This bridge is a Three-pin Timber Arch with a span of 90 feet. It is over the Partridge River 1 mile east of Hoyt Lakes, Minnesota in St. Louis County. It was completed in 1996 as part of Forest Highway No. 11. It is designed for three lanes of HS-25 trucks and a 6 foot wide sidewalk.

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Wheeler Lumber, LLC is a construction products company which specializes in treated timber. The name Wheeler has been around since Wheeler Lumber Bridge and Supply Company opened its doors in 1892. There have been several name and ownership changes over the years, but the focus of the company has remained constant, to manufacture and sell construction products. This company has filled a unique need in the construction industry by becoming the foremost supplier of treated timber bridges. The company has also become a leader in their industry by acquiring technical expertise on the design and use of treated timber. Engineers, architects, contractors and government officials have come to rely on Wheeler’s expertise over the years when they are designing or building anything using treated timber.
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INTRODUCTION

There are so many aspects of the repair and rehabilitation of highway bridges that any effort to produce a comprehensive paper on this subject is an overwhelming task and clearly beyond the limits of this endeavor. The focus of this paper will be on timber bridges and those instances where treated timber would be the most appropriate material to be used for the construction procedures proposed. Some of the considerations leading to the eventual choice of either to repair, rehabilitate or replace a specific structure are controlled by administrative policies and procedures and are specific to the governmental units directly involved. Discussion of those administrative considerations is a clearly outside of the scope of this paper.

The inclusion in this paper of repair and rehabilitation procedures correcting some type of deficiency does not in itself imply that that specific procedure should be done. Implementation of any of the procedures given must be evaluated considering all of the facts unique to the specific site. This author does not mean to imply that all timber bridges can or should be repaired or rehabilitated. Many bridges simply are not good candidates for either repair or rehabilitation. The bridges shown in Photograph No. 1 and 2 are typical of bridges that should be scheduled for replacement. That decision must be made by the owner. The purpose of this paper is to offer alternatives to be considered. Good public administration consists of carefully evaluating all of the options and selecting the one that is in its best interest.

The benefits of using treated timber for the repair and rehabilitation of highway bridges will be demonstrated by examples and case studies. The deficiencies corrected using these procedures represent many of the common problems facing road authorities today. Design data and procedures, along with construction details are provided to enable the engineer to design bridge repair and rehabilitation projects using treated timber. Also, several examples will be shown where treated timber can be used to rehabilitate bridges constructed from steel and concrete.

As the infrastructure of our nation continues to deteriorate faster than the current rate of replacement, this paper offers some very cost-effective methods of repairing and rehabilitating structurally and functionally deficient bridges.

Photograph No. 1 - Not all timber bridges are good candidates for rehabilitation. Many bridges have been neglected too long.

The information presented here will be arranged in chapters, each covering a specific topic. The first chapter will be devoted to the material itself. This will include useful information
concerning specifications and grades of material. The next chapter will cover repair. Repair is used here to indicate replacement of specific damaged or deteriorated components. For the most part, these will be remedial measures that will not dramatically change capacity of the bridge. The last chapter will cover rehabilitation techniques. These techniques are designed to increase the capacity or functionality of the structure and lengthen its useful life.

Photograph No. 2 - Bridges constructed with untreated, native timber generally should not be considered for rehabilitation.
GENERAL CONSIDERATIONS

Key to any decision concerning the repair or rehabilitation of a highway bridge is serviceability. The serviceability, or useful value, of all bridges is measured on different scales. First is the structural capacity of the bridge. Next is the ability of the structure to carry the traffic safely over the bridge; this measure is called the functionality. Bridges not meeting some minimal level are said to be functionally obsolete. Bridges are also evaluated on their hydraulic capacity. Failure of a structure to meet some minimum level of performance in one or more of these categories will render the structure deficient.

Only under very unusual circumstances do all three indices approach zero simultaneously. This independence of the different components and features of a bridge leads to a condition where some of the components have a useful life several times longer than other parts of the same structure. The rehabilitation of bridges is the process of addressing the deficiencies of a structure by capitalizing on the attributes. The bridges shown in Photographs No. 3 and 4 are typical of bridges 30 to 40 years of age that have the potential for 30 to 40 more years of useful service life with reasonable care and maintenance. These are the types of structures generally referred to in this paper.

Well-engineered rehabilitation alternatives should always be considered for any deficient structure. The evaluation of repair and rehabilitation alternatives is not an easy task for the owner of the bridge since all components do not age and deteriorate at the same rate nor do the proposed new components all have a known uniform expected life. The additional engineering required for rehab projects will in some cases discourage some engineers from proposing them. Highway departments, county boards, and other agencies that control highway funds must recognize that it is in their best interests to put additional engineering resources into well-engineered rehabilitation projects. We, as a society, can no longer allow the least durable attribute of a bridge to dictate the useful life of the structure.

The selection of treated timber as the material for bridge rehabilitation is based on several factors, the most important of which is that treated timber can be prefabricated into large, yet light, components that can be designed to fit specific site conditions. The use of large components allows for fast installation and the light weight of the timber reduces the size of the equipment necessary. The use of this material is not affected by freezing and wet weather so rehabilitation can be performed during off-peak seasons. Treated timber can be used to rehab structures constructed from other types of materials. Working with timber does not require specialized tools or highly technical skills.
Most possible bridge work can be divided into one of two categories, repair or rehabilitation. The distinction between these two categories is generally based on the premise that “repair work” does not significantly increase the capacity of the bridge, but only restores it to some recent precondition. Whereas, rehabilitation projects are generally thought to increase capacity, width or improve the safety of the structure. This paper will therefore present a separate section for each type of work. However, in a great many of the rehab projects there are also some repair work items to bring the structure up to some level of serviceability where the proposed rehab work will be economically justified.

Photograph No. 4 - Some treated timber structures are approaching 50 years of age and can provide 30 to 40 more years of service with some care.
MATERIAL

The design of repair and rehab projects using treated timber is not a complicated process. However, it does require some design data not generally available to many engineers. Much of the information needed is included in this paper. The *Standard Specifications For Highway Bridges* published by the American Association of State And Highway Officials contains additional information. The best source for allowable design values is the *Design Values For Wood Construction*, which is part of the *National Design Specifications For Wood Construction* published by the American Forest and Paper Association, 1111 19th Street NW, Suite 800, Washington, DC 20036, Phone (202) 463-2700.

The most important consideration in the design of any treated timber project is that as much as possible of the cutting and drilling of the timber members should be done before the wood is treated. Photograph No. 5 shows typical bridge fabrication where the parts are cut, drilled, and machined for special hardware prior to treatment. The long-term performance of a timber component is in jeopardy whenever the timber is cut or drilled in the field. Much thought must be given to the design so that all of the fabrication can be done prior to treatment. This means developing techniques and details so that all holes and lengths can be predetermined and fabricated with sufficient accuracy to be assembled in the field. This may require some components be assembled in the plant, match marked, and then disassembled for treatment. This will ensure that the parts fit properly.

Material specification for most timber rehab projects cover the following three general areas:

1. Timber
2. Preservative Treatment
3. Hardware

The timber is generally the largest segment of the material for a typical project and does not require extensive specifications. Most specifications for rehab work will specify the species, typically Coastal Region Douglas Fir, and the stress grade which is typically No. 1. A statement that this timber must meet the requirements as contained in the *National Design Specification for Wood Construction* (NDS) will be sufficient to ensure an enforceable specification. Most state highway department standard specifications make reference to the NDS as the base specification for timber.

Specifications for the preservative treatment are also simple. The American Wood-Preservers’ Association (AWPA) is the organization responsible for the development of specifications for treated timber. These specifications are updated each year and are
published as *American Wood-Preservers’ Association Standards*. The best thing about these “standards” is that they are “commodity standards.” This means that you do not have to detail all of the aspects of the preservative treatment process and the desired results. You only have to specify the species that you want and the type of preservative you want, like Copper Naphthenate. Once you have specified these two things you say in your specifications “meeting all applicable AWPA specifications.” You see, the AWPA specifications have specific sections based on the intended end use; in the case of highway bridges there is a complete section *C14 Wood For Highway Construction - Preservative Treatment By Pressure Process*. This section contains all of the detailed specifications that will ensure that the material furnished will meet your expectations. Just like the *National Design Standards for Wood Construction*, the *American Wood-Preservers’ Association Standards* are the base specification for most state highway department standard specifications.

There is no corresponding single source specification for galvanized bridge hardware. It will be easier to use your State Highway Department’s Standard Specifications for the bridge hardware. Most bridge hardware, bolts, washers, drift pins, dome-head bolts, drive spikes, shear ring, and related items are ASTM A307. However, if your project includes adding a crash-tested bridge railing system normally, those bolts in tension are ASTM A325. All bridge hardware must be hot-dipped galvanized.

**SELECTING MATERIAL**

Try to limit your design to stress grades of No. 1 or No. 1 and Better. There are higher stress grades available, however they tend to be much more costly and are many times not as economic as selecting a slightly larger size member using a lower stress grade. One other concern is that the size, length, and stress grade specified is available. It is worth the effort to contact a supplier prior to finalizing your materials list to make sure that all the components listed are readily available at a reasonable cost.

The selection of a preservative offers fewer choices than the timber components. The only treatments that should be accepted for the main structural members are oil-borne preservatives. Creosote is one of the better preservatives for highway bridges. However, it is meeting increased opposition from environmental interests. The preservative of choice today for most timber bridge construction is Copper Naphthenate. Like all oil-borne preservatives, Copper Naphthenate will protect timber from insects and organisms that cause decay. Additionally, Copper Naphthenate offers the benefit that it is water repellent and prevents changes in water content of the timber member. This prevents the splitting and checking caused by changes in moisture content. The heavy oil carrier of the Copper Naphthenate is noncorrosive to metal.

Waterborne preservatives such as Chromated-Copper-Arsenate (CCA) can be used for components that might be touched by pedestrians. This type of treatment provides a dry surface. However, there are several unfavorable characteristics of waterborne salt preservatives such as CCA. The most serious is that it reduces the modulus of rupture of the timber; consequently, the timber loses its ability to resist impact loading. Timber bridge design acknowledges this unique attribute by neglecting the impact factor that is used in the design of steel and concrete structures if the members in question are treated with an oilborne preservative. If a waterborne salt is used for members that are resisting impact loads, the engineer may want to consider the use of the AASHTO impact factor.
One other unfavorable characteristic is that waterborne salt preservatives are corrosive to metals. This causes concern about the fasteners. All of the hardware should be hot-dipped galvanized. Lastly, waterborne preservatives generally leave the wood more permeable and thus allow large and frequent moisture changes which will increase splitting and checking of the members.
INSPECTION

Inspection of bridges is the starting point for most repair and rehabilitation projects. Bridge inspection is so important and complex that its coverage is clearly beyond the scope of this paper. However, there are several aspects of bridge inspection unique to treated timber bridges and they will be briefly covered here. It is important to note however, that the very act of bridge inspection is controlled by the Federal Highway Administration and State Statutes. Any information presented herein must be seen as supporting State and Federal laws and procedures and any inconsistency must be resolved in favor of governmental laws and policies.

Highway bridges are by their very nature and use inevitably exposed to decay fungi. The use of preservative treated timber in bridge construction has significantly reduced the potential for decay and has extended the service life of timber bridges, but decay is still the leading cause for timber bridge replacement. For most timber bridges, a preventative maintenance program, including simple supplemental preservative treatments coordinated with the systematic inspections that are now mandatory, can significantly extend the service life of the bridge. Such a program is highly desirable, both from the standpoint of public safety and to help reduce the total cost of maintaining our infrastructure.

Decay in wood is caused by living fungi, which are simple plants having the unique capacity to break down and utilize wood cell wall material as food. The fungus in an area of decaying wood is generally invisible but present as a growing network of microscopic threads (hyphea) that penetrate randomly throughout the wood.

Once established, the fungus continues to grow as long as favorable conditions prevail. The requirements for growth are oxygen, favorable temperature, food, and water. Comparatively small amounts of oxygen are necessary for vigorous growth and even less is required for dormant existence, but decay fungi cannot survive without some free oxygen. Unfortunately, this essential requirement by the fungi is generally not subject to practical, manipulative control in bridge members. The primary exception is piling placed in water so that it remains completely submerged at all times.

A favorable temperature range is the second requirement for fungus growth. At freezing temperatures and below, fungi simply become dormant but remain alive and capable of a resumption of growth when prevailing temperatures rise above freezing. Growth rates gradually increase from near freezing to an optimum range at approximately 75° to 85° F, but growth drops off rapidly for most decay fungi as temperatures rise above 90° F. Only temperatures well in excess of 100° F are lethal for most decay fungi. Such high temperatures do not occur naturally in bridge members other than at the surface areas of dark wood exposed directly to summer sun. Thus, the temperature requirement also is not generally subject to control in bridge members to limit fungus growth. However, lethal high temperatures are an important factor in the elimination of living decay infections in new materials that are either kiln dried, steam seasoned, or preservative treated by heat and pressure methods.

An adequate supply of food is a third requirement for fungus growth. It is one of the essential needs of decay fungi that is subject to control and by which decay can be prevented. This is accomplished by preservative treatment of the wood.

Available water is the fourth requirement for initiation and spread of wood decay. Dry
wood will not decay although the widely used misnomer “dry-rot” may imply that possibility. Moisture content of wood is one of the most significant factors regarding wood decay because a considerable amount of water is required for fungus growth and in many instances, the moisture content of wood in service is subject to control.

Not only does the amount of water in wood directly control the possibility of fungus infection and growth, but it also is significant to the decay process in less direct ways. Prolonged or repeated wettings contribute to leaching and a consequent loss of natural decay resistance. Further, during the seasoning of large timbers, the loss of water is accompanied by shrinkage that normally results in the development of seasoning checks. Such checks may expose untreated parts of preservative-treated timbers and may also form water-trapping pockets that can become infection sites for decay fungi.

The tools and equipment needed for the inspection of timber bridges range from simple probes, such as a pocket knife, to state-of-the-art electronics. Again, a comprehensive paper on bridge inspection is beyond the scope of this effort. However, a few of the more common techniques for the inspection of timber bridges will be presented here. A systematic approach starting under the bridge with the substructure members is best. It allows the inspector to observe how the bridge reacts to the imposed vehicle loads passing over the bridge. The key is to look for deflections and looseness of joints and fasteners, and to listen for sounds.

Look for visual evidence of decay. Visual inspection requires a strong light as the details sought may be easily overlooked in poorly lit areas. Visible evidence of decay or probable decay will include three aspects: characteristic fungus fruiting structures; abnormal surface shrinkage or “sunken” faces; and insect activity.

Because of the very large number of similar structural areas in a bridge where decay-hazardous conditions may develop, the inspector must be alert for any evidence that suggests an increased likelihood that hazardous conditions have developed in specific areas or member parts. Such evidence should be a factor in the selection of probing or boring or coring sites. Visual evidence of decay-hazardous conditions includes water-marks of excessive wetting, rust stains, plants growing on the bridge, and joint interfaces.

After the initial survey for evidence of decay and conditions conducive to decay, more detailed examination of the questionable areas is in order. Probing with a moderately pointed tool as an awl or fine-bladed screwdriver is useful in detecting decay. Probing into end-grain faces or side-grain faces adjacent to joints may reveal the presence of decay by the excessive softness or lack of resistance to probe penetration. While detecting external decay is fairly simple, detecting internal decay is more involved, requiring the use of various devices, some of which are quite sophisticated.

It is difficult to establish a set of rules of thumb concerning how extensive an investigation should be made. This is a function of factors which the owner, or their engineer, must evaluate. There is relatively little information on strength loss correlated with either the amounts or stages of decay in bridge timbers. To calculate the residual strength of partially decayed structural members, it is important that the visible portion of the decay infection be outlined by the inspector, in both it’s cross sectional and longitudinal extent. In defining the area of decay, allowance must be made for nonvisible, incipient decay that extends outwardly from the visible decayed areas. The engineer can appraise the problem in relation to the type and intensity of stresses imposed on that portion of the member containing decay. The residual strength estimate should be based on the remaining uninfected material.
Considerable amounts of decay may not be service-limiting in some piling, abutments, and other members. However, in appraising the residual strength of members containing localized areas of decay, the engineers should be hesitant in assigning any strength value to the portion of wood showing definite evidence of decay.

Decay as a factor in bridge maintenance may be divided into three arbitrary categories based on the severity, or potential severity, of decay development. The first category includes existing decay that has progressed until severe loss of structural strength makes repair mandatory or restricted use of the bridge.

In the second category, immediate repair is not required for continued normal service. Although the incipient, moderate, or advanced decay may not be service-limiting, the bridge is likely to become more structurally damaged and load-limiting if corrective measures are not applied.

A third category that should be included in maintenance considerations is where decay has not started but where conditions conducive to decay are definitely present. The early development of damaging decay in individual hazardous areas can be predicted with little certainty, but in time decay will develop in such areas if the conditions favorable for fungus growth persist without correction.

A program of preventative maintenance for timber bridges offers considerable promise of: extending service life, reducing the frequency and extent of costly repairs, and improving bridge safety.
REPAIR

Member replacement may be indicated when its residual strength is found to be inadequate. If extended service is anticipated, the replacement member should be pressure treated with an appropriate preservative. When the defective member is removed, all adjacent and contacting members should be checked for possible decay infections that could have spread from the defective unit. Whenever repair or rehab of a timber structure is done, all effort should be made to include field-treatment of holes and cuts made as part of the work performed. There are many new products that are available for field treatment. Most of these products can be applied by maintenance crews and are available through most suppliers of treated timber.

In some instances, it may be more practical to remove and replace only the defective part of a member. In such cases the above recommendations apply. Additionally, it is important to remove an adequate length of the defective material to ensure the elimination of the infection.

Sister members or reinforcing elements may be added in establishing a load-carrying capacity to substitute for decay-weakened units. This would be logical where actual replacement is impractical due to the difficulty of working a new replacement into the position of the old member.

Some attempts at moisture control such as the use of roll roofing material or sheet metal as water-diverting cover or caps within the bridge structure, have not resulted in the improvement sought, and in some cases have contributed to increased wetting. For example, roll roofing used as stringer covers under decking or as piling top covers under caps may initially reduce wetting; however, as the material ages and is used repeatedly, it becomes permeable to water. In either new or old condition such material will inhibit the drying of any covered wood that was initially wet or that becomes wet.

Another example of faulty piling-top shielding is the use of sheet metal inserts between the pile top and cap in timber bents. The metal is usually installed over the pile before the cap is placed in position. The drift pin depresses the sheet metal slightly as it is driven through the cap and into the piling, and also creates a less than watertight hole in the metal. See Photograph No. 6 for a typical metal top cover for wing piles. This results in in a funneling of capillary water in the metal-to-cap interface to the piling wood around the drift pin hole. Nails through the metal into the pile top function similarly. The use of bituminous or asphaltic mastics as sealants or bedding

Photograph No. 6 - Covering the tops of piles with a metal cap can be detrimental to the pile. A better practice is to use asphaltic mastic.
compounds appear to be more effective for such functions.

**TIMBER STRINGER REPLACEMENT**

One of the most common repair procedures is the replacement of a broken or decayed timber stringer. This same method may be used to add an additional stringer adjacent to a stringer whose function has been impaired. See Figures 1, 2, and 3 for details of this procedure. This procedure requires ladders or scaffolding to provide access to the underside of the bridge. Place two jacks on each cap in adjoining bays next to the stringer to be replaced. Use steel plates under the deck to jack against. Jack up the deck until the stringer clears the cap 1/4 to 1/2 inch. Cut wedge out of one end of the stringer and bevel corners on the other end. If the new stringer is warped, always place camber up to provide bearing on all deck members. Place the wedge end of the stringer on one cap. Slide the stringer far enough to allow the other end of the stringer to be placed on the other cap. Pull stringer with come-along to final position. Remove jacks and come-along. Place the wedge cut at the wedged end of the stringer and nail decking to the stringer.

One question frequently asked by highway maintenance crews is the advisability of placing steel stringers in the same span as timber stringers. While this sounds like a logical suggestion it should be considered very carefully. The problem with doing this stems from the drastic differences in the stiffness of the two materials as measured by their respective modulus of elasticities (MOE). Steel has an MOE of approximately 30 million psi and Douglas Fir timber has an MOE of only 1.7 million psi. Putting stringers of two different materials, such as steel and timber, in the same span results in a system where they do not share equally in resisting the load. This makes rating the bridge complex. One has to assume that adjacent stringers of different materials deflect equally and that the load resisted is equal to the load necessary to deflect the two different stringers equally. It is readily apparent that it requires far less load to deflect the timber stringer than the steel stringer. There is an old saying amongst structural engineers that “stiffness attracts load.” It would be easy...
to have a situation where a steel stringer would fail before the adjacent timber stringers would fail. The mixing of stringers with differences in relative stiffness should be analyzed with some care.

**TIMBER CAP REPLACEMENT**

The replacement of timber caps on either abutments or intermediate timber bends is another common repair procedure. There are two different methods to be used for removing load from the existing cap. Details of these procedures are shown in Figures 4 and 5. These are jacking from cribbing and jacking from the bent piles. For replacement of abutment caps, place cribbing in front of the abutment, leaving room for the removal of the old cap and replacement of the new cap. For timber spans use a false cap to jack against. Spans with steel stringers jack against the bottom angle of the beams. Jack up the span about 1 to 2 inches. Cut off the drift pins connecting the cap to the piles using a Sawzall saw. Slide the old cap off the piling and move the new cap into position. Lower the jacks. Connect the cap to the piles with steel straps and drift pins.

The conditions which result in a cap failure can be decay. But, in a surprising number of instances, the cap fails as the abutment transfers earth pressure into the superstructure. This can be first observed as a twisting of the end of the cap. Many times this is mis-diagnosed as merely warping and twisting of the timber cap. Unfortunately, what is happening is that the cap is being restrained at the top by the superstructure and is being subjected to a large lateral load at the bottom of the cap by the piling. The pilings are being pushed by the earth pressure behind the abutment. As the cap starts to twist from these opposite forces it will start to crack. This will let moisture into the center of the member and decay will follow.
TIMBER CAP SCABS

Decay on the bearing ends of timber stringers can be repaired by increasing the width of bearing provided by the cap. This decay is generally the result of dirt allowed to build up on the cap between the stringers. Construction details are shown in Figures 6, 7, and 8. The conditions conducive to decay in the stringer must be corrected. The basic procedure is to attach a 6" x 12" to each side of an existing timber cap using split ring connectors and bolts. The existing cap must be sound timber. Raise the scab members into position and clamp to cap or use bolts to temporarily hold in position. Drill through the scabs and the cap. Remove the scab members and cut grooves for the shear rings. Put the scab members back into place and place bolts and washers to complete the repair. The primary resistance to the imposed load are the split-rings.

SHIMMING TIMBER PILES

There are many situations in the repair of timber bridges where the caps do not bear uniformly on the tops of the bearing piles. This condition can be caused by decay of the top of the pile or settlement of the pile or heaving of adjacent piles. Details for this procedure are shown in Figures 9 and 10. Frequently, in the repair of and rehab of timber bridges,
new piles are driven and pulled under the existing caps; this also results in uneven bearing. The construction procedure for the proper shimming between pile top and cap is not well understood by many repair crews. There is a misunderstanding that when shims are used the surfaces of the pile top are cut to match the bevel of the wedge. This is not correct. The correct construction procedure requires the use of sets of “matching wedges.” The wedges should be cut from good structural timber such as Douglas Fir. They should be pressure treated with Copper Naphthenate after cutting. Repair of bridges and the repair of heavy equipment each require some forethought to have the right parts for repair. Having several different sizes of wedges available prior to starting the bridge work is just good management.

![Diagram showing proper and improper shimming](image)

**Figure No. 10** - The correct method to shim the gap between pile top and cap is with two matching treated timber wedges.
REHABILITATION

The ability of a bridge to support the loads imposed on it starts with the deck. This is where the weight of the vehicle is transferred through the wheels to the structure. There may be a wearing course on the deck, but in most cases it does not contribute to the load-carrying capacity of the structure, it merely adds dead load. Bridge decks can be conveniently divided into two general classes, depending on how they function in transferring the wheel loads to other structural elements. In the first case, the deck itself provides all of the longitudinal bending resistance, as in the case of a slab span structure. The deck transfers the wheel loads directly to the transverse floor beams or substructure caps. In the other class, the deck provides transverse distribution of wheel loads to stringer elements. Most generally these stringer elements are longitudinal and bear on transverse floor beams of substructure caps.

Deck elements of most bridges are the first components to start to exhibit the effects of the traffic and the deicing chemicals. Most rehabilitation projects start with an in-depth inspection of the deck. This is generally a part of a systematic approach for looking at problems and possible solutions. Each bridge is unique and may have a variety of deficiencies. Bridges that are good candidates for rehab are generally in reasonable condition but have one or two glaring deficiencies. These bridges can often be placed in one or more of the following categories:

1. Roadway too narrow
2. Deck not structurally adequate
3. Stringers not structurally adequate
4. Railing system not adequate.

Many of the problems associated with bridge decks are independent of bridge type. This paper will look at several typical deficiencies in the deck and related elements and offer tested solutions to these problems. Any of the proposed solutions address multiple deficiencies, that is the rehab project may involve the replacement of the stringers, deck and railing system with all new components. Other solutions might only increase roadway width. These solutions will be presented in the form of case studies of actual projects where the procedures proposed were used.

Todd County

Todd County is a rural Minnesota county located in the center of the state. Bridge No. 89949, covered in this case history, is a steel beam span bridge with a concrete deck. It was structurally and hydraulically adequate, but was deficient in width. The curb-to-curb roadway width was only 22 feet and narrower than allowed by the geometric standards for the current average daily traffic (ADT). The County wanted to resurface the highway using 3R funds and had to provide the required roadway width. The two options were to replace the bridge with a new structure or widen the existing one. The estimated cost of the replacement structure was $74,000.

The existing bridge had an overall length of 30 feet and an out-to-out width of 25 feet. The abutments were concrete and both the main wall and the wing walls were supported by load-bearing piling. The renovation proposed was to remove the concrete curb and railing and
then widen the deck with prefabricated timber panels with a 10 kip timber railing attached. The deck panels were to be supported by pouring a bearing area on the abutment and wing wall.

The first step was to remove the curb and railing. There was a construction joint between the deck slab and the curb which made removal of the curb easy. The abutments were modified to support the deck panels. The back wall of the abutment extended about 2 feet outside the fascia edge of the deck slab. The wing walls were at about 45 degrees to the back wall. The portion of the back wall outside of the edge of the slab and a portion of the wingwall were cut down sufficiently to allow a bearing area to be poured so that the top of the panels matched the elevation of the concrete deck. Photograph No. 7 shows the new timber deck extending out over the edge of the original concrete deck.

The design of treated timber deck panels is covered in the AASHTO Standard Specifications For Highway Bridges. The panel is designed as a simple beam the width of which is the width of the tire plus twice the thickness of the deck. The width of a tire for an HS-20 load is 20 inches. The load that is applied to the simple beam is a wheel line, or one-half of the tabulated lane load for HS-20. The required thickness for this bridge is 14 inches. The effective span is less than 31 feet.

After the curb and rail had been removed, a 4-inch by 14-inch treated plank was attached to the fascia edge of each side of the bridge. This was accomplished using galvanized machine bolts and cinch-type anchors. This is shown in Figure No. 11. After the 14-inch plank was attached to the bridge, a plank half that width, or 7 inches, was attached to the bridge plank. This half-plank is called a splice plank. The other half of the splice plank is attached to the panel. The two splice planks are connected using 5/8-inch domehead drive spikes driven vertically through both splice planks. The 11/16-inch holes for the drive spikes are predrilled in the top splice plank. The 9/16-inch holes in the bottom splice plank are drilled in the field.

Two treated timber panels were prefabricated for shipping to the bridge site. The individual plank laminates were stair-stepped to follow the flare of the wing walls. This can be seen in Figure No. 12.
The planks in the panels are connected with 3/8-inch diameter ringshank dowels 15 inches in length. They are placed in a pattern so that there is one dowel every one foot longitudinally.

A treated-timber railing meeting AASHTO specifications for 10 kip loading was attached to each panel at the fabrication plant. The railing is shown in Figure No. 13. That detail has been altered slightly to show the new PL-1 type rail. This is the railing that has been crash-tested for use on federally funded projects. The actual railing on the Todd County bridge does not have the spacer block between the rail and the rail post.

At the plant, lifting eyebolts were put in place at the balance point of the assembled panels. When the panels were delivered to the jobsite, the only work remaining to be done was to drive the 5/8-inch drive spikes and to place the nuts and washers on the anchor bolts at the abutments.

The bridge was surfaced with a bituminous wearing course, which was crowned to provide deck drainage. The cost of the completed project in 1985 was $10,462, of which $7,200 was for material delivered to the site. Through a well-engineered rehabilitation project, this bridge now has the required 30 feet of roadway width and an HS-20 rating, and most importantly, it was done at a fraction of the replacement cost.
Morrison County

Morrison County is a rural county located in central Minnesota. It has an extensive county and township road system. The bridge featured in this case study was a 26-foot steel beam span with a concrete deck. The stringers were very inadequate structurally. The concrete abutments were in good condition and the bridge had sufficient hydraulic capacity. The roadway width was insufficient because it was less than 20 feet from curb to curb. The condition of the bridge was such that it would not support school buses, fire vehicles, or farm-to-market trucks. There were three alternatives considered: replacement with new bridge, replacement with large culverts, or rehabilitation. The rehab project was the most economical alternative.

The rehabilitation proposed was to remove the existing stringers, deck, and railing and replace them with a prefabricated dowel-laminated timber deck. See Photograph No. 8. The deck would be widened to provide the greatest possible roadway width that could be supported on the existing concrete abutments. The additional deck width would be supported on part of the wing walls. The bridge was constructed at 22 degrees right hand forward skew. The general layout of the bridge is shown in Figure No. 14.

The first step was to remove the existing span. Next, the tops of the wing walls were cut with a concrete saw. The concrete was broken out and then a new bridge seat was poured to support the additional deck width. Timber has an extremely high strength-to-weight ratio. This generally means that, for a given span, the depth of section for timber will be less than for any other type of material. This structure was no exception. The thickness of the required timber panel was only

![Photograph No. 8 - The existing concrete abutments were in good condition and the replacement of the steel stringers and concrete deck took this bridge off the list of deficient bridges.](image)

![Figure No. 14 - The roadway of the bridge was widened by supporting the additional width on the existing wing walls.](image)

![Figure No. 15 - A timber sleeper is used to adjust the finished elevation of the thinner deck panels. Also, the timber-to-timber connection is an easier connection to make with the large deck panels.](image)
Repair and Rehabilitation of Treated Timber Bridges

12 inches, compared to 18 inches for the steel stringers and concrete deck. The difference was compensated for by placing 6 inch by 12 inch timbers on the bridge seat under the deck panels. This can be seen in Figure No. 15.

This case history demonstrates that timber can be prefabricated to match site-specific conditions such as the 22 degree skew angle. See Photograph No. 9. The bridge deck was shipped to the job site in 4 panels, each of which was approximately 6.5 feet in width. These panels were placed by Morrison County Highway Department personnel using county equipment. Neither large complex equipment nor highly specialized skills are required to place this deck. See Photograph No. 10.

Sauk County

Sauk County, Wisconsin is near the Wisconsin Dells, which is a well-known tourist and recreational area. This section of Wisconsin has extremely seasonal traffic due to this attraction. One of the major routes serving this area is Sauk County Trunk Highway “A”. This case history involves a steel truss bridge on Trunk Highway “A” that was in need of a new deck. The existing concrete deck had deteriorated to the point that replacement was necessary. A reasonable detour did not exist for this structure, which prevented the closing of the structure during the summer season. The work would have to be done well after the normal construction season.

The structural system consisted of steel stringers which were supported by the floor beams of the truss. The stringers were not composite with the concrete deck. The stringers, floor beams, and remainder of the truss were structurally adequate. The sub-structures were also in good condition. The alternatives that were considered were to replace the deck in kind, that is with concrete, or to use treated timber. The estimated cost for a new concrete deck was $113,000.00.
The County decided to use treated timber and to do the work with their own crew and equipment. The design used prefabricated, dowel-laminated, timber panels which could be placed without removing the railing on the bridge. The panels were placed transverse to the stringers. The design of this type of deck is covered in AASHTO’s *Standard Specifications For Highway Bridges*. The material was Douglas Fir, No. 1 Plank and Joist, treated with creosote. The panels were prefabricated using 3-inch horizontal by 6-inch vertical members. The panels were approximately 6 feet in width and 27 feet in length.

The existing concrete deck was removed in small sections starting at one end of the structure. As a section 7 or 8 feet in length was removed, a timber panel was placed. While that panel was being secured to the stringers, the next section of concrete was removed. Using this method, the crews worked their way across the bridge. Some short panels were used at each end of the truss to bridge the gap between the last floor beam and the top of the concrete abutment. These were 3 feet in length and about 5.5 feet in width. This is shown in Figure No. 16.

The panels were attached to the stringers using 3/4 inch by 9 inch galvanized machine bolts with offset shoes. This detail can be seen in Figure No. 17. There is a compression spring between the nut and the offset shoe which compensates for any change in the thickness of the timber deck caused by changes in moisture content. The completed deck was surfaced with a bituminous wearing course.

An alternative method of attaching transverse timber deck panels to longitudinal steel stringers is shown in Figure No. 18. This method uses a timber block located...
between the stringers which bears on the top angles of the stringers. The block is attached to
the deck with either machine bolts or domehead bolts. This method has several advantages
over the use of offset shoes. Less precision is required in the location of the holes for the
bolts. This method is also less costly than the offset-shoe method, although the results are the
same. The timber block acts as a spring and keeps the deck secured to the stringers.

This project was constructed in the third week of November, 1982. It required only four
days of actual construction time. A summary of the costs is as follows:

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Equipment</td>
<td>$2,679.00</td>
</tr>
<tr>
<td>Labor</td>
<td>$6,000.00</td>
</tr>
<tr>
<td>Material</td>
<td>$21,344.00</td>
</tr>
<tr>
<td>TOTAL</td>
<td>$30,023.00</td>
</tr>
</tbody>
</table>

Deck Area: 2,511 square feet
Cost per sq. ft. of deck: $11.96

This project required that the road be closed for less than one week and was
constructed at one-third the cost of replacing the existing deck with concrete. A start-to-finish
history of this project is presented in Photograph Nos. 11 through 25.

**Photograph No. 11** - This bridge serving well-known tourist area in Sauk County needed a new
deck.

**Photograph No. 12** - The existing concrete deck was removed in sections approximately 7 feet
square.
Photograph No. 13 - The work was done by the County’s maintenance crews using County equipment.

Photograph No. 14 - The work was done in the last week of November so that the bridge would not be closed during a period of high ADT.

Photograph No. 15 - The longitudinal stringers and floor beams were inspected and found to be in good condition.

Photograph No. 16 - Once a portion of the old deck was removed the new deck installation was started.
Photograph No. 17 - The prefabricated bridge deck panels were designed to be placed under the existing railing without removing it.

Photograph No. 18 - The panels were 4 feet in width and designed to be lifted by the County’s equipment.

Photograph No. 19 - The removal of the old deck and the placement of the new deck were conducted concurrently.

Photograph No. 20 - The deck panels came from the fabricator with lifting eye bolts already installed.
Photograph No. 21 - Some minor welding was required to bridge over the floor beams between the ends of the stringers to form a continuous surface.

Photograph No. 22 - The dowel-laminated deck panels were shipped with banding to protect the panel from damage in handling.

Photograph No. 23 - A ship-lap joint was used to connect adjacent panels.

Photograph No. 24 - The original concrete deck was removed and this new deck was in place in one week.
Vernon County

This case history involves a bridge on Wisconsin Trunk Highway No. 131 in District No. 5. This bridge was deficient in structural capacity. The traffic levels were not sufficient to qualify this bridge for replacement at the time; however, some maintenance work was required. The deck and stringers were badly deteriorated. The proposed solution was to replace the stringers and deck with a longitudinal, dowel-laminated, treated timber panelized deck. This accomplished two things. It provided a new deck and, more importantly, the replacement of the concrete and steel with timber reduced the total dead load of the structure, which increased the resisting moment available for supporting the live load.

The existing stringers were removed and the floor beams cleaned and painted. There were five floor beams, creating four spans of approximately 16 feet each. The timber deck required to span from floor beam to floor beam was 10 inches thick. When timber panels are used in this manner, it is important that there be sufficient bearing area for the end of the panels on the floor beams. If there is insufficient area, this can be corrected by placing an inverted channel iron on the floor beams. This detail is shown in Figure No. 19.

The cost of material for this rehabilitation project was $13,240.00 and was purchased by the Wisconsin Department of Transportation in 1986.

Figure No. 19 - A channel iron was inverted over the floor beam to provide a wider bearing surface for the ends of the longitudinal panels to rest on.
Waseca County

This case history involves a very common type of deficient structure. Many concrete box culverts are structurally and hydraulically adequate, but, due to large increases in traffic volumes, are functionally deficient in terms of roadway width. This condition is illustrated by a project in Waseca County, Minnesota.

Prefabricated treated timber components that matched the as-built dimensions of the structure were shipped to the job site. These components were not extremely heavy, so they could be handled with small equipment such as tractor backhoes. The existing structure was left intact except for drilling a few holes for anchor bolts.

Holes were drilled on the inside of the sidewalls and used to attach horizontal 6 inch by 6 inch timbers, which in turn were attached to the timber box. Next, a 3 inch by 6 inch timber was attached to the outside face of the parapet wall over the culvert opening. This timber supported pieces of 3 inch by 12 inch planks, about 3 feet in length, which extended from the parapet wall to the top of the timber box. This can be seen in Figure No. 20.

The wall sections of the timber box were then placed into position and the top sections were placed on the wall sections. A 4 inch by 4 inch timber was attached to the concrete apron and to the inside of the timber wall sections. This timber was attached to the concrete apron with galvanized anchor bolts. The purpose of this timber was to prevent movement of the bottoms of the timber box side panels caused by earth pressure acting on side panels not connected to culvert bottom sections. The timber sections were placed on a prepared bed outside of the concrete apron. Once all of the box sections were set, the wing walls of the timber box were placed and held in position with deadmen.

This type of rehabilitation offers many advantages. The components can be designed to accommodate site-specific conditions. The wings can be positioned at any angle to the barrel and can be any length. The work can be done in freezing weather without any special considerations. The work can be done by road maintenance crews without special training or equipment. The construction sequence is illustrated in Photograph Nos. 26-34.

Figure No. 20 - This is a cross-section of the culvert extension showing the header block attached to the face of the concrete headwall and the ledger blocks attached to the oor of the apron of the culvert.

Photograph No. 26 - The first step is the installation of a header block to support members bridging to the first top section of the timber box culvert.
Photograph No. 27 - Next the side sections are placed and the ledger blocks holding the bottom are attached to the floor of the apron.

Photograph No. 28 - Horizontal braces connecting the side panels to the box are attached to the sides of the concrete box with Kwik Bolts.

Photograph No. 29 - The top sections are lowered into place to match up with the side sections.

Photograph No. 30 - The last top section with the parapet wall already attached is positioned.
Photograph No. 31 - Short treated timbers are placed bridging the gap between the concrete headwall and the start of the timber box culvert.

Photograph No. 32 - The bottom of the side sections are held in place by ledger blocks attached to the floor of the apron.

Photograph No. 33 - The wing wall is in position and the deadman is in position ready to be connected to the wing wall with tie rods.

Photograph No. 34 - The completed extension of a concrete box culvert using prefabricated treated timber components.
Sibley County

The bridges on our secondary road system are not currently being replaced at the same rate at which they appear to be deteriorating. This is not meant to be a criticism of our public works efforts but rather a realization that at current funding levels we, as engineers, must seek solutions that will extend the useful life of existing structures. Most of the research resources are being applied in the development of materials and designs that are directed toward construction of new structures and are not addressing the fundamental need to protect the public’s current investment in the infrastructure. The work detailed in this paper is an effort to develop a cost-effective procedure for improving load capacity and useful life of slab span timber bridges.

BACKGROUND

This procedure was developed for the rehabilitation of longitudinally laminated slab span timber bridges. This type of bridge represents over 70% of all timber bridges constructed in the United States since World War II. There are currently over 1,300 of this type of bridge off the trunk highway system only in the State of Minnesota, most of which were constructed since 1950. The design of this type of bridge consists of full sawn rough planks which are placed with the wide face vertical and are mechanically laminated together using large metal fasteners.

Prior to 1970, the lamination was done at the bridge site and the planks were nailed together using large spikes. This design is generally referred to as nail-laminated. After 1970 most slab span longitudinal deck bridges were prefabricated into large panels. These panels were shipped to the bridge site and were connected to each other by means of a ship-lap joint. This type of construction uses large steel dowels that are pressed into pre-drilled holes using a hydraulic press. This type of bridge is generally referred to as a dowel-laminated bridge.

The design of nail-laminated slab span bridges was based on an assumed lateral distribution of a line of wheel-loads equal to the width of the tire plus one thickness of the deck. Generally, every tenth laminae extends over the support in multi-span structures. The actual design calculations considered all laminae to be continuous at the region of maximum moment. Normally, this type of bridge would include a spreader beam placed at mid-span and attached to the deck with bolts. The purpose of the spreader beam is to increase the transverse distribution of the wheel loads.

Bridges of this type over time tend to lose some of the ability to distribute

Photograph No. 35 - A three span bridge in Sibley County has been used to demonstrate a new technique for rehabilitating a nailed in the field longitudinal slab span. The platform under the bridge supports the instruments used in the load tests.
the wheel loads laterally. This condition impacts the serviceability of the wearing surface. Some of this loss is the result of fiber crushing by the connectors. This may be the result of over loading of the bridge, poor field fabrication or inadequate design. The total resisting moment of all of the longitudinal members is still intact, however, fewer individual members are supporting the wheel loads. Greater live load deflections result.

One of the possible solutions for this condition was developed by the Ontario Ministry of Transportation and Communication. This technique is generally referred to as “post-tensioning” of wood. This procedure has been used successfully in re-establishing the transverse load distribution of nail-laminated timber decks. While this procedure is very effective for restoring load capacity and serviceability of timber bridges, the cost of that type of construction makes it prohibitive for rehabilitation. Because of the cost of that type of construction, most of the research and development has been directed toward new construction and not rehabilitation.

DEVELOPMENT

This investigation centered on methods to increase the transverse stiffness of longitudinal laminated slab-span bridges. The basic concept is to place an additional timber deck on the existing deck, but transverse to the existing deck. This type of timber deck will hereafter be referred to as a spreader deck. Testing of quarter-scale model indicated that favorable results could be obtained with a deck thickness of 75 mm (3 in) to 100 mm (4 in). A full scale test bridge was needed to investigate the effectiveness of the proposed solution. Any bridge selected for testing this procedure must have sufficient current load capacity to tolerate the additional dead load of the additional timber deck. The bridge would be load tested in its present condition and then retested after the spreader deck was installed. The live load deflections measured prior to the installations were used to determine the longitudinal stiffness of the existing bridge. The live load deflection taken with the spreader deck installed were used to calculate the increase in transverse load distribution.

SOUTH BRANCH OF RUSH RIVER BRIDGE

Sibley County in Minnesota has many longitudinal slab-span timber bridges on its road system. The bridge over the South Branch of Rush River is typical of the timber bridges in Sibley. It is a three span nail-laminated longitudinal slab-span bridge. It is supported with timber substructures and has three 7.32 metre (24 ft) spans set at 40° RHF skew with an 8.5 metre (28 ft) clear roadway. The railing system consists of galvanized plate beam rail supported by 200 mm (8 in) x 200 mm (8 in) curb and 200 mm (8 in) scupper blocks. The section of County State Aid Photograph No. 36 - Part of this project was the installation of a new crash-tested bridge railing system. Here the split rings are being installed.
Highway No. 18 containing this bridge was scheduled for upgrading and bituminous surfacing. This bridge had sufficient capacity to support the additional timber deck and an AASHTO HS-20 live load if the additional deck did not increase live load distribution sufficiently to offset the additional dead load.

**CONSTRUCTION OF SPREADER DECK**

The deck was prefabricated into panels approximately 1 meter in width. The individual laminates were Douglas Fir No. 1. The panels were constructed with the laminates parallel to the angle of skew; this eliminated the need for short panels at the ends of the bridge. The material was 3 inch nominal which was S1S to 70 mm (2.75 in). The first two panels on the west end of the bridge are 100 mm (4 in) in depth and the balance of the panels are 75 mm (3 in). The additional thickness on the west end of the bridge was to offset some additional bituminous surfacing due to a change in profile grade over the structure. The panels were 9.8 metres (32 ft) in length and designed to be placed between the existing timber curbs. Timber curb boxes were constructed with treated timber to facilitate deck drainage into existing scupper openings. See Figure No. 21 for typical cross section of deck.

The original bituminous wearing surface was removed from the bridge. The uncovered deck was remarkably clean and very little additional work was required to prepare the existing deck. The surface of laminated timber decks are irregular due to the manufacturing and construction tolerances both in the sawing of the timbers and in the construction of the decks. Those irregularities are generally placed in the top side of the panels for several reasons. First, this allows the bottom ends of the panels to bear evenly on the supports. Next, it allows all members to bear on the spreader beam. Last, the irregularities provide many gripping surfaces and edges or the bituminous surface to adhere to. All of these aspects of irregularities are positive.

The irregularities inherent in timber decks present a major problem to the uniform load transverse from one timber deck to another. The spreader deck was placed on the existing longitudinal deck. It was connected to the existing deck with 19 mm (3/4 in) diameter through bolts at the outside edge and with 16 mm (5/8 in) diameter dome-head drive-spikes in a pattern of approximately .75 meters (3 ft) on centers. The voids between the two decks were filled with near cement pumped through 32 mm (1-1/4 in) diameter holes through the spreader deck. These holes were approximately .75 meters (3 ft) on center, centered between the bolts and drive spikes attaching the spreader deck. The neat cement was pumped with a mud-jack pump. A systematic process of tapping on the deck with a large hammer to detect any hollow sound was used to ensure complete filling of all voids. The bridge was test loaded after completion of construction and curing of the grout.

![Figure No. 21 - A cross-section of the rail section for the bridge showing the overhang of the new deck.](image-url)
TESTING AND EVALUATION

The bridge was tested loaded using a tandem axle grave truck placed longitudinally with the center of the tandem axle group placed at the center of the span. The truck was placed transversely at three different positions, first with the wheels placed 230 mm (9 in) from the curb in the north lane and the same distance from the curb in the south lane, and then with the truck centered in the roadway width. Deck deflections were measured with self-leveling level along two lines, one at the center of the span along the angle of skew and at a line 100 mm (3 ft) from center of span again along angle of skew.

The vehicle used in the test produced a bending moment in the deck 12.5% greater than a standard AASHTO HS-20 design vehicle. The total weight of the vehicle was 22,155 kg (48,800 pounds) with 7,355 kg (16,200 pounds) on the front axle and 14,800 kg (32,600 pounds) on the rear tandem. The apparent modulus of elasticity (MOE) of the original deck system was computed based on the observed deflections and an assumed distribution width. The assumed distribution width for the purposes of this analysis is width of the wheel and two times the deck thickness. The original deck as existed prior to the installation of the spreader deck had an apparent MOE of approximately 1,700,000, indicating that the assumed distribution width is less than actual.

The distribution width after the installation of the spreader deck using the apparent MOE of the existing longitudinal deck is 1.85 meters (73 in) or about 166% of the AASHTO distribution width. This indicates that the spreader deck made a substantial increase in the transverse distribution of the wheel loads.

The total cost of this rehabilitation was less than $54.00 per square meter ($5.00 per square foot) of deck area including materials and installation.
ADDITIONAL APPLICATIONS

This same concept has been used on another bridge in Sibley County with some additional benefits. The second application is on a bridge where some additional roadway width was desirable. To accomplish this, the existing curb and railing system was removed. The new spreader deck was designed to overhang the existing deck by 600 mm (2 ft) on each side of the bridge. A new modern railing system was then attached to the new deck.

Photograph No. 39 - This bridge has been the subject of considerable research and testing by the University of Minnesota.

Photograph No. 40 - This rehabilitation of the existing structure brought the structure up to the standards needed for the widening and overlaying of this County highway.