Cascade Bridge Rehabilitation Evaluation

PREPARED FOR
CITY OF BURLINGTON
DEPARTMENT OF PUBLIC WORKS

SEPTEMBER 2012

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

COPY

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Registration renewal 12/31/2013

Cascade Bridge Rehabilitation Evaluation

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I. INTRODUCTION

In accordance with the request of the City of Burlington Department of Public Works, Shuck-Britson Inc. has performed an evaluation for rehabilitation of the Cascade Bridge.

The purpose of this evaluation is to:

- Assess the general condition of the structure.
- Identify structural deficiencies.
- Evaluate the feasibility of truss repairs (strengthening / replacing members and joints).
- Consideration of historical preservation requirements in repair details.
- Identify possible truss shoring and stabilization methods during repairs.
- Evaluate load rating / capacity of bridge (in current condition and after rehabilitation).
- Assess functional issues (bridge roadway width, rails, sidewalk width, ADA requirements).
- Provide estimated construction costs to implement repairs.
- Provide recommendation for feasibility of rehabilitation, for comparison to bridge replacement options previously developed by others.

Copies of recent inspection notes and photos, previous repair plans, and previous bridge analysis and load rating files were provided to Shuck-Britson Inc. by the City. Photos and inspection notes provided were from a 2008 inspection performed by the City. Previous repair plans were dated 1953, 1964/1978/1984, and 1998. It appears that the 1964 plans may have been developed in 1964 and perhaps partially implemented in 1964, 1978, and 1984, with some plan revisions occurring in 1978 and 1984. Previous analysis and load rating files were dated 1984 and 2006. Original bridge plans are not available. In addition, a Replacement of Cascade Bridge Report dated September 2009, prepared by another consultant, was provided.

A site visit was conducted by Steven M. Kunz, P.E. of Shuck-Britson for a general observation of the structure and existing site conditions; observations were made from the ground or from the bridge deck surface. Documentation files provided by the City, as noted above, were to be used for identifying truss geometry, truss framing member properties and condition, details of previous repairs, and as a basis for load rating. A thorough “hands on” inspection of the bridge, measurements of members, or identifying areas of damage and deterioration are beyond the scope of this evaluation, as are structural material testing and a survey of existing utilities.

Snyder and Associates, Inc. reviewed the approach geometry and condition, curbs, intakes, and approach sidewalks, utilizing photos and documentation provided by the City. A site observation by Snyder was not included in the scope of this evaluation.

The findings and conclusions included in this evaluation report reflect the conditions encountered based on the documentation provided by the City and the exercise of professional judgment, as well as conditions observed during the site observation. The nature of the evaluation does not permit assurance regarding the presence or absence of latent, hidden, or unknown defects in the structure. Therefore, no responsibility can

be assumed for lack of integrity of the structure from unknown conditions, unpredictable causes, or subsequent development of deterioration.

Preliminary estimated costs are presented in this report for repair. They are not intended as guarantees of prices which will be obtained by competitive bidding, but will prove useful for comparison of alternatives, preliminary budgeting, and determination of a course of action for rehabilitation of this structure. Costs are provided in September 2012 dollars, and may require adjustment for inflation and other economic factors if there is a significant delay before construction begins.

II. STRUCTURE CONDITION

The following narrative will give a description of the conditions of various bridge components. This description is based primarily on the previous inspection notes and photos provided by the City, previous repair plans, and our site observation.

The bridge was originally built in 1896, and was added to the National Register of Historic Places on June 25, 1998. The bridge has a roadway width of 29'-11", and a 4'-0" raised sidewalk on each side of the roadway. Several light posts and utility conduits are mounted to the bridge. The bridge consists of 4 deck truss spans, with lengths of 90'-0", 90'-0", 204'-0", and 60'-0", and a short 15'-9 stringer span across Pier 1. The total length of the bridge is 459'-9" measured from centerline to centerline of abutment bearings. Each span is comprised of two trusses tied together with lateral bracing. The trusses are pin-connected at panel points. Truss members serving primarily as compression members are built up from rolled channels and bar lacing, which are riveted together. Forged eyebars or rods are utilized for truss tension members. Steel floorbeams are placed transverse to the trusses at each panel point; deck stringers are placed parallel to the trusses and are supported by the floorbeams. The roadway deck is supported by the deck stringers. The end truss supports are high abutments constructed of concrete and are designed to retain fill behind the backwall and between the wings. Piers 1 and 2 are steel-framed structures and are supported on concrete footings; footing support conditions below the ground are not known. The truss of span 3 bears directly on Pier 3, which is a concrete grade beam supported on caissons cored into limestone bedrock.

Major work items associated with previous repair plans are as follows:

1953 repair plans:

- New truss bearing shoes
- New horizontal bracing
- Reinforcing of steel piers
- Reinforcing of some diagonal eyebars

1964/1978/1984 repair plans:

- Replacement of roadway deck joints
- North abutment bearing seat repairs and reinforcement of southeast abutment wing
- Installation of lateral bracing stiffener plates at panel points, reinforcing of some diagonal eyebars, seal welding of eyebar ends at panel points

- Cutting of holes in truss members to drain water
- Replacement of missing steel lacing with plates
- Clean and paint entire bridge
- Pier 2 steel repairs and concrete retaining wall at Pier 2
- Removal of existing sidewalk and exterior rail, and replacement with concrete sidewalk on corrugated steel deck, new sidewalk expansion joints, new exterior sidewalk rail
- Removal of laminated wood deck, and replacement with 5" open steel deck
- Removal of steel floorbeam bolsters, replacement with new steel wide-flange beams on tops of existing floorbeams
- Removal of shallow stringers, replacement with new steel wide-flange stringers
- New roller bearings at Piers 1 and 3
- New jacking posts at Pier 1 for bearing replacement
- New floorbeam at north abutment
- Floorbeam repairs at south abutment

1998 repair plans:

- Removal of existing concrete foundation at Pier 3, replaced with new concrete grade beam and caissons cored into limestone bedrock
- Concrete abutment repairs

A. Roadway Deck

The 5" steel roadway deck grating was installed in 1964. The steel deck is lighter than the original timber deck, thereby reducing the weight of dead load on the trusses. Deck drainage is not controlled, as water runs freely through the open steel deck. The steel deck appears to be in reasonably good condition (**see Photo A1, A2**).

The temperature at the time of the site observation was approximately 90 degrees F. The finger joint plates at the south abutment roadway expansion joint were in contact with each other (**see Photo A3**). There was a gap observed in the deck joint openings at the Pier 1 (**see Photo A4**) and north abutment roadway expansion joints (**see Photo A5**).

During our site observation, we observed that the alignment of the west roadway gutter line was not on a straight alignment, but instead appeared to have a slight bow to the west. Subsequent to our site observation, the City pulled a stringline along the deck using the longitudinal deck stringers as a reference line; offsets of 1 ½ inches and 1 inch were reported near the west side of the roadway and the east side of the roadway, respectively. It was noted that the alignment of the stringers may not provide an accurate reference for this measurement. Truss alignment can be verified during repair plan preparation, and possible alignment corrections can be considered at that time.

B. Sidewalks

The concrete sidewalk appears to be generally in fair condition, although the deck was not sounded to locate hollow or delaminated areas. Several isolated

locations of deterioration were observed, and several areas near the curb face exhibit exposed reinforcing steel (**see Photo B1**). The adjoining sidewalk deck segments and/or cover plates, are in contact with each other (see Photos A3, A4, and A5), indicating an inadequate joint capacity for thermal movement.

The width of each sidewalk does not meet current ADA standards. Per the Accessibility Guidelines for Pedestrian Facilities in the Public Right-of-Way, a publication released by the Access Board and published in the Federal Register, 5 foot wide passing spaces are required for narrower sidewalks at a minimum interval of 200 feet. In a practical sense, this means that, for any structure over 200 feet in length, a 5 foot wide sidewalk is needed.

C. Sidewalk Rails

Previous repair plans indicate that the original sidewalk rail was removed and reinstalled as part of the sidewalk reconstruction (**see Photo C1**). The rail height measures approximately 38 inches above the sidewalk surface. The current minimum design height for pedestrian railing is 42 inches, per the AASHTO LRFD Bridge Design Specifications. In addition, the rail does not meet the geometrical spacing restrictions of a 6-inch sphere for the lower 27 inches of railing, and an 8-inch sphere for the upper portion.

The rail appears to be in generally good condition, but exhibits paint deterioration with many locations of rust.

There is no separation barrier between the roadway and sidewalk. Current AASHTO specifications (C.13.7.1.1) do not require a separation barrier for posted speeds of 45 mph or less. It should be noted however, that there is no traffic rail on the bridge; the existing sidewalk rail does not meet current design standards for this purpose.

D. Deck Framing

Previous repair plans indicate that stringer bolsters were removed and replaced, and several deck stringer lines were replaced when the roadway deck was replaced and the sidewalk reconstructed. A deck cross section, included in the Appendix, illustrates the floorbeams, deck stringers, and sidewalk framing. Deck stringer designations are consistent with previous inspection reports. The condition of existing deck framing members appears to be consistent along the length of the bridge, with some sections of severe section loss and deterioration.

<u>Floorbeams:</u>

Floorbeams are built-up riveted sections utilizing steel plates as webs and steel angles as flanges. Wide-flange members were welded to the tops of the floorbeams during the deck replacement when the original deck stringer bolsters were removed. The floorbeams cantilever approximately 8 feet beyond the centerline of each truss, with the depth of the cantilever tapering. The cantilever section supports one deck stringer and the sidewalk framing. Previous inspections indicate deformations in built-up members due to pack rust, complete

section loss of flanges for significant length of floorbeams, and areas of severe section loss of flanges and webs (**see Photos D1, D2, D3, D4, D5, D6**). A review of the photos indicates significant corrosion and section loss in the areas of high stress at the ends of cantilevers.

Deck Stringers:
Some deck stringers are original double-channel members, and others are wide-flange members that were installed to replace some of the original smaller stringers during the deck replacement. Previous inspections indicate significant section loss, pack rust beneath splice plates, gusset plates supporting stringers perforated due to corrosion, and broken welds at splice plates due to corrosion (**see Photo D7**).

Sidewalk Framing:
Sidewalk framing consists of I-shaped members that were originally used as deck stringers. The deck stringers were salvaged during the deck replacement for the purpose of sidewalk framing. There is minimal information regarding their condition in the previous inspection reports. However, these members exhibit severe section loss and corrosion in some of the previous inspection photos (**see Photo D8**).

E. Truss Framing
The bridge comprises four truss spans; each span is comprised of two trusses tied together with lateral bracing. The trusses are pin-connected at panel points. Truss members serving primarily as compression members are built up from rolled channels and bar lacing, which are riveted together. Forged eyebars, rods, or channel sections are utilized for truss tension members. The general truss configurations are shown on the Elevation View of the Situation Plan included in the Appendix. Although the truss configurations are different for each span length, the construction details of each truss are similar.

Conditions of significant deterioration are included in the previous inspection reports and are noted below. Based on our cursory observation, we estimate a section loss of approximately 10% is not uncommon on the truss framing members. The condition of existing truss members appears to be consistent along the length of the bridge, with some sections of severe section loss and deterioration.

E1 Eyebars:
Previous inspection reports indicate several deficiencies relating to the truss eyebars. Severe section loss is common. Many tension members consist of sets of at least two eyebars between panel points, and bottom chord members of span 3 consist of sets of four eyebars. Many sets of eyebars were observed with “slack” in at least one of the eyebars – meaning that some of the eyebars were not carrying the tension load for which they were designed, and the adjacent eyebars in that set were carrying more tension load than they were designed to carry. This is readily apparent when observing the structure, but is difficult to

discern in the available photos. A damaged eyebar that was bent, possibly caused by impact, results in a more extreme example of this situation (**see Photo E1.1**).

Eyebars exhibit significant corrosion at panel points (**see Photos E1.2 thru E1.10)**. The 1953 repair plans include details to reinforce some of the diagonal eyebar ends of span 3 (**see Photo E1.11 thru E1.13**); but it does not appear that the details were implemented on all diagonal eyebars. The 1964/1978/1984 repair plans include similar details for diagonal eyebar reinforcement, but intended locations of repairs are not indicated in the plans. It does not appear that the details were implemented on all diagonal eyebars in other spans. The reinforcement details include an eyebar reinforcement plate welded to the end of the eyebar, and also supplemental plates welded between existing compression member elements or eyebars at the panel points, to which the eyebar reinforcement plate is welded. The effectiveness of this detail is questionable, given that the reinforcing plate utilized is welded to other eyebars at the panel point, or that the load path is not a direct transmission to the panel point. The 1964/1978/1984 repair plans also include details to seal weld the top perimeter of all adjoining eyebars and gusset plates at bottom panel point locations (**see Photo E1.14**).

E2 Rods:
Previous inspection reports do not include many comments regarding rod truss members. Two diagonals in the Span 4 truss are reported as consisting of only one rod instead of two as shown in the plans.

Some rods are indicated on previous plan sets as round rods, and others are indicated as square rods. The square rods were forged to form the closed loop at panel point connections. Previous inspection report photos indicate that the ends of several rods were reinforced (**see Photos E2.1 and E2.2**).

E3 Built-up Members:
Previous inspection reports indicate several deficiencies relating to the truss compression members. Severe section loss is noted in several areas, with complete section loss of portions of members in some locations (**see Photos E3.1 thru E3.3**). Many compression members have broken or missing lacing elements (**see Photo E3.4**). In addition, many rivets are missing from the built-up members (**see Photo E3.5**). Poor welds are reported from previous rehabilitations. Many built-up members exhibit pack rust accumulation between member elements, causing local distortions (**see Photos E3.6 thru E3.10**).

E4 Pinned Panel Points:
Previous inspection reports indicate significant section loss of panel point gusset plates. This can be seen in other photos included in sections E1 and E2.

The 1953 and 1964/1978/1984 repair plans include details that were intended to provide reinforcing for eyebar ends and a method of fastening lateral and horizontal bracing members to panel points. Eyebar / panel point reinforcing

details were mentioned in section E1 (**see Photos E4.1 thru E4.4**). The details include supplemental plates that were added between eyebars, around pins, and between eyebars and horizontal struts between trusses, and then welded to those members. Seal welds, as indicated in the repair plans, were made around the top perimeter of eyebar ends, presumably to prevent moisture intrusion. Such seal welds partially lock the joints and inhibit the ability of the members to rotate as designed, introduce residual stresses in the area of the eyes, and create notches and surface irregularities which could be a point of initiation for fatigue cracks.

E5 Horizontal and Lateral Bracing:
Previous inspection reports do not include any comments regarding horizontal or lateral bracing. Bracing members consist of round rods. The 1953 repair plans indicate that new horizontal bracing was installed in all spans. Previous inspection report photos indicate similar details for the horizontal and lateral bracing (**see Photo E5.1**)

E6 Truss Bearings:
Previous inspection reports do not include any specific notes on truss bearings. The 1953 repair plans include replacement details at the south abutment and for spans 3 and 4. The 1964/1978/1984 repair plans include replacement details for the north end of span 3 and the south end of span 2. Previous inspection photos at abutment bearings indicate heavy surface rust with debris surrounding the bearings (**see Photos E6.1 and E6.2).** Previous inspection report photos at pier truss bearings indicate heavy surface rust on bearings and eyebar ends, with section loss **(see Photos E6.3 and E6.4)**.

F. Steel Piers (Piers 1 and 2)

Previous inspection reports do not include any specific notes on steel piers. Review and comments regarding these piers is beyond the scope of this evaluation.

G. Structural Steel Paint

The date of the last paint job for this bridge is unknown, although the 1964/1978/1984 repair plans indicate that the entire bridge is to be cleaned and painted. It is not known if the previous paint layer was fully removed prior to repainting. Approximately 30% of the paint coating exhibits heavy deterioration. Many areas of eyebar ends, built-up members, panel point regions, floorbeams, and stringers exhibit significant surface rust with some pack rust.

The strength of adhesion of the remaining sound areas of paint is not known. During rehabilitation plan preparation, paint coating samples will be obtained and tested for measurements of lead and chromium to determine whether or not the material is considered hazardous.

H. Concrete Pier and Footings (Piers 1, 2, 3)

Previous inspection reports do not include any specific notes on concrete condition at these locations, but deterioration is visible on footings at Pier 1 in previous inspection photos. Review and comments regarding this pier are beyond the scope of this evaluation.

I. Concrete Abutments

Previous inspection reports indicate that concrete repairs are necessary for both abutments. Review and comments regarding these abutments are beyond the scope of this evaluation.

J. Approach Slabs / Approach Pavement / Approach Sidewalks

North Abutment: The north approach appears to consist of a seal coat roadway with concrete curb and gutter. The roadway grade is approximately 4% as it approaches the bridge. Open-throat intakes are provided approximately 10 feet from the bridge abutment. Aggregate washed from the seal coat surface is apparent at the north bridge abutment, particularly near the roadway centerline, indicating a significant amount of runoff from the roadway is not being intercepted by the intakes. This is likely a result of the limited roadway cross-slope with respect to the longitudinal grade. The roadway surface has scattered random cracking but generally appears to be in acceptable shape.

South Abutment: The south approach is a PCC slab with curb and gutter, which appears to be in good condition. Just south of the bridge is the entrance to Dankwardt Park, a seal coat roadway with concrete curb and gutter. The park entrance and exit roadways are split, although two-way detour traffic is currently utilizing the park exit. Sight distance was not measured at the park exit with this project. Although the roadway ostensibly drains to the south, aggregate washed from the seal coat roadway just south of the project limits has apparently washed and settled onto bridge approach slab, indicating a drainage problem may be present.

Sidewalks:
Although there is sidewalk along the east side of the bridge, there is no sidewalk along the roadway outside the bridge.

Along the west side of the roadway, a continuous pedestrian path is provided. Sidewalks both on and off the bridge are 4 feet wide. On the north approach, guy wires cross the sidewalk, limiting vertical clearance. Metal railings are provided along the west side of the sidewalk both north and south of the bridge; the height of the metal railings was not measured, and we are unable to determine if they meet current specifications. At both bridge approaches, asphalt has been placed on the sidewalk to smooth the approach to the bridge. Grades of transitions to the bridge were not measured to determine if they exceed the 5% maximum allowable grade. It appears that sidewalk surface discontinuities in

excess of 0.25 inches are present within the project limits, which will need to be corrected with the bridge rehabilitation.

III. TRUSS LOAD RATING AND PAINT SCRAPE TOXICITY TESTS

A. Truss Load Rating

A summary load rating using HS20 live load for each truss in the existing condition, and for each truss with replacement tension members installed, is included in the Appendix.

1. Methodology

   The truss load rating was performed by reviewing the methodology utilized and results obtained by previous load ratings. The methodology utilized in the 1984 load rating is consistent with our methods, which utilizes the influence of a load applied at panel points along the truss, and multiplies that influence by the actual applied load to determine the effect of the vehicular live load in question. This procedure was performed for all four truss spans for the HS20 loading, and checked for the Type 3-3, 3S3, and 4 rating trucks.

   During our review of the previous load ratings, it was found that the truss geometry utilized for the 204 ft truss analysis and the 60 ft truss analysis was not consistent with truss dimensions indicated in the existing plans. Therefore, member forces from previous ratings could not be utilized, and each truss was modeled to determine member forces for dead load and live load based on existing plan geometry. Only the HS20 live load was analyzed, as this is the controlling live load condition. Our truss rating models utilize member dimensions and properties identified in the previous load ratings.

2. Allowable Stresses for Rating

   Allowable stress values used in our truss load rating differ from those used in previous load ratings. The tension members of this bridge were forged in the late 1800's, and control the load rating. Steel manufactured in this era exhibits a high carbon content, and is very susceptible to failure from brittle fracture due to fatigue loading. Shortly after the inception of the federal bridge inspection program circa 1971, coupon samples from a steel truss of this vintage were tested to fatigue failure at the materials laboratory at the University of Iowa. The results of that testing indicated that an allowable tensile stress of 8.0 ksi is appropriate for Inventory Rating of tension members, resulting in a load level that which can safely utilize an existing structure for an indefinite period of time. An Operating Rating allowable tensile stress of 11.0 ksi is appropriate based upon the testing, to calculate the absolute maximum permissible load level to which the tension members within the structure may be subjected. Such allowable stress values are subject to the qualifier that the steel must be in reasonably good

condition, free of significant deterioration or defects such as heavy corrosion, hairline cracks, damage from vehicle impacts, fabrication errors, etc.

The allowable stresses for rating listed in the AASHTO tables are somewhat higher than the values shown above. AASHTO specifically states that, when the material is unsound or not reasonable equivalent in strength to new materials that would be in first class construction, the allowable stresses shall be fixed by the Engineer, based upon his investigation. In the case of the Cascade Bridge, we have test values on forged tension members of this vintage, and the proposed allowable stresses of 8.0 ksi (inventory) and 11.0 ksi (operating) are in line with values customarily used in Iowa to rate these types of structures.

3. Load Rating of Existing Bridge
   Determination of the maximum safe operating live load capacity of the Cascade Bridge was performed in accord with the AASHTO Manual for Bridge Evaluation. The Allowable Stress Rating method was utilized, as is customary for older bridges of this type which were designed using the allowable stress methodology. Applying the allowable tensile stress of 11.0 ksi, the load rating is governed by the tension members.

   The Operating Rating of the various spans for HS20 loading (36 ton tractor truck with semi trailer), assuming the members to be in reasonably good condition with no section loss, is presented here:

Span	**Controlling Member**	**Rating**
Span 1 (south)	L1L2, L3L4 Bottom Chord	11 Tons
Span 2	L1L2, L3L4 Bottom Chord	11 Tons
Span 3	U2M3, M9U10 Diagonal	4 Tons **
Span 4 (north)	U1L2, L2U3 Diagonal	10 Tons

   ** Controls Rating

   The 4 ton operating rating corresponds well with ratings of hundreds of other trusses of this vintage in Iowa, which generally fall within a range of 3 tons to 5 tons and are controlled by their forged tension members. Because of the advanced age and deteriorated condition of the Cascade Bridge noted previously in this report, as well as the unknown fatigue cycle history, the structure should be closed until repairs can be implemented, as it is susceptible to sudden, catastrophic failure of the tension members.

4. Load Rating with Tension Members Replaced
   Since the forged tension members limit the bridge to a prohibitively low capacity, we investigated the possibility of replacing those members

with modern steel members with a yield strength of 50 ksi. In our analysis, we kept the new members the same size as the existing ones, to maintain the same deflection characteristics for the trusses. The allowable operating stress for the new steel is 27.5 ksi, a considerable increase over the 11.0 ksi. When the trusses are analyzed using the upgraded tension members, the compression members control and the Operating Rating for each span is tabulated as follows:

Span	**Controlling Member**	**Rating**
Span 1 (south)	U0L0, U5L5 Vertical	52 Tons
Span 2	U0L0, U5L5 Vertical	52 Tons
Span 3	U4U6, U6U8 Top Chord	27 Tons
Span 4 (north)	U1L1, U3L3 Vertical	27 Tons

The tabulated ratings assume the existing compression members to be in reasonably good condition. Since these members exhibit some deterioration and advanced age, the safe load rating should be limited to no more than 20 tons, even if the tension members were replaced.

The Operating Rating for HS20 loading for stringers and floorbeams are 38 tons and 33 tons, respectively, assuming no section loss.

B. Paint Scrape Toxicity Tests

Material testing for this structure will include truss framing paint scrape tests to be obtained during rehabilitation plan preparation.

Samples of the paint top coat and primer coats will be obtained to measure levels of lead and chromium. Levels of either element exceeding 35,000 parts per million (ppm) or 5 ppm leachable indicate a hazardous material. Current environmental regulations and DOT specifications require that any hazardous materials removed during demolition or cleaning must be contained, collected, and disposed of properly.

IV. DISCUSSION

The Cascade Bridge is in need of significant repairs to maintain its structural integrity and serviceability, and to extend its useful life. Failure to address several repair items noted below can result in severe structure damage or potentially catastrophic failures.

It should be noted that truss tension members and floorbeams are fracture critical, meaning that a failure of one of these members would probably cause a portion of or the entire structure to collapse. Truss pins also serve a critical function, and sudden failure of one pin can result in sudden collapse.

Due to the specialized and labor-intensive nature of the repairs, the difficult site conditions, and the likelihood of unanticipated conditions once repair construction begins, it is likely that all repairs will take more than one construction season. It is recommended that all repairs be implemented under one contract for cost savings due to contractor mobilization and difficult site access. The bridge should remain closed during repairs to minimize bridge shoring complexity and traffic control requirements, and to reduce risk to the traveling public. The estimated base cost of repairs assumes full paint removal and repainting of the truss framing and superstructure members, but a partial paint removal and repainting option is discussed as well.

As discussed in Section III above, the existing bridge live load rating is low. We are recommending replacement of all eyebars, rods, gusset plates, and pins on all trusses as part of the bridge repairs (discussed in Section E below), which will significantly increase the live load capacity. Upgrading the bridge capacity further to carry legal loads would require replacement of approximately half of all compression members. The cost associated with this would be significant due to the high fabrication cost of new members to match existing for historical considerations – with multiple primary elements connected by bolted / riveted lacing. This level of truss strengthening is beyond the scope of this evaluation. A discussion of truss member replacement is included at the end of this section.

Some utilities are present on the bridge and adjacent to the bridge. The costs of utility relocation or preservation in the structure are not included in the cost estimate provided, but they should be considered in the decision to rehabilitate this structure.

Based on our evaluation of this bridge, the following is a summary of the general condition with items of visible deterioration and substandard conditions. Included with this discussion, we present our repair recommendations:

A. Roadway Deck

The 5 inch steel roadway deck grating appears to be in generally good condition and no repairs are recommended at this time. However, we recommend removing and reinstalling the finger joints at the south abutment to provide adequate clearance for thermal expansion and contraction.

The roadway width will need to narrowed by approximately 1’-2 along the west gutter line for widening of the sidewalk to provide an accessible route across the bridge that meets requirements of the ADA, as discussed below. Widening of the

east sidewalk is not recommended, as this sidewalk does not connect to any sidewalk off the bridge and would not be likely to be used. SUDAS typically requires that a new bridge be 4 feet wider on each side than the traveled way (which is defined as the travel lane width exclusive of curb offset), but also indicates that a bridge width equal to the face-to-face width of the approach roadway is acceptable. See below for discussion of approach roadway reconstruction.

B. Sidewalks

The concrete sidewalks and steel framing must be removed to accommodate a sidewalk that meets current ADA requirements, and to correct the severe corrosion and section loss observed in some of the steel supporting members. Although removal and reconstruction of both sidewalks is not necessary to meet ADA requirements, it is included in our cost estimate to represent a worst case scenario and to provide an allowance for sidewalk framing repairs that might be needed due to severe corrosion. The new sidewalk travel surface must have a width of at least 5'-0. Based on the new sidewalk width requirement, and the observed significant deterioration of some existing sidewalk framing members, we are assuming that all sidewalk framing members will need to be replaced. The new support framing will be constructed of new structural steel members using a configuration similar to the existing framing. A corrugated steel deck with concrete topping, similar to the existing sidewalk, is recommended to minimize weight. New expansion joints of similar construction to existing will be incorporated into the new sidewalk. Any existing framing members found to be in reasonably good condition during repair plan development can be re-used. A concept detail of the new sidewalk configuration and framing is included in the Appendix of this report.

C. Sidewalk Rails

The existing sidewalk rails will be reinstalled using details similar to existing. The existing rail section will need to be raised approximately 4-5 inches to meet the 42-inch minimum height requirement, which will result in a corresponding gap between the top of sidewalk and bottom of rail. In addition, a new horizontal rail member must be introduced across the top pattern of the rail to reduce the open space to a size that would restrict passage of an 8-inch sphere. Details for the new rail elements will be consistent with the existing rail details to maintain historic integrity. An illustration of this concept is already presented on Exhibit 7 of the previous Replacement of Cascade Bridge Report of September 2009, and is therefore not repeated in this document. Details of new rail elements will be reviewed with historic agencies during repair plan development.

The sidewalk rail should be fully shop blasted and repainted at this time. Rail elements found to be damaged should be repaired or replaced.

AASHTO Specifications indicate that the existing curb is adequate to channelize traffic at the posted speed.

D. Deck Framing

Floorbeams:

Deteriorated sections of floorbeams, primarily flange sections, should be repaired. Based on available information from previous inspection reports, an allowance of 10% of steel weight of floorbeams is included for replacement. Actual locations of steel repairs will be identified during the repair plan development.

Deck Stringers:

Deteriorated sections of deck stringers should be repaired. Stringers with extensive deterioration should be replaced. Based on available information from previous inspection reports, an allowance of 10% of steel weight of floorbeams is included for replacement. Actual locations of steel repairs will be identified during the repair plan development. It is intended that stringer replacements could be accomplished without removing the steel roadway deck.

Sidewalk Framing:

Previous inspection report photos indicate severe section loss in sidewalk framing members. Existing framing members will not match the geometry needed for new sidewalk framing, and therefore new steel members will be required.

E. Truss Framing

As noted previously, 10% section loss of truss framing members is common for the entire length of the bridge, with heavy section loss in isolated areas and at panel points / eyebar ends.

E1 Eyebars:

As noted in Section III, the low allowable stress of the existing eyebars severely restricts the live load capacity of the bridge.

Eyebar ends exhibit severe section loss and deterioration at panel points. In addition, the eyebar “slack” observed indicates that tension load is not equally shared in eyebar sets. The “slack” could be caused by original manufacturing practices or by localized eyebar deterioration at the pin connections.

The forged eyebars and rods were fabricated by bending each end of a flat bar around mandrels and then forging the bent end back on the bar to form the “eye”; or by upsetting the ends of large bars and punching the eyes while the metal is hot. This process was not done within strict fabrication tolerances and could result in varying lengths between eyes (which could result in “slack” in some eyebars), along with microcracks, defects, and inclusions within the forged region. Internal defects and microcracks may not be visible, particularly if they are obscured from view by gusset plates or adjacent eyebars. Stress levels around the pin, near the forged region, can be higher than the nominal stress in the rectangular section of the bar. Eyebars are prone to cracking in regions of high stress – with cracks propagating into the forged zone and eventually

resulting in failure. Eyebars carrying additional load for which they were not intended to carry (due to “slack” within an eyebar set) can be even more susceptible to cracking due to high stress.

The steel composition used to fabricate the eyebars and rods is not known; older steel generally has a high carbon content and can be brittle compared to modern steel and be very susceptible to stress corrosion cracking. Regardless of the steel composition, the high level of corrosion at the eyebar ends increases the susceptibility of any steel to stress corrosion cracking.

Existing microcracks in eyebar ends are difficult to detect because of the relatively large extent of the potential failure area, the presence of other eyebars or gusset plates which obscure the full surface of the eyebar ends, and the presence of paint or rust which can mask a crack.

Furthermore, it should be noted that failed eyebars have been recovered from similar bridges showing rust on part of the “eye” failure surface, with a bright area on the remaining “eye” failure surface. This condition indicates long-term stability of a partially cracked member, which then experienced a sudden or brittle failure. If live load on a bridge is reduced through load posting, a bar may function in this condition, undetected, for an extended period of time. A crack in one eyebar, within a set of eyebars, may shift more load to adjacent eyebars within the set – load for which those eyebars were not designed. Freeze-thaw conditions common in Iowa allow moisture to penetrate partial cracks and exert considerable force to continue the propagation of the cracks until failure of the member occurs.

Finally, it should be noted that the remaining fatigue life of this bridge is uncertain. It is not possible to accurately determine the remaining life of tension members because the history of load cycles is unknown. Fatigue is a process by which a material is weakened through cyclic loading (stressing). A weakening of the material occurs due to the propagation of cracks. The stress cycles producing these cracks do not need to be large, and are generally significantly lower than the member stress at full member capacity. Fatigue cracks can ultimately result in brittle fracture of a member, which can cause sudden and potentially catastrophic failures.

Based on the high levels of deterioration and corrosion at panel points, the inaccessibility of eyebar and rod ends for inspection and monitoring to ensure public safety, and the high stresses common to all eyebar ends, in addition to the low allowable stress of the eyebars, we recommend that all eyebars and rods be replaced with similar elements using modern materials.

Replacement members can be fabricated to closely match the geometry of original members to maintain historic integrity. Our concept for member replacement is to provide each tension member in two pieces, which are to be bolted together in the field. One end will be provided with pre-drilled bolt holes, and the other end will be field-drilled using the opposite end as a drilling

template. This method is intended to maintain the correct distance between panel points to maintain existing truss geometry.

Careful design of splice geometry will be needed during final design; splices may need to be staggered, and in some locations welded splices with acceptable fatigue profiles may be needed.

E2 Rods:
Previous inspection reports do not include many comments regarding rod truss members. For reasons similar to those noted for eyebars, we recommend that all rods be replaced with similar elements using modern materials and using the same procedure as noted for eyebars.

E3 Built-up Members:
Members exhibiting severe section loss should be repaired or replaced. Damaged lacing should be repaired, and missing rivets should be replaced with equivalent size high tensile strength bolts. It was previously noted that pack rust has accumulated between member elements and caused local distortion. There is no simple method to repair this damage. The deformations are similar to local buckling initiated under high member stress; the deformations can reduce the member capacity by an amount that is not readily quantified.

E4 Pinned Panel Points:
Significant rust and corrosion is present in gusset plates at panel points. In addition, previous repairs that utilized gusset plates at panel points to transfer forces in horizontal and lateral bracing, as well as the seal welds at pinned connections, affect the function of the pinned connections. These details restrict free and independent rotation of truss members at the pin connections, thereby resulting in bending stresses for which the truss members were not designed. Truss members are designed to function in pure axial tension or compression. A cursory computer modeling of the locked joints indicates that the resulting bending stresses are relatively small. However, the resulting local stresses at the panel points are high. These high stresses are placed on the eyebar ends, which already are subjected to stresses which can be higher than the rectangular section of the eyebars.

The pins and gusset plates at panel points should be replaced, and the bracing connections should be reconfigured to prevent interference with member rotation.

E5 Horizontal and Lateral Bracing:
Horizontal and lateral bracing should be closely observed during the final plan preparation to identify damaged or deficient members for repair or replacement.

E6 Truss Bearings:
Existing truss bearings should be blasted clean of all corrosion and pack rust, and should be repainted. Truss bearings should be closely observed during final plan preparation to identify elements that need to be replaced.

F. Steel Piers (Piers 1 and 2)

Previous inspection reports do not include any specific notes on steel piers. Review and comments regarding these piers are beyond the scope of this evaluation.

G. Structural Steel Paint

The date of the last paint job for this bridge is unknown, although the 1964/1978/1984 repair plans indicate that the entire bridge is to be cleaned and painted. It is not known if the previous paint layer was fully removed prior to repainting. Approximately 30% of the paint coating exhibits heavy deterioration. Many areas of eyebar ends, built-up members, panel point regions, floorbeams, and stringers exhibit significant surface rust with some pack rust. The adhesion of the remaining sound areas of paint is not known.

The steel framing members must be protected from the environment to prevent further corrosion, extremely costly future repairs, and to protect the integrity of the structure. We have identified two paint options as described in the following paragraphs. The estimated costs for these options are based on previous experience with projects requiring paint removal, power washing, spot painting, containment of paint residue, difficult structure configuration, and working in difficult site conditions. Although costs are based on historic data, recent bid costs for blasting, cleaning, and painting steel have been somewhat lower than the costs used for estimating. Appropriate paint topcoats, selected for economy, durability, sun exposure conditions, and compatibility with existing paints would be applied to the steel members for these options.

In the absence of a paint specification on the repair plans, we assume that the paint system was in compliance with then-current Iowa DOT Standard Specifications. The standard paint system of the 1964 and 1972 Standard Specifications includes a red lead primer and red lead first field-coat paint. We expect this paint to be characterized as hazardous. The standard paint system of the 1977 Standard Specifications includes a zinc silicate primer with vinyl top coat. We do not expect this paint to be characterized as hazardous. Results of scrape samples obtained during rehabilitation plan preparation should be used as a basis for plan specifications of paint removal, transport, and disposal.

For the purpose of estimating costs for this project, we assume the existing paint to be hazardous. The estimated cost for full paint removal and repainting of the entire structure is $25/sf. The estimated cost of partial paint removal and repainting of only that area is $40/sf, and an additional $13/sf for overcoating the remaining surfaces. The discrepancy in unit cost is due to the labor-intensive nature of this work and the cost of containment for a smaller area. These costs vary with containment methods adaptable to the site, site access, ground levels, platform construction, quantities, etc. Removal costs for nonhazardous paint may decrease the removal unit cost by approximately $5/sf. These costs do not include painting of the roadway deck.

Overcoating an existing paint is less costly than complete paint removal and repainting, but presents numerous uncertainties. The quality of the existing paint to be overcoated may not be consistent. The surface preparation before overcoating is extremely important to provide proper adhesion, and stringent quality control requiring a close level of inspection and monitoring would be necessary to ensure full removal of noncompatible paint. Paint can be overcoated as long as the base coat exhibits strong adhesion. The life expectancy of an overcoat paint (approximately 75% the life of a full removal / repainting) is relatively short.

The two options we have identified are as follows:

1. Partial paint removal and full overcoat. Includes removal of failed paint and recoating with primer, and overcoating the entire structure with a suitable topcoat. Blasting to bare steel is anticipated primarily at panel points, stringers, floorbeams, and bearing areas exhibiting paint failure where corroded and deteriorated steel is exposed. Enclosures would be used to allow blasting for surface preparation; shrouded hand tools would not be cost effective for the quantity of area involved or for the shape of structural elements. Judgment must be exercised at the time of the painting operations to identify zones desired to be cleaned and painted. Utilizing 30% of the structure surface area to be blasted, the estimated cost for this option is $1,750,000 assuming the existing paint is considered hazardous (or $1,612,000 if the existing paint is not considered hazardous). This is not our recommended option for this structure.

2. Full paint removal / repaint entire bridge framing. The City may prefer to reduce the uncertainty of future painting costs and the considerations of the unknown remaining paint life. Inspection and monitoring would be significantly simplified compared to the partial paint removal and full overcoat option. This option would require full containment of the framing and removal of existing paint and primer to bare steel. The life expectancy of a completely new paint system is approximately 20 years; it is not uncommon for modern quality paint systems to last considerably longer. The estimated cost for this option is $2,056,000 assuming the existing paint is considered hazardous (or $1,645,000 if the existing paint is not considered hazardous).

In either option, new structural members will be shop painted. Existing structural members to remain will be field painted. We recommend Option 2 for this structure due to the age of the existing paint system, the relatively small cost difference, and the long-term benefits of a new paint system applied to bare steel.

H. Concrete Pier and Footings (Piers 1, 2, 3)

Review and comments regarding this pier is beyond the scope of this evaluation. However, concrete repair and other minor repairs that may be necessary may be considered to be included in the cost allowance for contingencies.

I. Concrete Abutments

Previous inspection reports indicate that concrete repairs are necessary for both abutments. Review and comments regarding these abutments is beyond the scope of this evaluation. However, based on our site observation, we estimate that approximately 60% of the face of each abutment backwall is cracked and spalled. The condition of subdrains and the supporting elements (piles, spread footings) is not known. The abutments were not evaluated for plumb or apparent settlement. For the purpose of comparing bridge rehabilitation with bridge replacement, we have included a cost for concrete repair of the faces of abutment backwalls in Section V; other possible abutment repair costs are not included.

Spalled and delaminated concrete and areas of corroded rebar should be removed. Corroded reinforcing steel should be cleaned and replaced if necessary. Concrete should be replaced using cast-in-place concrete repair methods. It is possible that significant reinforcing steel and concrete deterioration may be exposed during the removal process. If necessary, the abutments should then be restored to their required capacity. It is likely that this can be achieved by splicing additional steel reinforcing bars with existing reinforcing steel, if foundation elements such as piles remain in good condition. A waterproofing membrane should be applied to the bearing seats.

J. Approach Slabs / Approach Pavement / Approach Sidewalks

North Approach: The approach will need to be reconstructed to provide a concrete approach slab and to match the new deck width; it is assumed that the approach reconstruction will be 70 feet long to match standard Iowa DOT bridge approach details. Additional roadway reconstruction to introduce a low point and potentially relocate the intakes may have drainage benefits, limiting or eliminating the volume of water that is running over the north abutment, but this isn't considered critical to the improvements and hasn't been included in the cost opinion. Reconstruction of the tops of the existing intakes is included with the proposed improvements.

South Approach: Removal and reconstruction of the existing slab west of the westernmost joint line will be necessary to accommodate the narrower bridge deck. The remaining portion of the PCC slab will not need to be removed and replaced. It is recommended that the seal coat roadway south of the PCC slab be reconstructed to ensure runoff drains south as intended, but those costs have not been included in the cost estimate for bridge rehabilitation.

Sidewalk: The sidewalk approaches to the bridge will also be reconstructed to accommodate the new 5 foot sidewalk and meet the requirements of the ADA. Reconstruction will extend through the roadway reconstruction limits to the north, and to Dankwardt Park Drive to the south, and will include construction of a new curb ramp crossing Dankwardt Park Drive and removal of the existing ramp crossing South Main Street. Pipe railing along the west side of the sidewalk is

assumed to be of acceptable height and in good condition, and no costs are included for railing removal and replacement.

Replacement of tension members, and replacing gusset plates and pins at panel points, will require external shoring to support the truss. This shoring will fully support the truss, eliminating member stress and thereby allowing truss members to be removed and replaced. It is recommended that only one panel point on a truss at any time be disassembled to help maintain overall truss geometry and stability. Replacement tension members will be fabricated in two pieces and spliced in the field. A truss shoring concept is included in the Appendix of this report.

A conceptual procedure for disassembling a single panel point and replacing truss members is as follows:

a. Install truss shoring.
b. Cut / grind welds from tension members and pins at bottom chord panel point.
c. Cut lateral and horizontal bracing gusset plates loose at panel point.
d. Install temporary lateral and horizontal bracing.
e. Cut tension bars.
f. Remove each cut tension member.
g. Remove pin and gusset plates from vertical member by removing rivets.
h. Install new gusset plates using high strength bolts.
i. Install new pin and replacement tension member end sections.
j. Move to one panel point with opposite end of a replacement tension member, and repeat steps b thru i at that panel point.
k. Splice two new replacement tension member end sections together using bolted connection (pull end sections together and clamp, field drill holes thru one end section using other end section as a drilling template, and bolt using high strength bolts).
l. Repeat step k at other panel points with opposite tension member end sections.
m. Move to top panel point, and remove pin and gusset plates by removing rivets.
n. Install new gusset plates using high strength bolts.
o. Install new pins and replacement tension member end sections.
p. Repeat steps b thru i at other panel points with opposite tension member end sections.
q. Repeat steps as necessary at remaining truss panel points.

V. ESTIMATED COSTS

This section provides an estimate of costs to perform the recommended repairs outlined in Section IV. These costs are approximate only and intended for preliminary budgeting purposes. Because of the specialized nature of many of the items, construction sequence, limited quantities of some repairs, difficult site conditions, and large structure dimensions, it is likely that repair unit prices will be somewhat higher than normal, making costs difficult to estimate. Traffic control or project schedule requirements may also impact the repair cost. All estimated costs are in September 2012 dollars.

In addition, a life cycle cost analysis was performed for the bridge rehabilitation and the bridge replacement for the purposes of comparing both options. The life cycle cost analysis is through 50 years. The Rehabilitation option includes the cost of initial repairs, plus repairs at 25 years. The remaining useful life of the structure is assumed as 50 years if recommended repairs are implemented. The Replacement option includes cost of initial construction, plus repairs at 25 and 50 years. Repair costs are based on experience with repairs for bridges of similar construction. It is assumed that the replacement bridge has remaining life after 50 years, and is reflected as a credit at the end of the 50 year duration.

An annualized rate of construction inflation is assumed at 3%, and an annualized investment rate of return is assumed at 5%. Engineering costs are not included in these cost estimates. The selected bridge replacement option for the comparison, taken from the Replacement of Cascade Bridge Report of September 2009, is a 5-span pretensioned prestressed concrete beam bridge with an estimated cost of $3,500,000.

The construction cost estimate for Rehabilitation and the life cycle cost analysis are shown on the following pages.

Cascade Bridge Rehabilitation
Construction Cost Estimate

9/14/2012

Bridge Items

No.	Item	Unit	Quantity	Unit Cost	Cost	Item Cost	Note
1	**External Shoring to Support Truss during Repairs**					**$ 332,420**	**Truss shoring sequence shown on concept in Appendix**
	Falsework design and submittal by Professional Engineer	LS	1	$ 25,000.00	$ 25,000		
	Install timber foundations	EA	46	$ 300.00	$ 13,800		Provide timber grillage at each shoring post, 85 kip min. working load
	Weld steel brackets to floorbeams for shoring bearing points	EA	44	$ 400.00	$ 17,600		May be left in place upon completion of construction, approx 100 lbs ea.
	Furnish vertical shoring posts	LF	1680	$ 14.00	$ 23,520		Furnish quantity matches quantity required for Span 3
	Install vertical shoring posts	LF	2340	$ 10.00	$ 23,400		Posts to be re-used as required
	Span 1 - 240 LF						
	Span 2 - 240 LF						
	Span 3 - 1680 LF						
	Span 4 - 180 LF						
	Strengthen floorbeam / truss connection to support truss dead load	EA	44	$ 400.00	$ 17,600		May be left in place upon completion of construction, approx 100 lbs ea.
	Furnish horizontal & transverse struts for vertical shoring posts	LF	1350	$ 20.00	$ 27,000		Furnish quantity matches quantity required for Span 3
	Install horizontal & transverse struts for vertical shoring posts	LF	2370	$ 10.00	$ 23,700		Struts to be re-used as required
	Span 1 - 330 LF						
	Span 2 - 420 LF						
	Span 3 - 1350 LF						
	Span 4 - 270 LF						
	Furnish diagonal tension cables for bracing vertical shoring posts	LF	2200	$ 13.00	$ 28,600		Furnish quantity matches quantity required for Span 3
	Install diagonal tension cables for vertical shoring posts	LF	3340	$ 10.00	$ 33,400		Cables to be re-used as required
	Span 1 - 180 LF						
	Span 2 - 680 LF						
	Span 3 - 2200 LF						
	Span 4 - 280 LF						
	Furnish & install ties between truss and vertical shoring posts	EA	58	$ 400.00	$ 23,200		
	Span 1 - 8 EA						
	Span 2 - 8 EA						
	Span 3 - 36 EA						
	Span 4 - 6 EA						
	Remove shoring sequentially after both trusses in a span are completed	LS	1	$ 25,000.00	$ 25,000		
	Remove timber foundations	EA	46	$ 100.00	$ 4,600		
	Daily monitoring of of shoring / bracing to ensure support of truss	LS	1	$ 36,000.00	$ 36,000		Survey posts to maintain plumb, adjust jacks & cables as necessary
	Restore disturbed areas of ground surface	LS	1	$ 10,000.00	$ 10,000		
2	**Sequentially Replace Tension Members, Pins, Gusset Plates**	**LB**	92202	$ 18.00	$ 1,659,636	**$ 1,659,636**	**Sequence for this procedure shown in Section IV**
	Approximate weight of structural steel per span						
	Span 1 - 12,827 LB						
	Span 2 - 12,827 LB						
	Span 3 - 64,813 LB						
	Span 4 - 1,735 LB						
3	**Disposal of Removed Steel**	**LB**	92202	$ 0.50	$ 46,101	**$ 46,101**	**Assumes paint on steel contains hazardous levels of lead, chromium, etc.**
4	**Floorbeam Repair**	**LB**	5100	$ 15.00	$ 76,500	**$ 76,500**	**Assumes 10% of floorbeams by weight require replacement due to corrosion**
5	**Stringer Repair / Replacement**	**LB**	15400	$ 10.00	$ 154,000	**$ 154,000**	**Assumes 10% of stringers by weight require replacement due to corrosion**
6	**Sidewalk Rail Repair**	**LF**	920	$ 125.00	$ 115,000	**$ 115,000**	**Remove, blast, repair, repair, reinstall**
7	**Sidewalk Reconstruction**					**$ 256,035**	
	Remove existing sidewalks	SF	3680	$ 12.00	$ 44,160		
	Remove existing steel framing	LB	33277	$ 0.50	$ 16,639		
	Reframe sidewalk with new framing (similar configuration)	LB	33277	$ 1.25	$ 41,596		
	Rebuild concrete sidewalk on steel form deck, with exp joints	SF	4600	$ 30.00	$ 138,000		
	New curb plate	LB	15640	$ 1.00	$ 15,640		
8	**Expansion Joint Repair**	**LF**	30	$ 90.00	$ 2,700	**$ 2,700**	**South roadway joint**
9	**Abutment Repair**					**$ 143,360**	**Approximated by visual observation, does not include wing walls**
	Concrete Repair - north abutment	SF	552	$ 120.00	$ 66,240		
	Concrete Repair - south abutment	SF	576	$ 120.00	$ 69,120		
	Waterproof membrane on abutment seats	SF	320	$ 25.00	$ 8,000		
10	**Clean and Paint Structural Steel Framing**	**LS**				**$ 2,056,425**	**Trusses, stringers, floorbeams - blast clean and repaint**
		SF	82257	$ 25.00	$ 2,056,425		

Roadway Items

11	**Removal of Pavement**	SY	278	$ 11.00	$ 3,058	**$ 3,058**	
12	**Bridge Approach, RK-20**	SY	270	$ 190.00	$ 51,300	**$ 51,300**	
13	**Removal of Sidewalk**	SY	46	$ 11.00	$ 506	**$ 506**	
14	**PCC Sidewalk, 6"**	SY	55	$ 50.00	$ 2,750	**$ 2,750**	
15	**Detectable Warnings**	SF	12	$ 40.00	$ 480	**$ 480**	
16	**Earth Shoulder Construction**	STA	1.7	$ 180.00	$ 306	**$ 306**	
17	**Removal of Intake**	EACH	2	$ 1,000.00	$ 2,000	**$ 2,000**	
18	**Intake, SW-545, Modified, Top Only**	EACH	2	$ 4,000.00	$ 8,000	**$ 8,000**	
19	**Traffic Control**	LS	1	$ 5,000.00	$ 5,000	**$ 5,000**	

BRIDGE SUBTOTAL	**$ 4,842,177**
ROADWAY SUBTOTAL	**$ 73,400**
COMBINED SUBTOTAL	**$ 4,915,577**
Contingency (15%)	**$ 737,337**
Mobilization (8%)	**$ 452,233**
TOTAL	**$ 6,105,146**

Cascade Bridge Life Cycle Cost Analysis

Rehabilitation vs Replacement

9/14/2012

Option A: **Rehabilitation**

Load posted at 20 tons

Bridge Deck Area (Existing): 17594 SF

Interval	0	25	50			Assumed Repair Cost at Interval		Assume 30% Blast plus Full Overcoat	
Year	2012	2037	2062						
Rehab	$6,105,000								
Repair	$1,145,014	$3,877,425		2012 Cost	$1,851,880	$20	/SF +	$1,500,000	Paint
Replace	$1,338,025		$15,343,671	2012 Cost	$3,500,000				
Present Cost	$8,588,040								
Remaining Value	$0			$0		At End of Useful Life			

Net Present Cost **$8,588,040**

Option B: **Replacement** **5-Span PPC Beam Bridge with Concrete Frame Piers**

Legal bridge - no load posting required

Bridge Deck Area (New): 19968 SF

Interval	0	25	50			Assumed Repair Cost at Interval	
Year	2012	2037	2062				
Replace	$3,500,000						
Repair	$246,924	$836,171		2012 Cost	$399,360	$20	/SF
Repair	$343,513		$3,939,203	2012 Cost	$898,560	$45	/SF
Present Cost	$4,090,437						
Remaining Value	-$501,759			-$5,753,877		Assume 3/8 of Useful Life Remains	

Net Present Cost **$3,588,677**

ASSUMPTIONS:

Construction Inflation assumed (annualized)	=	3%	0.03
Investment rate of return assumed (annualized)	=	5%	0.05

Negative value denotes credit

VI. CONCLUSIONS

The existing truss bridge is in poor condition, and is currently unsafe for public use. In our opinion, the options the City should consider are removal without replacement, rehabilitation, or replacement. If the bridge were to remain in place and closed to vehicle traffic, it will continue to deteriorate and be a threat to public safety based upon the danger of sudden, catastrophic collapse.

Based on the most recent information provided by the City, the cost of demolition of the bridge is estimated at $200,000. This cost may not be eligible for outside funding. If a bridge is desired at this location, the least cost option is a replacement with a modern structure engineered to current standards for load capacity and functionality. The replacement bridge could likely be constructed in one season.

Rehabilitation of the bridge is the most costly option, with an estimated cost of $6,105,000, and would most likely require two seasons to complete the work. The inclusion of the bridge on the National Register significantly complicates rehabilitation.

The life expectancy for the existing bridge, after rehabilitation, is about 50 years maximum; for a new bridge, about 80 years minimum. A new bridge would be open to all legal traffic, while the rehabilitated truss would require load posting estimated at 20 tons, based upon the procedures outlined in this report.

In our opinion, this bridge is not a good candidate for rehabilitation. A new bridge has a significant advantage regarding present cost, presents much less risk of unknowns, would not be fracture critical, and would last much longer than the repaired structure. In addition, the open nature of the truss bridge, with the steel grid decking, does not protect the structure below from the elements like a new bridge with a solid concrete deck. Painting this large truss bridge is extremely expensive, and is a recurring cost if the bridge is kept in service. For a 50 year life cycle, the net present cost of the rehabilitated existing bridge is $8,588,000, including future maintenance. In comparison, the net present cost of the replacement bridge is $3,589,000, including future maintenance and remaining value.

VI. RECOMMENDATIONS

If the City desires a bridge at this location, a replacement is the best option, based on findings included in this report, including initial costs, eligibility for outside funding, load capacity, functionality, estimated life, and periodic maintenance requirements. This option also eliminates the risk associated with the continued use of members which are currently 116 years old in the existing truss.

The existing bridge should remain closed until it is removed or rehabilitated, due to safety issues. If the bridge is to be removed, it should be archived in accord with required state and federal requirements for bridges on the National Register. If desired, representative pieces from the bridge could be displayed near the site, along with photographs, to commemorate the structure for posterity.

If the City elects to proceed with rehabilitation, the next step would be to select an engineer and authorize the preliminary and final design for the bridge rehabilitation. Preliminary and final design should take approximately 12-18 months to accomplish, including the time needed for historical coordination and agency reviews. An expected design fee for rehabilitation of a bridge of this type could range between 8% and 12% of the rehabilitation construction cost. Additionally, construction review and observation services could range between 4% and 10% of the rehabilitation construction cost, depending on the necessary scope of services and possible availability of city staff. Following completion of design and approval by the City, Iowa DOT, and other review agencies, the project can be let for construction.

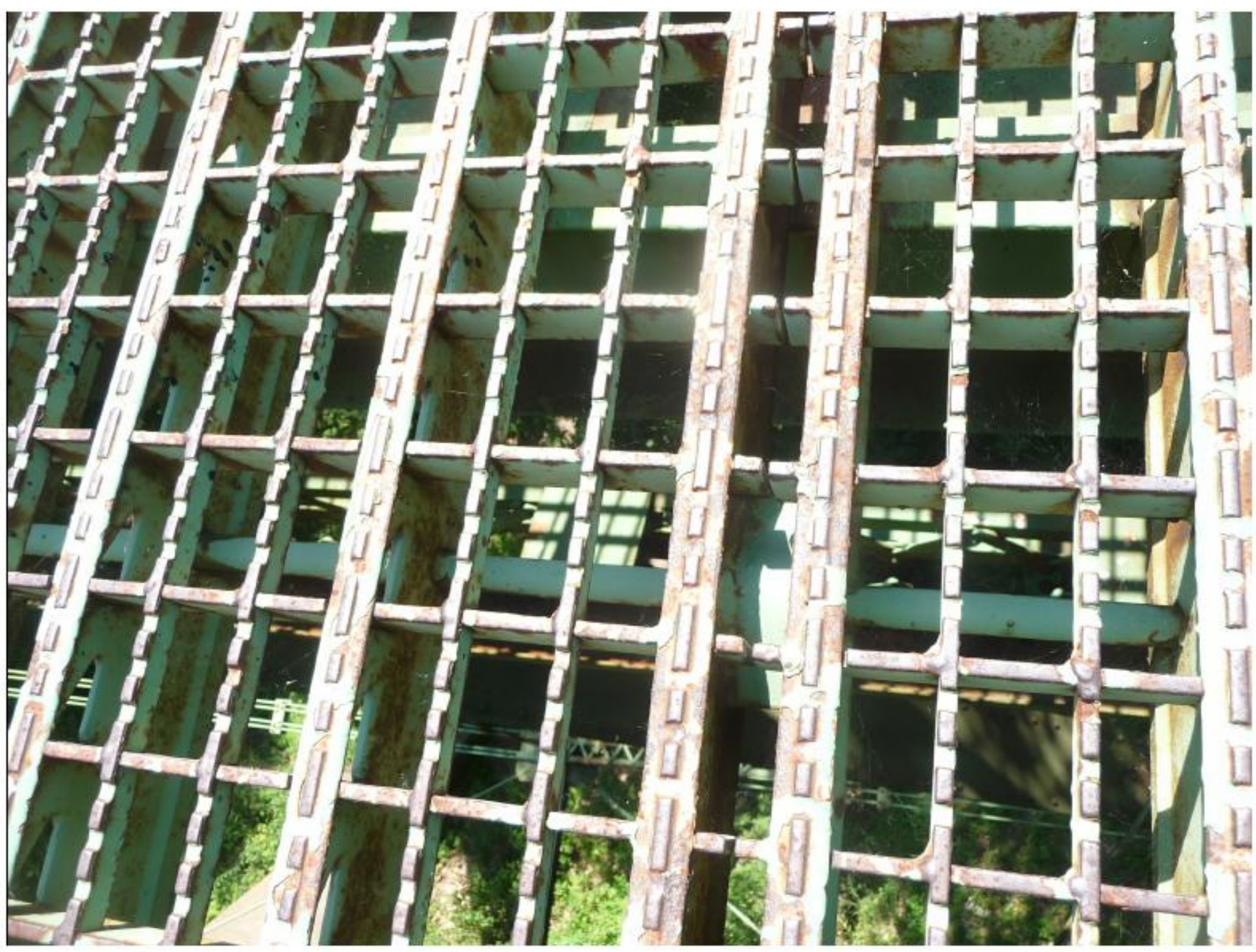
This concludes our structural evaluation of the Cascade Bridge. Please contact us if you have any questions or if you would like to discuss any material contained in this report. Shuck-Britson Inc. appreciates this opportunity to serve the City of Burlington.

Cascade Bridge Rehabilitation Evaluation

PHOTO LIST

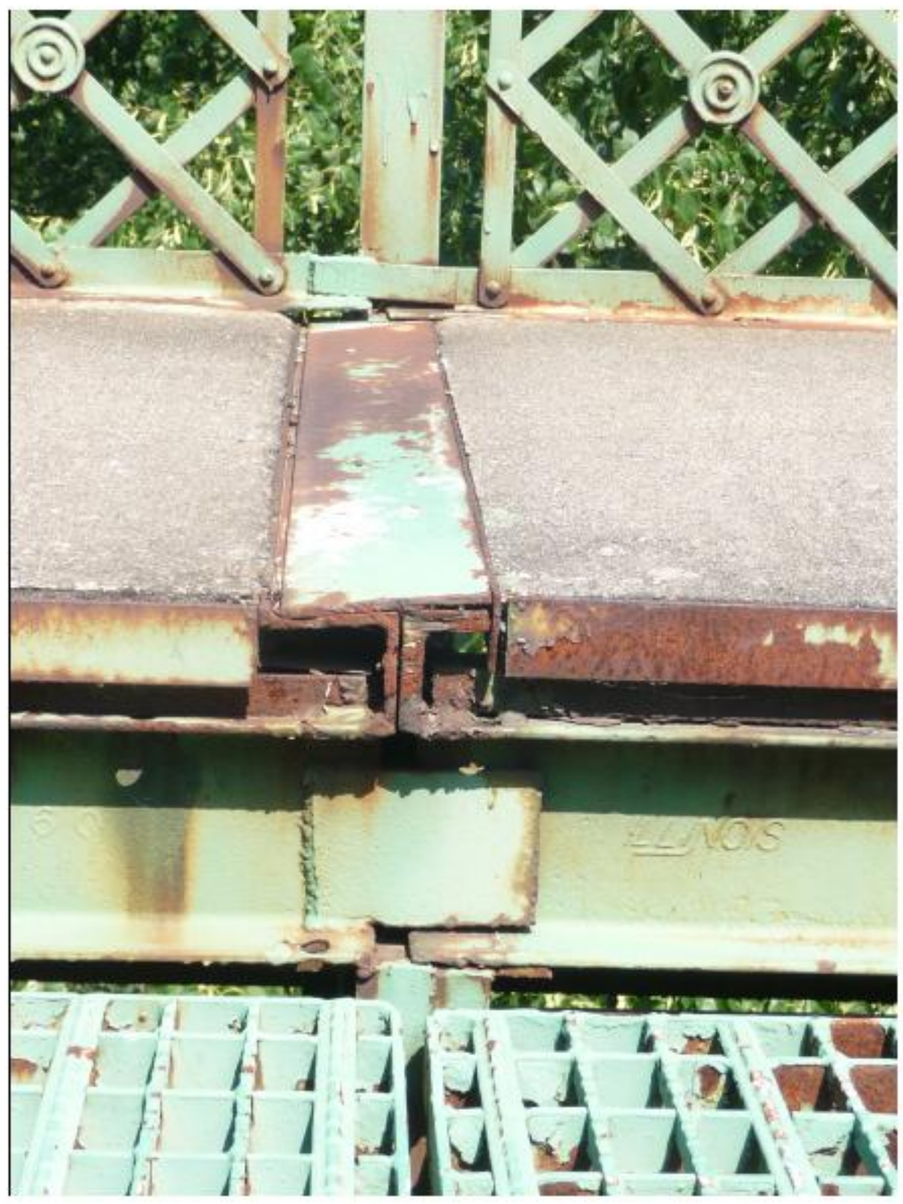
A1, A2	Steel bridge roadway deck
A3	South abutment deck expansion joint
A4	Pier 1 deck expansion joint
A5	North abutment deck expansion joint
B1	Sidewalk deterioration
C1	Sidewalk rail
D1 thru D6	Floorbeam corrosion and section loss
D7	Deck stringer corrosion and section loss
D8	Sidewalk framing corrosion and section loss
E1.1	Damaged eyebar
E1.2 thru E1.10	Eyebar corrosion and section loss at panel points
E1.11 thru E1.13	Previous repair eyebar reinforcement
E1.14	Previous repair seal welds at panel points
E2.1, E2.2	Previous repair rod reinforcement
E3.1 thru E3.3	Built-up member corrosion and section loss
E3.4	Damaged lacing on built-up member
E3.5	Missing rivet from built-up member
E3.6 thru E3.10	Pack rust distortion on built-up members
E4.1 thru E4.4	Eyebar / panel point reinforcing details
E5.1	Typical horizontal and lateral braces
E6.1, E6.2	Heavy surface rust at bearings
E6.3, E6.4	Section loss at truss bearings

A1



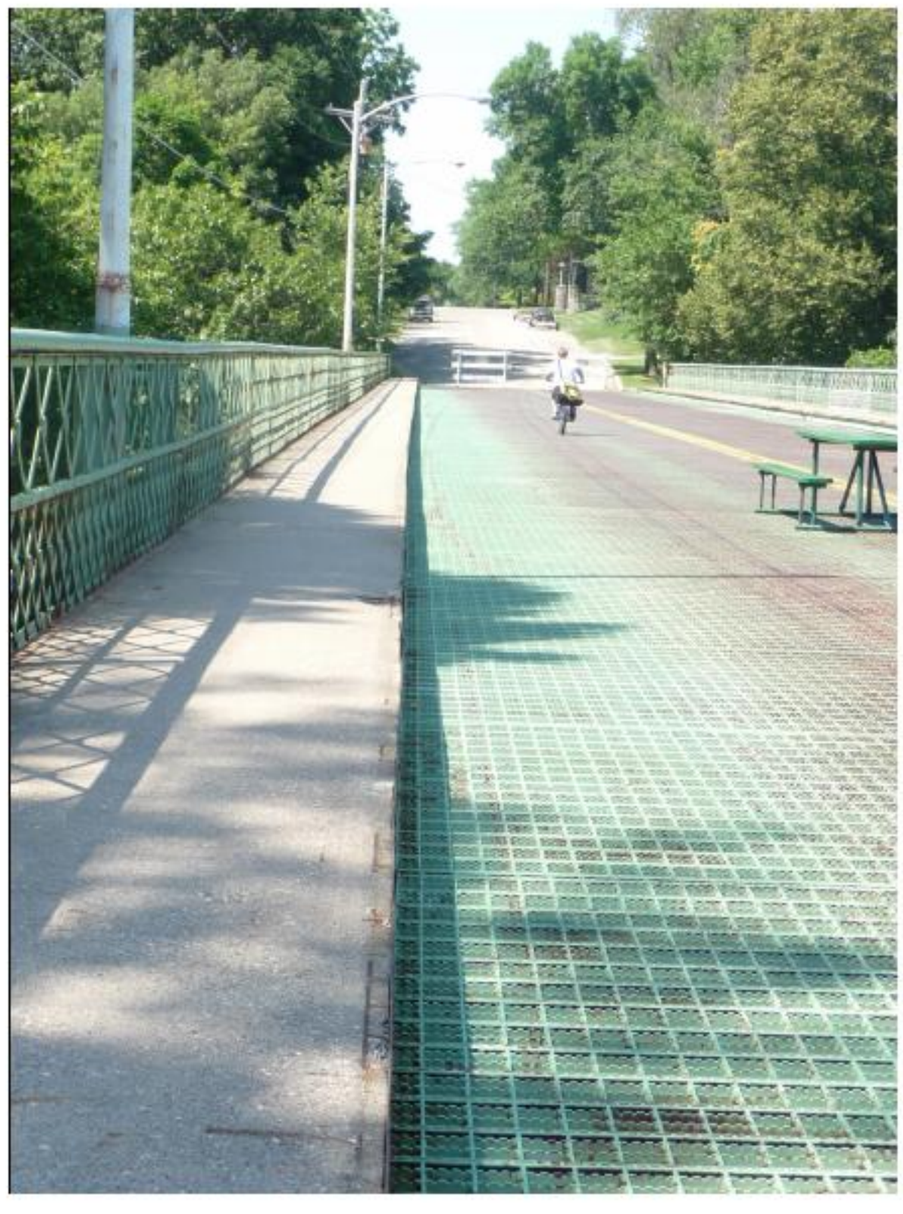
A2

A3



A4

A5



B1

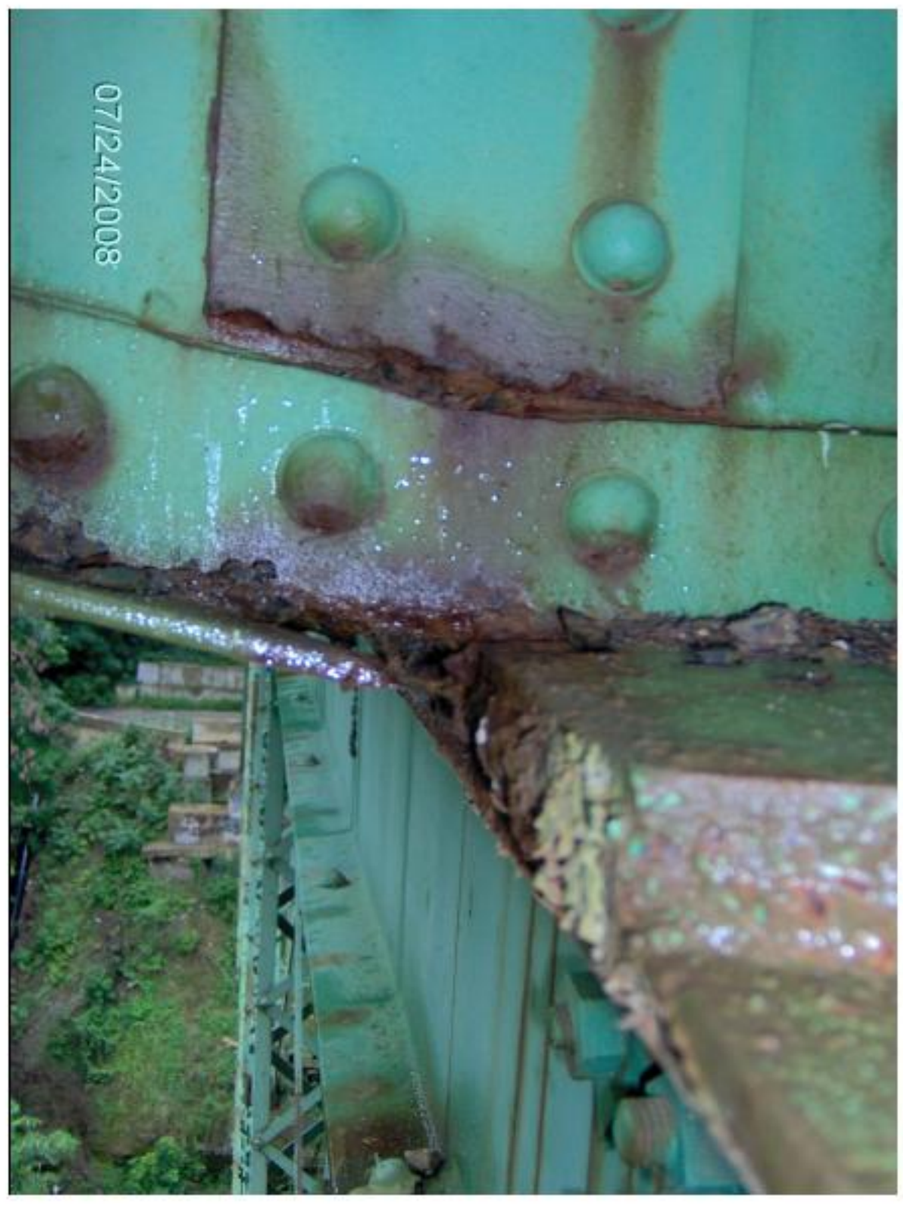
C1

D1

D2

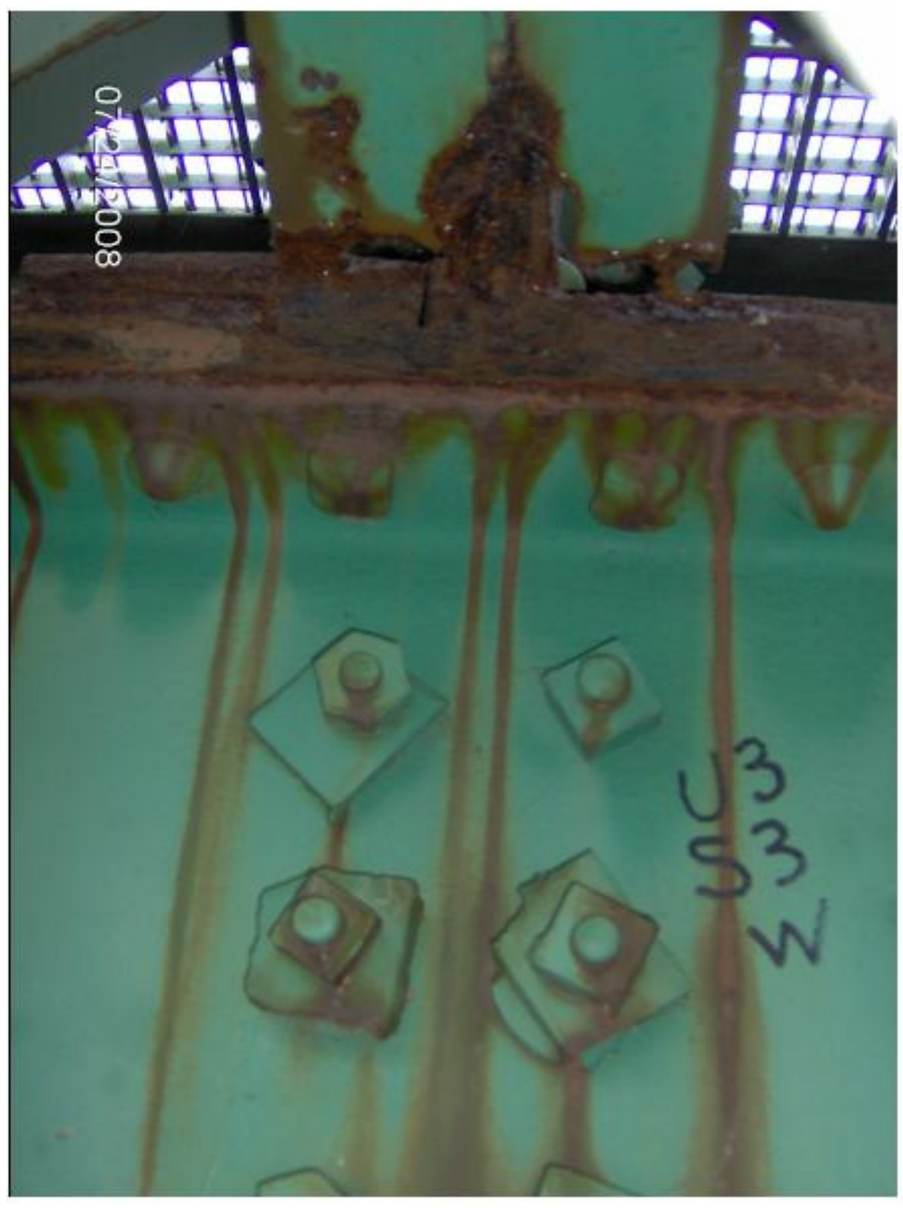
D3

D4



D5

D6



D7

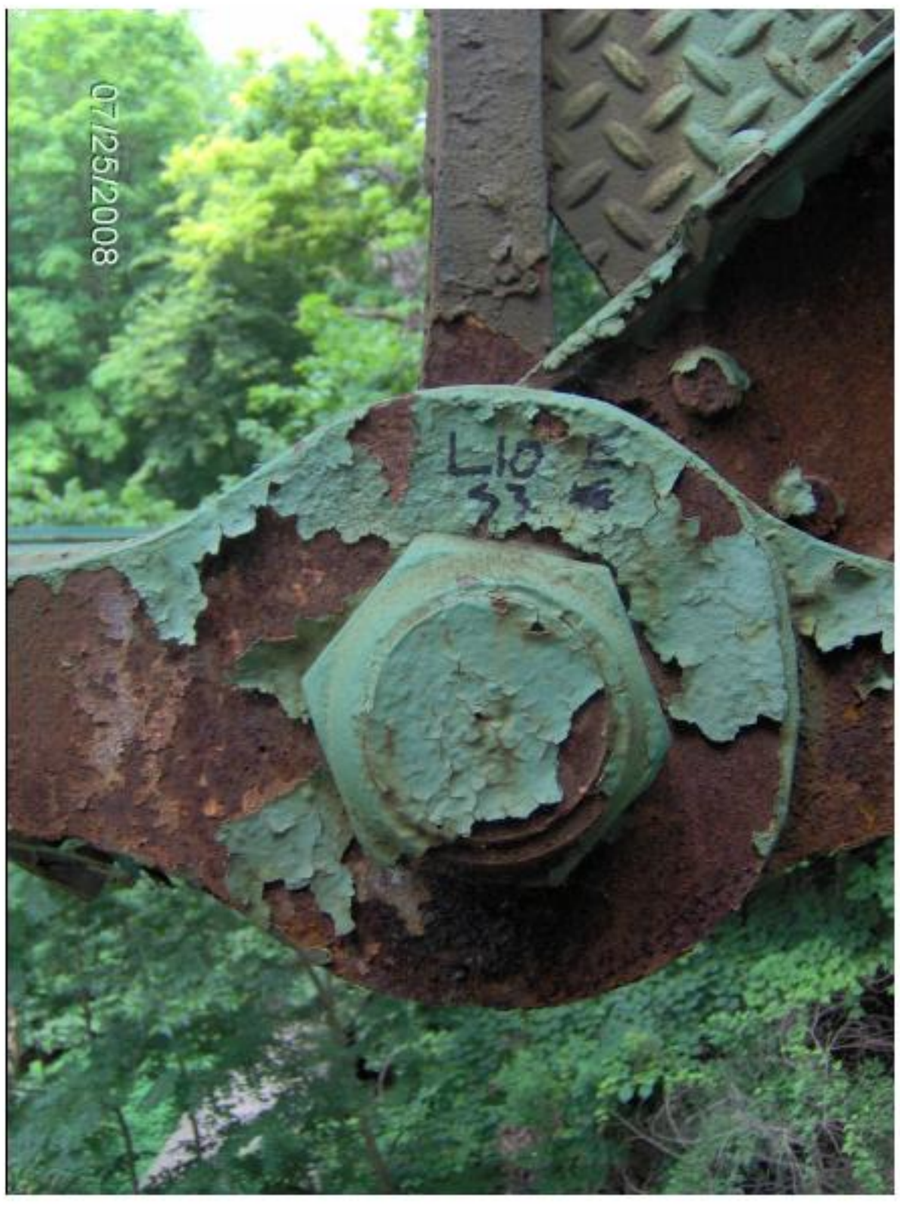
D8

E1_1

E1_10

E1_11

E1_12



E1_13

E1_14

E1_2

E1_3

E1_4

E1_5

E1_6

E1_7

E1_8

E1_9

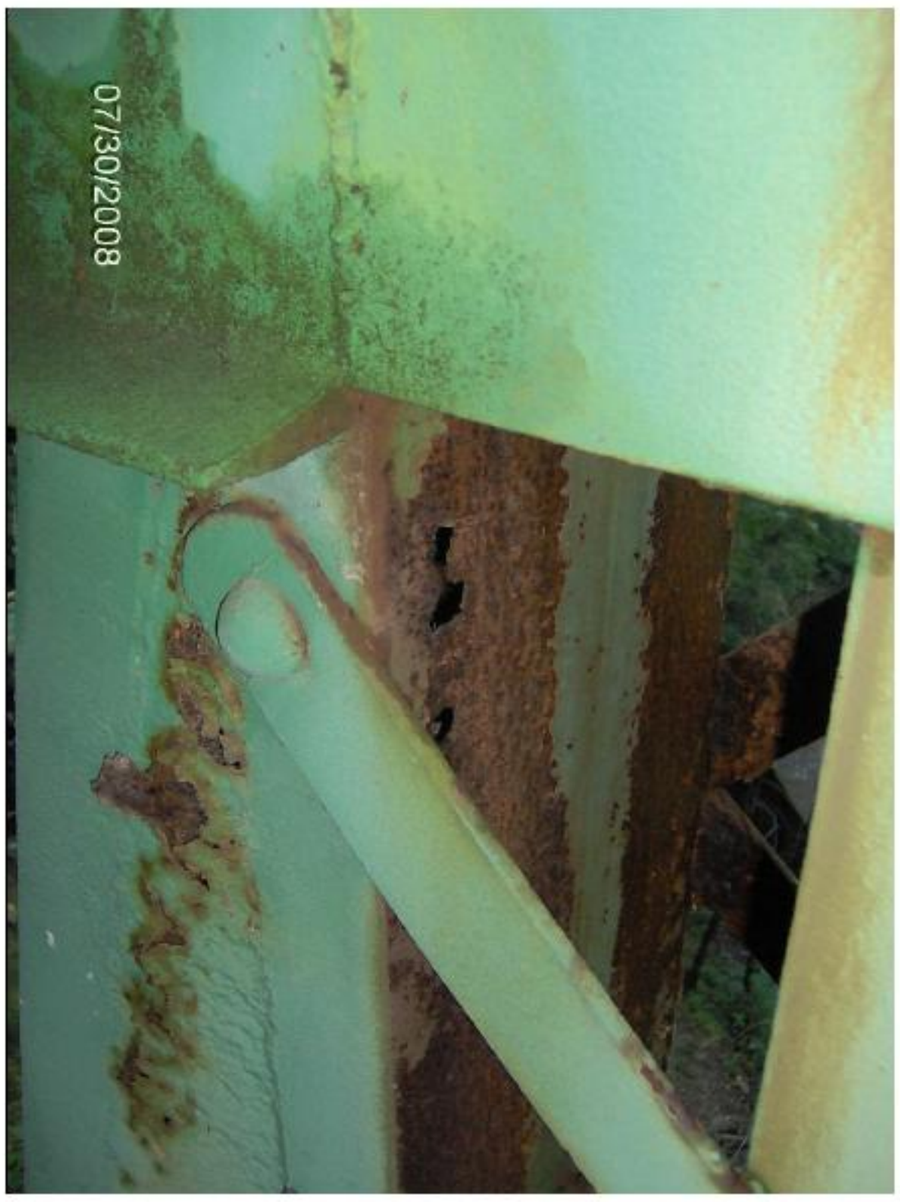
E2_1

E2_2

E3_1

E3_10

E3_2



E3_3

E3_4

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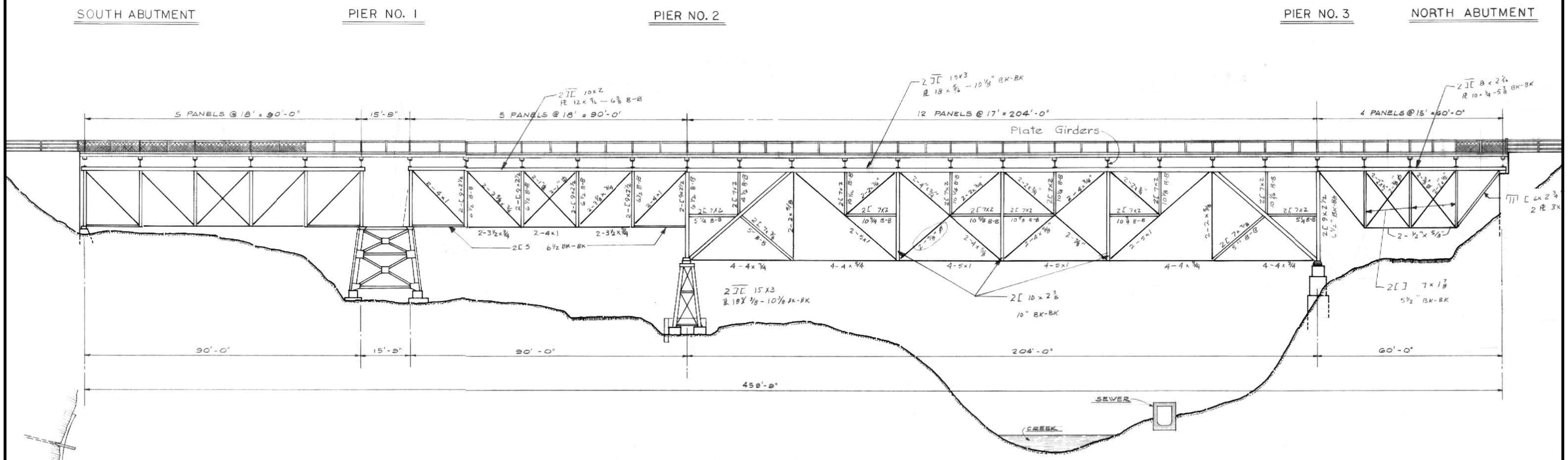
E5_1

E6_1

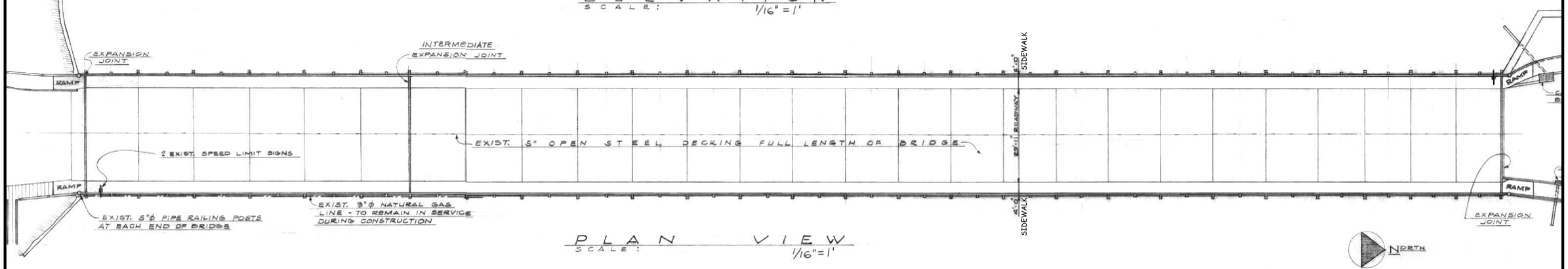
E6_2

E6_3

E6_4



ELEVATION
SCALE: 1/16" = 1'



PLAN VIEW
SCALE: 1/16" = 1'

NOTE: EXISTING CONDITIONS SHOWN.

459'-9x29'-11 DECK TRUSS BRIDGE

SITUATION PLAN

CITY OF BURLINGTON

SEPTEMBER, 2012

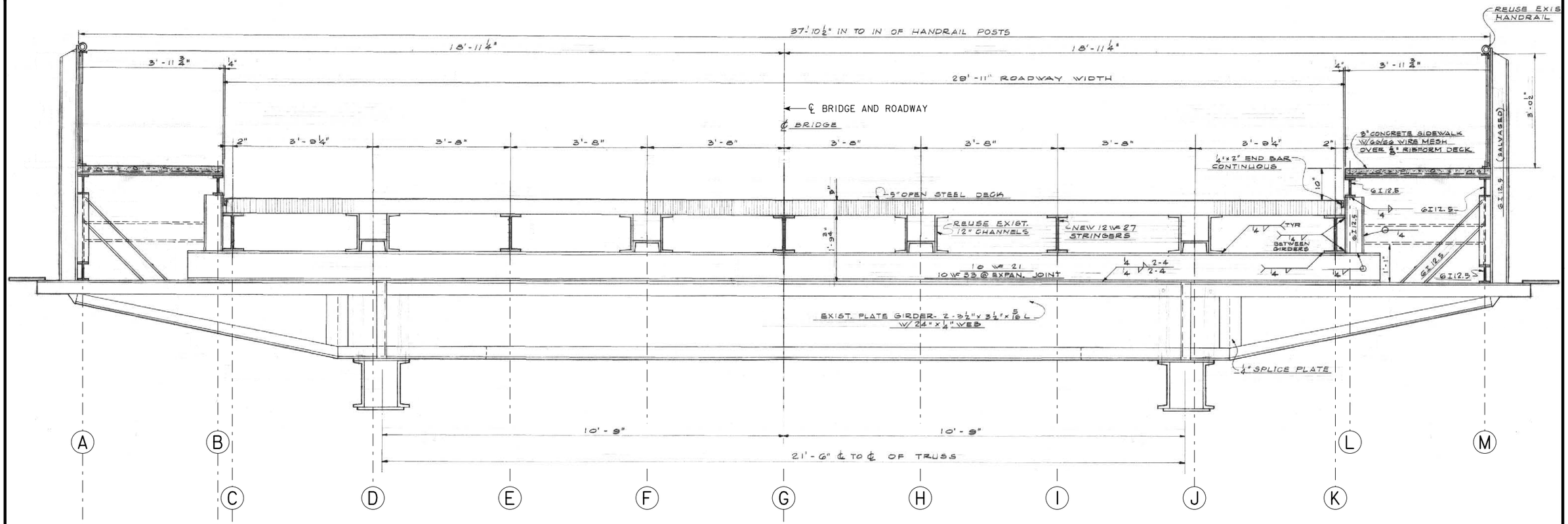
DESIGN TEAM SHUCK-BRITSON INC. 2409 GRAND AVENUE DES MOINES, IOWA 50312 515-243-4477

S.B.I. NO. 112.0583.03

DES MOINES COUNTY

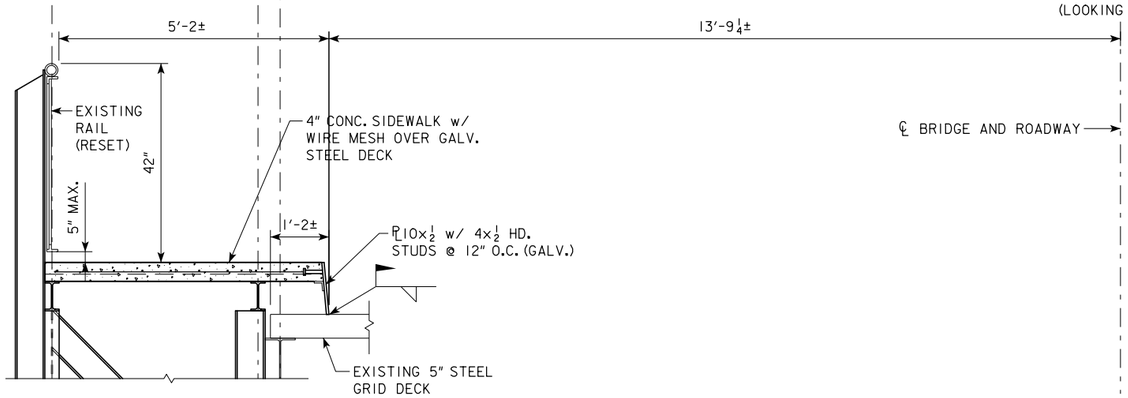
SHEET NUMBER V.01

9/13/2012 1:46:12 PM V8i_IDOTBridge N:\2012_Projects\112.0583 Cascade\Cadd\1120583.brg 1120583_V.01 11x17bw_pdf.plt



SECTION THRU EXISTING BRIDGE DECK FRAMING

(LOOKING NORTH)



PART SECTION

PROPOSED SIDEWALK FRAMING

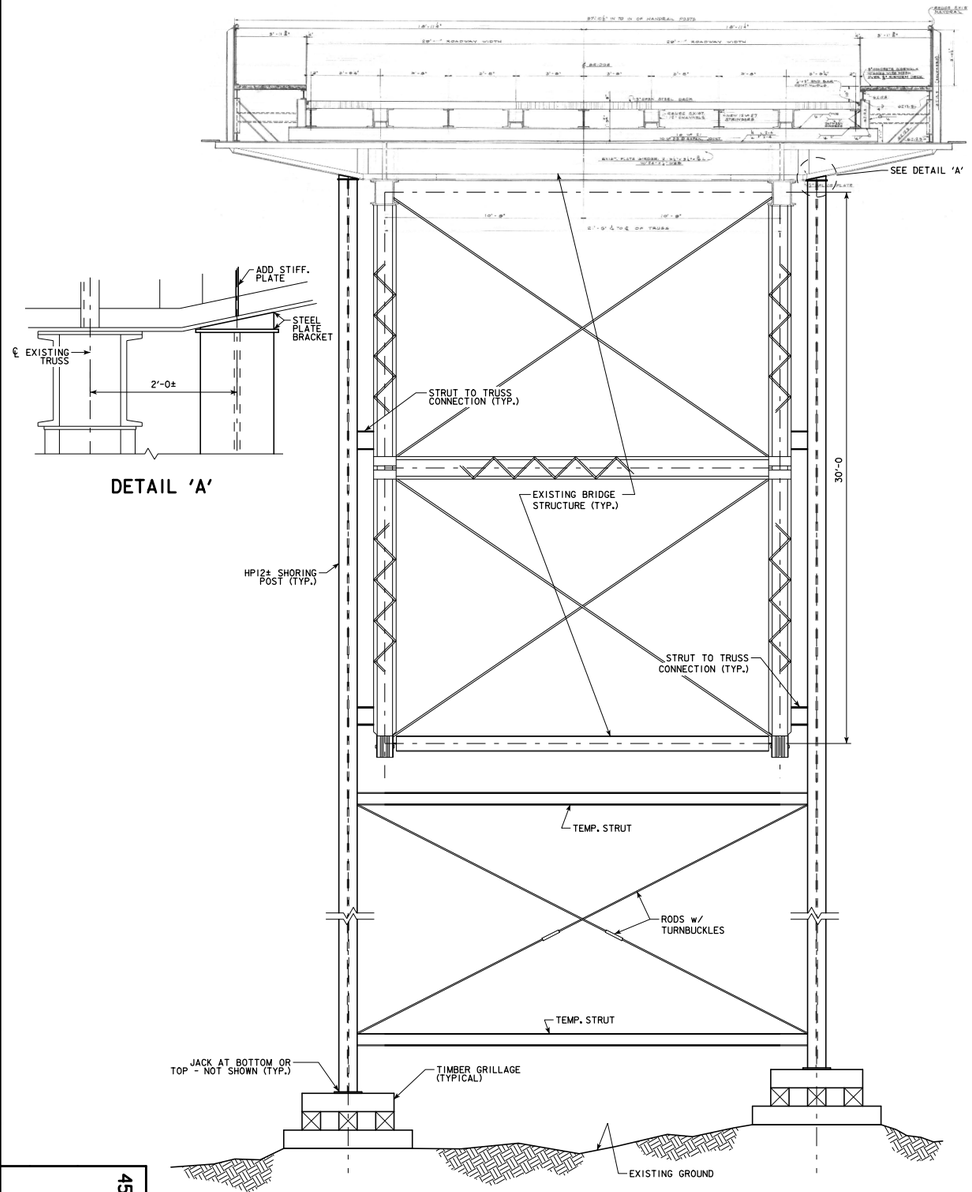
(TYPICAL BOTH SIDES)

459'-9x29'-11 DECK TRUSS BRIDGE

DECK CROSS SECTION

CITY OF BURLINGTON

SEPTEMBER, 2012



TYPICAL SECTION AT SHORING

(LOOKING NORTH)

TRUSS SHORING SEQUENCE

1. SHORING TO BE DESIGNED BY A PROFESSIONAL ENGINEER LICENSED IN THE STATE OF IOWA.
2. EACH SHORING POST AND FOUNDATION SHALL HAVE MINIMUM 85 KIP WORKING LOAD.
3. INSTALL TIMBER FOUNDATIONS.
4. FABRICATE AND INSTALL WELDED STEEL BRACKETS AT ENDS OF FLOORBEAMS FOR HP12 SHORING POSTS TO BEAR UPON.
5. SET VERTICAL HP12 SHORING POSTS AND JACK TO TIGHT CONDITION TO SUPPORT TRUSS DEAD LOAD.
6. STRENGTHEN CONNECTION OF FLOORBEAMS TO TOP CHORDS TO ACCOMMODATE TRUSS DEAD LOAD.
7. INSTALL BRACING FOR HP12 SHORING POSTS.
   a. HORIZONTAL STRUTS LONGITUDINAL AND TRANSVERSE.
   b. DIAGONAL TENSION CABLES.
   c. TIES STRUT BETWEEN SHORING POSTS AND TRUSS.
8. MONITOR SHORING AND BRACING DAILY TO ENSURE TIGHT CONDITION AND MAINTAIN FULL DEAD LOAD FORCE IN JACK. SURVEY HP12 SHORING POSTS TO MAINTAIN PLUMB, ADJUST CABLES AND JACKS AS NEEDED.
9. REMOVE SHORING SEQUENTIALLY AFTER BOTH TRUSSES IN A SPAN ARE COMPLETED.
10. REMOVE TIMBER FOUNDATIONS.

459'-9x29'-11 DECK TRUSS BRIDGE

TRUSS SHORING CONCEPT

CITY OF BURLINGTON

SEPTEMBER, 2012

DES MOINES COUNTY

DESIGN TEAM SHUCK-BRITSON INC. ARCHITECTS ENGINEERS PLANNERS

S.B.I. NO. 1120583.03

SHEET NUMBER V.03

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SHUCK-BRITSON INC.
Consulting Engineers
2409 Grand Avenue
Des Moines, Iowa 50312

TRUSS LOAD RATING SUMMARY - EXISTING CONDITION

City of Burlington
Cascade Bridge

DATE: 9/14/2012

Truss:	90' Truss	**YEAR BUILT**	1896
	Span 1 & 2	**NO. LANES**	2

Member	Area	Allowable Stress				Member Capacity		Dead Load	LL+I Capacity		LL+I Bar Forces	Inventory Rating (tons)	Operating Rating (tons)
		Inventory		Operating		Inven.	Operating	(kips)	Inven.	Operating	HS-20	HS-20	HS-20
	(sq. in)	Tens. (ksi)	Comp. (ksi)	Tens. (ksi)	Comp. (ksi)	(kips)	(kips)		(kips)	(kips)			
L0L1	4.71	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
L1L2	5.26	8.00	NA	11.00	NA	42.08	57.86	33.80	8.28	24.06	76.00	3.92	11.40
L2L3	8.00	8.00	NA	11.00	NA	64.00	88.00	50.70	13.30	37.30	84.70	5.65	15.85
L3L4	5.26	8.00	NA	11.00	NA	42.08	57.86	33.80	8.28	24.06	76.00	3.92	11.40
L4L5	4.71	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
U0U1	21.35	NA	11.33	NA	14.12	241.81	301.53	33.80	208.01	267.73	76.00	98.53	126.82
U1U2	21.35	NA	11.33	NA	14.12	241.81	301.53	50.70	191.11	250.83	108.40	63.47	83.30
U2U3	21.35	NA	11.33	NA	14.12	241.81	301.53	51.00	190.81	250.53	127.10	54.05	70.96
U3U4	21.35	NA	11.33	NA	14.12	241.81	301.53	50.70	191.11	250.83	108.40	63.47	83.30
U4U5	21.35	NA	11.33	NA	14.12	241.81	301.53	33.80	208.01	267.73	76.00	98.53	126.82
U0L0	11.72	NA	11.06	NA	13.79	129.62	161.62	50.70	78.92	110.92	76.00	37.38	52.54
U1L1	11.72	NA	11.06	NA	13.79	129.62	161.62	33.80	95.82	127.82	76.00	45.39	60.55
U2L2	11.72	NA	11.06	NA	13.79	129.62	161.62	17.20	112.42	144.42	79.10	51.17	65.73
U3L3	11.72	NA	11.06	NA	13.79	129.62	161.62	17.20	112.42	144.42	79.10	51.17	65.73
U4L4	11.72	NA	11.06	NA	13.79	129.62	161.62	33.80	95.82	127.82	76.00	45.39	60.55
U5L5	11.72	NA	11.06	NA	13.79	129.62	161.62	50.70	78.92	110.92	76.00	37.38	52.54
U0L1	8.00	8.00	NA	11.00	NA	64.00	88.00	47.80	16.20	40.20	107.50	5.43	13.46
U1L2	5.44	8.00	NA	11.00	NA	43.49	59.80	23.90	19.59	35.90	78.40	8.99	16.48
U2L3	2.00	8.00	NA	11.00	NA	16.00	22.00	0.50	15.50	21.50	47.50	11.75	16.29
L2U3	2.00	8.00	NA	11.00	NA	16.00	22.00	0.50	15.50	21.50	47.50	11.75	16.29
L3U4	5.44	8.00	NA	11.00	NA	43.49	59.80	23.90	19.59	35.90	78.40	8.99	16.48
L4U5	8.00	8.00	NA	11.00	NA	64.00	88.00	47.80	16.20	40.20	107.50	5.43	13.46

SHUCK-BRITSON INC.
Consulting Engineers
2409 Grand Avenue
Des Moines, Iowa 50312

TRUSS LOAD RATING SUMMARY - EXISTING CONDITION

City of Burlington
Cascade Bridge

DATE: 9/14/2012

Truss:	204' Truss	**YEAR BUILT**	1896
	Span 3	**NO. LANES**	2

Member	Area	Allowable Stress				Member Capacity		Dead Load	LL+I Capacity		LL+I Bar Forces	Inventory Rating (tons)	Operating Rating (tons)
		Inventory		Operating		Inven.	Operating	(kips)	Inven.	Operating	HS-20	HS-20	HS-20
	(sq. in)	Tens. (ksi)	Comp. (ksi)	Tens. (ksi)	Comp. (ksi)	(kips)	(kips)		(kips)	(kips)			
L0L2	12.00	8.00	NA	11.00	NA	96.00	132.00	112.20	-16.20	19.80	134.33	-4.34	5.31
L2L4	12.00	8.00	NA	11.00	NA	96.00	132.00	102.00	-6.00	30.00	122.10	-1.77	8.85
L4L6	20.00	8.00	NA	11.00	NA	160.00	220.00	145.20	14.80	74.80	188.42	2.83	14.29
U2U4	25.54	NA	11.88	NA	14.82	303.42	378.50	173.40	130.02	205.10	211.07	22.18	34.98
U4U6	25.54	NA	11.88	NA	14.82	303.42	378.50	193.80	109.62	184.70	238.97	16.51	27.82
U1M1	5.74	NA	10.83	NA	13.50	62.16	77.49	18.00	44.16	59.49	62.50	25.44	34.27
U2L2	2.50	8.00	NA	11.00	NA	20.00	27.50	9.00	11.00	18.50	31.21	12.69	21.34
U3M3	5.74	NA	10.83	NA	13.50	62.16	77.49	18.00	44.16	59.49	62.50	25.44	34.27
U4L4	8.98	NA	11.56	NA	14.42	103.81	129.49	54.00	49.81	75.49	83.86	21.38	32.41
U5M5	5.74	NA	10.83	NA	13.50	62.16	77.49	18.00	44.16	59.49	62.50	25.44	34.27
U6L6	8.98	NA	11.56	NA	14.42	103.81	129.49	18.00	85.81	111.49	78.64	39.28	51.04
LOM1	26.67	NA	11.59	NA	14.46	309.11	385.65	149.63	159.48	236.02	179.12	32.05	47.44
M1U2	26.67	NA	11.59	NA	14.46	309.11	385.65	136.02	173.09	249.63	162.87	38.26	55.18
M1L2	5.74	NA	9.00	NA	11.21	51.66	64.35	13.61	38.05	50.74	47.19	29.03	38.71
U2M3	10.00	8.00	NA	11.00	NA	80.00	110.00	95.22	-15.22	14.78	125.33	-4.37	4.25
M3L4	10.00	8.00	NA	11.00	NA	80.00	110.00	81.61	-1.61	28.39	109.15	-0.53	9.36
M3U4	3.00	8.00	NA	11.00	NA	24.00	33.00	13.61	10.39	19.39	47.25	7.92	14.77
U4M5	6.00	8.00	NA	11.00	NA	48.00	66.00	40.81	7.19	25.19	85.73	3.02	10.58
M5L6	5.00	8.00	NA	11.00	NA	40.00	55.00	13.61	26.39	41.39	72.44	13.11	20.57
L4M5	1.20	8.00	NA	11.00	NA	9.60	13.20	0.00	9.60	13.20	47.73	7.24	9.96
M5U6	3.00	8.00	NA	11.00	NA	24.00	33.00	0.00	24.00	33.00	58.95	14.66	20.15

SHUCK-BRITSON INC.
Consulting Engineers
2409 Grand Avenue
Des Moines, Iowa 50312

TRUSS LOAD RATING SUMMARY - EXISTING CONDITION

City of Burlington
Cascade Bridge

DATE: 9/14/2012

Truss:	60' Truss	**YEAR BUILT**	1896
	Span 4	**NO. LANES**	2

Member	Area	Allowable Stress				Member Capacity		Dead Load	LL+I Capacity		LL+I Bar Forces	Inventory Rating (tons)	Operating Rating (tons)
		Inventory		Operating		Inven.	Operating	(kips)	Inven.	Operating	HS-20	HS-20	HS-20
	(sq. in)	Tens. (ksi)	Comp. (ksi)	Tens. (ksi)	Comp. (ksi)	(kips)	(kips)		(kips)	(kips)			
L1L2	3.13	8.00	NA	11.00	NA	25.00	34.38	16.95	8.05	17.43	44.57	6.50	14.07
U0U1	10.58	NA	11.27	NA	14.06	119.24	148.75	16.95	102.29	131.80	55.65	66.17	85.26
U1U2	10.58	NA	11.27	NA	14.06	119.24	148.75	22.60	96.64	126.15	79.02	44.03	57.47
U1L1	5.74	NA	10.83	NA	13.50	62.16	77.49	20.34	41.82	57.15	75.86	19.85	27.12
U2L2	5.74	NA	10.83	NA	13.50	62.16	77.49	13.56	48.60	63.93	68.64	25.49	33.53
U0L1	8.94	8.00	NA	11.00	NA	71.52	98.34	26.48	45.04	71.86	86.90	18.66	29.77
U1L2	2.00	8.00	NA	11.00	NA	16.00	22.00	8.83	7.17	13.17	49.25	5.24	9.63
L1U2	1.20	8.00	NA	11.00	NA	9.60	13.20	0.00	9.60	13.20	17.58	19.66	27.03

SHUCK-BRITSON INC.
Consulting Engineers
2409 Grand Avenue
Des Moines, Iowa 50312

TRUSS LOAD RATING SUMMARY - TENSION MEMBERS REPLACED

City of Burlington
Cascade Bridge

DATE: 9/14/2012

Truss:	90' Truss	**YEAR BUILT**	1896
	Span 1 & 2	**NO. LANES**	2

Member	Area	Allowable Stress				Member Capacity		Dead Load	LL+I Capacity		LL+I Bar Forces	Inventory Rating (tons)	Operating Rating (tons)
		Inventory		Operating		Inven.	Operating	(kips)	Inven.	Operating	HS-20	HS-20	HS-20
	(sq. in)	Tens. (ksi)	Comp. (ksi)	Tens. (ksi)	Comp. (ksi)	(kips)	(kips)		(kips)	(kips)			
L0L1	4.71	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
L1L2	5.26	27.00	NA	37.50	NA	142.02	197.25	33.80	108.22	163.45	76.00	51.26	77.42
L2L3	8.00	27.00	NA	37.50	NA	216.00	300.00	50.70	165.30	249.30	84.70	70.26	105.96
L3L4	5.26	27.00	NA	37.50	NA	142.02	197.25	33.80	108.22	163.45	76.00	51.26	77.42
L4L5	4.71	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
U0U1	21.35	NA	11.33	NA	14.12	241.81	301.53	33.80	208.01	267.73	76.00	98.53	126.82
U1U2	21.35	NA	11.33	NA	14.12	241.81	301.53	50.70	191.11	250.83	108.40	63.47	83.30
U2U3	21.35	NA	11.33	NA	14.12	241.81	301.53	51.00	190.81	250.53	127.10	54.05	70.96
U3U4	21.35	NA	11.33	NA	14.12	241.81	301.53	50.70	191.11	250.83	108.40	63.47	83.30
U4U5	21.35	NA	11.33	NA	14.12	241.81	301.53	33.80	208.01	267.73	76.00	98.53	126.82
U0L0	11.72	NA	11.06	NA	13.79	129.62	161.62	50.70	78.92	110.92	76.00	37.38	52.54
U1L1	11.72	NA	11.06	NA	13.79	129.62	161.62	33.80	95.82	127.82	76.00	45.39	60.55
U2L2	11.72	NA	11.06	NA	13.79	129.62	161.62	17.20	112.42	144.42	79.10	51.17	65.73
U3L3	11.72	NA	11.06	NA	13.79	129.62	161.62	17.20	112.42	144.42	79.10	51.17	65.73
U4L4	11.72	NA	11.06	NA	13.79	129.62	161.62	33.80	95.82	127.82	76.00	45.39	60.55
U5L5	11.72	NA	11.06	NA	13.79	129.62	161.62	50.70	78.92	110.92	76.00	37.38	52.54
U0L1	8.00	27.00	NA	37.50	NA	216.00	300.00	47.80	168.20	252.20	107.50	56.33	84.46
U1L2	5.44	27.00	NA	37.50	NA	146.77	203.85	23.90	122.87	179.95	78.40	56.42	82.63
U2L3	2.00	27.00	NA	37.50	NA	54.00	75.00	0.50	53.50	74.50	47.50	40.55	56.46
L2U3	2.00	27.00	NA	37.50	NA	54.00	75.00	0.50	53.50	74.50	47.50	40.55	56.46
L3U4	5.44	27.00	NA	37.50	NA	146.77	203.85	23.90	122.87	179.95	78.40	56.42	82.63
L4U5	8.00	27.00	NA	37.50	NA	216.00	300.00	47.80	168.20	252.20	107.50	56.33	84.46

SHUCK-BRITSON INC.
Consulting Engineers
2409 Grand Avenue
Des Moines, Iowa 50312

TRUSS LOAD RATING SUMMARY - TENSION MEMBERS REPLACED

City of Burlington
Cascade Bridge

DATE: 9/14/2012

Truss:	204' Truss	**YEAR BUILT**	1896
	Span 3	**NO. LANES**	2

Member	Area	Allowable Stress				Member Capacity		Dead Load	LL+I Capacity		LL+I Bar Forces	Inventory Rating (tons)	Operating Rating (tons)
		Inventory		Operating		Inven.	Operating	(kips)	Inven.	Operating	HS-20	HS-20	HS-20
	(sq. in)	Tens. (ksi)	Comp. (ksi)	Tens. (ksi)	Comp. (ksi)	(kips)	(kips)		(kips)	(kips)			
L0L2	12.00	27.00	NA	37.50	NA	324.00	450.00	112.20	211.80	337.80	134.33	56.76	90.53
L2L4	12.00	27.00	NA	37.50	NA	324.00	450.00	102.00	222.00	348.00	122.10	65.45	102.60
L4L6	20.00	27.00	NA	37.50	NA	540.00	750.00	145.20	394.80	604.80	188.42	75.43	115.55
U2U4	25.54	NA	11.88	NA	14.82	303.42	378.50	173.40	130.02	205.10	211.07	22.18	34.98
U4U6	25.54	NA	11.88	NA	14.82	303.42	378.50	193.80	109.62	184.70	238.97	16.51	27.82
U1M1	5.74	NA	10.83	NA	13.50	62.16	77.49	18.00	44.16	59.49	62.50	25.44	34.27
U2L2	2.50	27.00	NA	37.50	NA	67.50	93.75	9.00	58.50	84.75	31.21	67.48	97.76
U3M3	5.74	NA	10.83	NA	13.50	62.16	77.49	18.00	44.16	59.49	62.50	25.44	34.27
U4L4	8.98	NA	11.56	NA	14.42	103.81	129.49	54.00	49.81	75.49	83.86	21.38	32.41
U5M5	5.74	NA	10.83	NA	13.50	62.16	77.49	18.00	44.16	59.49	62.50	25.44	34.27
U6L6	8.98	NA	11.56	NA	14.42	103.81	129.49	18.00	85.81	111.49	78.64	39.28	51.04
LOM1	26.67	NA	11.59	NA	14.46	309.11	385.65	149.63	159.48	236.02	179.12	32.05	47.44
M1U2	26.67	NA	11.59	NA	14.46	309.11	385.65	136.02	173.09	249.63	162.87	38.26	55.18
M1L2	5.74	NA	9.00	NA	11.21	51.66	64.35	13.61	38.05	50.74	47.19	29.03	38.71
U2M3	10.00	27.00	NA	37.50	NA	270.00	375.00	95.22	174.78	279.78	125.33	50.20	80.36
M3L4	10.00	27.00	NA	37.50	NA	270.00	375.00	81.61	188.39	293.39	109.15	62.14	96.77
M3U4	3.00	27.00	NA	37.50	NA	81.00	112.50	13.61	67.39	98.89	47.25	51.35	75.35
U4M5	6.00	27.00	NA	37.50	NA	162.00	225.00	40.81	121.19	184.19	85.73	50.89	77.35
M5L6	5.00	27.00	NA	37.50	NA	135.00	187.50	13.61	121.39	173.89	72.44	60.33	86.42
L4M5	1.20	27.00	NA	37.50	NA	32.40	45.00	0.00	32.40	45.00	47.73	24.44	33.94
M5U6	3.00	27.00	NA	37.50	NA	81.00	112.50	0.00	81.00	112.50	58.95	49.47	68.70

SHUCK-BRITSON INC.
Consulting Engineers
2409 Grand Avenue
Des Moines, Iowa 50312

TRUSS LOAD RATING SUMMARY - TENSION MEMBERS REPLACED

City of Burlington
Cascade Bridge

DATE: 9/14/2012

Truss:	60' Truss	**YEAR BUILT**	1896
	Span 4	**NO. LANES**	2

Member	Area	Allowable Stress				Member Capacity		Dead Load	LL+I Capacity		LL+I Bar Forces	Inventory Rating (tons)	Operating Rating (tons)
		Inventory		Operating		Inven.	Operating	(kips)	Inven.	Operating	HS-20	HS-20	HS-20
	(sq. in)	Tens. (ksi)	Comp. (ksi)	Tens. (ksi)	Comp. (ksi)	(kips)	(kips)		(kips)	(kips)			
L1L2	3.13	27.00	NA	37.50	NA	84.38	117.19	16.95	67.43	100.24	44.57	54.46	80.96
U0U1	10.58	NA	11.27	NA	14.06	119.24	148.75	16.95	102.29	131.80	55.65	66.17	85.26
U1U2	10.58	NA	11.27	NA	14.06	119.24	148.75	22.60	96.64	126.15	79.02	44.03	57.47
U1L1	5.74	NA	10.83	NA	13.50	62.16	77.49	20.34	41.82	57.15	75.86	19.85	27.12
U2L2	5.74	NA	10.83	NA	13.50	62.16	77.49	13.56	48.60	63.93	68.64	25.49	33.53
U0L1	8.94	27.00	NA	37.50	NA	241.38	335.25	26.48	214.90	308.77	86.90	89.03	127.91
U1L2	2.00	27.00	NA	37.50	NA	54.00	75.00	8.83	45.17	66.17	49.25	33.02	48.37
L1U2	1.20	27.00	NA	37.50	NA	32.40	45.00	0.00	32.40	45.00	17.58	66.35	92.15