English River Bridge
on
Nutmeg Avenue
Washington County, Iowa

Structural Study

Submitted to

Washington County Secondary Roads Department
&
Washington County Conservation Board

by

SHIVE HATTERY

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108140-0

INTRODUCTION

The Washington County Secondary Roads Department wishes to evaluate the English River Bridge south of Kalona, Iowa, for inclusion in a recreational trials corridor. The bridge structure, noted to be constructed in 1887, was closed to through traffic in 2001/2002. Due to the age of the facility and a previous recommendation to close the bridge to traffic, there was concern that the condition of the structure would not allow for use in a trails environment. The purpose of the study is to provide an opinion as to whether the structure could be incorporated into the trail system. Tasks associated with the study include:

- Field observation to collect information to allow for a structural analysis of the bridge and approaches and to observe for damage and deterioration.
- Structural analysis to provide an opinion as to the capacity of the structures.
- Evaluate whether strengthening of the structure (if necessary) is a viable alternative.
- Develop an opinion of cost for modifications to the structure.

The desired load capacity for pedestrian use is 85 pounds per square foot (psf) as designated by The American Association of State Highway and Transportation Officials (AASHTO). The desired load capacity to allow for maintenance vehicles is a single concentrated load of 10,000 pounds with an impact factor of 30%.

Location:

The bridge is located approximately 1.5 miles east of Highway 1 and 1.5 mile south of Highway 22 near Kalona, Iowa. An aerial view provided by the County is included on the Appendix. The legal description is:

Nutmeg Avenue over English River
Section 17
Township 77 North
Range 7 West
Washington County, Iowa

Description:

The structure consists of two primary elements; a through truss spanning 119'-0", and a series of six approach spans from the north of 28 feet each, for an approach length of 168'-0". The pinned truss is of Pratt configuration, characterized by upper chords and vertical members acting in compression, and lower chords and diagonals acting in tension.[1] The through aspect meaning that the top chord is sufficiently distant over the deck and bottom chord that traffic could pass

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[1] Historic Bridges of Iowa: Pratt Trusses

below the top chord or through the box formed by the top chord and it's bracing members. The south truss bearing consists of an abutment constructed of large rectangular stones. The north truss bearing consists of a single tubular steel pier for each truss.

The approach spans were formed with standard shaped steel beams at 2'-4" centers spanning between girders spaced at 28 foot centers. The girders were supported with braced steel bent systems anchored to concrete footings. The deck of both the truss and approach span is rough hewn timber 3x12's. A summary of element sizes and locations is included in the Appendix.

RESEARCH OF PERIOD STRUCTURES

Materials

Metallurgical tests of the framing elements were not taken, however we are fairly certain the framing systems consist of a combination of wrought iron and structural steel elements. The reasons being:

> Year of Construction (1887) - Steel beams were introduced for general use in 1884[2,3], however the availability of steel would have been somewhat limited in 1887. Prior to the introduction of steel, the material of choice was either a form of cast iron or wrought iron. Cast iron, as the name implies, was cast into the desired shape. Wrought iron was mechanically worked and mechanically formed into the desired shape. Cast iron tended to be strong in compression, but however lacked in ductility and was weak in tension. Early iron bridges were often constructed of cast iron compression members and wrought iron tension members in the shape of bars. Cast iron use in bridges was discontinued circa 1880[4]. Rolled wrought iron shapes (channels and standard beam sections) were introduced in 1861[5] and remained in use until the end of the century, at which point steel was in widespread use.
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> Observations - Both steel and wrought iron structural elements are rolled, which leads to similar shapes. Wrought iron members are also typically built up by riveting together iron plates or small rolled shapes[6]. Delaminated edges resulting from mechanical working can sometimes be observed in wrought iron elements. We did observe some delamination of one of the compression flange cover plates. The built up truss compression elements and the delaminated plate are indications that the truss elements are wrought iron.
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> We observed a manufacturer's stamp on a transverse beam element. A manufacturer's stamp which is characteristic of rolled steel, an indication that the larger beams were steel and not iron.

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[2] Alexander Newman, page 106.
[3] Llewellyn Edwards.
[4] Ibid.
[5] Ibid.
[6] Alexander Newman, page 107.

Known Use - The use of wrought iron elements in pre turn-of -the-century bridge construction is well documented. Some aspects regarding field and laboratory testing of similar bridge elements is referenced later in this report.

It is our opinion elements that form the truss portion of the bridge are of wrought iron construction and that the larger transverse deck beams and beams in the approach span are of steel.

Material Strengths and Allowable Stresses:

Published allowable design stresses for wrought iron formed between 1850 and 1900 generally range from 10,000 psi to 14,000 psi[7,8,9] . Data was reviewed from two studies that evaluated strength properties of wrought iron taken from same era truss bridges. A study on wrought iron strengths of Iowa bridges[10] performed by Iowa State University resulted in an average yield strength of 35 ksi (kips per square inch) and a failure strength of 49 ksi. A national study of wrought iron in historic bridges[11] resulted in a minimum tested yield strength of 32 ksi and a minimum tested failure strength of 48 ksi. We targeted an allowable stress value for wrought iron of 12,000 psi, which aligns with period design stresses. Tests on similar structures also indicates that a selected design stress of 12,000 psi would provide a factor of safety of 2.6 against yield and 4.0 against failure, which exceed factors of safety per current design standards.

Published allowable stresses for early steel range from 12,500 psi to 16,000 psi[12] depending on the application. We targeted an allowable value of 12,500 psi as published for bridge use. The selected allowable stress provides for a factor of safety against material failure of 4.8.

The allowable stresses as noted above are based on design approaches that would have prevailed at the time the bridge was constructed. Current design allows for higher allowable stresses up to 55% of yield, which results in 17,600 psi for a yield stress of 32,000 psi. The difference in allowables stems from an increased understanding of loads, materials and behavior of materials that has occurred through the years. Although we analyzed the structure in respect to period design approaches, we considered current allowables in evaluating recommendation options.

Timber design values, more so than other standard construction materials, are difficult to determine. Timber is often given general classifications or grades based on visual observation of the wood. Allowable stresses could vary considerably depending on an assigned classification. Lumber purchased today would have a classification noted on individual pieces. When working with older structures, particularly rough hewn timbers, the grade of the timber is not known, nor is it feasible to determine without strength testing of a sufficient number of elements to form a statistical conclusion. We generally use a lower grade of mixed southern pine when the grade is not known. Our observations of the timber stringers are relatively clear and true, which would

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[7] Alexander Newman, page 105.

[8] Robert Gordon, page 395.

[9] Herbert Ferris, page 5.

[10] W.W. Sanders, page 8.

[11] Robert Gordon, page 395.

[12] Herbert Ferris, page 5.

correspond to a higher grade of lumber. The summary table included on the Appendix is based on the lowest grade of mixed southern pine per AASHTO tables, which results in a basic bending allowable stress of 875 psi. The allowable stress can be increased 15% due to the use of multiple stringers, and increased 15% due to relatively short live load duration, resulting in an adjusted target allowable of 1160 psi.

Fatigue Strength:

Fatigue is the loss of capacity of an element due to cyclic stresses. This loss of capacity means that an element could fail well before it is stressed to the theoretical failure strength. The general approach to develop data for fatigue evaluation is to cycle through a select stress range until the material fails. This process is repeated multiple times until a correlation is developed between the applied stress range and the number of cycles to failure. The fatigue limit is the maximum stress which can be applied repeatedly without causing failure.[13]

The reason that fatigue is discussed in this report is that a structure that is 120 years old has cycled though loading many times in its lifetime. Applying an Estimated Average Daily Traffic as presented in the IDOT Appraisal Sheet of 160 vehicles per day will produce over 7,000,000 occurrences of traffic passing over the bridge. This number of loading occurrences is well into the range of those that could detrimentally impact the capacity of a structure.

To incorporate the impact of fatigue as it relates to the condition analysis, we consulted three sources; steel fatigue limits as described by AASHTO, a study titled "The Evaluation of Wrought Iron for Continued Service in Historic Bridges" by Robert Gordon and a study performed on Iowa bridges by W.W Sanders titled "Ultimate Load Behavior of Full-Scale Truss Behavior".

In general, wrought iron is perceived to be superior to steel in resisting corrosion and fatigue conditions.[14,15] Adherence to AASHTO steel requirements would limit the maximum allowable stress range to 24 ksi. This form of truss dictates that elements are either in tension or compression without variation, which then dictates a maximum applicable stress of 24 ksi. A fatigue stress of 24 ksi indicates that if the maximum stress does not exceed 24 ksi, the number of cycles will have little impact on capacity.

The Gordon study concluded that "The wrought iron used in historic bridges, if initially of good quality, can continue to give its original service indefinitely if protected from corrosion and the bridge's design load limitations are respected."[16]

The study by Iowa State University provided data on 24 wrought iron elements that were fatigue tested to failure. The minimum number of cycles at failure for an element subject to a 14 ksi stress range was 1,400,000.[17] These tests were on elements that had already been in use

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[13] Carl Keyser, page 63.

[14] T.B. Jefferson, page 61

[15] *Wrought Iron*, page 4.

[16] Robert Gordon, page 398.

[17] W.W. Sanders, page 36.

approximately 75 years at the time of the study. The tests showed there was considerable life remaining in the tested elements.

It is our opinion that an applied stress limitation of approximately 12,000 psi for wrought iron and 12,500 psi for steel will allow for continued service of the bridge.

OBSERVATIONS

Truss Elements:

Damage to exterior roadway structures, particularly those constructed of many small parts such as the truss, tends to relate primarily to corrosion. Corrosion is a natural result of unprotected ferrous metals being exposed to moisture and air. Some exposure conditions facilitate corrosion, such as locations that do not readily dry due to adjacency of other elements or absorptive materials. The structural framing has not received a protective coating and as such is affected by weather. Surface rusting was prevalent, however in general the observed corrosion was limited to surface only. Corrosion that extends into a metal surface sufficiently to cause material loss will be visible by the production of rust byproducts. The expansion of the metal to occupy several times the volume of the original metal is a rust byproduct. There were locations of rust byproduct observed, such as in the juncture between connected plates. Locations of observed corrosion of this nature were limited, and except for noted locations was not a controlling concern in regards to future use recommendations. The following discussion references truss elements that are identified by a panel point notation. Refer to the Longitudinal Section from a diagram of the panel point indentifiers.

Visual observations indicate that the truss elements are in fair shape, and they were found to be relatively plumb and true. There was widespread surface pitting of the wrought iron members, however, depth of pitting was relatively light. We adjusted the analysis for surface pitting by estimating a material loss of 2 mils for exposed surfaces. The inspection reports noted multiple bent members consisting of the tension verticals (L_1U_1 and L_5U_5) and the tension chord L_0L_1. Bending of the tension verticals appeared to be a result of vehicle impact. Bending of the lower chord appeared to be the result of debris impact. Because the bent elements are in tension, we did not feel the bends were significant in regards to the ultimate capacity of the structure. Damage to the railing rendered the railing unusable adjacent to the damage locations. Any reuse of the superstructure will require a replacement railing meeting current AASHTO requirements.

A critical aspect of pinned truss bridges is the pins. Because structural elements encapsulate the pins, it is not feasible to directly observe their condition. In addition, the upper chord pins are located within the channels of the compression elements, which further reduce ability to observe the pins. The observable surfaces of pins and attached elements were viewed for evidence of movement or corrosion that would indicate distress. Other than at the end bearing locations as discussed below we observed no significant indications of corrosion or other distress indicators.

There is concern with the tension elements located at the bearing ends of the trusses. The concern is the bearing ends are covered with soil and debris, which holds moisture against the metal elements and promotes corrosion. Although an attempt was made to clear the bearing ends, the collection of soil and debris was sufficient to hamper a thorough visual observation. We included a corrosion allowance of 1/8" in evaluating the capacity of the end tension elements that tie into the end pins. The corrosion allowance was estimated by feeling the metal areas adjacent near the pin. We included in the recommendations is a follow-up to clean and re-observe the bearing pins.

We did observe missing bolt connectors of the transverse beams at L_2, L_3 and L_4 to the corresponding vertical elements L_2U_2, L_3U_3 and L_4U_4. The missing bolts were noted in the previous inspection reports. The connections in which the bolts are missing are those where the beam extends through a double channel column and rest upon a plate attached to the column directly above the pin. The beam is vertically supported by the bearing plate, and as such the missing bolts are not load supporting. The bolts are intended to hold the elements together but would not carry vertical load. We feel it is feasible that many of the bolt holes placed at this particular connection were never filled with bolts, leading to the observation of inconsistently placed bolts. We do not feel the missing bolts are a concern to the capacity of the truss. Should the County elect to proceed with adapting the bridge for trail use, you may want to consider placing bolts in unused bolts holes if for no other reason than to prevent future confusion and concern.

Approach Span:

The approach span framing is in fair to poor condition. The timber deck above the framing protects the framing from direct weather conditions; however the decking also leads to longer durations of steel beams abutting wet timbers and potential collection of debris. This adjacency of wet timbers to steel affected the top flange of the "I" beams. The bents were plumb with visual evidence of surface corrosion only. We observed no evidence of distress in the foundation or the bent framing system.

The deck framing consists of channel exterior side beams and six standard shape "I" beams between the side channels. The side channels had been damaged by vehicle impact to the attached railing. There was also corrosion observed through the channel web at select locations. At some point in time after original construction, clips were attached to the timber deck and the top flange of the I beams to provide lateral support for the compression flanges. These support clips remain necessary for maximizing capacity of the I beam stringers. There was some limited corrosion of the top flange affiliated with moisture. We feel the I beams are in sufficient condition to warrant potential reuse, however we recommend removal of the existing side channels and railing system when considering reopening the approach spans.

Timber Deck:

The bridge and approach wear surface is timber deck that consists of rough sawn 3"x12" timbers placed side by side, with an orientation parallel to the direction of traffic. A thin (1/2" or less) asphaltic surface partially remains atop the timber deck. The asphaltic surface is most probably the remnants of a previous topping project. Below the wear surface planks are rough sawn 3"x12" timbers placed on their wide side at 24" transverse across the stringers. The stringers are rough sawn 3"x12" at 12" centers on the bridges and 10" steel I beams at 2'-4" centers at the approach spans.

The timber appears to be in fairly good condition with planks of relatively consistent size and clarity. We observed little deterioration of the wood. One section of planking was missing from the top deck and will require replacement. It is our opinion that the timber planking is in sufficient condition to continue use. An issue with the planking, however, is that the longitudinal orientation of the planks leads to gaps between the planks that are oriented in the direction of traffic. Gaps in the direction of traffic could potentially cause thin tires, such as bicycle tires, to become caught or wedged. We observed gaps up to an inch wide, which would not be acceptable in a trail scenario. Included in the Recommendations is an approach to alleviate the gap problem.

Abutments:

The south bridge abutment is constructed of large stone blocks stacked to a height of about six foot above the bank. Deteriorated mortar was observed in the joints between the stone blocks. It is not known whether the blocks were dry-stacked and the mortar was used to tuckpoint the stones or whether mortar was applied as a bed between the stone courses. The stone abutment, appears to be stable with no observed evidence of movement. It is our opinion that the abutment is stable and can remain in service with surface attention to the mortar joints.

Three courses of concrete masonry units (cmu) are placed atop the stone abutment. The cmu supports the south end of the south span of timber deck stringers. The cmu does not appear to be original construction. We surmise that the south end of the stringers was at one time supported by steel or wrought iron elements and the element corroded and was replaced by cmu. There were fractures and dislocations observed in the cmu, indicating that some movement has occurred. Repair of the cmu, consisting of pointing or block replacement, should occur prior to reconstructing the approach roadway.

The north bridge supports consist of a single four-foot diameter steel encased pier below each truss bearing point. The base of the piers is partially in the stream, causing the base of the piers to not be observable. The depth of embedment of the piers is not known, hence it is not known if scouring has affected the base of the piers. The piers are plumb and intact with no apparent distress. We saw no indications that use of the piers could not continue.

The north abutment of the approach spans is cast-in-place concrete, with a height approximately five feet above grade. We question whether the abutment is of original construction because of the building material. The north abutment of concrete is not consistent with the south abutment being of stacked stone block. The abutment appears intact and stable and continues to perform its intended function. Soil and debris has collected atop the foundation and should be removed to reduce the risk of corrosion near the beam bearings.

Waterway:

There is no riprap or other slope protection provided for the north bridge piers or the north approach spans. The northeast piers are partially located within the stream. As noted previously, there was no indication of movement or scouring damage observed, however damage of that nature could become a consideration. There is observable soil loss in areas below the approach spans, some adjacent to supports for the approach span bents. As in the piers, there was no movement or other signs of undermining associated with the bent supports observed, however an unprotected slope increases the risk of scouring and associated foundation movement. The County may wish to consider riprap slope protection near the approach spans and north bridge bearing piers.

There is riprap protection provided upstream and downstream of the south abutment. The riprap appeared to be effective in protecting the abutment against scouring. There was no immediate need observed to modify the slope protection on the south bank.

Approach Roads:

The approach roads have been torn up to prevent any possible traffic on the approach structure or the bridge. Should the structure reopen for use, it is expected that the County will reinstall the roadway. This report has not included a financial estimate of the roadwork in the concluding recommendations.

ANALYSIS

An analysis of the structure was performed based on two target uses; trail with mixed pedestrian and bicycle use, and a concentrated load for maintenance vehicles. The design live load requirement for pedestrian bridges per AASHTO is 85 pounds per square foot (psf). The desired capacity for a maintenance vehicle is 10,000 pounds concentrated load with an impact factor of 30%. In addition to vertical loads, the truss was evaluated for AASHTO lateral; wind loads of 75 psf on projected area, with a minimum chord applied load of 300 pounds per linear foot (plf) on the windward side and 150 plf on the leeward side.

Targeted allowable stresses, developed as described earlier, are summarized as:

Wrought iron -	12,000 psi tension and bending 32,000 psi yield with a maximum value of 12,000 psi
Steel -	12,500 psi tension and bending 35,000 psi yield with a maximum value of 12,500 psi

Allowable stresses were developed for compressive elements with AISC Allowable Stress Design criteria. Element areas as determined by field dimensioning were reduced by two mils for exposed surfaces. Tension element L_0L_1 was reduced an additional 1/8" width due to corrosion concerns.

Analysis summaries of various load combinations for elements comprising the truss and approach structure are included in the Appendix.

Truss Analysis:

Vertical Loads

It was apparent in the initial stages of the analysis that the truss would not be capable of supporting the pedestrian load for a full current deck width of 14 feet. Although compression members faired well, all tension members exceeded the target tension allowable stress. The tension stresses ranged from 113% to 183% of allowable. Alternative restricted deck widths were evaluated, concluding with an 8 foot wide restricted deck. Per the AASHTO guide for bicycle facilities[18], the minimum width allowed for combined bicycle and pedestrian use is 8 feet.

Analysis of the 8 foot wide deck produced only two elements in the truss that were significantly overstressed. The overstressed elements were matching interior tension diagonals L_3U_2 and L_3U_4 at approximately 16,000 psi. A third element of concern was the tension chord L_0L_1 that developed a stress of 13,600 psi.

The 10,000 pound concentrated load was placed at multiple locations within the span to determine the maximum impact to the structure. Placement of the concentrated load at the center node L_3 produced the controlling case, which was controlled by tension in the same L_3U_2 and L_3U_4 elements that controlled the uniform load analysis. The maximum stress in the noted tension element with a 10,000 pound load was approximately 15,600 psi, which was very close to that observed with the 8 foot wide deck. Based on the truss analysis only, it would be reasonable to support a 10,000 pound load under the same potential modifications as necessary to support an 8 foot wide trail deck. Other elements ultimately controlled the recommended maximum concentrated load.

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[18] *Guide for the development of bicycle facilities*, page 36.

Lateral Loads

The lateral bracing in the plane of the top chords and in the plane of the bottom chords was sufficient to withstand tributary area wind loads of 75 psf. The alternative minimum load of 300 plf for the windward chord and 150 plf per the leeward chord produced stresses in excess of twice the allowable stress.

Transverse Girders

Bridge

The 8 foot wide pedestrian loading stressed the transverse girders approximately 8% over target allowable. In evaluation of existing steel structures, we often would not be concerned for values up to 10% over target allowables. We believe that supporting the pedestrian loading on the girders is reasonable.

Concentrated loads of 10 kips, 8 kips and 6 kips stressed the girders 29% over, 11% over and 7% under, respectively. 8 kips would be pushing the reasonableness limit and 6 kips would be within target allowables.

Approach Structure

The 8 foot wide pedestrian loading stressed the transverse girders approximately 9% over target allowable. As per the bridge transverse girders discussion, we would view this as reasonable.

Concentrated loads of 10 kips, 8 kips and 6 kips stressed the girders 11% over, 4% under and 18% under, respectively.

Longitudinal Beams:

Bridge

The bridge longitudinal load distribution system relies solely on timber elements.
The summary table included on the Appendix is based on the lowest grade of mixed southern pine per AASHTO tables, which results in a basic bending allowable stress of 875 psi. The allowable stress can be increased 15% do to the use of multiple stringers, and increased 15% due to relatively short live load duration, resulting in an adjusted target allowable of 1160 psi. The target allowable as selected results in the 8 foot pedestrian deck loading that exceeds target allowable stresses by 16% and the 6 kip concentrated load that exceeds target allowable by 17%.

Approach Structure

Uniform live load of 85 psf stresses the approach beams to 14,600 psi, which corresponds to 17% over the target allowable of 12,500 psi. Concentrated loads of 10 kips, 8 kips and 6 kips stressed the girders 12% over, 4% under and 19% under, respectively.

Analysis Discussion:

Because of the way that uniform loads are additive and concentrated loads are sole applications, uniform pedestrian requirements tend to control where larger tributary areas apply, and concentrated loads tend to control where smaller areas apply. The uniform pedestrian load controlled maximum stress in the truss analysis, and led to the restriction of an 8 foot trail width. Even at an 8 foot width, twin diagonal tension rods on each truss are stressed 33% beyond target allowable. We feel that these elements will require strengthening to use the truss as a trail element. The tension chord stressed at 13,600 psi due to corrosion damage is discussed in Recommendations. With improvement of the tension elements in question, concentrated loads in consideration would not be a controlling factor with the truss. Transverse beams in the bridge and the approach were stressed 8% and 9% respectively over target approach stress. It is our opinion that these stresses are within the tolerance of the evaluation process and are within reason.

The largest stress variance occurs in the smaller structural elements, in particular the longitudinal timber stringers of the bridge and the longitudinal steel stringers of the approach structure. A uniform load applied to the timbers stringers resulted in a stress of 116% of target when applied to the timber stringers and 117% when applied to the steel stringers. A 6,000 pound concentrated load applied to the timber stringers produced a stress at 117% target stress. The timber stringers were the controlling load case for restricting the concentrated load to 6,000 pounds.

Stresses in the range of 116% and 117% over target stresses generally fall outside of a recommendable range. In this particular case, we feel it would be reasonable to maintain use of the stringer elements for the proposed function. The reasons it would be reasonable are:

- The target timber stress is somewhat arbitrary and based on a conservative value.
- Application of maximum loads with minimum allowable stresses would not cause collapse of the stringers. What could occur is that the stringers would deflect more than what is comfortable for the user, providing for a warning rather than a collapse.
- Multiple stringers provide a level of redundancy.

It is our opinion that with strengthening of select tension elements, it would be reasonable to allow for use of the structure an 8 foot wide bicycle/pedestrian trail or for an application of 6,000 pounds concentrated load.

DISCUSSION

It is not feasible to know with surety that an aged structure such as the English River Bridge is capable of supporting significant applied load. This lack of surety is due to aspects of the structure where the condition is not directly observable and verifiable. For example, it is not feasible to observe the majority of the surface of the panel pins or know the condition of the shaft of one of the multitude of rivets. The condition of non-observable elements is evaluated by looking for evidences of distress within the structure. Examples of distress are lack of plumbness, movement of elements or systems and fractures and/or bending of elements. This process of evaluating condition by looking for reflective distress is not foolproof, however it is the primary process available for our use. We do know from past studies that wrought iron elements from similar period bridges retain performance capability.[19,20] A past study of the ultimate capacity of similar bridges concluded that "The ultimate strength of the bridge is substantially higher in all cases than that found using present rating criteria."[21] We have concluded, based on the physical condition of the framing and on research of similar structures, that there is usable life remaining in the structure.

The capacity analysis indicates that the structure, with select reinforcement, can support a pedestrian bridge load of 85 psf for an eight foot wide trail. The structure can also support a 6,000 pound concentrated load. We recommend, at a minimum, that the following actions be taken should you wish to incorporate the structure into an active trail system.

1. Remove the soil and debris adjacent to the truss bearing ends and thoroughly clean the metals and pins. Condition and potential material loss should be evaluated and repairs made if necessary. The suggested allowance should repairs be necessary is $6,000.

2. Strengthen tension elements L_3U_2 and L_3U_4 on each truss, for a total of four elements. We have not developed a final design for the strengthening mechanism. However, in concept, we feel that adding a parallel element between the two rods with attachment to the truss verticals and chords with provide sufficient redundancy to ease concerns regarding the tension rods. The estimated cost for strengthening the tension rods is $15,000.

3. Remove the edge channels of the approach spans and remove the extending timber deck. The estimated cost to demolish and remove debris in preparation for new rail placement is $9,000.

4. Install new railings per IDOT requirements. The proposed rail is 4'-6" in height with a maximum opening size between rail elements of 6" for the lower half of the railing and 8" for the upper half. The anticipated cost for a new railing both sides is $45,000 for a steel rail. As an alternative the anticipated cost for a railing with chain link infill is $20,000.

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[19] W.W. Sanders.

[20] Robert Gordon.

[21] W.W. Sanders, page 54.

5. Install new decking over existing timber planks with side gaps perpendicular to the direction of traffic. The anticipated cost for 1" decking is $12,000.

6. Reconstruct the roadway bridge approaches to usability. It is anticipated that this can be done with county resources and a cost opinion has not been included.

Associated with the repairs above may be a consultant to develop connection designs and contractor guidance. The cost of a consultant will depend on the services desired and the capacity of County staff; we suggest an allowance of $15,000. Should you proceed with a reconstruction project, we expect items will be found that require attention that were not identified in the field review. A contingency should be established to covers these items.

Repairs you may wish to consider but are not deemed as a necessary requirement to utilize the bridge include:

1. Provide slope protection below the north approach spans. The anticipated cost for 12" of riprap below the approach spans is $10,000.

2. Tuckpoint the three courses of cmu atop the south abutment and the rock of the south abutment. The anticipated cost is $5,000.

Should the County decide to move forward with incorporation of the structure into the trail system, we suggest the following initial steps:

1. Clean and access the bearing conditions as noted above. It is feasible that deterioration at these locations is more extensive than expected, which could affect recommendations.

2. The items included in the recommendation are unique, and as such there is limited reference and guidance material in the development of a cost opinion. It is suggested that the County review the recommendations with a local contractor for a second opinion on costs. Local craftsmen may also have creative ideas on railing construction and potential flooring approaches that this report has not considered.

Alternative Deck:

The discussion above was based on retaining as much of the existing deck on the bridge and the approach as feasible. An alternative to maintaining the existing deck would be to remove and replace the deck entirely. The potential advantage of this approach is that reduced dead loads from a smaller, lighter deck could positively affect the analysis. A disadvantage would be increased cost.

Additional analysis was performed using a reduced dead load to account for removal and replacement of the existing deck with a smaller deck. Stress in the controlling element, which remains the L_3U_2 and L_3U_4 tension diagonals, is 13,400 psi. This is above the target value; however it is sufficiently close to not require reinforcement of the tension elements. Following this approach, reinforcement would be required only at the truss bearing pins should a further review reveal significant corrosion. The estimated cost to remove and replace the deck is $40,000 for the truss and $55,000 for the approach spans.

Longevity:

The bridge is currently approximately 120 years old, and the primary conerns regarding condition relate to areas where moisture was allowed to set against the structure. These areas consist of the collected soil near the truss bearings and to a lesser extent where timber planking abuts metal. We do not expect the bridge will be often loaded to the extent analyzed in the report, which would restrict fatigue from being a potential problem. It is felt that if these concern areas are alleviated, with proper maintenance the bridge should perform for many years to come, probably in excess of twenty years. We recommend a yearly inspection to monitor for changes occurring to the bridge condition as part of the ongoing maintenance program.

CONCLUSION

It is reasonable to consider the structure over the English River for use in the trail system. To prepare the structure for use, several inadequacies exist that will need to be addressed. These inadequacies consist of:

- Railings that are damaged and not code conforming.
- Gaps in the timber deck that are parallel to traffic.
- Questionable strength of select members.
- Concern regarding the condition of soil covered end bearing elements.

The above issues could be addressed independently at an estimated cost range of $62,000 to $87,000 depending on the type of replacement railing installed. As an alternative, the issues except that of the corrosion concern at the bearings could be addressed by removing the existing deck and replacing with a lighter deck with an integral railing. The anticipated cost to remove and replace the deck for the bridge and approach spans is $101,000. Consultant costs for the effort could be $15,000 to $20,000.

There were two items observed that were not considered as necessary to utilize the structure, however should be considered from a maintenance standpoint. The two items are additional slope protection for the adjacent banks and tuckpointing of the south abutment stone and masonry. The anticipated cost of the items is $15,000.

I hereby certify that this engineering document was prepared by me or under my direct personal supervision and that I am a duly licensed Professional Engineer under the laws of the State of Iowa.

Signature: ______________________ Date: __________

Name: Steven C. Bradley, P.E.

Iowa License Number: 10470

My license renewal date is December, 2009

Pages, Sheets, or Divisions covered by this seal: All

FOOTNOTES & BIBLIOGRAPHY

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