Upgrading and Recycling of Pin-Connected Truss Bridge by Pin Replacement

SHAHIN TAAVONI

Rehabilitation of the Carroll Road Bridge, a wrought iron Pratt through-truss bridge, built in 1879, is described. The goal was to preserve its historical character while upgrading the live-load capacity to meet all Maryland requirements for legal loads at the operating stress level. Inspection revealed that tension members had moved substantially; consequently, the load was no longer equally distributed. This effect was taken into account in pin and truss member analyses performed with the assistance of software developed by the author. Results indicated that the pins had minimal live-load capacity. To upgrade pin capacity, two methods were investigated. The first was to prevent transfer of all or part of the loads to the understrength pins. However, this could only be achieved by dramatically altering the appearance of the bridge and was, therefore, unacceptable. The second was to increase pin capacity by either prestressing or replacement. Prestressing was rejected because it could increase pin shear capacity, but not bending capacity. The only alternative was to replace all the pins with ones of higher strength. In-place replacement was cost prohibitive and time consuming, so the bridge was dismantled, and the two trusses transferred to a working space for repair. On completion of repairs and incidental substructure work, the bridge was reassembled and reopened to traffic. Three major conclusions emerged. First, in older bridge trusses, original symmetrical arrangement of the parallel components about the centerline of the pins usually no longer exists, resulting in a reduction of the load-rating capacity of truss members and pins. Second, in most cases, replacement of pins is the only acceptable solution. Third, the pin replacement method used was efficient, expeditious, economical, and suitable for similar situations.

This paper describes the rehabilitation study and subsequent rehabilitation of the 113-year-old Carroll Road Bridge. This wrought iron Pratt through-truss bridge crosses Carroll Run in a rural section of northern Baltimore County, Maryland. The bridge spans 92 ft and is supported at each end by stone masonry gravity abutments. The trusses consist of eight 11 ft 6 in. panels fabricated of pin-connected eye bars and riveted channels with lacing bars.

Most of the primary truss members have two components. Lower chord tension members consist of pairs of parallel eye bars, and compression members have two channels with lacing bars or batten plates riveted between the channels. Some of the diagonal tension members have an additional component parallel to the original pair, which was installed during previous repairs.

Spaced at 17 ft from center to center, the trusses accommodate a timber deck that provides a 14 ft 6 in. clear roadway between timber curbs. The deck is supported by stringers that transfer the loads to floorbeams. At each panel point, the floorbeams are suspended from the lower truss pins by U-bolts. Pin sizes are 2 and 2½ in. in diameter for upper chord and lower chord pins, respectively. Typical truss elevation and pin connection before rehabilitation are shown in Figures 1 and 2, respectively.

Kennedy, Porter & Associates, Consulting Engineers, 2319 Maryland Ave., Baltimore, Md. 21218.

PRELIMINARY REHABILITATION STUDY

In 1988 Kennedy, Porter & Associates (KPA) was retained by the Baltimore County Department of Public Works to make a preliminary rehabilitation study of the Carroll Road Bridge and seven other similar pin-connected wrought iron Pratt through-truss bridges. The goal was to preserve the historic character of the bridges and upgrade the live load capacity so that all Maryland legal loads could be sustained at the operating stress level.

As part of the rehabilitation study, it was necessary to check the load-bearing capacity of the pins and other connections, along with all truss and floor members, to ensure that all could sustain the proposed increase in live load.

Before the actual analyses could begin, a field visit to each bridge was necessary to measure pin size, evaluate the condition, and measure the spacing between components of the various members that bear on each pin. Small samples of material were taken from the pins and other truss members by lightly filing the surfaces. The samples were then sent to a laboratory for chemical analysis and metal identification. Because of the limited size of the recovered samples, it was not possible to perform mechanical tests. On the basis of the results of the chemical analyses and research of the technical literature, mechanical properties of the pins and other members were determined.

Research also indicated that the original design philosophy was to space adjoining truss members symmetrically about the pins (*1*). The symmetrical spacing was apparently adopted for the following reasons:

- To minimize pin stresses; and
- To induce equal strains and, hence, an equal force distribution through both components of any given truss member.

In the analyses, an equal distribution of force was assumed between parallel components of a particular truss member. Each pin was isolated as a free body with the truss member forces maintaining static equilibrium. Shears and moments were then computed on the basis of classical elastic assumptions.

The analysis was complicated because the pin loadings are a function of the placement of the truss members with respect to each other and the enormous quantity of possible live-load situations. With several pins in each truss and 11 sections of each pin to search for maximum bending and shear, the computations became extremely tedious. To expedite the process, a computer program

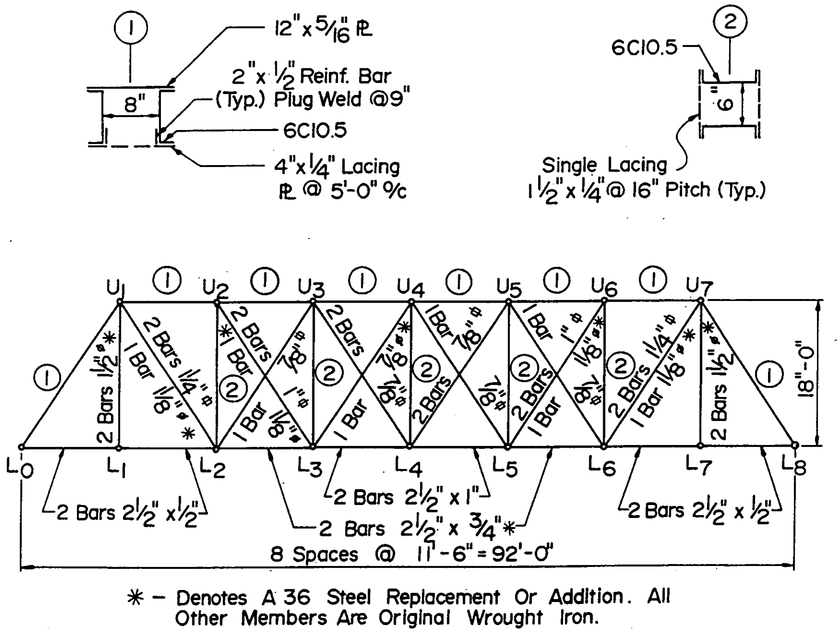

FIGURE 1 Typical truss elevation before rehabilitation.

was developed that completely evaluates truss pins under both static and moving loads (*2*).

In the pin analysis, each pin was treated as a beam with each vertical member component applying a vertical load, each horizontal member component applying a horizontal load, and each diagonal member component applying a vertical and a horizontal load. The computer program was used to calculate the vertical and horizontal components of bending and shear and the resulting bending and shear vector at each section of the pin, as a vehicle moves along the bridge. Subsequently, the maximum bending and shear at each pin were calculated and compared with the pin's capacity. These values allowed a rating value to be given to each pin. Note that as a vehicle moves along the bridge the bending and shear vectors at each section of a particular pin change direction. As a result, the live-load bending and shear vectors are not necessarily in the same direction as the corresponding dead load bending and shear vectors. Therefore, the usual method used for establishing the member rating could not be adopted. As a result, a vectorial analysis approach was adopted in the computer program for calculation of the bending and shear ratings. The computer analysis determined that in all cases the pins had a minimal live-load capacity and governed the rating of these bridges.

In spite of the analysis showing that some pins had very little live-load capacity, no pins have failed in any of these structures while operating under substantially higher live loads for the following reasons:

- The bridge was analyzed using a two-dimensional model, although in reality the two trusses and upper and lower bracings act together in a three-dimensional manner. Therefore, there are many different paths for the transfer of load from distressed pins to other less stressed parts of the structure.
- Ultimate plastic capacity of the pins is substantially higher than the elastic working capacity used in the capacity rating of the pins.

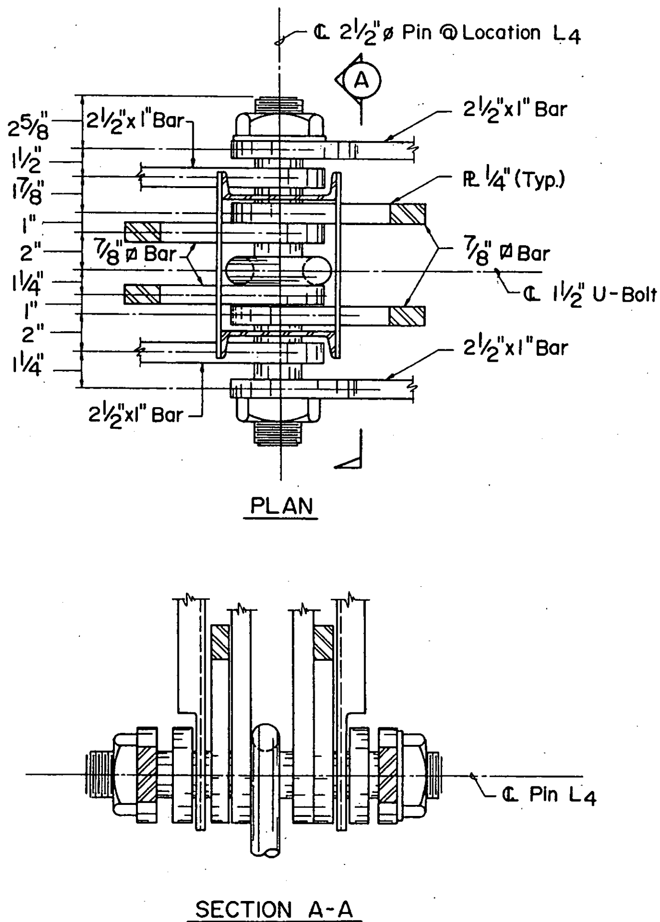

FIGURE 2 Typical pin connection at Joint L4 before rehabilitation.

• Because of the restricted width of the bridge, vehicles move slowly on the bridge. Therefore, the impact factor would be less than that stipulated by AASHTO.

FINAL REHABILITATION STUDY AND DESIGN

In 1990 the Department of Public Works decided to rehabilitate the Carroll Road Bridge to sustain all Maryland legal loads at the operating stress level. KPA was retained to provide the final rehabilitation study and contract documents for upgrading this structure. The main objectives of the design were to

• Preserve the historic character and original shape and form of the bridge because it is eligible for listing in the National Historic Register,
• Keep cost as low as possible,
• Protect the surrounding environment from the impact of the construction work in the area of the bridge, and
• Rehabilitate the bridge as quickly as possible.

For the final design, it was necessary to assess the load-carrying capacity of the pins and truss members as accurately as possible. Therefore, the truss pin analysis method used in the preliminary study was carefully reexamined. From this reexamination it became apparent that tension members, originally consisting of pairs of parallel eye bars placed symmetrically about the centerline of the pins, had in some cases moved substantially and were no longer symmetrical (see Figure 2). This condition has apparently resulted from the dynamic effects of the live load. As a result, pin deflections at points of contact with parallel truss member components were no longer equal. Consequently, the load was not equally distributed between parallel components, as was assumed in the preliminary study.

The original symmetric spacing of parallel components has also been disrupted over the years by various repairs and strengthening attempts that have added an additional component to some of the diagonal members without consideration of the effect on the pins and adjacent members (see Figure 1). This also causes an unequal distribution of force between parallel components of the diagonal members.

Note that fatigue and corrosion deformations of each parallel component (eye bar) surrounding the pins at the member ends may also cause unequal distribution of forces between or among parallel members of the truss. In other words, member ends may undergo a different sum of deformations as a result of different degrees of corrosion and fatigue deformations. However, visual inspection of the truss members and eye bars revealed that members were only moderately corroded, with no measurable loss of section. This was because most of the truss members were made of wrought iron and were well protected by paint. In addition, ultrasonic tests on all eye bars and other critical connections were conducted, and no fatigue cracks or distress were observed. Therefore, the effect of fatigue and corrosion deformations on load distribution was considered insignificant and was neglected in the analysis.

A method for distributing the force to each parallel component of a particular truss member on the basis of its location on the pin was developed. By means of this method, if the components of a truss member were not placed symmetrically about the centerline of the pin, the load would be distributed unequally between them.

The effect of unequal distribution of force between parallel components of a particular truss member was taken into account in both pin and truss member analyses. This was achieved with the help of a second computer program. This computer program is capable of providing a comprehensive analysis and rating of all of the pins and truss members (*3*).

The computer analysis of the Carroll Road Bridge, performed during the final design, determined that some of the pins and truss members had smaller rating capacities than originally found in the preliminary study (on the basis of the assumption of an equal distribution of load between different components of a truss member). The most profound result of the analysis was that most of the truss pins had minimal live-load capacity and were not capable of carrying the proposed increase in live load.

To verify the accuracy of the analysis, the Baltimore County Department of Public Works originally wanted to instrument the bridge, but because of insufficient funds the idea was abandoned.

Overall rating of the bridge was governed by the upper corner pin (pin U1 in Figure 3). The operating rating capacity of this pin for H and HS trucks was 3 and 4 tons, respectively. Other members with insufficient capacity were the deck, stringers, floor beams, and some of the truss members. This insufficiency was due to deterioration of the members or the proposed increase in the design live loads.

Replacement of the bridge with a new structure was not desirable because of the structure's historic significance. Therefore, it was necessary to identify the best rehabilitation alternative.

Retrofitting or replacement of the truss members, deck, and floor system is routine and straightforward. The most challenging aspect of the design was how to upgrade the pin capacity. Two methods were investigated:

1. Preventing transfer of all or part of the loads to understrength pins, and
2. Increasing the pin capacity.

The first method involves one of the following alternatives:

• Superposition of an additional structure, such as an arch, within the existing truss to carry a large portion of bridge live load and transfer it to the abutments;
• Placement of girders under the truss; or
• Prestressing the bottom chord of the truss to release the dead load and part of the live load on the bottom chord members and pins (*4*).

The first method would significantly change the bridge appearance and was therefore not acceptable from a historic preservation standpoint. Installation of girders under the existing truss was not possible because of intrusion on the hydraulic opening. Prestressing the truss bottom chord could reduce the load imposed on the bottom pins, thereby increasing the capacity, but could not release the load on the top chord pins to the desired amount.

The second method (i.e., increasing pin capacity) could have been achieved by prestressing or replacing the pins with new, higher strength pins. Because the prestressing method could only increase the pin shear capacity and not bending capacity, this method was discarded. Consequently, the only viable alternative was to replace all the pins with equal diameter pins of higher strength, and therefore higher capacity, material (FY = 100 ksi). Replacing the pins with larger diameter pins would also increase the pin capacity but would require increasing the eye bar hole diameter. This would require replacing the existing eye bars with larger diameter eye bars or increasing the eye bar hole diameter of the existing eye bars. These solutions were not desirable or acceptable.

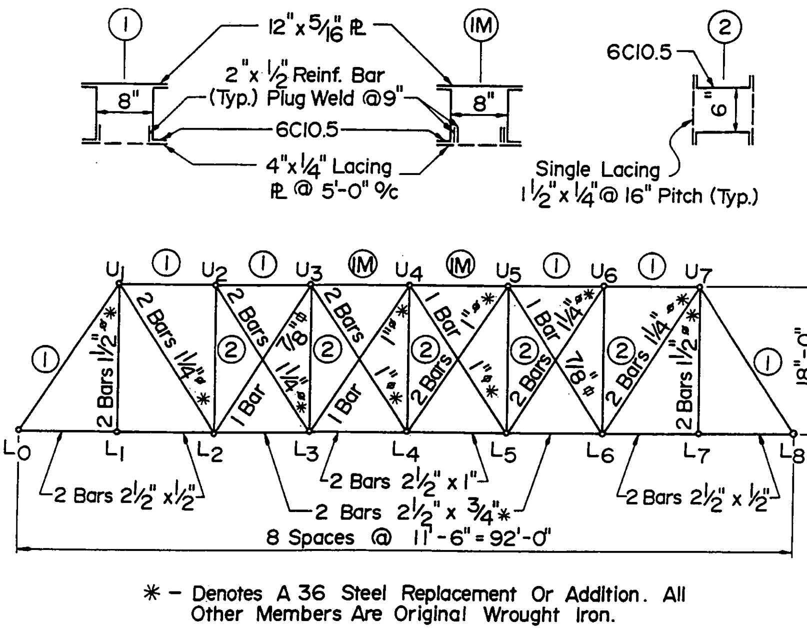

FIGURE 3 Typical truss elevation after rehabilitation.

To identify the best procedure for the replacement of the pins, several methods were considered, including:

1. Replacing pins in place;
2. Removing the complete bridge superstructure, transferring it to a work space, and replacing the pins; and
3. Dismantling the bridge, removing each truss completely, transferring them to a work space, and replacing the pins.

In-place replacement of pins requires shoring and supporting the entire bridge. This is because removal of one pin transforms the entire bridge into an unstable mechanism. This method proved to be cost prohibitive, time consuming, and obstructive to the normal flow of water. Removing the complete bridge superstructure could be done without any additional support for trusses during the lifting operation. However, the complete removal would require use of large cranes and removal of several trees near the bridge. The method finally selected was to dismantle the bridge and transfer each of the two trusses to a working space for pin replacement and repair. This method had the least adverse effects to the environment and was the most cost effective.

To reduce the stresses on the pins, optimize the existing truss member capacities, and return the trusses to their original form and shape, it was decided to remove all diagonal members that consisted of three components. These members were replaced with two components of adequate capacity, reinstating the symmetrical arrangement of truss member components about the centerline of the pins. Movement of the truss member components along the axis of the pins was evident on the existing structure and was attributed to the dynamic effect of the live load. To prevent this condition from occurring again, the rehabilitation introduced spacers where gaps between components existed. The spacers ensured that a symmetrical arrangement of member components was maintained. Figure 3 shows the members of the truss that were replaced or strengthened at the time of the rehabilitation. Because of deterioration of the deck, stringers, and severe corrosion of the floor beams, all of these members were replaced.

The existing timber deck was replaced with a new heavier timber deck of higher capacity. The decision to use timber deck to preserve the historical character of the bridge was taken. The assumed future service life of the rehabilitated bridge is 25 years.

CONSTRUCTION

Rehabilitation of the Carroll Road Bridge began in October 1990. The first step was to dismantle portions of the bridge. The existing timber deck and stringers were first removed, transferred to a working area behind the abutment, and later used as a horizontal platform for supporting the trusses during repair. Then, floorbeams and supporting U-bolts were removed and discarded. Next, the top and bottom lateral bracing was released and stored for future use. During the dismantling operation, Truss B was supported by guy wires at four points at the top chord joints and four points at the bottom chord. Truss A was supported by guy wires at two points at the top and two points at the bottom of the truss, in addition to the support provided by the lifting crane (See Figure 4). The guy wires were anchored to precast concrete deadmen, which were placed around the bridge. Figure 5 shows the connections of the guy wire to the concrete deadman and the top chord of the truss.

To strengthen the trusses during dismantling and lifting operations and to prevent buckling, two strongbacks were used for each truss (Figure 6). The first strongback was close to the lower chord and consisted of two channels placed horizontally and connected to the vertical posts by two 3/4 in. bolts. The second strongback consisted of two angles bolted to the vertical posts in the mid-height area of the truss.

To prevent any overstress in the truss members during the lifting operations, the dead load of the truss was transferred uniformly from the top joints of the truss to a lifting crane by a spreader beam.

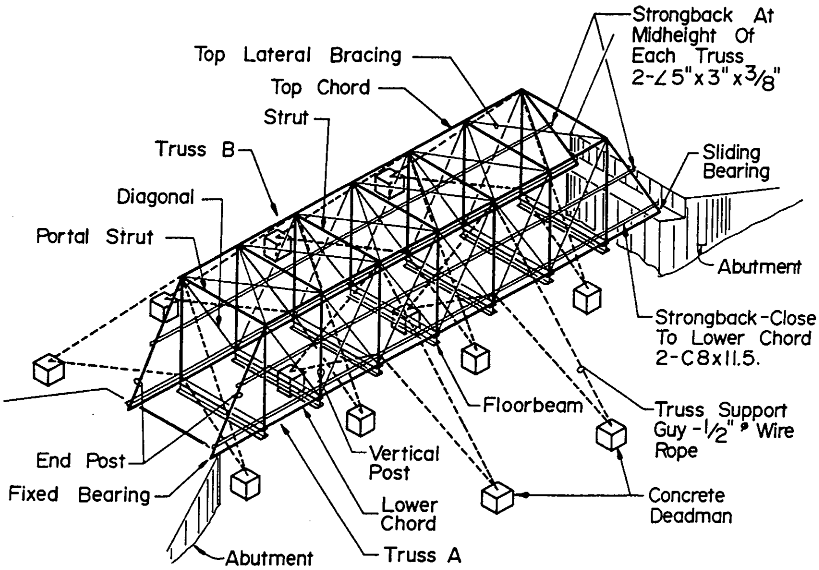

FIGURE 4 Truss stabilizing system during dismantling and re-erection of bridge.

Truss A was removed first and placed horizontally on the prepared platform, followed by Truss B. The entire dismantling of the bridge was done in one day.

Before removal of the pins, the members to be replaced were measured to determine their exact length, and the temperature was recorded. This information was later used in rebuilding these members to exact lengths, ensuring no change in the truss geometry.

Removal of the pins after more than 100 years of service proved to be a difficult task because of corrosion and permanent deflection of the pins. A special jack and sometimes heating were used for removing the pins.

After being removed, the pins were thoroughly inspected. Inspection revealed that some of the highly stressed pins had permanent deflections in the range of 1/8 to 1/4 in. This observation supported the theoretical calculations that indicated that these pins had been stressed beyond their elastic limits under live load. Several of the pins were corroded and pitted up to 1/4 in. deep; however, most were in good condition. Three pins from the upper chord and three from the lower chord were selected and sent to a laboratory for tensile and shear testing. Test results indicated that minimum yield stress was approximately 26 ksi. This value was close to what had been assumed in the rating analysis.

After removal of the pins and some other truss members, the remaining members were enclosed in a containment tent, sand blasted, and painted. Following the replacement of the removed members and the pins, the trusses were ready to be reerected.

The trusses were supported by lateral guy wires during the reerection procedures. The top and bottom lateral bracing, new steel floor beams, stringers, and timber deck were installed, and the bridge was reopened to traffic.

The overall cost of the rehabilitation of the bridge was close to $300,000.00. The operating rating for H and HS trucks was 23 and 37 tons, respectively. The member governing the rating of the rehabilitated bridge was bottom chord member L3-L4.

CONCLUSIONS

From this rehabilitation work, the following conclusions have been drawn:

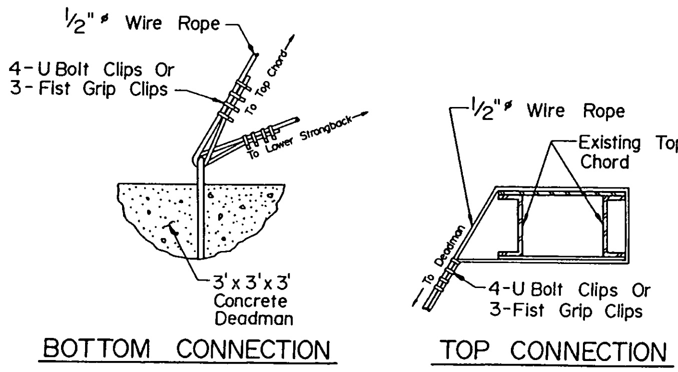

FIGURE 5 Deadman and truss connections.

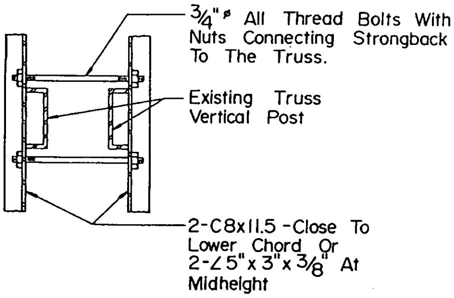

FIGURE 6 Strongback system.

• In old trusses, the original symmetrical arrangement of parallel truss member components about the centerline of the pins usually no longer exists. This results in an unequal distribution of loads in the parallel components of truss members and reduces load rating capacity of truss members as well as pins. This effect, often neglected in design and rating, should be taken into account.

• The rehabilitation of antique truss bridges requires special consideration of members that have been previously strengthened. In cases in which two component members have had a subsequent third component installed (as was the case at the subject structure), it is advantageous to replace completely the three-component members and return to a two component system. The new two-component member would be designed to sustain the proposed loads and be placed symmetrically about the centerline of the pin. This retrofit preserves the original character of the structure, is aesthetically appealing, and is preferable from a structural standpoint.

• It appears that the design and load rating of pin-connected trusses are often governed by the capacity of the pins. In cases in which the original form and shape of the truss must be preserved for historical reasons, pin replacement is the only practical solution.

• To eliminate fluctuations in the loads imposed on the pins and maintain the symmetry of the structure, it is advisable to introduce spacers where a gap exists between components bearing on the pin.

• The pin replacement method adopted for this project was efficient, expeditious, and economical, with the least adverse impact on the environment. Also it may be used in similar rehabilitation situations.

• According to the FHWA publications (*5*, p.9.8.45, *6*) definition, members with only one or two eye bars should be considered fracture critical members. In addition to the pin, each truss joint includes the ends of eye bar components, the ends of vertical components, connection plates, and so forth. These joint components are particularly vulnerable to fatigue stresses and corrosion and thereby contribute to additional nonredundancy of the trusses. It is critical for both rehabilitated and nonrehabilitated truss bridges that joints are frequently and meticulously inspected for any sign of distress due to fatigue or corrosion. Any required remedial action should be implemented immediately to ensure the safe operation of the structure.

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