United States Department of Agriculture

Evaluating Fire-Damaged Components of Historic Covered Bridges

Brian Kukay
Charles Todd
Tyler Jahn
Jenson Sannon
Logan Dunlap
Robert White
Mark Dietenberger

Forest Service
Forest Products Laboratory
General Technical Report
FPL–GTR–243
August 2016

In cooperation with the
United States Department of Transportation
Federal Highway Administration

A FHWA-NHCBP (National Historic Covered Bridge Program) publication

Abstract

Arson continues to claim many historic covered bridges. Site-specific, post-fire evaluations of the structural integrity of a bridge are often necessary in a fire's aftermath. Decisions on whether individual wood components can be rehabilitated, reconstructed, or replaced must be made. This report includes coverage of existing approaches and exploratory approaches that can be used for general guidance and a select number of more specific treatments that can be used to approximate the residual capacity of individual fire-damaged members. Topics such as fire-retardant treatments and other measures to prevent future fire damage are also discussed.

Keywords: covered bridge damage, rehabilitation of fire damaged bridge, fire resistant protection of structural wood

Cover photo: The Cedar Covered Bridge, Winterset, Iowa, was destroyed by a fire set by an unknown arsonist on the evening of September 3, 2002. The Madison County Board of Supervisors have constructed a replica of the original bridge. Image courtesy of the Winterset Madisonian/Dave Braga.

August 2016

Kukay, Brian; Todd, Charles; Jahn, Tyler; Sannon, Jenson; Dunlap, Logan; White, Robert; Dietenberger, Mark. 2016. Evaluating fire-damaged components of historic covered bridges. General Technical Report FPL-GTR-243. Madison, WI: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory. 34 p.

A limited number of free copies of this publication are available to the public from the Forest Products Laboratory, One Gifford Pinchot Drive, Madison, WI 53726–2398. This publication is also available online at www.fpl.fs.fed.us. Laboratory publications are sent to hundreds of libraries in the United States and elsewhere.

The Forest Products Laboratory is maintained in cooperation with the University of Wisconsin.

Acknowledgments

The authors acknowledge the many contributions of James Wacker, Sheila Rimal Duwadi, Mike Ritter, Carol Clausen, and Bob Ross.

This study is part of the Research, Technology and Education portion of the **National Historic Covered Bridge Preservation** (NHCBP) Program administered by the Federal Highway Administration. The NHCBP program includes preservation, rehabilitation and restoration of covered bridges that are listed or are eligible for listing on the National Register of Historic Places; research for better means of restoring, and protecting these bridges; development of educational aids; and technology transfer to disseminate information on covered bridges in order to preserve the Nation's cultural heritage.

This study is conducted under a joint agreement between the Federal Highway Administration–Turner Fairbank Highway Research Center, and the Forest Service–Forest Products Labortatory.

Federal Highway Administration Program Manager—Sheila Rimal Duwadi, P.E.

Forest Products Laboratory Program Manager—Michael A. Ritter, P.E.

Contents

In memory of

Robert White,

who will live on through the historic covered wood bridges that dot our great country, through the county bridge engineers that make use of this report, and through those who reference his many other works now and in the future.

Evaluating Fire-Damaged Components of Historic Covered Bridges

Brian Kukay, Ph.D., P.E, Associate Professor of General Engineering
Charles Todd, Ph.D., Tenured Full Professor, Department of Mathematical Sciences
Tyler Jahn, M.S., Graduate Student, General Engineering
Jenson Sannon, M.S., Graduate Student, General Engineering
Logan Dunlap, B.S., Student, General Engineering
Montana Tech of the University of Montana, Butte, Montana

Robert White, Ph.D., Research Forest Products Technologist
Mark Dietenberger, Ph.D., Research General Engineer
USDA Forest Service, Forest Products Laboratory, Madison, Wisconsin

Executive Summary

Covered bridges, which used to cover the American landscape, now number less than 900 bridges as a result of various factors including neglect, arson, vandalism, and natural disasters. Covered bridges tend to be in isolated locations and constructed of flammable materials; therefore, they are susceptible to arson and vandalism. Because it is typically difficult for firefighters to arrive at the bridge in a timely manner, any fire developing on the bridge can cause critical damage or completely destroy the bridge. Therefore, numerous security measures have been increasingly applied that provide deterrence, deny access, detect threats, provide protection, and educate the public. Passive protections such as lighting and fire retardant coatings are relatively low cost. Additional fire protection technologies are discussed in the *Guide for In-Place Treatment of Wood in Historic Covered and Modern Bridges* (Lebow and others 2012b). However, arsonists can overcome such protection and will require active security measures as described in the *Covered Bridge Security Manual* (Phares and others 2013). Although the famous Cedar Bridge in Madison County, Iowa, was destroyed in 2002 (cover page), along with many others at the same time and earlier, many more covered bridges are now surviving fires because of increased security measures implemented over the years. Because of the low costs involved, restoration of such limited damaged bridges is becoming increasingly viable rather than wholesale replacements, thereby extending the scarce restoration funding for the historic structures. Considerable confusion remains for understanding the level of fire damage and of the restoration process required. Much of this is not discussed in other manuals and is addressed here.

This guide will identify technologies and methodologies used to evaluate the fire-damaged components and the consequential methods of restoration and rehabilitation will also be identified and organized. The guide is provided in the logical order, as first understanding fire damage effects on the bridge structural properties (Sections 2 and 3), evaluating the fire-damaged wood elements (Sections 4 to 6), restoring the bridge with repairs, fire prevention, and controls (Sections 7 and 8), and citing standards, programs, and guidance (Sections 9 and 10). Perhaps the most important chapter to engineers is Section 3 on reduced section analysis of fire-damaged wood elements that provide a basis to guide the later chapters on inspection and restoration. The tools for post-fire inspection of the char regions mainly determine the depth of char, preferably using a minimal destructive device or NDE testing (see Appendix) of the char layer thickness determination. By having a value for the char layer thickness as input to the loading analysis of the reduced section will help determine if the char layer only needs to be removed or left as it is for posterity, or if the strength and load capacity of the damaged member is calculated to be so reduced by a deep char that it needs to be replaced. Although the presence of char on the covered bridge can look alarming, a careful post-fire inspection can reveal how minimal a restoration is actually required. This would be particularly true if the security measures installed provide timely fire protection that prevent or suppresses a historic covered bridge fire in its early stages.

1.0 Introduction

In 1804, the first documented covered bridge was built in America: the Union Bridge by Theodore Burr. This bridge was constructed in New York and spanned the Hudson River for 105 years (Griggs 2014). Although this bridge no longer exists, historic covered timber bridges grace landscapes across the country. Neglect, improper maintenance, and fire damage from both people and nature challenge the life of these bridges. It is important to properly maintain and preserve these historic, sometimes even romantic, landmarks as they connect us to the past. A 2012 photograph of the historic Red Covered Bridge, completed in 1864, is shown in Figure 1.

Figure 1—Red Covered Bridge near Princeton, Illinois.

Covered bridges began to spread westward and soon became a staple of historic transportation. Today, approximately 500 to 600 historic covered bridges are still standing in the United States (Wacker and Duwadi 2008). A proactive movement has begun to ensure proper rehabilitation, restoration, and preservation of these historic landmarks while simultaneously raising public awareness.

After a historic covered bridge has been affected by fire, a post-fire assessment is advantageous to investigate altered, individual members and their connections. The vast majority of historic covered bridges are made of timber stringers cut from solid sawn lumber. Unless the fire is severe, it is usually not necessary to replace all the large members after a fire. Selwyn Fox (1974) conducted a study using three 50-ft 24F Douglas-fir stress-grade beams that were salvaged from a warehouse after a severe fire had reduced each member's cross section by nearly 10%. Two-point loading tests were conducted on each beam after they were cut into six 10-ft sections and three 30-ft sections. The results showed that there was no evidence that the fire had reduced the strength of the beams beyond the reduced cross section and effected glue-line.

The goal for historic covered bridges is to return them to their original state while staying true to historic building practices and materials. Rehabilitation and reinforcement of fire-damaged timber stringers provide a method that is much less costly than replacing those damaged parts. This is important because bridge replacement may not be an option for various reasons (Wacker and Duwadi 2008). If appropriate analysis, repairs, and treatment are undertaken, fire-damaged wood members can often be restored instead of replaced (White and Ross 2014).

The objective of this publication is to provide guidance on the available methodologies to evaluate the residual load capacities of fire-damaged wood members. After a review of some background information, we discuss the reduced section approach normally used to evaluate the residual load capacity of a fire-damaged wood member. Next, we discuss items that should be included in a site visit of the damaged bridge. For situations where uncharred wood needs to be evaluated for reduced load capacity, we review the options for evaluating the residual wood for reduced mechanical properties. We conclude the publication with discussion of post-fire repair and actions that can be taken for fire prevention and damage control.

2.0 Background Information

When heated to high temperatures, wood undergoes thermal degradation to char and volatile gases. Ignition of the volatile gases results in the flames that facilitate the spread of the fire. Surface heating of a wood member in a fire results in surface charring and a steep temperature gradient. The stages of thermal wood degradation become zones of degradation in a structural wood member exposed to fire. In a

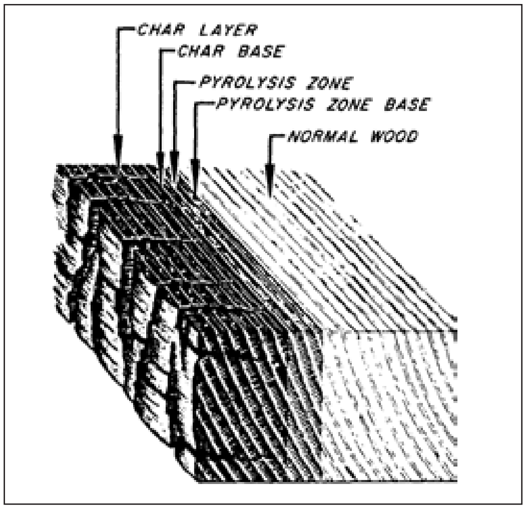


Figure 2—Degradation zones in a wood section.

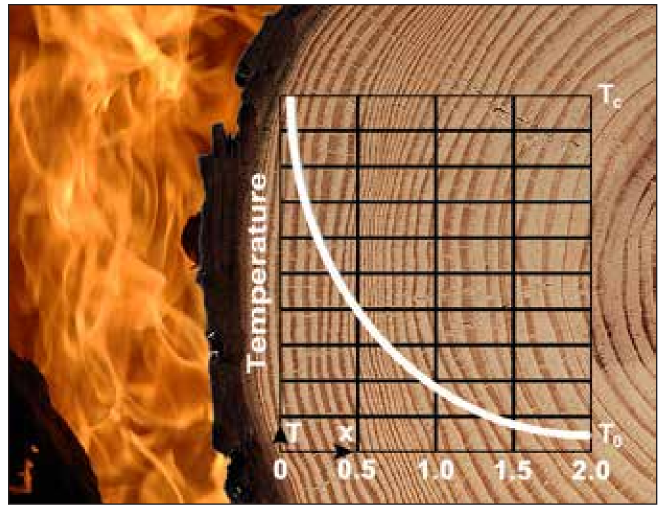


Figure 3—Temperature gradient beneath char layer in standard fire resistance test.

broad sense, there is an outer char layer, a pyrolysis zone, a zone of elevated temperatures, and the cool interior (Fig. 2). These zones of degradation reflect the temperature profile through the cross section. The extent of thermal degradation within these zones determines the residual load capacity of the wood components after a fire.

2.1 Thermal Degradation, Ignition, and Charring of Wood

Thermal degradation without apparent charring or mass losses can be detected first as losses in flexural properties at elevated temperatures (White and Dietenberger 2001, 2010). From 200 to 300 °C (392 to 572 °F), the wood components of extractives, hemicelluloses, and lignin begin to undergo significant degradation resulting in mass losses and complete loss of strength (Rowell and Dietenberger 2013; Wang and others 2014). Significant depolymerization of the cellulose component of wood occurs between 300 and 350 °C (572 and 662 °F). Thermal degradation of the wood to a char residue causes reductions in the density of the wood and the shrinkage of the surface char layer. The thermal degradation of a wood member depends upon the temperature and duration of the fire, with the char layer increasing as exposure increases.

Wood can ignite under piloted ignition or autoignition. Autoignition is where wood ignites in the absence of a direct flame. Piloted ignition is where an external flame or spark can ignite the combustible gases generated by the thermal degradation of the wood. Piloted ignition at heat fluxes sufficient to cause a direct-flaming ignition normally occurs at surface temperatures of 300 to 365 °C (Dietenberger 2004). Babrauskas (2001) summarized the findings of several works, spanning 30 researchers and more than 100 years, as they relate to the ignition temperature of wood. His findings show that all wood-based materials have similar ignition temperatures.

When “sufficiently sized” wood members undergo exposure to elevated temperatures from a direct flame, thermal penetration occurs a distance inward from the base of the char layer. The immediately adjacent region of thermal degradation of the wood is referred to as the pyrolysis zone (Fig. 2). The char layer itself acts to insulate the wood. The exposed surfaces of sufficiently sized members decrease gradually. The charring rate depends on the species, orientation, and size of the specimen (Firmanti and others 2004). In the standard fire-resistance tests used in the building codes to regulate building elements, charring of wood occurs at a predictable rate for heavy timbers. The char rate of panel products and dimensional lumber will increase as the temperature at the center or backside of the wood product rise above the initial ambient temperature.

As wood burns, marked zones of degradation become apparent (if a specimen is thick enough). In response to a fully developed fire, the base of the char layer of the wood is commonly associated with a temperature of 300 °C. In Fahrenheit, a temperature of 550 °F (288 °C) has been used to identify the base of the char layer. Wood exposed to temperatures in excess of approximately 300 °C will form a residual char layer. Obviously, any charred portion of a fire-exposed wood member has no residual load capacity. The wood beneath the char layer that is subject to some thermal degradation because of exposure to elevated temperature. This wood has residual load capacity but this residual capacity may be less than the load capacity prior to the fire. The normal wood (located beneath the zone of elevated temperatures) remains unaltered from the fire.

Temperature gradients (associated with timbers) occur at the base of a char layer. The temperature gradient of elevated temperatures in uncharred wood starts at the base of the char layer (300 °C) and fades into undamaged wood (Fig. 3). In the standard fire resistance test, reported temperatures include 177 °C (350 °F) at 6 mm (0.2 in.) and 104 °C (220 °F) at 13 mm (0.5 in.) beneath the char layer. Using the data of White and Nordheim (1992), Janssens and White (1994) determined the depth of the zone to be approximately 33 mm for large beams exposed in the standard fire resistance test. Beneath this layer is the normal or unaffected wood. There are equations and data for calculating char rates and temperature gradients that are valid when the member is thick enough to be considered a semi-infinite slab (White 2004). Additional information on thermal degradation, ignition, and charring of wood can be found in Browne (1958), White and Dietenberger (2001), and White and Nordheim (1992).

2.2 Elevated Temperature Impact on Mechanical Properties

Irreversible effects on the mechanical properties of wood can occur during the elevated temperatures associated with fire events. At a temperature of 66 °C (150 °F), the first loss of structural properties occurs, and significant degradation occurs within the temperature range of 200 °C to 300 °C (392 °F to 572 °F), with the latter temperature being the base of the char layer (Ross and others 2005a). The irreversible effects of elevated temperatures on mechanical properties depend on moisture content, heating medium, temperature, exposure period, and the species and size of the piece involved (Green and others 1999; Kretschmann 2010). The National Design Specification (NDS) for Wood Construction advises that prolonged heating to temperatures above 66 °C (150 °F) can cause a permanent loss of strength (American Wood Council 2012). Whereas days of heating at 66 °C can have a permanent effect on mechanical properties, the immediate temperature effect on mechanical properties is reversible for heating periods that are limited to hours at temperatures below 100 °C (Kretschmann 2010). In contrast to the immediate impact on strength properties of individual wood members at elevated temperatures, the permanent loss in compressive strength occurs at higher temperatures than a comparative loss in tensile strength (Schaffer 1977, 1982a,b).

Effects of elevated temperatures are primarily on the strength properties. The effect on stiffness is considerably smaller. The NDS supplied multipliers that are applied to the tabulated stresses for wood that will experience sustained exposure to temperatures of 100 to 150 °F are more severe for bending and tensile stresses than they are for compressive or buckling stresses. In addition to temperature, adverse effects depend on duration and type of exposure. In contrast to the impact on strength properties of wood at elevated temperatures, the loss in tensile strength after cooling to room temperature is greater than the loss in compressive strength. While days of heating at 66 °C (150 °F) can have a permanent effect on mechanical properties, the temperature effect on mechanical properties is reversible for heating periods of hours at temperatures below 100 °C (212 °F). Thus, the zone beneath the char layer between 100 °C (212 °F) and 300 °C (550 °F) has the potential of irreversible loss in mechanical properties at varying levels due to thermal degradation.

Without extinguishment, a fire has three phases in an interior structural fire: 1) the growth of the fire from ignition to flashover; 2) the fully developed post-flashover fire; and 3) the decay period of declining temperatures as the fuel sources are consumed. Flashover is the full involvement of the combustible contents of the compartment and is associated with flames out the door in the standard room-corner test. Whereas temperatures are lower, presuming that the wood members self-extinguish, the cooling period of the decay period prolongs the duration of elevated temperature exposure. A prolonged cooling period is accompanied by diminishing surface temperatures, while the temperatures in the center portion of the cross section may still increase. Hence, elevated internal temperatures may result in further decline of residual properties. Additional information on elevated temperatures impact on mechanical properties can be found in Kretschmann (2010) and Schaffer (1984).

2.3 Fire Resistance Ratings of Exposed Wood Elements

Requirements for structural integrity during a fire are a major component of the building code provisions pertaining to fire safety. These requirements for buildings are in terms of fire resistance ratings of the structural components. The ratings depend on construction type and occupancy classifications. ASTM E 119 (ASTM 2010) describes the full-scale test methods used to determine fire resistance of structural members. Structural failure occurs when a member is no longer capable of supporting its designed load during the standard fire exposure. The fire exposure of the standard fire resistance test approximates the second phase, or post-flashover portion, of the fire. Some of the specified temperatures for the standard time–temperature curve are 538 °C (1,000 °F) at 5 minutes, 843 °C (1,550 °F) at 30 minutes, and 927 °C (1,700 °F) at 1 hour.

The standardized fire test used to determine the fire resistance ratings of structural members has resulted in considerable data on the charring rate and temperature gradients of the remaining uncharred wood in a semi-infinite wood slab (Babrauskas 2001; Buchanan and Barber 1994) (Fig. 4). For such exposures, it is generally assumed that the temperature at the base of the char layer is 300 °C. As noted previously, the thickness of the wood layer with elevated temperature beneath the base of the char layer of elevated temperature is approximately 35 mm, and the temperature profile can be approximated by a parabolic curve.

For exposed wood members, there are methodologies for calculating the residual load capacity during the standard fire resistance test and thereby determine the fire resistance rating of the wood member. The methodology with current

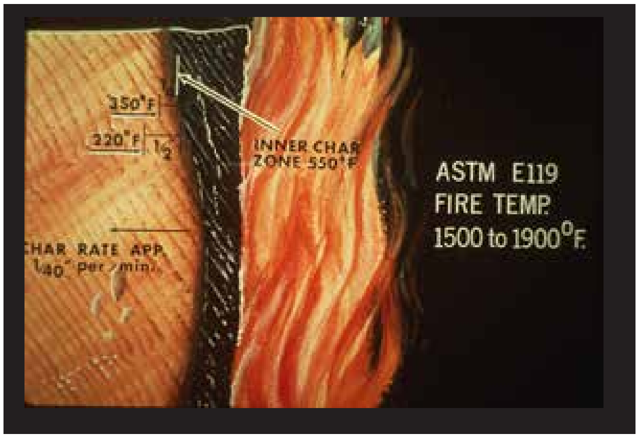


Figure 4—Illustration of a charring wood member exposed to the standard fire exposure of 815 to 1,038 °C.

building code acceptance is described in Technical Report 10 of the American Wood Council (2014) and includes information from the National Design Specification for Wood Construction (NDS) (American Wood Council 2012). The NDS specifies the design methodologies and property values to be used for the structural design of a wood structure. The equations used to estimate residual load capacity during fire exposure uses char rate equations to obtain reduced structural area of a member per time exposed to fire. One can account for additional thermal damage to uncharred wood in the load capacity calculations by including a zero-strength layer in the dimensions of the residual cross section. The NDS account for residual effects of wood in the zone of elevated temperature beneath the char layer as well as corner rounding by applying a multiplier factor of 20% to the char depth to obtain the reduced cross-sectional area of the wood (American Wood Council 2012). This reduced area is then used to solve for an estimated maximum capacity for a fire exposure time (American Wood Council 2014). The allowable design values in the NDS for the mechanical properties of a given grade and species are increased to their average ultimate strength values. Using the normal assumption of a char depth of 1.5 in. at one hour, the 20% calculation for the zero-strength layer is equal to 0.3 in. for a one-hour fire resistance rating. The methodology has been shown to be applicable to both dimension lumber members as well as heavy timber members.

2.4 Visual Grading of Lumber

From start to finish, wood construction incorporates numerous wood products into a number of primary and secondary structural applications. Wood is inherently variable and there are a number of factors that contribute to the performance of wood-based materials. Grading procedures account for the underlying factors of wood strength such as specific gravity, slope of grain, and the presence of knots. Grading procedures include visual grading criteria, nondestructive measurement such as flat-wise bending, stiffness, or density, or a combination thereof (American Wood Council 2012). In cases where graded lumber was used in the covered bridge, the grade stamp on the lumber may still be readable in areas protected from the weather.

For purposes related to post-fire assessments where the tabulated allowable design stresses for the grade are employed, it is recommended that the wood members be re-graded after the char is completely removed. The charring of the wood member is similar to ripping the wood member with a saw in terms of its impact on the mechanical properties and grade of the member. Visual stress grading rules are based on member size and characteristics (such as knots) of the outer zones. Thus, the removal of the outer zone as the result of the charring can greatly impact the stress grade of the member. Re-grading procedures take into account the impact of residual dimensions on the applicable grading rules for the reduced dimension as well as the altered relative locations of strength reducing characteristics in the cross section. To avoid confusion, the zero-strength layer could also be removed, or if it is to be retained as a protective surface, to account for it in the analysis. The re-grading of the members should be done by an appropriate a supervisory grading agency. The grading agency can make a qualified statement for each timber based on what grade characteristics are evident for each beam. A list of grading agencies is available on the web site of the American Lumber Standard Committee, Inc. (2015).

For the wood components that are not solid sawn lumber, the determination of the appropriate grade can be more difficult. Laminations of different grades are often used in construction of a glued-laminated timber. The inability to view the wide faces of the interior laminations will complicate the necessary re-grading of the residual beam. In addition, glued-laminated timbers use very high quality outer laminations and the loss of these laminations from a fire can severely affect the possibility of salvaging these timbers.

2.5 Preservative-Treated Wood

Some wood in a covered bridge may be preservative-treated wood. While little data are available, the ignition and flammability behavior of water-borne preservatives are considered to be similar to untreated wood. Oil-borne preservatives, such as creosote, can be much more flammable than untreated wood. Weathering of creosote-treated wood appears to reduce the adverse impact of the creosote treatment. There is special concern for covered bridges carrying railroad tracks using creosote treated wood, as the potential for fire related to overheating and grinding of tracks from stuck brake mechanism is high.

One common preservative treatment for wood is CCA (chromated copper arsenate). A specific fire performance issue with CCA-treated wood is the potential for after-glow. Because of this after-glow behavior, fence posts have been known to be completely consumed hours or days after a grass fire caused only minimal surface charring to the fence post before the grass fire self-extinguished. Because the continued after-glowing of the CCA wood is not easily visible, extra efforts need to be taken to insure that CCA-treated wood is truly extinguished. However, the fire risk posed by the CCA-treated wood may imply additional measures of fire protection than usual. The replacement of the CCA-treated wood itself will eliminate this concern.

For above-ground applications where leaching is not an issue, the boron-based preservatives are a common preservative. Boron is also a component of many interior fire-retardant treatments. Retentions levels for wood preservatives are much lower than those necessary for flame retardants.

3.0 Reduced Section Analysis of Fire-Damaged Wood Elements

The interior residual wood of timber stringers is protected from the intense heat of the fire by the char layer that develops. As a result, large structural wood members such as timbers do not necessarily need to be replaced after a fire (Buchanan 2001). Common practice with timbers (defined as those 5-in. by 5-in. and greater) neglects the reduced capacity associated with the elevated temperature zone. As such, once the char layer is removed, the remaining cross section is assumed to contribute fully to the load-carrying capabilities of the member (Firmanti and others 2004). Instances arise where negating the elevated temperature zone leads to un-conservative estimates of residual load-bearing capacity (Fig. 5). The wood beneath the char layer has residual load capacity; but this residual capacity may be less than the load capacity prior to the fire. A new cross section size is determined by carefully removing the char layer then measuring the char-free section dimensions. Research indicates that an additional reduction in the cross section may be warranted along with measurements of like members exposed to varying degrees of degradation (Kukay and others 2013).

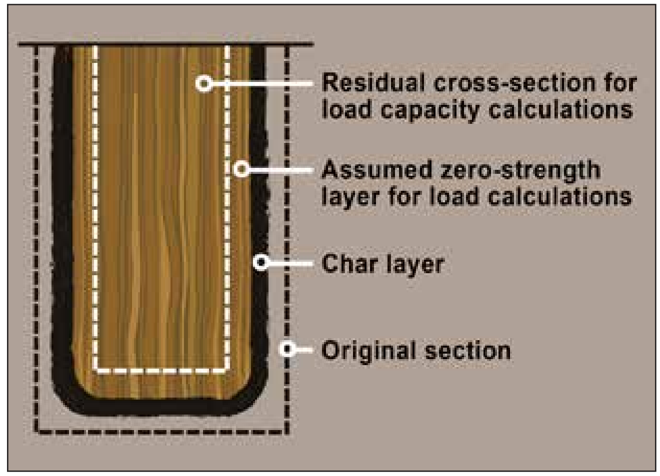


Figure 5—Reduced section approach to fire-damaged wood.

Most available data on the structural performance of wood in a fire are based on research using the standard fire-resistance test. For the post-fire assessment, the exposure of the structural wood members to elevated temperatures during the decay period of fire development should be considered. Although the temperatures are lower during the decay period, the duration of the exposure can be prolonged compared with the duration of the fully developed post-flashover fire phase. The steep temperature gradient near the fire-exposed surface assumed in the normal assessment of residual load capacity during a fire is based on transient heating coupled with progressive charring of the wood cross section. During a prolonged cooling, the surface temperatures will decline while interior layer temperatures on the cool side may increase. Tests have indicated that this increase in the temperatures in the interior of the wood member caused by redistribution of the heat after fire exposure is particularly common for wood protected with gypsum board. Because the decay or post-extinguishment period is one of reduced temperatures, many observations of damage at the fire scene will be less helpful in establishing the intensity and duration of the exposure during this period.

Recommendations for the additional reductions of cross sectional dimensions have varied. Often, it is assumed that the wood beneath the pyrolysis zone in a heavy timber structural member after a fire can be assumed to have full strength (Buchanan 2001). In *Evaluation, Maintenance and Upgrading of Wood Structures, A Guide and Commentary* published by the American Society of Civil Engineers in 1982, recommendations for evaluation of fire damage were for "removal" of a fixed amount of wood (Freas 1982). Recommendations included removal of the char layer plus approximately 1/4 in. or less of wood below the base of the char layer. For members controlled by compressive strength or stiffness, the recommendation was that no additional adjustment beyond removal of 1/4 in. was necessary to apply the basic allowable design stresses to the residual cross-sectional area. For members controlled by bending strength or stiffness, the recommendation was either removal of an additional 0.625 in. or removal of an additional 1/4 in. in combination with a 10% reduction in the allowable design value used to calculate the load capacity of the residual cross-sectional area.

In applying the methodology to the available strength data for permanent strength loss and the temperature profile reported for ASTM E 119 (ASTM 2010), White and Woeste (2013) concluded that 0.1 in. to 0.3 in. is a reasonable recommendation for the zero-strength layer of a member loaded in compression in a post-fire load capacity analysis when used with the NDS adjusted design values. For members loaded in tension or bending, the recommendations were a thickness of 0.3 in. to 0.5 in. These recommendations assume that the zero-strength layer is not physically removed from the member and the temperature at the center of the timber did not increase based on the likely temperature profile during the fire. One can further adjust the depth of the zero-strength layer downward by a fraction (e.g., 50%) of any uncharred depth removed for appearance reasons. Selection of values between 0.3 in. and 0.5 in. should be based on the duration of the fire as reflected in the observed char depth and location of members relative to direct exposure to flames. The observed thickness of the residual char layer will be less than the observed reduction in the dimensions of the charred member due to shrinkage of the char layer. In the context of these recommendations, 0.3 in. for compressive members and 0.5 in. for tension and bending members are the more conservative values for the thickness of the zero-strength layer in the calculation of load capacity.

One approach is to use a series of assessments in applying the reduced section analysis to the fire-damaged wood (White and Woeste 2013). As a simpler assessment indicates the potential for retention of the damaged wood elements, subsequent analysis refines the input data and assumptions used to make the assessment. In the initial simple analysis, calculations are done using the following:

- Reference properties listed in the NDS for a common grade and species used for bridge construction.
- Initial estimates of residual cross sections of the wood elements.
- The charred layer is assumed to have no residual strength and stiffness but residual wood is assumed to be undamaged.

For those members identified as having the potential to be adequate for continued use, a more refined set of calculations can be made using the following:

- Reference properties listed in the NDS for the identified species of the structural element in the bridge being evaluated and its likely grade.
- More detailed measurements of the residual cross-sectional areas and assumptions on the depth of an equivalent zero-strength layer can be refined.

Once the elements have reduced to their final dimensions, the final affirmative assessment of fire-damaged wood components needs to include visual grading of the residual structural elements. This re-grading needed for this final assessment would need to be done with the charred members cleaned of the char and reduced to their final dimensions. The maximum potential grade for each beam should be established and documented by an appropriate supervisory grading agency. Also, the responsible party should verify the final residual size of each timber for use when checking all fire-exposed timbers in the structure. The load history of the timbers may be important. The possibility of overloads in-service, or cumulative damage, should be investigated as well as damage from decay.

When dealing with calculations for residual load capacity of structural glued-laminated members, the structural grade variations of individual components that make up a composite wood member need to be considered. Glued-laminated members usually have higher grades for balanced and layup beams that are appropriate for the outer tension and compression laminations than the interior laminations near the neutral axis; therefore, damage to the outer higher grade laminations can have a more significant impact on the residual load capacity of glued-laminated beams after a fire event (Ross and Pellerin 1994). The determination of the appropriate grade is also more difficult. The inability to visually inspect the wide faces of the interior laminations will complicate the necessary re-grading of the residual beam.

4.0 Post-Fire Inspection and Site Visit

A site visit is needed to assess the damage of a fire to a historic covered timber bridge. As part of this site visit, visual post-fire inspection process of individual members should facilitate an appropriate course of action. In so doing, technical personnel need to assess the residual structural integrity of individual members that exhibit varying degrees of degradation. The objective of the initial assessment is to identify those members that need to be replaced due to the fire damage and those members that warrant further investigation of their residual load capacity.

Observable variants can be documented using various pieces of equipment such as a moisture content meter (James 1988), drill resistance press, increment core extractor and other equipment available for NDE assessment of wood members. Other issues related to decay and other pre-fire damage can also be noted at this time and analyzed. Pre-fire damage consists of degradation to a load-bearing solid wood member due in part to decay, insect damage, splitting or cracking, and overloading. Glued-laminated load-bearing beam damage can consist of the same degradations as solid wood, but also includes delamination and finger-joint defects (Garab and others 2010). All of these topics should be addressed during the post-fire evaluation.

4.1 Fire Itself

An understanding of the fire itself might aid in determining the degree of thermal damage beyond the base of the char layer. The standard for fire investigation is NFPA 921 *Guide for Fire and Explosions Investigations* (NFPA 2013), which can provide some guidance on the source of fire, the size of the fire, and its duration. Information gathered in the investigation of the fire itself may help establish likely maximum temperatures in various locations and the durations of the fire exposures. Although visual char depths may provide some insight, predicting a time of exposure to fire is difficult to determine from the depth of char and overall damage. Complicating factors including grain orientation, heat flux exposure, and moisture content can vary significantly, even within a given species. Accordingly, a uniform char rate cannot always be supported (Schroeder 1999). These caveats on wood charring should be taken into account when using the standard for fire investigations. To get further answers, professional fire investigators can be consulted, as they would have various investigative tools not available otherwise, such as chemical analysis of critical samples, fire modeling, and so on.

4.2 Decay Damage

While the inspection was initiated in response to a fire, an assessment of the damage should also verify that the timbers are otherwise “sound” and have not been damaged by factors other than fire exposure. Individual, biodegraded bridge components can be quantified with the use of NDE techniques to determine the impact of environmental conditions on the wood. Using NDE techniques, a complete analysis of the individual members can be determined without compromising structural integrity (Pellerin and others 1996). Many of the NDE discussed in this publication for potential use with fire-damaged wood are NDE techniques more widely used to detect decay or natural defects in structural member. The assessment of wood structures for decay is extensively discussed in the *Wood and Timber Condition Assessment Manual* (White and Ross 2014) and other publications.

The topics of biodeterioration of wood structures including bridges, options for in-place treatment to prevent or arrest the degradation, and the use of preservative treated wood in historic structures are discussed in other publications (Lebow and others 2012a, Lebow and Anthony 2012).

4.3 Moisture Content

A moisture meter is valuable for measuring the moisture content (MC) of wood members. Since the MC has a great effect on the strength properties of wood, the MC of individual members must be measured and recorded during the investigation. Guidelines for taking MC readings can be found in the Forest Products Laboratory (FPL) technical report, *Electric Moisture Meters for Wood* by William L. James (1988) and the *Standard Methods for Use and Calibration of Hand-Held Moisture Meters*, ASTM D 4444 (ASTM 1992). Electrical resistance type meters are recommended along with 3-in.-long, insulated pin probes.

4.4 Dimensions of Residual Cross Sections

The initial site visit can be used to identify critical members that might warrant further investigation of the degree of damage to the mechanical properties of residual wood. For those members identified as having potential to be adequate for continued use, a more refined assessment involves more detailed measurements of the residual cross-sectional areas. The thickness of the char layer is likely different than the thickness of the wood charred as the result of shrinkage and surface recession. Thus, it is not advisable to derive the thickness of residual sections by subtracting visual char depths from original dimensions.

Figure 6—Field test using commercial drill resistance test equipment.

Beyond simple visual measurements of the dimensions of the residual cross sections, increment cores and resistance microdrilling provides means to document the dimensions of the residual core cross section, particularly where accurate visual measurements may be difficult.

4.5 Increment Cores

One option to examine the interior cross section of wood elements is to use an increment borer to extract a wood core (Ross and Pellerin 1994). Cross sections of the cores can be used to make density-related determinations. Commercial increment borers are often marketed for use on living trees. In addition to providing a visual measure of the thickness of the residual cross section, the core can be used to examine density gradient, depth of penetration of any chemical treatment, and presence of any voids caused by decay or insect damage. As will be discussed later, sections along the core can also be subjected to chemical assay or near infrared (NIR) analysis for evidence of thermal degradation.

4.6 Drill Resistance Test

Whereas a simple hand or power drill can be used to attempt to determine the thickness of sound wood, the use of commercially available resistance microdrilling test equipment (Fig. 6) holds much promise. It is useful when qualifying the effects from decay, biodegradation, and fire damage. Past and current applications indicate that the drill-resistance test is useful when detecting the effects from decay and biodegradation (Ross and Pellerin 1994). By accurately denoting areas like this in individual members, inspectors can make a more informed decision on whether an individual member can remain in service (with or without repair), or if it should be replaced. Advantages to post-fire evaluations are that this technique may provide a direct qualifier of the residual capacity of individual members.

Modern equipment usually consists of an electric power drill, a specialized drill bit, and a software hardware interface to facilitate data collection and data analysis. Drill resistance tests are performed in much the same fashion

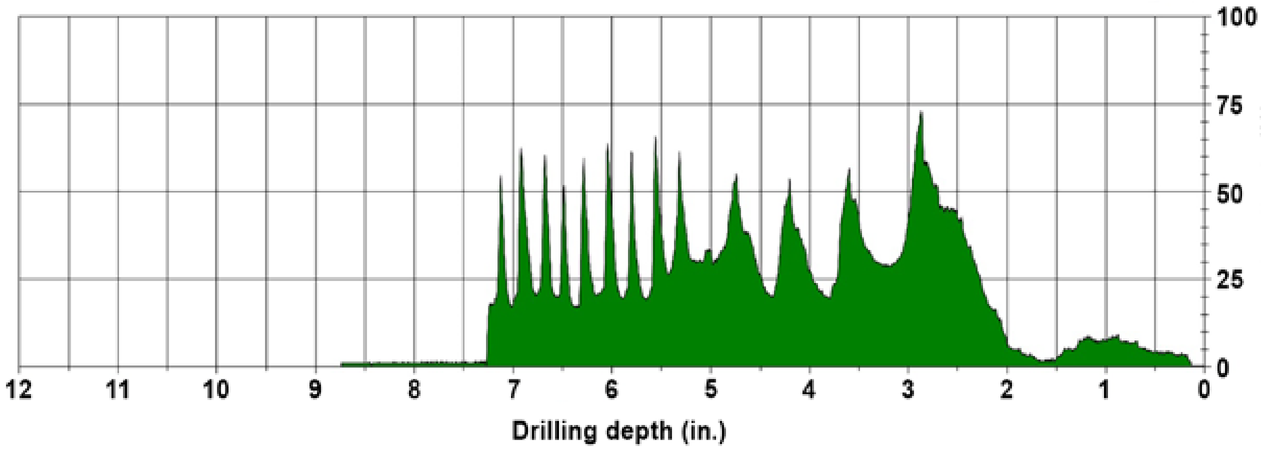


Figure 7—Residual dimensioning using data from commercial drill resistance test equipment.

as drilling a pilot hole or a series of pilot holes into wood members. The results from a drill resistance test, however, are plotted as a function of insertion-depth zones that exhibit less resistance (Fig. 7) and can be noted for areas of uncharred wood.

Tests have been traditionally classified as a quasi-nondestructive (semi-destructive) test as minor localized damage is imposed on the member of interest during testing. Drill-resistance tests are a function of drill-bit insertion depth. Generally speaking, a decrease in drill resistance or rather, an increase in the drill-penetration rate can be correlated to areas of decay, and aid in determination of the thickness of the residual shell (Emerson and others 1998).

Considering that current equipment can pick up on the changes in density from earlywood to latewood growth rings, a parallel may be drawn to that of the residual non-degraded wood. In the case of decay, the residual shell can be thought of as that portion of the cross-section that surrounds the deteriorated portion of wood; similarly, the residual shell is that portion of the cross-section that otherwise maintains load-bearing capacity. Conversely, for fire-degraded wood, the residual capacity is the residual core of the member. This type of test has also been performed when assessing fire damaged concrete as documented in the works of Felicetti (2006).

Until recently little research had been applied to microdrilling to the evaluation of fire-damaged wood. In a quick assessment of the application of technology to fire-damaged wood (White and others 2013b) resistance microdrilling measurements were made on charred beam near the mid-length of the beams in a vertical orientation (Fig. 7). The fire-charred outer portion of the glulam beam has considerably less density than sound wood and was easily detected with this NDE tool. The resulting relative density profiles were used to estimate (1) the remaining depth of uncharred wood and (2) the thickness of the thermal damaged wood layer. It was much more difficult to estimate the thickness of the thermal damaged wood layer.

5.0 Options for Evaluating Heat-Damaged Wood

The most obvious determination of damage from thermal degradation is visual. There can be various degrees of browning of the wood. Pyrolyzed wood retains the original cellular structure as observed via light microscopy or scanning electron microscopy (Zickler and others 2006). Zicherman and Williamson (1981) examined the microstructure of fire-damaged wood and found the demarcation between damaged and undamaged wood to be extremely narrow (several cell layers in thickness). Schroeder (1999) concurs with this observation of a narrow pyrolysis zone and attributed observations of a wider band to chemically borne moisture migration. For woods with resin contents, extrusion of the resin is evidence of exposure to elevated temperatures.

Charred wood is obvious. In some cases, there may be minimal or no char on the surface but smoke damage or surface resin is present to suggest some degree of fire exposure. In the reduced section approach, assumptions are made regarding the depth of the thermal degradation of the fire-exposed wood. Such assumptions are the basis for the thickness of the zero-strength layer that is added to charred wood to address the uncharred wood that has been thermally damaged and has reduced load capacity. The reality is that the fire exposure of a natural or real fire will be different than the fire exposure of the standard fire resistance test and there are considerable variations in such fire exposures. Assessment of light-frame elements and panel products that are not semi-infinite thick may also benefit from nondestructive evaluation (NDE) techniques to assess the degree of thermal damage. As previously discussed, an investigation of the fire severity and progression can provide some guidance. In this section, we discuss the potential NDE options for assessing thermal damage to wood.

While extensive work has been done on NDE techniques for assessing the residual capacity of wood affected by biological degradation, only limited work has been done on NDE for wood exposed to elevated temperatures (White and others 2013a).

One exception was research done in the late 1980s and early 1990s on NDE for the elevated temperature damage to fire-retardant-treated (FRT) plywood roof sheathings (Ross and others 1991a; Winandy and others 1991a,b.) Certain FRTs accelerated the strength reduction degradation at elevated temperatures. NDE technologies have also been used in research projects to investigate damage from production processes, such as heat treatment and to investigate the chemistry of thermal degradation.

In addressing problems of thermally degraded FRT roof plywood, the National Association of Home Builders (NAHB 1990) identifies five methods to determine the level of degradation in FRT plywood, including the following:

1. Concentrated proof loading
2. Removal of small samples for laboratory mechanical testing
3. Chemical analysis for chemical compositions of wood
4. Screw-withdrawal testing
5. Spectral analysis for end products of degradation

These methods are also potential methods for evaluating wood components for damage within the residual cross sections after a fire. As fire-damaged wood has a thermal gradient, removal of small samples for laboratory mechanical testing is probably not a practical option.

There are not standard or even commonly accepted methods for NDE of thermally degraded wood. A difficulty in evaluating wood for damage from elevated temperature exposure is the inherent variability in the properties of wood both in terms of the original strength properties and the properties being examined as an indicator of strength loss. Rather than predicting specific values of MOE and MOR, results for these techniques are likely to be better for comparing damaged members to similar members that are outside the area of elevated temperature exposure. From the comparative data, it may be possible to obtain evidence of either damage or the lack thereof.

As discussed, one detrimental outcome of the thermal degradation is the reduction in the density for wood exposed in temperature range of 200 to 350 °C. Taking into account both mass and volumetric changes, the change in density equates to 60% of original density at temperature of 340 °C (Zickler and others 2006). From 600 to 900 °C, the char density increases to about 80% of original density. Later at 1,800 °C, the density drops back to 70% of original density (Zickler and others 2006). Thus, one option for NDE evaluation is to measure density itself. As discussed previously, two methods pertaining to density are the increment core and drill resistance techniques. One problem with using weight loss or reduced density as an indicator is that significant loss in strength can occur before there is significant loss in weight (Ross and Pellerin 1994).

For this discussion, the potential NDE methods have been divided into three groups. Methods in the first group are those that measure at or near the surface and include those of hardness, microindentions, and penetration as well screw withdrawal and ultrasonic. These methods reflect loss in density near the surface and resulting effects on properties. The second group includes methods that measure chemical changes in the wood associated with loss of residual load capacity. Besides chemical composition analysis, chemical methods also include radiography. Methods in the third group are (1) near infrared, (2) stress, (3) vibration, and (4) proof loading. These methods measure the overall interior properties of the wood element. These methods either directly or indirectly provide measurements of the modulus of elasticity of the wood.

Where individual members are considered, the techniques mentioned in this report are meant to supplement, not necessarily replace, existing approaches as they pertain to individual members that otherwise expected to remain in service.

5.1 Tests for Surface Material

Surface measurements can be taken to examine for damage. As these methods are actually measuring a small portion of wood, they are particularly sensitive to the inherent variability of wood. The results will be sensitive to the grain orientation in the area of measurements as well as moisture contents. Thus, multiple measurements are required to improve the validity of the evaluation. Moisture content can be quantified in the field with a moisture meter.

The simplest test for surface degradation is the "pick test." It is a test used for detecting decay (Anderson and others 2003). In the "pick test," an ice pick or similar tool is used to poke the wood and ply out a splinter (parallel to grain) from it. With sound wood, the break will be some distance from the tool and from one end of the splinter. With degraded wood, the break in the wood will be at the location of the tool, and the brash, brittle break is perpendicular to the grain of the splinter. As with many of these tests, these subjective observations are best made by comparing results for the area of interests with results for similar wood elements in areas that are known to be undamaged. Although a simple technique, the subjective nature of the test makes it subject to misinterpretations. Two conditions that might lead to misinterpretation are water-softened wood and soft-textured wood (e.g., western red cedar) (White and Ross 2014).

The methods for the first group are those for hardness, indentation, and penetration. The methods for screw withdrawal and ultrasonic correspond to the second and third groups, respectively.

5.1.1 Hardness, Microindentations, and Penetration Tests

These techniques based on hardness or penetration may provide an indicator of thermal degradation near the surface of the wood member. Zickler and others (2006) conducted nanoindentation tests on spruce wood that had been pyrolyzed to temperatures up to 2,400 °C and cooled to room temperatures for the indentation tests. Hardness increases from 0.4 GPa at the heat-treated temperature of 220 °C to approximately 4.5 GPa as treated at 700 °C and a reduction in values occurs starting with treatment at 2000 °C. The indentation ductility index remained constant at about 0.8 until 300 °C treatment where it dropped to ~0.1 at around 500 °C treatment. Zickler and others (2006) concluded that below 400 °C treatment, decomposition leads to a minimum of the elastic modulus. Above 400 °C treatment is a gradual transition from a visco-plastic biomaterial into a brittle, glass-like carbonaceous residue with maximum values at around 800 °C treatment. Above 2,000 °C treatment, plastic deformability improves.

Hardness test is the force required to indent a material a specified amount. There are various methods in wood hardness testing including the Janka method and the one specified by Branco and others (2009). The significant difference between the two methods is the size of ball indented into the surface of the wood. The Janka method uses a ball with a radius of 11.28 mm, and the aforementioned authors use a 10-mm ball. The ball is indented into the wood to half of its diameter. The standard Janka method is a laboratory test of a specimen with specified dimensions. Portable commercial hardness devices exist but are largely intended for use on metal.

The microindentation technique is much like that of a traditional hardness test. A 1.5-mm (lengthwise), 0.5-mm diameter pin is pressed into the wood member with a very precise loading. Branco and others (2009) established a correlation between the force to indent the wood and modulus of elasticity in bending. This correlation was used in their experiment and would require more testing to be applicable in all cases (Branco and others 2009). Standard tests like this and the results can be found in Barletta (2005).

The amount of the pin that penetrates the wood has been used to generalize the mechanical properties of individual members. The penetration test, a variation of the micro-indentation test, uses a larger pin. This test measures the amount of strikes of a rebound hammer that is required to drive the large pin into the member a certain distance. The Pilodyn (KRS, Middelfart, Denmark) test uses the depth of penetration of a spring-loaded pin as a measure of degree of degradation (Ross and Pellerin 1994). Commercial Pilodyn devices are available.

Because very small localized compression is required to complete this test, the structural integrity of the member is not compromised during or after this test is completed. These tests would be run at the site of the member and could be completed relatively quickly. Given that the indentation is very small, many tests have to be taken to ensure that a proper sample of the properties of the wood are recorded. As a large portion of fire damage evaluation is measuring the residual strength for the member under the pyrolysis zone, this technique may not have the distance required to reach the zone of unaffected wood. A disadvantage is that the results may not indicate the initial strength losses because of the chemical changes in the early stages of thermal degradation.

5.1.2 Screw-Withdrawal Test

The screw-withdrawal test relates the maximum extraction load to residual flexural properties. Pilot holes are drilled into the undamaged end sections of each small specimens and a machine screw is then inserted so that it surfaces but does not extend beyond the specimen's base. The machine screw is then extracted with a digital screw extractor. The screw withdrawal loads are recorded. The screw-withdrawal test is relatively fast to perform, and no major repairs need to be made afterwards.

The screw-withdrawal test was extensively investigated as a method for evaluating FRT plywood that had been thermally degraded in roof applications (NAHB 1990; Winandy and others 1998; Ross and others 1991a; White and Ross 2014). During the 1990s, this issue was a serious problem in the United States, so the portable commercial screw-withdrawal devices were available. Screw-withdrawal FRT plywood tests use a strain gauge attached to a metal collar placed over the head of a standard wood screw inserted into the underside of a piece of roof sheathing in the field. The screw is then withdrawn from the plywood using a tensile force, and the maximum load is recorded. The strength of the FRT plywood can be compared to the strength of untreated plywood by correlations between screw withdrawal force and the breaking force (Ross and others 1991a; Winandy and others 1998). In an initial FPL study of thermally degraded FRT plywood, regression of screw withdrawal resistance and bending strength had a correlation coefficient of 0.88 (Ross and others 1991a).

Winandy and others (1998) inserted a No. 10 wood screw into various pieces of FRT plywood and untreated plywood that had been exposed to temperature greater than 130 °F for a specific time interval. They then measured the relationship between screw-withdrawal resistances versus the remaining plywood-bending strength and discovered that FRT plywood screw-withdrawal force is dependent on the thickness of plywood and various treatments methods. Based upon the method of fire-retardant treatment, the modulus of rupture was shifted either higher or lower. Models were developed for specific sub-groups, but data from different treatment/thickness could not be grouped into a single universal model to predict the bending strength.

Kukay and others (2008) and Kukay and Todd (2009) developed equations including variables for moisture content, specific gravity, moment of inertia, maximum screw withdrawal load, cross-sectional orientation with respect to the pith, treatment group (charred or uncharred), and various interactions between these variables. They concluded that the results obtained through screw-withdrawal tests are best represented when the results are limited to comparisons to similar members that have obvious degrees of residual load capacity. The following reduced model was found to be adequate based on a model comparison *F*-test procedure:

$$Y = \beta 0 + \beta\, 1\mathrm{MC} + \beta\, 2\mathrm{SG} + \beta\, 3I + \beta\, 4\mathrm{SW} + \beta\, 5O + \beta\, 6T + \beta\, 7O \bullet \mathrm{T} + \varepsilon$$

The variables include moisture content (MC), specific gravity (SG), moment of inertia (I), maximum screw withdrawal load (SW), cross-sectional orientation with respect to the pith (O), treatment group (T) (charred or uncharred), and various interactions between these variables.

Models like this are believed to be applicable under similar field conditions and are methodology-, material-, and grade-specific. Rather than predicting specific values of E_f and MOR, the results obtained through screw-withdrawal tests are best represented when the results are compared to similar members that have obvious degrees of residual load capacity. Variability of the results stem from changes in the predrilled pilot-hole size, the screw insertion depth, the screwtip to screw shank diameter, and the rate of extraction. For these reasons, care was needed when interpreting and extrapolating the results from screw-withdrawal tests. General correlations are likely to lack adequate precision to establish actual property values. The results of individual research studies that incorporate screw-withdrawal tests are generally not extrapolated. Additional work is needed to expand the models to account for the effects of a wider range of species and grades of materials as general correlations are likely to lack adequate precision to establish actual property values.

5.1.3 Ultrasonic Tests

Ultrasound technologies have been available since the 1960s. Historically used for medical applications, they have been converted into a tool used to detect density and variation in density, decay, and/or defects near the surface in a variety of materials, including wood. Ultrasonic inspection can be used as a direct test to determine whether there is an immediate density change near the surface of the member in question. Ultrasonic inspection techniques consist of high-frequency stress waves that disperse quickly over a short distance in wood. Ultrasonic inspection has predominantly been used in manufacturing to estimate product quality, but can also be used to detect common strength-reducing defects such as knots, slope of grain, and decay. MOE is computed from the sound wave measurements. Measurements for MOE are only valid for longitudinal measurements but the velocity and frequency domain signal amplitude are useful for defect detection in the transverse direction (Klinkhachorn and others 1999).

Reinprecht and Panek (2012) have shown that ultrasound technology can be used to detect differential density in wooden members. In their study, sawdust was used to model rot in wood. The ultrasound waves that propagated through the media were significantly slower in comparison to a solid member. Density, or rather, specific gravity, is considered to be an underlying factor of wood's strength; the greater the specific gravity, the greater the strength. The opposite could be said for members that show a decrease in specific gravity with regard to residual strength. Presuming that all other factors are the same, loss in specific gravity can be associated with wood members that have been exposed to elevated temperatures for an extended period of time. Klinkhachorn and others (1999) developed a portable ultrasonic device and conducted a few preliminary tests on charred yellow poplar specimens. There were little changes in the time delay or the velocity because of the charring of the two or four surfaces of the specimens but the area under Power Spectral Density (PSD) plots changed significantly as the result of the charring.

The position of the ultrasound probes can influence the results that are obtained from the ultrasonic inspection technique. The ability to obtain depthwise readings with in-situ technique like this becomes limited. Because of this limitation, some limit the technique to applications related to decay and other defects only (Emerson and others 1998).

5.2 Tests for Chemical Composition

Chemical analysis represents a different approach to NDE analysis of thermally degraded wood in that the effects being measured are the changes in the chemical composition of the wood. The changes in chemical composition are early indicators of thermal degradation and loss of residual load capacity. Small samples can be collected from the damaged wood and sent to a laboratory for chemical composition analysis. Alternatively, near infrared (NIR) technology can be used. In the investigation of options for assessing thermally degraded FRT plywood, the expense and specialization of testing were cited as disadvantages of the chemical analysis approach (NAHB 1990).

5.2.1 Chemical Analysis Tests

For initial exposures to heat, extractives and hemicelluloses are likely to be the first to be affected. Wood extractives such as fatty acids, fats, and waxes migrate to the surface of heat treated wood (Nuopponen and others 2004). Nuopponen and others (2004) used FTIR spectroscopy to examined Scots pine that had been heat treated under steam at temperatures of 100 to 240 °C. At temperatures of 100 to 180 °C, resin acids in the radial resin canals moved to the surface of the heat-treated wood and disappeared from the wood surface at higher temperatures (Nuopponen and others 2004). Although not providing a measure of strength loss, consideration of extractive content can provide indication of the level of temperature exposure.

In the thermal degradation process, the hemicelluloses are among the first of the three main wood components to be affected (LeVan and Winandy 1990). Hemicellulose measurements have potential to provide direct measure of strength loss since the xylose, galactose, and arabinose content were sensitive to the loss of mechanical properties (Winandy 2001). Arabinose was the most sensitive indicator of early strength loss. In a study on cyclic long-term temperature exposure at 82 °C, 30% RH, only arabinose showed a consistent reduction with increased durations of heating (Green and Evans 2008). Potential additional parameters that include holocellulose and alpha cellulose content and degree of polymerization has also been considered (Ross and others 1991a).

Reductions in pH (increased acidity) of wood are a potential indicator of thermal degradation and strength loss. Lebow and Winandy (1999) investigated pH of wood as a technique for evaluating thermally degraded FRT plywood. To determine pH of plywood, Lebow and Winandy (1999) drilled a 1/2-in.-diameter hole to a depth of 3/8 in. and collected the wood shavings to test the wood's pH. After collecting the sample and mixing the sample cuttings with deionized water, they were able to measure the pH of the samples within 20 min. Although some correlation ($R^2 = 0.74$) was observed between pH and strength loss in the plywood chemically treated with fire-retardant chemicals, this method was less sensitive to degradation in untreated plywood. Chemical analysis tests can be comparatively more expensive since most of the work has to be conducted in labs.

5.2.2 Near-Infrared Tests

The near-infrared (NIR) technology works by analyzing the interactions between materials and electromagnetic radiation. The wavelength and line speed are both variables predicted in models developed for detection of decay and other wood processing issues. Spectral analysis tests use infrared radiation to identify end products of chemical processes. A trained organic chemist is required to interpret the data gained from a spectrophotometer. More information on spectral analysis is provided in the American Plywood Association report, *Fire-Retardant-Treated Plywood Roof Sheathing: In-Situ Testing* (APA 1989).

The NIR technology can be used to accurately measure the chemical composition, mechanical properties, and a select few anatomical properties of wood (Brashaw and others 2009). Esteves and Pereira (2008) have investigated NIR spectroscopy to evaluate properties of heat-treated wood. Additional information on this technique can also be found in the technical review by So and others (2004), *Near Infrared Spectroscopy in the Forest Products Industry*. Particular advancements have been made for the use of NIR technologies to detect the presence of in-situ member decay with the majority of studies being done specifically on brown-rot fungi (Nicholas and Crawford 2003). Other researchers have used this technique to predict surface moisture distribution as well as wet pockets. It is thought that wet pockets are associated with bacterial activity (Watanabe and others 2010). The ability to detect local variants as well as general material distributive properties like moisture content make this technique appealing to applications beyond quality control. N is considered to be accurate and fast. More research is needed in the use of NIR technology to detect the early stages of degradation resulting in reduced load capacity. Considering its limitations, NIR is still a very attractive technology because it is lightweight and portable (Nicholas and Crawford 2003). These characteristics provide compelling reasons to further investigate its suitability for post-fire evaluation.

5.3 Tests for Whole Elements

For the interior characteristics of a wood element, the following measuring techniques are used. As useful tools for NDE assessments of wood decay, they are extended to thermally degraded wood elements. These include radiography, stress-wave, vibration, and proof loading. These techniques typically indicate changes in density or modulus of elasticity (MOE). Correlations are used to provide predictions for the modulus of rupture (MOR). Thus, the validities of the methods for predicting the residual load capacity based on MOE are limited by changes in the relationships between MOE and MOR caused by elevated temperature exposure. Stress-wave techniques are widely used to assess damage from decay (White and Ross 2014). Proof loading techniques include those designed to assess the entire bridge construction. In situations where there is a progression of damaged members, destructive testing of selected members may be an option.

5.3.1 Radiography Tests

Radiography tests, also known as X-ray tests are carried out using an X-ray source and detector to establish the internal density distribution of wood members (Niemz 2012). The technique typically involves either two-dimensional or three-dimensional imagery and is dependent on the equipment and access points. This technique, like many others, can quantify the extent of decay as it is correlated with density losses. Since density losses from thermal degradation are also correlated to losses in flexural properties, radiography technique is applicable to post-fire investigations. In particular, radiography can detect a redistribution of density in cases where bulk density does not change because of liquid solidification or volatile condensation within the wood bulk. Indeed, radiography is commonly used for detecting internal voids in wood mostly during manufacturing (Ross and others 2005b).

It has also been used to determine variants such as knots as well as the internal decay of individual wood members in laboratory settings (Niemz 2012). A technique such as this could currently apply to in-situ depictions of the pyrolysis zone of individual wood members.

X-ray technology is able to pick up the presence of knots or defects based on density changes in the members. This would have the potential to allow inferences to be made about the residual strength of individual members beyond simply reducing the cross-section and visual inspections.

Radiography could be used in the field for in-situ testing if further advancements are made in making the equipment more mobile. High costs of equipment, inspector safety factors, portability, member access, and expert interpretation of radiograph results are chiefly responsible for keeping X-ray techniques from being more widely used as a field-evaluation technique (Ross and others 2005b). As a result, the technique may not have advanced far enough at this point.

5.3.2 Stress-Wave Tests

Stress-wave evaluation has been successful in detecting decay in structures (White and Ross 2014; Brashaw and others 2004). Stress-wave evaluation is based on sound-wave physics. Stress waves (also referred to as sound waves) are generated from an impact on the surface of a member that is under investigation. Impact sound waves are measured as a function of their time to travel from one side of the member to the other side. Measurements are made so any and all regions of decay are captured (White and Ross 2014). A guide specific to stress-wave evaluation is *Stress-Wave Timing Nondestructive Evaluation Tools for Inspecting Historic Structures* by Ross and others (2000). The methodology is also extensively discussed in the *Wood and Timber Condition Assessment Manual* (White and Ross 2014). Sound waves have a lower velocity through decayed wood than through non-decayed wood. Knowing the member's cross section dimensions, an inspector can successfully locate areas of decay by making a series of measurements at incremental locations along the member (Emerson and others 1998). Stress-wave evaluation tests have more recently been used as indicators of residual wood stiffness for individual members.

The handheld device used to perform the stress-wave analysis is portable, which makes it a viable option for essentially any location. The test is non-invasive and very easy to complete in a field setting. The test is efficient and can run successive times in a timely manner. The moisture content of the member must be known before the analysis can be completed, which can also be found using a portable moisture content meter. Because this test relies on stress waves quickly propagating through the member, it is essential that the exact distance between the first and second probes is known. Any deviation in the measurements between readings would make the analysis highly unreliable because of the high-speed travel of the stress wave.

Impact sound waves are measured as a function of their time to travel from one side of the member to the other side. Dynamic modulus of elasticity or apparent MOE (Pa) is considered equal to the wave speed (m/s) squared times gross density (kg/m^3) (Ross and Pellerin 1994). As with some other NDE methods, stress wave is a better predictor of MOE than it is for MOR. Elevated temperature exposure changes the correlation between the MOE and the modulus of rupture (MOR). Thus, a disadvantage for NDE based on determining elastic properties is that the property of most interest in terms of safety (i.e., strength properties) are more sensitive to elevated temperature exposure than the properties being measured.

Garcia and others (2012) used stress wave for nondestructive determination of the MOE of wood before and after heat treatment. The dynamic modulus of elasticity decreased by about 13% in the most severe treatment (230 °C for 4 h) but not for the milder treatments. A strong relationship was found between energy absorption as measured by stress-wave techniques and residual strength of degraded FRT wood but the speed of sound was insensitive to the degradation (Ross and Pellerin 1994).

As part of our study on evaluating fire-damaged components of historic covered bridges, four 127-mm by 305-mm beams were subjected to a fire exposure in a fire test furnace that resulted in a narrow char layer but a deep layer of elevated temperatures that penetrated beyond the base of the char layer by nearly 25 mm. As a result of the fire exposure, average measured values indicate a 54% reduction in the maximum recorded load, a 46% reduction in strength (MOR) based on the residual cross-sectional area, and a corresponding 20% reduction in stiffness (MOE). These results are based on a standard three-point bending test with an increasing load applied at the lengthwise center of each beam. Sound wave measurements were taken near the top and bottom of each beam. The values and corresponding results were repeatable; resulting comparisons of the transmission rates indicated a reduction from the fire exposure. But the reductions in the times for fire exposure were not greater than the differences between the fastest and slowest times for the different specimens prior to fire exposure.

5.3.3 Vibration Tests

Vibration tests are conducted by placing an electric motor with a weighted wheel (attached) into motion at the midspan of a bridge or another readily identifiable location. This causes the bridge to resonate. Piezoelectric accelerometers are then placed and are used to measure the response to the vibration (Wang and others 2005). In so doing, a number of modes of vibration can then be recorded. These recordings are used to assimilate computer-generated models that are calibrated to individual bridges.

Typically, the nondestructive forced vibration method has been used to determine the modulus of elasticity of structural members and structural systems. When a number of individual members are targeted, the stiffness of a member or rather a decrease in stiffness may be used for evaluation purposes. An empirical correlation between stiffness and strength can be used to determine the soundness of a particular wood member (Emerson and others 1998).

Vibration tests continue to be employed when establishing the in-situ dynamic behavior of pre-stressed, pre-cast concrete bridges, steel bridges, and composite bridges. Depending on the goals, it is possible for vibration tests to be conducted on individual members after being removed from service.

The advantage of vibration tests such as this are that with further research it could be used to quickly determine the stiffness of a timber bridge as well (Wang and others 2005).

Whereas forced vibration tests can be used to assess the dynamic behavior, more research is needed in the area of post-fire assessment using current methods (Wang and others 2005). Also, the weight of the structure needs to be accurately determined to predict bridge stiffness based on the beam theory model (Wang and others 2005). Estimating the weight of a bridge structure is difficult and may be construed as unreliable.

Dynamic testing and diagnostic load testing are two of the most common methods used for assessing the overall condition of a bridge. The response to the dynamic system load can be evaluated against an analytical model of the bridge or, if available, a previous response record. Vibration analysis techniques have been demonstrated on simple-span members but are more complex when full-scale structures are under investigation because of the structures' many modes of vibration. Each mode of vibration must be investigated to determine the structural characteristics of the bridge (Emerson and others 1998).

5.3.4 Proof Loading Tests

Mechanical testing is a more direct approach to evaluating the effect of elevated temperatures on mechanical properties. In terms of NDE, this normally involves some sort of proof loading to determine the MOE. A bending proof load test was one of the options developed for the assessment of FRT plywood roof sheathing (NAHB 1990; Ross and Pellerin 1994). As part of the study on fire-damaged components of historic covered bridges, a proof loading method specific to field application to fire-damaged wood components was developed (Kukay and others 2013).

As part of this project on NDE of fire-damaged wood, new NDE proof-load techniques were developed (Appendix).

Dynamic testing and diagnostic load testing are two of the most common methods used for assessing the overall condition of a bridge. To directly determine a structure's load-carrying capacity, one approach requires that large proof loads be applied to the bridge. These large proof loads usually involve rolling heavy vehicles onto the bridge while measuring strain or deflections of various members. The measured strain data is then analyzed to establish the strength of the members. Although smaller static loads have also been used for this purpose (Emerson and others 1998), determining the maximum load-carrying capacity of the structure is generally not as reliable as when larger proof loads are used. This requires an assumption (or sampling) for the material properties to be reliable.

5.3.5 Destructive Mechanical Tests

Mechanical testing options include destructive methods such as removal of selected members for testing or removal of small samples for laboratory mechanical testing. Destructive testing will provide a direct measurement of modulus of rupture while proof-loading techniques only provide direct measurements of modulus of elasticity. The removal of small samples for laboratory mechanical testing is well suited for both the field and the laboratory. The span-to-depth ratios, based on uncharred dimensions, dictate the amount of material that must be removed for mechanical testing. A minimum span-to-depth ratio of 14:1 is common for small clear samples of sawn lumber products and is specified in the standard ASTM D 134 (ASTM 1994). The standard ASTM D 198 (ASTM 2009b) specifies a span-to-depth ratio between 17 and 21 that is typically used for bending tests of lumber in structural sizes, while 18:1 frequently serves as the international standard for wood products specified in standard ASTM D 3043 (ASTM 2000). Depth-wise reductions from charring would otherwise increase the span-to-depth ratios accordingly.

In accordance with ASTM D 143-94 (ASTM 1994), a single-point loading scheme can be applied at mid-span for all simply supported small members. Displacement measurements are taken during the test at the lengthwise midpoint, where the load is applied continuously to failure at an acceptable rate of motion. Flexural properties are then determined by analyzing the data that are collected from such tests.

Bending load strength tests are based on Test Method A that is specified in the standard ASTM D 3043 (ASTM 2000). Either 1- or 2-in. pieces are used for panel thicknesses less than 1/4 in. Two-inch pieces are used for greater thicknesses. The length of each test specimen depends upon the orientation of the laminations to the span. Perpendicular lamination test specimens shall not have a length of less than 24 times the depth plus 2 in. Parallel laminations test specimens shall not have a length of less than 48 times the depth plus 2 in. It is recommended to take at least three samples per panel.

Test samples shall be tested with a continuously prescribed loading rate—without shock—until failure to determine modulus of rupture. Measurements of deflection can be taken simultaneously to calculate modulus of elasticity, but one could be better served to use (Test Method B that is specified in the standard ASTM D 3043 (ASTM 2000)) instead for modulus of elasticity testing.

Bending load tests are fairly easy to perform with the correct equipment. Low costs are associated with the bending load tests, but the cost to repair the bridge must also be considered along with the shipping costs.

“The strength properties of various species are determined through standardized tests of small, clear (defect free) specimens at various moisture conditions. Large pieces, containing defects typical of standard grades of lumber, are also tested to develop strength data” and correlation between small clear specimen and standard structural sizes typically assessed from test method ASTM D 198 (ASTM 2009b). As addressed in the American Society for Testing and Materials, ASTM D 143-94 (ASTM 1994) Standard Test Methods for Small Clear Specimens of Timber, “The need to classify wood species by evaluating the physical and mechanical properties of small clear specimens has always existed”.

This technique presents one of five post-fire assessment options that were originally proposed by NAHB for directly determining the residual load-bearing capacity of thermally degraded plywood (Winandy and others 1998) across a broader range of materials. Similar approaches have been adopted in the works of LeVan and Winandy 1990 in their study on the effects of fire-retardant chemicals on bending properties of wood at elevated temps and Glos and Henrici (1991) when they assessed the bending strength and bending stiffness of structural timbers at temperatures up to 150 °C.

Advantages to post-fire evaluations that make use of this approach are such that much of parent material remains. This allows for the fire-damaged member to be sistered onto, and may otherwise eliminate the need for a temporary support mechanism, depending on the severity. This presumes that the primary connections are otherwise determined to be adequate. In instances where a fire is contained, the residual flexural values of like materials that exhibit varying degrees of degradation can be directly compared to one another with this technique.

Disadvantages are such that the removal of a smaller member from an existing member requires repair, if it is otherwise expected to remain in service.

Figure 8—Bearing style connection, Red Covered Bridge.

6.0 Other Post-Fire Assessments

6.1 Fire-Damaged Connections

During a fire, heat can also be transferred into the wood through exposed metal connections (Fuller and others 1992). Structural integrity of the covered bridge’s truss is heavily dependent on metal connectors. Therefore, all connections require a detailed post-fire inspection to determine their load-bearing capacity. Connection integrity is dependent on the quality of the metal and metal surface contact area. Chemical damage from corrosive effects of fire residues that may be present must be accounted for (Williamson 1982a,b). Figure 8 illustrates a bearing style connection.

Because metal can conduct heat into the interior of the wood component, assessment of fire damage around connections may be less feasible for visual inspection. Radiography tests, or also known as X-ray tests, may be advantageous for such assessment.

Inspections of the connections should also include examinations for corrosion of the metal fasteners. The surrounding wood should be examined for splits, decay, and insect damage. The tightness of the connections should be assessed.

6.2 Smoke Damage

Besides the stated effects of fire on wood (charring, reduced load-bearing capacity, and fire-damaged connections), smoke and other fire residuals will also be present; however, these do not affect the wood members’ load capacity (King 2002). Nevertheless, sealing and eliminating fire odors is still important. Paints, sealers, and other finishes are available to seal the odors in the member material. Before the application of these sealers, the wood member should be thoroughly cleaned and exhibit no traces of the previous fire damage. This is true even for members that will be hidden from sight. A post-fire restored historic covered bridge should not only look as it did pre-fire, it must also exhibit no lingering fire damage odor. The National Institute for Disaster Restoration (NIDR) has guidelines for fire and smoke repair. These guidelines highlight methods for removal of fire residues, removing fire odors, neutralizing acid residues, and the use of sealing and encapsulation.

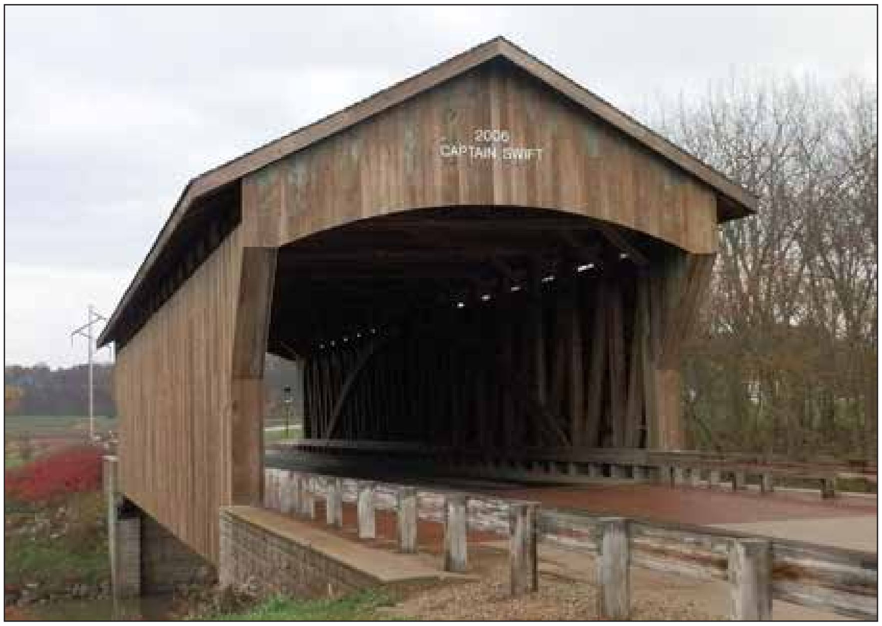


Figure 9—Captain Swift Bridge near Princeton, Illinois.

7.0 Post-Fire Bridge Repair

7.1 Historic Covered Bridge Rehabilitation Guidelines

Wacker and Duwadi (2008) state that when undergoing historic covered bridge rehabilitation, strengthening methods should conform to the Secretary of Interior's guidelines for preservation of historic structures. In that regard, any rehabilitation necessary to increase or improve the strength of the structure should be done in a way that the historical external appearance is preserved (Figs. 9 and 10).

According to the Vermont Historic Bridge Program (2014), a historic covered bridge restoration needs to ensure the maximum load capacity associated with the anticipated use of the bridge is maintained in accordance with a prescribed set of preservation guidelines. This methodology will ensure that rehabilitated historic covered bridges will maintain their historic status and charm, but will also meet traffic and environmental demands.

The Vermont Historic Bridge Program (2014) issued the following set of rehabilitation guidelines in order of priority, with the highest priority listed first and decreasing thereafter:

1. Retain all existing historic materials that have not deteriorated beyond the point of repair. Where existing rot or other damage is not severe enough to require replacement, the materials should be repaired rather than replaced. This treatment should be applied to each member individually, and deterioration of a large number of bridge elements should never justify the replacement of any single member capable of being replaced.

Figure 10—Underside of bridge deck, Red Covered Bridge.

2. Replacement of existing materials in kind, meaning identical species, quality, and dimension to the maximum extent feasible, or restoration of original materials and design. Preferably, material origins should be from the same region of the specimen. If a different species or quality is considered and/or materials from the same region of the country are not available, substitutions may be considered with justification.
3. Application of historic methods of strengthening such as the application of sister lattices in Town lattice truss bridges.
4. Introduction of glued-laminated beams as a co-functional, reversible structural system. The beams must be designed to work in conjunction with the historic structural system to achieve required load capacity, and the historic structural system must be restored according the Rehabilitation Guidelines 1, 2, and 3 above.

5. Replacements of limited pieces of existing load-bearing members with materials identical in species, quality, and origin to the maximum extent feasible. Dimensions may be larger but must not cause alterations to the dimensions of any other important bridge components. For example, increasing the depth of bottom cords of Town lattice trusses may increase capacity without requiring alteration to either overall bridge dimension or the design of the floor system.
6. Replacement of existing load-bearing members with glued-laminated members (beams or chords) of identical dimension.
7. Reinforcement of load-bearing members with non-obtrusive modern materials such as steel rods or plates, glass fiber, carbon plates, or other material. Placing these modern materials hidden from view, to maintain the historic integrity of the bridge, whenever possible.
8. Protection of load-bearing members by the introduction of steel beams that provide a safety-net for the bridge. The redundant structure must allow the existing timber frame to continue functioning, and a minimum clearance between steel beams and floor beams should be designed. The purpose of this treatment is to protect the historic bridge in case of structural failure, not to increase carrying capacity.
9. Replacement of load-bearing members with, in order of priority: (a) timber of larger dimension but otherwise identical in terms of species and quality; or (b) timber of larger dimension and different species.
10. Replacement of existing load-bearing members with modern materials.

These guidelines may help post-fire investigators, bridge engineers, and bridge owners to develop a rehabilitation strategy that incorporates appropriate guidelines and requirements. With a consistent approach, case studies can be conducted to further assist an engineer with any problems or questions that occur during their rehabilitation efforts.

7.2 Historically Accurate Reinforcement and Rehabilitation

Conventional strengthening/reinforcing methods for timber structures may be accomplished through the use of member augmentation such as steel plates or bars, aluminum plates, or simply timber patches. On structures where it is impractical or impossible to replace a member, member augmentation may be a method that is capable of replenishing the load capacity of a damaged member. Sistering and splicing are the two most commonly used methods of member augmentation. When a member's existing capacity is insufficient, sistering a member by adding reinforcing pieces either over a portion or even the entire member is the most popular choice. Sistering may also be used to repair members that are exposed to weathering or exhibit significant changes in moisture content as compared to splicing (Ritter 1992).

The sistered members can be attached to the damaged member in a number of ways. Two approaches include either adhesive bonding or steel plates with bolts. Adhesive bonding can have several advantages over steel plates: the stress can be more uniformly dispersed throughout the member, little weight is added to the structure, little to no damage occurs to the adherents, and it is generally less expensive. Of all adhesives for in-situ repair, epoxies have demonstrated minimal shrinkage during curing, remain dimensionally stable after hardening, and maintain a high resistance to water. After a surface has been cleaned, the epoxy augmentation process should begin immediately. A mechanically sound adhesion can be produced with relatively little force being applied to the scabbed member during curing (Custodio and others 2009).

7.3 Modern Reinforcement and Rehabilitation

Conventional repair methods may increase dead loads, installation costs, and transportation expenses because heavier reinforcement materials are used. Modern reinforcement methods using fiber reinforced polymers (FRP) have lower strength-to-weight and stiffness ratios. This allows FRPs to be used without a significant increase to the dead load of a structure.

FRP composites are usually bonded to the higher stress zone, more commonly known as the tension side of timber beams, which increases their load-carrying capabilities while decreasing deflection. FRP composites bonded on the tension face are either adhered to the surface as a sheet or are inserted into a cut narrow groove and secured into place with a bonding agent. Near-surface mounting of FRP bars placed in cut grooves is a new technology designed to increase the load-carrying capacity and energy absorption capacity when compared to unstrengthened counterparts (Kim and Harries 2010). Installing the bars in a cut groove on the tension side of the members means that repairs will typically be hidden from view. This is very important to historic covered bridges that need to maintain their historic integrity and charm.

Glass-fiber reinforced plastic (GFRP) bars are generally less expensive than other FRP bars, carbon fiber and aramid fiber, but still significantly improving a member's load-carrying capacity and stiffness (Raftery and others 2009). When the Tourand Creek Bridge was strengthened with GFRP bars, it cost less than 15% of the estimated cost to replace the entire bridge ($800,000) (ISIS Canada 2000).

GFRP bars have a high strength-to-weight ratio and are non-corrosive. Both of these attributes provide engineers with a secure long-term option to reinforce in-situ members of historic timber bridges.(Gentile and others 1999) installed GFRP bars into grooves cut into the tension side of select structural Douglas-fir stringers. The GFRP bars were bonded in place with an epoxy resin and the reinforced stringers were tested against similar non-reinforced stringers. Results showed a strength increase of 25% to 50% for the GFRP members. Ultimately, the study concluded that GFRP bars are a viable treatment method to increase the flexural strength of sawn timber beams.

The use of epoxy adhesives to bond the FRP bars into place has increased more recently based on their gap-filling qualities and low clamping pressures (Raftery and others 2009). To prepare the wood surface prior to bonding a FRP, it must be planed where bonding is going to occur; adequate bonding has to occur; degreasing of the surface has to be done; and the surface also has to be free of loose material (DOT 2007).

The U.S. Department of Transportation conducted a large-scale bending test with GFRP plates that were bonded to the tension face of several timber members. This research concluded that the strength and stiffness of timber members was vastly improved (DOT 2007).

Aramid fibers are another FRPs option that consists of a synthetic organic polymer fiber. Yahyaei-Moayyed and Taheri (2011) found that these fibers exhibit the highest tensile strength-to-weight ratio of all reinforcing fibers and are mainly used in industry, military, and aerospace applications. They also demonstrated that aramid fibers can successfully be used to increase the flexural strengthening of southern yellow pine and Douglas-fir species by 74% and 34%, respectively.

Another method of strengthening wood beams is to bond steel cords to the beam's tension face. Borri and Corradi (2011) demonstrated the effectiveness of bonding high-strength steel filaments to the tension face of wood beams by simply placing steel filaments on the wood surface and covering them with a thin layer of epoxy.

For all cases of FRP bar installed into timber, the bond shear-slip between the two materials has a minor effect on the ultimate loading capacity of the members. During analysis, a continuous bond between the two materials was observed in most of the tested cases (Valipour and Crews 2011).

7.4 Member Replacement

After a post-fire analysis of the damage of the structure is complete and residual properties of the materials are determined, it may be necessary to decide whether to replace or to reinforce and rehabilitate a damaged member. More specifically, structural members that do not meet current load requirements and applicable codes should be repaired, reinforced, or replaced. Selected members should be analyzed based on which repair method is the most feasible, while maintaining historic covered bridge repair principles and guidelines. A list of these guidelines is presented in the next section. Abrasive blasting may be employed to remove char and member discoloration. Various media types can be used including ground corn cob, sand, and baking soda. The final wood surface should be treated for residual odors, and protective sealers should be applied (White and Ross 2014). If the member is damaged beyond repair, replacement is typically the final option. For historic covered bridges, replacement should be identical down to its cross section, species, and harvest location to retain the bridge's historical integrity. When this option is not available, glued-laminated members may be used in accordance with the applicable guidelines (see APA 2009). Rodgers and others (1997) showed that replacing existing solid-sawn stringers that had accumulated more than 50 years of service with new glued-laminated stringers reduced the overall deflection of the structure under load and made its load-bearing response more uniform.

8.0 Design for Fire Prevention and Control

Minimizing fire damage to wood members is of great importance to the rehabilitation, repair, and restoration of historic covered bridges. Post-fire considerations should include an assessment of actions that can be taken to reduce the likelihood and severity of future damaging fires. With their construction and often isolated locations, covered bridges are a challenge for fire protection engineers. Risk-based strategies associated with fire containment and suppression through fire-retardant treatments are intended to prevent a bridge from collapsing in the event of a fire, but does not necessarily prevent a fire from occurring (Barbaresi and others 2012). Approaches like this should also be considered to be part of the design. Site-specific conditions and proximity to resources necessitate a combination of effective techniques into the design and/or retrofit. Design considerations related to fire prevention and extinguishment programs should incorporate serviceability, preservation, and aesthetics.

Although arson is a major cause of fire ignition to covered bridges, the possibility of naturally occurring and unintentional fires near a covered bridge is possible near the wildland–urban interface. Diligent removal of ignitable materials can curtail a fire's preferential pathway. Thus, one obvious consideration should be the improved maintenance to reduce contents or surrounding vegetation that can add to fire intensity or ease of ignition.

Whereas it is beneficial to remove surrounding vegetation or other combustible items that provide a path for a wildfire to spread to the covered bridge, it needs to be recognized that a major avenue for the spread of wildfires to structures is the firebrands and embers lofting considerable distances from the wildfire itself.

In situations where elements of the bridges are being replaced, consideration of the relative fire performance of different building materials should be a factor in the selection of materials. The flame spread behavior of wood is dependent on the species. Chemical treatments are available to further reduce the rate of flames spread when the wood is ignited. Larger structural elements are less conducive for ignition and sustained flaming. Selected installation of panel products such as gypsum board can be used to reduce the areas of potential fire spread into smaller compartments.

Historic covered timber bridges are often in remote locations. Fire on these structures will likely be reported after the fire is fully developed or even after the decay period of a fire. Remote security and fire detection monitoring systems for these historic covered timber bridges can be installed to reduce response times, preferably to where only a small or no fire damage occurs. Design considerations that accommodate the retrofit of fire notification and extinguishment systems should include routine operation and maintenance inspections; both of which can be reduced through tamper-resistant installations among other factors. The isolated locations of many historic covered bridges make some options for improved security topics of debatable benefits in terms of vandalism. Improved lightening generally will deter vandalism but in remote locations it may facilitate the unauthorized use of the covered bridge as a gathering place.

8.1 Flame Spread Index of Wood

Treatments are available to reduce the flammability of lumber and other wood products. In the building codes, regulations pertaining to surface flammability of interior finish are based on the surface burning characteristics test described in ASTM E 84 (ASTM 2009a). This test is widely known as the 25-ft tunnel test. In the test, observations of flame travel along the specimen are used to calculate a dimensionless flame spread index (FSI). The standard ASTM E 84 test has a duration of 10 minutes or less. Historically, the index was equivalent to zero for asbestos and 100 for red oak flooring. Measurements of the obscuration of a light beam are used to calculate a dimensionless smoke development index (SDI) which has a value of 100 when red oak flooring is tested. In the building codes, the provisions pertaining to the fire ratings of interior finish are in terms of the three classes of A, B, and C. Class A is the most restrictive and requires a FSI of 25 or less. For wood products, Class A classification typically requires a fire-retardant treatment. Class B requires a FSI of 75 or less (i.e., 26 to 75). For lumber, some domestic softwood species and some high-density imported hardwood species have reported FSI in the Class B range. Class C is a FSI of 76 to 200. Most wood panel products, lumber of some domestic softwood species, and the lumber of the domestic hardwood species have reported FSI in the Class C range. Lumber is typically tested in the E84 test with a nominal thickness of 1 in. (actual thickness of 19 mm). The FSI may increase with reductions in the thickness of the test specimen. Some wood panel products have been reported with FSI that exceed the upper limit of 200 for Class C. The requirement for the SDI is typically 450 for all three classifications. Available SDI data for wood products are for SDI less than this 450 requirement. The American Wood Council maintains a publication on their web site (www.awc.org) with listings of FSI data for domestic wood species. Only limited public data is available for the imported wood species. Further studies are needed to determine if the test environment of ASTM E 84 is applicable to potential fires in a covered bridge. It may be possible that Class B rated lumber may result in limited flame spread behavior for the typical fire scenarios that would result in minimal fire damage, with attendant low restoration costs. In any case, the test severity of ASTM E 84 usually ensures that the Class A rated lumber will provide the adequate passive protection from fire progression in realistic fire scenarios.

8.2 Pressure-Impregnated Fire-Retardant-Treated Wood

Wood’s fire performance can be improved with fire-retardant treatments. Commercial fire-retardant treatments for wood are intended to reduce the flame spread index as tested in accordance with ASTM E 84 (ASTM 2009a). In addition to slowing the spread of flames, the treatment will likely also delay ignition and reduce the heat release rate. The main treatment options are pressure impregnated treatment or surface application as a coating. Pressure treatments use pressure to impregnate the chemical solutions into wood members. This process is similar to the method used to impregnate chemical preservatives into wood members.

For building code purposes, specifications for FRTW go beyond the requirements for a Class A flame spread index of 25 or less. To qualify for the FRTW classification, the product must pass a modification of the standard E84 test in which the test is extended by 20 minutes to a total duration of 30 minutes. The test is often referred to as the 30-minute E 84 test and more recently has been described in ASTM E 2768 (ASTM 2013a). Along with these added fire performance requirements, the requirements in the building codes for FRTW also require the treatment be a pressure-impregnated treatment and meet other performance requirements pertaining to hygroscopity and strength loss. Treated wood products that meet the requirements for FRTW can be used in applications where FRTW is a prescribed alternative material in the building code. The requirements for FRTW also include requirements for pre-weathering of the test specimen prior to the fire test when the application would subject the product to exterior weather exposures. Thus, specifications for FRTW need to specify whether the application is interior or exterior. In terms of the existing market, available FRTW products are limited to softwood lumber and softwood structural plywood. Available listings of species for

FRTW lumber are devoid of the hardwood species. FRTW products are required to have appropriate labels with information about the treatment.

The compositions of the commercial fire-retardant treatments are largely proprietary. Truax and others (1935) concluded that using different combinations of chemicals was the most effective way to provide fire-retardant treatments. Traditional treatments include the use of inorganic salts, such as boric acid, borax, diammonium phosphate, monoammonium phosphate, ammonium sulfate, and zinc chloride. Because the inorganic salts are subject to leaching, such treatments are limited to interior applications. Organic phosphate salts have also been used as a fire retardant on wood. The two common elements in FR treatments for wood are phosphorus and boron. While pressure-impregnated treatments are marketed as either a wood preservative or as a fire retardant, FR treatments incorporating boron will be likely to have improved resistance to decay and insect damage, as boron is also used as a wood preservative. Boron wood preservative treatments are likely to have little fire retardancy because the treatment levels for fire retardancy are much greater than those used for wood preservation.

As the result of high treatment levels, FR treatments are known to adversely impact the strength properties of the wood. In the structural design process, the reduced structural capacity is accounted for prescriptively through reduction factor for the allowable stresses. The reduction factors need to be obtained from the treater or the FR manufacturer. The specifications for FRTW also include test requirements intended to address potential for thermal degradation of the FRT product in roof or other applications involving elevated temperature exposures.

FR treatments for wood are not marketed to improve the fire resistance of a wood member as evaluated in the ASTM E 119 test (ASTM 2010). The effect of the FR treatment on the charring rate is mixed. Many FR treatments lower the temperature required to initiate the thermal degradation of the wood. The result of this temperature shift is a reduction in the volatile gases and an increase in the residual char.

There are two published specifications for FRTW, namely NFPA 703: Standard for fire-retardant impregnated wood and fire-retardant coatings for building materials and the applicable sections of the AWPA standard (NFPA 2015). Manufacturers of FRTW as well as FR coatings may also have evaluation reports issued by the ICC Evaluation Service.

In applications where the building code provisions or other specifications are limited to just the requirements for the Class A or Class B flame spread index, the requirement for the FR treatment is limited to the FSI provision. In such cases, the use of a fire-retardant coating may be acceptable to the code official. FR composite products such as fiberboard and particleboard are likely to be marketed for just the Class A FSI provision.

8.3 Fire Protective Coatings

For use in historic covered bridges, the FR coating is advantageous as a field application of the FR to the existing members. Pressure impregnation is obviously not an option for field application. Pressure impregnation would require removal of the wood for treatment or the use of new FRTW lumber. FR coatings can be sorted into two categories, intumescent and non-intumescent. Intumescent coatings are foam surfactants that expand when exposed to heat. When the foam expands, the volume increases while the density decreases and this provides a thermally insulative protective coating for the wood members. Specifications for FR coatings are incorporated in NFPA 703. FR coatings are marketed as meeting FSI of either Class A, B, or C. The classification will depend on the specifics of the wood substrate and the quantity of the application. As discussed previously, untreated lumber will typically meet either Class B or C depending on the species. Review of fire test documentation for a FR coating should take into consideration the likely classification of the uncoated substrate. As with any field application, quality control measures for the field application of the FR coating should be reviewed.

Similar to pressure impregnated FR treatments, FR coatings for wood for the most part are not marketed to stop the wood from charring. They are intended to reduce the spread of the initial flame. Fire protective coatings for structural steel are intended to improve their performance in the fire resistance test, ASTM E 119 (ASTM 2010). Research has shown that some fire protective coatings can in fact improve the fire resistance of wood members (White 1984, 1986; Richardson and Cornelissen 1984). These coatings are typically the intumescent coatings or thick coatings that can provide a degree of thermal insulation to the wood surface. More recently, the non-intumescent coatings tend to have an enhanced ability to absorb heat, such being hydrated with water, of which the brand "pyrotite" is one example. This FR coating is marketed as a 15-minute thermal barrier and as a 20-minute rating for use in assemblies.

8.4 Panel Products

Gypsum board is the most common panel product used to improve the fire resistance (as measured by the ASTM E 119 test (ASTM 2010) of a wood member or assembly. Whereas regular gypsum board will provide improved fire resistance, it does not have fire resistance performance requirement. Fire rated gypsum board is known as Type X gypsum. Per its definition in ASTM C 1396/C 1396M (ASTM 2014), Type X designates gypsum board that provides not less than 1-hour fire resistance rating for boards 5/8 in. thick, or 45 minutes fire-resistance rating for boards 1/2 in. thick, as applied on each side of a typical wood frame wall assembly of 2 by 4 studs spaced at 16 in. on center. There is also a higher grade of fire-rated gypsum board that is often referred to as Type C. Gypsum board provides

Figure 11—Examples of fire deterrent and notification techniques used on the Red Covered Bridge, circa 2012.

substantial protection because of its high content of bound water. In terms of performance, the differences between regular gypsum board and the different grades of fire-rated gypsum boards is their ability to maintain their own integrity after sustained fire exposure. During sustained fire exposure, cracks are formed in the gypsum board.

8.5 Roof Coverings

The various types of roof coverings are rated for their relative fire performance. The series of fire tests that are used to classify roof coverings are specified in the standard ASTM E 108 (ASTM 2013b). The tests include those for flame spread as well as those intended to address fires being spread by burning brands or embers. The classifications for roof coverings are also Class A, B, and C, with Class A being the most restrictive. In the case of roof coverings and the ASTM E 108 test, untreated wood does not satisfy the requirements for Class C roof coverings. Fire-retardant-treated wood shingles are available that will satisfy Class C or Class B roof covering requirements. The roof shingles are typically western red cedar, a species that has some natural decay resistance. Wood roofs with Class A ratings are for the roof systems, which includes the specified underlayments (ASTM 2013b).

8.6 Security and Fire Alarm Options

The first means of protection against fire damage is prevention. Since historic covered bridges are often located in rural areas, fire-monitoring devices such as flame detectors can be used to alert officials of a possible threat of fire (Phares and others 2010). Faster response times minimize fire damage, thereby increasing the chances of successful post-fire rehabilitation (Fig. 11).

Minimizing fire damage to wood members is of the greatest importance to the rehabilitation, repair, and restoration of historic covered bridges. Because fire-retardant chemicals may reduce the structural capacity of wood members, the most effective way to minimize fire damage is to reduce burn time.

Historic covered timber bridges are often in remote locations. Fire on these structures will typically be reported after post-flashover has occurred or even after the decay period of a fire. Remote security and fire-detection monitoring system can be installed on all historic covered timber bridges to reduce response times; however, the cost effectiveness of these types of technologies should be considered.

A study carried out by Phares and others (2010) studied remote fire and vandalism monitoring systems using multiple fire detection devices and cameras on historic covered bridges. Iowa’s Cedar Covered Bridge was outfitted with an infrared camera, flame detector, and fiber optic sensors. The most expensive component of this system was the infrared camera, which cost $15,000. An infrared camera was used not only to detect trespassers after hours; it was also able to communicate with the other detection devices to minimize false fire alarms. Infrared cameras in this study were able to detect a pan-controlled fire in less than 10 seconds for each test and in less than 5 seconds for 75% of the tests. The pan-controlled fire is a contained fire in a 12-in.-diameter pan burning alcohol, because the standard gasoline fire was considered too volatile for use on a historic covered bridge.

Flame detectors also proved to be a relatively reliable fire detector, as they were able to detect a pan-controlled fire in 7 seconds at a distance of 15 ft; however, they were unable to detect lit butane lighter beyond 24 in. Even though the detection of small fires such as a lighter may be a limitation, the detection times associated with this technology proved to be successful. By installing fiber optic sensors near the timber bridge deck, they were also able to detect the presence of a fire when the fire was within a few feet of the sensor. This may be beneficial, as most fires are lit on the bridge deck alongside an interior wall of the bridge.

Of the three devices tested, the flame detectors proved to be the method of choice for detecting historic covered timber bridge fires because of their quick detection time for fire, ease of installation, reliability, and their relatively low cost.

8.7 Fire Extinguishment Options

The most popular forms of fire extinguishment include suppressing agents in foam or powder form, carbon dioxide, and water. All of these forms can be administered in different manners to increase the firefighting effectiveness of the media being used. Examples include but certainly aren't limited to automated systems, water additives, fire hydrants, and portable fire extinguishers.

Depending on the locations of historic covered bridges, the standard fire extinguishment may very well be through automated systems. Pipe and nozzle systems have long been used alongside water. Environmental impact is minimized because water, while readily available in most instances, is also inert. Water application systems must be properly maintained to avoid unnecessary exposure-related issues leading to the degradation of wood components. Water-mist applications can be differentiated into one of three types of systems: total, local, and zoned.

With respect to water additives, the advantages of a multi-composition (MC) additive to water-dispersing systems are well documented. The MC additives are made from mixing five different components. Of the additives, a fluorocarbon surfactant is used to form a thin film layer on members that are in line with the spray. A viscosity modifier can be used to improve the mobility and runoff of the mist spray; this allows for a quicker dispersion of the water and additive. An organic metallic compound can also be added to improve the fire extinguishing capabilities of the water by producing active radicals. Carbamide is used to absorb energy from the flame. It generates inert gases as a byproduct. The final compound, N,N-dimethylformamide, serves as an anti-freeze. Collectively, these chemicals, when added to a water mist spray, allow for a more effective way of extinguishing a remote bridge fire before irreparable damage to the bridge occurs (Zhou and Guangxuan 2006).

Should remote systems prove to be prohibitive, fire hydrants have also been used to extinguish bridge fires. Water must be supplied to the fire hydrant, which may or may not be a practical solution for all bridges in question. Similarly, portable fire extinguishers require an operator. Fire detection and response are not addressed through approaches such as this. Concerns of remoteness and readiness remain. As standalones, techniques like this could be construed as a last resort option for fighting, but have real advantages when incorporated into a larger extinguishment plan.

9.0 Concluding Remarks and Summary

Arson continues to claim many historic covered bridges. Site-specific post-fire evaluations of the structural integrity of a bridge are often necessary in a fire's aftermath. Decisions must be made on whether individual wood components can be rehabilitated, reconstructed, or replaced. This project on evaluating fire-damaged components of historic covered bridges includes post-fire assessment guidelines for evaluations, existing methodologies and techniques, and the customized application of a nondestructive technique, all of which can be employed during site-specific post-fire evaluations of historic covered bridges. More specifically,

- The existing approaches to post-fire evaluations were surveyed based on their relevance to historic covered bridges. A focal point included the rehabilitation, reconstruction, and replacement of individual fire-damaged flexural members.
- A nondestructive technique for directly determining the residual capacity of individual fire-damaged glued-laminated beams that are expected to remain in service was also produced.

10.0 Federal Program for Covered Bridges

Several important federal agencies participate in the preservation and registry of historic covered bridges. The largest database is the National Register of Historic Places. Owners and funding agencies alike can apply to register a bridge, an important step in the preservation and repair of such bridges.

The Historic American Engineering Record (HAER), part of the National Park Service, develops well documented records of historic structures including covered bridges. The HAER collection on covered bridges houses more than 3,500 sheets of measured and interpretive drawings, 72,000 photographs, and 61,000 data pages on over 7,000 sites, structures, and objects (Pierce and others 2005). All of these forms of data can be found at the U.S. Library of Congress, or the National Digital Library.

The U.S. Secretary of Interior's Standards for Historic Preservation has provided the *Standards and Guidelines for Archeology and Historic Preservation* (Federal Register 1983). These guidelines provide technical advice on preservation activities and methodologies including, but not limited to, rehabilitation, reconstruction, and restoration.

The FPL actively tests and evaluates different aspects of wood bridge design and fire-damage evaluation. The tests conducted by FPL allow for technological advances that would not otherwise be possible. The works of this agency are far reaching. For more extensive information on these and other agencies and/or organizations, refer to Chapter 18 of the *Covered Bridge Manual* (Pierce and others 2005).

The *Covered Bridge Manual* (Pierce and others 2005) provides excellent comprehensive support to those involved with assessing, maintaining, and rehabilitating covered bridges. While this resource states that site-specific post-fire evaluations are necessary in a fire's aftermath, additional guidance is absent. In particular, there is no direction on how to decide whether individual wood components can be rehabilitated, reconstructed, or replaced. Recognizing the need for additional information on this topic, this report is thought to complement and enhance the *Covered Bridge*

Manual current coverage of post-fire evaluations. It is hoped to be worthy of being included in the *Covered Bridge Manual* or similar future publications of the National Historic Covered Bridge Preservation Program.

In terms of historic preservation, engineers are to comply with the Historic Sites Act of 1935, Section 106 of the National Historic Preservation Act of 1966 and its amendments. The Federal Highway Administration's National Historic Covered Bridge Preservation Program started with its highway legislation the Transportation Equity Act for the 21st Century (TEA 21) (FHA 1999). It was subsequently renewed under the next Highway Bill the Safe, Accountable, Flexible, Efficient Transportation Equity Act: A Legacy for Users (SAFETEA-LU) (FHA 2009–2012), which funded rehabilitation, restoration, and preservation of covered bridges listed or eligible for listing on the National Register of Historic Places, and research and education. Information pertaining to the National Register for Historic Places application process can be found on the United States Department of Transportation website under their National Historic Covered Bridge Preservation Program (http://www.fhwa.dot.gov/bridge/covered.cfm). Procedures that relate to historic covered bridge improvements such as strengthening also need to conform to the Secretary of Interior's guidelines for preservation of historic structures.

The TEA 21 and SAFETEA-LU transportation bills provided funding for the repair and preservation of historic covered bridges, and for research, but did not provide funds for the full reconstruction of a historic bridge if it cannot be repaired (Wacker and Duwadi 2008). This program, however, expired with the expiration of SAFETEA-LU. With the loss of this approximately $10 million per year program, repairing and rehabilitating historic covered bridges becomes an even more difficult task. Preservationist groups could still apply for historic covered bridge grants, but it would be necessary to compete directly with other bridge and highway projects for funds. Therefore, the rehabilitation and reinforcing of historic covered bridges damaged by fire is extremely important to saving these historic landmarks from destruction after a fire event.

For timber bridges listed under the National Historic Registry (www.nps.gov/nr/), post-fire repair should aim to satisfy strength requirements from a structural, aesthetic, and historical perspective. Whenever possible, the strengthening methods should conform to the Secretary of Interior's guidelines for preservation of historic structures (Federal Register 1983).

The Vermont Historic Bridge Program (2014) has developed a progressive list of rehabilitation techniques. When such techniques are warranted, their adoption helps ensure that covered timber bridges accommodate historically-accurate engineering procedures that utilize similar materials. The Vermont Historic Covered Bridge Program mentions that if this is not feasible, member augmentation may be used after other options have been evaluated and are otherwise ruled out. In this case, modern materials like glass-fiber reinforced plastics (GFRP) exhibit extremely low profiles once installed and have been shown to increase the load-carrying capacity (Kim and Harries 2010; Gentile and others 1999).

References

Anderson, C.A.; Loferski, J.R.; Woeste, P.E. 2003. Detecting early wood decay. The Inspector's Journal, Online. Reprint accessed on 08/25/14 at http://www.inspectorsjournal.com/forum/topic.asp?TOPIC_ID=2101&SearchTerms=pick+test

American Lumber Standard Committee, Inc. 2015. American Softwood Lumber Standard Rules. Germantown, Maryland: American Lumber Standard Committee, Incorporated. http://www.alsc.org/greenbook%20collection/LumberProgram_facsimile.pdf

American Wood Council. 2012. National Design Specification for wood construction with commentary. ASD/LRFD. Washington D.C.: American Wood Council.

American Wood Council. 2014. Calculating the fire resistance of exposed wood members: Technical Report 10. Washington D.C.: American Wood Council. 51 p.

APA. 1989. Fire retardant treated plywood roof sheathing: In-situ testing. Tacoma, WA: American Plywood Association.

APA. 2009. Calculating fire resistance of glulam beams and columns. EWS Y245B. Tacoma, Washington: American Plywood Association – The Engineered Wood Association. http://www.drjlumber.com/wp-content/uploads/2014/07/Calculating-Fire-Resistance.pdf

ASTM. 1992. Standard test methods for use and calibration of hand-held moisture meters. Designation D4444-92. In: Annual book of ASTM standards. West Conshohocken, PA: American Society for Testing and Materials International.

ASTM. 1994. Standard test methods for small clear specimens of timber. ASTM D143-94. West Conshohocken, PA: American Society for Testing and Materials International.

ASTM. 1999. Standard guide for moisture conditioning of wood and wood-based materials. Designation D4933-99. In: Annual book of ASTM standards. West Conshohocken, PA: American Society for Testing and Materials International.

ASTM. 2000. Standard test method for structural panels in flexure. Designation D3043 In: Annual book of ASTM standards. West Conshohocken, PA: American Society for Testing and Materials International.

ASTM. 2009a. Standard test method for surface burning characteristics of building materials. Designation E84-09. In: Annual book of ASTM standards. West Conshohocken, PA: American Society for Testing and Materials International.

ASTM. 2009b. Standard test methods of static tests of lumber in structural sizes. Designation D198-09. In: Annual book of ASTM standards. West Conshohocken, PA: American Society for Testing and Materials International.

ASTM. 2010. Standard test methods for fire tests of building construction and materials. Designation E119-10a. In: Annual book of ASTM standards. West Conshohocken, PA: American Society for Testing and Materials International.

ASTM. 2013a. Standard test method for extended duration surface burning characteristics of building materials (30 min tunnel test). Designation E2768-11. In: Annual book of ASTM standards. West Conshohocken, PA: American Society for Testing and Materials International.

ASTM. 2013b. Standard test methods for fire tests of roof covering. Designation E108-11. In: Annual book of ASTM standards. West Conshohocken, PA: American Society for Testing and Materials International.

ASTM. 2014. Standard specification for gypsum board. Designation C1396/C1396M-14a. In: Annual book of ASTM standards.West Conshohocken, PA: American Society for Testing and Materials International.

Barbaresi, T.; Cozzani, V.; Di Padova, A.; Tallone, F.; Tugnoli, A. 2012. Mitigation of fire damage and escalation by fireproofing: A risk-based strategy. Reliability Engineering and System Safety. 105: 25.

Babrauskas, V. 2001. Ignition of wood: a review of the state of the art. London: Interflam, Interscience Communications, Ltd.: 71–88.

Barletta, M. 2005. Application of instrumented micro-indentations to 'in situ' mechanical characterization of wooden structures: Part I—Analysis of highly selected and decayed pinewood samples. Measurement. 39(4): 274–284.

Borri, A.; Corradi, M. 2011. Strengthening of timber beams with high strength steel cords. Composites Part B: Engineering. 42(6): 1480–1491. doi:10.1016/j.compositesb.2011.04.051

Branco, J.; Piazza, M.; Cruz, P. 2009. Structural analysis of two King-post timber trusses: Non-destructive evaluation and load-carrying tests. Construction and Building Materials. 24(3): 371–383.

Brashaw, B.K.; Bucur, V.; Divos, F.; Gonçalves, R.; Lu, J.; Meder, R.; Pellerin, R.; Potter, S.; Ross, R.; Wang, X.; Yin, Y. 2009. Nondestructive testing and evaluation of wood: A worldwide research update. Forest Products Journal. 59(3): 7–14.

Brashaw, B.K.; Wang, X.; Ross, R.J.; Pellerin, R.F. 2004. Relationship between stress wave velocities of green and dry veneer. Forest Products Journal. 54(6): 85–89.

Browne, F.L. 1958. Theories of the combustion of wood and its control. FPL Report 2136. Madison, WI: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory.

Buchanan, A.H. 2001. Structural design for fire safety. New York: John Wiley & Sons, Ltd.

Buchanan, A.; Barber, D. 1994. Fire resistance of epoxied steel rods in glulam timber. In: Proceedings, Volume 1, Pacific Timber Engineering Conference; 1994 July 11–15; Gold Coast, Australia. Fortitude Valley MAC, Queensland, Australia: Timber Research and Development Advisory Council: 590–598.

Custódio, J.; Broughton, J.; Cruz, H. 2009. A review of factors influencing the durability of structural bonded timber joints. International Journal of Adhesion and Adhesives. 29(2): 173–185. doi:10.1016/j.ijadhadh.2008.03.002

Dietenberger, M.A. 2004. Ignitability of materials in transitional heating regimes. In: Wood and fire safety. Proceedings, 5th international scientific conference, April 18–22, 2004. Slovak Republic, Svolen, Slovakia: Faculty of Wood Sciences and Technology, Technical University of Zvolen: 31–41.

DOT. 2007. Strengthening historic bridges to carry modern traffic. FHWA-HRT-07-041. McLean, VA: United States Department of Transportation, Federal Highway Administration.

Emerson, R.N.; Pollock, D.G.; Kainz, J.A.; Fridley, K.J.; McLean, D.L.; Ross, R.J. 1998. Nondestructive evaluation techniques for timber bridges. In: Proceedings of the 5th world conference on timber engineering, Swiss Federal Institute of Technology, Lausanne, Switzerland: 670–677.

Esteves B.; Pereira, H. 2008. Quality assessment of heat-treated wood by NIR spectroscopy. Holz als Roh- und Werkstoff. 66(5): 323–332.

Federal Register. 1983. Thursday, September 29, 1983, Part IV, Department of the Interior, National Park Service, Archeology and Historic Preservation: Secretary of the Interior's Standards and Guidelines. 48(190): 44738–44739.

Felicetti, Roberto. 2006. The drilling resistance test for the assessment of fire damaged concrete. Cement & Concrete Composites. 28(2006): 321–329.

FHA. 1999. Public Law 105–178, as amended by Title IX of Public Law 105–206. Transportation Equity Act for the 21st century. TEA 21. Washington, D.C.: U.S. Federal Highway Administration. https://www.fhwa.dot.gov/tea21/h2400idx.htm#sec1

FHA. 2009–2012. Safe, Accountable, Flexible, Efficient Transportation Equity Act: A legacy for users (SAFETEA-LU) Extension Acts. Washington, D.C.: U.S. Federal Highway Administration. https://www.fhwa.dot.gov/safetealu/legis.htm

Firmanti, A.; Backtiar E.T.; Surjokusumo, S.; Komatsu, K.; Kawai, S. 2004. Mechanical stress grading of tropical timbers without regard to species. Journal of Wood Science. 51: 339–347.

Fox, S.P. 1974. Residual strength of fire-damaged laminated beams. Forest Products Journal. 24(1): 48–50.

Freas, A.D. 1982. Evaluation, maintenance and upgrading of wood structures: a guide and commentary. American Society of Civil Engineers, Committee on Wood, Subcommittee on Evaluation, Maintenance and Upgrading of Timber Structures. New York: American Society of Civil Engineers. 428 p.

Fuller, J.J.; Leichti, R.J.; White, R.H. 1992. Temperature distribution in a nailed gypsum-stud joint exposed to fire. Fire and Materials. 16: 95–99.

Garab, J.; Toth, A.; Szalai, J.; Bejo, L.; Divos, F. 2010. Evaluating glued laminated beams using a nondestructive testing technique. Transactions of FAMENA. 34(4): 33–46.

Garcia, R.A.; Carvalho, A.M.; Latorraca, V.F.; Matos, J.L.M.; Santos, W.A.; Silva, R.F.M. 2012. Nondestructive evaluation of heat-treated *Eucalyptus grandis* Hill ex Maiden wood using stress wave method. Wood and Science Technology. 46:41–52.

Gentile C.; Svecova, D.; Saltzberg W.; Rizkalla, S. 1999. Strengthening of timber bridges. World Wise ’99, Winnipeg, Canada, December 6–8, 1999. 6 p.

Glos, P.; Henrici, D. 1991. Bending strength and MOE of structural timber (*Picea abies*) at temperatures up to 150°C. Holz als Roh-und Werkstoff. 49(1991): 417–422. Translated by Barbara J. Marshment, May 1992.

Green, D.W.; Evans, J.W. 2008. The immediate effect of temperature on the modulus of elasticity of green and dry lumber. Wood and Fiber Science. 40(3): 374–383.

Green, D.W.; Winandy, J.E.; Kretschmann, D.E. 1999. Mechanical properties of wood–wood as an engineering material. General Technical Report FPL–GTR–113. Madison, WI: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory.

Griggs, Frank. 2014. Union Bridge: First bridge across the Hudson River, 1804. Structure Magazine. February 2014.

ISIS Canada. 2000. Intelligent Sensing for Innovative Solutions (ISIS) Canada. Solution-oriented research. Annual Report (2000–2001). ISIS Canada Administrative Centre, The University of Manitoba, Winnipeg, Manitoba, Canada. http://www.isiscanada.com/archived-isiscanada/annual_report/ARE_01.pdf

James, W.L. 1988. Electric moisture meters for wood. General Technical Report FPL–GTR–6. Madison, WI: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory. 17 p.

Janssens, M.L.; White R.H. 1994. Short communication: temperature profiles in wood members exposed to fire. Fire and Materials. 18: 263–265.

Kim, Y.J.; Harries, K.A. 2010. Modeling of timber beams strengthened with various CFRP composites. Engineering Structures. 32(10): 3225-3234. doi:10.1016/j.engstruct.2010.06.011

King, M. 2002. NIDR guidelines for fire and smoke damage repair. Millerville, MD: The National Institute of Disaster Restoration.

Klinkhachorn, P.; Halabe, U.B.; GangaRao, H. 1999. Portable ultrasonic device to monitor timber properties: Assessment of charred timber members. In: Proceedings of the 28th International Conference on Fire Safety, Columbus, OH: 89–93.

Kretschmann, D.E. 2010. Mechanical properties of wood. In: Wood handbook, wood as an engineering material. General Technical Report FPL–GTR–190. Madison, WI: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory.

Kukay, B.M.; Barr, P.J.; Friel, L.; Coster, D.C.; Halling, M.W. 2008. Post-fire assessments, methodology, and equations for directly determining wood’s residual flexural properties. Forest Products Journal. 58(1–2): 40–46.

Kukay, B.M.; Todd, C. 2009. Determining wood’s residual flexural properties using nondestructive testing. Forest Products Journal, Wood Design Focus. 19(2): 8–12.

Kukay, B.; White, R.; Todd, C.; Jahn, T. 2013. Evaluating fire-damaged components of timber bridges and structures. Structures Congress 2013, Fire Risks and Impacts to Bridges Track. Pittsburgh, PA: 758–767.

Lebow, S.T.; Anthony, R.W. 2012. Guide for use of wood preservatives in historic structures General Technical Report FPL–GTR–217. Madison, WI: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory. 59 p.

Lebow, S.; Kirker, G.; White, R.; Amburgey, T.; Barnes, H.M.; Sanders, M.; Morrell, J. 2012. Guide for in-place treatment of wood in historic covered and modern bridges. General Technical Report FPL–GTR–205. Madison, WI: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory. 43 p.

Lebow, S.T.; Winandy, J. E. 1999. Effect of fire-retardant treatment on plywood pH and the relationship of pH to strength properties. Wood Science and Technology. 33(4): 285–298.

LeVan, S.L.; Winandy, J.E. 1990. Effects of fire retardant chemical on the bending properties of wood at elevated temperatures. Research Paper FPL–RP–498. Madison, WI: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory.

NAHB. 1990. Home builders guide to fire retardant treated plywood. Evaluation, testing and replacement. Upper Marlboro, MD: National Association of Home Builders, National Research Center. 49 p. http://www.awc.org/helpoutreach/faq/FAQfiles/FRT-Publications-listandref.html

NFPA. 2015. Standard for fire retardant-treated wood and fire-retardant coatings for building materials. Standard 703. Quincy, MA: National Fire Protection Association.

NFPA. 2013. Guide for fire and explosion investigations. NFPA 921. Quincy, MA: National Fire Protection Association. 305 p.

Nicholas, D.D.; Crawford, D. 2003. Concepts in the development of new accelerated test methods for wood decay. In: ACS Symposium Series. 845: 288–312. Washington, DC: American Chemical Society.

Niemz, P.; Mannes, D. 2012. Non-destructive testing of wood and wood-based materials. Journal of Cultural Heritage. 13(3): S26–S34, Supplement.

Nuopponen, M.; Wikberg, H.; Vuorinen, T.; Maunu, S.L.; Jämsä, S.; Viitaniemi, P. 2004. Heat-treated softwood exposed to weathering. Journal of Applied Polymer Science. 91: 2128–2134.

Pellerin, R.F.; Lavinder, J.A.; Ross, R.J.; Falk, R.H. 1996. NDE of timber bridges. In: Proceedings, SPIE, Nondestructive Evaluation of Materials and Composites, November 15, 1996. Volume 2944: 275–284. doi:10.1117/12.259068

Phares, Brent M.; LaViolette, Michael D.; Wipf, Terry J.; Ritter, Michael A. 2010. Remote monitoring of historic covered timber bridges for the prevention of arson and vandalism. General Technical Report FPL–GTR–191. Madison, WI: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory. 27 p.

Phares, B.; Wipf, T.; Sievers, R.; Hosteng, T. 2013. Covered bridge security manual. General Technical Report FPL–GTR–223. Madison, WI: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory. 34 p.

Pierce, P.C.; Brungraber, R.L.; Lichtenstein, A.; Sabol, S. 2005. Covered bridge manual. FHWA-HRT-04-098. McLean, Virginia: Federal Highway Administration. http://www.fhwa.dot.gov/publications/research/infrastructure/structures/04098/index.cfm.

Raftery, G.M.; Harte, A.M.; Rodd, P.D. 2009. Bond quality at the FRP wood interface using wood-laminating adhesives. International Journal of Adhesion and Adhesives. 29(2): 101–110. doi:10.1016/j.ijadhadh.2008.01.006

Reinprecht, L.; Panek, M. 2012. Ultrasonic technique for evaluation of biodefects in wood: Part 1 – influence of the position, extent, and degree of artificial rots. International Wood Products Journal. 3(2): 107–115.

Richardson, L.R.; Cornelissen, A.A. 1984. Fire-resistant coatings for roof/ceiling deck timbers. Fire and Materials. 11: 191–194.

Ritter, M.A. 1992. Maintenance and rehabilitation of timber bridges. In: Timber bridges: design, construction, inspection, and maintenance. Chapter 14. Washington, DC: U.S. Department of Agriculture, Forest Service.

Rodgers, P.; Otter, D.E.; Wipf, T.; Ritter, M.; Wood, D. 1997. Effectiveness of glued laminated components in strengthening timber bridges. Technical Document TD 97-026. Washington D.C.: Association of American Railroads.

Ross, R.J.; Brashaw, B.K.; Wang, X.; White, R.H.; Pellerin, R.F. 2005a. Post-fire assessment of structural wood members. In: Wood and timber condition assessment manual. Chapter 4. Madison, WI: Forest Products Society: 29–46.

Ross, R.J.; Wang, X.; Brashaw, B.K. 2005b. Detecting decay in wood components. In: Inspection and monitoring techniques for bridges and civil structures. Cambridge, England: Woodhead Publishing Limited: 100–114.

Ross, R.J.; Cooper, J.; Wang, Z. 1991a. In-place evaluation of fire retardant treated wood products. In: Proceedings, Eighth International Nondestructive Testing of Wood Symposium, Pullman, WA: 247–252.

Ross, R.J.; Pellerin, R.F. 1994. Nondestructive testing for assessing wood members in structures: A review. General Technical Report FPL–GTR–70. Madison, WI: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory.

Ross, R.; Pellerin, R.F.; Volny, N.; Salsig, W.W.; Falk, R.H. 2000. Stress wave timing nondestructive evaluation tools for inspecting historic structures: A guide for use and interpretation. General Technical Report FPL–GTR–119. Madison, WI: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory.

Rowell, R.M.; Dietenberger, M.A. 2013. Thermal properties, combustion, and fire retardancy of wood. In: Rowell, R., ed. Handbook of wood chemistry and wood composites. Second edition. New York: CRC Press. pp. 127–149. Chapter 6.

Schaffer, E. 1977. State of structural timber fire endurance. Wood and Fiber. 9(2): 145–170.

Schaffer, E.L. 1982a. Temperature. In: Evaluation, maintenance and upgrading of wood structures: a guide and commentary. New York: American Society of Civil Engineers: 69–92. Chapter 2, Section 2.3.

Schaffer, E.L. 1982b. Fire. In: Evaluation, maintenance and upgrading of wood structures: a guide and commentary. New York: American Society of Civil Engineers: 120–126. Chapter 2, Section 2.7.

Schaffer, E.L. 1984. Structural fire design: wood. Research Paper FPL 450. Madison, WI: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory. 16 p.

Schroeder, R.A. 1999. Post-fire analysis of construction materials. Berkeley, CA: University of California Berkeley. Ph.D. dissertation. 61–03, 1–1–275.

So, C.L.; Via, B.K.; Groom, L.H.; Schimleck, L.R.; Shupe, T.F.; Kelley, S.S.; Rials, T.G. 2004. Near infrared spectroscopy in the forest products industry. Forest Products Journal. 54(3): 6–16.

Truax, T.R.; Harrison, C.A.; Baechler, R.H. 1935. Experiments in fireproofing wood—fifth progress report. In: Proceedings of the American Wood Preservers' Association: 231–245.

Valipour, H.R.; Crews, K. 2011. Efficient finite element modeling of timber beams strengthened with bonded fibre reinforced polymers. Construction and Building Materials. 25(8): 3291–3300. doi:10.1016/j.conbuildmat.2011.03.017.

Vermont Historic Bridge Program. 2014. A Vermont Government Website. Copyright 2014. State of Vermont. http://historicbridges.vermont.gov/

Wacker, J.P.; Duwadi, S.R. 2008. Covered bridges in the United States and the preservation program. Federal Highway Administration. In: Proceedings of the 10th world conference on timber engineering. Miyazak, Japan; 2008 June 2–5. 8 p.

Wang, N.; Fu, Y.; Liu Y.; Yu, H.; Liu, Y. 2014. Synthesis of aluminum hydroxide thin coating and its influence on the thermochemical and fire-resistant properties of wood. Holzforschung. 68(7): 781–789.

Wang, X.; Wacker, J.P.; Ross, R.J.; Brashaw, B.K. 2005. Nondestructive assessment of timber bridges using a vibration-based method. In: Proceedings of the 2005 structures congress. New York: The Structural Engineering Institute of the American Society of Civil Engineers.

Watanabe, K.; Hart, J.F.; Mansfield, S.D.; Avramidis, S. 2010. The future of quality control for wood and wood products. In: Near-infrared technology applications for quality control in wood processing. Proceedings, The Final Conference of COST Action E53, 4–7, May 2010, Edinburgh.

White, R.H. 1984. Use of coating to improve fire resistance of wood. In: Lieff, M.; Stumpf, F.M., eds. Fire resistive coatings: the need for standards. ASTM STP 826. Philadelphia: American Society for Testing and Materials: 24–39.

White, R.H. 1986. An empirical model for predicting performance of fire-resistive coatings in wood construction. Journal of Testing and Evaluation. 14(2): 97–108.

White, R.H. 2004. Post-fire assessment of structural wood members. In: Wood and timber condition assessment manual, first edition. Madison, WI: Forest Products Society: 29–46. Chapter 4.

White, R.H.; Cramer, S.M.; Shrestha, D.K. 1993. Fire endurance model for a metal-plate-connect wood truss. Research Paper FPL–RP–522. Madison, WI: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory. 12 p.

White, R.H.; Dietenberger, M.A. 2001.Wood products: thermal degradation and fire. In: Encyclopedia of materials: science and technology. Elsevier Science, Ltd.: 9712–9716.

White, R.H.; Dietenberger, M.A. 2010. Fire safety of wood construction. In: Wood handbook—wood as an engineering material. General Technical Report FPL–GTR–190. Madison, WI: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory: 18-1–18-22.

White, R.H.; Kukay, B.; Wacker, J. 2013a. Options for NDE assessment of heat and fire-damaged wood. 18th International Nondestructive Testing and Evaluation of Wood Symposium, September 2013, Madison, Wisconsin.

White, R.H.; Kukay, B.; Wacker, J.; Sannon, J. 2013b. Options for evaluating fire-damaged components of historic covered bridges. Second National Covered Bridge Conference. Dayton, OH.

White, R.H.; Nordheim, E.V. 1992. Charring rate of wood for ASTM E 119 exposure. Fire Technology. 28(1): 5–30.

White, R.H.; Ross, R.J. (Eds.). 2014. Wood and timber condition assessment manual: second edition. General Technical Report FPL–GTR–234. Madison, WI: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory. 93 p.

White, R.H.; Woeste, F. 2013. Post-fire analysis of solid-sawn heavy beams. Structure Magazine. November 2013.

Williamson, T.G. 1982a. Rehabilitation of fire-damaged members.In: Evaluation, maintenance and upgrading of wood structures: a guide and commentary. New York: American Society of Civil Engineers: 172–178. Chapter 4, Section 4.5.4.

Williamson, T.G. 1982b. Rehabilitation of fire-damaged timber—the Filene Center. In: Evaluation, maintenance and upgrading of wood structures: a guide and commentary. New York: American Society of Civil Engineers: 417–422. Chapter 8, Section 8.8.

Winandy, J.E. 2001. Thermal degradation of fire-retardant-treated wood: predicting residual service-life. Forest Products Journal. 51(2): 47–54.

Winandy, J.E.; Ross, R.J.; LeVan, S.L. 1991a. Fire retardant treated wood: Research at U.S. Department of Agriculture, Forest Service, Forest Products Laboratory. Proceedings, International Timber Engineering Conference, London, 1991 Sept. 1–5. p. 4.69–4.74.

Winandy, J.E.; LeVan, S.L.; Ross, R.J.; Hoffman, S.P.; McIntyre, C.R. 1991b. Development and evaluation of a test protocol to compare the effects of extended exposure to elevated temperature on untreated and fire-retardant-treated plywood. Research Paper FPL–RP–501. Madison, WI: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory.

Winandy, J.E.; Lebow, P.K.; Nelson, W. 1998. Predicting bending strength of fire-retardant-treated plywood from screw-withdrawal tests. Research Paper FPL–RP–568. Madison, WI: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory. 20 p.

Yahyaei-Moayyed, M.; Taheri, F. 2011. Experimental and computational investigations into creep response of AFRP reinforced timber beams. Composite Structures. 93(2): 616–628. doi:10.1016/j.compstruct.2010.08.017

Zicherman, J.B.; Williamson, R.B. 1981. Microstructure of wood char. Wood Science and Technology. 15: 237–249.

Zickler, G.A.; Schoberl, T.; Paris, O. 2006. Mechanical properties of pyrolysed wood: a nanoindentation study. Philosophical Magazine A: Physics of condensed matter: structure, defects and mechanical properties. 86(10): 1373–1386.

Zhou, X.; Guangxuan, L.; Bo, C. 2006. Improvement of water mist's fire-extinguishing efficiency with MC additive. Fire Safety Journal. 41: 39–45.

Additional Sources

American Forest & Paper Association. 2001. The allowable stress design (ASD) manual for engineered wood construction. Washington DC: American Forest & Paper Association.

American Wood Council. 2014. Design for code acceptance. http://www.awc.org/Publications/dca/dca1/DCA1.pdf

ASTM. 2010. Practice for sampling and data-analysis for structural wood and wood-based products. Designation D2915-10. In: Annual book of ASTM standards. West Conshohocken, PA: American Society for Testing and Materials International.

ASTM. 2009. Standard test method for evaluating the flexural properties of fire-retardant treated softwood plywood exposed to elevated temperatures. Designation D5516-09. In: Annual book of ASTM standards. West Conshohocken, PA: American Society for Testing and Materials International.

ASTM. 2010. Standard test method for evaluating the effects of fire-retardant treatments and elevated temperatures on strength properties of fire-retardant treated lumber. Designation D5664-10. In: Annual book of ASTM standards. West Conshohocken, PA: American Society for Testing and Materials International.

ASTM. 2013. Standard test methods for standardization and calibration of in-line dry lumber moisture meters. Designation D6782-13. In: Annual book of ASTM standards. West Conshohocken, PA: American Society for Testing and Materials International.

AWPA. 2014. AWPA book of standards. AWPA Technical Committee. American Wood Protection Association.

White, R.H. 2008. Analytical methods for determining fire resistance of timber members. In: SFPE handbook of fire protection engineering, fourth edition. Quincy, MA: National Fire Protection Association: 346–366.

Appendix—Development of New NDE Test for Fire-Damaged Wood

A.1 Technical Basis for Technique

The goal of this research is to adapt a direct test to estimate the residual flexural properties of individual framing members for post-fire evaluations. The concentrated proof loading tests are typically conducted in the field. The proof loading technique summarized below is similar to that which is mentioned in the NAHB FRT evaluation guide, created by the American Plywood Association for untreated plywood panels (NAHB 1990). The adaptation incorporates partial-composite action with an aluminum stock and a cycle of increasing load tests that are applied gradually. The evaluations of the methodology were based on eight, glued-laminated beams of which four were fire-damaged. The fire-damaged specimens were exposed to a specific time-temperature curve intended to produce a significant zone of damaged wood due to elevated temperatures within the residual wood section. The proposed test applies when flexural properties are to be refined beyond estimates that rely solely on reductions in cross-section and tabulated values for fire-damaged members and members that can otherwise be expected to remain in service.

A.2 Material Description

A.2.1 Design Values

The evaluations are based on tests of eight 24F-V8 DF/DF 5-1/8-in. by 12-in. by 10-ft (130-mm by 305-mm by 3-m) glued-laminated beams. All beams were ordered from the same supplier and were used as the test specimens for this project.

A.2.2 Moisture Content and Conditioning

Upon arrival, each of the glued-laminated beams was conditioned until equilibrium moisture content to limit the variability amongst the beams because of moisture content (ASTM D 4933, ASTM 1999). The beams were conditioned using the heated-room dry method at a temperature of 83 °F (28.5 °C) with a 14.7% ± 1.0% relative humidity. Both the moisture content and relative humidity were tracked on a regular basis using a Delmhorst Moisture Check Meter and a Delmhorst Thermo-Hygrometer. All of the beams were then wrapped in Visqueen polyethylene sheeting to avoid reconditioning. Four of these beams served as the control group. These beams were not fire-damaged. The other four beams were fire-damaged in a furnace at FPL.

A.2.3 Field Procedure and Equipment

The technique involved affixing a 6061-T651, type 200, aluminum block to the bottom of a beam with 1/4-20x2 socket cap screws once 7/32-in. (5.6-mm) pilot holes were drilled into the bottom of the beam at these locations. Each screw was inserted to a torque of 30 lb-in. (3.4 N-m) at an insertion depth of 0.75 in. (19 mm). Next, increasing loads were applied and deflections were recorded. Both the beam and the aluminum block were subject to bending, although at different rates; as such, measurable changes in average torque readings resulted from the interaction between the aluminum block and bottom of the beam after the load was removed for the partial-composite load tests.

The purpose of this was to establish a definitive target deflection that the beam could be reloaded to and that was within the linear elastic range. In so doing, the corresponding load, otherwise identified as the "40% estimate of maximum" load (hereafter referred to as the 40% load) is achieved once mean torque readings correspond to values in this range. Here, deflections were recorded along with values for the increasing, applied loads using a data acquisition system. The estimated maximum load is simply a multiplier of the 40% load. With this value, both the residual stiffness (apparent modulus of elasticity, E_f) and the residual strength (modulus of rupture, S_R) can be accurately estimated using conventional equations found in the appendix of D 198-09 (ASTM 2009b). The difference is that the flexural property approximations now stem from a definitive load estimate in the linear elastic range. The tests performed on two expendable beams eliminated a considerable amount of trial and error that would have otherwise accompanied the load tests. The field equipment items include a digital level, a calibrated torque wrench, a pair of calipers, and a pre-drilled aluminum bar (attached to the bottom of a fire-damaged beam).

A.3 Laboratory Procedure

A.3.1 Fire Damage

Four beams were exposed to a specific time-temperature curve intended to produce a significant zone of damaged wood from elevated temperatures within the residual wood section. The 4-ft by 7-ft (1.2-m by 2.1-m) horizontal furnace has a series of eight diffusion-flame natural gas burners on the floor of the furnace. Each of the four 5-1/8-in. by 12-in. by 10-ft (130-mm by 305-mm by 3-m) test beams was inserted in openings on the ends of the furnace and exposed to a relatively low fire exposure for a period of three hours. The tops of the beams were covered with gypsum board so three surfaces were exposed to the fire. Approximately 6 ft (1.8 m) of the middle of each beam was subjected to fire exposure.

Exposure to the standard ASTM E 119 (ASTM 2010) fire exposure of fire-resistance tests would have resulted in a fairly steep temperature gradient within the wood beam. Since the intent was to evaluate the ability to determine loss in strength of the residual section, a longer and lower temperature fire exposure was used in hope of a more gradual temperature gradient within the charred member.

Table 1—Idealized time–temperature curve

Temperature (°F)	Temperature (°C)	Time (min)
149	65	10
199	93	20
370	188	30
500	260	45
621	327	60
662	350	120
707	375	180

A modification of a curve previously used to represent the plenum exposure in a protected wood truss assembly was employed (White and others 1993). This idealized time–temperature curve is summarized in Table 1.

At the end of the approximately three hours of fire exposure, the beams were quickly removed from the furnace and sprayed with water. A series of thermocouples were inserted 4 in. (102 mm) down from the top surface of the specimen at various distances of 0.25 to 2.5 in. (6 to 64 mm) from the surfaces of the sides of the beam. The temperature data were used to plot a temperature profile at the time of test termination. Analysis of the combined temperature data for all four tests indicated that the base of char layer with a temperature of 572 °F (300 °C) was approximately 0.3 in. (8 mm) inward. As was expected, the center temperature was much cooler at approximately 212 °F (100 °C).

In contrast, the base of the char layer for the semi-infinite slab has a different temperature profile. Specifically, 356° F (180 °C) at 0.25 in. (6 mm) and 68° F (20 °C) at 1.3 in. (33 mm). When compared to a semi-infinite slab subjected to a standard fire-resistance test, the temperature was 491° F (255 °C) at 0.25 in. (6 mm) and was 252° F (122 °C) at 1.3 in. (33 mm) inward.

There was more charring along the bottom of the glued-laminated beams than along the sides. Each glued-laminated, fire-damaged beam was cooled to room temperature prior to being packaged and shipped back to Montana Tech of The University of Montana for further evaluation and testing.

A.3.2 Laboratory Tests

After removing the char layer from each of the fire-damaged beams, the section properties for all beams were measured and recorded. The beams were then instrumented with epoxied (Vishay foil strain gauge) and removable Transducer Model BDI ST350 uniaxial strain gauges (Bridge Diagnostics, Inc., Boulder, Colorado). The two types of strain gauges were affixed at comparable locations, above, below, and near the depth-wise center of each beam.

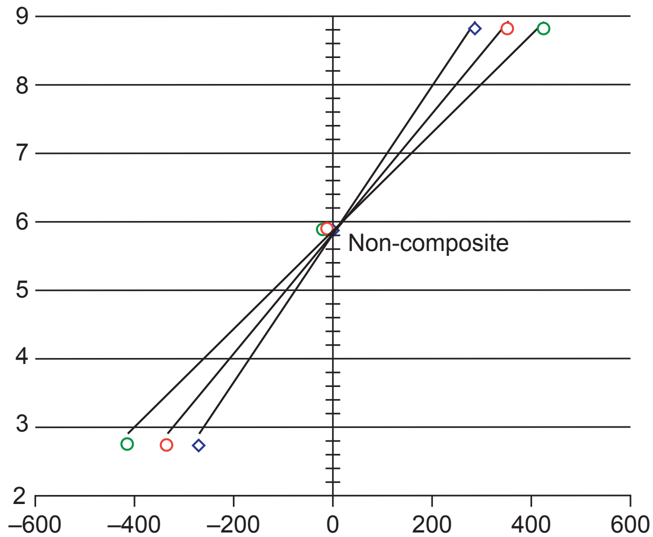


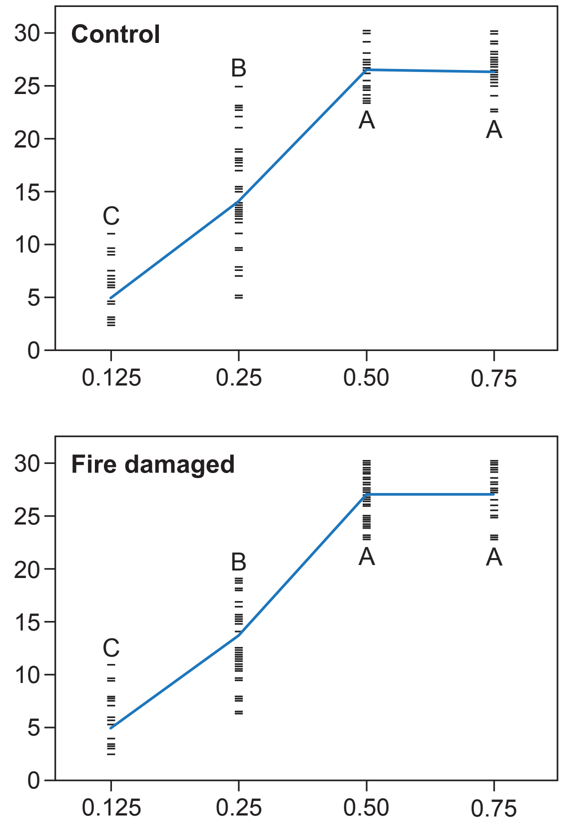
Figure 12—Neutral axis plot for a typical, non-composite beam.

Values collected from the gauges were used to verify the amount of composite action that was achieved. All beams were load-tested at Montana Tech of The University of Montana using a steel load frame (Fig. 12). The load frame consists of four HP Shapes, a reaction beam, and various support beams. Hydraulic loads were applied with a 10,000 psi (69 MPa) capacity hydraulic hand pump and cylinder equipped with a tilt-saddle. The values associated with the applied loads and corresponding deflections were recorded using load cells and displacement transducers during the appropriate stages of testing (Fig. 12).

A.3.3 Creating Partial Composite Action between the Aluminum Stock and the Wood Beam

The results from the load-test data indicate that a modest, neutral axis shift of 2.5% or less was achieved. Further evaluation of these results indicated that the partial composite action between the aluminum stock and a glued-laminated beam was appropriate at this level. This is because the measurable changes in the mean machine screw torques were also germane to the proposed field test. The shift in the neutral axis of each beam was identified and the amount of partial-composite action was assessed from the strain measurements at three different locations. To measure this, values for strain were plotted at three distinct locations for three distinct linear-elastic loads. The intersection of the resulting lines that were fit to the data denotes the measured neutral axis. The neutral axes were measured with the aluminum stock affixed to the tension face of the beam (partial-composite action) and without the aluminum stock affixed to the tension face (non-composite action). Fig. 12 depicts such a plot for a typical non-composite beam test.

The neutral axis shifts 2% on average when the partial-composite tests are compared to the non-composite tests using



Screw insertion depth (in.)	Control	Fire damaged
0.125	5.47, 8.09	4.22, 6.03
0.25	12.43, 15.94	12.05, 14.34
0.50	25.95, 27.14	26.48, 27.67
0.75	25.78, 26.97	26.59, 27.72

Figure 13—Summary of statistics for the control and fire-damaged groups.

epoxied stain gauges. This shift is 2.5% on average when removable strain gauges are used. The results indicate that both the removable strain gauges and epoxied strain gauges are comparable. However, field applications may make one type of gauge preferable to the other.

A.3.4 Torque Readings

The purpose of the screw-withdrawal test is to establish a definitive, target deflection that individual beams can be re-loaded to that was within the linear elastic range. Torque readings of four different insertion depths were explored in this study: 0.75 in. (19 mm), 0.5 in. (13 mm), 0.25 in. (6.3 mm) and 0.125 in. (3.2 mm). The results from laboratory experiments and statistical analyses indicate that for both the control and fire-damaged group, the variability at an insertion depth of 0.25 in. (6.3 mm) was greater than the variability at insertion depths of 0.5 in. (13 mm) and 0.75 in. (19 mm). These results are based on the Brown-Forsythe test with an initial p-value <0.0001 when testing the equality of all variances for both groups. Note, at an insertion depth of 0.125 in. (3.2 mm), the variability of the machine screw torque values were unreliable because of missing values that occurred because the machine screws stripped out upon contact.

The mean torque readings were compared using the 2-way ANOVA F-tests as well as the Tukey HSD multiple comparison procedure. Plots of the mean torque readings, along with individual 95% confidence intervals collected at these depths are depicted in Figure 13.

When comparing across the control and fire-damaged groups, as well as across insertion depths, the Tukey HSD multiple comparison procedure results, calculated at the 95% experiment-wise error rate, are significantly different where the levels are not connected by the same letter (Fig. 13). Note that at the conclusion of the partial-composite load tests, residual machine screw depths of 0.25 in. (6.3 mm) or less are to be avoided because of an increased potential for variability in torque readings and (or) the inability to obtain them. We recommend that the average values for torque be used on a given beam and that the test is performed on a number of beams as a way of controlling for naturally occurring variability inherent to these measurements.

We recommend that torque readings collected after a load is removed should conclude when torque readings fall between 20 and 25 lb-in. (2.3 and 2.8 N-m) based on the results from the laboratory experiments. This recommendation is also supported through the results of the statistical analyses wherein the variability in torque readings is reduced as the insertion depth passes 0.25 in. (6.3 mm) and levels off at 0.5 in. (13 mm). Torque readings within this range can be achieved by incrementally loading each of the beams in the laboratory and collecting torque readings once the load is removed. This range falls below the 95% confidence intervals for the average screw torque readings at insertion depths of 0.75 in. (19 mm) and 0.5 in. (13 mm). As supported by the Tukey HSD procedure, any measurable changes in average torque readings at these levels is non-significant. Moreover, any measurable changes in average torque are attributed to the resulting interaction between the aluminum stock and bottom of the beam. It should also be noted that this range is above the 95% confidence intervals for mean screw torque readings at insertion depths of 0.25 in. (6.3 mm) and 0.125 in. (3.2 mm). These values are also presented in Figure 13.

A.3.5 Calculation of the Modulus of Elasticity and the Modulus of Rupture

The purpose of the screw-withdrawal test is to establish a definitive target deflection that individual beams can be re-loaded to and that was within the linear elastic range. In so doing, the corresponding load, otherwise identified as

Table 2—Actual and estimated values for S_R and E_f, all beams

Beam ID	Average torque reading, lbf-in (N-m)	Target deflection, in. (mm)	Apparent E_f, $\times 10^3$ lbf/in^2 (MPa)		S_R, lbf/in^2 (MPa)	
			Actual	Estimate	Actual	Estimate
Control						
Beam 3	24 (2.7)	0.327 (8.3)	749 (5,170)	793 (5,470)	5,060 (34.9)	4,640 (32.0)
Beam 7	24 (2.7)	0.339 (8.6)	736 (5,070)	729 (5,030)	6,740 (46.5)	5,530 (38.1)
Beam 8	20 (2.3)	0.218 (5.5)	1,109 (7,650)	1,106 (7,630)	5,000 (34.5)	4,900 (33.8)
Beam L	22 (2.5)	0.192 (4.9)	1,075 (7,410)	1,090 (7,520)	7,440 (51.3)	4,150 (28.6)
Average	—	—	917 (6,320)	930 (6,410)	6,060 (41.8)	4,810 (33.1)
Fire damaged						
Beam 2	24 (2.7)	0.247 (6.3)	810 (5,590)	818 (5,640)	3,620 (25.0)	3,600 (24.8)
Beam 4	22 (2.5)	0.306 (7.8)	645 (4,450)	652 (4,500)	3,890 (26.8)	3,750 (25.9)
Beam 5	25 (2.8)	0.145 (3.7)	887 (6,120)	891 (6,150)	2,560 (17.6)	2,440 (16.8)
Beam 6	23 (2.6)	0.277 (7.0)	570 (3,930)	567 (3,910)	3,060 (21.1)	2,990 (20.6)
Average	—	—	728 (5,020)	732 (5,050)	3,280 (22.6)	3,190 (22.0)

the “40% estimate of maximum” load (hereafter referred to as the 40% load) is achieved once mean torque readings correspond to values in this range. (Here, deflections were recorded along with values for the increasing, applied loads using a data acquisition system.) The estimated maximum load is simply a multiplier of the 40% load. With this value, both the residual stiffness (apparent modulus of elasticity, E_f) and the residual strength (modulus of rupture, S_R) can be accurately estimated, using conventional equations found in the appendix of D 198-09 (ASTM 2009b).

A.4 Results—Flexural Properties

The estimated values for flexural properties were obtained by using the methodology established through the proposed field test. Table 2 presents the results for the torque tests, the corresponding target values for deflection, along with the actual and estimated flexural properties for all of the beams evaluated herein.

The determination of the actual values for the beams tested destructively without the aluminum bar was made using the methodologies described in ASTM D 198 (ASTM 2009b) for each beam with solid rectangular homogeneous cross-section and center point loading. The modulus of rupture, SR, was calculated using the maximum bending moment at the maximum load borne by the beam. The apparent modulus of elasticity, E_f, was calculated using the equation in the appendix of D 198 with the load-deflection slope determined for the range of 20% to 40% of the maximum load.

The data collected from the laboratory tests indicate that the averages for the estimated and actual E_f differed by less than 2% for the control group and less than 1% for the fire-damaged group. Should beam L be omitted, the averages for the S_R for the control group differ by approximately 10% where the actual values and estimated values are compared to one another. Including beam L, this average becomes larger. A comparison of averages made within the fire-damaged group shows that the S_R differs by less than 3% for the actual and estimated values. These results are depicted in Figure 14.

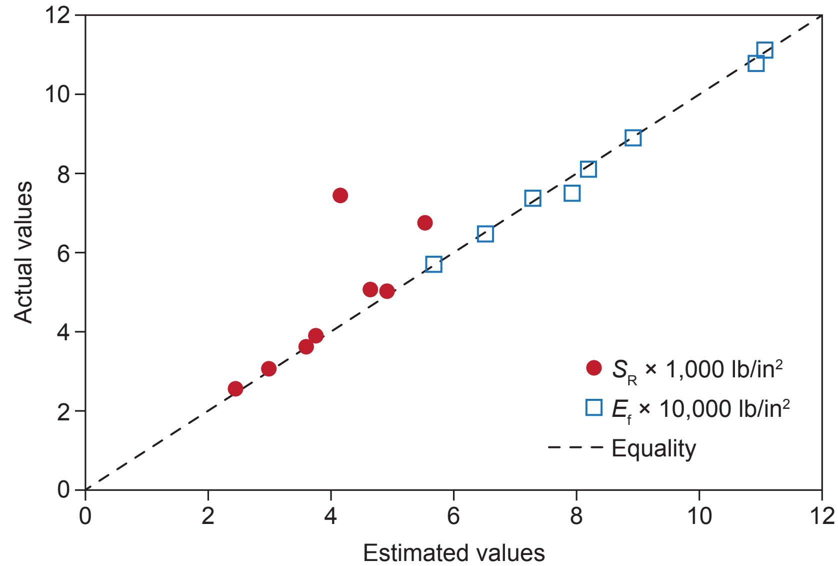


Figure 14—Estimated and actual values for S_R and E_f, all beams.

A.5 Conclusions

This research presents a proposed field technique that can be used to determine the residual flexural properties in glued-laminated beams that have been fire-damaged. Two treatment groups were established: a control group and a fire-damaged group. The nondestructive values, arrived at through the proposed field test, were compared to the destructive values for the beams from both groups. The following conclusions are based on these results.

Results show that on average the relative changes in the apparent modulus of elasticity and the modulus of rupture changed by 20% and 46% after the beams had been fire-damaged. This marked decrease in flexural properties goes beyond simply reducing the cross-section once the char layer has been removed.

Results of this research indicate that torque readings collected after a load is removed should conclude when torque readings fall between a range that is inclusive of 20 and 25 lb-in. (2.3 and 2.8 N-m). The same finite range was applicable to beams in either group. This approach has been validated in the laboratory and is specific to the beams tested herein.

Results corroborate the proposed field technique when direct determinations of estimates of the residual flexural properties are sought. This is important where like members that exhibit various degrees of degradation are tested in the field and are otherwise expected to remain in service because the flexural property approximations now stem from a definitive load estimate in the linear elastic range.