77	74					
Horiz. Top adj	354			462	444					
Horiz. Bottom raw	56			67	73					
Horiz. Bottom adj	336			402	438					
Core Sampe Data		24"								
Top Out SG / MC		0.58	24.3							
Top In SG / MC		0.42	28.7							
Bottom In SG / MC		0.53	22.2							
Bottom Out SG / MC		0.65	25.9							

Figure 3-30: Span 3 Stringer 1. Vertical Core, bottom to left.
All of these stringer cores show high moisture contents, especially Stringers 2 and 3. Stringers 1 through 3 also show reduced specific gravity in portions of sample.

Span 19 Stringer 1	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans: 17-20			
Location:	2"	24"	48"		63"					
Horiz. Top raw	87	105	194		103					
Horiz. Top adj	522	630	1164		618					
Horiz. Bottom raw	101	53	66		67					
Horiz. Bottom adj	606	318	396		402					
Core Sampe Data			48"							
Top Out MC/SG			0.50	22.5						
Top In MC/SG			0.41	29.4						
Bottom In MC/SG			0.39	24.6						
Bottom Out MC/SG			0.43	24.4						

Figure 3-31: Span 19 Stringer 1. Vertical Core, bottom to left.

3.3.3 Abutment Timbers

One sample was taken from Abutment 1, from Sill Beam 4. The sample was located directly below Post 4A; there appears to be a vertical drift pin through the cribbing beams below each of the posts in Abutment 1. This sill beam was constructed from salvaged timber when the towers were rebuilt in 2009. The sample was examined under a microscope, and the species was determined to be Spruce. This is believed to be the case for all of the timbers in the abutment towers.

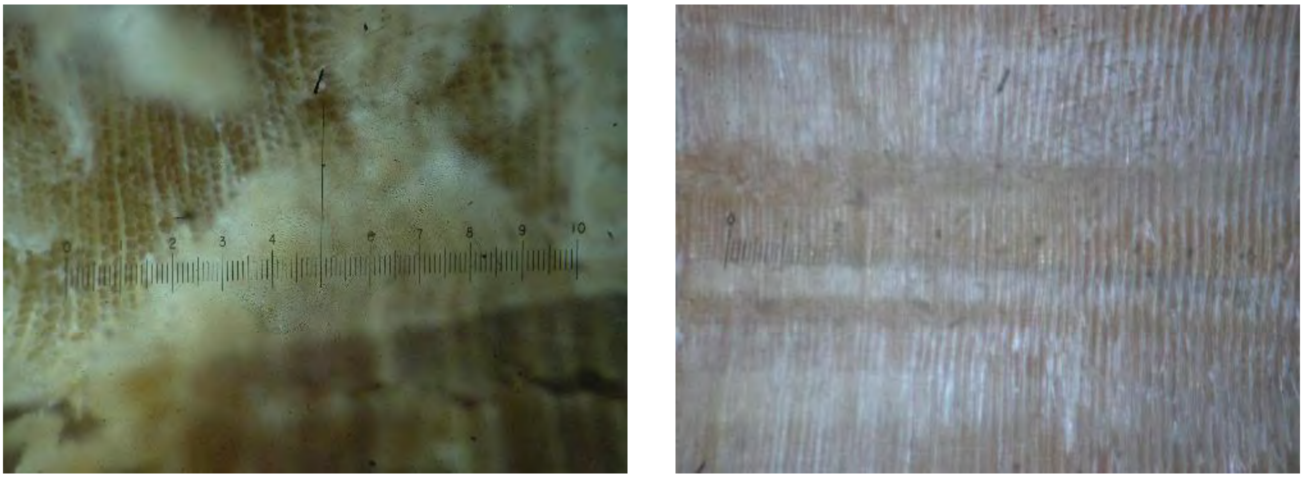

Figure 3-32: The sample from Abutment 1 under high magnification. The species is Spruce.

Sill 4	Length: 126.0" Depth: 12.0" Width: 12.0"								
Location:	4"	12"	17"	22"	26"	48"	72"		...
Vert. raw	295				250	236	264		
Vert. adj	295				250	236	264	248	
Horiz. raw	643		324		268	275	318	551	
Horiz. adj	643		324		268	275	318	551	
Horiz. Top raw		293		259					
Horiz. Top adj		293		259					
Horiz. Bottom raw			320						
Horiz. Bottom adj			320						
Core Sampe Data		13"							
Side 1 Out SG / MC		0.58 17.5							
Side 1 In SG / MC		0.43 23.0							
Side 2 In SG / MC		0.47 18.8							
Side 2 Out SG / MC		0.56 17.9							

Figure 3-33: Abutment 1, Sill 4. 13 inches from End 1, 1-1/2 inches from Top. Horizontal core Side 1 to left.

The abutment timbers are Douglas fir. The specific gravity of 0.56 and 0.58 on the outers is higher due to the treatment and the fact that the treatment doesn't penetrate very far beyond the incising in the outer zones of the timber. Particularly in Spruce which is not considered a treatable species. This sample was taken directly below the post, and there is believed to be a vertical pin through the cribbing members at this location.

In summary there were important findings developed from the assay sampling and subsequent laboratory testing that are worth noting. The first important point to note is that the moisture content of the bridge elements is a lot higher in many locations than the typical bridge moisture contents for timber bridges in situ (approximately 14% under a tight deck and 16% under a loose deck on average). There were many locations where the moisture content readings were well above the level that allows decay to begin which is 22 to 23%, some readings were over 100% which is a very adverse condition for bridge longevity. This is a result of the heavy solids paint and vertical fasteners that originate at the top of the element. The paint should be removed and vertical fasteners originating from the top replaced with horizontal fasteners. The second point to be made is that the Specific Gravity values for the cores correlated very well with the SWT values strongly indicating that the SWT values are properly representing the condition of the elements. Further, in points where there was no decay the SG values correlated well with published values for Spruce; .45 to .46. The third point to be made is that the cross beams are in very poor condition around the vertical fasteners connecting the hanger cables to the suspension cable. The samples were often found to be totally degraded into powder. Clearly many of the cross beams are gone at their ends and the SWT values properly indicate the condition of the cross beams at their ends particularly. The closure of the bridge was critical for safety as these results indicate. The fourth point to be made is that the stringers also have areas of decay and should be replaced where necessary. Finally, the fifth point to be made is that the abutments are in good condition however, if the heavy solids paint is not removed they won't stay this way long as the paint will continue to trap high moisture content amounts in the wood and cause decay to accelerate.

4.0 RESULTS AND FINDINGS

Many of the main structural elements of Miles Canyon Bridge are highly decayed, especially the crossbeams and stringers. The following repairs are recommended to restore the bridge's capacity and achieve a load-rating of 5 kPa for pedestrian live loads.

Crossbeams: Replace Crossbeams 1, 2, 3, 5, 8, 9, 11, 12, 17, 20, and 21, which all contain advanced decay around the connections. Replacement would be completed by working across the bridge hanging new tension cables and installing new transverse elements. Remove all heavy solids paint and stain properly.

Remaining crossbeams should be carefully monitored for further decay and replaced as needed, especially Crossbeams 4, 6, 7, 10, 13, 14, and 15, which contain Yellow Zones indicating reduced capacity due to decay. Remove all heavy solids paint and stain properly.

	Replacement beams should be installed without the use of vertical fasteners which penetrate the top of the member. It is recommended that the connection to the suspender cables be made using a connector strap which wraps around the outside of the member.
Stringers	Replace stringers which contain Red Zones. These include: Stringer 1, Spans 5-8; Stringer 1, Spans 9-12; Stringer 1, Spans 13-16; Stringer 1, Spans 17-20; Stringer 1; Spans 21-22; Stringer 2, Spans 10-13; and Stringer 4, Spans 15-18. Remove all heavy solids paint and stain properly.
	Remaining stringers should be carefully monitored for further decay and replaced as needed. All of the tested stringers contained Yellow Zone SWT readings indicating reduced capacity due to decay. Replace bad elements and remove all heavy solids paint and stain properly.
Deck	The deck is in fair condition. Monitor for decay and replace planks as needed. Remove all heavy solids paint and stain properly.
Railings	Replace damaged elements. Areas that were noted as needing repair included the cross braces along Side 1 in Spans 15-21.
Abutments	Timber elements in the towers are in good condition overall and do not require repair. See notes above and below about updates to improve longevity. These apply especially to the painted elements which have scattered Yellow and Red Zone readings, and the Cap at Abutment 2, which has Yellow Zone readings throughout. Remove all heavy solids paint and stain properly.
All	All new timber elements should be preservative treated, and all machining (cutting, drilling, etc.) should be completed prior to treatment.
	New connection details should be designed to avoid the use of vertical fasteners which penetrate the tops of members. For example, deck planks may be secured using angle-clips and screws installed from below the deck. Where this is not possible, all holes should be machined prior to treatment.
	Existing timber elements should be treated with Decaystop™ diffuser rods to prevent further decay. This is especially important where the elements are already showing signs of decay and Yellow or Red Zone SWT readings.
	Any exposed bright wood (bolt holes, field-cut ends, etc.) should be field treated Copper Naphthenate preservative, and the end-grain should be sealed with Anchorseal paraffin sealant.
	Heavy solids paint should be removed from all timber elements. The application of a stain with lower than 29% solids is acceptable. Removing the paint allows the timber to breath and dry out and will slow the rate of decay growth. Similarly, flashings (such as those on the abutment tower caps) should either be removed or redesigned to provide ample air-flow beneath the flashing, to allow the timber to dry.

5.0 RESTORATION STRATEGY

The structure can be restored by placing new hanger cables and transverse elements properly treated with machining that is necessary completed before treatment. It is recommended that Penta pressure treated, incised cross beams be utilized for these elements, with all holes drilled prior to treatment. The uptake should be a minimum of 5 kg/m3 with a target of 9 kg/m3. The best material for these elements would be coastal Douglas-fir. Once new transverse elements and hangers are placed then the deck should be removed and new longitudinal stringers placed using the same material as with the transfer beams. This would provide the lightest weight, least maintenance solution as steel would rust, even if galvanized, and concrete is not an option. More expensive options in metal such as Stainless steel or aluminum could be considered but they are more expensive, difficult to install and problematic for downstream maintenance.

The deck can be stripped and reused where possible, supplemented with new pieces. The railing system can be replaced as necessary. The whole system should be stripped and stained. The abutments elements should be diffused. The main transverse elements and longitudinal elements could be replaced immediately to keep the bridge from failing during a snow fall and making the job of restoration more difficult. The remaining works could be completed in the spring.

6.0 CONCLUSION

The overall condition of elements hanging from the suspension cables in Miles Canyon Bridge is poor. Many of the crossbeams have advanced decay around the connections to the vertical suspender cables, and several of the longitudinal stringers show signs of decay as well. However, the abutment towers, the main suspension cables are in good condition. While the decayed and damaged elements revealed by this inspection are critical to the safety of the structure, they can be replaced relatively easily to restore the capacity of the bridge.

With the repairs listed above, it will be possible to restore the bridge to safely carry 5kPa pedestrian loads; however, due to the severe decay in many of the main structural elements, it is recommended that the bridge remain closed until the above repairs are completed. Immediate works to prevent snow build up induced failure or failure under a trespass load might be considered. In such a case immediate remedial works on the transverse, longitudinal and deck elements could be undertaken with the remaining restoration to take place in the next building season.

The recommendation to close the bridge was based on the non-destructive testing results, which show high levels of decay in several of the main crossbeams. The assay sample testing correlated well with the SWT data and confirmed that the decision to close the bridge was the correct one. While the longitudinal stringers with staggered joints would provide some level of redundancy should one of the crossbeams fail, this level of redundancy is not enough to ensure the safety of the bridge in the case of a partial failure. In several instances, two or more adjacent crossbeams are severely weakened by decay. If one beam failed, the adjacent beams may not be able to carry the increased loads. In addition, several of the stringers have advanced decay; it is likely that they would not be able to resist the applied stresses that would result from a crossbeam failure. These scenarios could potentially lead to a progressive failure of a large portion of the structure.

Fortunately, the main suspension cables and the abutment towers are in good condition, requiring only minor upgrades; if major repairs to these elements were required, they would be much more costly to complete. The required repairs to the crossbeams and stringers can be completed fairly easily through piece-by-piece replacement of the degraded elements. Furthermore, the upgrades listed above will greatly improve the longevity of the structure, extending the useful life of this landmark structure for many years to come.

Dan Tingley Ph.D., P. Eng. (Canada), P. Eng., MIEAust, CPEng, RPEQ
Senior Wood Technology/ Structural Engineer
Wood Research and Development

Robert Keller P.E.
Project Engineer
Wood Research and Development

APPENDIX

Appendix A Bridge Information Table

Appendix B SWT Tables

Appendix C Assay Sample Lab Results

Appendix D Biography and Curriculum Vitae for Dr. Dan Tingley P.Eng. (Canada)

Appendix E Original article prepared by Author on Decay and Retrofit of Existing Structures.

Appendix F Paper and Article written by Tingley for Australian Small Bridge Conference July 09

Appendix G Paper written by Tingley and Richards presented at the Australian Small Bridge Conference July 2009

Appendix H Discussion on reasons for not using coatings with over 30% solids on heavy dimension timber and ferric degradation around on galvanized connectors.

Appendix I Tingley discussion on Advanced Inspection Techniques and banding of timber piles.

Appendix J Top Ten Modifications to Timber Bridge Maintenance Manuals.

Appendix A

Table 3: Miles Canyon Bridge Info	
Bridge Name:	**Miles Canyon Bridge**
Location:	Whitehorse, Yukon
For:	Yukon Government
Inspection Date:	28-Oct-15 through 31-Oct-15
Inspected by:	Chris Legg, Randy Lewis, Matt Cole
Lat:	60°39'44.05"N
Long:	135° 1'44.16"W
Main Span Length:	130'-10"
Deck Width:	4'-0"
Skew:	None

Appendix B

Table 1: Miles Canyon Bridge SWT Data 28 Oct 2015 through 31 Oct 2015

Distance measurements:	Ft and inches	Temp:	
Abutment 1 Direction:		RH:	

SWT > 1000: Red Highlight Shows Elements requiring Immediate restoration or replacement (12in gauge length)

700 < SWT < 999: Yellow Highlight Shows Elements requiring caution in use frequent inspection (12in gauge length)

Stringers

Length and locations measured from face of crossbeam at End 1

Horizontal readings taken 1 inch from top or bottom face

Span 1 — AB1 — Beam 1

Stringer 1	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans: 1-4			
Location:		16"	40"	62"						
Horiz. Top raw		74	78	68						
Horiz. Top adj		444	468	408						
Horiz. Bottom raw		37	60	55						
Horiz. Bottom adj		222	360	330						

Stringer 2	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans: 1			

Stringer 3	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans: 1-3			

Stringer 4	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans: 1-2			
Location:		16"	35"	62"						
Horiz. Top raw		132	101	61						
Horiz. Top adj		792	606	366						
Horiz. Bottom raw		131	118	74						
Horiz. Bottom adj		786	708	444						

Span 2 — Beam 1 — Beam 2

Stringer 1	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans: 1-4			
Location:	2"	24"	46"	62"						
Horiz. Top raw	64	60	86	87						
Horiz. Top adj	384	360	516	522						
Horiz. Bottom raw	87	77	59	60						
Horiz. Bottom adj	522	462	354	360						

Stringer 2	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans: 2-5			

Stringer 3	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans: 1-3			

Stringer 4	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans: 1-2			
Location:	2"	26"	47"							
Horiz. Top raw	73	77	99							
Horiz. Top adj	438	462	594							
Horiz. Bottom raw	90	79	133							
Horiz. Bottom adj	540	474	798							

Table 1 Continued: Miles Canyon Bridge SWT Data 28 Oct 2015

Span 3

Beam 2 — Beam 3

Stringer 1	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans: 1-4			
Location:	2"	24"	32"	48"	63"					
Horiz. Top raw	117	136	85	94	78					
Horiz. Top adj	702	816	510	564	468					
Horiz. Bottom raw	63	131	79	58	46					
Horiz. Bottom adj	378	786	474	348	276					

Stringer 2	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans: 2-5

Stringer 3	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans: 1-3

Stringer 4	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans: 3-6			
Location:		16"	48"	63"						
Horiz. Top raw		59	77	74						
Horiz. Top adj		354	462	444						
Horiz. Bottom raw		56	67	73						
Horiz. Bottom adj		336	402	438						

Span 4

Beam 3 — Beam 4

Stringer 1	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans: 1-4			
	3"	22"	44"							
Horiz. Top raw	126	148	104							
Horiz. Top adj	756	888	624							
Horiz. Bottom raw	129	67	81							
Horiz. Bottom adj	774	402	486							

Stringer 2	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans: 2-5

Stringer 3	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans: 4-7

Stringer 4	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans: 3-6			
Location:	2"	24"	48"	63"						
Horiz. Top raw	112	91	96	114						
Horiz. Top adj	672	546	576	684						
Horiz. Bottom raw	90	134	86	113						
Horiz. Bottom adj	540	804	516	678						

Table 1 Continued: Miles Canyon Bridge SWT Data 28 Oct 2015

Span 5 **Beam 4** **Beam 5**

Stringer 1	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans: 5-8			
Location:		24"	44"	63"						
Vert. raw				154						
Vert adj				308						
Horiz. Top raw		113	149	151						
Horiz. Top adj		678	894	906						
Horiz. Bottom raw		99	65	81						
Horiz. Bottom adj		594	390	486						

Stringer 2	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans: 2-5			
Location:				63"						
Vert. raw				223						
Vert adj				446						

Stringer 3	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans: 4-7			
Location:				63"						
Vert. raw				203						
Vert adj				406						

Stringer 4	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans: 3-6			
Location:	2"	26"	47"	63"						
Vert. raw				155						
Vert adj				310						
Horiz. Top raw	107	86	98	101						
Horiz. Top adj	642	516	588	606						
Horiz. Bottom raw	81	90	90	90						
Horiz. Bottom adj	486	540	540	540						

Table 1 Continued: Miles Canyon Bridge SWT Data 28 Oct 2015

Span 6

Beam 5 — **Beam 6**

Stringer 1	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans: 5-8			
Location:	2"	24"	44"	63"						
Vert. raw	141									
Vert adj	282									
Horiz. Top raw	186	110	190	192						
Horiz. Top adj	1116	660	1140	1152						
Horiz. Bottom raw	160	116	180	102						
Horiz. Bottom adj	960	696	1080	612						

Stringer 2	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans: 6-9			
Location:	2"									
Vert. raw	210									
Vert adj	420									

Stringer 3	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans: 4-7			
Location:	2"									
Vert. raw	228									
Vert adj	456									

Stringer 4	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans: 3-6			
Location:	2"	26"	45"							
Vert. raw	140									
Vert adj	280									
Horiz. Top raw	60	118								
Horiz. Top adj	360	708								
Horiz. Bottom raw	81	85	86							
Horiz. Bottom adj	486	510	516							

Span 7

Beam 6 — **Beam 7**

Stringer 1	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans: 5-8			
Location:	2"	24"	48"	63"						
Horiz. Top raw	144	151	170	187						
Horiz. Top adj	864	906	1020	1122						
Horiz. Bottom raw	151	100	136	110						
Horiz. Bottom adj	906	600	816	660						

Stringer 2	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans: 6-9

Stringer 3	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans: 4-7

Stringer 4	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans: 7-10			
Location:		19"	37"	49"	63"					
Horiz. Top raw		72	97	88	87					
Horiz. Top adj		432	582	528	522					
Horiz. Bottom raw		113	99	66	130					
Horiz. Bottom adj		678	594	396	780					

Table 1 Continued: Miles Canyon Bridge SWT Data 28 Oct 2015

Span 10

Beam 9 — **Beam 10**

Stringer 1	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans: 9-12			
Location:	2"	20"	44"	63"						
Horiz. Top raw	121	115	116	126						
Horiz. Top adj	726	690	696	756						
Horiz. Bottom raw	160	89	121	233						
Horiz. Bottom adj	960	534	726	1398						

Stringer 2	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans: 10-13			

Stringer 3	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans: 8-11			

Stringer 4	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans: 7-10			
Location:	2"	19"	48"							
Horiz. Top raw	96	100	86							
Horiz. Top adj	576	600	516							
Horiz. Bottom raw	99	134	70							
Horiz. Bottom adj	594	804	420							

Span 11

Beam 10 — **Beam 11**

Stringer 1	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans: 9-12			
Location:	2"	24"	48"	63"						
Vert. raw				184						
Vert adj				368						
Horiz. Top raw	199	79	112	114						
Horiz. Top adj	1194	474	672	684						
Horiz. Bottom raw	118	105	118	96						
Horiz. Bottom adj	708	630	708	576						

Stringer 2	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans: 10-13			
Location:				63"						
Vert. raw				564						
Vert adj				1128						

Stringer 3	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans: 8-11			
Location:				63"						
Vert. raw				180						
Vert adj				360						

Stringer 4	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans: 11-14			
Location:		17"	40"	63"						
Vert. raw				182						
Vert adj				364						
Horiz. Top raw		102	93	100						
Horiz. Top adj		612	558	600						
Horiz. Bottom raw		79	79	89						
Horiz. Bottom adj		474	474	534						

Table 1 Continued: Miles Canyon Bridge SWT Data 28 Oct 2015

Span 8

Beam 7 — **Beam 8**

Stringer 1	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans: 5-8			
Location:	2"	24"	47"							
Horiz. Top raw	198	89	176							
Horiz. Top adj	1188	534	1056							
Horiz. Bottom raw	188	92	119							
Horiz. Bottom adj	1128	552	714							

Stringer 2	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans: 6-9

Stringer 3	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans: 8-11

Stringer 4	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans: 7-10			
Location:	2"	26"	51"	63"						
Horiz. Top raw	131	92	152	105						
Horiz. Top adj	786	552	912	630						
Horiz. Bottom raw	152	137	147	124						
Horiz. Bottom adj	912	822	882	744						

Span 9

Beam 8 — **Beam 9**

Stringer 1	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans: 9-12			
Location:		20"	42"	63"						
Horiz. Top raw		160	151	191						
Horiz. Top adj		960	906	1146						
Horiz. Bottom raw		76	100	125						
Horiz. Bottom adj		456	600	750						

Stringer 2	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans: 6-9

Stringer 3	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans: 8-11

Stringer 4	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans: 7-10			
Location:	2"	26"	41"	63"						
Horiz. Top raw	57	137	82	68						
Horiz. Top adj	342	822	492	408						
Horiz. Bottom raw	122	92	118	86						
Horiz. Bottom adj	732	552	708	516						

Table 1 Continued: Miles Canyon Bridge SWT Data 28 Oct 2015

Span 12 — **Beam 11** — **Beam 12**

Stringer 1	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans:	9-12		
Location:	2"	20"	48"							
Vert. raw	201									
Vert adj	402									
Horiz. Top raw	120	117	106							
Horiz. Top adj	720	702	636							
Horiz. Bottom raw	81	193	118							
Horiz. Bottom adj	486	1158	708							

Stringer 2	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans:	10-13		
Location:	2"									
Vert. raw	380									
Vert adj	760									

Stringer 3	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans:	12-15		
Location:	2"									
Vert. raw	215									
Vert adj	430									

Stringer 4	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans:	11-14		
Location:	2"	26"	50"	63"						
Vert. raw	178									
Vert adj	356									
Horiz. Top raw	93	86	102	110						
Horiz. Top adj	558	516	612	660						
Horiz. Bottom raw	81	89	88	105						
Horiz. Bottom adj	486	534	528	630						

Table 1 Continued: Miles Canyon Bridge SWT Data 28 Oct 2015

Span 13

Beam 12 — **Beam 13**

Stringer 1	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans:	13-16		
Location:		16"	42"	63"						
Horiz. Top raw		85	172	123						
Horiz. Top adj		510	1032	738						
Horiz. Bottom raw		84	82	77						
Horiz. Bottom adj		504	492	462						

Stringer 2	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans:	10-13

Stringer 3	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans:	12-15

Stringer 4	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans:	11-14		
Location:	2"	26"	48"	63"						
Horiz. Top raw	109	100	96	117						
Horiz. Top adj	654	600	576	702						
Horiz. Bottom raw	95	78	93	73						
Horiz. Bottom adj	570	468	558	438						

Span 14

Beam 13 — **Beam 14**

Stringer 1	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans:	13-16		
Location:	2"	24"	48"	63"						
Horiz. Top raw	78	113	80	89						
Horiz. Top adj	468	678	480	534						
Horiz. Bottom raw	98	113	100	71						
Horiz. Bottom adj	588	678	600	426						

Stringer 2	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans:	14-17

Stringer 3	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans:	12-15

Stringer 4	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans:	11-14		
Location:	2"	28"	50"							
Horiz. Top raw	91	69								
Horiz. Top adj	546	414								
Horiz. Bottom raw	95	76	95							
Horiz. Bottom adj	570	456	570							

Table 1 Continued: Miles Canyon Bridge SWT Data 28 Oct 2015

Span 15

Beam 14 — **Beam 15**

Stringer 1	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans:	13-16		
Location:	2"	24"	48"	63"						
Horiz. Top raw	101	107	112	112						
Horiz. Top adj	606	642	672	672						
Horiz. Bottom raw	92	93	70	93						
Horiz. Bottom adj	552	558	420	558						

Stringer 2	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans:	14-17

Stringer 3	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans:	12-15

Stringer 4	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans:	15-18		
Location:		17"	41"	62"						
Horiz. Top raw		94	74	88						
Horiz. Top adj		564	444	528						
Horiz. Bottom raw		57	62	71						
Horiz. Bottom adj		342	372	426						

Span 16

Beam 15 — **Beam 16**

Stringer 1	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans:	13-16		
Location:	2"	24"	46"							
Horiz. Top raw	100	121	108							
Horiz. Top adj	600	726	648							
Horiz. Bottom raw	166	92	94							
Horiz. Bottom adj	996	552	564							

Stringer 2	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans:	14-17

Stringer 3	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans:	16-19

Stringer 4	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans:	15-18		
Location:	2"	25"	52"	64"						
Horiz. Top raw	85	96	117	161						
Horiz. Top adj	510	576	702	966						
Horiz. Bottom raw	93	118	80	151						
Horiz. Bottom adj	558	708	480	906						

Table 1 Continued: Miles Canyon Bridge SWT Data 28 Oct 2015

Span 17 — **Beam 16** — **Beam 17**

Stringer 1	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans:	17-20		
Location:		16"	42"	63"						
Horiz. Top raw		86	81	88						
Horiz. Top adj		516	486	528						
Horiz. Bottom raw		88	114	62						
Horiz. Bottom adj		528	684	372						

Stringer 2	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans:	14-17

Stringer 3	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans:	16-19

Stringer 4	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans:	15-18		
Location:	2"	26"	39"	62"						
Horiz. Top raw	140	81	117	92						
Horiz. Top adj	840	486	702	552						
Horiz. Bottom raw	102	80	70	105						
Horiz. Bottom adj	612	480	420	630						

Span 18 — **Beam 17** — **Beam 18**

Stringer 1	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans:	17-20		
Location:	2"	24"	48"	63"						
Horiz. Top raw	114	113	158	146						
Horiz. Top adj	684	678	948	876						
Horiz. Bottom raw	123	94	86	95						
Horiz. Bottom adj	738	564	516	570						

Stringer 2	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans:	18-21

Stringer 3	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans:	16-19

Stringer 4	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans:	15-18		
Location:	2"	25"	49"							
Horiz. Top raw	184	113	144							
Horiz. Top adj	1104	678	864							
Horiz. Bottom raw	86	81	77							
Horiz. Bottom adj	516	486	462							

Table 1 Continued: Miles Canyon Bridge SWT Data 28 Oct 2015

Span 19

Beam 18 — Beam 19

Stringer 1	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans:	17-20		
Location:	2"	24"	48"	63"						
Horiz. Top raw	87	105	194	103						
Horiz. Top adj	522	630	1164	618						
Horiz. Bottom raw	101	53	66	67						
Horiz. Bottom adj	606	318	396	402						

Stringer 2	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans:	18-21

Stringer 3	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans:	16-19

Stringer 4	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans:	19-22		
Location:		17"	42"	63"						
Horiz. Top raw		131	121	112						
Horiz. Top adj		786	726	672						
Horiz. Bottom raw		81	103	104						
Horiz. Bottom adj		486	618	624						

Span 20

Beam 19 — Beam 20

Stringer 1	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans:	17-20		
Location:	2"									
Horiz. Top raw	245									
Horiz. Top adj	1470									
Horiz. Bottom raw	89									
Horiz. Bottom adj	534									

Stringer 2	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans:	18-21

Stringer 3	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans:	20-22

Stringer 4	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans:	19-22		
Location:	2"	21"	41"	63"						
Horiz. Top raw	80	68	86	101						
Horiz. Top adj	480	408	516	606						
Horiz. Bottom raw	74	86	86	93						
Horiz. Bottom adj	444	516	516	558						

Table 1 Continued: Miles Canyon Bridge SWT Data 28 Oct 2015

Span 21

Beam 20 — **Beam 21**

Stringer 1	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans:	21-22		
Location:		18"	42"	63"						
Horiz. Top raw		63	54	62						
Horiz. Top adj		378	324	372						
Horiz. Bottom raw		92	51	83						
Horiz. Bottom adj		552	306	498						

Stringer 2	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans:	18-21

Stringer 3	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans:	20-22

Stringer 4	Length:	65.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans:	19-22		
Location:	2"	24"	40"	63"						
Horiz. Top raw	63	88	153	100						
Horiz. Top adj	378	528	918	600						
Horiz. Bottom raw	70	106	107	101						
Horiz. Bottom adj	420	636	642	606						

Span 22

Beam 21 — **AB2**

Stringer 1	Length:	73.0"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans:	21-22		
Location:	2"	24"	48"	60"	70"					
Horiz. Top raw	225	71	50	74	78					
Horiz. Top adj	1350	426	300	444	468					
Horiz. Bottom raw	194	53	58	61	60					
Horiz. Bottom adj	1164	318	348	366	360					

Stringer 2	Length:	72.8"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans:	22

Stringer 3	Length:	72.8"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans:	20-22

Stringer 4	Length:	72.5"	Depth:	6.0"	Width:	2.0"	Cont. Across Spans:	19-22		
Location:	2"	27"	49"	69"						
Horiz. Top raw	73	81	134	140						
Horiz. Top adj	438	486	804	840						
Horiz. Bottom raw	92	97	87	124						
Horiz. Bottom adj	552	582	522	744						

Table 1 Continued: Miles Canyon Bridge SWT Data 28 Oct 2015

AP1 Span 1 — **Approach Sill** — **Approach Bent**

Stringer 1	Length:	98.0"	Depth:	6.0"	Width:	4.0"			
Location:		21"	45"	69"	93"				
Horiz. Top raw		171	231	225	112				
Horiz. Top adj		513	693	675	336				
Horiz. Bottom raw		145	221	168	168				
Horiz. Bottom adj		435	663	504	504				

Stringer 2	Length:	98.0"	Depth:	6.0"	Width:	2.0"			
Location:		21"	45"	69"	93"				
Horiz. Top raw		162	115	84	81				
Horiz. Top adj		972	690	504	486				
Horiz. Bottom raw		168	106	145	115				
Horiz. Bottom adj		1008	636	870	690				

Stringer 3	Length:	98.0"	Depth:	6.0"	Width:	2.0"			
Location:		21"	45"	69"	93"				
Horiz. Top raw		177	116	120	80				
Horiz. Top adj		1062	696	720	480				
Horiz. Bottom raw		109	110	116	87				
Horiz. Bottom adj		654	660	696	522				

Stringer 4	Length:	98.0"	Depth:	6.0"	Width:	4.0"			
Location:		21"	45"	69"	93"				
Horiz. Top raw		179	125	123	144				
Horiz. Top adj		537	375	369	432				
Horiz. Bottom raw		150	142	130	173				
Horiz. Bottom adj		450	426	390	519				

Table 1 Continued: Miles Canyon Bridge SWT Data 28 Oct 2015

AP1 Span 2

Approach Bent — AB1

Stringer 1	Length:	187.0"	Depth:	6.0"	Width:	2.0"				
Location:	20"	44"	68"	192"	116"	140"	164"	176"		
Horiz. Top raw	120	130	152	185	96	98	102	110		
Horiz. Top adj	720	780	912	1110	576	588	612	660		
Horiz. Bottom raw	53	115	89	76	82	86	79	92		
Horiz. Bottom adj	318	690	534	456	492	516	474	552		

Stringer 2	Length:	187.0"	Depth:	6.0"	Width:	2.0"				
Location:	20"	44"	68"	192"	116"	140"	164"	176"		
Horiz. Top raw	99	98	95	125	130	107	98	100		
Horiz. Top adj	594	588	570	750	780	642	588	600		
Horiz. Bottom raw	110	82	100	100	122	101	90	72		
Horiz. Bottom adj	660	492	600	600	732	606	540	432		

Stringer 3	Length:	187.0"	Depth:	6.0"	Width:	2.0"				
Location:	20"	44"	68"	192"	116"	140"	164"	176"		
Horiz. Top raw	106	90	103	124	90	96	106	110		
Horiz. Top adj	636	540	618	744	540	576	636	660		
Horiz. Bottom raw	60	83	104	79	86	80	76	87		
Horiz. Bottom adj	360	498	624	474	516	480	456	522		

Stringer 4	Length:	187.0"	Depth:	6.0"	Width:	2.0"				
Location:	20"	44"	68"	192"	116"	140"	164"	176"		
Horiz. Top raw	94	81	108	99	101	113	104	98		
Horiz. Top adj	564	486	648	594	606	678	624	588		
Horiz. Bottom raw	51	69	70	88	85	78	102	97		
Horiz. Bottom adj	306	414	420	528	510	468	612	582		

AP2

AB2 — Approach Sill

Stringer 1	Length:	96.0"	Depth:	6.0"	Width:	4.0"				
Location:	2"	24"	48"	72"	94"					
Horiz. Top raw	163	168	177	139	149					
Horiz. Top adj	489	504	531	417	447					
Horiz. Bottom raw	129	147	118	117	138					
Horiz. Bottom adj	387	441	354	351	414					

Stringer 2	Length:	96.0"	Depth:	6.0"	Width:	4.0"				
Location:	9"	24"	48"	72"	94"					
Horiz. Top raw	137	129	126	158	140					
Horiz. Top adj	411	387	378	474	420					
Horiz. Bottom raw	107	106	112	115	124					
Horiz. Bottom adj	321	318	336	345	372					

Stringer 3	Length:	96.0"	Depth:	6.0"	Width:	4.0"				
Location:	2"	24"	48"	72"	94"					
Horiz. Top raw	104	66	82	80	88					
Horiz. Top adj	312	198	246	240	264					
Horiz. Bottom raw	120	138	183	131	143					
Horiz. Bottom adj	360	414	549	393	429					

Table 1 Continued: Miles Canyon Bridge SWT Data 28 Oct 2015

Cross Beams

Length and locations measured from end of beam at Side 1 of bridge
Horizontal readings taken at centerline of beam
Vertical readings taken at centerline or 1 inch from Side 1 or Side 2 of beam

AB 1	Length:	72.0"	Depth:	6.0"	Width:	6.0"				
Location:	4"		24"			48"		68"		
Horiz. raw	209		239			159		148		
Horiz. adj	418		478			318		296		

Beam 1	Length:	72.0"	Depth:	6.0"	Width:	6.0"				
Location:	4"	8"	16"	32"	40"	56"	64"	68"		
Horiz. raw	786	1015	729	315	266	229	488	309		
Horiz. adj	1572	2030	1458	630	532	458	976	618		
Vert. raw	528	739	549	365	352	262	405	341		
Vert. adj	1056	1478	1098	730	704	524	810	682		
Vert. Side 1 raw	764	810					258	290		
Vert Side 1 adj	1528	1620					516	580		
Vert. Side 2 raw	313	406					427	339		
Vert Side 2 adj	626	812					854	678		

Beam 2	Length:	72.0"	Depth:	6.0"	Width:	6.0"				
Location:	4"	8"	16"	32"	40"	56"	64"	68"		
Horiz. raw	269	268	265	256	172	142	550	387		
Horiz. adj	538	536	530	512	344	284	1100	774		
Vert. raw	168	239	189	284	264	235	233	276		
Vert. adj	336	478	378	568	528	470	466	552		
Vert. Side 1 raw	207	223					3100	394		
Vert Side 1 adj	414	446					6200	788		
Vert. Side 2 raw	256	293					591	888		
Vert Side 2 adj	512	586					1182	1776		

Beam 3	Length:	72.0"	Depth:	6.0"	Width:	6.0"				
Location:	4"	8"	16"	32"	40"	56"	64"	68"		
Horiz. raw	336	347	266	258	185	438	540	380		
Horiz. adj	672	694	532	516	370	876	1080	760		
Vert. raw	233	250	191	194	289	234	577	374		
Vert. adj	466	500	382	388	578	468	1154	748		
Vert. Side 1 raw	203	156					215	321		
Vert Side 1 adj	406	312					430	642		
Vert. Side 2 raw	325	213					308	390		
Vert Side 2 adj	650	426					616	780		

Beam 4	Length:	72.0"	Depth:	6.0"	Width:	6.0"				
Location:	4"	8"	16"	32"	40"	56"	64"	68"		
Horiz. raw	216	174	208			323	252	232		
Horiz. adj	432	348	416			646	504	464		
Vert. raw	297	395					452	258		
Vert. adj	594	790					904	516		

Table 1 Continued: Miles Canyon Bridge SWT Data 28 Oct 2015

Beam 5	Length:	72.0"	Depth:	6.0"	Width:	6.0"				
Location:	4"	8"	16"	32"	40"	56"	64"	68"		
Horiz. raw	205	246	293	191	219	257	263	578		
Horiz. adj	410	492	586	382	438	514	526	1156		
Vert. raw	253	172	162	146	260	260	224	794		
Vert. adj	506	344	324	292	520	520	448	1588		
Vert. Side 1 raw	194	285					241	413		
Vert Side 1 adj	388	570					482	826		
Vert. Side 2 raw	174	186					339	408		
Vert Side 2 adj	348	372					678	816		

Beam 6	Length:	72.0"	Depth:	6.0"	Width:	6.0"				
Location:	4"	8"	16"	32"	40"	56"	64"	68"		
Horiz. raw	282	372	287			208	336	424		
Horiz. adj	564	744	574			416	672	848		
Vert. raw	355	363					368	336		
Vert. adj	710	726					736	672		
Vert. Side 1 raw							295	425		
Vert Side 1 adj							590	850		
Vert. Side 2 raw							278	336		
Vert Side 2 adj							556	672		

Beam 7	Length:	72.0"	Depth:	6.0"	Width:	6.0"				
Location:	4"	8"	16"	32"	40"	56"	64"	68"		
Horiz. raw	182	194	279			180	194	205		
Horiz. adj	364	388	558			360	388	410		
Vert. raw	179	252					212	445		
Vert. adj	358	504					424	890		
Vert. Side 1 raw							270	283		
Vert Side 1 adj							540	566		
Vert. Side 2 raw							324	290		
Vert Side 2 adj							648	580		

Beam 8	Length:	72.0"	Depth:	6.0"	Width:	6.0"				
Location:	4"	8"	16"	32"	40"	56"	64"	68"		
Horiz. raw	712	4011	7021			339	699	713		
Horiz. adj	1424	8022	14042			678	1398	1426		
Vert. raw	1379	3509					705	628		
Vert. adj	2758	7018					1410	1256		
Vert. Side 1 raw							526	1319		
Vert Side 1 adj							1052	2638		
Vert. Side 2 raw							784	1547		
Vert Side 2 adj							1568	3094		

Beam 9	Length:	72.0"	Depth:	6.0"	Width:	6.0"				
Location:	4"	8"	16"	32"	40"	56"	64"	68"		
Horiz. raw	387	485	8325			333	355	452		
Horiz. adj	774	970	16650			666	710	904		
Vert. raw	884	1202					440	410		
Vert. adj	1768	2404					880	820		

Table 1 Continued: Miles Canyon Bridge SWT Data 28 Oct 2015

Beam 10	Length:	72.0"	Depth:	6.0"	Width:	6.0"				
Location:	4"	8"	16"	32"	40"	56"	64"	68"		
Horiz. raw	197	181	273			370	260	233		
Horiz. adj	394	362	546			740	520	466		
Vert. raw	267	322					451	408		
Vert. adj	534	644					902	816		

Beam 11	Length:	72.0"	Depth:	6.0"	Width:	6.0"				
Location:	4"	8"	16"	32"	40"	56"	64"	68"		
Horiz. raw	277	298	303	208	277	742	1282	610		
Horiz. adj	554	596	606	416	554	1484	2564	1220		
Vert. raw	421	264	176	203	220	1024	1000	850		
Vert. adj	842	528	352	406	440	2048	2000	1700		
Vert. Side 1 raw	353	277					279	327		
Vert Side 1 adj	706	554					558	654		
Vert. Side 2 raw	319	272					1086	757		
Vert Side 2 adj	638	544					2172	1514		

Beam 12	Length:	72.0"	Depth:	6.0"	Width:	6.0"				
Location:	4"	8"	16"	32"	40"	56"	64"	68"		
Horiz. raw	203	527	484			417	623	270		
Horiz. adj	406	1054	968			834	1246	540		
Vert. raw	289	520					590	500		
Vert. adj	578	1040					1180	1000		

Beam 13	Length:	72.0"	Depth:	6.0"	Width:	6.0"				
Location:	4"	8"	16"	32"	40"	56"	64"	68"		
Horiz. raw	140	167	204			327	325	318		
Horiz. adj	280	334	408			654	650	636		
Vert. raw	176	302					358	312		
Vert. adj	352	604					716	624		

Beam 14	Length:	72.0"	Depth:	6.0"	Width:	6.0"				
Location:	4"	8"	16"	32"	40"	56"	64"	68"		
Horiz. raw	163	162	239			319	198	280		
Horiz. adj	326	324	478			638	396	560		
Vert. raw	159	293					457	482		
Vert. adj	318	586					914	964		

Beam 15	Length:	72.0"	Depth:	6.0"	Width:	6.0"				
Location:	4"	8"	16"	32"	40"	56"	64"	68"		
Horiz. raw	271	293	181			352	351	320		
Horiz. adj	542	586	362			704	702	640		
Vert. raw	200	313					355	251		
Vert. adj	400	626					710	502		

Beam 16	Length:	72.0"	Depth:	6.0"	Width:	6.0"				
Location:	4"	8"	16"	32"	40"	56"	64"	68"		
Horiz. raw	175	156	231			164	180	166		
Horiz. adj	350	312	462			328	360	332		
Vert. raw	223	184					184	174		
Vert. adj	446	368					368	348		

Table 1 Continued: Miles Canyon Bridge SWT Data 28 Oct 2015

Beam 17	Length:	72.0"	Depth:	6.0"	Width:	6.0"				
Location:	4"	8"	16"	32"	40"	56"	64"	68"		
Horiz. raw	517	842	776			267	627	507		
Horiz. adj	1034	1684	1552			534	1254	1014		
Vert. raw	507	1181					965	736		
Vert. adj	1014	2362					1930	1472		

Beam 18	Length:	72.0"	Depth:	6.0"	Width:	6.0"				
Location:	4"	8"	16"	32"	40"	56"	64"	68"		
Horiz. raw	274	277	235			246	129	161		
Horiz. adj	548	554	470			492	258	322		
Vert. raw	327	327					236	181		
Vert. adj	654	654					472	362		

Beam 19	Length:	72.0"	Depth:	6.0"	Width:	6.0"				
Location:	4"	8"	16"	32"	40"	56"	64"	68"		
Horiz. raw	229	258	231	149	121	173	150	170		
Horiz. adj	458	516	462	298	242	346	300	340		
Vert. raw	200	181	173	163	126	181	254	164		
Vert. adj	400	362	346	326	252	362	508	328		
Vert. Side 1 raw	187	196					179	184		
Vert Side 1 adj	374	392					358	368		
Vert. Side 2 raw	200	170					198	212		
Vert Side 2 adj	400	340					396	424		

Beam 20	Length:	72.0"	Depth:	6.0"	Width:	6.0"				
Location:	4"	8"	16"	32"	40"	56"	64"	68"		
Horiz. raw	250	244	204	134	155	236	389	320		
Horiz. adj	500	488	408	268	310	472	778	640		
Vert. raw	250	225	180	128	151	146	460	879		
Vert. adj	500	450	360	256	302	292	920	1758		
Vert. Side 1 raw	227	216					266	298		
Vert Side 1 adj	454	432					532	596		
Vert. Side 2 raw	238	192					636			
Vert Side 2 adj	476	384					1272			

Beam 21	Length:	72.0"	Depth:	6.0"	Width:	6.0"				
Location:	4"	8"	16"	32"	40"	56"	64"	68"		
Horiz. raw	212	300	170	171	174	362	1402	411		
Horiz. adj	424	600	340	342	348	724	2804	822		
Vert. raw	195	196	166	190	194	287	516	432		
Vert. adj	390	392	332	380	388	574	1032	864		
Vert. Side 1 raw	261	250					536	474		
Vert Side 1 adj	522	500					1072	948		
Vert. Side 2 raw	205	426					684	682		
Vert Side 2 adj	410	852					1368	1364		

AB2	Length:	72.0"	Depth:	6.0"	Width:	6.0"				
Location:	4"		24"			48"		68"		
Horiz. raw	158		171			210		183		
Horiz. adj	316		342			420		366		

Table 1 Continued: Miles Canyon Bridge SWT Data 28 Oct 2015

AB1

Posts

Height and locations measured from bottom of cap

Readings readings centerline of post

Post 1A	Height: 173.5"		3/9: 12.0"		6/12: 12.0"				
Location:	16"	26"	48"	72"	88"	102"	120"	144"	165"	172"
6/12 raw		450	291	249	267	288	224	230	311	284
6/12 adj		450	291	249	267	288	224	230	311	284
3/9 raw	394	351	321	246	307	338	316	430	223	
3/9 adj	394	351	321	246	307	338	316	430	223	

Post 1B	Height: 173.5"		3/9: 12.0"		6/12: 12.0"					
Location:	16"	24"	48"	72"	88"	102"	120"	144"	165"	172"
6/12 raw		244	457	277	523	240	236	304	330	334
6/12 adj		244	457	277	523	240	236	304	330	334
3/9 raw	258	457	348	466	395	300	364	274	341	
3/9 adj	258	457	348	466	395	300	364	274	341	

Post 2A	Height: 173.5"		3/9: 12.0"		6/12: 12.0"					
Location:	16"	24"	48"	72"	88"	102"	120"	144"	165"	171"
6/12 raw		279	260	276	340	281	327	284	338	268
6/12 adj		279	260	276	340	281	327	284	338	268
3/9 raw	250	254	312	327	319	298	450	276	303	
3/9 adj	250	254	312	327	319	298	450	276	303	

Post 2B	Height: 173.5"		3/9: 12.0"		6/12: 12.0"					
Location:	16"	24"	48"	72"	88"	102"	120"	144"	165"	171"
6/12 raw		604	372	365	496	400	420	352	370	318
6/12 adj		604	372	365	496	400	420	352	370	318
3/9 raw	563	309	338	367	325	492	310	336	370	
3/9 adj	563	309	338	367	325	492	310	336	370	

Post 3A	Height: 173.5"		3/9: 12.0"		6/12: 12.0"					
Location:	16"	24"	48"	72"	88"	102"	120"	144"	165"	171"
6/12 raw		424	350	272	299	265	266	265	273	212
6/12 adj		424	350	272	299	265	266	265	273	212
3/9 raw	325	314	376	352	268	238	277	255	279	
3/9 adj	325	314	376	352	268	238	277	255	279	

Post 3B	Height: 173.5"		3/9: 12.0"		6/12: 12.0"					
Location:	16"	24"	48"	72"	88"	102"	120"	144"	165"	171"
6/12 raw		241	245	321	297	305	637	398	348	465
6/12 adj		241	245	321	297	305	637	398	348	465
3/9 raw	273	245	247	305	454	335	284	284	360	
3/9 adj	273	245	247	305	454	335	284	284	360	

Post 4A	Height: 173.5"		3/9: 12.0"		6/12: 12.0"					
Location:	16"	24"	48"	72"	88"	102"	120"	144"	165"	171"
6/12 raw		288	294	305	255	243	263	277	345	371
6/12 adj		288	294	305	255	243	263	277	345	371
3/9 raw	204	266	249	326	263	308	245	293	291	
3/9 adj	204	266	249	326	263	308	245	293	291	

Table 1 Continued: Miles Canyon Bridge SWT Data 28 Oct 2015										
Post 4B	Height: 173.5"		3/9: 12.0"		6/12: 12.0"					
Location:	16"	24"	48"	72"	88"	102"	120"	144"	165"	171"
6/12 raw		262	278	306	284	342	397	254	286	275
6/12 adj		262	278	306	284	342	397	254	286	275
3/9 raw	263	277	277	289	265	504	286	360	330	
3/9 adj	263	277	277	289	265	504	286	360	330	

Horizontal Members

Length and locations measured from end of member at Side 1 or End 1 of bridge

Readings taken at centerline of member or 2 inches from edge, as noted

Cap	Length: 154.0"		Depth: 12.0"		Width: 12.0"					
Location:	4"	24"	48"	72"	96"	120"	144"	150"		
Horiz. Top raw	386	420	387	416	372	390	498	545		
Horiz. Top adj	386	420	387	416	372	390	498	545		
Horiz. Bottom raw	234	491	480	311	380	328		308		
Horiz. Bottom adj	234	491	480	311	380	328		308		

Sub Cap	Length: 96.0"		Depth: 12.0"		Width: 12.0"					
Location:	4"	24"	48"	72"	92"					
Vert. raw	294				610					
Vert. adj	294				610					
Vert. Side A raw		422	411	386						
Vert. Side A adj		422	411	386						
Vert. Side B raw		489	428	418						
Vert. Side B adj		489	428	418						
Horiz. raw	444	270	251	244	854					
Horiz. adj	444	270	251	244	854					
Horiz. Bottom raw		480	738	432						
Horiz. Bottom adj		480	738	432						

Table 1 Continued: Miles Canyon Bridge SWT Data 28 Oct 2015

Sill 1	Length:	126.0"	Depth:	12.0"	Width:	12.0"					
Location:	4"	12"	17"	22"	26"	48"	72"	96"	104"	109"	114"	122"
Vert. raw	352				293	255	241	275				303
Vert. adj	352				293	255	241	275				303
Horiz. raw	316		332		277	264	300	288		312		321
Horiz. adj	316		332		277	264	300	288		312		321
Horiz. Top raw		297		307					297		383	
Horiz. Top adj		297		307					297		383	
Horiz. Bottom raw			257							292		
Horiz. Bottom adj			257							292		

Sill 2	Length:	142.0"	Depth:	12.0"	Width:	12.0"									
Location:	4"	24"	28"	33"	38"	48"	60"	70"	78"	96"	117"	120"	125"	129"	138"
Vert. raw	666	521				382	281		338	307	362				349
Vert. adj	666	521				382	281		338	307	362				349
Horiz. raw	363	327		341		328	313	378	296	260	290		309		382
Horiz. adj	363	327		341		328	313	378	296	260	290		309		382
Horiz. Top raw			359		292			340				319		339	
Horiz. Top adj			359		292			340				319		339	
Horiz. Bottom raw				316				494					315		
Horiz. Bottom adj				316				494					315		

Sill 3	Length:	142.0"	Depth:	12.0"	Width:	12.0"									
Location:	4"	24"	28"	33"	38"	48"	60"	70"	78"	96"	117"	120"	125"	129"	138"
Vert. raw	369	354				297	347		359	322	329				486
Vert. adj	369	354				297	347		359	322	329				486
Horiz. raw	762	562		364		260	272	290	262	255	255		259		657
Horiz. adj	762	562		364		260	272	290	262	255	255		259		657
Horiz. Top raw			269		284			237				260		352	
Horiz. Top adj			269		284			237				260		352	
Horiz. Bottom raw				276				336					478		
Horiz. Bottom adj				276				336					478		

Sill 4	Length:	126.0"	Depth:	12.0"	Width:	12.0"					
Location:	4"	12"	17"	22"	26"	48"	72"	96"	104"	109"	114"	122"
Vert. raw	295				250	236	264	248				472
Vert. adj	295				250	236	264	248				472
Horiz. raw	643		324		268	275	318	551		331		376
Horiz. adj	643		324		268	275	318	551		331		376
Horiz. Top raw		293		259					325		302	
Horiz. Top adj		293		259					325		302	
Horiz. Bottom raw			320							303		
Horiz. Bottom adj			320							303		

Sill A	Length:	241.0"	Depth:	12.0"	Width:	12.0"										
Location:	4"	21"	29"	37"	48"	72"	84"	96"	120"	144"	156"	168"	192"	211"	219"	237"
Vert. raw	311	336		329	222	270		436	232	254		306	367		497	354
Vert. adj	311	336		329	222	270		436	232	254		306	367		497	354
Horiz. raw	299	351	342	270	288	315	338	314	266	272	303	358	325	340	363	320
Horiz. adj	299	351	342	270	288	315	338	314	266	272	303	358	325	340	363	320
Horiz. Top raw			308				524				476			367		
Horiz. Top adj			308				524				476			367		
Horiz. Bottom raw			365				280				276			265		
Horiz. Bottom adj			365				280				276			265		

Sill B	Length:	241.0"	Depth:	12.0"	Width:	12.0"										
Location:	4"	21"	29"	37"	48"	72"	84"	96"	120"	144"	156"	168"	192"	211"	219"	237"
Vert. raw	288	404		438	400	312		295	277	293		470	278		393	330
Vert. adj	288	404		438	400	312		295	277	293		470	278		393	330
Horiz. raw	360	292	277	292	299	283	312	255	278	272	312	355	265	278	468	358
Horiz. adj	360	292	277	292	299	283	312	255	278	272	312	355	265	278	468	358
Horiz. Top raw			432				339				394			323		
Horiz. Top adj			432				339				394			323		
Horiz. Bottom raw			237				365				368			355		
Horiz. Bottom adj			237				365				368			355		

Table 1 Continued: Miles Canyon Bridge SWT Data 28 Oct 2015

Subsill 1	Length:	144.0"	Depth:	12.0"	Width:	12.0"						
Location:	4"	17"	25"	33"	48"	72"	96"	111"	119"	122"	127"	140"
Vert. raw	353			301	304	518	313	551				473
Vert. adj	353			301	304	518	313	551				473
Horiz. raw	444	461	447	216	338	486	422	453		392	563	431
Horiz. adj	444	461	447	216	338	486	422	453		392	563	431
Horiz. Top raw			422						598			
Horiz. Top adj			422						598			
Horiz. Bottom raw			291						256			
Horiz. Bottom adj			291						256			

Subsill 2	Length:	144.0"	Depth:	12.0"	Width:	12.0"						
Location:	4"	17"	25"	33"	48"	72"	96"	111"	119"	122"	127"	140"
Vert. raw	480			550	364	334	434	411				390
Vert. adj	480			550	364	334	434	411				390
Horiz. raw	639	344	400	396	418	327	443	402		541	374	646
Horiz. adj	639	344	400	396	418	327	443	402		541	374	646
Horiz. Top raw			537						441			
Horiz. Top adj			537						441			
Horiz. Bottom raw			852						324			
Horiz. Bottom adj			852						324			

Subsill 3	Length:	144.0"	Depth:	12.0"	Width:	12.0"						
Location:	4"	17"	25"	33"	48"	72"	96"	111"	119"	122"	127"	140"
Vert. raw	577			340	493	287	376	395				345
Vert. adj	577			340	493	287	376	395				345
Horiz. raw	601	323	343	365	324	328	357	303		454	333	405
Horiz. adj	601	323	343	365	324	328	357	303		454	333	405
Horiz. Top raw			532						262			
Horiz. Top adj			532						262			
Horiz. Bottom raw			372						458			
Horiz. Bottom adj			372						458			

Subsill 4	Length:	144.0"	Depth:	12.0"	Width:	12.0"						
Location:	4"	17"	25"	33"	48"	72"	96"	111"	119"	122"	127"	140"
Vert. raw	344			433	425	334	374	448				432
Vert. adj	344			433	425	334	374	448				432
Horiz. raw	504	272	299	306	339	342	301	385		393	346	587
Horiz. adj	504	272	299	306	339	342	301	385		393	346	587
Horiz. Top raw			452						283			
Horiz. Top adj			452						283			
Horiz. Bottom raw			392						627			
Horiz. Bottom adj			392						627			

Table 1 Continued: Miles Canyon Bridge SWT Data 28 Oct 2015

Crosspiece A1	Length:	22.0"	Depth:	12.0"	Width:	12.0"				
Location:	6"	16"								
Vert. raw	512	397								
Vert. adj	512	397								
Horiz. raw	381	379								
Horiz. adj	381	379								

Crosspiece A2	Length:	22.0"	Depth:	12.0"	Width:	12.0"				
Location:	6"	16"								
Vert. raw	343	413								
Vert. adj	343	413								
Horiz. raw	488	418								
Horiz. adj	488	418								

Crosspiece B1	Length:	22.0"	Depth:	12.0"	Width:	12.0"				
Location:	6"	16"								
Vert. raw	593	714								
Vert. adj	593	714								
Horiz. raw	445	385								
Horiz. adj	445	385								

Crosspiece B2	Length:	22.0"	Depth:	12.0"	Width:	12.0"				
Location:	6"	16"								
Vert. raw	352	359								
Vert. adj	352	359								
Horiz. raw	377	471								
Horiz. adj	377	471								

Crosspiece 1	Length:	46.0"	Depth:	12.0"	Width:	12.0"				
Location:	6"	24"	40"							
Vert. raw	604	3623	706							
Vert. adj	604	3623	706							
Horiz. raw	636	813	684							
Horiz. adj	636	813	684							

Crosspiece 2	Length:	46.0"	Depth:	12.0"	Width:	12.0"				
Location:	6"	24"	40"							
Vert. raw	479	463	753							
Vert. adj	479	463	753							
Horiz. raw	754	744	763							
Horiz. adj	754	744	763							

Crosspiece 3	Length:	46.0"	Depth:	12.0"	Width:	12.0"				
Location:	6"	24"	40"							
Vert. raw	392	593	329							
Vert. adj	392	593	329							
Horiz. raw	488	588	331							
Horiz. adj	488	588	331							

Crosspiece 4	Length:	46.0"	Depth:	12.0"	Width:	12.0"				
Location:	6"	24"	40"							
Vert. raw	593	358	346							
Vert. adj	593	358	346							
Horiz. raw	585	464	430							
Horiz. adj	585	464	430							

Table 1 Continued: Miles Canyon Bridge SWT Data 28 Oct 2015

AB2

Posts

Height and locations measured from bottom of cap

Readings readings centerline of post

Post 1A	Height: 173.5"	3/9: 12.0"	6/12: 12.0"							
Location:	16"	24"	48"	72"	88"	103"	120"	144"	165"	171"
6/12 raw		487	271	565	372	419	330	240	498	281
6/12 adj		487	271	565	372	419	330	240	498	281
3/9 raw	328	302	257	396	300	260	276	275	280	
3/9 adj	328	302	257	396	300	260	276	275	280	

Post 1B	Height: 173.5"	3/9: 12.0"	6/12: 12.0"							
Location:	16"	24"	48"	72"	88"	103"	120"	144"	165"	171"
6/12 raw		430	378	390	365	468	361	304	325	412
6/12 adj		430	378	390	365	468	361	304	325	412
3/9 raw	282	280	405	412	486	442	305	309	430	
3/9 adj	282	280	405	412	486	442	305	309	430	

Post 2A	Height: 173.5"	3/9: 12.0"	6/12: 12.0"							
Location:	16"	24"	48"	72"	88"	103"	120"	144"	165"	171"
6/12 raw		610	238	533	521	500	527	393	681	331
6/12 adj		610	238	533	521	500	527	393	681	331
3/9 raw	318	510	520	610	303	642	520	451	658	
3/9 adj	318	510	520	610	303	642	520	451	658	

Post 2B	Height: 173.5"	3/9: 12.0"	6/12: 12.0"							
Location:	16"	24"	48"	72"	88"	103"	120"	144"	165"	171"
6/12 raw		266	264	230	304	240	291	286	319	287
6/12 adj		266	264	230	304	240	291	286	319	287
3/9 raw	322	274	370	412	312	215	214	230	317	
3/9 adj	322	274	370	412	312	215	214	230	317	

Post 3A	Height: 173.5"	3/9: 12.0"	6/12: 12.0"							
Location:	16"	24"	48"	72"	88"	103"	120"	144"	165"	171"
6/12 raw		571	432	221	272	286	216	263	250	270
6/12 adj		571	432	221	272	286	216	263	250	270
3/9 raw	290	264	280	256	306	269	253	238	254	
3/9 adj	290	264	280	256	306	269	253	238	254	

Post 3B	Height: 173.5"	3/9: 12.0"	6/12: 12.0"							
Location:	16"	24"	48"	72"	88"	103"	120"	144"	165"	171"
6/12 raw		253	273	318	305	309	255	579	415	377
6/12 adj		253	273	318	305	309	255	579	415	377
3/9 raw	234	274	281	265	415	484	371	350	665	
3/9 adj	234	274	281	265	415	484	371	350	665	

Table 1 Continued: Miles Canyon Bridge SWT Data 28 Oct 2015

Post 4A	Height:	173.5"	3/9:	12.0"	6/12:	12.0"				
Location:	16"	24"	48"	72"	88"	103"	120"	144"	165"	171"
6/12 raw		380	322	274	420	544	264	255	301	486
6/12 adj		380	322	274	420	544	264	255	301	486
3/9 raw	294	332	274	240	236	653	227	249	289	
3/9 adj	294	332	274	240	236	653	227	249	289	

Post 4B	Height:	173.5"	3/9:	12.0"	6/12:	12.0"				
Location:	16"	24"	48"	72"	88"	103"	120"	144"	165"	171"
6/12 raw		420	248	281	273	288	236	315	302	273
6/12 adj		420	248	281	273	288	236	315	302	273
3/9 raw	288	261	265	332	345	241	250	236	262	
3/9 adj	288	261	265	332	345	241	250	236	262	

Horizontal Members

Length and locations measured from end of member at Side 1 or End 1 of bridge

Readings taken at centerline of member or 2 inches from edge, as noted

Cap	Length:	156.0"	Depth:	12.0"	Width:	12.0"				
Location:	4"	24"	48"	72"	96"	120"	144"	152"		
Horiz. Top raw	436	398	357	437	603	346	346	397		
Horiz. Top adj	436	398	357	437	603	346	346	397		
Horiz. Bottom raw	417		920	720	864	797		936		
Horiz. Bottom adj	417		920	720	864	797		936		

Sub Cap	Length:	99.0"	Depth:	12.0"	Width:	12.0"				
Location:	4"	24"	48"	72"	96"					
Vert. raw	650				819					
Vert. adj	650				819					
Vert. Side A raw		745	500	624						
Vert. Side A adj		745	500	624						
Vert. Side B raw		604	582	739						
Vert. Side B adj		604	582	739						
Horiz. raw	490	1047	537	700	1285					
Horiz. adj	490	1047	537	700	1285					

Table 1 Continued: Miles Canyon Bridge SWT Data 28 Oct 2015

Sill 1	Length:	126.0"	Depth:	12.0"	Width:	12.0"				
Location:	4"	24"	48"	72"	96"	120"				
Vert. raw	552	450	556	318	380	264				
Vert. adj	552	450	556	318	380	264				
Horiz. raw	528	245	289	493	412	324				
Horiz. adj	528	245	289	493	412	324				

Sill 2	Length:	126.0"	Depth:	12.0"	Width:	12.0"				
Location:	4"	24"	48"	72"	96"	120"				
Vert. raw	360	387	450	385	397	343				
Vert. adj	360	387	450	385	397	343				
Horiz. raw	384	320	280	284	300	295				
Horiz. adj	384	320	280	284	300	295				

Sill 3	Length:	126.0"	Depth:	12.0"	Width:	12.0"				
Location:	4"	24"	48"	72"	96"	120"				
Vert. raw	456	469	311	500	512	334				
Vert. adj	456	469	311	500	512	334				
Horiz. raw	431	518	340	332	357	420				
Horiz. adj	431	518	340	332	357	420				

Sill 4	Length:	126.0"	Depth:	12.0"	Width:	12.0"				
Location:	4"	24"	48"	72"	96"	120"				
Vert. raw	385	382	527	405	378	487				
Vert. adj	385	382	527	405	378	487				
Horiz. raw	413	524	320	350	288	403				
Horiz. adj	413	524	320	350	288	403				

Crosspiece A1	Length:	22.0"	Depth:	12.0"	Width:	12.0"				
Location:	6"	16"								
Vert. raw	394	420								
Vert. adj	394	420								
Horiz. raw	510	527								
Horiz. adj	510	527								

Crosspiece A2	Length:	22.0"	Depth:	12.0"	Width:	12.0"				
Location:	6"	16"								
Vert. raw	743	953								
Vert. adj	743	953								
Horiz. raw	550	992								
Horiz. adj	550	992								

Table 1 Continued: Miles Canyon Bridge SWT Data 28 Oct 2015

Crosspiece B1	Length: 22.0"	Depth: 12.0"	Width: 12.0"							
Location:	6"	16"								
Vert. raw	593	373								
Vert. adj	593	373								
Horiz. raw	346	363								
Horiz. adj	346	363								

Crosspiece B2	Length: 22.0"	Depth: 12.0"	Width: 12.0"							
Location:	6"	16"								
Vert. raw	494	450								
Vert. adj	494	450								
Horiz. raw	452	508								
Horiz. adj	452	508								

Crosspiece 1	Length: 47.0"	Depth: 12.0"	Width: 12.0"							
Location:	6"	24"	41"							
Vert. raw	367	295	423							
Vert. adj	367	295	423							
Horiz. raw	434	636	482							
Horiz. adj	434	636	482							

Crosspiece 2	Length: 47.0"	Depth: 12.0"	Width: 12.0"							
Location:	6"	24"	41"							
Vert. raw	462	253	310							
Vert. adj	462	253	310							
Horiz. raw	580	290	314							
Horiz. adj	580	290	314							

Crosspiece 3	Length: 47.0"	Depth: 12.0"	Width: 12.0"							
Location:	6"	24"	41"							
Vert. raw	365	532	433							
Vert. adj	365	532	433							
Horiz. raw	281	233	420							
Horiz. adj	281	233	420							

Crosspiece 4	Length: 47.0"	Depth: 12.0"	Width: 12.0"							
Location:	6"	24"	41"							
Vert. raw	320	337	252							
Vert. adj	320	337	252							
Horiz. raw	377	290	509							
Horiz. adj	377	290	509							

Appendix C: Assay Sample Lab Results

Moisture Content -- ASTM D 4442

Project #:	**8529**		
Starting Date:	11/4/2015	Lab Temp	72
Ending Date:	11/4/2015	RH%	55
Oven temperature:	212 °F		
Recorded By:	SS		

Measuring Equipment			
Moisturemeter WRD # (optional):		Next calib. due date:	
Weight measurement WRD #:	123	Next calib. due date:	5/1/2016
Temperature measurement WRD #:	21	Next calib. due date:	May-16
The measurement of uncertainty (MU) was calculated to be: 0.029			

	Specimen ID	Optinal moisturemeter reading (%)	Initial weight (g)	Initial Volume	Initial Diameter	Initial Length	Weight (g)	Weight (g)	Weight (g)	Weight (g)	Weight (g)	Dry Volume	Minimum weight (g)	SG	Moisture content (%)	Minimum weight (g)	Calculated moisture content
			Date:	11/4/2015			11/4/2015						11/4/2015				
			Time:	9:00			1:00						3:00				
1	AB1S4S1IN		0.561	1.15			0.457					1.05	0.456	0.43	23	0.456	23.0%
2	AB1S4S1OUT		0.778	1.15			0.663					1.15	0.662	0.58	17.5	0.662	17.5%
3	AB1S4S2IN		0.645	1.2			0.542					1.15	0.543	0.47	18.8	0.543	18.8%
4	AB1S4S2OUT		0.493	0.75			0.419					0.75	0.418	0.56	17.9	0.418	17.9%
5	B24"S1IN		0.254	0.55			0.192					0.5	0.19	0.38	33.7	0.19	33.7%
6	B24"S1OUT		0.288	0.55			0.227					0.5	0.226	0.45	27.4	0.226	27.4%
7	B24"S2IN		0.305	0.6			0.241					0.55	0.242	0.44	26	0.242	26.0%
8	B24"S2OUT		0.344	0.65			0.268					0.6	0.269	0.45	27.9	0.269	27.9%
9	*B268"S1IN		0.084				0.063						0.063		33.3	0.063	33.3%
10	B268"S1OUT		0.184	0.4			0.156					0.4	0.155	0.39	18.7	0.155	18.7%
11	*B268"S2IN		0.147				0.123						0.121		21.5	0.121	21.5%
12	B268"S2OUT		0.303	0.45			0.226					0.4	0.226	0.57	34.1	0.226	34.1%
13																	
14																	
15																	
16																	
17																	
18																	
19																	
20																	
21																	
22																	
23																	
24																	
25																	
26																	
27																	
28																	
29																	
30																	
	*FINE CRUMBS															average:	25.0%

Moisture Content -- ASTM D 4442

Project #: **8529**

Starting Date: 11/5/2015 Lab Temp 72

Ending Date: 11/5/2015 RH% 55

Oven temperature: 212 °F

Recorded By: SS

Measuring Equipment			
Moisturemeter WRD # (optional):		Next calib. due date:	
Weight measurement WRD #:	123	Next calib. due date:	5/1/2016
Temperature measurement WRD #:	21	Next calib. due date:	May-16
The measurement of uncertainty (MU) was calculated to be: 0.029			

#	Specimen ID	Optinal moisturemeter reading (%)	Date:	11/5/2015			11/5/2015						11/5/2015			Minimum weight	Calculated moisture content
			Time:	8:40			12:40						2:40		Moisture		
			Initial weight (g)	Initial Volume	Initial Diameter	Initial Length	Weight (g)	Weight (g)	Weight (g)	Weight (g)	Weight (g)	Dry Volume	Minimum weight (g)	SG	content (%)	(g)	
1	CB38"VTIN		0.366	0.6			0.279					0.6	0.278	0.46	31.7	0.278	31.7%
2	CB38"VTOUT		0.197	0.45			0.167					0.45	0.166	0.37	18.7	0.166	18.7%
3	CB38VBIN		0.236	0.6			0.177					0.5	0.177	0.35	33.3	0.177	33.3%
4	CB38VBOUT		0.236	0.5			0.199					0.5	0.199	0.4	18.6	0.199	18.6%
5	CB366VTIN		0.307	0.7			0.258					0.65	0.257	0.4	19.5	0.257	19.5%
6	CB366VTOUT		0.227	0.55			0.198					0.5	0.199	0.4	14.1	0.199	14.1%
7	CB366VBIN		0.235	0.6			0.203					0.45	0.203	0.45	15.8	0.203	15.8%
8	CB366VBOUT		0.159	0.4			0.132					0.35	0.133	0.38	19.5	0.133	19.5%
9	*B1154TIN		0.087				0.073						0.072		20.8	0.072	20.8%
10	*B1154TOUT		0.118				0.098						0.098		20.4	0.098	20.4%
11	*B1154BIN		0.094				0.091						0.09		4.4	0.09	4.4%
12	B1154BOUT		0.218	0.4			0.181					0.35	0.181	0.52	20.4	0.181	20.4%
13																	
14			Date:	11/5/2015			11/5/2015						11/5/2015				
15			Time:	9:10			1:10						3:10				
16	B198TIN		0.292	0.7			0.236					0.65	0.235	0.36	24.3	0.235	24.3%
17	B198TOUT		0.25	0.65			0.209					0.55	0.207	0.38	20.8	0.207	20.8%
18	B198BIN		0.408	0.75			0.341					0.75	0.34	0.45	20	0.34	20.0%
19	B198BOUT		0.264	0.6			0.233					0.6	0.234	0.39	12.8	0.234	12.8%
20	B1966TIN		0.291	0.6			0.217					0.55	0.217	0.39	34.1	0.217	34.1%
21	B1966TOUT		0.251	0.55			0.2					0.45	0.2	0.44	25.5	0.2	25.5%
22	B1966BIN		0.29	0.65			0.243					0.6	0.242	0.4	19.8	0.242	19.8%
23	B1966BOUT		0.249	0.6			0.198					0.55	0.196	0.36	27	0.196	27.0%
24	**B20S28TIN		0.387	0.5			0.166					0.45	0.165	0.37	134.5	0.165	134.5%
25	**B20S28TOUT		0.742	0.7			0.233					0.6	0.233	0.39	218.5	0.233	218.5%
26	B20S28BIN		0.335	0.65			0.163					0.45	0.162	0.36	106.8	0.162	106.8%
27	B20S28BOUT		0.25	0.5			0.201					0.4	0.2	0.5	25	0.2	25.0%
28																	
29																	
30																	
																average:	37.8%

*FINE CRUMBS

**SMALL PIECES

Moisture Content -- ASTM D 4442

Project #: **8529**

Starting Date: 11/6/2015 Lab Temp 74

Ending Date: 11/6/2015 RH% 49

Oven temperature: 212 ^oF

Recorded By: SS

Measuring Equipment			
Moisturemeter WRD # (optional):		Next calib. due date:	
Weight measurement WRD #:	123	Next calib. due date:	5/1/2016
Temperature measurement WRD #:	21	Next calib. due date:	May-16
The measurement of uncertainty (MU) was calculated to be: 0.029			

	Specimen ID	Optinal moisturemeter reading (%)	Date:	11/6/2015												Minimum weight	Calculated moisture content
			Time:	8:30			10:30	12:30					2:30		Moisture		
			Initial weight (g)	Initial Volume	Initial Diameter	Initial Length	Weight (g)	Weight (g)	Weight (g)	Weight (g)	Weight (g)	Dry Volume	Minimum weight (g)	SG	content (%)	(g)	
1	*B20S266VTIN		0.258				0.115	0.114					0.114		126.3	0.114	126.3%
2	*B20S266VTOUT		0.233				0.112	0.109					0.108		115.7	0.108	115.7%
3	*B20S266VBIN		0.368				0.136	0.128					0.128		187.5	0.128	187.5%
4	*B20S266VBOUT		0.327				0.137	0.13					0.129		153.5	0.129	153.5%
5	SP3S124TIN		0.296	0.55			0.242	0.238				0.6	0.238	0.4	24.4	0.238	24.4%
6	SP3S124TOUT		0.308	0.5			0.26	0.253				0.45	0.252	0.56	22.2	0.252	22.2%
7	SP3S124BIN		0.314	0.65			0.271	0.256				0.7	0.257	0.37	22.2	0.257	22.2%
8	SP3S124BOUT		0.358	0.6			0.308	0.293				0.6	0.294	0.49	21.8	0.294	21.8%
9	SP3S224TIN		0.307	0.65			0.257	0.247				0.6	0.246	0.41	24.8	0.246	24.8%
10	SP3S224TOUT		0.209	0.3			0.171	0.158				0.25	0.158	0.63	32.3	0.158	32.3%
11	SP3S224BIN		0.328	0.65			0.281	0.265				0.6	0.265	0.44	23.8	0.265	23.8%
12	SP3S224BOUT		0.313	0.55			0.274	0.255				0.6	0.256	0.43	22.3	0.256	22.3%
13																	
14	Specimen ID		Date:	11/6/2015													
15			Time:	9:00			11:00	1:00					3:00				
16	SP3S324TIN		0.242	0.45			0.187	0.181				0.45	0.18	0.4	34.4	0.18	34.4%
17	SP3S324TOUT		0.212	0.4			0.169	0.159				0.4	0.16	0.4	32.5	0.16	32.5%
18	SP3S324BIN		0.399	0.7			0.337	0.332				0.7	0.332	0.47	20.2	0.332	20.2%
19	SP3S324BOUT		0.323	0.5			0.267	0.258				0.5	0.257	0.51	25.7	0.257	25.7%
20	SP3S424TIN		0.381	0.65			0.306	0.297				0.7	0.296	0.42	28.7	0.296	28.7%
21	SP3S424TOUT		0.399	0.6			0.333	0..321				0.55	0.321	0.58	24.3	0.321	24.3%
22	SP3S424BIN		0.359	0.55			0.306	0.293				0.55	0.294	0.53	22.2	0.294	22.1%
23	SP3S424BOUT		0.326	0.4			0.276	0.26				0.4	0.259	0.65	25.9	0.259	25.9%
24	SP19S148TIN		0.348	0.65			0.289	0.269				0.65	0.269	0.41	29.4	0.269	29.4%
25	SP19S148TOUT		0.305	0.55			0.261	0.25				0.5	0.249	0.5	22.5	0.249	22.5%
26	SP19S148BIN		0.314	0.65			0.266	0.252				0.65	0.252	0.39	24.6	0.252	24.6%
27	SP19S148BOUT		0.321	0.65			0.27	0.259				0.6	0.258	0.43	24.4	0.258	24.4%
28																	
29																	
30																	
	*Fine crumbs															average:	45.5%

Appendix D: Biography and Curriculum Vitae for Dr. Dan Tingley P.Eng. (Canada)

Daniel A. Tingley, Ph.D., P. Eng. Photograph from Discovery Channel series "How Stuff Works"

Dan Tingley serves as Executive Director for Wood Research & Development Ltd. He is the inventor of the award-winning FiRP® Panel reinforcement technique, which makes use of high-strength reinforced plastics to strengthen wood products. He holds a number of associated patents.

Here is a brief biography of Dr. Tingley. A complete curriculum vitae is available in PDF format with more information on his past experience.

ACADEMIC BACKGROUND:

- Ph.D. Oregon State University, 1997, Major in Forest Products, Minor in Civil Engineering
- M.Sc.C.E. University of New Brunswick, 1988, Structural Engineering in Wood
- B.Sc.F.E. University of New Brunswick, 1975, Forest Engineering

BRIEF CAREER SUMMARY:

Dr. Tingley has worked in the wood products industry for over 25 years. He received his Bachelor of Science in Forest Engineering and Master of Science in Civil Engineering from the University of New Brunswick. He completed his Ph.D. at Oregon State University in Wood Science, Technology and Civil Engineering.

Dr. Tingley currently holds more than 25 published patents in the reinforced wood field in the US and other countries. He has authored over 105 conference proceedings, publications, and articles in the area of reinforcement of wood and wood composites.

<u>SPECIAL AWARDS/PRIZES, DECORATIONS:</u>

- Charles Pankow Innovative Applications Award 1996
- NOVA Award for Innovation 1997 (only person to win NOVA and Charles Pankow Awards back to back, only person to win both awards in wood and high strength fibers)
- Applied Science Technologists & Technicians of British Columbia (ASTTBC) Award for advanced technology (worldwide competition), 1996
- Association of Professional Engineers of Nova Scotia design award for designing "Hector Heritage Quay," an all timber connector building with adjustable base connectors creating fixed and Moment connectors in a green wood situation.

<u>MEMBERSHIPS:</u>

- AITC (American Institute of Timber construction), Technical Activity Committee, Voting Member
- ASCE (American Society of Civil Engineers)
- APENB (Association of Professional Engineers of New Brunswick)
- APEBC (Association of Professional Engineers of British Columbia)
- APENS (Association of Professional Engineers of Nova Scotia) (License to Practice)
- APENZ (Association of Professional Engineers of New Zealand)
- ASTM (American Society Testing Materials), Voting Member
- International Bamboo Code for Structural Development Committee
- Canadian Forestry Association
- Canadian Society for Civil Engineering
- Forest Products Society
- Society of American Foresters
-

<u>PUBLICATIONS:</u>

More than 100 publications including:

- “Current State-of-the Art of Reinforcement Methodologies for Glued Laminated Timber” *Wood and Fiber Science,* 1998
- “The Effects of Creep on High Strength Fiber Reinforced Plastic Reinforced Douglas-fir Glulams”, *Wood and Fiber Science,* 1998
- “Geometric Considerations for Internal Fixed Moment Connectors for Glulams”, *Journal of Structural Engineering,* 1998
- “High Strength Fiber-Reinforced Plastic Reinforced Glulam Highway Bridges”, *Journal of Structural Engineering,* 1998

7.0 CURRICULUM VITAE - DANIEL A. TINGLEY

ACADEMIC BACKGROUND:

- Ph.D. Oregon State University, 1997, Major in Forest Products, Minor in Civil Engineering
- M.Sc.C.E. University of New Brunswick, 1988, Structural Engineering in Wood
- B.Sc.F.E. University of New Brunswick, 1975, Forest Engineering

CURRENT EMPLOYMENT:

Wood Research and Development Ltd.
PO Box 70
10476 Sunnyside Rd SE
Jefferson, OR 97532
USA
Tel: 541-752-0188
Email: dant@woodrandd.com

MEMBERSHIPS:

- AITC (American Institute of Timber construction), Technical Activity Committee, Voting Member
- ASCE (American Society of Civil Engineers)
- APENB (Association of Professional Engineers of New Brunswick)
- APEBC (Association of Professional Engineers of British Columbia)
- APENS (Association of Professional Engineers of Nova Scotia)(License to Practice)
- APENZ (Association of Professional Engineers of New Zealand)
- ASTM (American Society Testing Materials), Voting Member
- International Bamboo Code for Structural Development Committee
- Canadian Forestry Association
- Canadian Society for Civil Engineering
- Forest Products Society, Publications Reviewer.
- Society of American Foresters

AWARDS:

- 1997 - CIF "Nova" Award for Innovation (worldwide competition all construction products)

- 1996 - Applied Science Technologists & Technicians of British Columbia (ASTTBC) Award for advanced technology (worldwide competition).
- 1996 - Civil Engineering Research Foundation (CERF) Charles Pankow Innovative Applications Award (worldwide competition only structural products).
- 1993 - Association of Professional Engineers of Nova Scotia design award for designing the "Hector Heritage Quay," an all timber connector building with adjustable base connectors creating fixed end moment connectors in a green wood situation.
- 1993 - Advanced Material Center, Oregon State University, $3,000.00 scholarship for graduate study.
- 1992-94 - Graduate Fellowship for three years to complete Ph.D. at Oregon State University
- 1988 - Nova Scotia Architects Association Innovation in Engineering Award (Timber frame distillery).
- 1982-83 - Association of Professional Engineers of New Brunswick, post graduate studies scholarship (two consecutive years).

TEACHING AND CURRICULUM DEVELOPMENT:

- Part-time Instructor - Technical University of Nova Scotia, Civil Engineering Department and Department of Extension - Wood Design and Wood Technology
- Assisted in the establishment of a certificate program in construction management
- cosponsored by the Canadian Society of Civil Engineers (CSCE) and the Technical University of Nova Scotia (TUNS) Department of Extension.
- Developed curriculum and taught two structural wood design courses at TUNS.
- Currently developing curriculum for project costing and scheduling course, TUNS and CSCE.
- Taught second year structures at University of New Brunswick, Department of Civil Engineering,1983-84.
- Developed and taught course curriculum for construction and construction management for residential and light commercial construction.
- Established methodology for total input/output computer analysis of all sawmills in New Brunswick, Canada. Conducted three years of data collection.
- Level I and II Engineered Log Structure Training Course text.

CONSULTING FOR PRIVATE INDUSTRY:

- Forensic Engineering

- Structural analysis of decayed roof systems in four school facilities, Dawson Creek,
- B.C. Designed retrofit of glulams and roof decking.
- Infrared roof analysis to detect water penetration and subsequent stability of roof
- system, Halifax, N.S.
- Structural redesign and analysis of decayed 146' x 256' glued laminated arch arena,
- Penticton, B.C.
- Design and construction methodology for a major foundation collapse, Dartmouth, N.S.
- Shop drawings for wood and wood composite manufactures, i.e., Ramlam, Inc., Louisville, P.Q., Trus Joist, Inc., Toronto, Gang Nail, Inc., Toronto.
- Commercial design, cost projections and supervision of construction of Knox United Church, Bedford, N.S.
- Design of Hector Heritage Quay Interpretative Center ($500,000.00 solid sawn wood superstructure), Cape Breton, N.S.
- Design and costing for the Mira Lodge and Resort ($21 million resort complex).
- Participant in Canadian trade delegation to Jakarta, Indonesia.
- Design and supervision of construction over 100 wood structures.

RETROFIT EXPERIENCE

- **Wingspread (Summer 1996):** Developed and tested roof members for the retrofit of Frank Lloyd Wright's "Wingspread" in Racine, Wisconsin. The roof members consisted of wood and carbon composites arranged to optimize strength and stiffness.
- **Dawson Creek Schools (Summer 1998):** Inspected several schools for possible decay and changes in dead and live loads in Dawson Creek, British Columbia. The detailed on-site inspection included taking core samples, stress wave timing, and visually inspecting various members to determine their current structural capacity. With the strength of the existing members now known, a FiRP® Reinforced tension lamination was designed to support new loading requirements. The FiRP® Reinforced tension lamination design carries the majority of the new live load stresses thereby limiting the live load stresses in the existing members. Also, designed a mitigation plan to repair and prevent future decay. Provided project oversight during the installation of the retrofit laminations and the decay repairs to assure materials were installed correctly.
- **McCloud Bridge (Fall 1998):** Designed and provided installation oversight for the retrofit of a timber bridge in McCloud California. Used FiRP® Reinforced tension laminations to increase the moment capacity of the solid timber members.

- **Quoddy and Hay Cove Bridges (January 2000):** Performed inspections of each bridge to locate any possible decay and to determine the allowable strengths of the existing wood members in Nova Scotia, Canada. The inspections included taking core samples, stress wave timing, and visually inspecting various members to determine their current structural capacity. Developed a retrofit design using FiRP® Reinforced tension laminations to support the new live loads. The retrofit design reduced the stresses in the existing members under live load. Provided oversight during the installation of the FiRP® Reinforced tension laminations to assure materials were installed correctly.
- **Skier's Bridge (October 2000):** Performed a preliminary inspection to determine the extent of decay in the bridge girder and bent beams in the Skier's Bridge in Whistler, British Columbia. The inspection included taking core samples, stress wave timing, and visually inspecting various members to determine their current structural capacity. Designed a retrofit using FiRP® Reinforced laminations, steel, and wood to allow the bridge to support the design loads and provided a mitigation plan for preventing further decay. Provided project oversight during the installation of the retrofit materials to assure materials were installed correctly.
- **Nechaka Learning Center (November 2000):** Performed an inspection to determine the extent of decay and cracking in roof beams subjected to moisture from leaky roof in Vanderhoof, British Columbia. The inspection included taking core samples, stress wave timing, and visually inspecting various members to determine their current structural capacity. Designed a retrofit consisting of FiRP® Reinforced tension laminations and FiRP® Reinforced plywood to carry the moment and shear forces, respectively. Provided project oversight during the installation of the retrofit materials to assure materials were installed correctly.
- **Paper Mill (Spring 2001):** Performed a detailed inspection of a 40,000 square foot building located on a paper mill facility in St. Helens, Oregon. The inspection included taking core samples, stress wave timing, and visually inspecting various members, including the decking material, main beams, and purlins, to determine their current structural capacity. Developed a retrofit design for the entire roof structure, including main beams, secondary beams, purlins, and decking to reduce the stresses in the main beam under dead and live loads. Used FiRP® Reinforced LVL to repair all of the members. Provided project oversight to assure materials were installed correctly.
- **First Baptist Church of Fair Oaks (May 2002):** Developed a retrofit design for a beam that had cracked due to an overload situation using FiRP® Reinforced tension laminations for the First Baptist Church of Fair Oaks, California. To reduce the dead load stresses, the existing members were loaded with an uplift force, via a jack. Provided project oversight during the

jacking and the installation of the retrofit laminations to assure materials were installed correctly.

RESEARCH ACHIEVEMENT:

- Completed coordination of a reinforcement application development project with a total budget of $25 million initiating reinforcement procedures for all wood and wood composites using high strength fiber reinforced plastic. Supervised all technical and testing details leading to code compliance with the International Conference of Building Officials (ICBO) in the United States. Research is now code approved (ICBO PFC-5100) and marketed under the trade name FiRP® Glulam.

APPLIED AND FUNDAMENTAL RESEARCH:

- Investigated and prepared report outlining design and construction methodology for a true pipe/spline fixed end moment connection for long length glued laminated beams (Western Wood Structures, Inc., Tualatin, OR).
- Investigated and prepared report on the effects of lateral loads on log walls (Timberline Cedar Log Homes, Washington).
- Investigation and reporting for proper construction techniques for oak truck trailer floors (Sunbury Transport, New Brunswick).
- Manufacturing techniques for "Flat Dowel Biscuit Production Using Veneers" (Black & Decker, U.S. and Canada).
- Solid wood furniture manufacturing (Craftique Furniture, New Brunswick).
- Sawmill streamlining (Woodstock Cedar Sawmill, Ltd.).
- Specialized pallet production (Arrowhead Wood Products, St. Louis, MO).
- In-depth investigation of measures for edge stabilization of OSB manufacturing (Huber & Sons,Maine, USA).
- Bamboo utilization in diversified markets.

GRANT AND CONTRACT SUPPORT:

- $1.2 million in various government/industry funding and several commercial research contracts, including a major project for the coordination and development of grading rules for Radiata Pine in Chile.
- Bamboo utilization in developing countries.

WORKSHOPS AND SEMINARS:

- Served as columnist and wood expert for *Log Home Living* magazine.
- Prepared and presented a one-day and two-day seminar on designing and constructing with wood and wood composites in conjunction with the Extension Department at TUNS.
- Prepared a two-day seminar on designing and constructing bridges with wood and wood composites in conjunction with the Extension Department at TUNS.
- Prepared and presented a half-day seminar on rehabilitation of marine structures constructed with wood.
- Developed and presented over 20 seminars on wood polymer composites across Canada and in ten countries.

PROFESSIONAL SERVICE:

- Expert witness for two construction civil liability legal actions.
- Mandatory mediator for construction disputes.
- Inspector for Atlantic New Home Warranty Corporation.
- Served on advisory committee for Certification Program Construction Management.
- Currently serve on the editorial committee of *Wood Design Focus*.
- Advisor to Canada Mortgage and Housing Commission on log home construction.
- Technical Advisor, Wood Technology, *Log Home Living* magazine.
- Developed criteria with International Network for Bamboo and Rattan for testing and evaluation of bamboo to determine allowable structural properties. This criteria was adopted by the International Conference of Building Officials as an Acceptance Criteria for bamboo.

PUBLICATIONS:

Published Research Papers:

- "Partially Reinforced Glulam Girders Used in a Light Commercial Structure," *Wood Design Focus*, Winter 1994.
- "The Taylor Lake Bridge: A Reinforced-Glulam Structure," *Wood Design Focus*, Summer 1993.
- "Mechanical Properties of Polymer-Impregnated Sugar Maple," *Forest Products Journal*, January 1990.
- "Toughness of Polymer Impregnated Sugar Maple at Two Moisture Contents," *Forest Products Journal*, June 1989.

Research Report Papers:

- “Design Criteria for Internal Pipe-Spline Moment Connectors,” 1996, Oregon State University.
- “Long Term Load Effects on High-Strength Fiber-Reinforced Plastic Used as a Reinforcement in Glulams,” 1996, Oregon State University.
- “Modeling and Testing Internal Pipe-Spline Moment Connectors,” 1996, Oregon State University.
- “Reinforcement of Curved Laminated Wood Structural Products Using High Strength Fiber Reinforced Plastic,” 1994, WSTI.
- “The Effects of Juvenile Wood on Tensile Strength and Modulus of Douglas-fir,” October 1993.
- “Partial Impregnation of Oak Hardwood Flooring,” February 1990, WSTI.
- “Impregnating Oak Hardwood Flooring,” January 1990, WSTI.
- “Polymer Impregnation of Heartwoodâ,” June 1989, WSTI.
- “Polymer Impregnation of Orientated Strand Board,” June 1989, WSTI.
- “Stabilizing Cellulose Insulation,” June 1989, WSTI.
- “Polymer Impregnation of Orientated Strand Board to Seal Edges,” March 1989, WSTI.
- “Partial Impregnation of Billiard Cues,” January 1989, WSTI.
- “Impregnating Billiard Cues with WSTIWOODTM Process,” November 1988.
- “Design of Flat Dowel Biscuit Manufacturing Facility,” September 1988, WSTI.
- “Manufacturing Flat Dowel Biscuits,” June 1988, WSTI.
- “Developing Strength and Grading Standards for Radiata Pine,” Spring 1988.
- “Design of Leather Polymer Composite Processing Facility,” June 1987, WSTI.
- “Predicting Strength Criteria for Kevlar and Fiberglass Reinforced Plastic (KRP and FRP) Glued Laminated Beams,” WSTI.
- “Axially Loaded Glulams Reinforced with High Strength Fiber Reinforced Plastic,” Oregon State University.
- “Wood Decay,” Wood Science & Technology Institute, Ltd.

Conference Proceedings:

- “New Compression Based Design Principle for Reinforced Glulams,” November, 1996, ASCE Annual Convention 96’, Washington, DC.
- “Glued-Laminated Timber Reinforced with Fiber-Reinforced Plastic,” October 1996, Composites 96’, Dallas, TX.

- “Long Term Load Performance of FRP Reinforced Glulam Bridge Girders,” October, 1996,USDA Transportation Meeting, Madison, WI.
- “High-Strength Fiber-Reinforced-Plastic Reinforced Glulam Highway Bridges,” October, 1996,IWEC 96’ Conference, New Orleans, LA.
- “Shear Stress Distributions in ASTM D143-89 Shear Blocks,” June 1996, Forest Product Society Annual Meeting, Minneapolis, MN.
- “The Effects of Test Setup and Apparatus on Full-Scale Glued Laminated Timber Beam Shear Strength,” April, 1996, ICBO Shear Issue Meeting, Los Angeles, CA.
- “High-Strength Fiber-Reinforced Plastic Reinforcement of Wood and Wood Composite,” March1996, SAMPE, Anaheim, CA.
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Appendix E - Original article prepared by Author on Decay and Retrofit of Existing Structures.

 Taken from Text Chapter prepared by Dr. Tingley while working as Executive Director of Wood Science & Technology Institute LLC. Text Name, Structural Renovation of Building Methods, Details and Design Examples by Alexander Newman, P.E.

CHAPTER 8
RENOVATION OF WOOD STRUCTURES

8. Renovation of Wood Structures

8.1 Introduction

Wood is one of the oldest and most widely used building materials. It is used in a variety of structural applications: for beams, columns, girders, panels, wall and truss systems, for piles, poles, railway ties, and temporary forms in concrete construction.

As a building material, wood is unique, innovative, and dependable. Most light structures built in North America, including school buildings, single-family homes, two - to three-story commercial and apartment buildings, are built using wood and wood products. Wood is also used in large construction projects such as bridges and buildings. Dependability has been demonstrated in numerous timber buildings worldwide. Residential timber buildings in Greece and pagodas and temples in China and Japan have survived hundreds of years of use and environmental loadings. At present, China has more than a dozen timber buildings, in seismically active regions, with a history of 1,000 years or more[6]. Many pagodas in china, built entirely of wood, are hundreds of years old.

Until the beginning of the twentieth century, timber was the primary material used in a variety of structures for transportation, rail systems, and residential, commercial, and sea structures.

With wood structures being so prevalent the issue of rehabilitating or strengthening wood members can be common. Often rehabilitation of wood structures involves retrofitting an existing structure to carry increased load or repair of decayed wood members. In some cases repairing the existing structure is the goal and in others strengthening the members by retrofitting provides the most economical solution.

In renovating wood structures the two most encountered issues are deterioration and strength enhancement. An assessment of deterioration, or other present load and geometric conditions must precede strength enhancement. This chapter focuses on wood deterioration and detection, and prevention and strength enhancement of wood beams.

8.2 Wood Deterioration

Wood deteriorates for numerous reasons, and as deterioration implies this adversely affects woods properties. The two primary causes of deterioration in wood are: biotic (living) agents and physical (nonliving) agents. In many cases the agents that first alter the wood, provide the conditions for other agents to attack (e.g. insects bring woodpeckers). The effectiveness of an inspection of deteriorated wood depends upon the inspector's knowledge of the agents of deterioration. A well-trained inspector is essential for accurately assessing wood deterioration.

8.2.1 Wood Deterioration Due to Biotic Agents

Biotic, or living, organisms that attack wood include bacteria, fungi, insects, and marine borers. As living organisms, they require certain conditions for survival such as moisture, oxygen, temperature, and food, which is usually the wood. When the basic living conditions are provided biotic agents of wood deterioration are free to proliferate, but if any one of them is removed the wood is safe from further biotic attack.

8.2.1.1Bacteria

In very wet environments bacteria can colonize untreated wood[7]. Bacterial damage can include softening of the wood surface, increased permability, and even degradation of chemical preservatives so that the wood becomes more susceptible to less chemically tolerant organisms[8]. Usually the process bacterial attack is very slow, but under extensive exposure for long periods damage can become significant.

Fungi

When exposed to favorable conditions, most types of wood become an attractive food source for a variety of decay-producing fungi. The fungi require moderate temperature, oxygen, and a moisture content of approximately 19% or greater (oven dry basis) to become active. Decay progresses most rapidly at temperatures between 10°C (50°F) and 35°C (95°F), outside this range decay growth slows considerably, and ceases when the temperature drops as low as 2°C (35°F) or rises as high as 38°C (100°F). Wood can be too wet for decay also. If the wood is water-soaked, the supply of

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[6] Hu, S. 1991. The earthquake-resistant properties of Chinese traditional architecture. Earthquake Spectra 7(3):355-389.

[7] Ritter, Michael A. 1992. Timber Bridges Design, Construction, Inspection, and Maintenance. United States Department of Agriculture. Forest Service. EM 7700-8.

[8] Ellwood, E.L. Eklund, B.A. 1959. Bacterial attack of pine logs in storage. Forest Products Journal 9: 283-292.

oxygen may be inadequate to support development of typical decay fungi[9]. Thus, wood will not decay, and decay already present from prior infection will not progress if appropriate conditions are not met.

Examples of wood preservation by environmental conditions are common. Timber pagodas in China have survived hundreds of years, and in some cases over 1,000 year'[s], because the wood was kept dry. Entrepreneurs in the U.S. are recovering old growth wood from sunken transport ships and selling the recovered wood. The sunken wood has been almost perfectly preserved from being kept saturated.

Decay fungi may be generally classified into two categories by the appearance on the wood surface.

1. Brown rot: Appears darker and can crack across the grain. Brown rot fungi attack the cellulose in the wood fibers. The brown color is due to the remaining lignin (the binder which holds the cellulose structure together), which is not consumed by the fungi. The decayed wood tends to form into small cubic shaped sections, which is a sign of advanced decay.
2. White rot: Appears lighter in color and does not crack across the grain until severely degraded. In contrast to brown rot, white rot consumes both the lignin and cellulose and leaves the surface appearing generally intact, but with little or no significant mechanical strength. The surface of the decayed wood tends to have a "white" appearance.

Dry rot is a common type of decay fungi in which the wood becomes brown and crumbly and an apparent dry condition. However, dry rot is a misnomer, because the wood must have some moisture in it to decay, although it may become dry later. A few fungi have water-conducting strands (hyphae) which are capable of carrying water, usually from the soil, into buildings or wood piles where they moisten and rot wood that would otherwise be dry

Interior decay damage can occur even when some precaution has been taken. Surface treated wood material can form cracks, which extend beyond the treated surface into untreated core material. Water can also get into the core of "protected" wood by the fungi hyphae. In either case water enters the core material and provides the adequate conditions for decay fungi to live.

Surface decay can be identified by both visual and probing techniques. Decayed wood tends to be very rough in texture with closely spaced cracks and grooves. With a pocketknife or flat-head screwdriver, decayed wood can easily be penetrated and partially removed. These techniques are only suitable for identifying possible surface decay. The depth of the damage may be determined by taking core samples, which is further discussed in section

Detecting Deterioration

8.2.1.3 Effect of Decay on Mechanical Properties of Wood

The primary effects of fungi attack on wood can be characterized by the following points[10]:

1. Change of color
2. Change of odor
3. Decreased weight
4. Decreased strength
5. Decreased stiffness
6. Increased hygroscopicity (easier absorption of water)

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[99] Forest Products Laboratory. 1999. Wood Handbook: Wood as an Engineering Material. U.S. Government Printing Office. Agric. Handb. 72. Washington DC: U.S. Department of Agriculture; rev. 1999.

[10] Bodig, J., Jayne, B. A. 1993. Mechanics of Wood and Wood Composites. Krieger Publishing Co., Florida. pp. 586-589.

7. Increased combustibility
8. Increased susceptibility to insect attack

The incipient stages of fungi attack are characterized by a change of color and perhaps a change in the odor and may not be detected by changes in hardness or by surface tests, such as the pick test. This stage may be very difficult to detect visually. Brown rot may reduce the mechanical properties by 10 percent before any significant weight reduction is noticed. When weight loss is between 5 and 10 percent, the reduction in mechanical properties may be reduced 20 to 80 percent[11]. Usually when decay is discovered by visual inspection, the damage has already been done.

Advanced stages of fungi attack reduce the specific gravity (weight) which decreases nearly every other mechanical property, including strength and is indicated by soft, punky, or crumbly wood. The compression perpendicular to the grain capacity is typically reduced the most by decay.

A common example of decay occurring where large compression perpendicular to the grain stresses act, is mushrooming, or bulging, of a beam over a support. Untreated beams that span past the exterior walls and supported by exterior columns are very susceptible to mushrooming decay. Water can get trapped by the steel beam-to-column connector, and settle directly between the steel and bottom surface of the wood beam. This moisture allows decay growth to occur in this bearing area. As decay progresses, significant vertical deformation can occur, due to the wood material weakening where compression stresses perpendicular-to grain are high, Figure shows this phenomena. Another example of decay occurring where large compression perpendicular to the grain stresses act, is given in Figure and

Figure , showing a poor connection detail on a bridge. The connection shown in Figure and

Figure , has lag screws drilled into the top of the beam allowing water to be drawn into the wood (like a tube holding water); this can easily seen by the water seeping out between the laminations on the side of the beam.

[11] Forest Products Laboratory. 1987. Wood Handbook: Wood as an Engineering Material. U.S. Government Printing Office. Agric. Handb. 72. Washington DC: U.S. Department of Agriculture; rev. 1987. pp 4-43.

Figure 1. The left and right photos show the exterior and interior respectively of a bulging, or mushrooming, glulam. Note the discoloration near the bottom of the beam on the left, and the moisture entering the inside of the building on the right, both indicative of a good environment for biotic agents of deterioration.

Inspection by Wood Science Technology Institute (N.S.) Ltd[23]. The owner is school District 59, Dawson Creek, B.C.

Figure 2. A glulam beam supporting the glulam girders of a bridge shows evidence of water damage.

Figure shows a close up of this damage.

Inspection by Wood Science Technology Institute (N.S.) Ltd[23]. "Skiers Bridge" managed by Intrawest Corporation, Whistler, B.C.

Figure 3. The left and right photos show evidence of water inside the beam. The lag screws drilled into the topside of the beam have allowed water to enter and build up in the wood and then seep out the sides as shown. The right photo shows a core sample being taken to further define the extent of internal deterioration.

Inspection by Wood Science Technology Institute (N.S.) Ltd[23]. "Skiers Bridge" managed by Intrawest Corporation, Whistler, B.C.

8.2.1.4 Insects

Many insect species have developed the ability to use wood as food or shelter7. Termites, beetles, bees, wasps, and ants are the primary insects causing wood deterioration. Damage by insects is usually noticeable from cavities or tunnels in the wood, or wood powder or frass (insect feces) near the outside of the wood[7].

Insects in the wood can lead to further damage as well by being food for woodpeckers. The woodpeckers break the wood to eat the insects. Severe woodpecker damage has been reported in wooden utility poles[29].

Marine Borers

Marine borers can affect timber substructures located in salt or brackish water. In 1965 the U.S. Navy reported that collectively these organisms cause over $250 million in damage each year[12].

8.2.2 Deterioration due to Physical Agents

Although physical agents of wood deterioration are not as common as biotic agents, their impact can become quite serious in specific locations. Physical agents can damage the wood material and preservative treatments that can lead to increased susceptibility of attack by biotic agents. Included among physical agents are abrasion, mechanical impact, by products of metallic corrosion, highly acidic or basic substances and ultraviolet light.

8.2.2.1 Mechanical Damage

Mechanical damage is caused by a number of factors and varies considerably in its effects on the structure. Most commonly, abrasion, vibrations, overloads, and foundation settlements cause mechanical damage.

8.2.2.2 Metallic Corrosion

Wood degradation can occur from metal fasteners in the wood reacting with moisture to release ferric ions that deteriorate the wood cell wall. Wood strength can be severely reduced in the affected area. Wood attacked by this type of corrosion ifs often dark and appears soft. The effect of wood metal corrosion can be limited by using galvanized or non-iron fasteners.

8.2.2.3 Chemical Degradation

The presence of strong acids or bases can substantially affect the wood. Strong acids degrade the cellulose and hemicellulose of wood which causes weight and strength loss7. The appearance of wood damaged by acid is dark in color almost as if it has been charred by fire. Strong bases degrade the hemicellulose and lignin. Wood exposed to a strong base will be bleached of color. Chemical exposure of this type is rare, except in cases of accidental spills.

8.2.2.4 Degradation by Ultraviolet Light

Ultraviolet light reacts with lignin near the surface of wood, resulting in degradation of the lignin and subsequent deterioration that is highly visible. Ultraviolet degradation changes the color of the wood; light-colored woods darken and dark woods lighten. However, this damage only penetrates a short distance below the surface[13] so there is little strength loss in the wood member exposed to ultraviolet light.

8.3 Detecting Deterioration

Methods for detecting wood deterioration can be broken into two categories: interior detection and exterior detection methods. In each case specific methods or tools are appropriate for different types of damage and structures. There is no certain method that will accurately determine the condition of a given structure, but a combination of the methods, tools, and a well-trained inspector can provide a reasonably accurate assessment of the deterioration present.

8.3.1 Exterior Detection Methods

Exterior detection methods are easy to employ, because of easy access to exterior wood. The methods most commonly used include visual inspection, probing, and the pick test. These methods provide a basis for further interior detection methods to define the extent of damage.

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[12] United States Navy. 1965. Marine bioligical operational handbook: inspection, repair, and preservation or waterfront structures. NAVDOCKS MO-311. Washington, D.C.: U.S. Department of Defense, Bureau or Yards and Docks.

[13] Feist, W. 1983. Weathering and protection for wood. In: Proceedings American Wood Preservers's Association; 1983; 79: 195-205.

8.3.1.1 Visual Inspection

Visual Inspection is the simplest method for locating wood decay on the outside (exterior) of the member and is suitable for detecting decay in more advanced stages. Visual inspection may not be an effective method to find early stages of decay, when control is most effective. Some common indicators of decay, which can be found by a visual inspection, are listed below[7]:

Fruiting bodies: Some types of fungi produce fruiting bodies, which appear on the surface during the decay process. These types of indicators can easily be partially cleaned off by weathering. If fruiting bodies are observed on exterior wood members, the decay is most likely extensive.

Sunken faces: Localized surface depressions are often a sign of decay near the surface. The wood may be intact or partially intact at the surface.

Staining or discoloration: A surface blemish can indicate if the wood member has been subject to surface water.

Bulging of wood over the bearing points in beams. The decrease in specific gravity caused by fungi attack greatly diminishes the perpendicular to the grain bearing capacity of wood (as shown in Figure).

Insect activity can be identified by holes, piles of wood powder, or frass.

Plant or moss growth indicates that relatively high moisture is present, a condition suitable for decay.

8.3.1.2 Probing

Probing can be done with a pointed tool to locate soft areas of the wood surface. This can indicate decay or water softened wood, so experience is necessary to interpret the results.

A probing tool, the Pilodyn has also been developed and used extensively in Europe. The Pilodyn is a spring-loaded device that drives a pin into the wood surface. The depth of penetration, according to the moisture content and wood species type, gives a measure of surface decay[14].

8.3.1.1 Pick Test

If an area of surface decay is suspected by inspection, a “pick test” may help to determine if decay is present. The “pick test” is conducted by driving a metal pick or screw driver a short distance into the wood surface and bending the tool back to pry off a small area of wood. If the wood splinters, it is most likely sound. If the break is brash or crumbles, the wood is most likely decayed and may require treatment or removal. An experienced wood inspector should interpret the results of the “pick test”.

8.3.2 Interior Decay Detection

Due to lack of visible indicators interior deterioration is difficult to detect. Several methods and tools exist for assessing interior damage, which include hammer sounding, moisture meters, drilling and coring, sonic evaluation and to a limited extent x-ray devices.

8.3.2.1 Hammer Sounding

Sounding the wood surface is done by striking it with a hammer and evaluating the tonal quality. A trained inspector can interpret dull or hollow sounds that may indicate internal decay. Of course many factors other than decay can influence the sound of wood struck with a hammer, so this provides only a partial understanding. This method is easy, quick and inexpensive and suspect areas can then be verified by other methods such core sampling.

8.3.2.2 Moisture Meters

[14] Smith, S.M.: Morrell, J.J. 1986. Correcting Pilodyn measurements of Douglas-fir for different moisture levels. Forest Products Journal. 36(1):45-46.

Moisture meters can help identify wood at high moisture content, and high moisture content wood is a suspected area of potential decay. Untreated wood moisture contents higher than 20-25% indicates conditions suitable for decay. Further information on the use of moisture meters has been published[15].

8.3.2.3 Drilling and Coring

Drilling and Coring are some of the most common methods of interior decay detection[16]. Due to their similarities they are included together.

Drilling can be done with either a hand or power drill. Usually the inspector drills into the structure at different locations noting the torque resistance and observing the drill shavings for evidence of decay. The advantage of using a hand drill is that it allows the inspector to better feel the drill bit's torque resistance.

Core samples can be taken with increment borers that provide a solid wood core that can be examined for evidence of decay, or void pockets. An increment borer and core samples are shown in Figure . Core samples can show the limit and extent of deterioration and provide lab samples. Lab samples can be cultured to indicate the presence of decay fungi and provide an assessment of future risk[17] and also to analyze the woods' specific gravity.

Suspected decay areas, determined by moisture meters, visual inspection, or other methods can be confirmed by drilling and coring. It is important when drilling or coring to use sharp tools so that crushed wood, caused by a dull bit, will not be mistaken for decay.

8.3.2.3 Sonic Evaluation

Several different sonic wave propagation methods have been recently developed. These include sonic wave velocity, acoustic emission, and stress wave analysis. The simplest of these methods measure the velocity change of a sound wave moving across wood. More recent efforts have measured how a sonic wave is altered by wood defects and also various deterioration agents[18]. These methods are often referred to as nondestructive evaluation, and much research has been done on the topic[19].

Progress is being made in the development of technologies for assessing the residual performance of wood in structures. Researchers have shown that using acoustic emission techniques can be used to detect the presence of termites in wood members[20]. The wooden ship USS constitution, built under

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[15] James, W.L. 1975. Electric moisture meters for wood. Gen. Tech. Rep. FPL 6. Madison, WI: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory. 28 p.

[16] Maeglin, R.R. 1979. Increment cores-how to collect, handle and use them. Gen. Tech. Rep. FPL 25. Madison, WI.: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory. 19 p.

[17] Morrell, J.J.; Helsing, G.G.; Graham, R.D. 1984. Marine wood maintenance manual: a guide for proper use of Douglass fir in marine exposure. Res. Bull. 48. Corvallis, OR: Oregon State University, Forest Research Laboratory. 62 p.

[18] Ross, R.J., Brashaw, B.K., and Pellerin, R.F. 1998, Nondestructive evaluation of wood. Forest Products Journal. 48:1:14-19.

[19] Ross, R.J. and Pellerin, R.F. 1994. Nondestructive testing for assessing wood members in structures: A review. GTR-70. USDA Forest Service, Forest Products Laboratory. Madison, WI.

[20] Lemaster, R.L., Beall, F.C., and Lewis, V.R. 1997. Detection of termites with acoustic emission. Forest Products Journal. 47:2:75-79.

orders from George Washington, was renovated for its 200th anniversary using ultrasonic, and stress-wave testing to locate areas of deterioration[21,22].

Advances have been made in the development and use of these nondestructive test methods for inspection purposes. Some inspection professionals[23] are using commercially available adaptations of these technologies to aid in their inspection work. Coupled with a thorough visual examination, these technologies can add significantly to the quality of an inspector's evaluation by providing information on the internal condition of members and the residual load-carrying capacity of an in-situ wood member[18].

Design examples 8.6.1 and 8.6.2 show use of the stress wave timer.

8.3.2.3 X-Ray Devices

X-ray scanners are now being developed to provide internal images of uncut logs in attempt locate defects to optimize yield and cutting patterns[18]. In the past X-ray scanners were used to locate internal voids in wood[24]; but the high cost of equipment, safety factors and expertise needed have curtailed its use. Other developments in Europe have used X-ray scanners to move up and down a pole to provide internal images of wooden poles7.

8.3.3 Summary of Decay Detection Methods

The process of locating potential decay problems in an existing structure is both an art and a science. This section summarizes the methods explained for interior and exterior decay detection, to locate potential decay problems.

No one method exists to accurately detect decay in a given structure, but a number of tools used in combination give a good estimate of the amount and degree of wood deterioration present. Exterior detection methods such as visual inspection, probing, or picking, can reveal signs of decay or give indications that internal decay may be present. The presence of internal decay can be explored using methods such as core sampling, moisture meters, and several modern nondestructive sonic evaluations. The development and use of these nondestructive evaluation technologies will add significantly to the inspector's evaluation by providing information on the internal condition and residual load carrying capacity of an in-situ wood member.

<u>8.4 Preventing Wood Deterioration</u>

Preventing wood deterioration involves many factors, which are mostly related to moisture control, or preservative treatments. Generally, preservative treatments are used only when absolutely necessary, due to their toxicity. Good construction detailing can be much more effective to resist decay than using harsh toxic chemicals. The most effective construction detail is to keep the wood dry (less than 19% moisture content). Keeping the wood covered may not necessarily keep the wood dry. Wind blown rain is often responsible for wetting exposed roof beams and water can be absorbed into the wood through other construction materials such as concrete (in footing or roof slabs) or metals (from thermal condensation).

Trees use multiple defense systems to prevent or slow fungi. The first is the bark, which provides an effective barrier against fungi attack. Second is the sapwood (the living part of the xylem) which can

[21] Witherell, P.W., Ross, R.J. and Faris, W.R. 1992. Using today's technology to help preserve USS constitution. Naval Engineers Journal. 104:3:124-134.

[22] Mardin, L. 1997. Restoring Old Ironsides. National Geographic. 191 (6):38-53.

[23] Wood Science and Technology Institute, Ltd., 1600 SW Western Blvd. Suite 190, Corvallis,OR. 97333. (541) 753-4548. On the web at: http:/www.wsti-wce.com

[24] Mothershead, J.S.; Stacey, S.S. 1965. Applicability of radiography to inspection of wood products. In: Proceedings 2nd symposium on Non-Destructive Testing of Wood; 1965; Spokane, WA.

respond to fungi attack by terminating the cellular metabolism in the affected areas which may create an adverse environment for the fungi. The sapwood can also transport resins to seal off the infected area thereby reducing the extent of the decay. Third, the heartwood (the nonliving portion of the xylem) contains chemicals and extractives hostile to fungi. The heartwood of some species of redwood and cedar can be used outside in decks with no chemical treatments because of the excellent fungi resistant chemicals naturally contained in the heartwood. Heartwood can also become plugged with growths called tyloses which restrict the movement of water and fungi.

There are many types of man-made chemical preservatives, which are used to prevent fungi attack. The best known is creosote, which is often used to preserve wood utility structures. Pentachlorophenol is used to treat bridge girders and other wood members where human exposure is limited. Due to the leaching of pentachlorophenol and its toxicity, its use is limited. Chromate copper arsenate (CCA) is an effective wood preservative which is relatively safe for humans. Unfortunately, the treatment process for CCA uses water as the transport mechanism, which can cause splits and checks, especially for larger wood members. The effectiveness of CCA in the heartwood is in question due to generally poor penetration (often caused by tyloses).

Most chemical treatments require special pressure tanks to obtain the necessary penetration depth for effective decay resistance. Surface treating is not nearly as effective as pressure treatment because once the protective coating is broken by localized splits, checks, and moisture cracks, an avenue for fungi attack is created. This creates problems for fixing existing wood structures or components in-situ. Fumigants were developed to provide chemical protection without the requirement for pressure treatment. This allowed structures already in the field to be treated. The first use of the technology was applied to wood utility poles and has developed from there to use in beams and columns.

8.4.1 Moisture Control

One of most effective and least expensive methods to prevent wood decay in an existing structure is to lower the moisture content. If the moisture content of the wood drops below a certain value (usually less than 19% moisture content), the fungi becomes dormant and further wood decay is prevented. Access to free water needs to be completely terminated by means of sealers or construction details. A classic example of construction detailing used to prevent water from contact with the wood structural system is covered bridges. These bridges were built with a roof to prevent rainwater from contacting the wood structural system, thereby preventing the moisture content from exceeding 19%. Now, preservatives are available to allow wood to exceed 19% moisture content without risk of fungi attack.

Preventing water from being absorbed into the end-grain of wood is of paramount importance. Due to the cellular makeup of wood, water is most easily transported along (parallel to) the grain. Access to free water at the end-grain may allow the moisture content of the wood to reach 19% at great distances from the water source. Often, exposed cantilevered roof beams may experience excessive crushing at the wall support, many feet away from the beam end due to decay because water traveled from the exposed beam end to the support and provided the fungi with the necessary moisture. Most species of wood allow water transport perpendicular to the grain, although at a fraction (1/100) of the rate of parallel-to-grain transport. Paint and sealers can be an effective moisture barrier although they should be used in conjunction with construction details designed to prevent water from reaching the surface of the wood. Paint and sealers with high solids content can seal in moisture as well as keep it out, thus at times paints and sealers can provide good decay conditions. Generally large dimension timber and glulam beams (nominal 4 in. and greater widths) are not painted, just stained.

8.4.1.1Oxygen Deprivation with Moisture

Another example of preventing decay by simple means is storage and transport of logs in water. Historically, the ideal place to store logs prior to mill shipment is a body of water such as a river or lake. In this environment, the fungi are deprived of oxygen, which is essential for their chemical processes. In modern times, logs are often stored on land and continuously soaked by sprinklers to ensure water saturation. Logs are also stored wet to reduce checking and the potential for fire damage.

8.4.2 In-Place Preservative Treatments

To arrest decay in existing wood members or structures, in-place preservative treatment methods are used. The two common types of in-place treatment methods are surface treatments, usually used to prevent decay from starting, and fumigants, usually used to treat internal decay. In place treating can significantly extend the life of wooden structures. Several case studies[7] have shown in-place preservative treatment to extend the life of timber bridges by as much as 20 years or more.

8.4.2.1 Surface Treatments

Surface treatments are useful for decay prevention, but their shallow penetration limits their effectiveness against internal decay. Surface treatments are usually in liquid, gel or paste form. Liquid preservatives can be applied by brushing, squirting, or spray-flooding the wood surface. Other preservatives are available in semisolid greases or pastes, and these are useful for vertical surfaces. The different chemical preservatives commonly used are published7, and possible health risks associated with the different preservatives must be fully understood before use.

Surface treatments are more effective for drier wood. Tests have shown improved treatment of wet wood using double the preservative concentration[25]. Field tests have shown surface treatments can prevent decay for 20 years[26], though it is recommended to reapply treatment at 3-5 year intervals.

Painted surfaces tend to develop small cracks and pores, which provide routes for moisture. Paints slow the movement of water into the wood, and the exit of water from the wood. Therefore if a paint barrier slows the exit of moisture, the fungus has more time to continue degrading the wood. Beams should not be coated with a finish that contains more than 30% solids, as this may prevent the exit of water. Stains are usually acceptable, and free from the problems associated with paints.

Example: Sodium Fluoride Paste

Osmose Wood Preserving, Inc. uses a proprietary sodium fluoride paste which is applied to the outer surface of existing utility poles and is very effective in penetrating the wood and halting decay and wood destroying organisms. Sodium fluoride can be supplied in a gel, rod, or paste form to suit the particular need of the repair. This type of preservative is effective at low moisture contents, however it is very toxic to humans. Unfortunately, the most effective preservatives are also the most toxic to humans and the environment.

8.4.2.2 Fumigants

Fumigants are preservative chemicals in liquid or solid form placed in predrilled holes to stop internal decay. Over time the fumigants vaporize into gas and move through the wood eliminating decay and insects. Fumigants can diffuse 8 feet or more from the point of application in vertical members, and 2-4 feet in horizontal members7 and are most effective when applied to sound wood. Different fumigants diffuse at different rates and will eventually diffuse out of the wood requiring a reapplication.

Example: Fumigant Types: Boron Rods and Sodium Fluoride

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[25] Clark, J.W.; Eslyn, W.E. 1977. Decay in wood bridges: inspection and preventative & remedial maintainence. Madison, WI: USDA, Forest Service, Forest Products Laboratory. 51 p.

[26] Scheffer, T.C.; Eslyn, W.E. 1982. Twenty-year test of on-site preservative treatments to control decay in exterior wood of buildings. Material u. Organsimen 17(3): 181-198.

Boron is a type of fumigant and is very effective in controlling wood decay and is relatively less toxic to humans as other chemical preservatives. Boron can be processed into rods, similar to glass rods, and inserted into predrilled holes in a structural wood member. The boron rods slowly dissolve over time and the natural moisture in the wood facilitates the migration of the boron through the pores.

Because the boron rods depend on moisture to transport the preservative, treatment with these rods may not be appropriate in areas where construction detailing, flashing, or roof repair has been performed which eliminated the moisture supply for the fungi. Research has indicated that the moisture content of the wood needs to be greater than 40% for adequate boron transport through Douglas-fir heartwood[27]. However, for exposed beams or structural members in contact with the ground, the boron rods are ideal.

Sodium fluoride is another diffusing fumigant, which functions similar to boron. Using the natural moisture in the wood, sodium fluoride rods dissolve and travel through the cellular structure of the wood.

Chemical preservatives are available which may be applied in-situ to slow or stop further fungal decay in sensitive environments such as homes and schools. Boron based preservatives are typically a low toxicity pesticide which is designed to penetrate wood and wood composites and protect from termites, boring insects, ants, and fungus. Rather than using high toxicity to kill insects directly, the boron based preservative kills the microbes in the insects' digestive system, which leads to death by starvation. Boron based preservative can be supplied in may forms: solid rods, powders, gels, and sprays.

See design example 8.6.3 for calculation of boron rod requirements.

8.5 Retrofit Repair

Retrofit mechanical repair methods typically involve the use of fasteners and additional wood, steel, or more recently fiber reinforced plastic (FRP) components to strengthen and reinforce existing wood members. The two main methods of repair discussed here are member augmentation, and clamping and stitching. Also examined are the advantages of FRP components used in the retrofit of wooden structures.

8.5.1 Member Augmentation

Augmentation is the addition of new material to an existing member with the purpose of increasing the strength of the existing member. The material added is usually wood, steel and more recently fiber reinforced plastic's (FRP's). These materials are usually attached to the existing wood in place, using steel bolts and sometimes with an epoxy adhesive also. Two common methods of member augmentation are splicing and scabbing. Splicing is the addition of a splint like cast, which restores strength at a break, split or other defect, as shown in **Figure 4**. It is recommended that the member be fully cut through to equally distribute load to the splice plates[28].

[27] Morrel, J. J., Sexton, C. M., Preston, A. F. 1990. Effect of Moisture Content of Douglas-fir Heartwood on Longitudinal Diffusion of Boron from Fused Borate Rods. Forest Products Journal. 40(4): 37-40.

[28] American Society of Civil Engineers. 1982. Evaluation, maintenance, and upgrading of wood structures. Freas, A., ed. New York: American Society of Civil Engineers. P. 428.

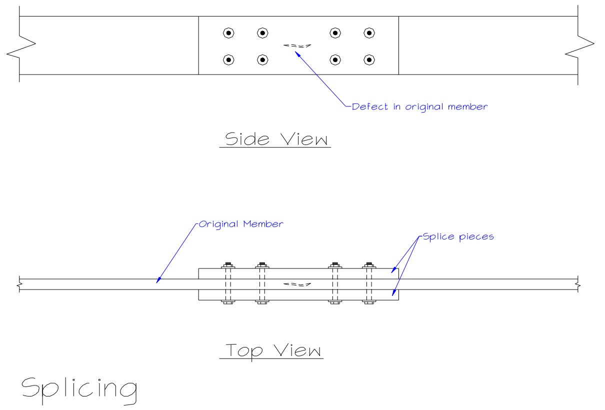


Figure 4. Member augmentation with splice plates.

Scabbing is the addition of reinforcement over a substantial length of the existing member to strengthen it, as seen in **Figure 5**. Other augmentation methods used on utility poles and timber pilings have involved sleeves, such as FiberWrap™ [29] (a FRP) or reinforced concrete bound by FRP jackets[30].

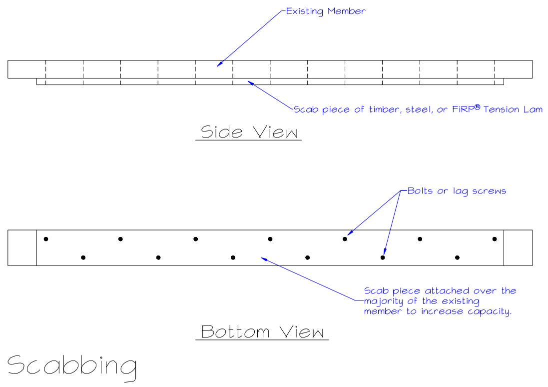


Figure 5. Member augmentation by scabbing.

8.5.1 Member Augmentation Using High Strength Fibers

High-strength fiber reinforced plastics are used to increase the strength and stiffness of existing beams in-place. These FRP's have several advantages over other materials, including better wood compatibility (allows optimization of material strengths), small size and weight, very high allowable strengths, and low cost.

Currently, member augmentation by scabbing is the most common retrofit application using FRP's. These retrofits have included the use of FRP tension laminations on glue-laminated timber (glulam) and solid sawn beams, and FRP plywood used as tensile reinforcement of roof decking and shear reinforcement of beams[23], the FRP tension laminations being most common. Fiber reinforced tension laminations consist of single or multiple layers of FRP reinforcing panel bonded under controlled conditions to the wide face of high quality lumber or laminated veneer lumber. The grade and species of the lumber used in the FRP reinforced tension laminations should be of the highest quality; BF 2400 (Canada) and 302-24 (U.S.). The purpose of the high quality tension lumber is for increased strength, safety and to facilitate installation of the thin layer(s) of FRP. Typically, the lumber is end-jointed (finger joint or scarf joint) every 8 to 10 ft to produce any desired length. The composite FRP and lumber lamination is marketed under the trade name FiRP® Retrofit tension

[29] Osmose Wood Preserving, Inc. 980 Ellicott Street Buffalo, New York 14209. (716) 882-5905

[30] Better Roads. 1980. Bridge pilings can be protected; FRP jackets stop deterioration. Better Roads. May: 20-25.

lamination[23]. A photograph of a FRP reinforced tension lamination bonded to an existing glulam beam is shown in Figure 6 and Figure 7. Structural epoxy and lag screws are used to attach the reinforced tension lamination to the bottom side of the existing beam.

Figure 6: Photograph of a FRP reinforced tension lamination attached to an existing glulam beam.

Retrofit design by Wood Science Technology Institute (N.S.) Ltd[23]. FiRP® Retofit Lamination manufactured by Structurlam Products, Ltd[31]. The owner and contractor are school District 59, Dawson Creek, B.C.

The FRP reinforced tension lamination design was an adaptation of the results from long term research and development on high-strength fiber reinforced glulam beams (FiRP® Glulams). After nearly 1,000 full scale bending tests and component tests on wood and fiber reinforcement[32], the FRP reinforced glulam was accepted by the International Conference of Building Officials (ICBO) under the Evaluation Report No. 5100 (ER5100).

The bending capacity and bending stiffness of an existing glulam beam retrofitted with FRP reinforcement is estimated using transformed section methods and elasticity theory.

Since the purpose of the FRP reinforced tension lamination is to increase the bending stiffness and bending strength, it may not need to be extended under the support. Therefore, retrofit of the supports or bearing walls may not be required and mechanical units may be left in place. Figure 7 shows a photograph of a complicated FiRP® Reinforced tension lamination retrofit.

[31] Structurlam Products Ltd. Penticton, B.C. V2A 3M2. 250-492-8912.

[32] Tingley, D. 1995. Over a decade of research results in new, improved glulam. Canadian Consulting Engineer. March/April, 1996.

Figure 7: Complicated FRP reinforced tension lamination installation.

Retrofit design by Wood Science Technology Institute (N.S.) Ltd[23]. FiRP® Retofit Lamination manufactured by Structurlam Products, Ltd[33]. The owner and contractor are school District 59, Dawson Creek, B.C.

See design example 8.6.4 of FiRP® Retrofit of existing glulams

8.5.1.1FRP-to-Wood Compatibility: Why FRP is Better than Steel

Material compatibility between FRP reinforcement panels and lamination grade wood allows the capacity of the wood beam to be fully utilized at failure[34]. Steel used for reinforcement of wood beams reaches its yield point much sooner than wood. The yield strain (ratio of the elongation to the original length) for mild steel (ASTM A36) is 0.12%, and for Douglas-fir it is approximately 0.4% (varies depending on grade and size). When wood and steel are used together as in a composite beam, the maximum steel stress is reached when the adjacent wood stress is approximately 40% utilized. This prevents the composite beam from maximum energy absorption in the wood at the yield point. On the other hand, aramid-reinforced plastic (ARP) used in FRP has a yield strain of 2%[34], which is much greater than that of wood. The high yield strain of ARP allows full energy absorption to take place in the wood. Therefore, the yield strain properties allow a FRP and wood composite beam to stronger and more efficient than a steel and wood composite beam.

Bonding to wood is another area where FRP reinforcement excels over steel. The dimensions of FRP reinforcement and steel are not effected by changes in environmental conditions. Douglas-fir, however, may expand 0.1% and 0.2% for every 1% of moisture content change (between 0% and 30%) in the radial and tangential directions, respectively (Figure 8 defines the radial and tangential direction of wood). There are no significant dimensional changes in the longitudinal direction for Douglas-fir. No matter what type of reinforcement is used, the seasonal expansion and contraction of the wood must be accommodated. The stiffness properties of steel are uniform in all directions. The stiffness properties of FRP reinforcement can be designed for a specific use. Consequently, as wood expands in the radial or tangential direction, steel (which is 15 times stiffer than wood) will effectively resist the expansion which will destroy the adhesive bonding the wood and steel. The

[33] Structurlam Products Ltd. Penticton, B.C. V2A 3M2. 250-492-8912.

[34] Tingley, D. 1995. High-strength-fiber-reinforced plastic compatibility with structural wood composites. Wood Science and Technology Institute, ltd., Corvallis,OR. Paper # 24.

stiffness of FRP reinforcement perpendicular to the grain of the wood is practically the same as Douglas-fir. Therefore, as wood expands or contracts in changing environmental conditions, large stresses in the adhesive between the wood and reinforcement are not developed.

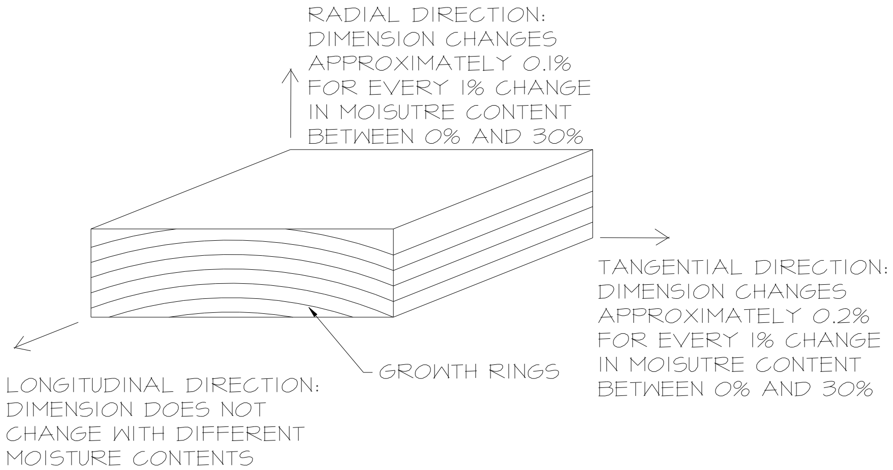


Figure 8: Orientation of the three principal directions in wood and corresponding dimensional sensitivity to changes in moisture content.

Fiber reinforced plastics have been specifically designed as a reinforcement to be compatible with wood using conventional adhesives. These are marketed under the trade name FiRP® Reinforcement[23]. Although steel is twice as stiff, it has only 25% of the ultimate tensile strength of FiRP® Reinforcement and is not compatible for bonding to wood[34]. Thus, FiRP® Reinforcement allows for a strong wood composite member.

8.5.2 Clamping and Stitching

A common problem in timber members is the development of longitudinal splits. These can develop as the member seasons and checks, or from overloading, or poor design details such a notching at a support. The effect of splits on the structural effects has been documented[35,36].

Clamping and stitching involves the use of fasteners and, or steel assemblies to prevent cracks, splits, or delamination from further development. Clamping usually uses bolts with steel plate assemblies. Stitching usually involves bolts or lag screws through the member. Figure 9 shows clamping and stitching details.

[35] American Society of Civil Engineers. 1982. Evaluation, maintenance, and upgrading of wood structures. Freas, A., ed. New York: American Society of Civil Engineers. P. 428.

[36] Ketchum, V.T.; May, T.K.; Hanrahan, F.J. 1944. Are timber checks and cracks serious? Engineering News Record. July 27: 90-93.

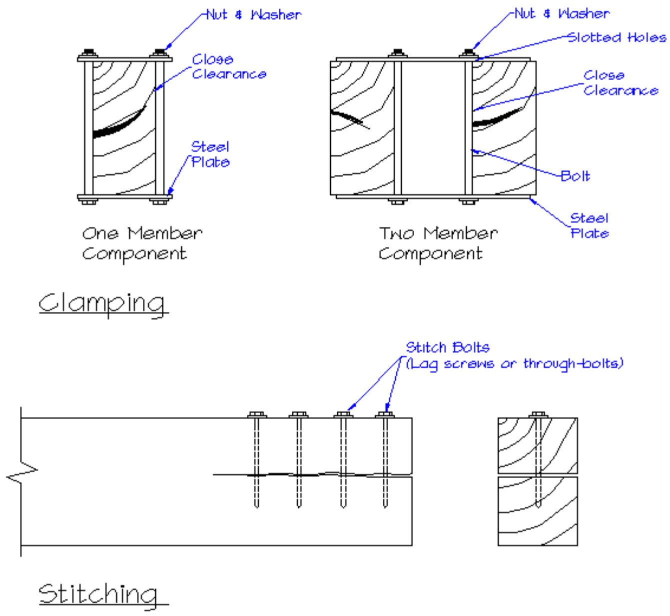


Figure 9. Clamping and stitching details.

8.6 Case Study Design Examples

Introduction to Design Examples 8.6.1 - 8.6.4

Two timber structures in Dawson Creek, British Columbia collapsed in 1996 and 1997 due to lateral instability and poor attachment of the structural components combined with an unknown, but not believed to be maximum loading[37]. The collapses initiated a structural review of all the schools within the local district. The following examples are actual case studies of inspection and renovation work performed at Dawson Creek public schools.

8.6.1 Example Using the Stress Wave Velocity Timer

Stress wave measurement techniques to locate internal decay have recently become popular because of its non-destructive nature. Stress wave analysis consists of sending a “sound” wave through a medium (wood) and measuring its velocity. The sound wave is introduced into the material by striking it with a hammer or blunt object. When the vibrations reach an accelerometer, an accurate timer is started; when the sound reaches a second timer, the timer is stopped. The distance between the “start” and “stop” accelerometer is measured. Knowing distance and time, the average velocity of the stress wave (sound wave) can be measured.

The modulus of elasticity of the material is theoretically related to the velocity of the stress wave and the density according to Equation 1.

$$E = \frac{c^2}{\rho}$$ …Equation 1

Where

E=Modulus of elasticity

c=Velocity of the stress wave

ρ=Density of the material

[37] Shipton, Brad. G.H. Cook and Associates, Inc. Dawson Creek, B.C., Canada. (250) 782-9275.

It is the measured modulus of elasticity (*E* in Equation 1) which indicates if decay is present or not. Typically, the modulus of elasticity for sound Douglas-fir ranges from $1.5x10^6$ psi to $2.2x10^6$ psi. The range can be tightened if the exact grade is known. If the sample of Douglas-fir has been subject to fungi decay, the specific gravity (weight relative to water) of the wood will decrease. The decrease in specific gravity causes a decrease in the modulus of elasticity, which decreases the velocity of the stress wave. Therefore, if decay is present, the measured modulus of elasticity using the stress wave timer will be significantly lower than the expected range.

Calibration of the stress wave timer is a critical step. Taking core samples of sound wood and the wave propagation velocity in the same sound wood establishes the calibration. The wave propagation velocity in sound (non-decayed) wood must be well established and repeatable before attempting to locate decayed areas. The manner in which the velocity is measured must be consistent to minimize variation and false readings.

The calibration curve for the stress wave timer when used on Douglas-fir beams and columns in place as shown in Figure 13. The curve indicates the relationship between stress wave time and specific gravity. Because fungal decay tends to reduce the specific gravity, the stress wave timer may be used to indicate potential areas of decay. Generally, stress wave times greater than 300 microseconds per foot (for the calibrated Douglas-fir) indicate that fungal decay may have significantly degraded the strength and stiffness properties.

The stress wave timer calibration curve was created by measuring stress wave times on an existing glulam and then taking an assay sample with a core drill. Specific gravity of the assay samples was measured in a laboratory to develop the curve in Figure 13.

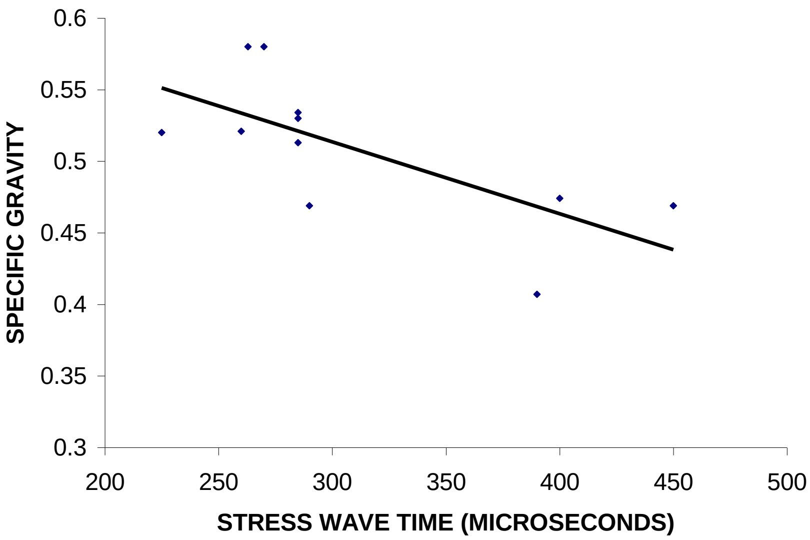


Figure 10: Stress wave timer calibration curve.

8.6.2 Using Stress Wave Timing for Slash and Cut Repair

A school building has glulam beams cantilevered out (towards the outside) with the ends exposed to weather. Since the wood of the glulam was untreated, and exposed to moisture, decay present was suspected.

Stress wave times through the wood were measured.

Figure 14 shows a contour map drawn on the side of a beam, created with the stress wave timer. This figure shows that the very end of the beam has relatively high stress wave time values (greater than 300 microseconds per foot) indicating areas of low density due to fungal decay. The stress wave times tend to decrease toward the wall indicating higher density. The covered area next to the wall has the lowest stress wave times because this region is subjected to the least amount of moisture.

Water is easily absorbed and transported parallel to the grain. This can often provide enough moisture for active fungal attack many feet from the source. An especially harmful situation occurs when an exposed beam collects moisture from the exposed end and transports the water over the load bearing exterior wall.

A combination of a slash-cut (to remove decay and limit the amount of exposure), end-sealing with a high solids coating and paraffin wax, and preservative treatment is used to renovate the beam ends. The design of the slash-cut meets two key parameters:

1. The remaining portion of the beam end should be adequate to support the weight of the roof and applicable snow load.
2. The exposed end-grain is of high enough quality to be properly planed and end-sealed. This may be determined using the stress wave timer.

Figure 11: Stress wave time contour map used to determine areas of decay.

Inspection and design by Wood Science Technology Institute (N.S.) Ltd[23]. The owner is school District 59, Dawson Creek, B.C. A photograph of the slash-cut retrofit design is shown in Figure .

Figure 12: Photograph of a slash-cut detail.

Inspection and design by Wood Science Technology Institute (N.S.) Ltd[23]. The owner and contractor are school District 59, Dawson Creek, B.C.

8.6.3 Example: Boron Rod Fumigant Requirement

Further wood decay can be prevented through the use of boron rods. The number of boron rods required for a specific treating situation is based on the volume of exposed wood. For effective long-term protection 6 ounces of boric-acid-equivalent (BAE) are required for each exposed cubic foot of wood. Rods are supplied in many different sizes and potencies. Therefore, the number of boron rods required depends on the size and potency of the rods.

One method for determining the amount of a particular type of boron rod is shown below:

- Type of rod:¾” x 3” (per specifications)
- BAE:2.03 oz / rod (per manufacturers specifications)
- Required concentration:6.00 oz BAE / cubic foot (per specifications)
- Rods required: $\frac{6.00\ oz\ BAE/cubic\ \ foot}{2.03\ oz\ BAE/rod} = 2.96\ Rods/cubic\ \ foot$
- Volume per rod: $\frac{\pi}{4}(0.75\ in^2)(3\ in) = 1.33\ in^3 = 7.67x10^{-4}$ cubic feet
- Dosage:($7.67x10^{-4}$ cubic feet)(2.96 Rods/cubic foot) =0.00227 ≅ **0.23%** by volume

Where the above “Dosage” of 0.23% is the amount of boron required to treat the wood. For example, if 100 cubic inches of wood is exposed and needing preservative treatment, then 0.23 cubic inches of boron rods are required. The boron should be as evenly distributed as possible.

8.6.4 Example: FiRP® Retrofit Tension Lamination

After a gymnasium addition to a grade school in Canada, the roof snow load on a lower adjacent roof increased significantly. The new gym roof elevation was about 10-feet higher than the roof elevation of the adjacent existing building. The difference in elevation of roofs caused snow to accumulate (drift) onto the lower elevation roof subjecting the older roof to greater snow loads than designed for. A load analysis showed that the maximum tension stresses in the glulam beams supporting the older roof were being exceeded by about 20%.

Since the lower elevation roof was covering a work shop area, adding columns and/or increasing the depth of the beams significantly, was ruled out. The design team decided to reinforce the existing glulams with FRP reinforced tension laminations to increase the strength while minimizing the shop area lost.

A retrofit of the glulams using the FRP reinforced tension lamination is relatively easy both in design and execution. The dimensions of the glulams were 5.25-inches wide by 28.5-inches deep with a span of 36-feet. The amount of FRP needed, 0.28-inches (4 layers of 0.07-inch aramid reinforced plastic), was found using methods of transformed sections and elastic analysis. The 0.28-inches of FRP is attached to a wood tension lamination for additional strength and ease of installation. The FRP reinforced tension lamination was then attached to the bottom (region of maximum tension stress) of the existing glulams, over the entire span length using epoxy and ¼-inch lag screws. The FRP reinforced tension lamination was left exposed and this completed the job, see Figure 6 and Figure 7.

Completion of this retrofit reduces the maximum tension stress in the original glulam beam by 24%. The new composite beam, now an FRP retrofit glulam, has a 129% increase in moment capacity over the original beam with only a 5% increase in depth.

Appendix F: Paper and Article written by Tingley for Australian Small Bridge Conference July 09

ADVANCED INSPECTION, NON DESTRUCTIVE TESTING, REMOTE MONITORING AND REFURBISHMENT TECHNIQUES FOR TIMBER BRIDGES

Dan Tingley Ph.D., P.Eng.

Wood Research & Development Ltd.

Corvallis, OR, USA, 97333

www.woodrandd.com

dant@peak.org

KEYWORDS:

Timber Bridges, Restoration, High-Strength-Fiber Reinforcements, Preservation Techniques, Non-Destructive Testing, reinforced glulams, glulam bridge girders, fiber reinforced plastic (FRP).

ABSTRACT

From early days to the middle of the nineteenth century, wood was the predominant bridge building material. Great timber truss bridges up to 460 feet long testify to the skill of our early bridge designers and builders.

Preservative treatments were introduced in 1910 and helped to assure the longevity of timber bridges. Glued laminated timber (glulam) was introduced into the United States in the 1930s and around the world by the 1940's and 1950's. The development of waterproof adhesives in the 1940s made it practical to use glued laminated timber in bridge construction. The advent of pressure treated timbers with preservative further increased the importance of wood as an engineering material for highway bridges.

The modern glued laminated timber bridge offered many advantages in the construction of highway bridges using a systems approach to bridge design. However, the Achilles heel for wood was the limited decay resistance when exposed to moisture contents over 20%. Thus, if poor design and maintenance detailing was instituted in the bridge it would soon show signs of degradation. Further, wood has a limited modulus of elasticity and allowable design strength compared to steel and concrete. However, on a specific strength and modulus basis wood was superior and in the age to come where carbon trading will become a major consideration in construction material choice wood will again become a major player in the bridge construction marketplace. Part of the renaissance of wood in bridges is the utilization of advanced techniques for inspection, non-destructive testing, remote monitoring and refurbishment techniques. This paper reviews the current state-of-the-art in these areas.

INTRODUCTION

There are many advantages achieved by using wood in the construction of bridges. Components of a timber bridge can be prefabricated at the plant, permitting rapid on-site assembly which reduces labor costs and construction time. A variety of configurations, including vertically curved girders and arches, can be manufactured so that the bridge designer has a wide degree of latitude in designing structural components. Wood is virtually unaffected by the chemicals or corrosive materials commonly applied to roadway surfaces, therefore de-icing salts do not corrode or deteriorate the decking as may occur with other deck materials. Timber is a relatively lightweight construction material which permits the transportation of large prefabricated structural units such as girders, girder and deck panels. Glued laminated timbers are relatively lightweight and its high strength to weight ratio in comparison to other materials permits use of smaller mobile erection equipment and may reduce foundation costs. Assembly of modular units can be accomplished at the construction site by semi-skilled labor. Esthetically, timber fits most environments, particularly in rural and suburban areas where a natural appearance is desired. Wood exhibits excellent short term duration of load characteristics to resist dynamic loads that can occur during construction or in high earthquake risk areas. Wood is a renewable natural resource.

Reinforcing glue-laminated timber with high-strength fiber-reinforced plastic (FRP) is now being used as reinforcement for glulam to improve its performance of glulam bridge girders (Tingley and Leichti 1993). Since 1993, many pedestrian and vehicular bridges using FRP technology have been installed in the United States and Japan. The use of high-strength fiber-reinforced plastic (FRP) to reinforce glue-laminated timber (glulam) members is being internationally commercialized. The first use of this product has been in the United States where the product is marketed under the trade name of FiRP® Glulam. Thousands of pedestrian, light vehicular and highway bridges with up to T 66 load ratings have been installed since the first glulam pedestrian bridge was installed in the summer of 1993 (Leichti et al, 1993).

In the face of this significant increase in the number of timber bridges being built around the world concrete and steel still control the market and the vast majority of bridges constructed today utilize steel and concrete. Superior strength and stiffness qualities are for the most part the key reason for this but other reasons are also important to note for this trend. The most significant reason is the fact that engineers are not trained in the design and utilization of timber to nearly the same degree as they are in steel and concrete. Further, they do not understand the current state-of-the-art in inspection, non destructive testing, remote monitoring and refurbishment techniques for timber bridges. This paper discusses these state-of-the-art concepts. This paper combines three previous papers written by the author and reworks those utilizing examples from around the world with timber bridges.

HISTORY OF WOOD UTILIZATION

The following sections are taken from a chapter written by the author for renovation of wood structures for McGraw Hill.

Wood is one of the oldest and most widely used bridge construction materials. It is being used and has been used in a wide variety of structural bridge applications: for beams, columns, girders, piles, ties, and temporary forms in concrete construction.

Wood is unique, innovative, and dependable. Around the world, school buildings, single-family homes, two-to three-story commercial and apartment buildings, are built using wood and wood products. Dependability has been demonstrated in numerous timber buildings worldwide. Timber buildings around the world have survived hundreds of years of use and environmental loadings. In China more than a dozen timber buildings, in seismically active regions, have survived 1,000 years or more.

Wood has been the predominate construction material throughout history and only during the last 80 years have concrete and steel surpassed wood in the volume, on a monetary basis, for larger construction projects. However, when the total of construction project dollars considers residential housing wood still surpasses concrete and steel as the primary construction material in the world today.

With wood structure use so prevalent, the issue of rehabilitating and/or strengthening wood members is an important factor in the engineering world. In addition the rehabilitation of wood structures often involves retrofitting an existing structure to carry increased load or the repair of decayed wood members within the structure.

WOOD DETERIORATION

Wood deteriorates for a wide variety of reasons. The two main causes of wood deterioration are: biotic (living) agents and physical (nonliving) agents. In many cases the agents that first alter the wood, provide the conditions for other agents to attack (e.g. insects bring woodpeckers). The effectiveness of an inspection of deteriorated wood depends upon the inspectors' knowledge of the agents of deterioration. A well-trained inspector is essential for accurately assessing wood deterioration. For example just because the sapwood has decayed due to White Rot in a round log hardwood girder does not mean the round log girder should be taken out of service. Perhaps a sand blasting to remove the lousy material and a fumigation of the heartwood is in order. Understanding how to properly prescribe a repair/refurbishment strategy is critical to properly upgrading and maintaining bridges.

Biotic Agents

Examples of biotic, or living, organisms that attack wood are; bacteria, fungi, insects, and marine borers. These organisms require certain conditions for survival such as moisture, oxygen, temperature, and food, which is usually the wood. When the basic living conditions are provided biotic agents of wood deterioration are free to proliferate, but if anyone is removed the wood is safe from further biotic attack.

Bacteria

In very wet environments bacteria can colonize in untreated wood. Bacterial damage can include softening of the wood surface, increased permeability and even degradation of chemical preservatives so that the wood becomes more susceptible to less chemically tolerant organisms. Usually the process bacterial attack is very slow, but under extensive exposure for long periods damage can become significant. See the photograph in Figure 1 of a beam that completed degraded due to biotic agents. The first agent was fungi, the second was insect infestation, the third was wood peckers, and the fourth was rodents. These steps all occurred in 22 years in a treated glulam bridge girder where the end of the beam had been cut back in the field after pressure treating and were not properly field treated. The end in the cantilever connector was completely decayed and degraded and the beam was about to fall out of the connector when discovered in an inspection by the author.

Figure 1. Photographs of a glulam beam that completely degraded due to biotic agents over a short 22 year period.

Fungi

Wood that is exposed to favorable conditions becomes an attractive food source for a variety of decay-producing fungi. Fungi require moderate temperature, oxygen, and a moisture content of approximately 19% or greater (oven dry basis) to become active. In addition decay progresses most rapidly in environments where the temperatures are between 10°C (50°F) and 35°C (95°F), outside this temperature range decay growth slows considerably. It ceases when the temperature drops as low as 2°C (35°F) or rises as high as 38°C (100°F).

Wood can be too wet for decay to continue also. If the wood is saturated by water, the supply of oxygen may be inadequate to support development of typical decay fungi. Thus, wood will not decay, and decay already present from prior infection will not progress if appropriate conditions are not met. This is important in considering underwater and underground piers. Typically the splash zones at the ground line or water line are the worst areas for pier degradation.

Decay fungi may be generally classified into two categories by the appearance on the wood surface.

Brown rot: Brown rot appears darker and typically will crack across the grain. Brown rot fungi attack the cellulose portion of the wood fibers. The brown color is due to the remaining lignin (the binder which holds the cellulose structure together), which is not consumed by the fungi. The decayed wood often forms into small cubic shaped sections, which is a sign of advanced decay.

White rot: Appears lighter in color and does not crack across the grain until severely degraded. In contrast to brown rot, white rot consumes both the lignin and cellulose and leaves the surface appearing generally intact, but with little or no significant mechanical strength. The surface of the decayed wood tends to have a “white” appearance. White rot is more typical in hardwoods. See figure 2 for a round log bridge girder with white rot.

Figure 2. Photographs of a white rot on a round hardwood log girder in Victoria, Australia.

Dry rot is a common type of brown rot decay fungi in which the wood becomes brown and crumbly and is in an apparent dry condition. However, dry rot is a misnomer, since the wood must have some moisture in it to decay, although it may become dry later. A few fungi have water-conducting strands (hyphae) which are capable of carrying water, usually from the soil, into buildings or wood piles where they moisten and rot wood that would otherwise be dry.

Interior fungal decay damage can occur even when some precaution has been taken. Surface treated wood material can form shrinkage cracks, which extend beyond the treated surface into untreated core material. Water can also get into the core of “protected” wood by the fungi hyphae. In either case water enters the core material and provides the adequate conditions for decay fungi to live.

Surface decay can be identified by both visual and probing techniques. Decayed wood tends to be very rough in texture with closely spaced cracks and grooves. With a pocketknife or flat-head screwdriver, decayed wood can easily be penetrated and partially removed. These techniques are only suitable for identifying possible surface decay. The depth of the damage may be determined by taking core samples, which is further discussed in section

Detecting Deterioration - Effect of Decay on Mechanical Properties of Wood

The primary effects of fungi attack on wood are as follows:

Change of color

Change of odor

Decreased weight

Decreased strength

Decreased stiffness

Increased hygroscopicity (easier absorption of water)

Increased combustibility

Increased susceptibility to insect attack

A change in color is evidence of incipient stages of fungi attack and in some cases perhaps a change in the odor. The decay may not be detected by changes in hardness or by surface tests, such as the pick test. This stage may be very difficult to detect visually. When specific gravity drops due to decay between 5 and 10 percent, the reduction in mechanical properties may be reduced as much as 20 to 80 percent. Usually by the time decay is discovered by visual inspection, the damage has already been done. The use of advanced detection techniques discussed later in this paper assists inspectors in determining the extent of strength reduction well before visual detection can be performed.

Advanced stages of fungi attack reduce the specific gravity (weight) which decreases nearly every other mechanical property, including strength and is indicated by soft, punky or crumbly wood. The compression perpendicular to the grain capacity is typically reduced the most by decay.

A common example of decay occurring where large compression perpendicular to the grain stresses act, is mushrooming, or bulging, of a girder over a support see

Figure , showing a poor connection detail on a bridge. The connections shown in Figure 4-5, contain lag screws drilled into the top of the beam allowing water to be drawn into the wood (like a tube holding water); this can easily seen by the water seeping out between the laminations on the side of the beam.

Figure 3. Bridge round log girder, showing advanced signs of decay with compression bulbing over the reaction point, Victoria, Australia.

Figure 4. A glulam beam supporting the glulam girders of a bridge shows evidence of water damage in Whistler B.C., Canada.

Figure below shows a close up of this damage.

Figure 5. The lag screws drilled placed in the topside of the beam have allowed water to enter and build up in the wood and then seep out the sides as shown. The right photo shows a core sample being taken to further define the extent of internal deterioration.

Insects

Many insect species use wood as food or shelter. Termites, beetles, bees, wasps, and ants are the typical insects that cause wood deterioration. Damage by insects is usually noticeable from cavities or tunnels in the wood, or wood powder or frass (insect feces) near the outside of the wood. See Figure 6 below for examples of white ant damage to wood curb timbers in a bridge in Victoria, Australia.

Figure 6. White Ant deterioration to a bridge timber in Victoria, Australia.

Marine Borers

Marine borers can degrade timber substructures found in salt or brackish water. In 1965 the U.S. Navy reported that collectively these organisms cause over $250 million in damage each year. See photograph in Figure 8 of such damage to a pier in the Barwon Heads Bridge in Victoria, Australia.

Figure 8. Marine borer damage to a pier and walers in the Barwon Heads bridge in Victoria, Australia.

Physical Agent Damage

Physical agents can damage the wood material and degrade away preservative treatments. This allows increased susceptibility of attack by biotic agents. Abrasion, mechanical impact, by products of metallic corrosion, highly acidic or basic substances and ultraviolet light are examples of such physical agent damage. See Figure 9 below for examples of tire wear on a bridge deck that could have been significantly reduced by the use of running boards on sleepers to allow for proper moisture flow. The runners could have been replaced easily and thereby saved the deck timbers.

Figure 9. Deck timbers worn by traffic. Runners like the one shown in the center for cyclists would have allowed easier lower cost bridge deck maintenance.

Mechanical Damage

Mechanical damage is caused by a wide variety of factors. Most commonly, vibrations, overloads, and foundation settlements cause mechanical damage.

Metallic Corrosion

Wood degradation most often occurs from metal fasteners in the wood reacting with moisture to release ferric ions that can excessively deteriorate certain types of wood cells. Wood strength can be severely reduced in the area around such connectors which is often a very important area in a bridge structure. Wood attacked by this type of corrosion ifs often dark and appears soft. The effect of wood metal corrosion can be limited by using galvanized or non-iron fasteners. See Figure 10 for photograph of such a non galvanized fastener utilized in an old bridge

Figure 10. Rusted metal connectors, ferric damage in the wood and fruiting bodies from fungal decay.

Chemical Degradation

The application of strong acids or bases can significantly affect the wood. See Figure 11 for a photograph of wood heavily damaged by chemicals in a paper processing plant. The strong acids degrade the cellulose and hemicellulose of wood which causes weight/strength reduction. Wood exposed to a strong acid will be dark in color as shown in Figure 11. Strong bases will also degrade the hemicellulose and lignin. Chemical exposure of this type is rare, except in cases of accidental spills in bridge situations.

Figure 11. Strong base degradation of wood beams in a roof system of a paper manufacturing facility.

Degradation by Ultraviolet Light

Ultraviolet light reacts with lignin near the surface of wood, resulting in degradation of the lignin and subsequent deterioration that is highly visible. Ultraviolet degradation changes the color of the wood; light-colored woods darken and dark woods lighten. However, this damage only penetrates a short distance below the surface so there is little strength loss in the wood member exposed to ultraviolet light.

Detecting Deterioration

Methods for detecting wood deterioration can be broken into two categories: interior detection and exterior detection methods. In each case specific methods or tools are appropriate for different types of damage and structures. There is no certain method that will accurately determine the condition of a given structure, but a combination of the methods, tools, and a well-trained inspector can provide a reasonably accurate assessment of the deterioration present.

WOOD STRUCTURAL MEMBER EXTERIOR DETECTION METHODS

Methods to detect exterior degradation of wood members are easy to employ, because of easy access to exterior wood. Most commonly these methods include visual inspection, probing, and the pick test.

Visual Inspection

Visual inspection has been in use predominately and is the simplest method for locating wood decay on the outside (exterior) of the member and is suitable for detecting decay in more advanced stages. The key problem with exterior methods of degradation detection in wood is that the visual inspection does not work very well for detecting early stages of decay, when control is most effective. The common indicators of decay, which can be found by a visual inspection, when the decay or degradation is advanced, are listed below:

Fruiting bodies: Often fungi will produce fruiting bodies, which appear on the surface during the decay process. If fruiting bodies are observed on exterior wood members, the decay is most likely extensive underneath. By the time fruiting bodies are observed the decay has spread far within the wood structure underneath.

Sunken faces: Another visual evidence of decay is localized surface depressions near the surface of the wood member. The wood may be intact or partially intact at the surface.

Staining or discoloration: A surface blemish or discoloration can indicate if the wood member has been subject to surface water.

Bulging of wood over the bearing points in beams. The decrease in specific gravity caused by fungi attack greatly diminishes the perpendicular to the grain bearing capacity of wood (as shown in some of the above figures. When this happens the wood will bulge or expand in the lateral direction as it compacts excessively in the vertical direction.

Insect activity can be identified by holes, piles of wood powder, or frass.

Plant or moss growth indicates that relatively high moisture is present, a condition suitable for decay.

Probing

Probing can be done with a pointed tool or drill bit to locate soft areas of the wood surface. This can indicate decay or water softened wood, so experience is necessary to interpret the results.

A probing tool, the Pilodyn, is a spring-loaded device that drives a pin into the wood surface. The depth of penetration, according to the moisture content and wood species type, gives a measure of surface decay.

Pick Test

If an area of surface decay is suspected by inspection, a “pick test” may be used. The “pick test” involves the use of a metal pick or screw driver to probe a short distance into the wood surface and bending the tool back to pry off a small area of wood. If the wood splinters, it is most likely sound. If the break is brash or crumbles, the wood is most likely decayed and may require treatment or removal. An experienced wood inspector should interpret the results of the “pick test”.

WOOD STRUCTURE INTERIOR DECAY DETECTION METHODS

The following section is reworked from a previous article written by the author. Degradation of the wood may occur (as often the case) on the inside of the wood member due to a lack of visible indicators. Such interior deterioration is difficult to detect. Several advanced methods and equipment exists for assessing interior damage. Such advanced equipment involves sonic evaluation and such devices as x-ray equipment. Other, less advanced equipment include hammer sounding, moisture meters, drilling and coring,

Hammer Sounding

Sounding the wood surface is completed by striking the wood piece or structural member with a hammer and evaluating the tonal quality. A trained inspector can interpret dull or hollow sounds that may indicate internal decay. Many factors other than decay can influence the sound of wood struck with a hammer, thus this method is prone to misinterpretation.

Moisture Meters

Moisture meters can help identify wood at high moisture content, and high moisture content wood is a suspected area of potential decay. Untreated wood moisture contents higher than 20-25%

indicates conditions suitable for decay. See Figure 12 below for a bridge inspector in Victoria utilizing a moisture meter to assess a log girder that has very high moisture content.

Figure 12. Moisture content meter in use to determine moisture content in a log girder in Victoria, Australia.

Drilling and Coring

The use of core and drilling devices such as shown in Figure 5 above are the most common methods of interior decay detection. Due to their similarities they are included together.

Drilling can be completed using either a hand or power drill to drill into the structure at different locations noting the torque resistance and observing the drill shavings for evidence of decay. The advantage of using a hand drill is that it provides the inspector a better feel of the drill bits' torque resistance.

Core samples can be taken with increment borers provide samples that can be analyzed to show the limit and extent of deterioration and provide lab samples. Lab samples can be further analyzed and processed to provide samples for cultures to indicate the presence of decay fungi and provide an assessment of future risk and also to analyze the woods' specific gravity.

Suspected decay areas that are cored should utilize sharp tools so that crushed wood, caused by a dull bit, will not be mistaken for decay. See Figure 13 below for photographs of core sample recovered from Red Box, a typical Victorian hardwood utilized in round log form several years ago to construct bridges.

Figure 13. A core sample taken with an increment borer from Red Box round log girder in Victoria, Australia.

Advanced Detection Techniques for Internal Degradation Detection - Sonic Evaluation

Several different sonic wave propagation methods are now in use to detect degradation inside a wood element. Included in this method are sonic wave velocity, acoustic emission, and stress wave analysis. The simplest of these methods measures the velocity change of a sound wave moving across the wood element are various locations. The velocity can be directly related to the density of the wood inside the structural elements.

These internal degradation methods are often referred to as nondestructive evaluation methods. A large volume of research by the author and others in this area has now allowed very accurate measurement of the internal degradation levels in wood structural members in situ. Progress has now been made in the development of the stress wave velocity technology and other technologies for assessing the residual performance of wood in structures. For example researchers have now shown acoustic emission techniques can be now be used to detect the presence of termites in wood members. Ships like the famous last great whaling ship called the Morgan are now being considered for a full internal investigation of the all the timbers utilizing sound wave velocity.

The stress wave analysis method consists of sending a “sound” wave through a medium (wood) and measuring its velocity. The sound wave is introduced into the material by striking it with a hammer or blunt object. When the vibrations reach an accelerometer, an accurate timer is started; when the sound reaches a second timer, the timer is stopped. The distance between the “start” and “stop” accelerometer is measured. Knowing distance and time, the average velocity of the stress wave (sound wave) can be measured. See Figure 14 below for photographs of the stress wave time machine being utilized to determine a bearer internal degradation level in the Seymour bridge in Victoria, Australia. In addition Figure 15 shows the process being utilized in the waterline area from a boat in the Goulburn River at the Seymour bridge in Victoria, Australia.

Figure 14. Stress Wave Timer equipment being utilized to determine the internal degradation in a bearer in the Seymour Bridge in Victoria, Australia.

Figure 15. Stress Wave Timer equipment being utilized to determine the internal degradation in a pier at the water line in the Goulburn River in the Seymour Bridge in Victoria, Australia.

The Modulus of Elasticity of the material can be related to the velocity of the stress wave and the wood density.

Typically, the MOE for sound hardwoods and softwoods can be calibrated utilizing cores from the structure to develop reasonably accurate calibration tables for use with the stress wave time data. If the wood sample has been subject to fungi decay, the specific gravity (weight relative to water) will decrease. The decrease in specific gravity causes a decrease in the modulus of elasticity, which decreases the velocity of the stress wave. Therefore, if decay is present, the measured MOE using the stress wave timer will be significantly lower than the expected range.

As discussed above the calibration of the stress wave timer is a critical step. Taking core samples of sound wood and the wave propagation velocity in the same sound wood establishes the calibration. The wave propagation velocity in sound (non-decayed) wood must be well established and repeatable before attempting to locate decayed areas. The manner in which the velocity is measured must be consistent to minimize variation and false readings.

A sample calibration curve for the stress wave timer when used on Douglas-fir beams and columns in situ is shown in Figure 13 below. The curve indicates the relationship between stress wave time and specific gravity. Because fungal decay tends to reduce the specific gravity, the stress wave timer may be used to indicate potential areas of decay. Generally, stress wave times greater than 300 microseconds per foot for softwoods and 250 ms for hardwoods indicates that fungal decay may have significantly degraded the strength and stiffness properties.

The stress wave timer calibration curve was created by measuring stress wave times on an existing glulam and then taking an assay sample with a core drill. Specific gravity of the assay samples was measured in a laboratory to develop the curve in Figure 13.

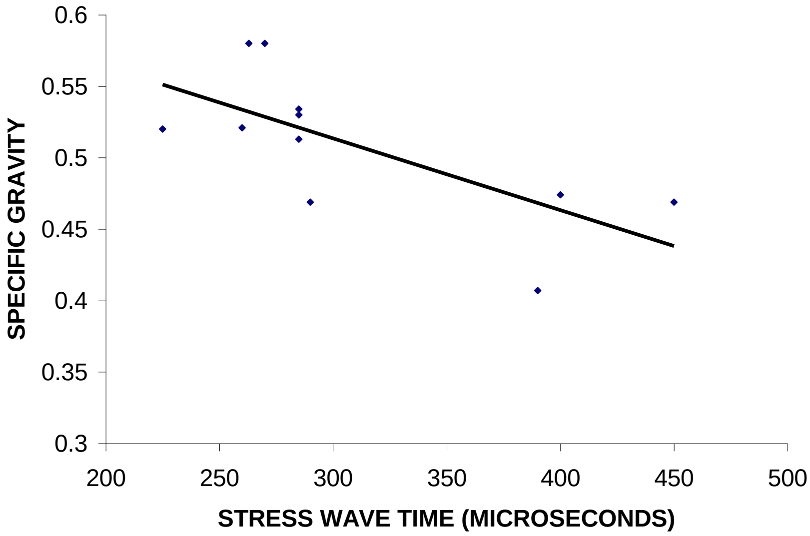


Figure 13. Stress wave timer calibration curve.

Utilizing Stress Wave Time Data

Amputation and fumigation details can be developed during the rehabilitation of structures utilizing stress wave time data.

Figure 14 below shows a contour map drawn on the side of a beam, created with the stress wave timer. This figure shows that the very end of the beam has relatively high stress wave time values (greater than 300 microseconds per foot) indicating areas of low density due to fungal decay. The stress wave times tend to decrease toward the wall indicating higher density. The covered area next to the wall has the lowest stress wave times because this region is subjected to the least amount of moisture.

Water is easily absorbed and transported parallel to the grain in wood structure elements. This factor can often provide enough moisture for active fungal attack many feet from the source. An especially harmful situation occurs when an exposed beam end collects and absorbs water to areas in the beam where loads are great and decay must be limited such as the load bearing exterior wall in the beam shown in Figure 17.

A combination of an amputation in a slash direction to limit further water exposure coupled with end-sealing with a high solids coating and paraffin wax, and preservative treatment is used to renovate the beam ends. See Figure 18 below. The design of the slash-cut meets two key parameters:

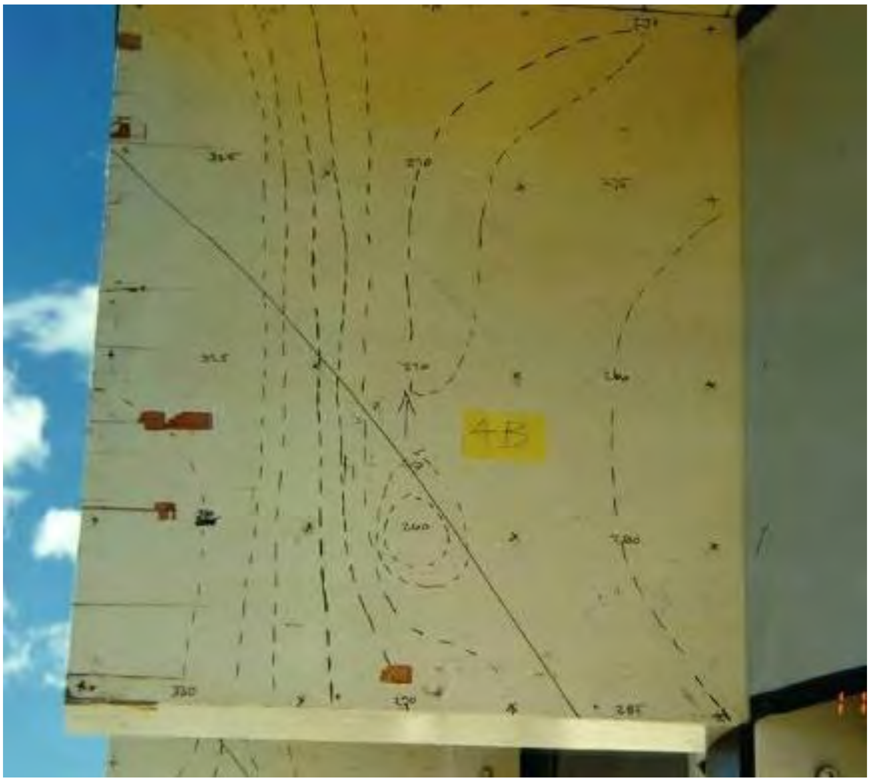


Figure 14: Stress wave time contour map used to determine areas of decay.

Figure 18: Photograph of a slash-cut detail coupled with end sealing with paraffin wax.

X-Ray Devices

X-ray scanners are now being developed to provide internal images of wood elements in attempt locate defects but the high cost of equipment, safety factors and expertise needed have curtailed its use.

ADVANCED METHODS OF PREVENTING WOOD DETERIORATION IN BRIDGES

Preventing wood deterioration involves many factors mostly related to moisture control, or preservative treatments. Good construction detailing can be much more effective to resist decay than preservative chemicals. Trees use multiple defense systems to prevent or slow the growth of fungi. The first is the bark, which provides a very effective barrier against fungi attack. Second is the sapwood (the living part of the xylem) which can respond to fungi attack by terminating the cellular metabolism in the affected areas which may create an adverse environment for the fungi. The sapwood can also transport resins to seal off the infected area thereby reducing the extent of the decay. When the tree is felled and sawn into lumber or used in round log form as a girder in a bridge the sapwood can be particularly susceptible to decay and in some cases must be removed to keep the decay causing fungi from spreading. The heartwood (the nonliving portion of the xylem) contains chemicals and extractives hostile to fungi. The heartwood of some species of redwood, red box and cedar can be used outside in decks with no chemical treatments because of the excellent fungi resistant chemicals naturally contained in the heartwood. Heartwood can also become plugged with growths called tyloses which restrict the movement of water and fungi.

There are many types of man-made chemical preservatives, which are used to prevent fungi attack. The best known is creosote, which was often used to preserve wood bridges. Pentachlorophenol is also used to treat bridge girders where human exposure is limited. Due to the leaching of pentachlorophenol and its toxicity, its use is limited. Chromate copper arsenate (CCA) is an effective wood preservative which is relatively safe for humans that is often utilized in bridge components. Unfortunately, the treatment process for CCA uses water as the transport mechanism, which can cause splits and checks, especially for larger wood members. The effectiveness of CCA in the heartwood is in question due to generally poor penetration (often caused by tyloses). Thus, its use is often limited to treatment after all machining of the bridge element has been completed.

Most chemical treatments require special pressure tanks to obtain the necessary penetration depth for effective decay resistance. Surface treating is not nearly as effective as pressure treatment because once the protective coating is broken by localized splits, checks, and moisture cracks an avenue for fungi attack is created. This creates problems for fixing existing wood structures or components in-situ. Fumigants were developed to provide chemical protection without the requirement for pressure treatment. This allowed structures already in the field to be treated. The first use of the technology was applied to wood utility poles and has developed from there to use in beams and columns.

In-Place Preservative Treatments

To restrict decay in existing wood members or structures, in-place preservative treatment methods are used. The most common type of in-place treatment method for bridges is fumigants.

Fumigants

Fumigants are preservative chemicals in liquid or solid form placed in predrilled holes to stop internal decay in bridge structure components. Over time the fumigants vaporize into gas and move through the wood stopping decay from continuing and insects from inhabiting the piece. With some fumigants like the borates this vaporization continues based on the moisture content of the wood and will conveniently become active when the moisture content in the wood moves upward past the point at which decay begins (20% MC). Fumigants can diffuse 8 feet or more from the point of application in vertical members, and 2-4 feet in horizontal members and are most effective when applied to sound wood. Different fumigants diffuse at different rates and will eventually diffuse out of the wood requiring a reapplication.

Boron is a type of fumigant and is very effective in controlling wood decay and is relatively less toxic to humans than other chemical preservatives. Boron is processed into rods, similar to glass rods, and inserted into predrilled holes in a structural wood member and plugged with pressure treated bungs. The boron rods will slowly dissipate over time and the natural moisture in the wood facilitates the migration of the boron through the pores.

Further wood decay can be prevented by using boron rods. The number and type of boron rods required for a specific treating situation is based on the volume of exposed wood.

ADVANCED BRIDGE RESTORATION, REFURBISHMENT, IN SITU TESTING AND UPGRADING USING HIGH STRENGTH FIBERS

High-strength fiber reinforced plastics are used to increase the strength and stiffness of existing bridge beams in-place. These FRP's have several advantages over other materials, including better wood compatibility (allows optimization of material strengths), small size and weight, very high allowable strengths, and low cost. The following section contains a copy of a paper prepared by the author on a bridge retrofit project in Nova Scotia Canada.

Three Nova Scotia Transportation and Public Works (NSTPW) bridges in Nova Scotia, Canada: Hay Cove, Soldiers Cove and Quoddy Bay were inspected. Subsequent to the inspection utilizing advanced non-destructive testing techniques the in-situ properties were determined, a retrofit utilizing high-strength fiber was designed, and then the bridges were rehabilitated utilizing high-strength fiber. Two of these bridges; Hay Cove and Quoddy Bay had been load tested before inspection and retrofitting. These bridges were again load tested after the retrofit work to ascertain the accuracy of the retrofit design methodology and the retrofit material performance. This paper discusses this work and the results that were obtained.

The goal of the inspection work on the bridges was to determine the in-situ condition of the longitudinal girders and to evaluate their current load carrying capacity. The inspection techniques included visual inspection, core sampling, moisture content readings and stress wave time measurements. The combination of the various inspection techniques provides a reasonable estimate of the amount, and degree, of wood deterioration that was present. The assessment that followed the inspections provided the in-situ bridge load limitations. These values were then utilized to design a high-strength fiber reinforcement retrofit that brought the bridges up to the desired load rating for current traffic requirements. The bridges had been in service for about 50 years, and the current live load rating factors were less than 1, and in the case of one of the bridges, Quoddy Bay was .46. Thus, two issues were presented. The first pertained to the condition of the wood in the girders. The second, best addressed with full knowledge of the first – the wood condition was how to increase the load carrying capacity to safely carry heavy truck loads. The bridge surfaces were constantly cracking (see Figure 24) due to overloading and maintenance of the bridges was expensive due to the overload traffic and amount of traffic. The scope of this report will be limited to the results of the investigation of the condition of the wood girders.

The following inspection techniques and findings were common to each bridge site, Hay Cove, Soldiers Cove, and Quoddy Bay. Inspection was limited to the girders.

Inspection Techniques

Visual inspection, moisture-content measurements, stress wave timing, and core sampling were the main inspection techniques performed for girder assessment. Measurements of girder depth, width, spacing and span length were recorded as well as decking and asphalt thickness. Measurements affecting structural performance of the girders were recorded. Observations were made throughout the inspection for fruiting bodies, sunken faces, staining and discoloration, bulging of wood grain, insect activity, plant or moss growth and other signs of deterioration. Notes were taken at each site, and are included as appendix material. Moisture content measurements were recorded using a Wagner Moisture Meter L601-3, which is a surface type meter (i.e. does not penetrate wood with pins).

Stress wave times were obtained using a Stress Wave Timer. All transmission times were determined perpendicular to the grain. Each bridge girder had the stress wave times recorded for the end and mid-span regions. The end regions were about 0.5 m from the girder supports, and the mid-span region was within 0.5 from the exact mid span. At each region the transmission times were determined at the "top" (near the decking), mid-depth, and bottom (nearest the water) of the beam. The top and bottom measurements were actually 50-75mm from the actual top and bottom towards the mid-depth. Core samples were taken to confirm or further define the extent of decay given by the stress wave time measurements. The specific gravity analysis of the cores is part of the design segment and will be included in a later report.

Stress Wave Time Analysis

Previous research[6] of creosote treated Douglas-fir, has found the following perpendicular to grain stress wave transmission times: for sound wood 1279 μs/m (390 μs/ft), moderately decayed wood 1827 μs/m (557 μs/ft), and severely decayed wood 2430 μs/m (741 μs/ft). These velocities are in good agreement with previous findings by WSTI and they will provide the basis for WSTI's judgments of decay based on SWT's for the NSTPW bridges.

Species Identification

The wood species of the girders, for each of the three sites, is confirmed to be Douglas-fir. Longitudinal sections from core samples obtained from the girders were observed under a microscope. The presence of spiral thickenings on the inside of the cell wall identifies the wood species as Douglas-fir.

Creosote Preservative

The girders were all treated with creosote, showing penetration of about 10-15 mm from the core samples.

HAY COVE BRIDGE FINDINGS

Figures 19-20 provide direction orientation and show the girders.

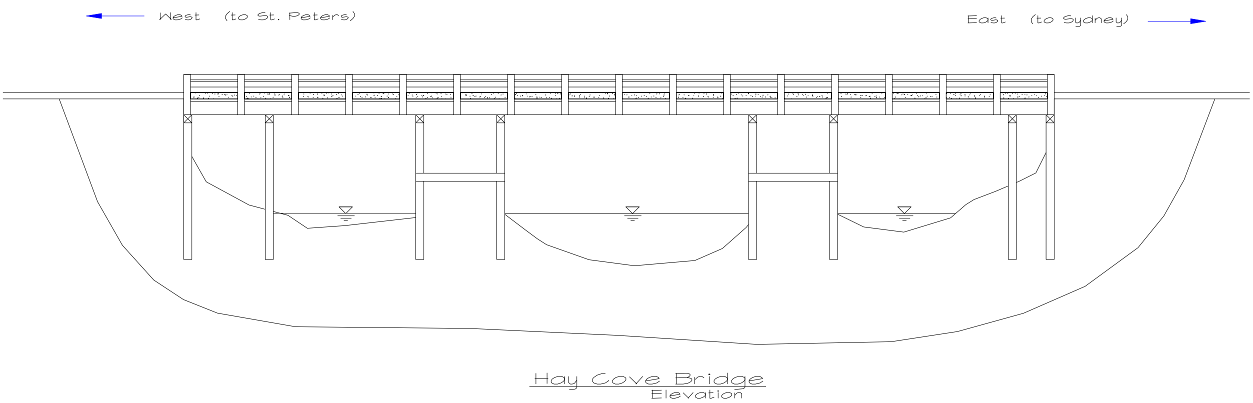


Figure 159. Hay Cove bridge elevation showing East-West direction.

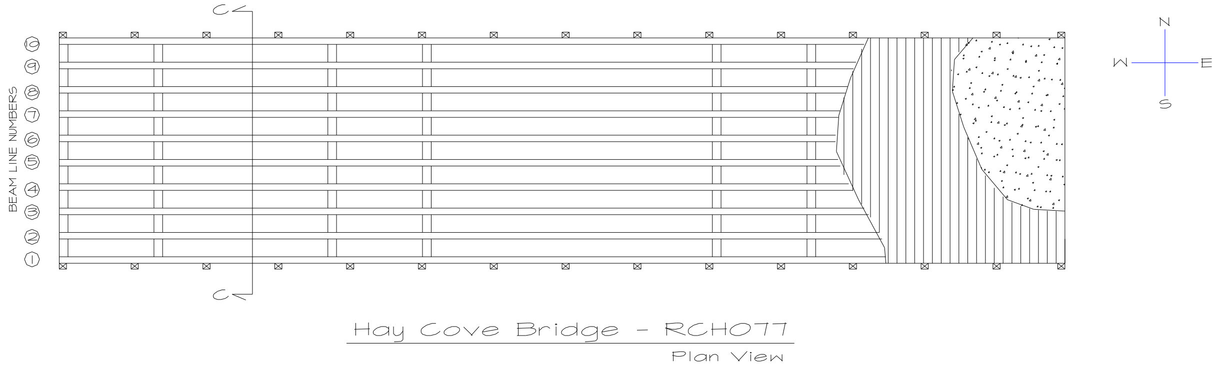


Figure 160. Hay Cove plan view showing girder numbering with respect to direction.

Visual Inspection

The girder dimensions are approximately, 225-mm wide, 490-mm deep (9”x20”) and have a 9.38-m (30.5 ft.) clear span. The center to center spacing is not equal for all girders but is averaged to be 840-mm (33 in.) contains a photograph which shows the girders.

Figure 21. Hay Cove girders.

Girder (#6) had a large crack at mid-span at the bottom of the beam, possibly due to overload. Stress wave times through this cracked region were much slower, indicating the crack is significant through the beam width. Figure 22 shows a picture of this cracked girder.

Figure 22. Cracked girder.

The north outer girder (#1) had visible water runoff from the deck onto the side of the beams. Accordingly, this girder had the highest measured moisture content, otherwise the girders showed no obvious signs of deterioration.

Stress Wave Times

Stress wave transmission times (SWT's) perpendicular to the grain were obtained at the ends and mid-span of each girder, and the results are given in Table 1. The decay severity was determined as previously discussed. Table 1 shows that only the outside girders showed signs of decay. These outer girders had the highest moisture content, visible water on the surfaces due to road runoff of the crowned road surface, and also had bolt holes through the side to fasten guard railing as well as spikes from the decking penetrating the top side of the girder through the creosote region. The combination of high moisture content, penetrating spikes, and bolt holes had led to some decay.

Table 1. Stress Wave Transmission Times Perpendicular to Grain for Hay Cove.

		Velocity of Sound Waves Through Stringers						
		East End (To Sydney)		Midspan		West End (To St. Peters)		Moisture Content
Stringer	Depth	μs/m	(μs/ft)	μs/m	(μs/ft)	μs/m	(μs/ft)	%
1 (South)	Top	1333	(406)	1911	(583)	4889	(1490)	20-30
	Middle	800	(244)	1867	(569)	933	(284)	
	Bottom	1556	(474)	1378	(420)	978	(298)	
2	Top	1111	(339)	1200	(366)	1156	(352)	14-16
	Middle	800	(244)	1111	(339)	889	(271)	
	Bottom	1422	(433)	1156	(352)	1111	(339)	
3	Top	978	(298)	1156	(352)	1111	(339)	15-20
	Middle	933	(284)	844	(257)	1422	(433)	
	Bottom	933	(284)	1111	(339)	1067	(325)	
4	Top	1200	(366)	1244	(379)	1333	(406)	25
	Middle	933	(284)	889	(271)	978	(298)	
	Bottom	1244	(379)	1111	(339)	1111	(339)	
5	Top	1022	(312)	1022	(312)	1022	(312)	16-20
	Middle	978	(298)	933	(284)	800	(244)	
	Bottom	1022	(312)	1156	(352)	1067	(325)	
6	Top	933	(284)	889	(271)	978	(298)	18-22
	Middle	889	(271)	933	(284)	889	(271)	
	Bottom	933	(284)	1200	(366)	978	(298)	
7	Top	1111	(339)	1244	(379)	1022	(312)	18-24
	Middle	978	(298)	889	(271)	844	(257)	
	Bottom	1200	(366)	1333	(406)	1156	(352)	
8	Top	1022	(312)	1289	(393)	1156	(352)	14-18
	Middle	933	(284)	844	(257)	933	(284)	
	Bottom	1289	(393)	1244	(379)	1333	(406)	
9	Top	1156	(352)	1067	(325)	1111	(339)	14-18
	Middle	978	(298)	1022	(312)	1244	(379)	
	Bottom	1156	(352)	1022	(312)	978	(298)	
10 (North)	Top	1778	(542)	2000	(610)	2667	(813)	17-18
	Middle	1156	(352)	1778	(542)	2889	(881)	
	Bottom	1422	(433)	1067	(325)	5333	(1626)	

Key to shading:	Sound wood	Moderate decay	Extensive decay

Note; The SWT's show that the majority of the girders are in sound condition.

Moisture Content

The moisture content for each girder is given in Table 1. The outside girder had the highest moisture content due to roadway runoff.

Core Samples

Core samples were taken to verify the species type, SWT's, and determine the specific gravity. Core samples were taken at the slow SWT locations to determine extent of decay. The core samples gave a good visual agreement with the SWT analysis.

SOLDIERS COVE BRIDGE FINDINGS

The following Figures 23-24-25 provide direction orientation and show the identifying numbers for each girder.

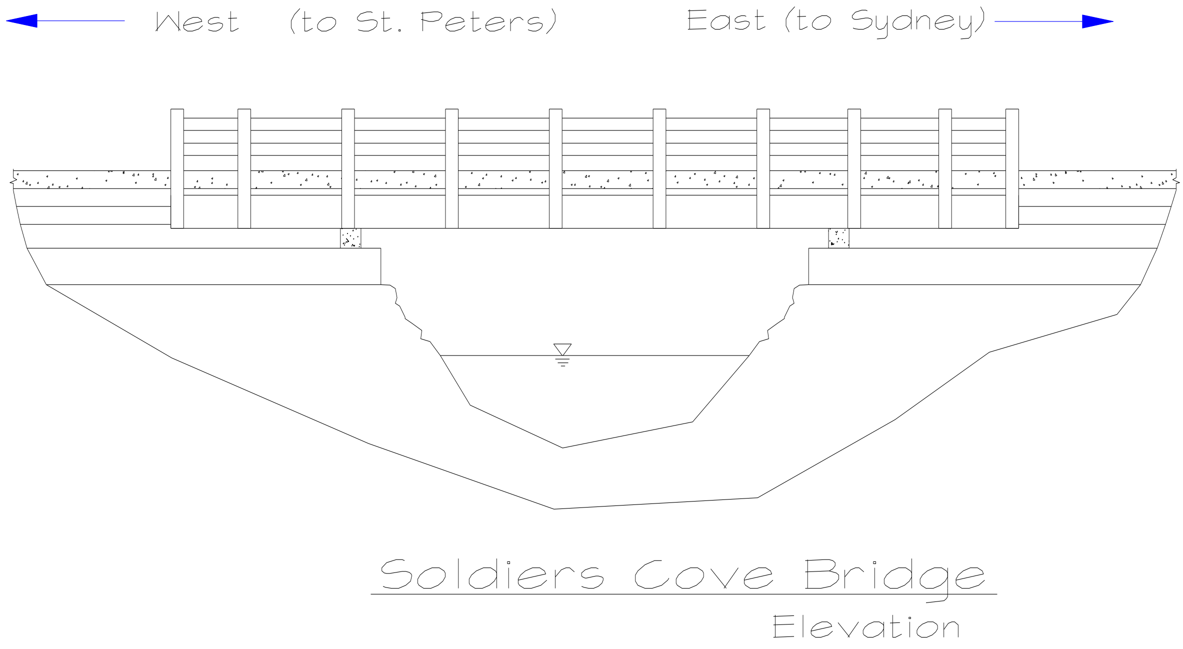


Figure 23. Soldiers Cove elevation view showing East-West direction.

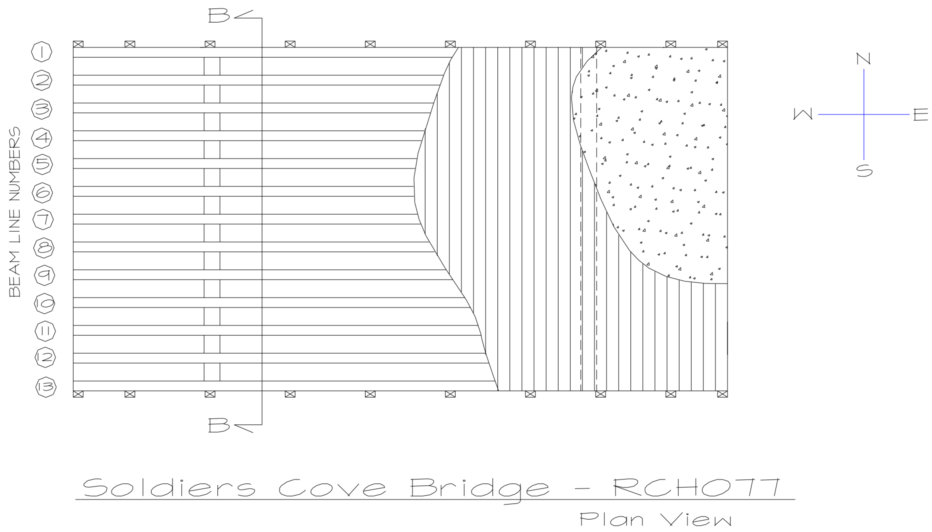


Figure 24. Soldiers Cove plan view showing girder numbering.

Visual Inspection

The girder dimensions are approximately, 250-mm wide, 490-mm deep (10 in. x 20 in.) and have a 9.02-m (30 ft.) clear span. The center to center spacing is not equal for all girders but is averaged to be 724-mm (28.5 in.). The north and south outer girders had visible water runoff from the deck onto the side of the beams. Accordingly these girders had the highest measured moisture content. After a core sample was taken from girder #1 (north outer girder), water began dripping out the hole, indicating the wood fibers were saturated with water (moisture content greater than 30%), Table 2 shows severe decay in this region. The other outer girder (#13) had a damage “hole” in the side of it, and this damaged area also had SWT’s indicating decay.

Figure 25. Soldiers Cove outer girder showing damage “hole” on side (top) and girder end shear crack (bottom)

Stress Wave Times

Stress wave transmission times (SWT's) perpendicular to the grain were obtained at the ends and mid-span of each girder, and the results are given in Table 2. The decay severity is determined as previously discussed.

Table 2 Stress wave transmission times perpendicular to grain for Soldiers Cove.

		Velocity of Sound Waves Through Stringers						
		East End (To Sydney)		Midspan		West End (To St. Peters)		Moisture Content
Stringer	Depth	μs/m	(μs/ft)	μs/m	(μs/ft)	μs/m	(μs/ft)	%
1 (North)	Top	12000	(3658)	2800	(853)	4800	(1463)	20-30
	Middle	1400	(427)	880	(268)	960	(293)	
	Bottom	8000	(2438)	840	(256)	1080	(329)	
2	Top	920	(280)	1080	(329)	920	(280)	
	Middle	800	(244)	880	(268)	680	(207)	
	Bottom	840	(256)	920	(280)	1200	(366)	
3	Top	3400	(1036)	960	(293)	960	(293)	
	Middle	1000	(305)	760	(232)	760	(232)	
	Bottom	1000	(305)	1000	(305)	1000	(305)	
4	Top	880	(268)	840	(256)	840	(256)	
	Middle	680	(207)	720	(219)	760	(232)	
	Bottom	1800	(549)	880	(268)	840	(256)	
5	Top	2000	(610)	880	(268)	1120	(341)	
	Middle	840	(256)	800	(244)	760	(232)	
	Bottom	1000	(305)	860	(262)	960	(293)	
6	Top	1280	(390)	920	(280)	920	(280)	
	Middle	1040	(317)	720	(219)	680	(207)	
	Bottom	920	(280)	1120	(341)	1000	(305)	
7	Top	1200	(366)	880	(268)	840	(256)	
	Middle	2800	(853)	800	(244)	800	(244)	
	Bottom	2000	(610)	1080	(329)	1200	(366)	
8	Top	2600	(792)	860	(262)	920	(280)	
	Middle	4000	(1219)	760	(232)	720	(219)	
	Bottom	1600	(488)	920	(280)	720	(219)	
9	Top	1000	(305)	880	(268)	880	(268)	
	Middle	720	(219)	800	(244)	800	(244)	
	Bottom	1000	(305)	840	(256)	840	(256)	
10	Top	960	(293)	880	(268)	1000	(305)	
	Middle	760	(232)	800	(244)	840	(256)	
	Bottom	920	(280)	880	(268)	960	(293)	
11	Top	760	(232)	860	(262)	800	(244)	
	Middle	800	(244)	840	(256)	720	(219)	
	Bottom	920	(280)	800	(244)	920	(280)	
12	Top	2920	(890)	960	(293)	1600	(488)	
	Middle	2000	(610)	880	(268)	1000	(305)	
	Bottom	2800	(853)	980	(299)	920	(280)	
13 (south)	Top	6000	(1829)	4000	(1219)	2800	(853)	
	Middle	3600	(1097)	1800	(549)	1280	(390)	
	Bottom	3800	(1158)	1720	(524)	1480	(451)	

Key to shading:	Sound wood	Moderate decay	Extensive decay

Table 2 shows that the outside girders and the east-end of several interior girders showed signs of decay. These outer girders had the highest moisture content, visible water on the surfaces due to road runoff of the crowned road surface, and also had bolt holes through the side to fasten guard railing as well as spikes from the decking penetrating the top side of the girder through the creosote region.

The combination of high moisture content, penetrating spikes, and bolt holes had led to some decay. The decayed portions of the outer girders extend along the entire length near the top of the girder. The SWT values show that the majority of the girders, all interior girders, are in sound condition.

Moisture Content

The moisture content for the girders is given in Table 2. The outer girders had the highest moisture content, likely due to roadway runoff.

Core Samples

Core samples were taken to verify the species type, SWT's, and determine the specific gravity. Core samples were taken at the slow SWT locations to determine extent of decay. The core samples provided good visual agreement with the SWT analysis.

QUODDY BAY BRIDGE FINDINGS

The following Figures (26-27) provide direction orientation and show the identifying numbers for each girder.

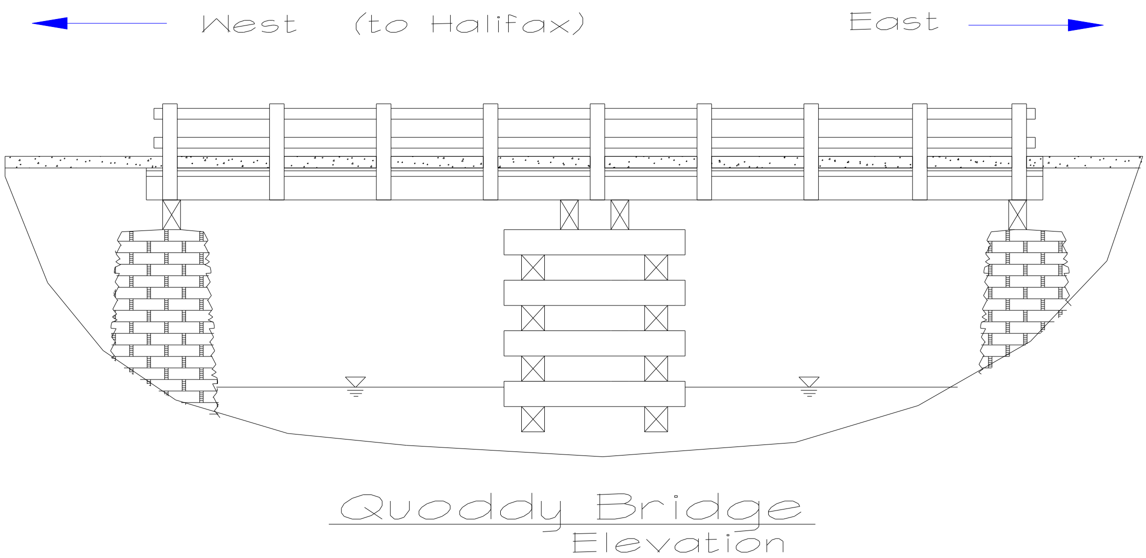


Figure 26. Quoddy Bay bridge elevation showing east-west directions.

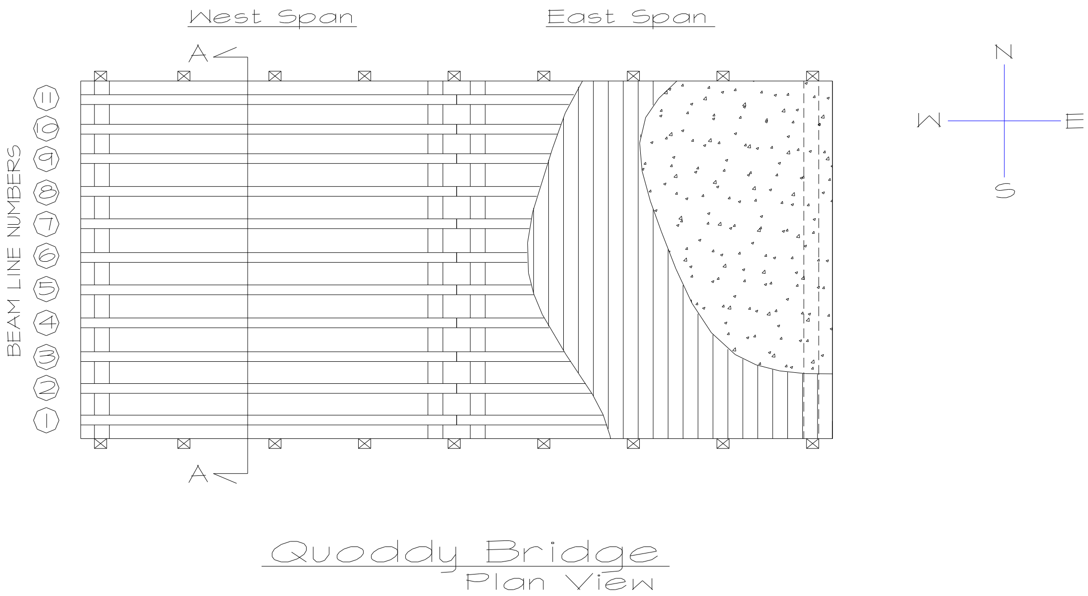


Figure 27. Quoddy Bay Bridge plan view showing spans and girder numbering.

Visual Inspection

The girder dimensions are approximately, 200-mm wide, 403-mm deep (8 in. x 16 in.) and have a 7.007-m (23 ft.) clear span. The center to center spacing is not equal for all girders but is averaged to be 671-mm (26.4 in.). This bridge has two equal spans, referred to as east and west spans. The bridge is shown carrying a logging truck in Table 3

Figure 17. Quoddy Bay Bridge carrying a heavy log truck.

Again, the north and south outer girders had the highest measured moisture content, from the crowned road runoff. The girders showed no obvious signs of deterioration.

Stress Wave Times

Stress wave transmission times (SWT's) perpendicular to the grain were obtained at the ends and mid-span of each girder, and the results are given in Table 3, for the west and east spans respectively. The decay severity was determined as previously discussed. Table 3, SWT's for the west span, showed decay only at the top edge on the outer girders, and inner girder #2, otherwise the wood appeared sound. For girder #2 the decay occurred at the top edge extends about 1-m to each side of the mid-span, based on stress wave times not reported in Table 3.

Table 3 Stress wave transmission times perpendicular to grain for Quoddy West Span.

		Velocity of Sound Waves Through Stringers						
West Span		**West End**		**Midspan**		**East End**		**Moisture Content**
Stringer	**Depth**	**μs/m**	**(μs/ft)**	**μs/m**	**(μs/ft)**	**μs/m**	**(μs/ft)**	**%**
1 (South)	Top	1900	(579)	2000	(610)	3750	(1143)	20-25
	Middle	900	(274)	750	(229)	900	(274)	17-23
	Bottom	1100	(335)	1600	(488)	1600	(488)	15-20
2	Top	1500	(457)	7500	(2286)	1050	(320)	18
	Middle	1100	(335)	1000	(305)	850	(259)	15
	Bottom	850	(259)	1000	(305)	1000	(305)	13
3	Top	750	(229)	900	(274)	900	(274)	18
	Middle	1100	(335)	900	(274)	850	(259)	15
	Bottom	900	(274)	1000	(305)	1000	(305)	13
4	Top	850	(259)	1050	(320)	1250	(381)	18
	Middle	900	(274)	900	(274)	1100	(335)	15
	Bottom	1250	(381)	1000	(305)	1100	(335)	13
5	Top	1100	(335)	650	(198)	800	(244)	18
	Middle	1250	(381)	1100	(335)	950	(290)	15
	Bottom	1150	(351)	1050	(320)	1100	(335)	13
6	Top	1250	(381)	950	(290)	900	(274)	18
	Middle	750	(229)	900	(274)	750	(229)	15
	Bottom	1200	(366)	1200	(366)	1000	(305)	13
7	Top	900	(274)	950	(290)	900	(274)	18
	Middle	950	(290)	950	(290)	850	(259)	15
	Bottom	1150	(351)	900	(274)	850	(259)	13
8	Top	900	(274)	750	(229)	1500	(457)	18
	Middle	1300	(396)	1200	(366)	1000	(305)	15
	Bottom	1050	(320)	1150	(351)	1250	(381)	13
9	Top	1050	(320)	800	(244)	1000	(305)	18
	Middle	1000	(305)	900	(274)	800	(244)	15
	Bottom	1050	(320)	1600	(488)	1050	(320)	13
10	Top	1250	(381)	1100	(335)	1300	(396)	18
	Middle	1350	(411)	1100	(335)	1050	(320)	15
	Bottom	1000	(305)	1000	(305)	1050	(320)	13
11 (North)	Top	2375	(724)	1000	(305)	1050	(320)	20-25
	Middle	1075	(328)	700	(213)	750	(229)	17-23
	Bottom	1250	(381)	1100	(335)	850	(259)	15-20

Key to shading:	Sound wood	Moderate decay	Extensive decay

Table 4, SWT's for the east span, also showed decay only at the top edge on the outer girders, otherwise the wood appears sound.

Table 4 Stress wave transmission times perpendicular to grain Quoddy Bay Bridge East Span.

East Span		Velocity of Sound Waves Through Stringers						Moisture Content
		West End		Midspan		East End		
Stringer	Depth	µs/m	(µs/ft)	µs/m	(µs/ft)	µs/m	(µs/ft)	%
1 (South)	Top	1050	(320)	2500	(762)	2150	(655)	20-25
	Middle	800	(244)	1200	(366)	750	(229)	17-23
	Bottom	900	(274)	1000	(305)	1250	(381)	15-20
2	Top	1050	(320)	1200	(366)	900	(274)	18
	Middle	1100	(335)	950	(290)	900	(274)	15
	Bottom	1100	(335)	900	(274)	900	(274)	13
3	Top	1000	(305)	900	(274)	850	(259)	18
	Middle	800	(244)	800	(244)	800	(244)	15
	Bottom	1000	(305)	950	(290)	950	(290)	13
4	Top	1150	(351)	1000	(305)	1000	(305)	18
	Middle	1000	(305)	750	(229)	675	(206)	15
	Bottom	900	(274)	750	(229)	950	(290)	13
5	Top	950	(290)	900	(274)	900	(274)	18
	Middle	750	(229)	950	(290)	900	(274)	15
	Bottom	950	(290)	850	(259)	1000	(305)	13
6	Top	850	(259)	950	(290)	950	(290)	18
	Middle	900	(274)	1000	(305)	850	(259)	15
	Bottom	1150	(351)	1100	(335)	1050	(320)	13
7	Top	1100	(335)	1100	(335)	1200	(366)	18
	Middle	900	(274)	700	(213)	1000	(305)	15
	Bottom	1100	(335)	950	(290)	1000	(305)	13
8	Top	1000	(305)	1000	(305)	1000	(305)	18
	Middle	850	(259)	1000	(305)	1000	(305)	15
	Bottom	800	(244)	1500	(457)	850	(259)	13
9	Top	1050	(320)	1150	(351)	1100	(335)	18
	Middle	850	(259)	900	(274)	1000	(305)	15
	Bottom	1050	(320)	1100	(335)	1250	(381)	13
10	Top	900	(274)	1200	(366)	1000	(305)	18
	Middle	1000	(305)	900	(274)	800	(244)	15
	Bottom	1000	(305)	900	(274)	1000	(305)	13
11 (North)	Top	1000	(305)	1000	(305)	2000	(610)	20-25
	Middle	700	(213)	850	(259)	800	(244)	17-23
	Bottom	1000	(305)	1200	(366)	1000	(305)	15-20

Key to shading:	Sound wood	Moderate decay	Extensive decay

The outer girders had the highest moisture content, and also had bolt holes through the side to fasten guard railing as well as spikes from the decking penetrating the top side of the girder through the creosote region. The combination of high moisture content (from road runoff), penetrating spikes, and bolt holes had led to some decay in these outer girders. The decayed portions of the outer girders extended along the entire length near the top of the girder, for the west span. It was more limited on the east span.

Moisture Content

The moisture content for the girders is given in Table 4. The outer girders had the highest moisture content due to roadway water runoff.

Core Samples

Core samples were taken to verify the species type, SWT's, and determine the specific gravity. Core samples were taken at the slow SWT locations to determine extent of decay. The core samples gave good visual agreement with the SWT analysis.

Bridge Inspection Findings and Conclusions

The condition of the girders in each of the three bridges was similar, mostly sound, except the outer girders, which showed the most decay usually near the top edge. A summary of each bridge follows:

Hay Cove – The interior girders appeared sound. The exterior girders showed moderate to extensive decay, especially the northern most girder (#10).

Soldiers Cove – About half of the interior girders showed moderate to severe decay, though only at the east (to Sydney) end. The exterior girders showed moderate to extensive decay, especially the southern girder (#13), which had visual damage exposing untreated wood.

Quoddy Bay – The west span showed slightly more decay than the east, and all decayed areas were limited to the top 50-100 mm of the girder depth. Interior girder #2 (south) of the west span had internal moderate to severe decay extending 1 m from each side of the mid span, otherwise no other interior girder gave indication of internal decay. The outer girders showed moderate and/or extensive decay at the top 50-100 mm of the beam depth.

Of the three bridges, Soldiers Cove had the most internal decay relative to the others. Only Soldiers Cove had multiple interior girders showing moderate to severe decay. All three bridges showed moderate to extensive decay in the outer girders. Quoddy Bay had the least internal decay relative to the others.

The recommendations for the three bridges were as follows:

-fumigation with a preservative treatment of all the girders with any sign of decay, and

-retrofit of existing girders to provide adequate strength and safety.

Retrofit Design Engineering for Hay Cove, Soldiers Cove and Quoddy

The followings section discusses the engineering methods employed for the retrofit of three Nova Scotia Department of Transportation and Public Works (NSDOT) bridges: Hay Cove, Soldiers Cove, and Quoddy. The retrofit and analysis are limited to the bridge girders only, and Canadian Limit

States Design methodology is followed. The retrofit is considered a new design, in which a composite laminate consisting of wood and high strength fiber is bonded to the existing girders to increase the existing girder moment resistance. The moment and shear resistance of the existing bridge girders was considered to determine the magnitude of retrofit required. The moment resistance of the existing stingers was less than the factored design moment. However, the shear resistance was greater than the factored design shear. Therefore the retrofit design was limited to increasing the girder moment resistance, so that the design resistance was greater than the design load. Deflection limitations were not considered at the request of NSDOT

The design load analysis was provided by the NSTPW, and the retrofit design is designed so that the limiting resistance is 5% greater than the maximum design load. The FiRP® Tension Lam was developed from the success of the FiRP® Reinforced Glulam. The moment capacity of the FiRP® Reinforced beam is directly related to the tensile (or ends joint) capacity of the tension zone wood, provided the percent of FiRP® Reinforcement per cross-section is adequate, thus in retrofitting the Nova Scotia bridges the limiting factors for the retrofit was the weakest strength reducing defect in the bottom tension face of the longitudinal girders.

Retrofit Design Method

The retrofit design includes the attaching of a high strength FiRP® Tension Lam bonded to the bottom (tension side) of the existing girders. The design objective was to find the most efficiently sized FiRP® Tension Lam that met code load requirements. The design was based on transformed sections and linear elastic analysis. The FiRP® Tension Lam is applied in a manner that allows complete stress transfer at the FiRP® Tension lam and girder interface, allowing the linear elastic design assumption to remain valid. A plastic design methodology, e.g. compression based design methodology, is now utilized in retrofitting wood structures. This method could have been implemented in the designs for the three **NSTPW** bridges, providing higher utilization of materials and lower retrofit costs. However, extensive testing proving this fact was not completed at the time of the retrofit of the **NSTPW** bridges. Thus, for this project, the linear-elastic FiRP® Technology analysis methodology was utilized. All current retrofit projects utilizing high strength fibers utilize the new compression based design strategy. It assumed stresses beyond conventional allowable stresses in the compression zone. The FiRP® Technology code approval for glulam reinforcement assumes full compression zone yielding. The compression zone contribution to resisting moment is approximated by using a blocked section with a constant compressive stress value (F_c). This compression block is offset in the moment couple by the tension in the reinforcement. The tensile stress in the reinforcement is limited by the tensile strain at the FiRP® Lamination wood interface on the neutral axis side of the reinforcement.

Transformed Section; A transformed section was calculated based on the modulus of elasticity of each material in each of the reinforced girders. Each material is “transformed” to an equivalent width based on the modulus of elasticity of the wood in the existing girders therefore the transformed section had a modulus of elasticity equal to the wood in the existing girders. This transformed section was then used to determine the stress distribution induced by the live loads, based on elasticity theory.

Loads; The dead load stresses acted on the existing girder section due to the weight of the girders, decking, and wearing surface. The live load stresses acted on the composite section after retrofit. The stress distribution for each load case was superposed (added) to obtain the total load stress distribution acting on the composite section. The limiting stress in this retrofit design was the tensile stress in the existing wood (bottom surface of the existing girders).

Fastening The FiRP® Reinforced Tension Laminations; The FiRP® Tension Lam was bonded to the existing wood with a structural epoxy. The shear stress at the bond was well under the allowable bond strength. Lag screws are used to provide adequate clamping pressure at the bond interface. The shear strength of the structural epoxy between untreated lumber and creosote treated lumber was addressed in a separate report submitted to the NSTPW. In summary, the strength of the epoxy bond was adequate to transfer the applied shear stresses at the interface.

Retrofit Cutoff Length; To determine the length of the FiRP® Tension Lams, the theoretical cutoff point was determined. The theoretical cutoff point was the location where the design strength of the existing girder equals the load applied (factored). This cutoff point determined by “moving” the design truck along the existing girder span, to find where the equal moments are closest to the support (i.e. maximize the moment in the beam near the support). The FiRP® Tension Lam was then extended at least 0.5 meters beyond the theoretical cutoff point.

Core Sample Analysis; The core samples obtained from the on-site investigation were tested to determine the specific gravity (oven dry basis). Although the samples were small, careful and exacting measurements allowed the laboratory staff to accurately obtain the specific gravity information.

The specific gravity results are shown below and are consistent with Douglas-fir:

Average	0.50
Coefficient of Variation	14%

Minimum	0.43
Maximum	0.64

RETROFIT INSTALLATION FOR QUODDY BAY AND HAY COVE

The water under the Quoddy Bay Bridge is tidal therefore the work was completed around low tide times. See 30 and

Figure 18 for photographs of the tension laminations.

Figure 30. Drilled and treated tension laminations for Quoddy Bay bridge.

Figure 18. Close-up of the tension laminations for the Quoddy Bay bridge.

Unfortunately, the tide dismantled the scaffolding several times during the retrofitting process making the retrofitting job very difficult and a better method had to be developed for subsequent bridge retrofit work. The heating of the area under the bridge was important as it was in January in Eastern Canada and temperatures were low and conditions due to snow were very difficult. However, after a few modifications methods were developed utilizing portable furnaces and tarps that allowed the temperature in the environment under the bridge to reach acceptable levels for working and for the epoxy to cure. The actual tension lam installation went smoothly after these modifications were developed. See

Figure,

Figure , and Figure 34 for installation photographs

Figure 32. Applying epoxy for the Quoddy Bay Bridge tension laminations

Figure 33. Drilling pilot holes (foreground) and installing lag screws (background) on the Quoddy Bay bridge.

Figure 34. Epoxy squeeze out on the Quoddy Bay bridge.

See Figure 35 for completed retrofit on a girder in the Quoddy Bay Bridge. For the most part the bond achieved between the bridge girder and the tension laminations was acceptable, but there was one tension lam that was bonded tightly on one side and had a gap on the other side, see Figure this had to be repaired. Considering the conditions under which the retrofit work was completed the work was well done.

Figure 35. Installed tension lamination on the Quoddy Bay bridge.

Figure 36. Gap between tension lamination and existing girder on the Quoddy Bay Bridge.

Figure 37. Exterior view of the Hay Cove Bridge during tension lamination installation.

The tension lams went up fairly smoothly. Placing two tension lams side by side on a girder was a little tricky, but manageable. The first girder retrofit took a long time to install all the lag screws because the beams were 25 ft long with a total of 51 lag screws per girder. This meant that the installation crews had to be very careful with the set time of the epoxy and they were forced to dispose of a few rollers and trays. To eliminate as much waste as possible, the installation crew tried to mix only enough epoxy for one tension lam.

There was some concern about some of the epoxy bonds because of the creosote layer on the existing girders in both bridges. When they were tightened the lag screws the epoxy squeeze out would have a fine layer of creosote covering the surface, see Figure . This concern was addressed in future retrofit projects and the concern eliminated with a preparation strategy that specified that at least 1/8" of wood was removed from the bottom of a creosoted beam. See Figure 39 for photographs of a properly prepared creosoted beam tension surface. See Figure 40 for an improperly planned surface on the bottom of a bridge girder. The beams for Quoddy Bay Bridge were installed in one day.

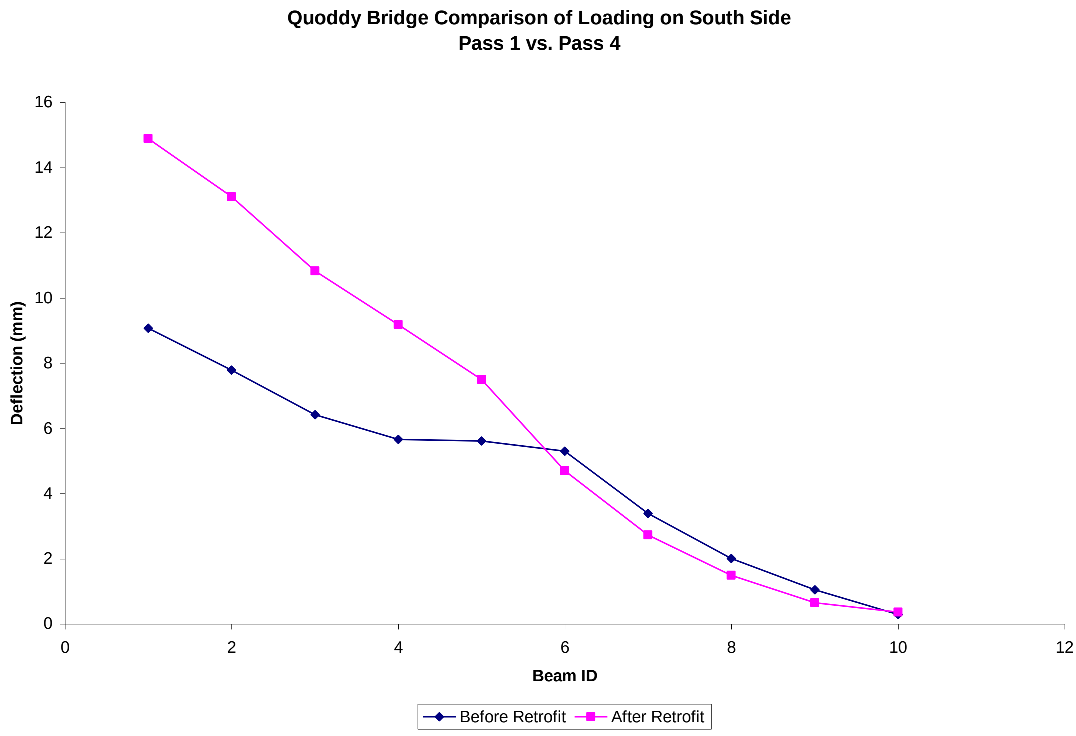
Figure 38. Creosote covered epoxy squeeze out.

Figure 39. Properly prepared surface on creosoted bridge girder in terms of depth of planning.

Figure 40. Properly prepared surface on creosoted bridge girder in terms of depth of planning. However note the improper uneven surface and planner gouge that had to be smoothed out for completion of retrofit installation.

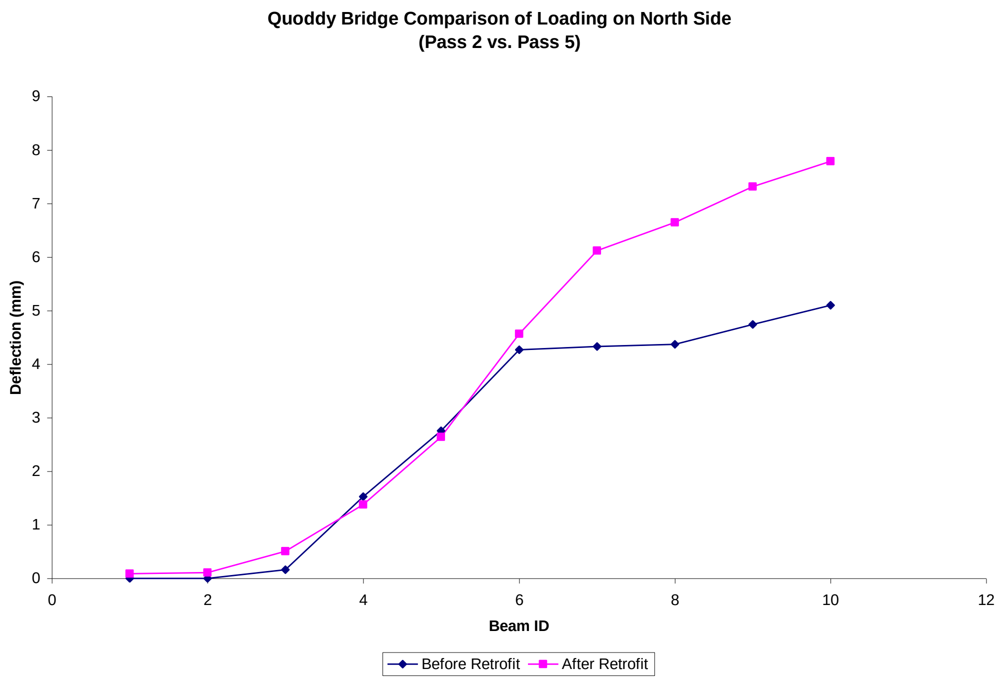
IN-SITU LOAD TESTING BEFORE AND AFTER FOR QUODDY BAY AND HAY COVE BRIDGES

Figures 41 and 42 below, show the results for load deflection testing before and after for Quoddy Bay and Hay Cove respectively. The results showed that stiffness was dramatically improved for the bridges. The testing (See Figures 43-44) involved the use of extensometers and **NSTPW** gravel trucks of known weights. These trucks were positioned at specified locations and deflection data recorded. Then these vehicles are driven across the bridge at various known speed and deflection measurements recorded. The trucks that were used had two back axles and on front axle had provided a 36T axle load.



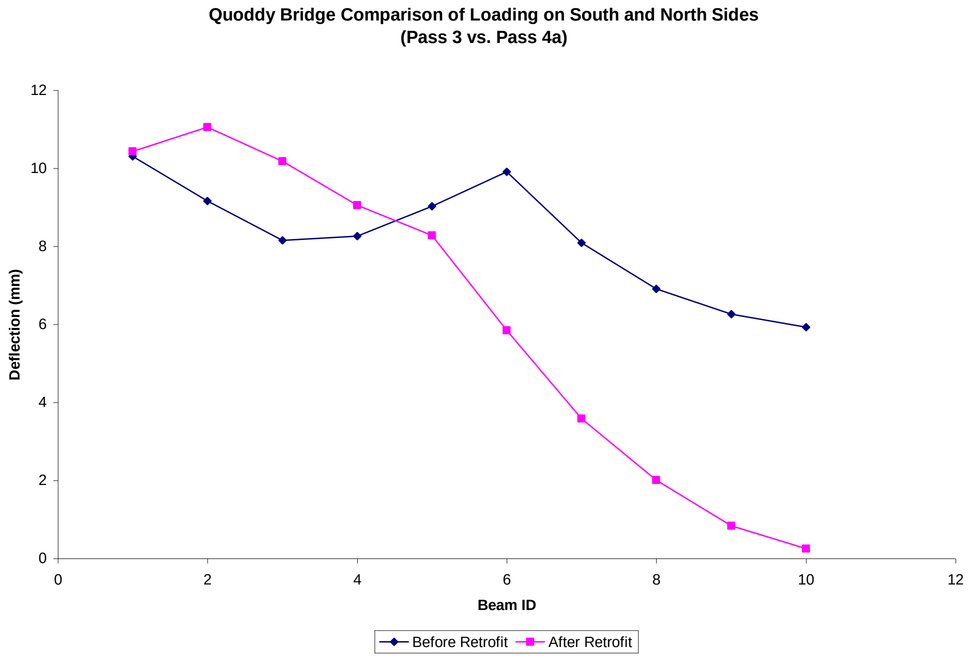
Quoddy Bridge Comparison of Loading on South Side
Pass 1 vs. Pass 4
Deflection (mm)
Beam ID
Before Retrofit
After Retrofit

(a)



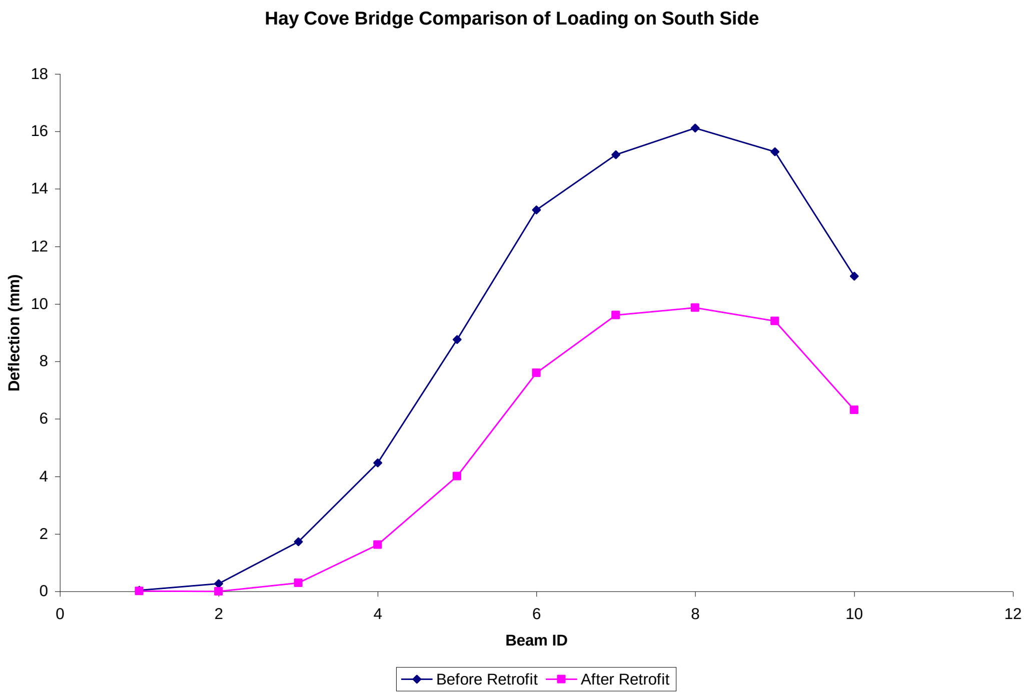
Quoddy Bridge Comparison of Loading on North Side
(Pass 2 vs. Pass 5)
Deflection (mm)
Beam ID
Before Retrofit
After Retrofit

(b)



(c)

Figure 41. Deflection data comparison between, before retrofit, and after retrofit for Quoddy Bay bridge. South side a). north side b.) and merged data c.)



(a)

(b)

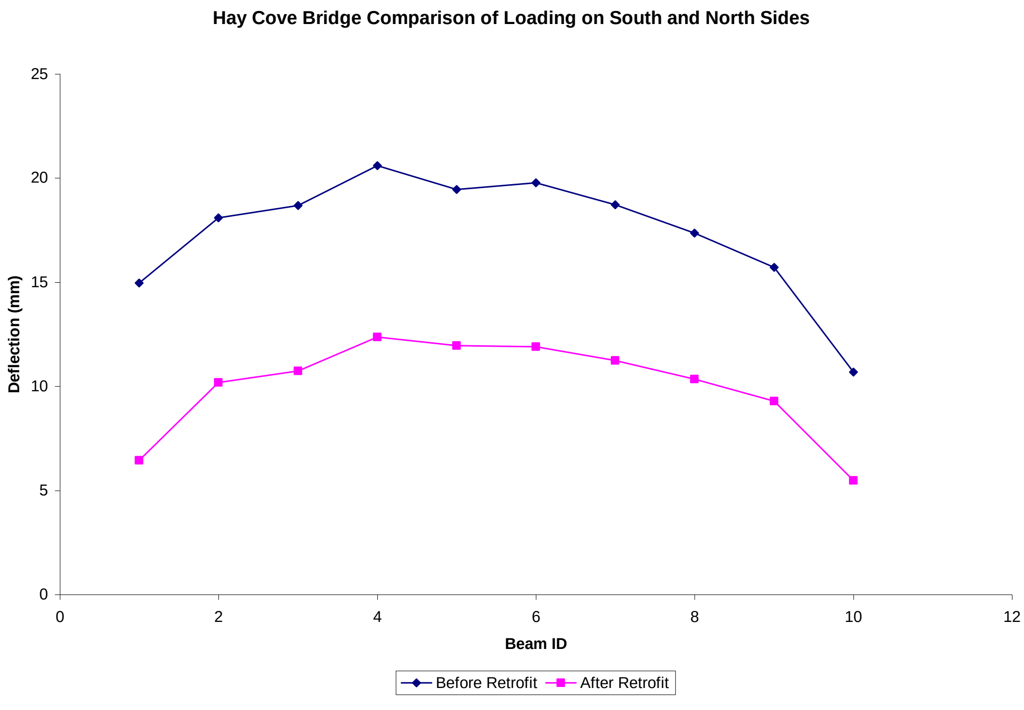


Figure 42. Deflection data comparison between before retrofit and after retrofit for Hay Cove Bridge. South side a), north side b) and merged data c).

Figure 43. Gravel truck utilized to create known weight (36 ton per axle) positioned at specified locations. Also note excessive cracking in roads over the bridge deck from overloading prior to retrofitting.

Figure 44. Extensometers and data gathering equipment

Please note that there were some data errors in the test results shown for the Quoddy Bay bridge. The deflection for the pre-retrofit case on the south side appeared to be reversed. The data logger out feed identification was improperly labeled it appeared. However, this couldn't be confirmed with the **NSTPW** engineering department. Clearly, the data is spurious and incorrect for the south side girders as the deflection would be less after the installation of the retrofit. See Figure 44 for photographs of load testing process.

COST COMPARISONS BETWEEN HIGH STRENGTH FIBER RETROFIT AND CONVENTIONAL RETROFIT

The cost of retrofitting the three bridges is shown in Table 5 below, for the first year of installation, 1999/2000. The cost for retrofitting per bridge in the second year of retrofit work beyond the initial three bridges in Nova Scotia is also shown below in Table 5, 2000/2001. The reduction in cost is due to multiples of 10 versus 3 and also is due to improved skill at installation. The downstream cost beyond the second year, when retrofitting multiples of 10 bridges, is lower than the second year beyond the first three retrofits and is shown to be less than $20,000 USD per bridge, 20002+. This assumes no testing and a conventional commercial operation for installation. Access costs are not shown in the cost table below.

Table 5. Costs of Retrofit of NSTPW timber bridges.

	1999/2000	2000/2001	2002+
LABOUR			
PLANING	$1,600	$1,600	$1,600
INSTALLATION & TRAFFIC CONTROL	11,800	7,000	7,000
	$13,400	$8,600	$8,600
MATERIAL			
TENSION LAMINATIONS	$6,500	$6,500	$6,500
EPOXY	2,400	2,400	2,400
PRESSURE TREATING	350	350	350
LAG SCREWS	1,200	1,200	1,200
	$10,450	$10,450	$10,450
TESTING			
CAD/CAM CENTER	$3,540	$3,540	$0
TRAFFIC CONTROL & STAGING	1,500	1,500	0
	$5,040	$5,040	$0
ENGINEERING (WSTI)			
INITIAL ASSESSMENT	$5,000	$0	$0
DESIGN	5,760	5,760	0
CONSTRUCTION REVIEW	5,000	0	0
	$15,760	$5,760	$0
TOTAL	$44,650	$29,850	$19,050

A typical retrofit of a timber bridge without the use of high-strength fiber retrofits involves the following process;

Remove asphalt

Remove deck

Add girders

Re-deck

Pave

Traffic control

The cost of a typical retrofit of a timber bridge is $90,000 to $110,000 USD. This means that the savings that is realized utilizing a FiRP® Retrofit is 85% or in other words a typical retrofit is 5 times more expensive. In addition the bridge is out of service totally while the conventional retrofit is underway. With a FiRP® Retrofit the bridge is always in service. In summary the following benefits are achieved utilizing a FiRP® High Strength fiber retrofit;

Simple installation procedure, 80% less time.

Utilize highways department work forces

1 crew chief versus 3 crew chiefs

crewmen (minimum)

Lower cost

Reduced traffic control

Technology transfer

CONCLUSION FOR RETROFIT METHODS FOR BRIDGES UTILIZING HIGH STRENGTH FIBERS

The NSTPW timber bridge retrofit project was a tremendous success and provided detailed information on the correct procedure to utilize to inspect, identify in-situ structural properties, design a retrofit utilizing high-strength fiber, install the retrofit and properly treat the finished bridges. The bridges have been operating with increased loads with no further road surface cracking. The cost savings gained by utilizing FiRP® High-Strength Fiber Reinforcements allows more bridges to be upgraded for the same money. Nearly 5 times the number of bridge upgrades can be completed with the same maintenance and repair budget in most cases. Clearly high-strength fiber retrofits coupled with state-of-the-art investigation and analysis processes is extremely beneficial to highways departments around the world.

ADVANCED LONG TERM REMOTE MONITORING METHODS A CASE STUDY OF THE LIGHTHOUSE BRIDGE IN WASHINGTON STATE.

The following section is a reworked paper written by the author from another conference.

A two-span 160 ft. long HS-25 reinforced glulam highway bridge called the “Lighthouse Bridge” was built near Clallam Bay, Washington. Long term monitoring devices were installed in the bridge and continuous monitoring is ongoing. The long term performance of fiber reinforced plastic (FRP) reinforced glulam has been shown to be better than unreinforced glulam. Recent studies have shown that Douglas-Fir glulam, reinforced with as little as 0.3% by cross section aramid reinforced plastic (ARP), has a 95% reduction in additional deflection beyond initial loading over a six month period. This has a noted affect on the performance of bridge girders. This paper presents the benefits of using FRP reinforced glulam in this project as well as the results of long term monitoring to date.

Advantages of Reinforced Glued Laminated Timber bridges The first heavy loading vehicular bridge (HS25-44) was constructed in August 1995 - the Lighthouse Bridge. The reinforced girders for this bridge were instrumented with embedded strain gauges to allow continuous monitoring of the girders over a long-term period.

Significant cost reductions in the bridge girder cost have been achieved since the FRP reinforcement greatly increases the moment capacity of the beam. The width of a reinforced beam can be reduced one or more standard widths while maintaining approximately the same depth as an unreinforced beam. The reduction results in less volume of lumber used. Reinforced glulams can be made of lower grade laminations. These lower grades of lumber are more available than the higher grades and can be obtained from smaller trees.

Reinforced glulam beams have significantly lower variability than conventional glulams. The strength of conventional glulams is affected by the natural growth characteristics of the timber. Strength reducing defects such as knots, slope of grain variations, coarse grain timber and placement of finger joints affect the ultimate strength of the beam. The reinforced glulams are not as affected by these natural defects as unreinforced glulams.

Use of FRP reinforcement in the tension zone increases the ductility of the beam significantly. The FRP allows larger yields train values in the wood and in the tension zone by allowing higher tensile stresses at failure at localized anomalies e.g. finger joints or slope of grain (Tingley and Gai 1998).

The Lighthouse Bridge

The Lighthouse Bridge (Figure 45) was constructed on Frontier Street over the Clallam River in Clallam Bay, Washington. The bridge is located approximately 400 m from the mouth of the Clallam River, where the Clallam River meets the Strait of Juan de Fuca. The owner of the bridge is the government of Clallam County, WA.

Figure 45. Lighthouse Bridge, Clallam Bay, Washington

The bridge consists of two simple spans, each 24.8 m, for a total span of 49.6 m with an HS-25 load rating. Six Douglas-fir girders (L2 grade), spaced 1.6 m center-to-center, were used for each span. The exterior girders were 222 by 1459 mm with 15.2 mm of aramid-reinforced plastic (ARP) as a tensile reinforcement placed between the outer two tension zone laminations. The reinforcement ratio, the percentage of reinforcement to wood cross section, is 1.04%. The interior girders were 171 by 1459 mm reinforced with 12.5 mm of carbon/aramid-reinforced plastic (CARP). The reinforcement ratio is 0.86%. Figure 46 shows interior and exterior girders. The bridge deck was a conventional transverse glulam deck composed of 52 panels. Glulam was also used for deck stiffeners and curbs. All of the bridge components were pressure treated with pentachlorophenol.

Figure 46. Reinforced glulam girders

The estimated unreinforced glulam beam size is 273 mm x 1524 mm for the interior and exterior girders for the same bridge. Thus, the reinforced bridge used only 67% of the wood fiber that a conventional glulam design would have required. The cost savings, which included reduced wood fiber and treatment costs, was nearly 24% as compared to conventional glulam (Douglas-fir, 24F-V4). This did not include additional savings due to pier size reductions caused by reduced dead weight loads. FiRP® Reinforced girders also provide an estimated 10% cost savings in concrete piers and foundations as less the reinforced glulams weight less than 10% of the weight of the originally proposed precast concrete girders. Another significant advantage of FiRP® Reinforced glulam beams is the reduced variation in the modulus of elasticity and the strength. The reduced variation permits significantly increased design properties and a closer approximation of the deflections.

Procedure and method

The purpose of this study was to investigate the short term and long term performance of FiRP® Reinforced glulam bridge girders subjected to in-service conditions such as dead loads, live loads, and environmental factors such as moisture content and temperature fluctuations. Both electronic data acquisition of strain gauges and direct surveying using precision optical levels are used to assess the response of the Lighthouse bridge to applied loads and environmental conditions.

Strain Gauge Data Acquisition

Three of the main girders, two exterior and one interior, were fitted with internal strain gauges internally on the reinforcement and on the wood. Figure 47 shows the location of the gauged girders in the bridge. The strain gauges are located at the center span point and at 3.05 m and 6.10 m toward each end from the centerline. Figure 48 shows the internal strain gauge layout in the girder.

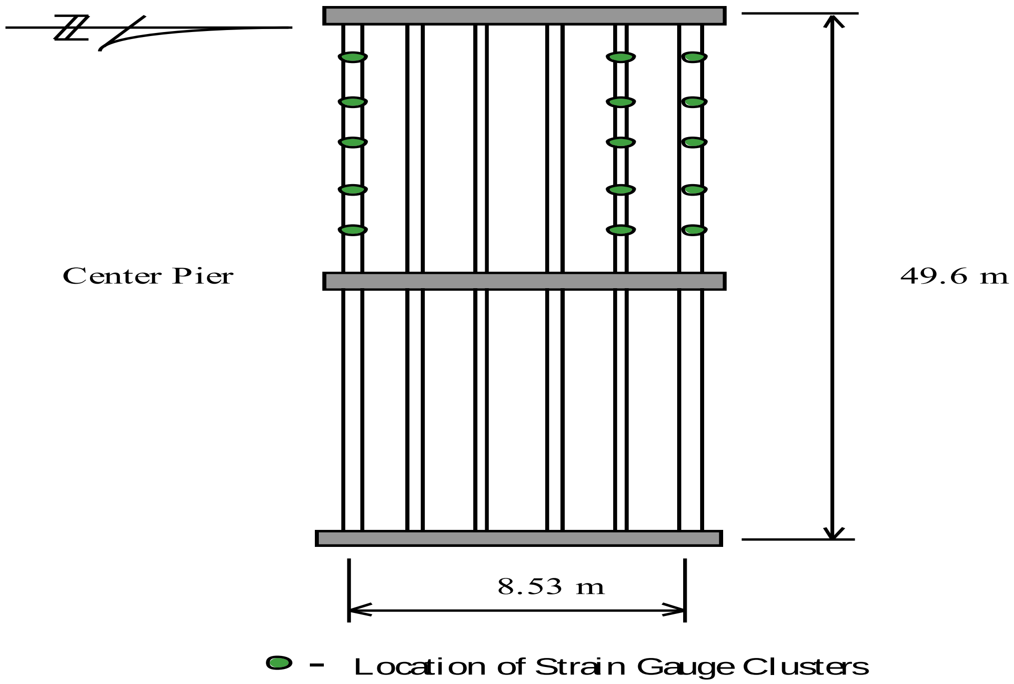


Figure. 47 Locations of gauged girders in the Lighthouse Bridge

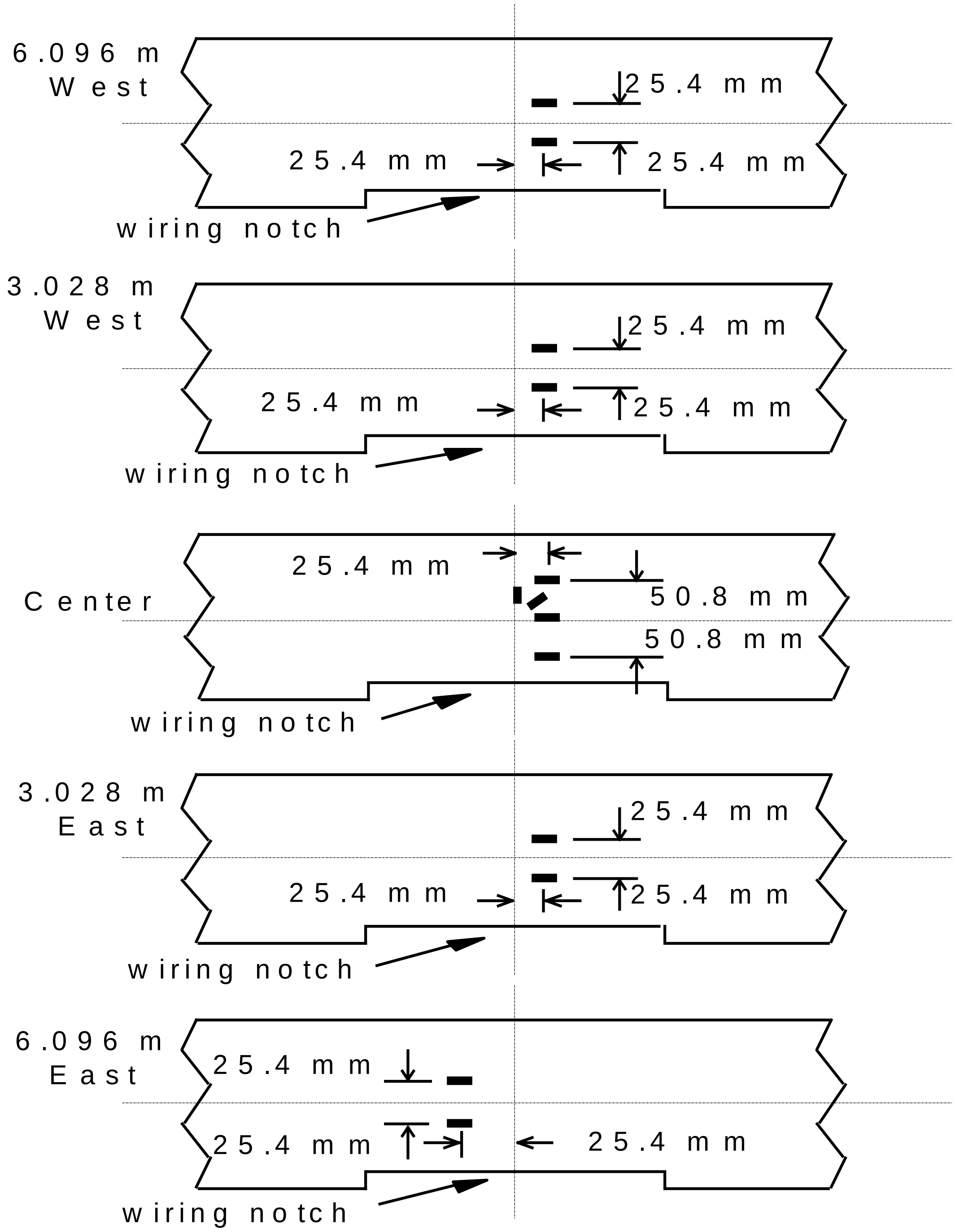


Figure 4-Plan view of strain gauge layout in the girders.

Figure. 48 Internal Strain Gage Layout

General purpose strain gauges (type EA-06-10CBE-120) with a 25.4 mm effective gauge length and 120.0 ± 0.15% ohms resistance at 24° C are used for the long term monitoring. This type of strain gauge has a working temperature range from -75° C to 175° C and a maximum of 5% strain.

The wires from each gauge are connected to the monitoring apparatus in a central instrument panel located near the centerline bridge pier. For the first portion of this study, battery power was provided in a separate enclosure to power the instrumentation. Strain gauge data is collected at 108 min. intervals and is routed to a 32 channel multiplexer for each girder then stored in the memory of a Campbell data logger.

Survey of the Northwest Exterior Girder

Deflection of exterior girder was measured by direct surveying. Locations on the bottom of the northwest exterior girder corresponding to the internal strain gauge locations are marked to allow consistent level measurements to be made. Benchmarks are located at the supports and the center pier. The initial survey took place on September 21, 1995 and the elevation of the deflected shape of the northwest exterior girder was established. During this time, only the dead load of the girder and the bridge deck contributed to the deflection. Upon returning to the bridge on January 26, 1996 (after the construction was complete), another survey was conducted to measure the deflected shape of the northwest exterior girder.

Results

Strain

Strain gauge data down loaded from the Campbell data logger was processed in Microsoft Excel 5.0 (1994). The voltage output from the gauges was converted to micro strain through a calibration constant of 1904 $^{\text{micro strain}}/_{\text{Volt}}$. The strain was subsequently zeroed-out based on the first recorded strain reading.

Figure 49 displays the axial strain on the wood of the northwest exterior girder at the centerline, 3.05 m from the centerline and 6.10 m from the centerline. The axial strain in the northwest exterior reinforced glulam beam in the extreme tension zone shows an initial increase due to application of the asphalt concrete wearing surface. However, after the initial strain increase, no significant creep had been observed.

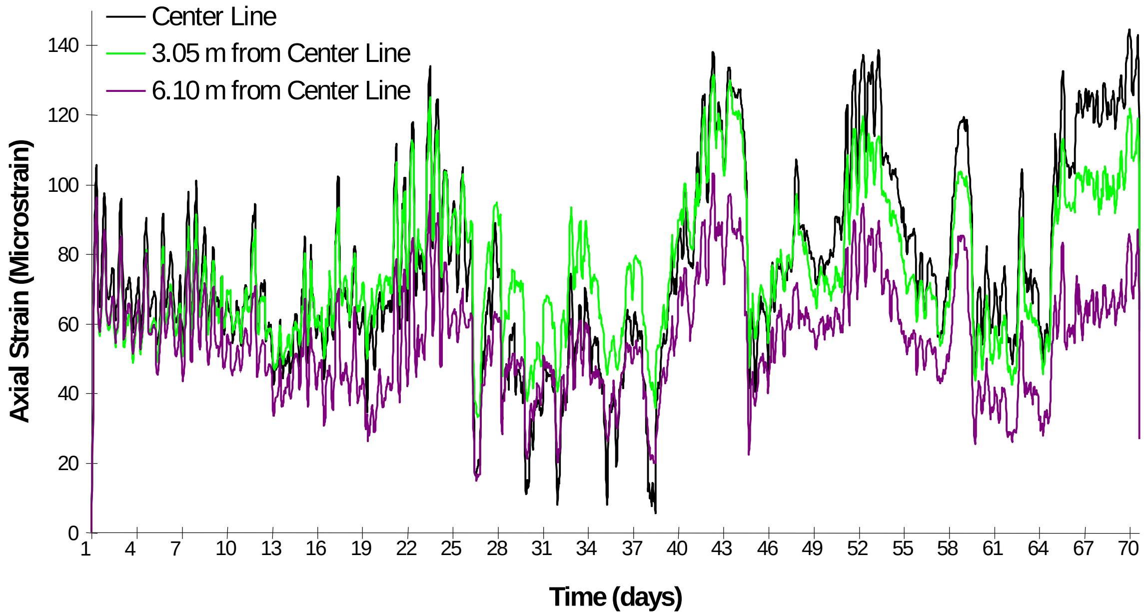


Figure. 49 Response of strain gauges located at the centerline of each of the three gauged girders.

A Fourier analysis was conducted to transform the time-domain to the frequency-domain using a fast Fourier transform algorithm to check for the influence of cyclic events. Figure 50 displays the Fourier amplitude spectrum of a strain gauge mounted on the wood between the bottom lamination and the reinforcement on the northwest exterior girder. Perhaps the most important result from the Fourier analysis is the presence of a spike at the one-per-day frequency indicating that there is significant cyclic behavior at one-day intervals (the corresponding harmonics can be seen at higher frequencies). All of the strain gauges experienced the cyclic behavior. The most reasonable explanation of this response is day-to-day temperature fluctuations.

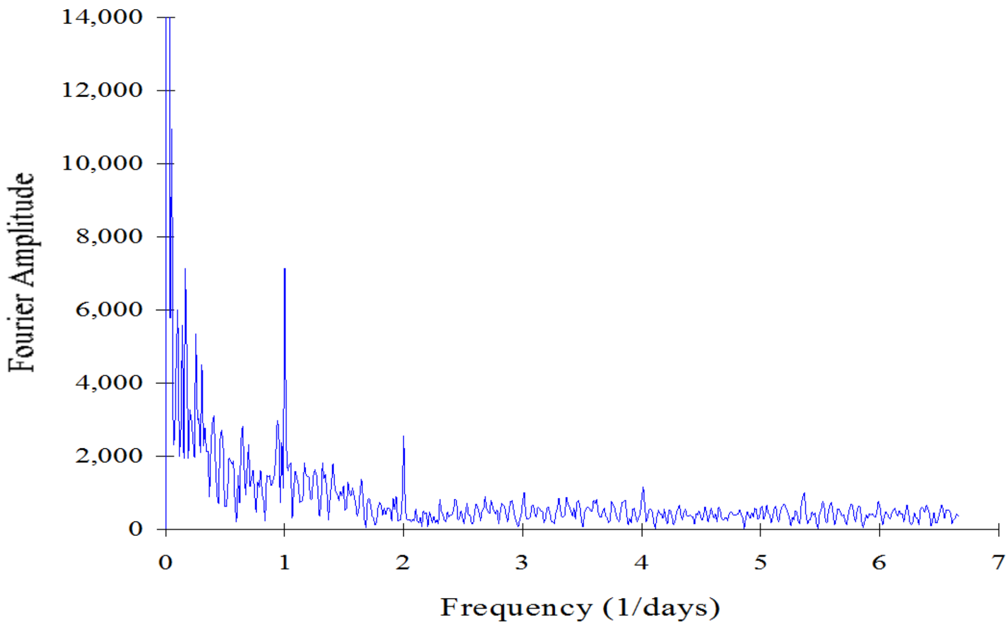


Figure 50 Fourier amplitude spectrum of the strain gauge response.

Generally, the strain gauges placed on the two exterior girders exhibited a larger degree of sensitivity to temperature than the interior girder due to the more direct exposure to the environment.

Deflection

Using a precision automatic level, the elevation of the bottom of the northwest exterior girder was measured in reference to a brass benchmark located on the north side of the center pier (assumed elevation of 6.096 m). Figure 51 displays the results of the two surveys. A difference in the centerline deflection of 1.016 mm was observed. It was considered as a result of the increased dead load from the asphalt wearing surface which had been recorded on the strain gauge data.

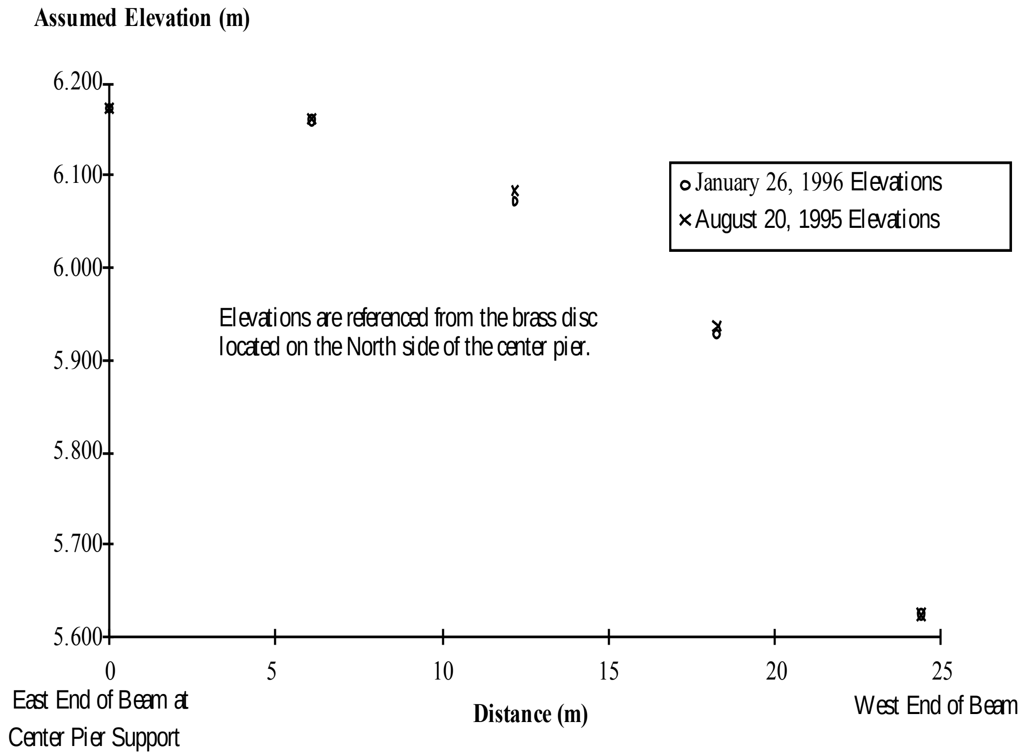


Figure. 51 Deflection of northwestern exterior girder measured by surveying

This confirms the results from a prior study of load duration effects in the glulam beams conducted at WSTI (Tingley, 1995). The creep in a reinforced glulam versus unreinforced glulam of the same size and design load is reduced by 80% in the first 24 hours. The reduction of deflection is 95% compared to an unreinforced beam under design load.

Conclusions

Overall performance of Lighthouse Bridge has been found satisfactory. The study has shown that Douglas-Fir glulam, reinforced with as little as 0.3% by cross section aramid reinforced plastic (ARP), have a 95% reduction in additional deflection beyond initial loading over a six month period. Use of FRP reinforcement in the tension zone increases the ductility of the beam significantly.

The data collected to-date shows no appreciable creep in the reinforced girders. The slight increase in strain can be attributed to temperature effects. The strain gauge output from the data logger is heavily influenced by environmental conditions. It is apparent that modifications need to be implemented to the strain gauge acquisition system to account for the influence of environmental conditions.

Significant cost reductions in the beams have been achieved since the FRP reinforcement greatly increases the moment capacity of the beam. The reduction results in less volume and lower grade laminations used. Reinforced glulam beams have significantly lower variability than conventional glulams.

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Washington State Highway Department.

Appendix G. Paper written by Tingley and Richards presented at the Australian Small Bridge Conference July 2009

INVESTIGATION OF AUSTRALIAN SHORT AND MEDIUM SPAN TIMBER BRIDGES

Dan Tingley Ph.D., P.Eng.

Wood Research & Development Ltd.

Corvallis, OR, USA

www.woodrandd.com

dant@peak.org

&

Stephen Richards

Manager Asset Services

Mitchell Shire

Broadford, VIC, Australia, 3658

StephenR@mitchellshire.vic.gov.au

KEYWORDS: Timber Bridges, Restoration, High-Strength-Fiber Reinforcements, Preservation Techniques, Non-Destructive Testing, Round log girders, Heritage Timber Bridges

ABSTRACT

In Australia, inspection of old timber bridges, particularly round log heritage bridges, is being completed by local highways departments, shire and municipality engineers utilizing advanced techniques. Work is underway to complete advanced levels of non destructive testing utilizing such methods as stress wave analysis. New techniques are being developed to utilize high strength fibers like carbon, aramid and glass to reinforce round log girders in-situ. This paper discusses the findings and methods employed in this work. The paper provides examples of timber bridges, some built in the 1920's and earlier, with round hardwood log girders and piles, and advanced non destructive testing methods and rehabilitation procedures being utilized to improve the performance of same. These procedures include fumigation techniques utilizing non toxic natural basalt fumigants and advanced compression-based high strength fiber reinforcement design techniques. Examples provide methods utilized to complete designs and subsequent retrofitting techniques that lead to improvements in the in-situ design load capacity by as much as 400%. The rehabilitation methods provide for live load and

total load upgrades and increase the bridge performance to a 44T or higher as needed from such low values as 8T .

INTRODUCTION

Australian governments at all levels are struggling to replace and refurbish an ever aging population of timber bridges. This is particularly true of shires where there are vast numbers of aging secondary highway bridges that are either in a seriously degraded condition or significantly under capacity for the highway requirements that they service. In most cases primary or arterial wood highway bridges have been replaced but this is not the case in many shires where budgets are not adequate to replace all the timber bridges. In such cases asset managers have to determine how to best utilize their limited resources to firstly properly categorize their timber bridges in terms of needed repairs and secondly, determine how to upgrade certain bridges to meet increased demand or load requirements without simply replacing them with a new bridge. The money required to replace all the timber bridges is simply not available. Australia, like Canada has a relatively small population compared to its land mass. The civil infrastructure needs are great compared to the population tax base and governments at all levels struggle to meet the needs of the urban sprawl as well as continue to maintain an ever increasing number of bridges. At the dawn of the age of vehicular traffic many bridges were built to handle the traffic requirements of the day. Today's load requirements are higher with 44T being the conventional benchmark and ratings as high as 60T for raw resource truck traffic being common. Thus, many old bridges are in need of upgrading.

This job of upgrading is made more complex by the fact that many older bridges were constructed with wood and, in many cases, with round log girders. These wood elements are very susceptible to decay, insect infestation and weathering. Frequently, the timber bridges that first develop problems are the bridges around smaller communities that are growing and sprawling outwards engulfing bridges that normally handled collector road ratings. These bridges are rated at lower loading to start with because of the loading requirements when they were built e.g. 18T. They can be located on arterial highways that provide truck traffic access to enable the community to function. They receive regular loadings in excess of their ratings and this further accelerates their degradation. When the shire or municipality engineers begin to deal with the bridge they interact with state highway engineers and usually contract consulting engineers are brought in to inspect the bridges. Since most engineers do not receive a significant amount of training in timber engineering and even fewer receive training in non destructive techniques for testing wood bridges and identification methods for determining in-situ design properties these bridges often get flagged for replacement and/or re-plated with a lower capacity as well as reduced speeds.

This interaction sequence is happening all over Australia and local governments are banding together to formulate effective means of dealing with degraded wood bridges that involves non destructive techniques, in-situ design property assessment and restoration and reinforcement techniques. The process of dealing with the bridges needs to be comprehensive and involve all three of the above elements or it is ineffective. Having a retrofit technique is useless unless the engineers know how to properly design with it and understand accurately the in-situ properties in comparison to the desired loading requirements. In addition to the above considerations another important aspect of this process is the national and local heritage registries. Often the heritage foundations and associations have significant governmental authority over key arterial bridges that constitute the only means of access for some communities. This authority can trump the local highways engineering departments such that even if the money is available for complete replacement they are unable to proceed due to heritage requirements which require that the historical bridge in question is restored and utilized for current traffic requirements. This factor means that the three step process discussed above becomes mandatory. This paper discusses a program now underway in Mitchell Shire in Victoria, Australia to properly investigate and assess timber bridge in-situ capacity and requirements for refurbishment to properly meet the expectations of the community that utilize the bridge. Seven bridges are featured in this paper ranging from small short spans to longer intermediate span wood pier, steel girder, timber bearer and deck highway bridges. See Figures 1-6 for photographs of the six bridges currently being investigated and refurbished. See Figure 7 for photograph of another bridge inspected two years ago in Victoria Australia. Barwon Heads Bridge was a bridge listed as a Heritage site that was over 90 years old constructed with Turpentine log girders.

Figure 1. Photographs of the Cameron's Creek Bridge.

Figure 2. Photographs of the Costello Road Bridge.

Figure 3. Photographs of the Bruce's Creek Bridge.

Figure 4. Photographs of the Pyalong Bridge.

Figure 5. Photographs of the Smith's Creek Bridge.

Figure 6. Photographs of the Seymour Bridge.

Figure 7 Photograph of the Barwon Heads bridge with Turpentine log girders.

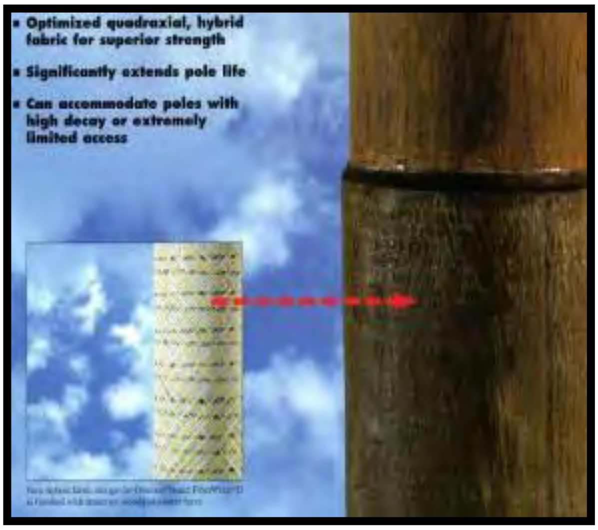
The Barwon Head bridge has gradually deteriorated over time. In response to this deterioration various rehabilitation techniques and strengthening strategies have been utilized. See Figure 8 which contains a photograph of steel I-beams that have been installed along side of the log longitudinal log girders. These I-beams have also deteriorated in certain places.

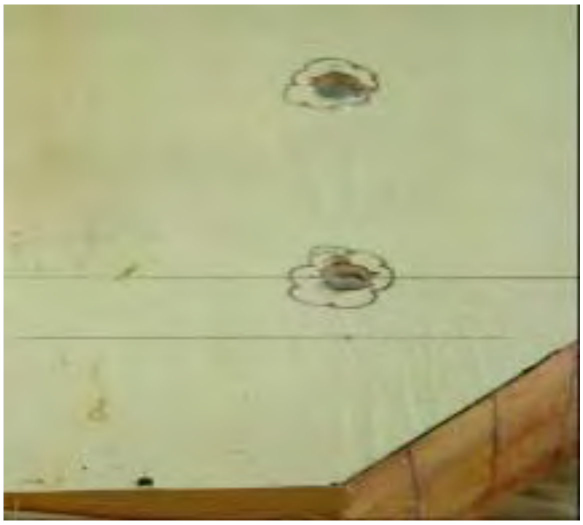
Figure 8 Photograph of Steel I-Beams Utilized to Strengthen the Barwon Heads Bridge.

The deterioration is found in a broad cross section of the structural elements. The causes of this deterioration have been basically due to environmental factors. In some cases the deterioration could have been prevented and in other cases it would have at been reduced by the utilization of better construction and design techniques. See Figure 9 which contains a photograph of a downspout for a drain that is dumping on a structural member and subsequent accelerated decay and degradation. Note in Figure 8 the steel I-beams have been placed at a very critical shear location in the pile cap transverse girder support beam; a distance 2.5 to 3 “d” from the reaction where “d” is the depth of the beam. The applied shear stress reaches a maximum at the neutral axis at this point in the beam. The placement of the sister I-beam girders at this location has increased the loss of strength in the pile caps due to the decay and degradation of the bridge.

Figure 9. Photograph of Improperly Placed Downspout Causing Accelerated Decay in Structural Members below.

The Barwon Heads Bridge (BHB) could have been restored to its original condition and upgraded to higher load limits utilizing current non destructive testing techniques to isolate the external and internal degraded areas. Subsequently the in situ properties including the strength and stiffness of each of the members and member connections could have been ascertained. With this information designs using advanced rehabilitation techniques such as high strength fiber technology can be employed to restore the members in place. Internal non toxic agents could be utilized to prevent further decay and degradation. Better drainage, protection, connector and member placement strategies could be employed to improve the bridge performance as well. Figure 10 contains a series of photographs of fumigation, and rehabilitation of bridge and building structural components restored to service utilizing high strength fiber retrofit technology.





10a. High Strength Fiber Pile Reinforcements

10b. Non Toxic Fumigants Utilized To Preserve Timbers

10c. Amputation and End Sealing
Large Dimension Timber
Restored with High Strength Fibers
and Fumigants

10d. Similar Age Bridge In
Similar Condition and Location
Restored with High Strength Fibers

10e. Large Dimension Timbers In Two Older Wood Bridges Reinforced
With High Strength Fibers

A plan was circulating throughout various governmental and quasi-government groups that involved the construction of a sister bridge and the rehabilitation of half of the existing timber bridge at Barwon Heads. The sound timbers in the half were to be taken down and recovered and utilized to replace the timbers in the half to stay in place. See Figure 11 for a cross sectional drawing of the intended old and new bridge configuration.

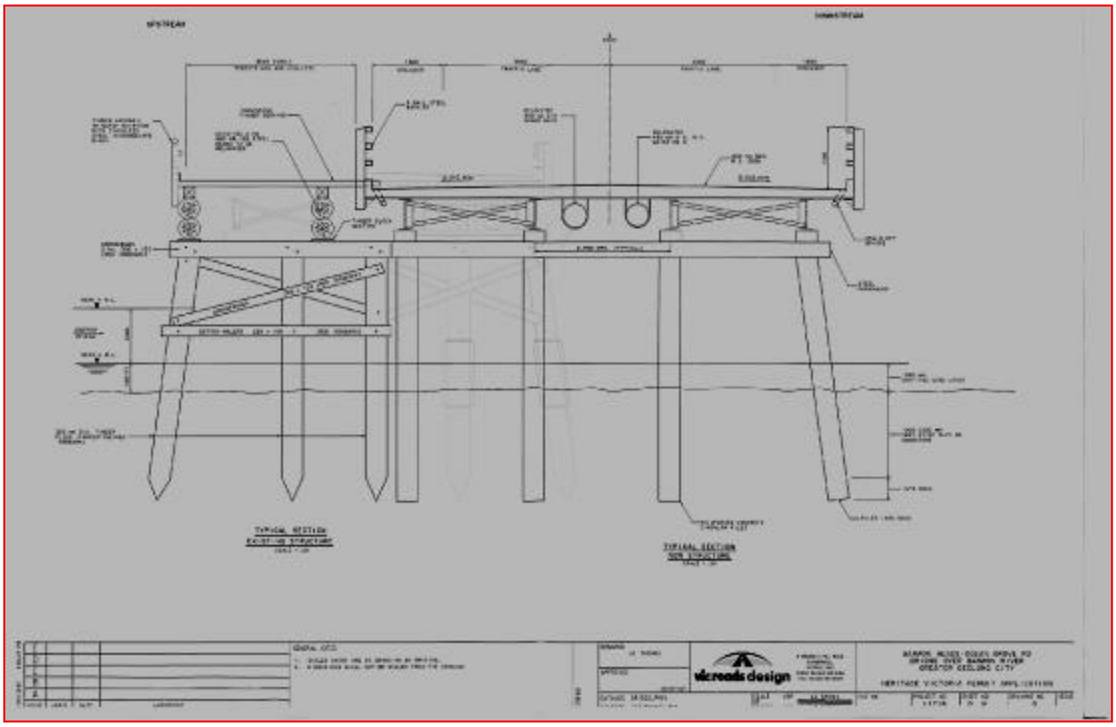


Figure 11. Cross Sectional Drawing of the Intended Old and New Barwon Heads Bridge Configuration as Presented by State of Victoria, Australia, Bridge Engineering Staff (Victoria Bridge Engineering Department).

Unfortunately it has now been determined to tear down the grand old BHB and to replace it with a new two lane concrete bridge as shown in Figure 11. A major part of the decision making process involved a lack of understanding of the in-situ properties of the existing structure. The first step in that process is the proper assessment of the existing structure. All of the timbers including the piles have to be assessed and their in-situ design properties established. The subsequent phases of this work involve the design of retrofits, preservative techniques and system connection strategies to restore the half of the bridge to remain. The bridges shown in Figures 1 to 6 above located in Mitchell Shire are currently being assessed utilizing non destructive techniques to determine the best way to properly upgrade and refurbish them.

The Mitchell shire is a typical shire in Victoria where budget monies are always tight to maintain and upgrade its various bridges. Mitchell shire wants to stretch its bridge replacement and maintenance money by utilizing advanced inspection techniques and retrofit methods.

MITCHELL SHIRE

Mitchell Shire is a rural municipality with an urban focus. It lies just beyond the metropolitan fringe, due north, of Melbourne in Victoria. The Goulburn River, Tallarook Ranges, Mt. Disappointment State Forest, and other diverse and magnificent scenery abound throughout Mitchell Shire. In addition to Mitchell Shire's natural attractions, tourism incentives include railway heritage sites and museums, Puckapunyal Military Tank Museum, award winning wineries, festivals, art galleries, explorers' history sites and horse racing.

Mitchell Shire has a residential population approaching 35,000. There are four major towns with populations ranging from 3000 to 8000 people. Each of these towns has its own business community and support services. In addition to these towns, there are also six communities with populations ranging from 300 to 1500 persons. Council has proposed a draft 2009/10 budget of $40.3 million to service all of these communities. The Capital Works Program component of that budget is $8.48 million. This comprises funds specifically earmarked for infrastructure asset renewal programs.

Mitchell Shire Council manages on behalf of its various communities, infrastructure assets as listed in the following table;

Asset Class	Current Value
Roads and Pavements	$148,043,872.00
Bridges	$ 22,547,798.00
Buildings	$ 38,336,591.00
Drainage	$ 21,971,546.00
Parks and Reserves	$ 5,962,846.00
TOTAL INFRASTRUCTURE ASSETS	$236,862,653.00

Note also that the table contains valuations for each asset class under management as reported in Mitchell Shire's 07/08 Annual Report.

MITCHELL SHIRE HIGHWAY INFRASTRUCTURE

Mitchell shire has the following numbers and types of structures in its Bridges Asset Class:

33 Concrete bridges

11 Composite bridges

10 Steel bridges

6 Timber bridges

158 Culverts / floodways.

Since Mitchell Shire was formed in 1994, there has been a philosophy of replacing timber bridges due to their poor condition at the time. Timber bridges have been primarily replaced with concrete bridges, steel composite bridges and reinforced concrete culverts.

Mitchell Shire has implemented a 10 point condition rating score system within its Asset Management Information System (AMIS). Council does not possess a sophisticated Bridge Management System hence its reliance on accurate condition scores in its AMIS. The AMIS has a maintenance management system (MMS) attached to it and any defects that are recorded during routine patrols or at the condition audit are recorded against the bridge asset in the AMIS. The collection of bridge defects allows for the repair works to be programmed by repair type, by crew or by locality.

Once the bridge condition is understood, then the Level Of Service (LOS) that a bridge provides to its users needs to be assessed to determine what is the appropriate capital investment in that structure to provide the required LOS and what is the appropriate maintenance investment in that structure to maintain the provision the required LOS by the bridge structure.

All of Council's asset inspections for defects and asset condition contain a risk assessment process which assesses the impact on the organization should an incident be caused by the defect. This includes an assessment of the likely damage that may be caused as a result of such incident.

Defects are determined to be “above intervention”, “nearing intervention” or “free of defects” and, together with the level of risk that the defect poses for the Council and / or its community, are used to establish priorities for the repair of recorded defects.

As a result of comparing the condition rating of all bridge structures with the renewal intervention level, and reviewing the cost of the defects recorded in the MMS section of the system, an annual renewal plan can be prepared submitted for consideration in conjunction with the preparation of the Council’s Annual budget. In preparation for the introduction of the Federal Government’s CPRS, there is an opportunity to review the LOS necessary to be provided by each of Council’s bridges. Once the LOS is established, an assessment of the cost of the various replacement options can be carried out against a standard specification. The results of such cost analysis can then be compared with an organization's or a community’s ability to fund such necessary works.

When you bring together the ability to rehabilitate and strengthen existing timber bridges, the use of plantation softwoods to manufacture timber members of the required sizes, the ability to carbon sequestrate and reduce an organization's carbon footprint and potentially reduce whole of life costs too; it appears that timber may still be a viable bridge material for use in Australia.

MITCHELL SHIRE ICLEI Data Capture Project

In 2008, Mitchell Shire was one (1) of just ten (10) municipalities nationally to be selected to participate in the International Council for Local Environmental Initiatives (ICLEI) Data Capture project. This is an important project in light of the Federal Government’s impending Carbon Pollution Reduction Scheme (CPRS). Only Cities for Climate Protection (CCP) participants were eligible for consideration to be awarded a place within the project. Mitchell Shire has been a member of Cities for Climate Protection since 2001 and has now achieved CCP+ status by achieving all of its milestones.

Participating is such a project has exposed the project team members to many of the issues that will be confronting Local Governments worldwide in the not too distant future:

Embedded energy in products and materials;

“Scopes” of responsibility for energy consumption in the supply chain;

Proposed legislation;

We begin to understand that the manufacture of concrete and steel are energy intensive processes and hence those materials have a high level of embedded energy within them. With the commencement of the CPRS as early as the second half of 2010, it is expected that the price of concrete and steel products will rise due to the high levels of embedded energy contained.

Conversely plantation timbers are understood to contain a low level of embedded energy. The plantations will also generate substantial benefits in that they will be able to be accounted positively in any organizations assessment of its carbon footprint due to Carbon Sequestration.

Presently, Mitchell Shire Council and associated organizations seem happy to specify timber bridges for pedestrian and shared pathway applications but do not express any interest in recommending a timber bridge for any vehicular application. This is probably the result of the difficulty experienced in obtaining timber of appropriate sizing. In addition, the price of hardwood timbers has risen substantially during the past decade.

New timber technologies present an opportunity for Bridge Managers and Asset Managers to re-evaluate their future maintenance and renewal strategies. If we continue to focus on hardwoods, then nothing will change, however softwoods appear to present an opportunity to prepare the timber to compete with steel and concrete as a viable structural material.

The use of plantation grown softwoods addresses a number of Australian environmental concerns in that the focus can be shifted from “old growth forests”, appropriately treated and designed softwood structures have performed equally or better than similar hardwood ones, the timber plantations are applicable for carbon sequestration and the lower embedded energy within the timber reduces an organization’s carbon footprint.

In preparation for the introduction of the Federal Government’s CPRS, there is an opportunity to review the LOS necessary to be provided by each of Council’s bridges

Once the LOS is established, an assessment of the cost of the various replacement options can be carried out against a standard specification. The results of such cost analysis can then be compared with an organization's or a community’s ability to fund such necessary works.

When the rehabilitation and strengthening of existing timber bridges is combined with the use of plantation softwoods to manufacture timber members of the required sizes, the ability to carbon sequestrate and the reduction of an organization's carbon footprint and potentially reduce whole of life costs it appears that timber may still be a viable bridge material for use in Mitchell shire and in Australia.

BRIDGE INVESTIGATION AND ASSESSMENT

The investigation of the Mitchell Shire bridges shown in Figures 1 to 6 has involved the following advanced techniques and equipment.

1. Stress wave timing

The velocity of a sound wave through wood is proportional to the specific gravity, which is heavily impacted by the presence of decay damage. The use of a precision device to initiate a sound wave at a known location and to subsequently measure the time it takes the wave to travel to another known location is called SWT analysis. The time is measured in microseconds (millionths of a second).

2. Visual inspection

The overall condition of the bridge has been assessed by utilizing wood technology expertise that involves a proper understanding of what constitutes wood degradation and what does not in a visual inspection. Poor construction practices affecting bridge performance and longevity must be isolated. A visual inspection will locate surface decay which is an indicator of interior decay damage.

3). Assay samples

Small core samples have been obtained in locations where internal decay is suspected (connections, wood-to-wood contact areas, supports) and where SWT data has been recovered to properly calibrate the SWT data. The samples have been bagged and will be taken to the laboratory for specific gravity, moisture content, and microscopic evaluation. A quick understanding of though-beam-soundness is achieved when cores undergo initial evaluation. Samples will also be used for calibration of stress wave data.

4). Species/Grade Identification

The species and grade for each beam will be determined.

5). Structural Details

The structural connection and system methods and details have been investigated and recommendations developed for proper modifications to be employed in the retrofit design phase.

6). Pier Investigation

The piers have been investigated with the above techniques along with pulse echo analysis techniques (PET) in the case of the Smiths Bridge. This investigation is limited to the areas below and above the pier concrete jackets where they are present. See Figure 12 for photograph on a typical timber pile with a concrete jacket wrap and the subsequent condition many such piers are found in at old bridges such as the Barwon Heads Bridge.

Figure 12. Photograph of typical timber pile pier with a concrete jacket wrap. Splash zone degradation from borers, decay, weathering and water has occurred in the pier above the jacket (and likely inside the jacket) to a point where the removal of the jacket, fiber wrap and epoxy/chopped-fiber injection under the fiber wrap is required. Also extensive fumigant preservation will be required.

To analyze the wood pier under the concrete jacket x-ray technology is utilized. In most cases however, removal of the concrete jacket is necessary and the preferred solution. Subsequently the restoration of the pier in place utilizing a light weight high strength pre-cured or cured on site high-strength-fiber reinforced polymer wrap is the preferred rehabilitation strategy for older wood piers where the section at the splash zone has been severely reduced and compromised. Epoxy injection of the area between the reinforcement and the pier will then be completed utilizing a chopped fiber-polymer matrix. When the jackets are removed the pier sections under the jackets can be assessed with stress wave time analysis techniques. See later figures for pictures of these wraps and Figure 10a above.

THE REQUIRED ASSESSMENT ANALYSIS OUTPUTS

The investigation team will prepare a full report with a thorough analysis. The staff of the local government engineering departments will be involved in this process providing the required loading details for the bridge and interacting with the retrofit design team to develop the final results. The analysis report will include the following details;

A listing of the in-situ condition of the structural elements including the deck, walkway, log girders and heavy timber pile caps. This should include a numbering scheme in the field on each element, cross referenced to the report listing.

A listing of the required upgrade requirements for the structural elements (deck, walkway, log girders and heavy timber pile caps) in the portion of the bridge to remain in place.

A recommended course of action with regard to how to upgrade the structural elements (deck, walkway, log girders and heavy timber pile caps).

A recommended course of action with regard to the connectors, environmental protection details e.g. proper drainage techniques, preservation methods e.g. fumigants and pastes for the portion of the bridge to remain.

A recommended course of action with regard to the piers regarding replacement and/or rehabilitation strategy for the piers. This will include recommendations as to the removal of concrete jackets, reinforcement of original piers utilizing advanced high strength fibers, epoxy injections and preservatives through the use of fumigants.

A recommended course of action with regard to non wood structural elements (steel I beams and concrete piers) in the portion of the bridge to remain in place.

All of the recommendations must be backed by in-situ test data and substantiation by state-of-the-art investigation techniques. The recommendations must include all necessary information to complete a retrofit design.

RETROFIT DESIGN

The data obtained from the investigation is used to determine appropriate in-situ design values for the existing wood structural members for bending strength, shear strength, and bending stiffness. The design values will be used to determine the level of retrofit based on the current bridge high department design requirements which are now established at various levels from 12 to 44T. The load ratings desired by the highway department will be compared to the allowable capacity as determined during this technical investigation to determine the increase in capacity (if required) for each structural element that is needed in a subsequent retrofit of the portion of the structure that will remain in place.

Bending strength, shear strength, and bending stiffness can be increased using high-strength-fiber reinforcement techniques. Bending strength and stiffness of a wood beam can be increased with the use of thin profile high-strength fiber reinforcements applied to the bottom surface of the beams in the form of reinforcements as shown in Figure 7 and 8 below or partial wraps applied to the bottom of round log girders. Alternatively, narrow width strips (1 inch-2.5 cm) of high-strength reinforced-fiber are utilized on the bottom ¼ of the circumference of the round log girders. These fiber wraps can be (see Figures 5a) applied to the piles as well, as discussed earlier (see Figure 10a above and Figure 13 and 14 below). Shear strength can be increased with the use of composite bar (C-bar) dowels epoxied in place at a calculated spacing near the supports or with the use of reinforcement applied to the side of the beam near the supports. These side reinforcements are applied at calculated depths to allow for moisture content (MC) change induced shrinkage and expansion of the wood. It is never a good idea to encase the wood with reinforcements as is often done with concrete and steel as the wood is unable to move with MC change and the reinforcements will separate from the wood and plane sections will no longer remain plane rendering the reinforcements useless. The high-strength-fiber reinforcement design engineers investigate the most cost effective retrofit method for each situation. Figures 15 and 16 contain photographs of retrofit examples utilizing high strength fiber reinforcements.

Figure 13: Close-up view of interface between reinforced tension lamination and existing wood beam. Chetwynd Secondary School, School District 59, Dawson Creek, B.C.

Figure 14: Reinforced tension lamination installation under timber bridge. McCloud Bridge, near McCloud, California.

Figure 15: High Strength Fiber Wrap of Log Columns with diffusers and Alternative Brown Stain Pigmentation

Figure 16. Round Log Fiber Wrap With Diffusers Installed and Brown Stained Pigmentation

The “residual” allowable bending strength may vary between 10 to 100% of the original bending strength depending of the service life, upkeep, presence of chemical preservative, and other environmental conditions. In addition the required increase in plate capacity of the bridge may require an addition 100% increase, or more, in design capacity from the bridge. A retrofit can be designed to increase the current allowable bending strength up to 500% (varies depending on the amount and location of decay damage). The engineering team must design reinforced tension laminations to increase the allowable bending strength to meet the requirements of the situation. Such a strength increase can be accomplished by adjusting the type and/or amount of reinforcement. Should stiffness or shear capacity be in question, retrofit products can address these situations also.

The retrofit design team must specify nontoxic wood preservatives which can be applied to existing components to halt further decay or insect damage. Application of such preservatives does not require specialized equipment.

LONG TERM MONITORING

Performance of retrofitted wood bridges can be monitored with the use of electronic sensors and survey methods. During installation of the retrofit materials, the engineers can install electronic sensors (strain gauges, tilt meters, position transducers) and a data logger, based on budget availability. These devices will record the performance of the bridge over time. Data can be down loaded by local government representatives and analyzed at predetermined intervals. In the design engineers should setup bench marks for the road authorities to reference for differential leveling to monitor the bridge profile.

CONCLUSION

Clearly the level of understanding at the local level in the methods described above needs to be improved so that local shires in Australia can make more informed decisions on their bridges. More education is needed in the state-of-the-art non-destructive testing, analysis and retrofit design processes. The use of trained experienced professionals to educate and train local authorities in the use of non destructive testing, analysis of in-situ wood structural elements, and design and use of retrofit with high-strength fiber reinforcements is necessary. These methods provide economical methods to obtain a significantly increased service life from wood bridges.

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Appendix H: Discussion on reasons for not using coatings with over 30% solids on heavy dimension timber and ferric degradation around on galvanized connectors.

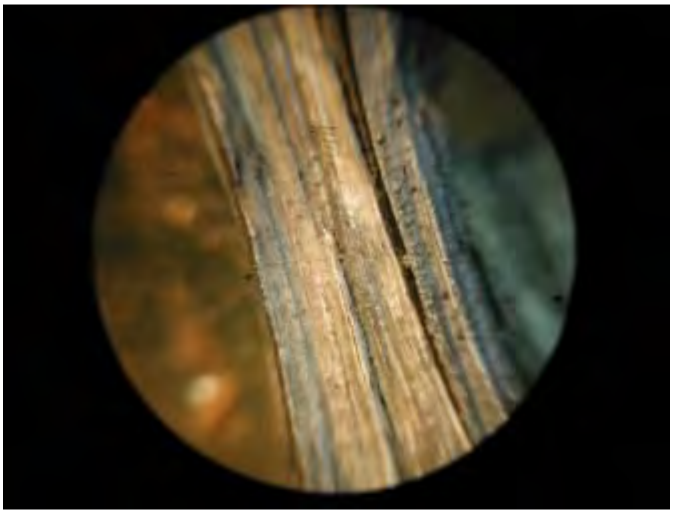
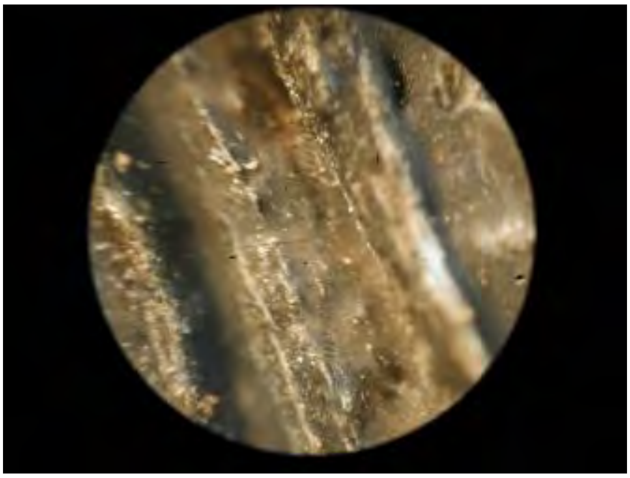
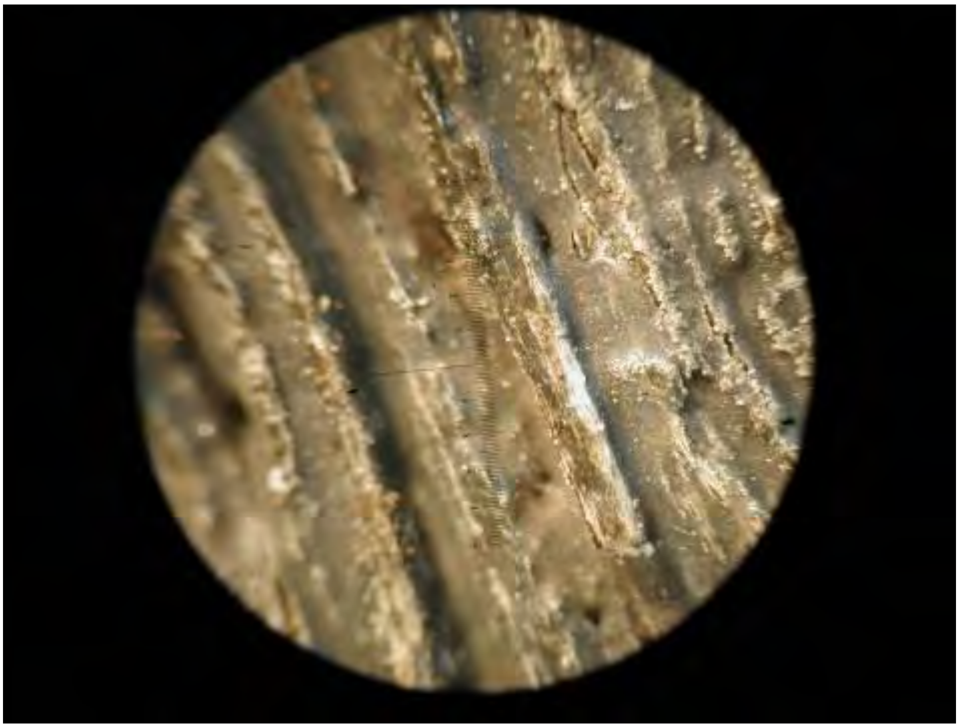


Figure G-1. Surface high magnification (HM) shot of surface showing coating.

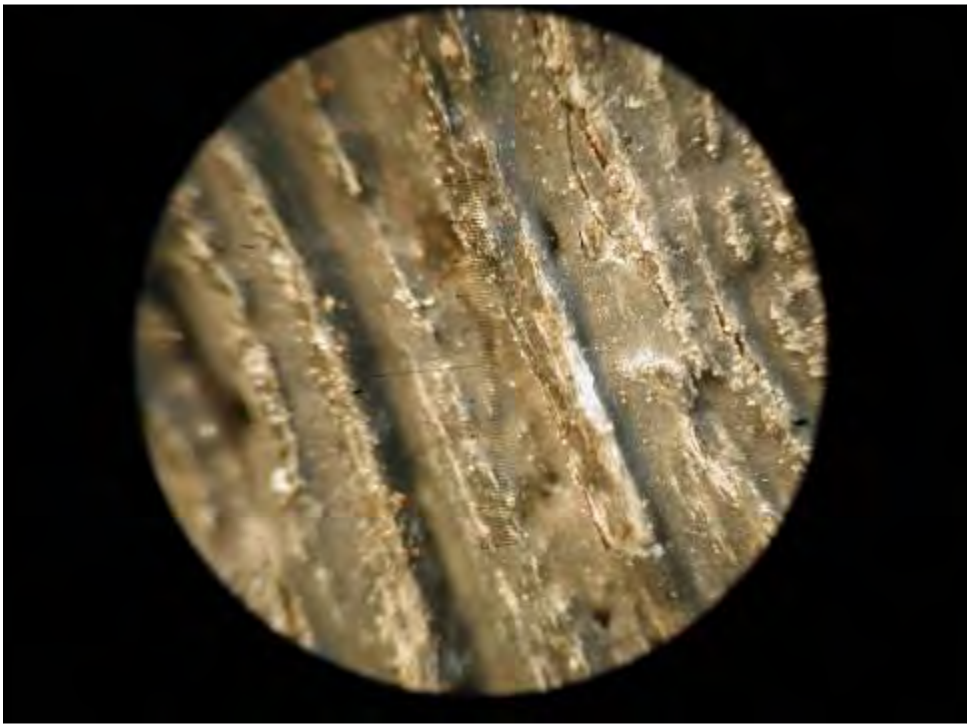


Figure G-2. Surface high magnification (HM) shot of surface showing coating. In these shots note the cracking of the surface that occurs on a brittle coating when the wood shrinks and expands with moisture content change. This allows the moisture in and subsequently does not allow the surface to breath allowing decay.

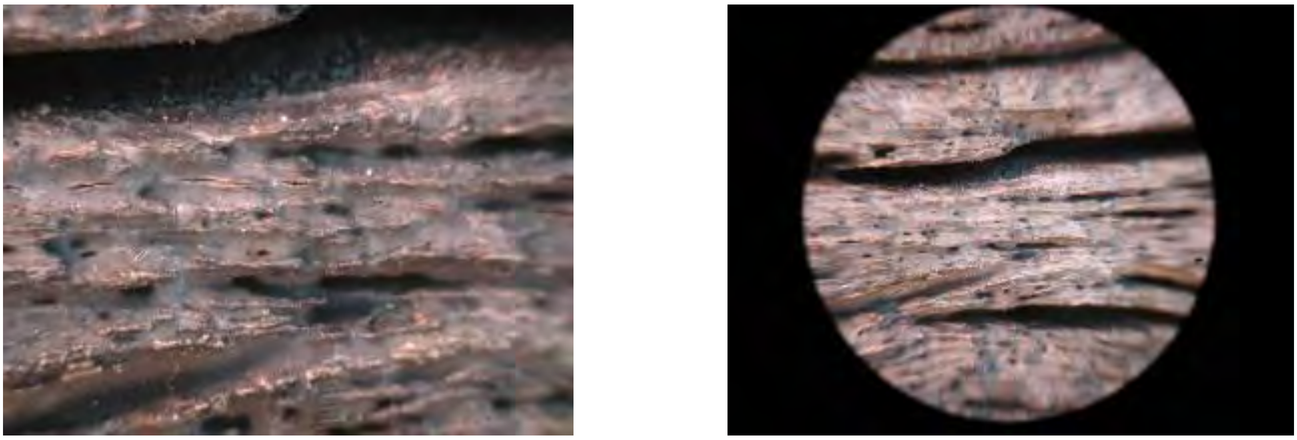


Figure G-3. Surface high magnification (HM) photograph of surface, showing coating. In these photographs note the cracking of the coating on the ridges of deeply fissured surface that occurs on a brittle coating when the wood shrinks and expands with moisture content change.

Figure G-4. Transverse HM photograph showing surface coating extending down into the fissures. In these photographs note how the expansion and contraction of the wood at the fissured points will cause the cracking of the coating. These are the logical entrance points for decay and other degradation causing organisms e.g. moss, insects etc.

Figure G-5. Transverse HM photographs showing surface coating carrier (petroleum based likely) movement into the wood laterally through late wood pores without tyloses and depth wise through rays without tyloses.

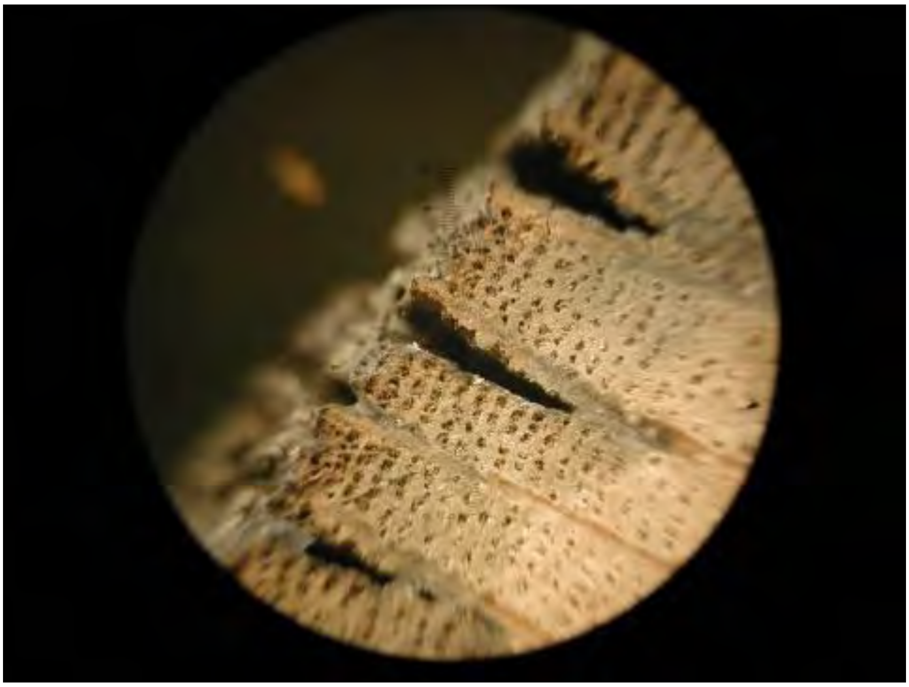
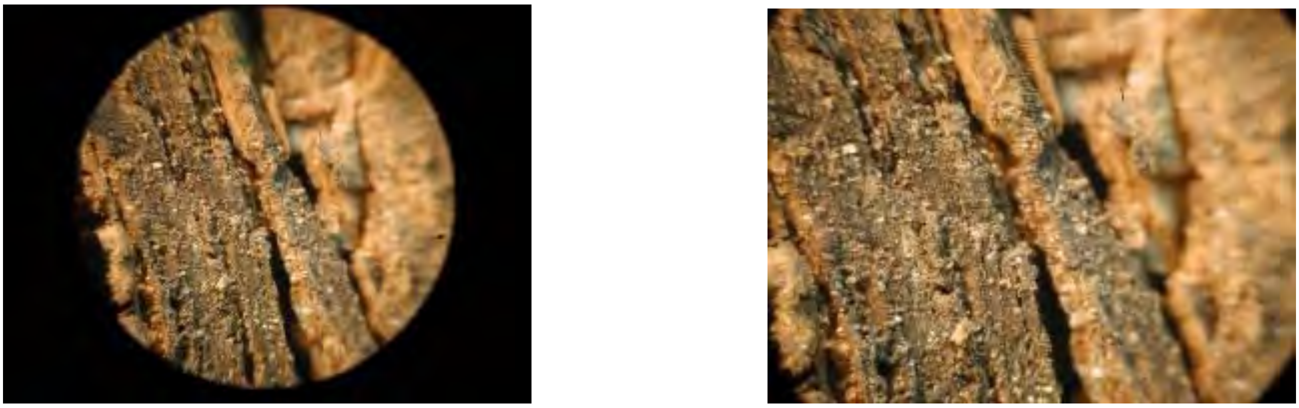


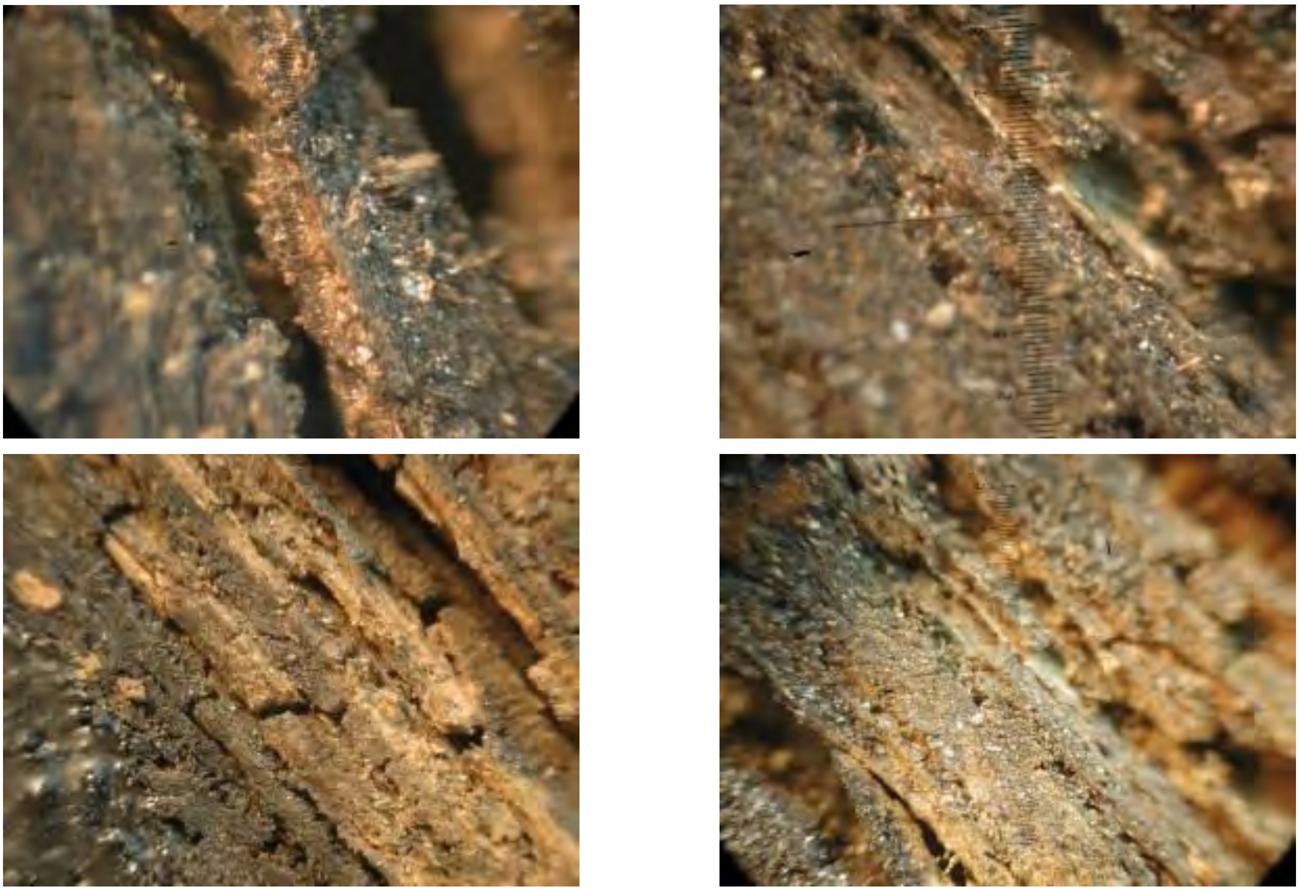
Figure G-6. Transverse HM photographs, showing surface coating and surface cracks/fissures focused in the rays which are often the point where surface cleavage takes place as they are planes of weakness across a tangential face.

(a)

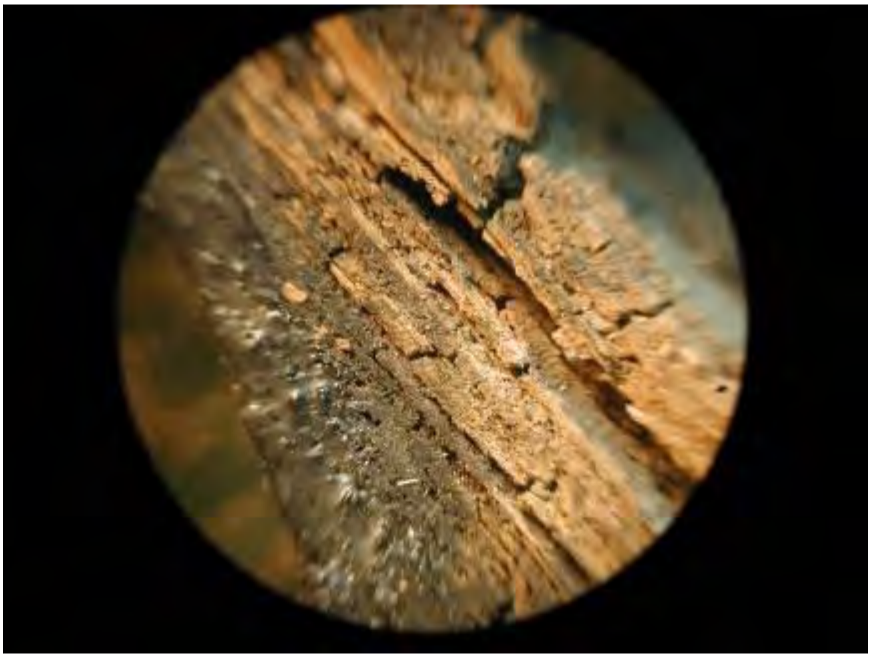
(b)



(c)

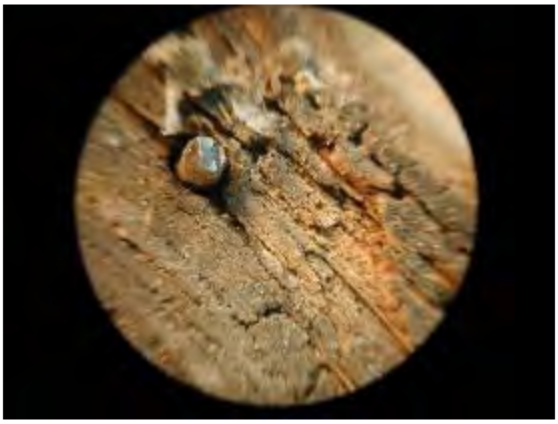


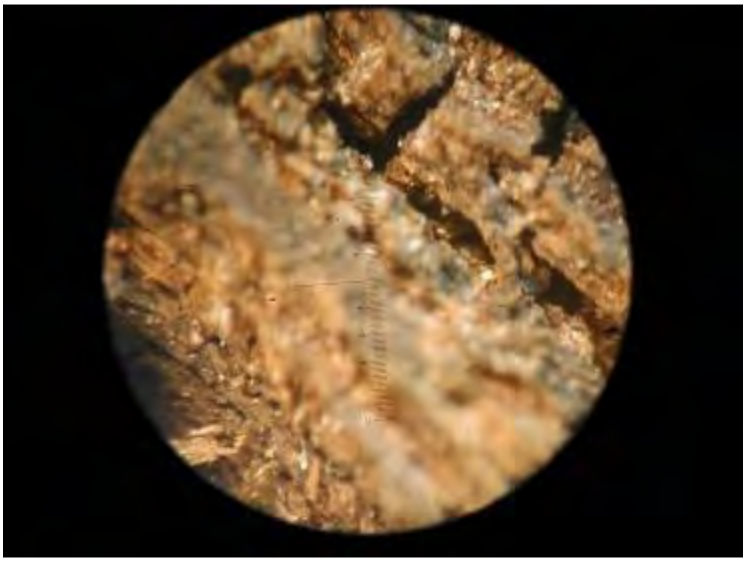
(d)

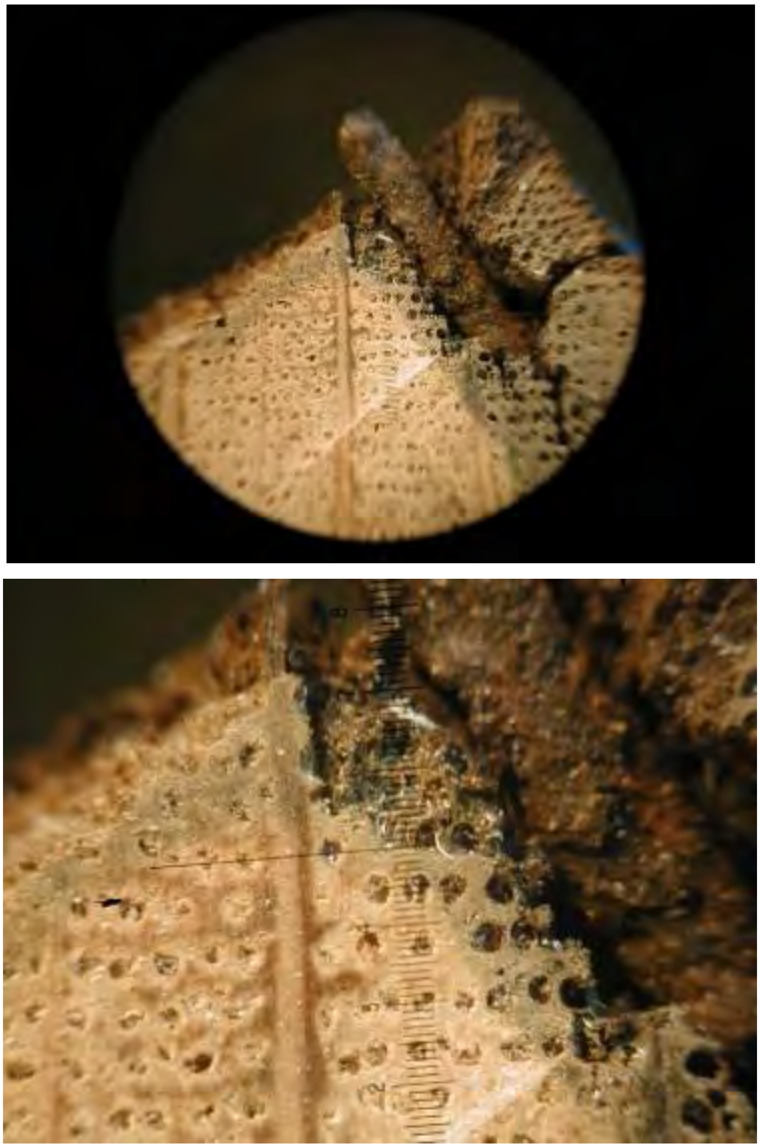


(e)

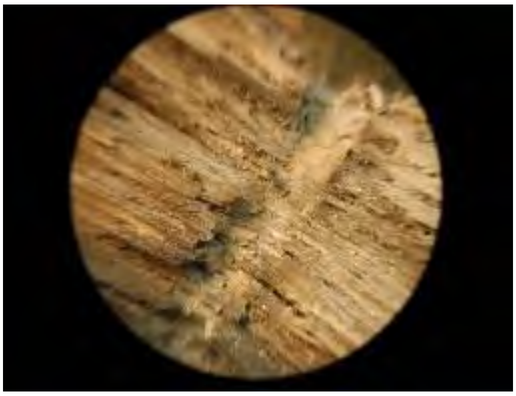
Figure G-7. Photographs showing the interior face of the recovered piece of white oak façade (a.). No coatings on this face. An abundance of decay is evident from periods of high MC. Tangential face shows low magnification (LM) left side and HM on the right side. Note the fruiting bodies in photograph (b.). Note the dramatically reduced SG of the wood in photographs in (c.). The fungal activity has moved further in from the surface and a severely reduced SG decayed zone is left behind. Photographs in (d.) show fungal spores (white). The wood in photograph (d.) is completely decayed and no integrity in the structure is intact. When this happens to zones around fasteners the wood falls out of position due to its own dead weight. Photograph (e.) shows the coating on the surface and non-coated inner surface where the piece narrows decay has moved from behind to completely decay the congruent faces.

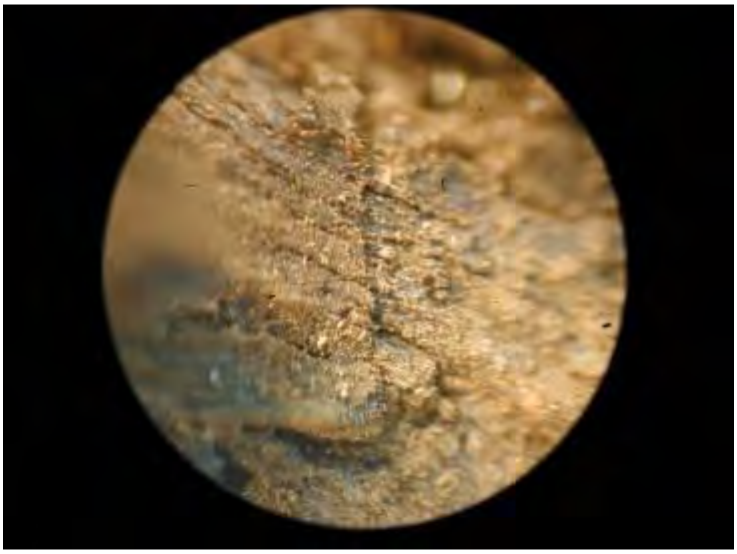





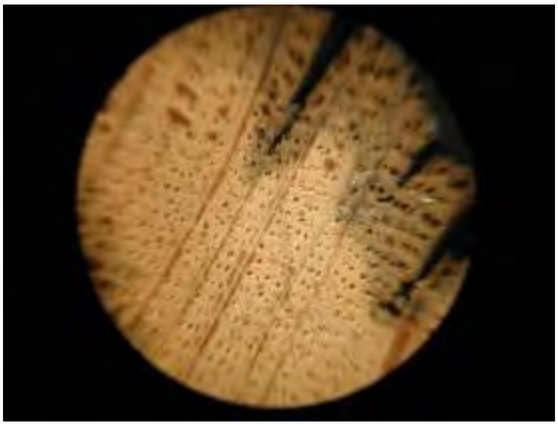
(a)

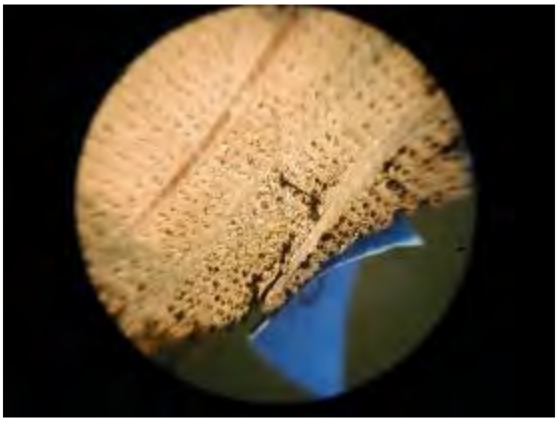
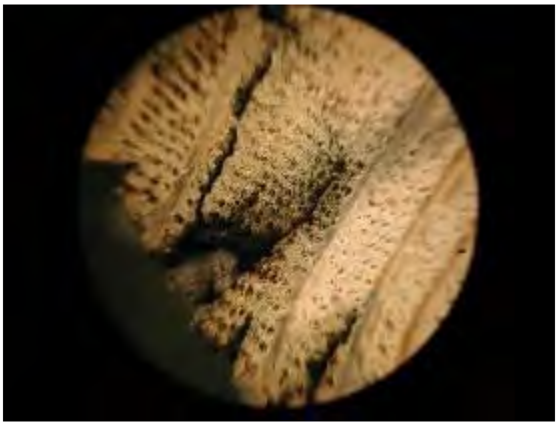



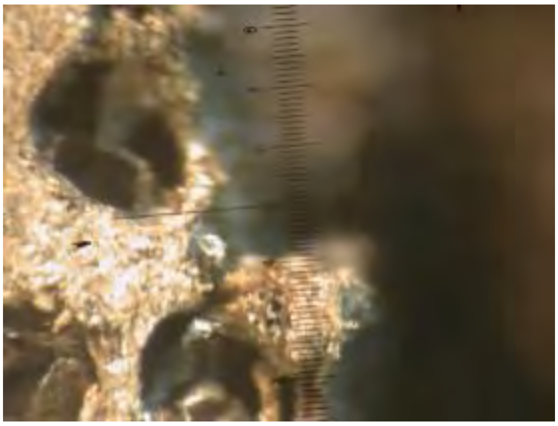



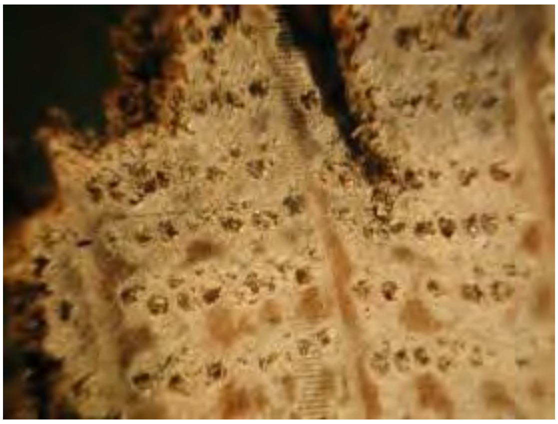
(b)

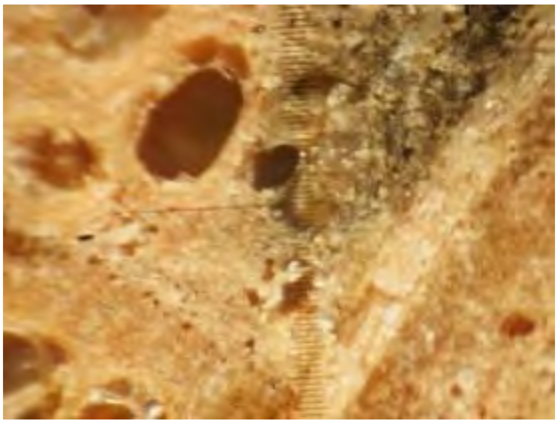
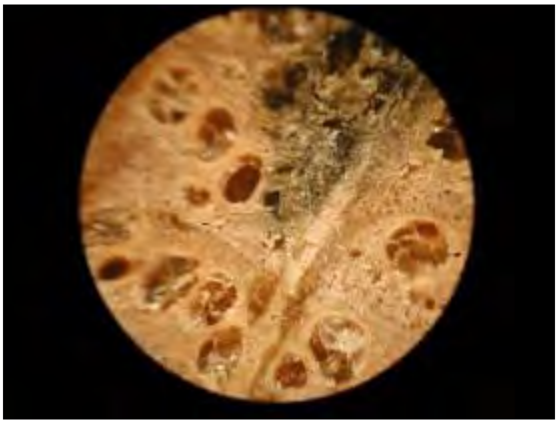
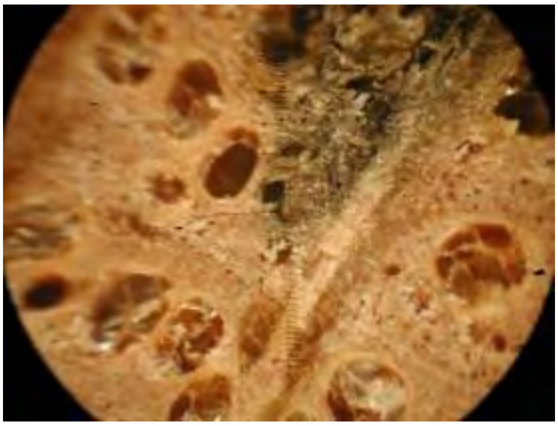
Figure G-8. Interior face of the recovered piece of white oak at a fastener location (nail) (a.). This nail depends on cleavage resistance in the wood to maintain holding force. This cleavage resistance is one of the first strength characteristics that is reduced in a decaying piece of wood (a.). In addition old nails had an iron oxide impact on the wood in cleavage around nails. The iron oxide reduces woods resilience and embrittles the cell wall and lignin that holds the cells together (b.). The hemicellulose surface of the cell wall loses its flexibility and the wood deformation zone around the nail hardens and the nail loses its fastening capability. When this happens to zones around fasteners the wood falls out of position due to its own dead weight.

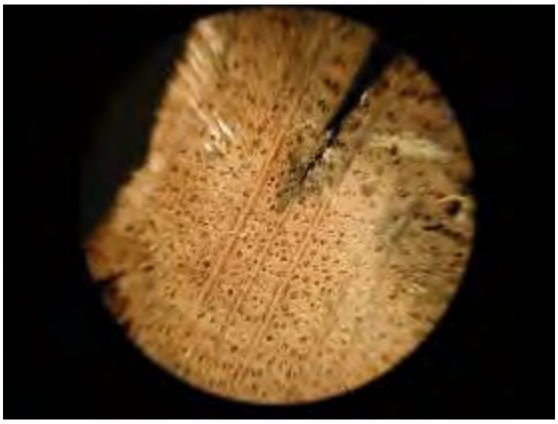


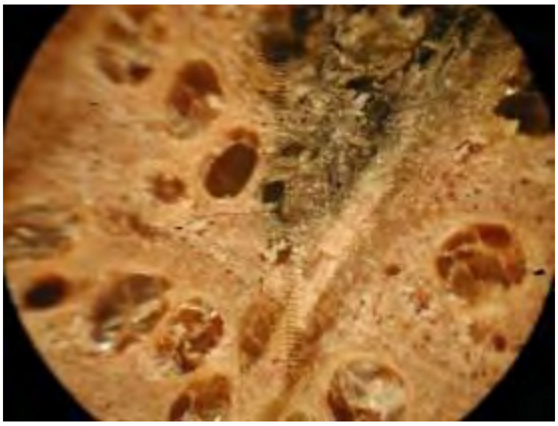



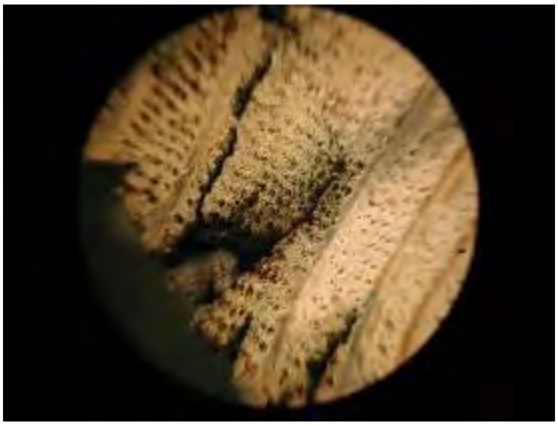


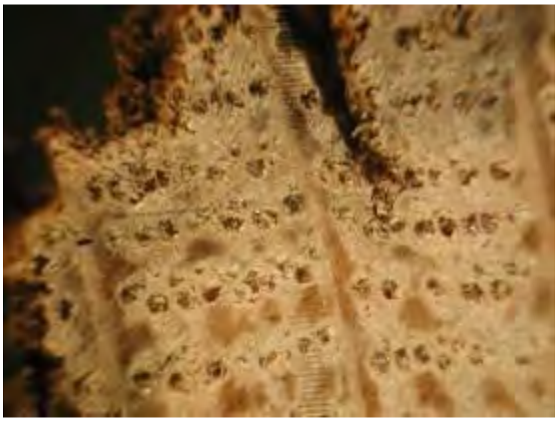
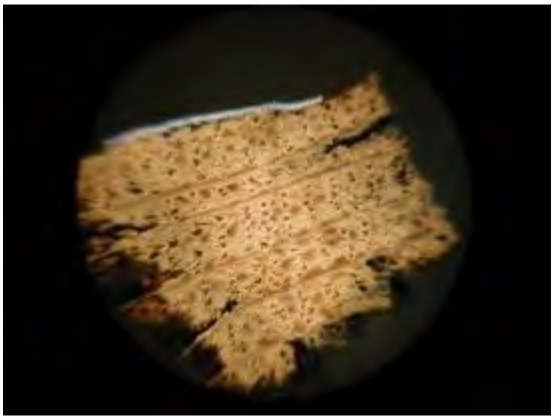


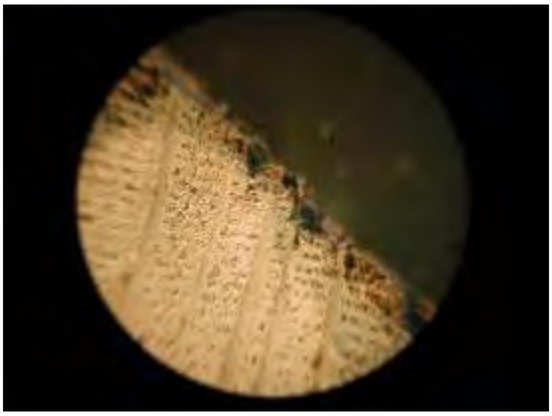





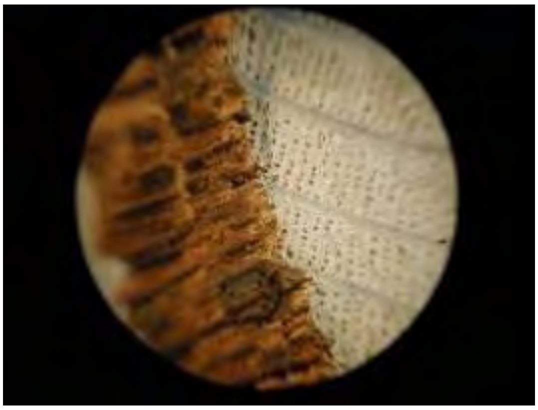










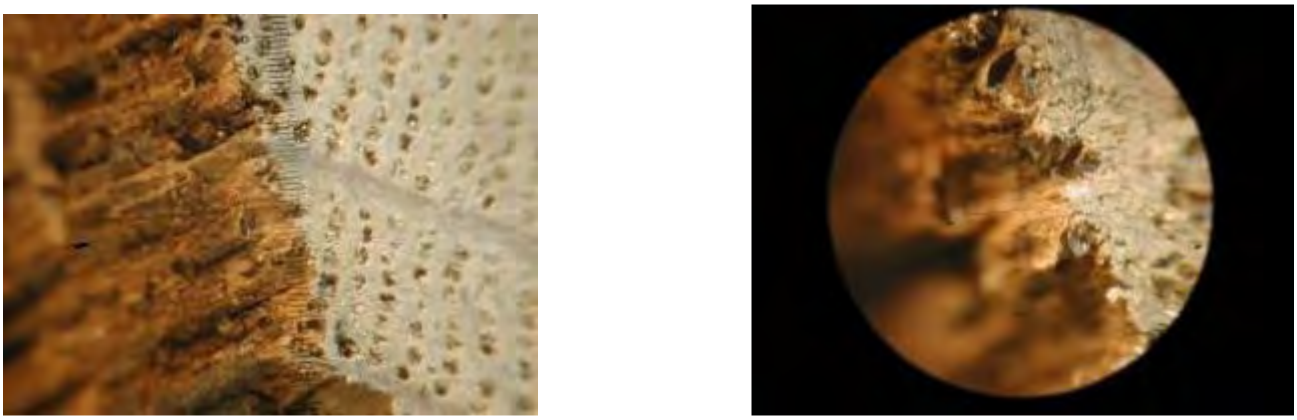


Figure G-9. Collection of HM and LM shots of interest.

Appendix I: Tingley discussion on Advanced Inspection Techniques and banding of timber piles.

Being Anisotropic wood responds to loads that are applied in different ways and directions differently than steel and concrete. This is the biggest problem I find when approaching a timber bridge from a steel or concrete point of view. The second biggest problem is typically engineers and bridge inspections lack and understanding of how decay moves through wood. By the time a pile is piped or cored/cavitated by decay the outer ring of wood has lost significant percentages of Specific Gravity. Thus, the piles become more and more incapable of withstanding axle compressive loads combined with lateral loads.

Banding for cored and cavitated broomed piles is no longer considered an effective way to improve pile axle capacity with heavily piped or cavitated piles. The bands end up being loose most of the time and the side walls buckle inward when the piles are heavily cavitated or piped. Waste of time and money. Infilling with special polymers and wrapping is the way forward coupled with fumigants.

Vertical stress wave times were extremely high through the depth of the caps/cross heads at the locations under the RSJ's. This is a direct indication of discontinuities in the cross heads/caps. Typically the through bolts allow internal decay and subsequent cavitation's. White rot is a typical cross head strength reducer. It decays the spaces between the cells (lignin) and significantly reduces longitudinal shear capacity. When the RSJ"s are placed away from the tops of the piles (in a range of 2 d or more) and the associated compression perpendicular to grain stresses, the point loads they deliver to the caps develop a more traditional parabolic stress distribution through the depth of the cross head. This distribution has a maximum at the neutral axis (3V/2A) versus a maximum shear stress near the bottom over the reaction (4V/2A). Even though the shear stress maximum value is more at the reaction, wood has a very interesting characteristic, it has up to 50% higher shear stress resistance when there is associated compression perpendicular to grain stresses. Thus, engineers often neglect shear stress a distance equal to d away from the support in a structural element. When engineers put designs in place where heavy point loads are delivered to caps away from the pile tops often a problem develops with shear cracking due to the above.

These factors usually mean that localized shear cracking either micro or macro due to overstressing in longitudinal shear. This overloading coupled with reductions in the shear resisting section due to decay typically around the vertical through bolts in the center of the cross head/cap mean that the caps/cross heads will deflect excessively or shear crack (micro or macro). The SWT wave timer will pick up these discontinuities.

I have attached some photos of other bridges in your area where Level II and III reports had been prepared by local engineers using traditional techniques and methods employed by Vic Roads and other agencies. As you can see when the Stress Wave Timer and other similar non destructive testing equipment was used they revealed that a different situation than was determined by the local engineer. These conditions were revealed for certain when more in-depth mechanical methods were employed to expose the core of the structural elements.

Appendix J: Top Ten Modifications to Timber Bridge Maintenance Manuals

Many years of observation have shown that some standard maintenance practices are, in fact, detrimental to the health and longevity of timber structures exposed to the elements. Modifying the following ten maintenance practices will prevent premature deterioration and add years of useful service life to these structures.

1. Change vertical through bolting to horizontal bolting or verticals that do not pass through the upper surface. Vertical through bolts allow moisture to penetrate the elements below, where it becomes trapped, causing decay.


Decay caused by vertical through bolts and spiking

The alternative horizontal fastening systems shown above do not allow moisture to seep down from above and become trapped in the lower members.

2. Stop the use of malthoid barrier between the deck and girders. Moisture follows the fastener shank down to where it becomes trapped between the malthoid barrier and the element below, decaying the lower element.

The use of bent metal or waterproof paper coverings has trapped moisture in these elements, causing decay.

3. Insure positive drainage that does not fall directly onto structural elements below.

Deck drainage must be directed out and completely away from the timber elements below to avoid deterioration such as shown above.

4. Accommodate dimensional change in timber elements due to changes in moisture content. Provide oval holes in side plates and other similar measures to allow for expansion and contraction of the timber.

Slotted holes have been provided in the steel parts above to allow for the expansion and contraction of the timber members. S-clips attaching the deck have been inserted into slots in the girders to allow for movement of the deck panels.

5. Stop the use of banding. Steel banding is no longer a recommended practice for stabilization of timber piles that are degraded by decay, splits, cracking, or have broomed/feathered tops. It does not prevent buckling inward; and the pile is driven tip down so the taper is down and the bands slip down the pile becoming ineffective.




Steel bands on the piles above have not strengthened the pile or stopped deterioration.

6. Stop the use of near end drift pinning.


These splits were caused by the near end drift pinning. Substitute horizontal connectors to locate elements without damaging them.

7. Provide proper clearance for timber elements to breathe and dry out.


The end of the girder on the left is buried in damp debris and cannot dry out. To prevent deterioration, the girder end should be open and free to breathe. The concrete encasing the pile and wales on the right is meant to strengthen the pile, but is instead holding moisture against the timber, accelerating the rate of decay.

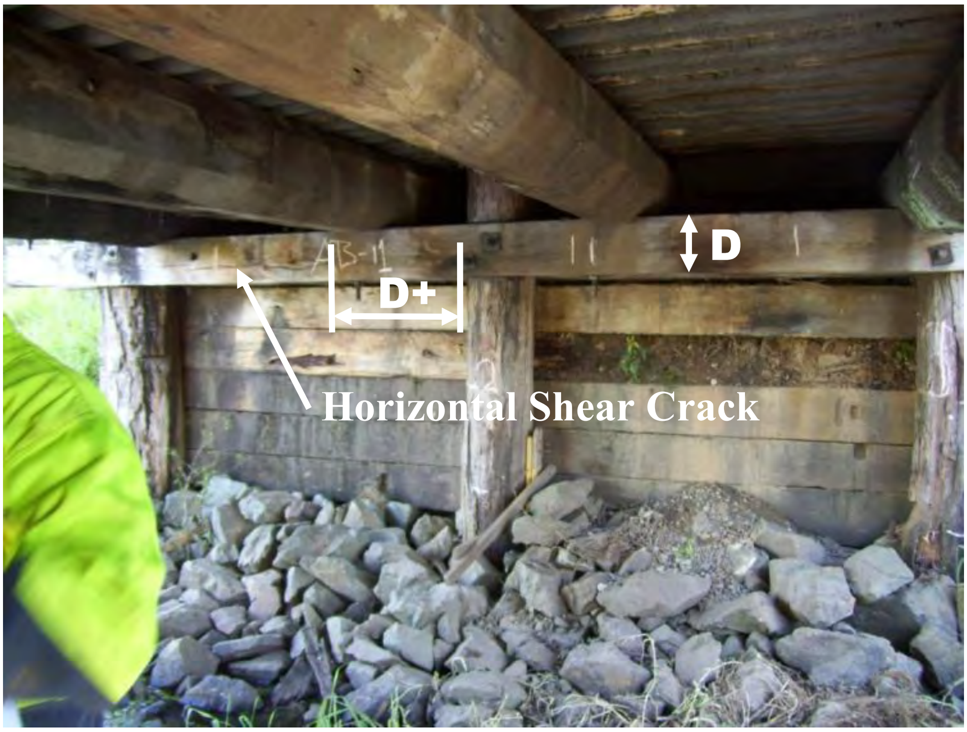
8. Stop the use of heavy percentage solid coatings (over 29% solids). Moisture can penetrate the coatings at vertical holes or damaged surfaces and become trapped against the timber, promoting decay. Use stains (less than 29% solids) that allow the wood to breathe.

Heavy solids paint has been applied to these kerbs and rail. Vertical through bolting and impact damage have then given moisture a way to infiltrate under the paint where it cannot evaporate, accelerating the rate of decay.

9. Stop the use of heavy notching which promotes reentrant corner cracking. Use a 6:1 slope cut instead.

Reentrant corner cracking at the snipe or notch in these girders could have been eliminated if a longer (6:1) slope was cut. These girders could be stitched or injected to stabilize the cracking after a longer slope cut is machined. Such cuts should be CN and sealed. The girders should be diffused to prevent decay.

10. Use properly sized timbers in pile bents and place loads within D of the pile to prevent micro-checking (horizontal shear cracking) in undersized cross heads.



Girders should be located no more than D (the depth of the crosshead) away from the piles to prevent shear failures. These girders are too far from the piles, and a horizontal shear crack is visible in the crosshead.