AN INVESTIGATION OF A PREFABRICATED STEEL TRUSS

GIRDER BRIDGE WITH A COMPOSITE CONCRETE DECK

by

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ABSTRACT

Steel truss girder bridges are an efficient and aesthetic option for highway crossings. Their relatively light weight compared with steel plate girder systems make them a desirable alternative for both material savings and constructability. A prototype of a welded steel truss girder constructed with an integral concrete deck has been proposed as a potential alternative for accelerated bridge construction (ABC) projects in Montana. This system consists of a prefabricated welded steel truss girder topped with a concrete deck that can be cast at the fabrication facility (for ABC projects) or in the field after erection (for conventional projects). To investigate possible solutions to the fatigue limitations of certain welded member connections in these steel truss girders, bolted connections between the diagonal tension members and the top and bottom chords of the steel truss girders were evaluated.

A 3D finite element model was used to more accurately represent the distribution of lane and truckloads to the individual steel truss girders. This distribution was compared to an approximate factor calculated using an equivalent moment of inertia with expressions for steel plate girders from AASHTO. A 2D analytical model was used to investigate the fatigue strength of the bolted and welded connections for both a conventional cast in place deck system and an accelerated bridge deck system (cast integral with the steel truss girder).

Truss members and connections for both construction alternatives were designed using loads from AASHTO Strength I, Fatigue I, Fatigue II, and Service II load combinations. A comparison was made between the two steel truss girder configurations and 205 ft. steel plate girder used in a previously designed bridge over the Swan River. Material and fabrication estimates suggest the cost of the conventional and accelerated construction methods is 10% and 26% less, respectively, than the steel plate girder designed for the Swan River crossing.

CHAPTER ONE

INTRODUCTION

A prototype bridge structure has been proposed as a potential alternative for accelerated bridge construction (ABC) projects in Montana. Accelerated bridge construction is rapidly gaining momentum in the United States as a common bridge building practice due to the increased safety and decreased impact on the public that results from the associated reduced construction times. The proposed system consists of a prefabricated welded steel truss girder topped with a composite concrete deck cast-in-place at the fabrication facility. These composite members are transported to the site, where they are set next to each other on a prepared foundation to create the bridge.

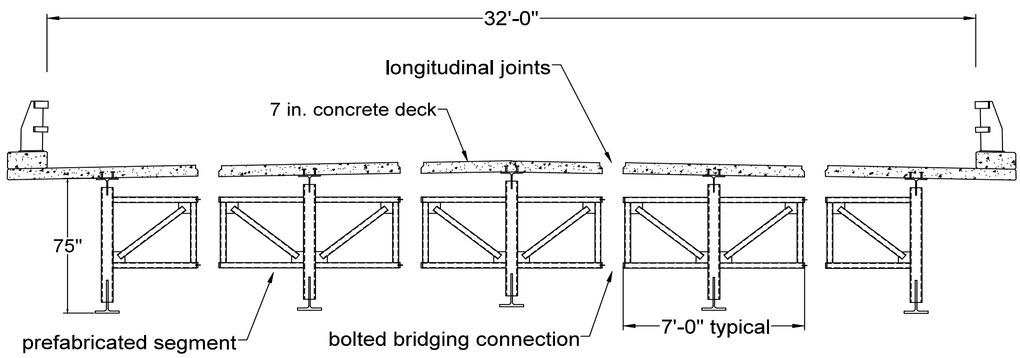
Description of Proposed Prefabricated Bridge System

Allied Steel completed preliminary designs for three different prefabricated steel truss girder/integral concrete deck bridge systems, namely two configurations of a 148 ft. bridge over Cooper Creek (Thompson Falls, MT) and a 108 ft. bridge over Big Dry Creek (Jordan, MT). The prefabricated elements for these systems consist of a single steel truss girder supporting 10 ft. - 4 in. (Big Dry Creek) and 7 ft. (Cooper Creek) wide concrete decks cast at the steel fabrication facility. Member sizes for these preliminary designs are shown in Table 1.

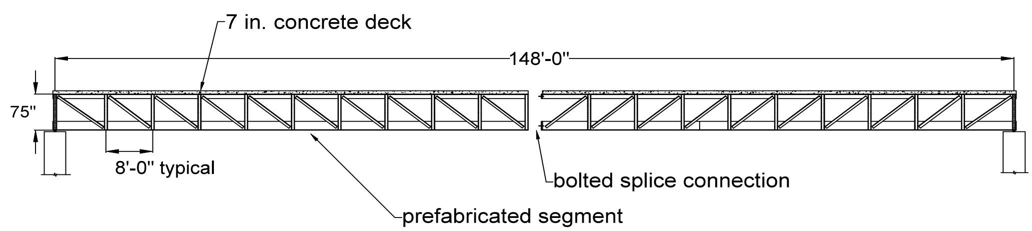
In all cases, the vertical and diagonal steel truss girder members are welded to the top and bottom chords of the steel truss girder. Two (or more) prefabricated elements are bolted together longitudinally to create the final bridge span. The longitudinal and transverse joints between the prefabricated elements are reinforced and filled with concrete to create continuity between the segments. A cross-section and elevation view of Option 1 in Table 1 is shown in Figure 1.

Table 1. Prototype Bridge Systems

Option	Span (ft.)	Deck Thickness (in.)	Top Chord Member	Bottom Chord Member	Vertical Member	Diagonal Member	Steel Weight (lbs.)
1	148	7	WT12x38	WT18x97 / WT20x147	HSS6x6 / HSS5x5	LL5x3 / LL6x3 / LL7x4	29,100
2	148	7	WT12x38	WT18x97 / WT20x147	W8x15-31	W6x16 / W8x21-28	28,000
3	108	8-1/4	PL3/4x12	PL1-3/4x12 / PL2x6	W8x18-24	PL1x6	10,080



(a) Cross-Section



(b) Elevation

Figure 1. Proposed (a) Cross-Section and (b) Elevation View of the Prefabricated Steel Truss Girder Bridge Option 1

Summary of Work

The literature review (Chapter 2) identified the current state-of-practice related to the analysis, design, and construction of similar bridge systems constructed on an accelerated schedule. The review focused on four primary topics pertinent to the proposed bridge system and this project: 1) modular systems, 2) concrete decks, 3) welded connections subjected to fatigue, 4) full-scale experimental studies, and 5) live load distribution factors.

The objectives of Chapter 3 were to 1) identify any impacts on the projected service life of the prototype steel truss girder bridge configurations based on fatigue of the welded member-to-member connections, 2) perform a cost analysis for the proposed systems and compare the results with the cost of plate girder alternatives, 3) as necessary and possible, suggest potential generic changes in member connection details to improve fatigue performance, and 4) for a specific 205 ft. span, identify a steel truss bridge configuration with the greatest potential for material and construction efficiencies. The 205 ft. span was selected so that these results could be readily compared with the Swan River plate girder project currently being designed by MDT.

In Chapter 4, a bolted/welded prefabricated steel truss girder bridge was investigated as an alternative to the welded steel truss girder bridge. Use of bolted connections at selected locations in the steel truss girders offers improved fatigue performance, allowing for lighter weight members, and making it a viable alternative for bridge replacement projects using either conventional or accelerated construction methods. The proposed system consists of bolted diagonal and welded vertical member connections to the top and bottom chords. Tasks included 1) developing a 3D finite element model used to more accurately calculate the distribution of lane and truck loads to the steel truss girder members, 2) investigating approximate distribution factors using an equivalent moment of inertia, and 3) determining member sizes and connection geometry to satisfy AASHTO Strength I, Fatigue I, and Service II load combinations for both conventional and accelerated construction methods.

The final designs of the two steel truss girder configurations developed in Chapter 4 were used in Chapter 5 to estimate potential cost savings related to materials, fabrication, and construction of these alternatives compared with the 205 ft. Swan River plate girders.

Summary and conclusions are presented in Chapter 6.

CHAPTER TWO

LITERATURE REVIEW

In reviewing prefabricated bridge systems with a view toward investigating their deployment, five subject areas of interest were identified and researched in the literature: 1) modular steel systems, 2) concrete decks, 3) welded connections subjected to fatigue, 4) full-scale experimental studies, and 5) live load distribution factors. Each topic, discussed in the following subsections, was selected for its impact on the analysis, design and construction of a prefabricated steel-truss bridge in Montana.

With these topics in mind, a thorough search was performed using four resource databases: Engineering Village, MDT Library, Transportation Research Board, and Google Scholar. The keyword "Prefabricated Bridges" was successfully combined with "Steel Truss," and "Deck Systems" to identify potential works of interest. The articles were reviewed and further organized into categories related to the components of the proposed modular steel system. This review and filtering process identified 37 sources (journal publications, trade journal articles, and state, federal, and private reports) as the most relevant to the proposed prefabricated steel truss girder bridge.

Modular Steel Systems

Prefabricated steel bridges have been constructed using a truss configuration, most notably in the Bailey Bridge and its successors. Other prefabricated steel systems include steel girders with composite concrete decks and composite space trusses.

Steel Trusses

One of the earliest forms of prefabricated bridges was the Bailey Bridge. Patented in 1943, the Bailey Bridge was designed by Sir Donald Bailey for use by the Allied Forces to build crossings during World War II (SDR Engineering Consultants 2005). A typical longitudinal section of a Bailey Bridge is shown in Figure 2. This section has a width of 10 ft. and a height of 4 ft. – 9 in. These sections, designed to fit in a standard military truck, are bolted together in the field at the top and bottom chords to form a through-truss bridge. Five different steel bridge configurations are available using Standard Bailey Bridge System components (Figure 3). Constructing the Bailey Bridge can be done using a crane to hoist the assembled configuration in place or launching the structure from one side of the gap to be bridged as shown in Figure 4. Portable Bailey panel bridges are currently available from Bailey Bridges, Inc.

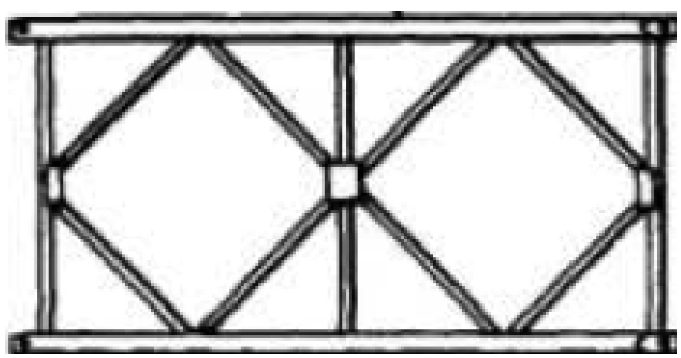


Figure 2. Detail of a Bailey Bridge Panel (Klaiber and Wipf 2004)

Since the expiration of the Bailey Bridge patent, Acrow Corporation of America and U.S. Bridge have developed modular bridge systems that are similar to the Bailey Bridge. These portable bridge configurations are often used for pedestrian bridges, although many state DOT's, including Montana, have used them as temporary structures during bridge construction or in the event of an emergency.

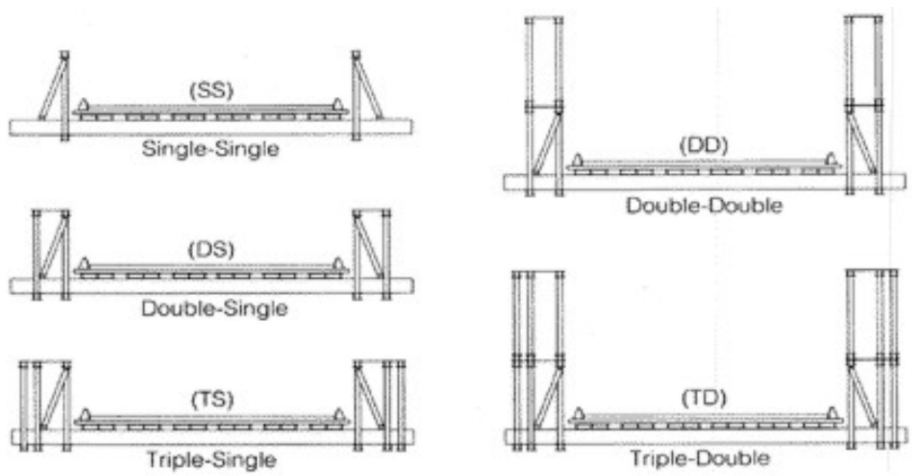


Figure 3. Bailey Configurations (SDR Engineering Consultants 2005)

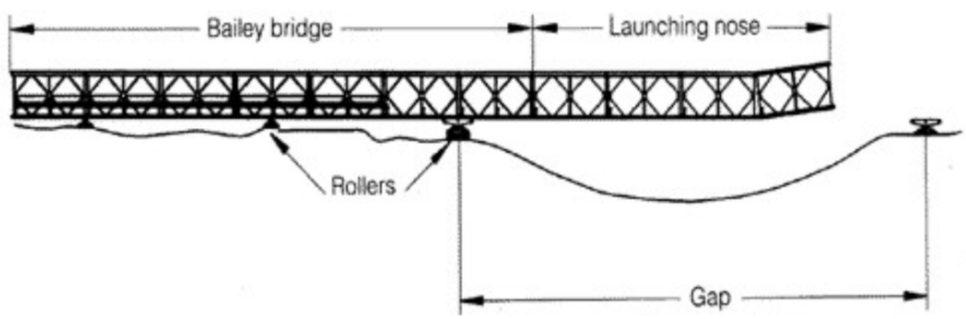


Figure 4. Bailey Bridge Launching Diagram (SDR Engineering Consultants 2005)

The Acrow Panel Bridge is made up of three different stock items that are assembled to form the desired configuration. A typical Acrow bridge is shown in Figure 5. The truss segments are 10 ft. wide, 7.2 ft. tall, and 6.5 in. wide. Spans of up to 230 ft. can be created by bolting the panels together and are capable of supporting three lanes of HS 25 load. Standard floor beams span between the trusses, and decking panels span longitudinally along the bridge length between the floor beams. Prefabricated steel orthotropic panels are the most common deck type, although steel grids and timber options can be incorporated (Klaiber and Wipf 2004).

Figure 5. Acrow Bridge assembled using Several Layers of Panels to Achieve the Span (Acrow Corporation of America 2015)

The Bailey Bridge System has been used in Montana for several temporary crossings where bridges were damaged, deteriorated, or collapsed. A search of Montana's Treasure State Endowment Program (TSEP) project applications and reports, the Department of Commerce project evaluations and funding recommendations, and the Department of Transportation bid packages revealed the following projects used prefabricated steel bridges (State of Montana 2016):

- A 100 ft. span, double-single M2 Bailey Bridge configuration was installed over the existing bridge structure crossing Box Elder Creek, near Hammond, MT. Bids were received in August 2009 to replace the temporary structure with a permanent one.
- Park County installed a temporary Bailey Bridge to replace the Ninth Street Bridge over the Yellowstone River in June 2008, in Livingston. The bridge was

installed over the existing structure and was posted with a speed limit of 5 mph and a maximum vehicle weight of 3 tons.

- A collapsed bridge over Fish Creek near Ryegate, in Golden Valley County, was replaced with a temporary Bailey Bridge. Bids were received in August 2014 to replace the temporary structure with 83 ft. pre-stressed bulb-tee beams.
- TSEP emergency funds were used to construct a temporary Bailey Bridge over a damaged bridge crossing Racetrack Creek in Powell County (pre-2005).
- Mineral County used a temporary Bailey Bridge over the 52 ft. damaged timber Cedar Creek Bridge (pre-2005).
- In December of 2002, Madison County installed a Bailey Bridge over the deteriorating Upper South Boulder Bridge to provide a temporary crossing until a permanent solution could be implemented.

The panel sizes, span lengths, and load capacities of the Bailey type bridges are consistent with the proposed systems considered in this investigation. Their long history demonstrates that modular prefabricated truss systems are an effective bridge construction strategy. That being said, these bridges are used in a through truss configuration, while the proposed systems use an underslung truss arrangement. The decks in these systems do not act compositely with the trusses, while composite action between the concrete decks and steel trusses in the proposed systems is expected to offer improved structural efficiency and stiffness.

U.S. Bridge, a descendent of the Ohio Bridge Corporation, offers prefabricated truss options that are designed for the Association of State Highway and Transportation

Officials (AASHTO) HS10, HS15, HS25, and HL93 loadings (U.S. Bridge 2015). Unlike the Bailey/Acrow Panel Bridge, where identical panel segments are bolted together in the field, the U.S. Bridge System uses longer, all-welded truss systems that can then be bolted together in the field. The trusses panels are prefabricated with standard W-sections, and the entire welded segments are then hot-dipped galvanized (Klaiber and Wipf 2004). The trusses are through-type with parallel top and bottom chords and are available in standard lengths of up to 150 ft. For longer spans, a camel back configuration is used and is shown in Figure 6. A common deck system includes underslung floor beams carrying simply supported stringers. Traditional concrete filled pans and timber decks can also be provided.

Figure 6. US Bridge Design, the “Viking Bridge” (U.S. Bridge 2015)

Completely prefabricating steel-truss bridge superstructures could potentially be a more cost- effective and permanent solution for counties that install temporary bridge structures. Albany County in New York State investigated this alternative to find cost-efficient bridge solutions in rural areas with lower traffic volumes (Heine 1990). The county replaced a 70 ft. truss bridge built in 1898 with Warren trusses and welded connections prefabricated by the Ohio Bridge Corporation. The estimated cost to install the bridge on the existing abutments was $50 per sq. ft. and included the cost of material, erection, and placement of a wooden deck. Bid prices were 5 to 6 times this amount for a standard replacement (Heine 1990).

A second example of a permanent welded prefabricated truss installation is the Crosier Bottom culvert in Meade County, Kentucky (McConahy 2004). The solution for the bridge replacement was a design-build process using 80 ft. prefabricated steel trusses (Figure 7). This alternative was substantially cheaper than a cast-in-place concrete bridge (McConahy 2004). The steel trusses were a U.S. Bridge product, and each truss was shipped in two 40-foot sections that were bolted together to form the final 80 ft. length and then lifted by crane onto the abutments. The bridge was finished with a cast-in-place concrete deck. The entire project, including a soil investigation, design, and construction was completed in 30 days. A detailed timeline of the construction was not provided. The Crosier Bottom bridge replacement highlights the benefits that prefabricated steel trusses can provide.

Rolled Wide-Flange Sections

Another type of prefabricated modular system consists of wide-flange beams topped with a composite concrete deck, as shown in Figure 8. One such system, originally patented under the name “Inverset,” is now marketed by Fort Miller Co., Inc. (Schuylerville, NY) as Prefabricated Bridge Units (PBU). The composite system is similar to the proposed prefabricated system of the current study; however, the assemblies consist of two wide-flange sections, rather than steel trusses, topped with a concrete deck. Common or typical segment sizes are not provided on Fort Miller Company's website.

Figure 7. Crosier Bottom Crossing (McConahy 2004)

The PBU/Inverset system uses an innovative fabrication method to obtain a more efficient composite cross-section. The segments are cast in an upside down orientation, as shown in Figure 9, in such a manner that upon subsequent erection, stresses in the composite elements are near zero in the bottom steel flange and are tensile in the top concrete flange (Klaiber and Wipf 2004). The result is a more efficient section for short to medium span bridges where stresses are dominated by live loading. The Fort Miller PBU's have been used for spans up to 126 ft. long with skews that exceed 45 degrees (Fort Miller Company 2016). The specific span and width of the prefabricated segments were not provided. Keys cast in the overhanging slabs are grouted together with nonshrink grout during construction. A similar joint system was investigated by Au et al. (2008) and is discussed in the following section of this report.

Figure 8. Prefabricated Wide-Flange Beams topped with a Composite Concrete Deck

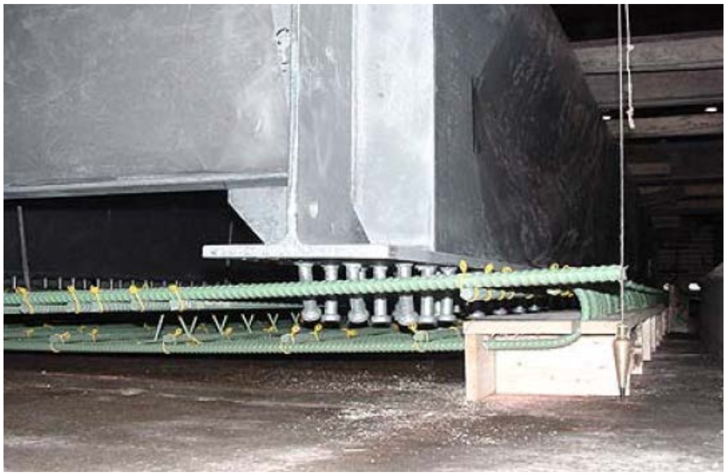


Figure 9. Prefabricated Bridge Units cast Upside-Down (Fort Miller Company 2016)

The New York State Department of Transportation used PBUs for the north and south bound bridges over the Mohawk River to minimize disruptions of the 110,000 vehicles that use these bridges each day. Two hundred and twenty-four prefabricated assemblies were used, including assemblies with monolithically cast traffic barriers, which is the same concept proposed for the system considered herein. High-performance concrete was used for the longitudinal and transverse joints between modular units. Installation of the prefabricated members and one of the joints is shown in Figure 10. More recent installations of Fort Miller PBU's are listed in Table 2.

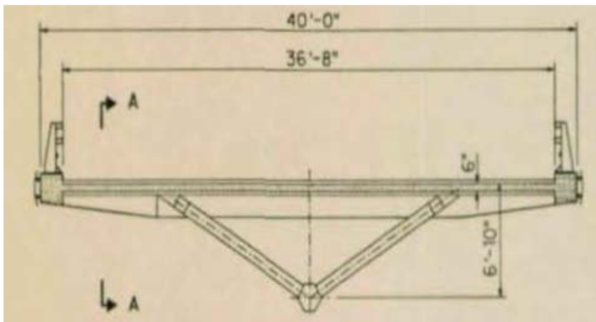
Table 2. Recent Bridge Installations using Fort Miller PBU's (Fort Miller Company 2016)

Project	Date	No. Longitudinal Segments	Length (ft.)
Garden State Parkway, NJ	April 2016	4	53
Route 28, MA	April 2016	4	90

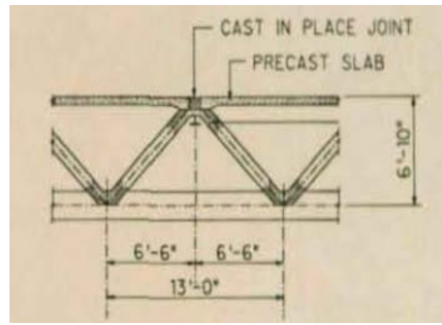
Figure 10. I-87 Prefabricated Bridge Unit Installation, I-87 Bridge Reconstruction (Fort Miller Company 2016)

Space Trusses

In an attempt to discover methods for reducing the weight of bridge superstructures for medium-span (50 to 150 ft.) bridges, the French Highway Administration invested nearly 10 years of research before selecting a steel space truss design for demonstration deployment over the Roize River (Montens and O'Hagan 1992). The Roize Bridge was completed in 1990 and was the first structure to combine an innovative steel space truss with pre-stressed concrete deck panels. Similar to the proposed prefabricated system, the Roize Bridge used modular building methods and composite action between the space truss and concrete deck, with the concrete deck effectively acting as the "top chord" of the truss system. The bridge consisted of three spans: two 118 ft. end sections and a 131 ft. long center span. A typical cross-section and elevation view are shown in Figure 11.



(a) Cross-Section



(b) Elevation

Figure 11. Roize (a) Cross-Section and (b) Elevation View (Muller 1993)

The bottom chord of the space truss is a hexagonal cross section made of two bent steel plates joined by a continuous longitudinal weld (Figure 12). Four diagonals are welded to stiffeners in the bottom chord, forming two inclined Warren-type trusses. The top of the diagonals is welded to I-shaped transverse floor beams spaced at 13 ft. These 13 ft. long tetrahedrons (four diagonals, one bottom chord, and one floor beam) were mass produced in a factory and assembled on-site. Rigid nodes were created along the bridge deck by extending the inclined truss members through the transverse floor beams and into the deck closure pour.

The precast concrete deck panels were 40 ft. wide and 12 ft. - 4 in. in length. The panels were pre-stressed with 54 - 0.5 in. bonded strands in the longitudinal direction and post-tensioned with two 4-strand tendons located on either side of the floor beams after the closure joints were cast. After the bridge deck was assembled and cast, the superstructure assembly was continuously post-tensioned with five external draped 12-strand tendons (Figure 12). The concrete was a high-strength silica-fume with specified compression strength of 11.5 ksi. The combination of high-strength concrete and draped longitudinal post-tensioning helped reduce the long-term creep effects due to flexural loads (Montens and O'Hagan 1992).

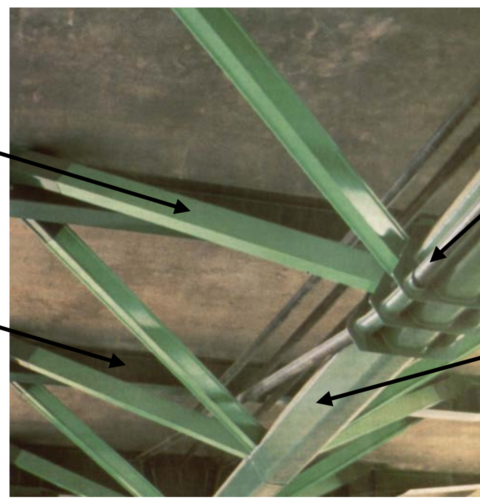
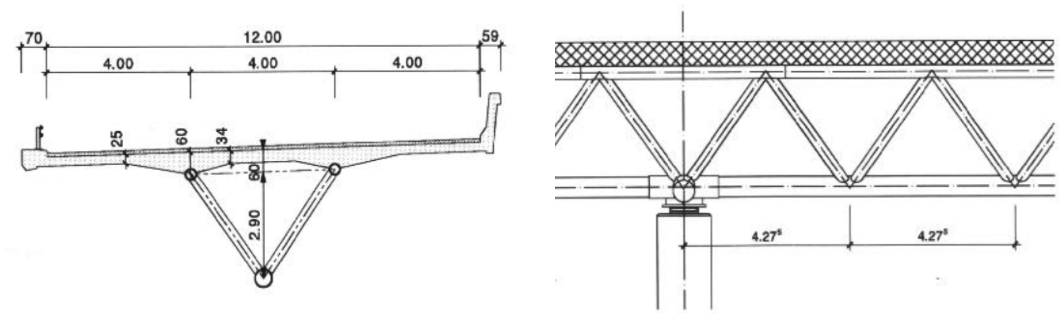


Figure 12. Space Truss Superstructure of the Roize Bridge (Muller 1993)

The Lully Viaduct in Switzerland is a similar composite, prefabricated space truss bridge that was selected over two pre-stressed concrete box girder alternatives for its aesthetic qualities (Dauner et al. 1998). A typical cross-section and elevation view of this bridge is shown in Figure 13. Average spans of the 1000 m bridge were 43 m, and the space truss depth was 2.9 m. Circular pipes were used for all truss members and resulted in complicated node geometry that created challenges with cutting and preparing the member ends for full penetration welds. Special equipment was used to cut the contact and welding surfaces. The prefabricated space trusses were erected in one-half span lengths (22 m). Longitudinal and transverse post-tensioning was used after curing of the cast-in-place concrete deck. The completed structure is shown in Figure 14.



(a) Cross-Section (b) Elevation

Figure 13. Lully Viaduct (a) Cross-Section and (b) Elevation View, SI Dimensions (Dauner et al. 1998)

Figure 14. Lully Viaduct Space Truss (Dauner et al. 1998)

<u>Modular System Comparison</u>

A detailed evaluation and assessment of six different modular bridge types was done by SDR Engineering Consultants (2005). Numerical ratings were assigned for each bridge in four categories of performance: aesthetics; design flexibility and service life; construction and erection; and future maintenance. The overall score was the summation of the ratings for each category and is shown in Table 3. On a scale of 0 – 100, scores ranged from a low value of 62 (temporary truss and permanent precast systems) to a high value of 87 (steel girders and concrete deck). The proposed prefabricated system being considered in this project has elements that are most similar to Bridge Type 3, composite space truss, and Bridge Type 4, steel girders and concrete deck, which ranked 1st and 3rd, respectively, for the bridge types considered by SDR. Unlike the proposed system where the bridge is supported by the bottom chord, the under-slung truss (Bridge Type 5) evaluated by SDR was supported by the top chord and was not as modular as the other bridge types considered.

The highest total score for the performance criteria shown in Table 3 was a bridge with steel girders with precast composite concrete decks (No. 4). For this reason, SDR investigated a new modular precast concrete system that is shown in Figure 15. To reduce live load deflections, SDR's concept could also include continuity reinforcement at interior supports, as shown in Figure 16.

SDR also commented that the use of modular precast concrete systems can be limited by transportation constraints. A general weight limit for traditional transportation is 200 kips, and panel widths wider than 8 ft. require special permitting (SDR Engineering Consultants 2005).

The third highest total score for the bridge types shown in Table 3 is a composite space truss. These systems have high strength and stiffness-to-weight ratios; however, their lack of standardized members and details leads to higher initial costs (SDR Engineering Consultants 2005). Despite their high ranking, this option was not selected for further study by SDR. The research team contacted several bridge manufacturers to determine if fabrication of a space truss with existing equipment and fabrication techniques could be accomplished. All fabricators interviewed expressed reservations on the practicality of such a system.

Table 3. Comparison of Modular Bridge Systems, adapted from SDR Engineering Consultants (2005)

No.	Bridge Type	Unit Configurations and Aesthetics (30)	Design Flexibility and 75-Year Service Life (25)	Construction and Erection (25)	Future Maintenance (20)	Total Score (100)
1	Temporary Truss and Permanent Precast System	21	15	18	8	62
2	Railroad Flatcar	24	18	24	14	80
3	Composite Space Truss	23	21	17	16	77
4	Steel Girders and Concrete Deck	26	22	23	16	87
5	Under-Slung Truss	17	19	21	12	70
6	Cold-Formed Steel Plate Box	23	16	22	11	72

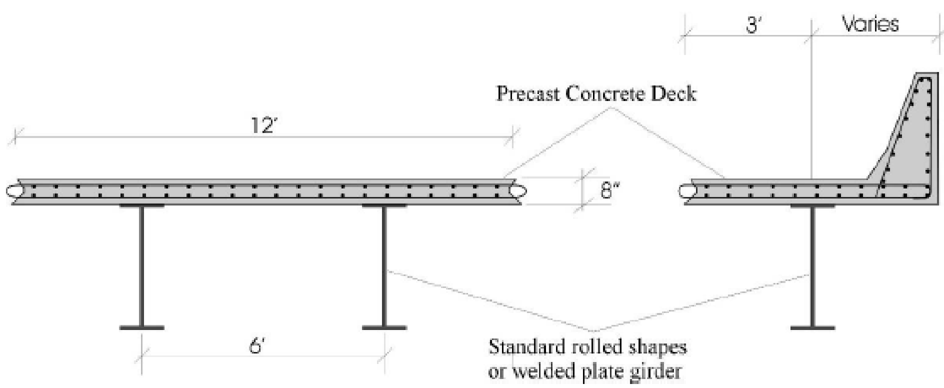


Figure 15. Modular Precast Concrete Bridge Concept (SDR Engineering Consultants 2005)

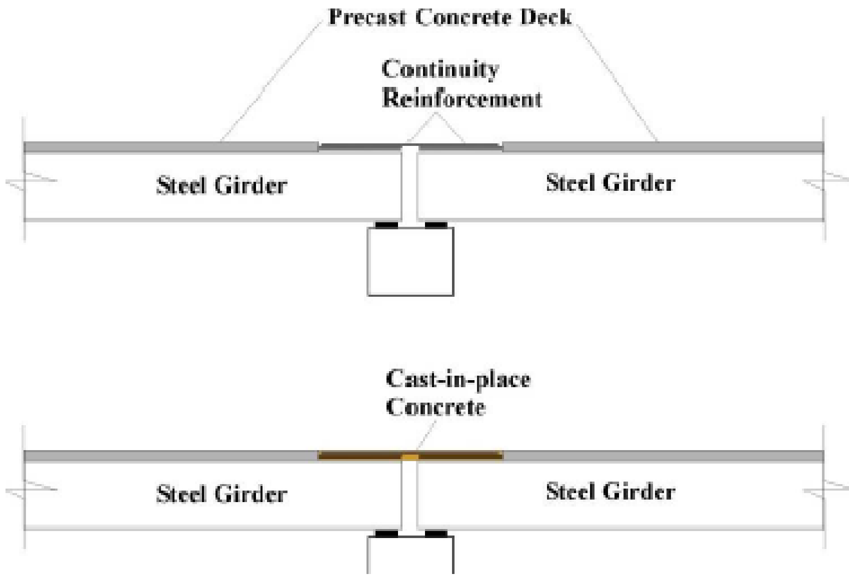


Figure 16. Continuous Precast Modular Bridge Concept (SDR Engineering Consultants 2005)

The predominant discouragement to the widespread, continued use of modular bridges in the United States, despite growing prevalence in Europe and Asia, is the fatigue-sensitive nature of some of the details (SDR Engineering Consultants 2005). In addition, more complete, modular bridge systems such as those by Bailey Bridges, U.S. Bridge, Acrow, and Fort Miller may not be cost-effective due to the proprietary nature of their designs.

Concrete Decks

Several different concrete deck systems have been investigated for use in accelerated bridge construction. The systems were designed with the intent of reducing the time needed to construct a deck while maintaining equal or better performance and durability than conventionally constructed decks. These systems include precast, cast-in-place, and post-tensioned concrete decks.

Precast Concrete

Advantages of precast concrete decks include quick installation and increased quality control with higher strength and performance concrete than typically is used in cast-in-place concrete decks. A concern with precast concrete decks is the durability and structural integrity of the joints between elements (Culmo 2011). The Ministry of Transportation in Ontario, Canada performed structural testing on reduced scale precast panel joints (Au et al. 2008) to investigate the performance of different joint configurations. The prefabricated bridge systems were selected to meet the requirements of one, two, or three-span bridges with spans ranging from 66 to 164 ft.

Two types of precast panel joints were investigated and are shown in Figure 17. System A consisted of a concrete deck precast on a single steel girder forming a T-shaped prefabricated member, similar to the proposed system. Closure strips for this deck system are located between the girder supports. As an alternative to offset the potentially heavy and difficult-to-transport prefabricated T-shaped members, System B consisted of separate precast concrete deck panels that were attached to the pre-stressed or steel girders after they were placed at the bridge site. The panel closure strips were located over the girder.

Due to practical limitations (size effects, design criteria, laboratory restrictions, and material availability), the bridge specimens were constructed with one-third scale dimensions in the vertical direction, one-seventh scale in the longitudinal direction, and one-quarter scale in the transverse direction. The authors performed an analysis of both the prototype and scaled bridge models and determined the behavior of the two systems were similar.

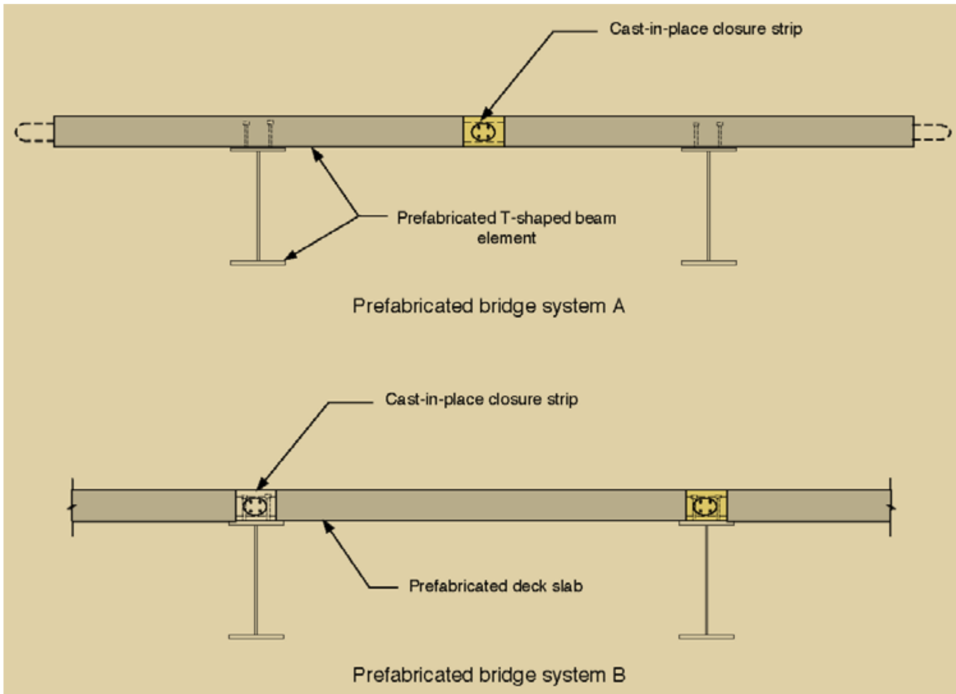


Figure 17. Typical Transverse Sections of Prefabricated Bridge System Specimens (Au et al. 2008)

Two different joint configurations were constructed for each system. Specimens 1 and 2 for System A used different arrangements of top and bottom reinforcement, which are shown in Figure 18. Specimens 3 and 4 for System B utilized L-shaped and U-shaped reinforcement within the closure strip over the steel girders, which also are shown Figure 18.

A total of 7 million load cycles were applied to Specimens 1 through 3. Specimen 4 was subjected to a total of 16 million load cycles. To investigate the condition of the specimens during the cyclic tests, a static load test was performed after every 1 million cycles of loading. After all cyclic load tests, punching load tests were performed to determine the post-elastic behavior of the specimens by applying a concentrated load over an area that represented a single wheel. Several loadings and unloading cycles were completed before the maximum failure load was reached.

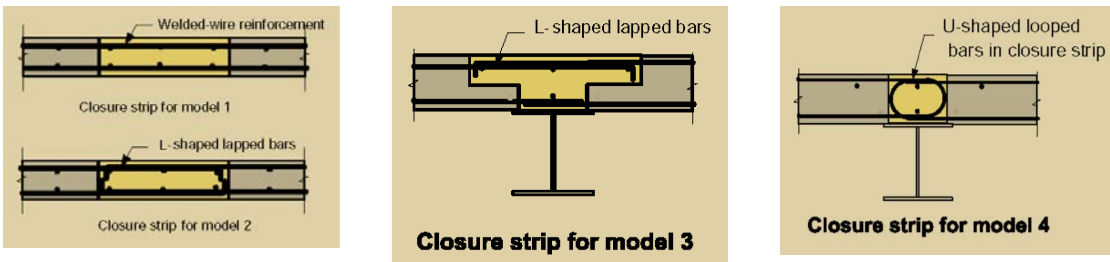


Figure 18. Closure Strip Details for Four Configurations Considered (Au et al. 2008)

The experimental program concluded that 1) long-term performance of the longitudinal joints was acceptable, 2) higher transverse deck stiffness was achieved when the longitudinal joints were located over the beams, and 3) the smooth bars used in the closure strip in Specimen 2 had a lower initial stiffness.

Successful or unsuccessful field deployments of this type of structural system were not found in the literature; however, a similar bridge system was recently constructed over Maxwell Coulee, 22 miles East of Jordan, MT. The bridge was 38 ft. – 4 in. wide by 100 ft. long and construction was completed in 2013. The bridge is currently being evaluated, and a final report on the bridge performance was due in 2017 (Montana Department of Transportation 2012).

Post-Tensioned Concrete

Transverse post-tensioning in concrete deck slabs is a common method for connecting precast concrete segments and could be used with the proposed bridge system. The tendons could be threaded through ducts in the prefabricated slab and grouted after post-tensioning. Research has shown that transverse post-tensioning improves the performance of the shear key joint and the durability of the bridge decks by reducing the number and width of cracks (Grace et al. 2012; Poston 1984). Satisfactory performance of transverse post-tensioned joints was observed in an experimental program conducted on a precast concrete deck panel system subjected to static and fatigue loading (Yamane et al. 1995). This deck system was designed and developed specifically for rapid construction and rehabilitation.

One of the challenges with post-tensioning deck panels assembled on site are construction tolerances. In a case study in Michigan (Attanayake et al. 2014), post-tensioning ducts were misaligned because the skew of the bridge was not correctly considered. When placing the precast panels on the pre-stressed bridge girders, some of the shear connector pockets did not provide enough tolerance for the twist (sweep) of the beams. This particular case study demonstrated the importance of providing adequate tolerances on precast members for efficient construction.

Cast-In-Place Concrete

Full-depth cast-in-place concrete decks are not a viable option for accelerated bridge construction due to the formwork and shoring required during construction. A partial-depth cast-in-place system that includes a precast or pre-manufactured form system could mitigate some of these construction issues and result in a cast-in-place top surface that minimizes joints on the surface of the deck. Such a concept was studied by SDR (2005), where a cold-formed steel plate is welded to steel girders to form a metal stay-in-place form as shown in Figure 19. The metal form acts as tension reinforcement for the composite system. A welded wire mesh-reinforcing cage is welded to the steel plate at the factory and acts as top reinforcement for the slab.

On-site, the form and reinforcement assemblies are bolted together in the longitudinal and transverse directions. A mat of steel mesh is then placed over the top of the joint to splice the reinforcement meshes together. This new concept was selected by SDR for further study because like the modular precast system described above, it also falls into the steel girder and concrete deck bridge type that had the highest total score in their evaluation and assessment (Bridge Type No. 4 in Table 3).

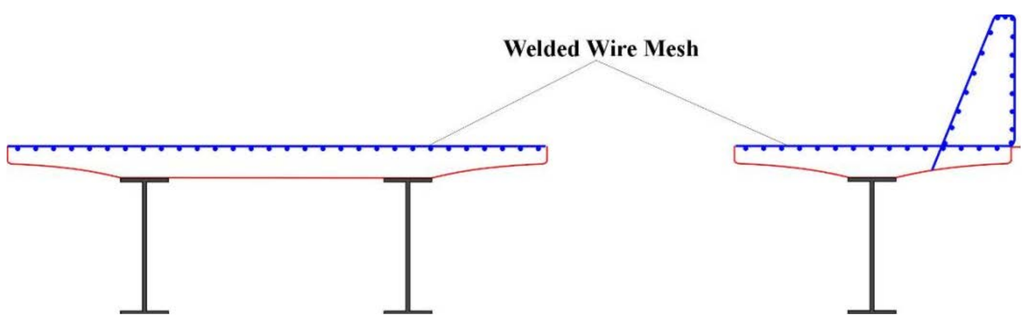


Figure 19. Proposed Cross-Section for a Cast-In-Place Concrete Deck without Formwork (SDR Engineering Consultants 2005)

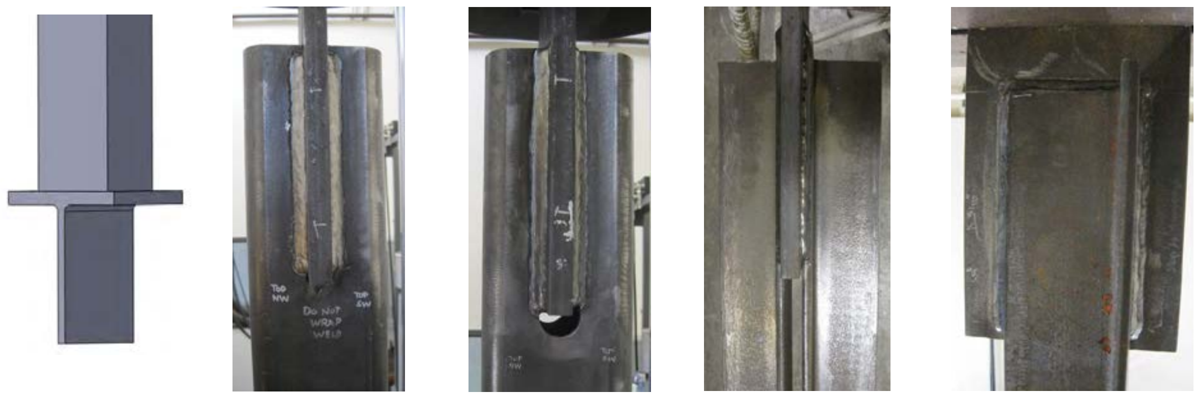
Welded Connections Subjected to Fatigue

Fatigue in steel and notably in welded steel connections is always a concern in cyclic loading environments, which is an obvious consideration with the composite steel truss/concrete deck modular system being studied in this project. The welded connection types included in the proposed prefabricated system are longitudinal welds in a knife-plate configuration and transverse welds made at the ends of the vertical and diagonal web members. The research summarized below identifies recent articles related to connection geometry and weld configuration that can be applied to the investigation of the proposed system.

Connection Geometry

Extensive testing was carried out at the University of Texas at Austin with regard to fatigue strength of welded connections used in steel bridges (Battistini et al. 2014). The experimental program investigated the fatigue performance of five cross-frame connection configurations by measuring stiffness, ultimate strength, and fatigue resistance. The project objectives were to determine the connection type that was most economical to fabricate and construct, while still providing adequate strength and stiffness for the connecting members.

The five connections tested (Figure 20) were the (a) T-stem, (b) knife plate without a stress relief hole, (c) knife plate with a stress relief hole, (d) double angle, and (e) single angle. A stress relief hole was included in three of the six knife plate specimens to mitigate stress concentrations at the forward edge of the fillet weld. The T-stem variations tested did not reach the minimum AASHTO fatigue requirement for Detail Category E and are not included in this review. In addition, because the back-to-back single-angle connection performance was similar to the double angle, the remainder of this section will focus on the two knife plate connections (b, c) and the double- angle connection (d) shown in Figure 20.



(a) T-Stem | (b) Knife Plate (KP) | (c) KP with Stress Relief | (d) Double-Angle | (e) Single-Angle

Figure 20. Connection Configurations Tested (Battistini et al. 2014)

Many of the results presented were related to the specific behavior of different brace configurations, such as X-, Z-, and K-frames. Improvements to fatigue behavior were observed in some of these frame configurations when thicker center gusset plates were used and when an additional transverse weld was included on the reverse side of the angle. The following specific conclusions were made related to the fatigue tests and welded connections:

- The T-stem connections (square, round, and diamond) had poor fatigue performance, likely due to a small local eccentricity that existed in the geometry.
- The knife plate connection performed adequately in fatigue, with 5 of the 6 specimens achieving E classification; the stress relief hole further increased the connection fatigue life.

- The double-angles achieved connection E classification. The fatigue cracking initiated in the angle when the member stress range was larger than the gusset plate stress range.
- The measured fatigue life of the connections tested in this study correlated well with the tabulated fatigue categories provided by AASHTO for common connection geometries.

Weld Configuration

The influence of weld geometry was investigated by McDonald and Frank (2009) to determine if balanced welds had an influence on the fatigue strength of single-angle connections. This study attempted to estimate fatigue performance based on the geometry and the angle of connection. The specimens consisted of single-angle members attached to a plate on each end as shown in Figure 21.

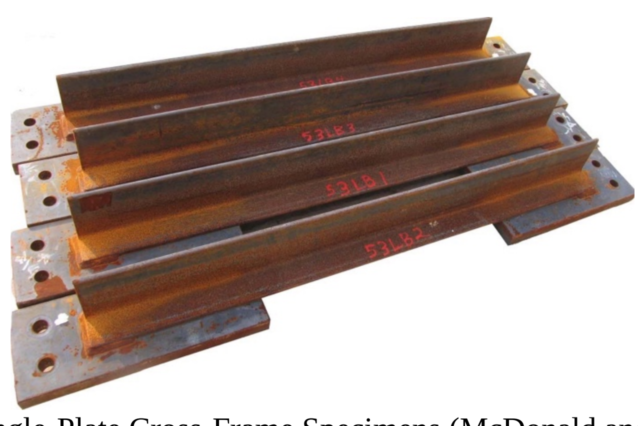


Figure 21. Angle-Plate Cross-Frame Specimens (McDonald and Frank 2009)

A total of 25 specimens and 6 weld configurations were tested, with a stress range from 8-12 ksi in fatigue by applying axial load to the two end plates. Both eccentric and balanced welds with short and long angle legs welded to the plate were included. The balanced welds were detailed to meet the requirements of AASHTO (2012). The study concluded that the balanced welds consistently performed better than specimens with equal length welds; however, due to the fact that angle and plate length varied, it was inconclusive as to whether the balancing of welds or frame geometry led to improved fatigue performance.

A parametric study using finite element analysis (FEA) was also performed by McDonald and Frank (2009) to investigate the factors affecting the stress concentrations in the steel plate connected to the single angles. The results of the parametric study suggested that the factor with the highest influence on the stress concentration was the length of the outstanding leg of the angle. Battistini et al. (2014) focused their parametric analysis on the axial stiffness reduction factor for a single angle cross frame. They concluded that the length of the diagonal member of a frame affects the stiffness as well, with a longer diagonal increasing the magnitude of the reduction factor.

Full-Scale Experimental Studies

Full-scale tests on bridge systems with elements similar to those being investigated here were identified in the literature and provide information relevant to the strength and analytical modeling aspects of steel trusses.

Research by King et al. (2013) included laboratory load tests on two full-scale, Bailey bridge segments. Two 10 ft. panel segments (Figure 22) were pin-connected to form 20 ft. spans for each specimen. A vertical load was applied through a thick plate on both sides of the top chord at the central nodes. The test specimen is shown in Figure 22.

Lateral buckling was observed in the top chord members adjacent to the central node at a load of 112 kips and 114 kips for the two specimens.

Figure 22. Full-Scale Bailey Bridge Model (King et al. 2013)

A comparison was made with the AASHTO specifications (2012) for members that failed by lateral buckling. The ratio of tested capacity (P_{test}) to the calculated nominal strength (P_n) ranged from 0.81 to 1.1 and showed that AASHTO generally recommends conservative design strengths for members in compression (King et al. 2013). The composite concrete deck will brace the top chord compression members for the proposed prefabricated truss; however, the conservative strength predictions by AASHTO are relevant to the diagonal members in compression.

Based on test results of the two specimens and isolated tests of the individual connections, elastic and nonlinear analyses were performed. From the elastic analysis, it was found that the effect of partial fixity of the connections was not significant due to the connections remaining elastic during the test. Results from the 2D nonlinear analysis compared well with the measured load displacement response, but the predicted capacity was higher because the analytical model could not capture the out-of-plane stability behavior that was observed in the test (King et al. 2013).

A second full-scale experimental investigation was performed on the Hillsville Truss bridge over the New River in Virginia (Hickey et al. 2009), shown in Figure 23. The objective of the study was to calibrate an analytical model that was used to estimate loads that could cause the bridge to collapse. This study was part of a larger endeavor to better understand the collapse of the I-35W bridge in Minneapolis, Minnesota by conducting field tests and detailed structural analysis on a similar bridge. The Hillsville Truss was similar to other mid-twentieth century steel truss bridges that used riveted gusset plate connections between members.

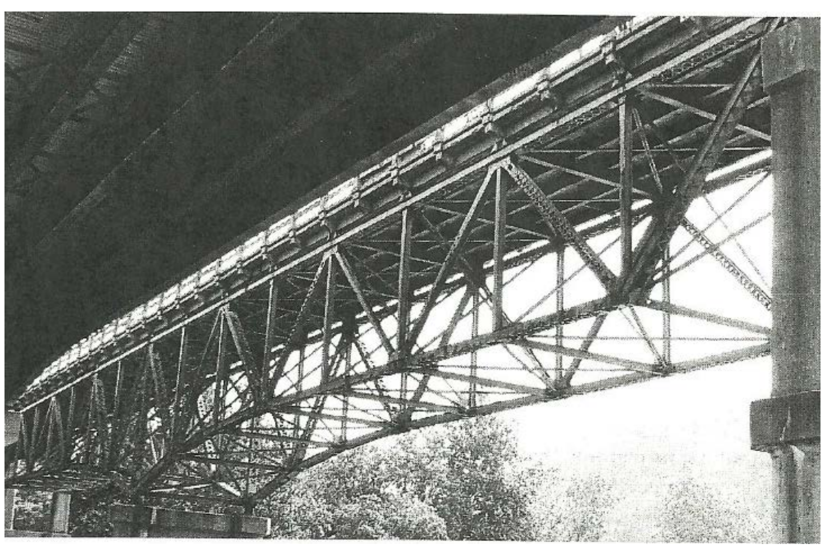


Figure 23. Hillsville Truss (Hickey et al. 2009)

Loaded trucks with known dimensions and weights were parked along the bridge, and strain gauges were strategically placed to record various member responses. The field test results were used to calibrate a 2-dimensional linear elastic steel truss bridge model, after which a failure analysis was conducted. The truss model with simple connections at the joints did not correlate with the data, so the model was updated to a frame model where bending moment could be considered. Adding the transverse floor beams and stringer elements to the frame model resulted in calculated results that most closely correlated with the collected data (Hickey et al. 2009). The authors concluded that the models provided evidence that moment was being transferred through the connections of the truss members, and therefore the connections should be evaluated to include flexural stresses.

An important observation from the analytical modeling of the Bailey Bridge segments and Hillsville Truss is that different conclusions were made related to the restraint provided by the connections. The welded connections for the Bailey Bridge did not provide significant restraint to member rotations and the results suggested the connections could be modeled as pinned. The pinned connections assumed for the riveted gusset plate connections in the Hillsville Truss however, did not compare well with the measured data, and additional connection restraint was necessary. These are important observations for the analytical modeling task of the current research project and were included in the analysis of the proposed prefabricated system.

Live Load Distribution Factor

The live load distribution factor equations in the AASHTO LRFD Bridge Design Specification (2014) were developed under National Cooperative Highway Research Program Project 12-26 (Zokaie et al. 1991). This project was initiated to improve the accuracy of the *S/D* formulas contained in the AASHTO specifications (Standard 1996). The girder spacing is *S*, and *D* is a constant based on the bridge type and the number of design lanes loaded. The *S/D* formulas generated valid results for bridges of typical geometry (i.e., girder spacing near 6 ft. and span length of about 60 ft.), but notably were less accurate for bridge spans shorter or longer than 60 ft. (Zokaie 2000). For these cases, span length and stiffness properties must be considered to gain better accuracy. This study led to the development of a set of formulas that provided better accuracy and included a broader range of bridges. These new equations first appeared in the 1998 LRFD Bridge Design specification.

Other LDF Formulas

Because of the complexity of current LDF equations, a new format for the equations has been proposed by Cai (2005), to help improve user understanding of effects different parameters have on load distribution. The parameters included in the proposed LDF's by Cai have practical and meaningful effect on load distribution while maintaining simplicity and intuition. The new LDF equation is represented by:

$$C_1 + \frac{S}{C_2} + C_3 R$$

The girder spacing is *S*, and *R* represents the ratio of longitudinal to transverse stiffness. The coefficient C_1 represents a nonzero LDF as the girder spacing decreases, C_2 reflects a linear relationship of the LDF versus girder spacing, and C_3 represents the effect of the relative stiffness in the longitudinal and transverse directions. The maximum difference in LDF's between the current AASHTO equations and the Cai equations for the bridge configurations was 7%, with an average difference of 1% (Cai 2005).

Another method to estimate LDF's was proposed by Huo et al. (2004) that established modification factors that were applied to a simplified method developed by the Tennessee Department of Transportation (1996). This procedure, known as Henry's method, begins by dividing the total bridge width by 12 ft. to determine the fractional number of lanes. Linear interpolation is used to determine multi-presence factors between 1.00, 0.90, and 0.75 for two, three, or greater than four traffic lanes, respectively. The total number of lanes is then divided by the total number of beams. The modification factor proposed by Huo et al. (2004) for Henry's method is 1.0 for steel beams and was based on the evaluation of LDF's for 24 bridges with six different types of superstructures using the current AASHTO equations and finite element analyses. The study showed that the simplified or modified Henry's method provides reasonable and reliable LDF's.

The range of bridge configurations considered for the development of the three methods and for which the LDF expressions can be considered applicable is shown in Table 4.

Table 4. Bridge Configurations Considered

LDF Equation	Girder Spacing, S (ft.)	Concrete Deck Thickness, t_s (in.)	Girder Span, L (ft.)	Number of Girders, N_b
AASHTO	$3.5 \leq S \leq 16.0$	$4.5 \leq t_s \leq 12.0$	$20 \leq L \leq 240$	$N_b \geq 4$
Cai	$5.0 \leq S \leq 10.0$	$7.0 \leq t_s \leq 7.5$	$41 \leq L \leq 125$	$5 \leq N_b \leq 10$
Huo et al.	$9.0 \leq S \leq 11.5$	$8.0 \leq t_s \leq 9.0$	$124 \leq L \leq 182$	$3 \leq N_b \leq 9$

Finite Element Analysis (FEA)

Finite element modeling allows engineers to generate complete, full-scale bridge models that can be used to evaluate the accuracy of the current LDF equations. In the finite element model, the effect of the live loads on the girders is determined by applying the loads in the model to the concrete deck, and calculating the support forces subsequently generated in the girders. These support forces can be compared to the applied loads to determine the load distribution factors. Research conducted by Yousif and Hindi (2007) further investigated four different finite element modeling techniques using SAP2000 in order to select the most accurate and practical method to determine load distribution.

Model A idealizes the bridge superstructure as a three-dimensional system where the main girders and the end diaphragm beams are modeled as two-node space elements with 6 degrees-of-freedom (DOF) at each node. Model A is shown in Figure 24. The bridge deck was modeled as a four-node quadrilateral shell element with 6 DOF at each node with the center of gravity of the slab coincident with the center of gravity of the girder. Bridge supports consist of hinges at one end and rollers at the other.

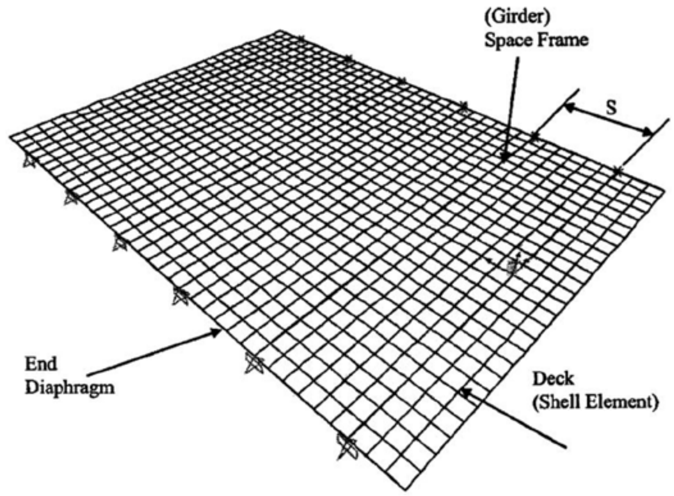


Figure 24. Finite Element Model A Technique (Yousif and Hindi 2007)

Model B idealizes the bridge superstructure as a three-dimensional system. The bridge deck is modeled as quadrilateral shell elements with 6 DOF at each node and the main girders and the end diaphragm beams are modeled as space frames with 6 DOF at each node. Model B is shown in Figure 25. The girders are eccentrically connected to the bridge deck with rigid links to account for the differences in their centers of gravity. The same support conditions from Model A were used.

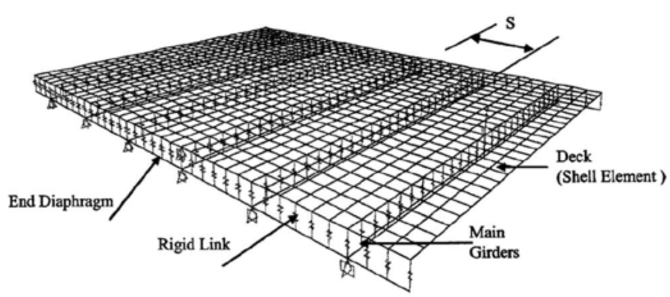


Figure 25. Finite Element Model B Technique (Yousif and Hindi 2007)

Model C idealizes the bridge superstructure as a 3D system where the bridge deck, main girders, and diaphragms are modeled as four-node quadrilateral shell elements with 6 DOF at each node. The girder flanges are modeled as two-dimensional shell elements with thick plates instead of space frame elements. The components are connected together using rigid links to allow for full composite action. This type of modeling is very time consuming in order to calculate the total forces at each girder and is not compatible with a moving load analysis using SAP2000.

Model D idealizes the bridge superstructure as a 3D system where the bridge deck is modeled as 3D solid elements and the girders are modeled as shell elements. This method is very similar to Model C, and therefore a moving load analysis is not possible. In addition, it requires more computation time and larger computer memory than Model C, even for simple bridge configurations.

Yousif and Hindi (2007) concluded that Model A provided the most accurate results. A comparison of the four models by Hayes et al. (1986) and Mabsout et al. (1997) indicated that the load distribution for Model A was better than Models B,C, and D. Yousif and Hindi (2007) performed additional checks to validate Model A using verification procedures reported by Hays et al. (1995), which was also verified by Chen and Aswad (1996). This validation showed good agreement between the three studies providing a high level of confidence for selecting Model A. Using Model A, results of the study by Yousif and Hindi (2007) indicated that the LDF equations overestimated the live load distribution by a maximum of 55% when compared to FEA for a significant number of cases. LDF equations gave very comparable results to the FEA models for bridges with parameters intermediate within the ranges considered and tended to deviate for bridges with parameters toward the limits considered. This study suggested that the LDF equation limitations need to be revised to reduce the deviation from the FEA, as in the case of large deviations costs could be unnecessarily high in some cases, and safety could be jeopardized in other cases.

Full-Scale Field-Testing

Field-testing is an increasingly important activity in bridge engineering, because it can reveal the hidden strength reserve of bridge members and verify the adequacy of a bridge even if it is deteriorating. Actual load-carrying capacity is often much higher than what can be determined by analysis due to more favorable loading sharing, effect of nonstructural components, actual support conditions, and other difficult to quantify factors. Eom and Nowak (2001) developed a field testing program that was carried out on about 20 existing bridges to determine the accuracy of the AASHTO LDF equations based on the behavior of actual bridges.

Each bridge had strain gauges attached to the lower and/or upper surfaces of the bottom flange of the steel girders with all girders instrumented at mid-span. Some of the bridges had strain gauges installed near supports to measure the moment restraint provided by the supports. Measurements were taken under passages of one and two vehicles, each being a Michigan three-unit, 11-axle truck with known weight and configuration. Superposition of strain data for a single truck in one lane and a single truck in the other lane were compared with the results obtained for two trucks side-by-side to verify a linear-elastic behavior of the bridge. The test trucks were driven at a crawling speed to simulate static loads and at regular speed to observe dynamic effects on the bridge. The LDF was equal to the ratio of the static strain in the girder and the sum of all the static strains in the other girders.

Results indicated that measured strain is lower than what is predicted by analysis due to the partial fixity of the supports. Measured LDF's are consistently lower than those of the AASHTO codes-specified values, and this difference indicates that the actual bridge performance can be different from what is assumed in the analysis procedure. When evaluating existing bridges, use of the code-specified LDF equations without considering the effect of possible partial support fixity can be too conservative.

Summary

The proposed prototype bridge structure consists of a prefabricated welded steel truss girder with a composite concrete deck, cast-in-place at the fabrication facility. These modular elements are then transported to the site, where they are lifted onto the foundation. This specific bridge and prefabricated construction technique is not well represented in the literature, and thus there is a need to identify potential bridge spans and traffic volumes where the proposed system is viable and economical. The most applicable information obtained from the literature review for this project is summarized below.

- The most common application for modular prefabricated steel truss systems has been for temporary bridge crossings. Two cases of permanent welded truss bridge replacement projects (Heine 1990; McConahy 2004) were identified in the literature for short spans with low-volume traffic. For these projects, these systems were significantly more economical than traditional solutions.

- Several investigations have been performed on details of longitudinal and transverse joints between prefabricated elements. This research has resulted in recommendations on joint configurations by the American Concrete Institute (Austin et al. 2001) and AASHTO (Culmo 2009).
- Measured fatigue stresses for a connection configuration similar to one of the proposed welded connections by Allied steel were consistent with the AASHTO (2012) Fatigue Detail Category E (Battistini et al. 2014).
- Full-scale experimental investigations of two steel truss bridges resulted in different conclusions related to the degree of rotational restraint provided by the truss connections. In one study, partial fixity of the connections was not significant (King et al. 2013). A study by Hickey et al. (2009), found that modeling the restraint at the connections was necessary to match the measured stresses in the full-scale bridges.
- Current AASHTO LDF equations are thought of as to complex so simpler equations would be welcomed by the design community.
- Finite element modeling is a key tool for structural engineers to test the accuracy of current LDF equations with 3D models of the bridge under consideration
- Field-testing allows engineers to get a better understanding of how the loads are shared between the bridge girders and the influence the actual support conditions plays in the load distribution

With these observations in mind, the service life, fatigue strength, and joint restraint of the proposed welded steel truss girders were included in the following analysis of a 148 ft. span steel truss girder.

CHAPTER THREE

ANALYSIS OF A 148 FT. SPAN STEEL TRUSS GIRDER WITH WELDED CONNECTIONS

An analysis was performed of a 148 ft. span steel truss girder with welded connections to 1) identify any impacts on the projected service life of the prototype steel truss girder bridge configurations based on fatigue of the welded member-to-member connections, 2) perform a cost analysis for the proposed systems and compare the results with the cost of a plate girder alternative, 3) as necessary and possible, suggest potential generic changes in member connection details to improve fatigue performance, and 4) investigate additional bridge spans to evaluate the change in steel weight of a steel truss girder compared to a steel plate girder.

Projected Fatigue Impacts of the Welded-to-Welded Member Connections

Of the three proposed bridge options shown in Table 1, the longer spans of Options 1 and 2 were identified by MDT to be more representative bridge span for which the proposed system would be used in Montana. For this reason, Option 1 shown in Figure 1 was selected to make a preliminary assessment of the load-induced fatigue stresses on the welded connections. Steps involved in executing this assessment consisted of developing a 2D finite element model of a typical subsection of the bridge system, determining appropriate factors to distribute applied loads to this subsection of the system, identifying fatigue life stress thresholds, and comparing predicted stress levels at various locations in the system as determined from the 2D finite element model with these fatigue life stress thresholds.

2D Finite Element Model

The two-dimensional model shown in Figure 26 was created using the program SAP2000, a finite element program by Computers and Structures, Inc. The restraints at the ends of the diagonal and vertical truss members were released to create pinned connections as permitted by AASHTO section 4.6.2.4. The top and bottom chords were modeled as both pinned and fixed connections to evaluate the effects of the continuous members per AASHTO section 4.6.3.5. A comparison between the two conditions resulted in member forces that were within 5%. Pinned connections were subsequently used for the bottom chord. A continuous member was used for the top chord because the member is fabricated as continuous, and loads are applied from the concrete deck slab between panel points. To simplify modeling and to generate composite action, the 7 ft. wide concrete deck and top chord elements were coincidently located at the composite neutral axis, similar to the procedure used by Yousif and Hindi (2007). Calculated self-weight deflections from this model were 2.5 in. (L/710) and were in reasonable agreement (~10%) with approximate hand calculations and the estimated dead load deflections of 2.7 in. (L/660) shown on the Allied Steel drawings.

The diagonal and bottom chord tension members that were the focus of this preliminary analysis are labeled in Figure 26. The AASHTO Fatigue I load combination considered with the un-factored permanent loads did not produce stress reversals in the vertical compression members, and therefore design for fatigue and fracture was not required for these members (AASHTO Section 6.6.1.2.1).

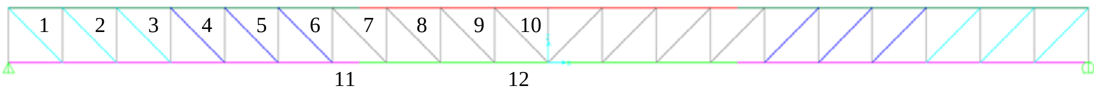


Figure 26. SAP2000 Model with Diagonal and Bottom Chord Tension Member Labels

Distribution Factors

The lever rule was used to distribute the axle and lane loads in the transverse direction. The joints connecting the pre-fabricated segments were assumed to create a continuous member spanning between the steel truss girders. The loading diagrams used for an interior steel truss girder are shown in Figure 27 and Figure 28. Two loaded lanes were considered with the Strength I load combination and resulted in a distribution factor of 0.79. The distribution factor calculated with fatigue load combinations using a single loaded lane is 0.57.

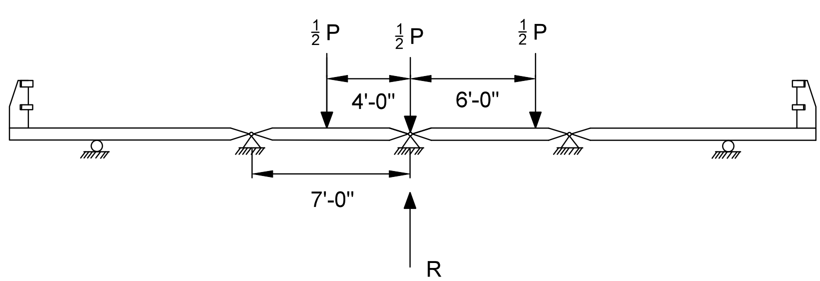


Figure 27. AASHTO Lever Rule Loading Diagram for Strength I Load Combination with Two Lanes Loaded

Fatigue Thresholds

In fatigue analysis, the threshold stress a member can experience is significantly affected by the fatigue susceptibility of the basic connection configuration, and the number of load cycles it will experience over its design life. A typical welded connection detail in the proposed steel truss girder is shown in Figure 29. Illustrative examples of the relevant detail categories for these members from AASHTO Table 6.6.1.2.3 are shown in Figure 30a for the diagonal members and Figure 30b for the bottom chord member. Relative to fatigue susceptibility, the welded connection between the diagonal members and top and bottom chords falls in AASHTO (2014) Detail Category E'; the connection between the vertical members and bottom chord is in AASHTO (2014) Detail Category C'.

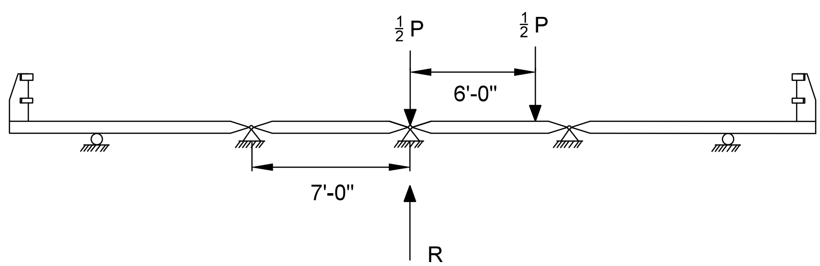


Figure 28. AASHTO Lever Rule Loading Diagram for Fatigue Load Combination with One Lane Loaded

Both the diagonal and bottom chord tension members were evaluated for fracture and fatigue limit states at the largest tension load occurring in the diagonal member at the end panel point. Relative to associated fatigue environment and attendant design life, one situation of interest is to keep stresses below the threshold for an infinite-life design addressed by the Fatigue I load combinations. The stress threshold for an infinite-life design is 2.6 ksi for Detail Category E' and 12.0 ksi for Detail Category C' (AASHTO Table 6.6.1.2.3-1).

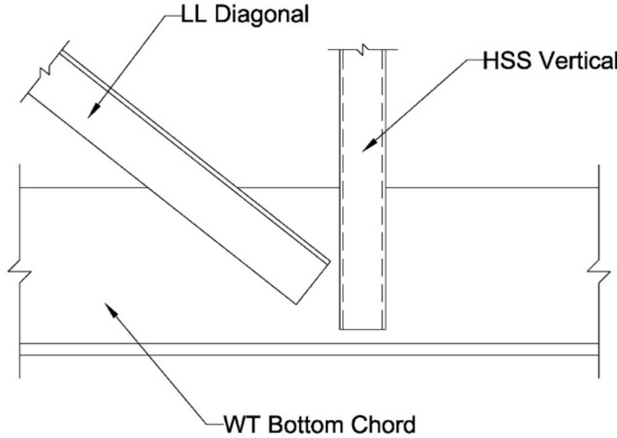
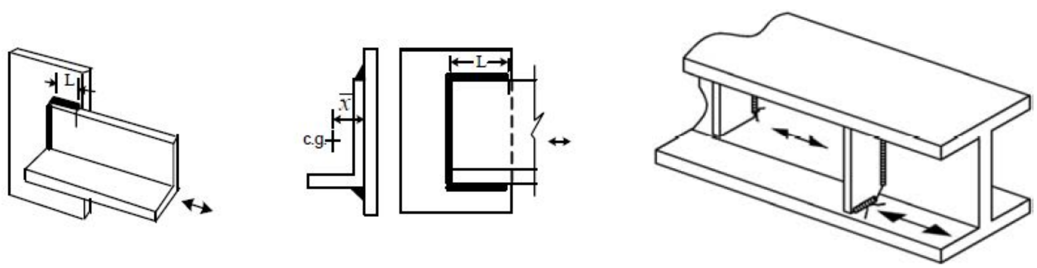


Figure 29. Proposed Connection Detail



(a) Detail Category E’ (b) Detail Category C’

Figure 30. Connection Examples of Detail Category E’ and C’ for Welded Attachments (AASHTO, 2014 Table 6.6.1.2.3-1 Description 7.2 and 4.1)

A second situation of interest is a finite-life design of 75-years, which is addressed by the Fatigue II load combination. The associated fatigue stress threshold is dependent on the expected number of fatigue cycles across a 75-year design life, as reflected by the projected single-lane average daily truck traffic (AADT). Thus, to determine this stress threshold, some level of assumed traffic is necessary. In this case, the fatigue demands on a bridge over Maxwell Coulee on Highway 200 by Jordan, MT were considered. This bridge is a prefabricated structure installed in 2013 by MDT and is representative of at least one situation in which the proposed steel truss girder/composite deck system would be used.

Current traffic data was obtained from MDT’s website for three different bridge crossings on Hwy 200 east of Jordan, MT. The AADT for each bridge was approximately 500 vehicles in 2014. Assuming a value for the traffic growth factor of two (which corresponds to a growth rate of 1 percent per year), an expected average AADT of 1,000 over a 75-year design life was determined. An estimated AADT value was obtained by assuming 15% of average daily traffic (ADT) were trucks (AASHTO Table C3.6.1.4.2-1). Based on the above assumptions, a 75-year design life threshold fatigue stress of 4.6 ksi for Detail Category E’ and 10.2 for Detail Category C’ was determined (AASHTO Section 3.6.1.4).

Calculated Stresses vs. Stress Thresholds

Three AASHTO load combinations were used in the preliminary analysis of the proposed prefabricated bridge. The impact, distribution, and multiple presence factors applied to the design truck and tandem loads with AASHTO’s Strength I, Fatigue I and Fatigue II combinations are summarized in Table 5.

The Strength I load combination results for the diagonal members are shown in Figure 31. Member labels on the x-axis of this figure correspond with the member numbers shown in Figure 26 above. The preliminary analysis suggests that 8 of the 12 diagonals and both bottom chord members proportioned by Allied Steel satisfy tension yielding of the gross section. Diagonal members 1, 4, 5, and 6 may require slightly larger cross-sections.

Table 5. Factors Applied in Analytical Model

Load Combination	Load	Impact Factor	Multiple Presence Factor, *m*	Distribution Factor	Load Factor
Strength I	Dead Load	NA	NA	NA	1.25
	Design Lane Load	NA	1.00	0.79	1.75
	Design Truck	1.33	1.00	0.79	1.75
	Design Tandem	1.33	1.00	0.79	1.75
Fatigue I	Design Truck	1.15	NA	0.57	1.50
Fatigue II	Design Truck	1.15	NA	0.57	0.75

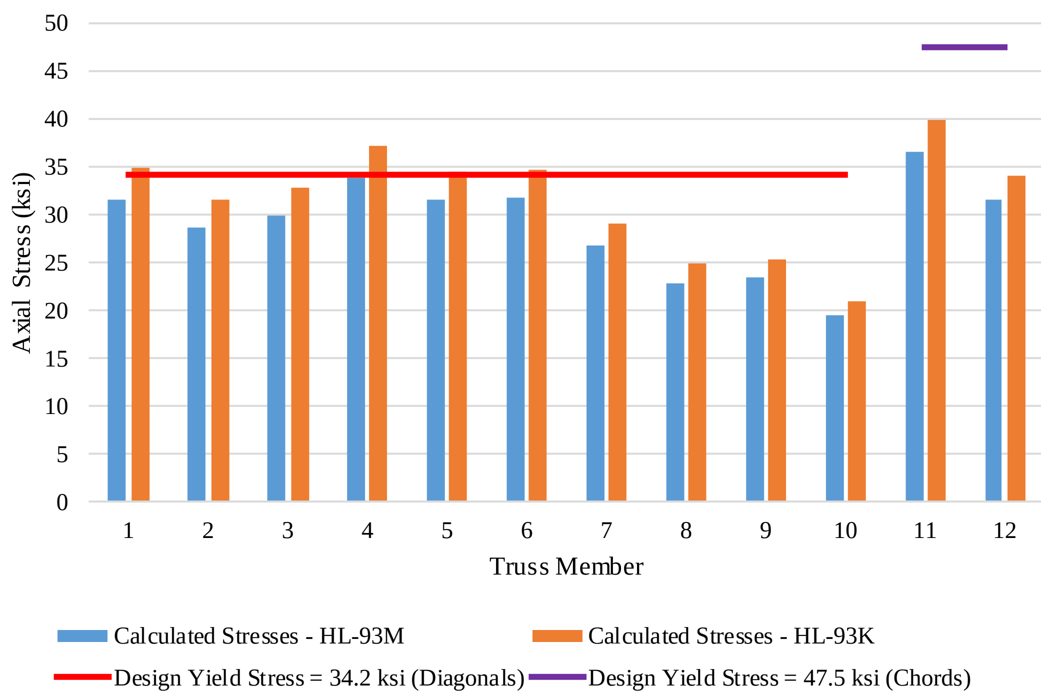


Figure 31. Axial Stress in the Diagonal and Bottom Chord Members with the Welded Connection for the Strength I Load Combination

Calculated axial loads from the Strength I load combination were used to estimate required weld lengths to include the effect of connection geometry on load-induced fatigue stresses. Calculated stresses in the diagonal and bottom chord members, shown in Figure 32, ranged from 9.0 to 12.0 ksi for the Fatigue I load combination. The fatigue thresholds for Fatigue Categories E’ and C’ are also shown in Figure 32 and indicate that the diagonal members are inadequate for an infinite-life design under the Fatigue I load combination.

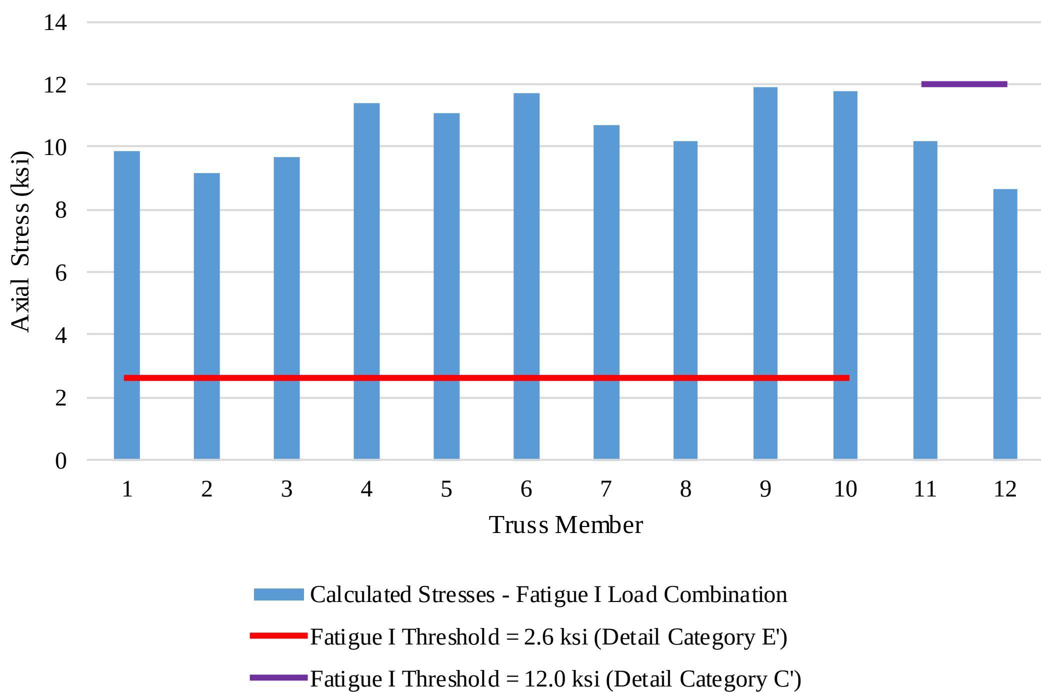


Figure 32. Axial Stress in the Diagonal and Bottom Chord Members with the Welded Connection for the Fatigue I Load Combination

Calculated stresses in the diagonal and bottom chord members, shown in Figure 33, ranged from 4.0 to 6.0 ksi for the Fatigue II load combination. While these stress levels are closer to the threshold values for this case, the thresholds are still exceeded in many of the members.

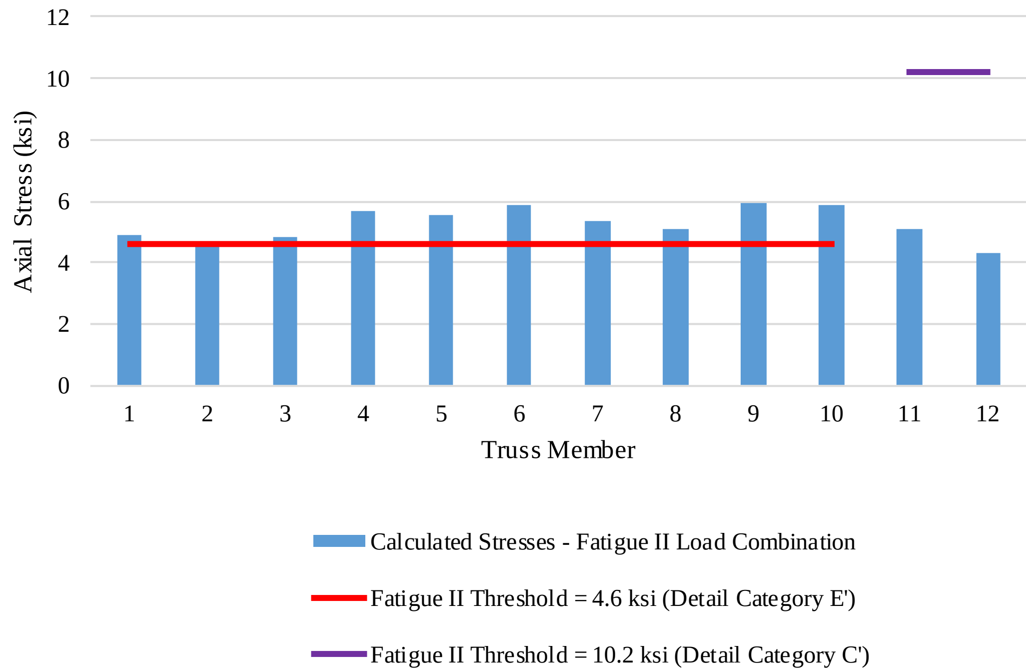


Figure 33. Axial Stress in the Diagonal and Bottom Chord Members with the Welded Connection for the Fatigue II Load Combination

Material and Fabrication Costs

Before further pursuing the prefabricated welded steel truss girder options, the cost of materials and fabrication were investigated, to determine if these options indeed offered some degree of economic advantage over alternative systems, as was generally expected. The steel truss girder configurations shown in Table 1 specifically were considered, notably in comparison with material and fabrication costs for equivalent steel plate girder systems. A preliminary design was completed for a 148 ft. plate girder with transverse stiffeners using the same span and depth of Options 1 and 2 (Table 1). An elevation view of the plate girder is shown in Figure 34.

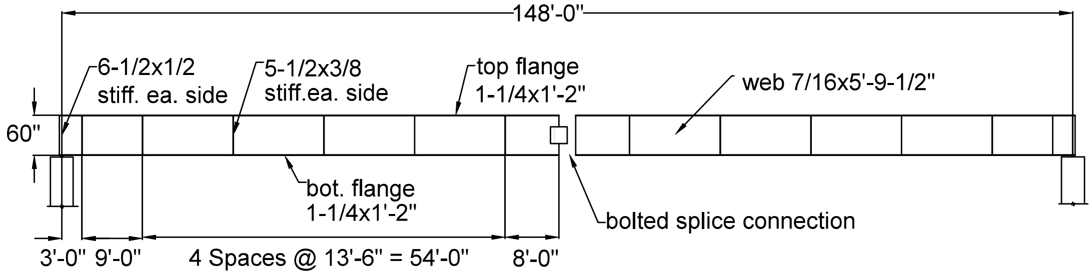


Figure 34. Elevation View of Plate Girder

Fabrication and cost information was obtained from AVEVA (Denver, CO), a supplier of software solutions and services to the steel fabrication industry, RTI Fabrication (Plains, MT) and Allied Steel Co. (Lewistown, MT). Note that based on conversations with all three companies, Option 3 (Table 1) was identified as non-viable due to the difficulty of fabricating the connections between the web and chord members of this configuration. The cost of cutting and beveling the vertical and diagonal members to make partial penetration welds to the top and bottom chord plates would be significantly more expensive than the fillet weld member connections used in the other two steel truss girder configurations. For this reason, Option 3 is not included in the cost comparison described below.

AVEVA

AVEVA provided the most detailed cost estimate for the two steel truss girders and steel plate girder options. Their cost-estimating software includes separate approximations for materials, labor, and fabricator profit to obtain the total cost. The cost estimates for the steel truss girder and steel plate girder options provided by AVEVA are summarized in Table 6.

Table 6. AVEVA Price Estimates

Bridge Option	Weight (lbs.)	Material Cost ($)	Labor Cost ($)	Fabricator Profit ($)	Total Price ($)
Option 1	29,100	34,940	5,020	5,900	45,950
Option 2	28,800	36,640	3,940	6,390	43,210
Plate Girder	36,560	35,720	6,120	6,280	48,120

RTI Fabrication

RTI Fabrication (Plains, MT) provided a cost estimate based on the total weight of steel used for each alternative. Their estimated price range was $1.30/lb. to $1.50/lb. for the total cost of material and fabrication. An average of $1.40/lb. was used to determine the cost estimates shown in Table 7.

Table 7. RTI Fabrication Price Estimates

Bridge Option	Total Weight (lbs.)	Total Price ($)
Option 1	29,100	40,740
Option 2	28,800	40,320
Plate Girder	36,560	51,190

Allied Steel

Allied Steel did not offer a price for each steel truss girder but instead estimated a savings of approximately 15% for the two steel truss girder options compared with the steel plate girder cost, based simply on the total weight of steel in each alternative.

Cost Estimate Summary

To compare the costs from the three sources described above, a plate girder price is needed to calculate Allied Steel's 15% savings estimate. This was accomplished by using the average cost of the plate girder prices provided by AVEVA and RTI

Fabrication and reducing it by 15%. A summary of the cost estimates is presented in Table 8.

Table 8. Steel Price Estimates

Company	Option 1 ($)	Option 2 ($)	Plate Girder ($)	% Difference (minimum)
AVEVA	45,950	43,210	48,120	5
RTI Fabrication	40,740	40,320	51,190	20
Allied Steel	42,210	42,210	49,660	15

The costs shown in Table 8 suggest the savings of 5% to 20% would be realized by using the two steel truss girders compared to a steel plate girder. In reaching this conclusion, it is important to recognize the various features and assumptions inherent in these cost estimates shown in Table 8. For example, specific fabrication procedures for RTI Fabrication and Allied Steel may be included in their estimates, but only approximated by costs provided by AVEVA. In addition, different shops may specialize in certain types of fabrication and these efficiencies may not be accurately included in the estimates above.

Alternative Steel Truss Girder Configurations

Based on further discussion with Allied Steel and AVEVA and the desire to improve the fatigue performance of the proposed steel truss girder system, revisions were considered for selected members and their connections. Allied Steel suggested that a steel truss girder utilizing double-angle diagonal members and wide-flange vertical members could be more economical. In addition, a bolted connection between the diagonal member and top and bottom chord would improve the fatigue performance of the connection to meet infinite-life design requirements using AASHTO's Fatigue I load combination. This bolted connection geometry results in an AASHTO (2014) Detail Category B and is shown in Figure 35. The stress threshold for the Fatigue I load combination for Detail Category B is 16.0 ksi and is a significant improvement over the 2.6 ksi threshold for the welded connection with a Detail Category E'.

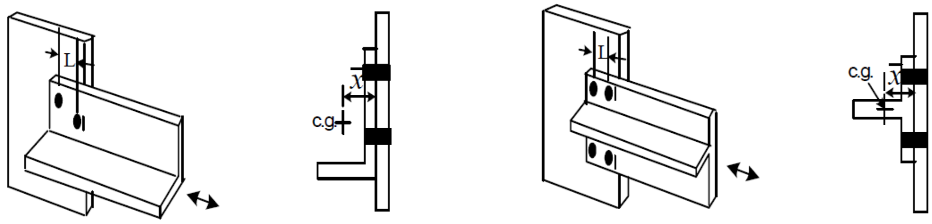


Figure 35. Diagonal Member Connection Examples of Detail Category B for Longitudinally Loaded Bolted Attachments (AASHTO 2014 Table 6.6.1.2.3-1 Description 2.5)

The welded knife-plate connection between the wide-flange vertical members and the web of the top and bottom chord WT-sections is most closely represented by AASHTO (2014) Detail Category C' shown in Figure 36. The stress threshold for the Fatigue I load combination is 12.0 ksi.

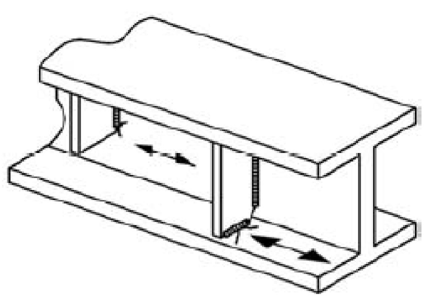


Figure 36. Example of Detail Category C' for Longitudinally Loaded Bottom Chord with Transverse Welded Attachments (AASHTO, 2014 Table 6.6.1.2.3-1 Description 4.1)

A drawing of a single steel truss girder panel showing the wide flange vertical members for this new option is shown in Figure 40.

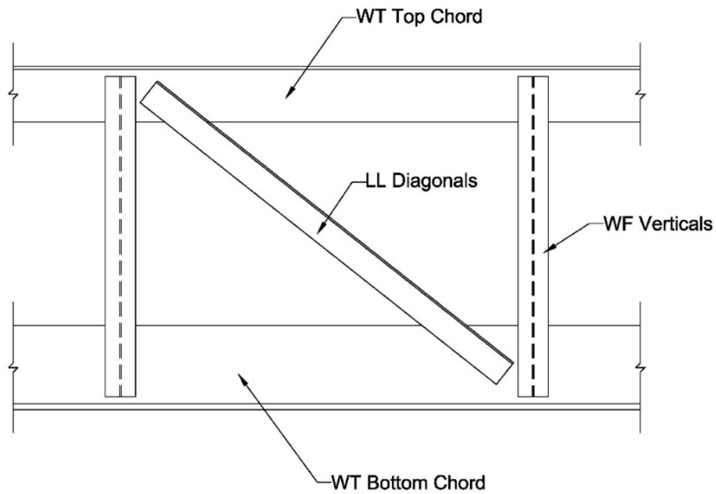


Figure 37. Typical Panel Layout of Option 4

To further explore this new steel truss girder configuration (labeled Option 4), a preliminary design was completed for the 148 ft. span using the AASHTO Strength I load combination. The weight comparison for the three steel truss girder options and the plate girder are shown in Table 9.

Table 9. Weight Comparison

Span (ft.)	Option 1 (lbs.)	Option 2 (lbs.)	Option 4 (lbs.)	Plate Girder (lbs.)
148	29,100	28,800	30,000	36,560

As previously mentioned, although the preliminary design indicates Option 4 is slightly heavier than Options 1 and 2, the lower price-per-pound for wide-flange material compared with hollow structural shapes could contribute to a more-economical steel truss girder.

Before continuing with the fatigue analysis for the new steel truss girder configuration, three additional bridge spans were analyzed and compared with comparable steel plate girders to evaluate the change in steel weight for different span lengths. A preliminary design was performed for 100 ft., 125 ft., and 193 ft. spans to determine the truss member sizes and plate girder proportions for each span. The apparent random 193 ft. span was selected to match a recently constructed plate girder project by MDT in which the actual girder weight was known. A plot of steel weight vs. span length is shown in Figure 38. The difference between the total weight of steel for the two systems increases for larger spans.

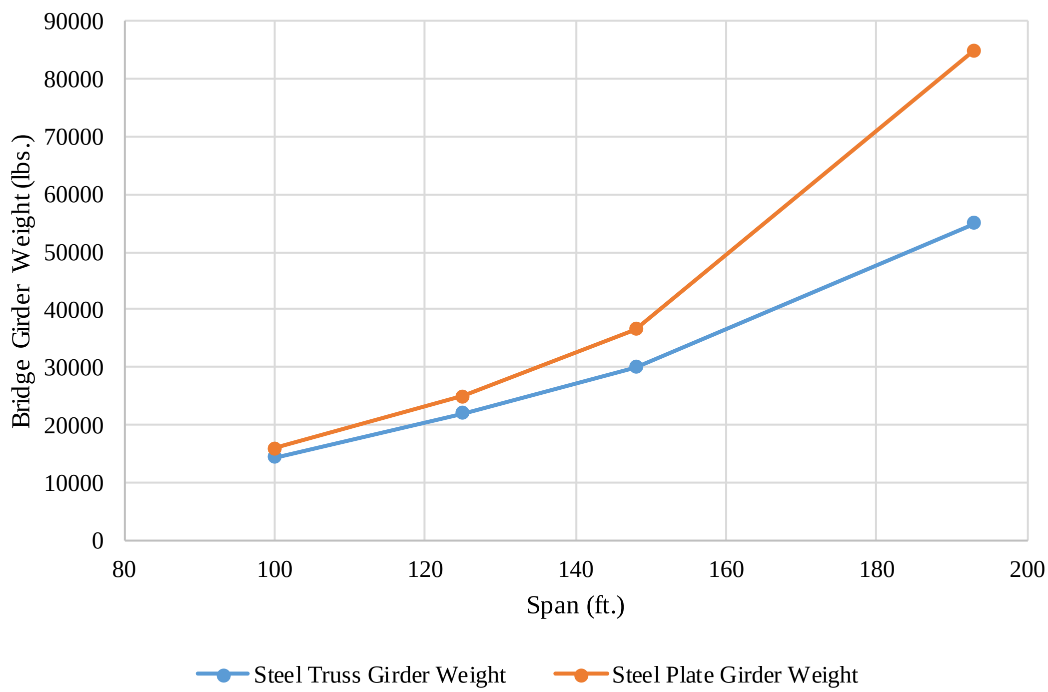


Figure 38. Comparison of Steel Truss Girder and Steel Plate Girder Weight as Span Changes

Summary

A preliminary analysis of a 148 ft. span prefabricated steel truss girder system was completed using AASHTO's Strength I, Fatigue I, and Fatigue II load combinations. Results indicate that 4 of the 12 diagonal truss members may need larger cross-sections to meet Strength I requirements. Load-induced fatigue stresses for the Fatigue I load combination exceed threshold values by a factor of approximately 4.0 for an infinite-life design. For a 75-year design life using Fatigue II load combinations, fatigues stresses exceed threshold values by approximately 18% based on measured traffic on Hwy 200 East of Jordan, MT.

Material and fabrication cost estimates were obtained from three sources for two of the 148 ft. steel truss girder configurations and a comparable plate girder. The estimates suggest the welded steel truss girder options cost approximately 5% to 20% less than a comparable steel plate girder.

Because of the cost savings estimated for the lighter 148 ft. steel truss girder compared with a comparable steel plate girder and the potential additional savings available for longer spans, a 205 ft. steel truss girder was selected for further investigation. Notably, MDT is currently completing a 205 ft. span bridge project using a steel plate girder design, allowing for a better system-to-system comparison. To overcome fatigue limitations, bolted connections were evaluated. The analysis of this new configuration is described in the following chapter.

CHAPTER FOUR

ANALYSIS OF A 205 FT. SPAN STEEL TRUSS GIRDER WITH BOLTED AND WELDED CONNECTIONS

A preliminary analysis and design of a 205 ft. steel truss girder bridge was next undertaken in this study, as MDT is currently designing a 205 ft. steel plate girder bridge for the Swan River crossing on Route S-209 . Bolted and welded connections were used for the steel truss girder to help improve the fatigue response. To further investigate the potential material and fabrication cost savings for the lighter steel truss girder system, a three-dimensional finite element model was created to more accurately estimate the distribution of multiple lane and axle loads to the steel truss girders in the system and attendant individual truss members. This distribution was compared to that obtained using an approximate factor calculated using an equivalent moment of inertia with expressions for steel plate girders from AASHTO (2014).

The resulting load distribution, less conservative than that calculated using the lever rule, was then used to determine design demands on individual truss members and connections for the Strength I, Service II, and Fatigue I load combinations. Two steel truss girder configurations were evaluated. The first was a conventional construction alternative where the concrete deck is cast after truss erection at the site. The second configuration utilized accelerated construction where the concrete deck is cast prior to shipping the prefabricated system to the bridge site. Member sizes were subsequently selected for both steel truss girder configurations, and selected connection details determined.

Preliminary 205 ft. Steel Truss Girder Design

The preliminary analysis and design of the 205 ft. steel truss girder bridge was completed to evaluate the fatigue response of the new connections. Bolted connections were used between the diagonal members and top and bottom chords. The vertical wide-flanges were assumed to be welded to the top and bottom chord. Double-channel sections were selected as the diagonal members to improve the connection geometry for the bolted connections. The spacing of the steel truss girder was 8.75 ft. and the concrete deck was 8 in. thick to match the plate girder design by MDT. The preliminary truss member sizes are shown in Table 10. The finite element program SAP2000 was again used for the analysis of this new steel truss girder system with the same modeling parameters as the 148 ft. model. An elevation view of the bridge is shown in Figure 39. A bolted connection detail was designed based on the largest tension demand due to the fracture limit state. The bolted connection geometry is shown in Figure 40. The distribution factor calculated using the lever rule for the 205 ft. configuration was 0.93 for the Strength I load combination using two loaded lanes (Figure 27) and 0.66 for the Fatigue I Load Combination using one loaded lane (Figure 28).

Table 10. 205 ft. Bolted/Welded Steel Truss Girder Properties

Span (ft.)	Deck Thickness (in.)	Top Chord Member	Bottom Chord Member	Vertical Member	Diagonal Member	Steel Weight (lbs.)
205	8	WT16.5x65	WT20x162 / WT16.5x193.5	W10x39	2MC10x33.6 / 2MC10x25 / 2MC 8x18.7	69,000

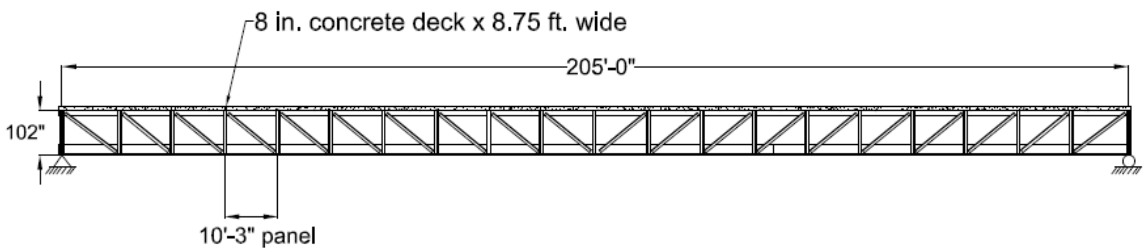


Figure 39. 205 ft. Bolted/Welded Steel Truss Girder Elevation View

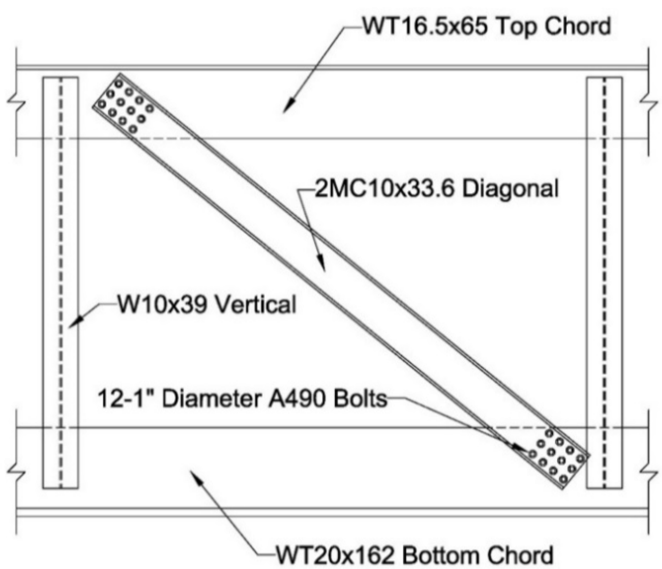


Figure 40. Bolted Connection Detail

Results indicate that the new truss members and bolted connection configuration satisfy strength and fatigue requirements for an infinite-life design. The same members identified in Figure 26 were evaluated in this analysis. Tensile stresses in the diagonal members and bottom chord members, shown in Figure 41, are well below their design yield stresses of 34.2 and 47.5 ksi, respectively, for the Strength I load combination. Tensile stresses in the diagonal and bottom chord members, shown in Figure 42, are also well below the 16.0 ksi and 12.0 ksi thresholds, respectively, using the Fatigue I load combination.

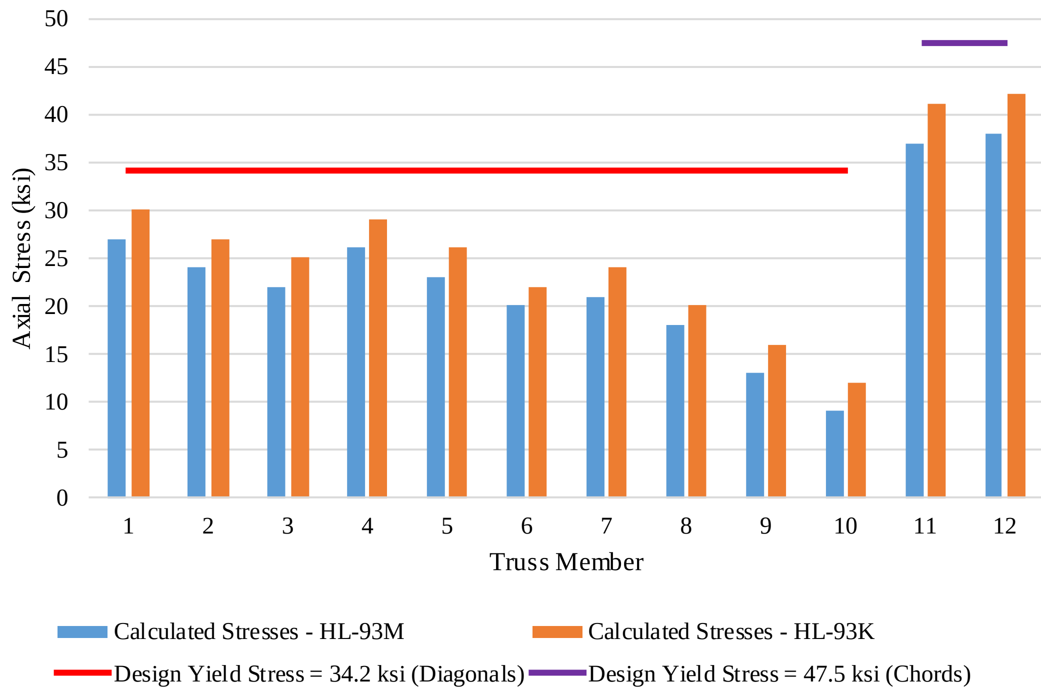


Figure 41. Axial Stress in the Diagonal and Bottom Chord Members with the Bolted Connection for the Strength I Load Combination

Live Load Distribution Factor Refined Approach

The live load distribution factors (LDF's) used for composite reinforced concrete deck on steel beams can be estimated by the equations in the AASHTO LRFD Bridge Design Specification (2014). To calculate the LDF's using the AASHTO equations requires a moment of inertia of the girder and is therefore not directly applicable to steel truss girders. For these cases, AASHTO permits the use of the lever rule or direct loading in a space analysis to calculate the LDF. The distribution factor calculated using the lever rule for the 205 ft. configuration was 0.93 for the Strength I load combination.

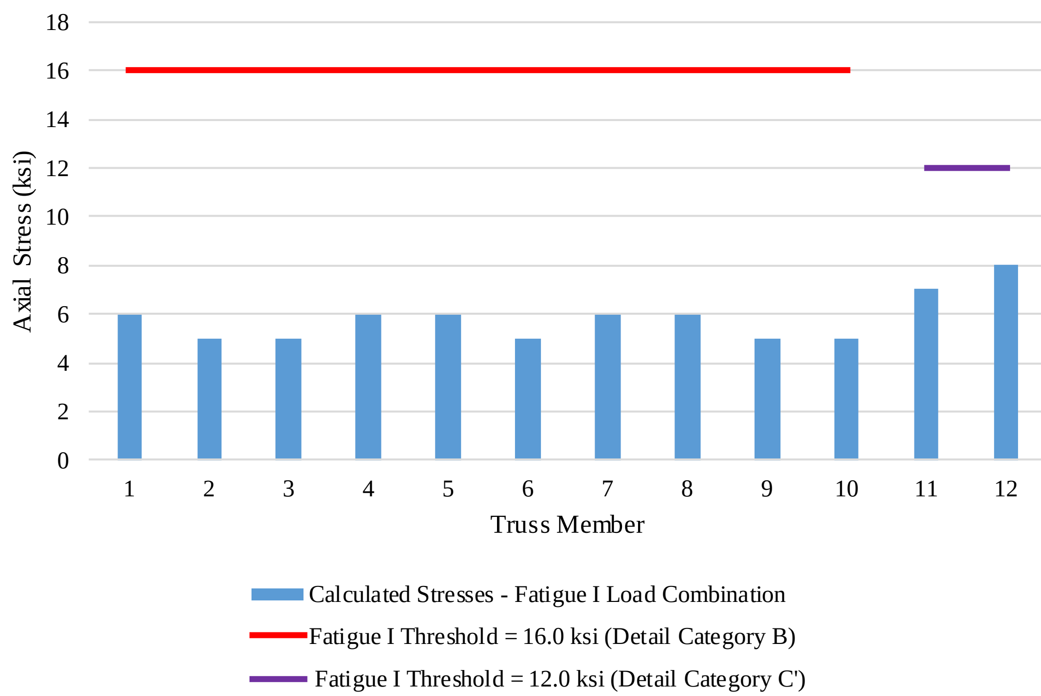


Figure 42. Axial Stress in the Diagonal and Bottom Chord Members with the Bolted Connection for the Fatigue I Load Combination

To investigate the accuracy of the distribution factor calculated using the lever rule (0.93), a 3D finite element model was created using SAP2000 (Computers and Structures Inc. 2014) to distribute the live loads to the truss members through the concrete deck (Figure 43). The 8 in. concrete deck was modeled with approximately 1 ft. x 1 ft. shell elements. Concrete strength was 4,000 psi. To simplify modeling and appropriately generate composite action, the slab and top chord elements were coincidently located at the composite neutral axis. An effective moment of inertia of one-half of the gross moment inertia ($I_e = 0.5I_g$) was used for the concrete slab in the transverse direction (consistent with a cracked cross-section) and gross section properties were assumed in the longitudinal direction (consistent with an un-cracked cross-section in compression). Similar to the 2D model used in the preliminary analysis for the 148 ft. span steel truss girder, the bottom chord, diagonal and vertical members were pin-connected at the panel points and a continuous member was used for the top chord.

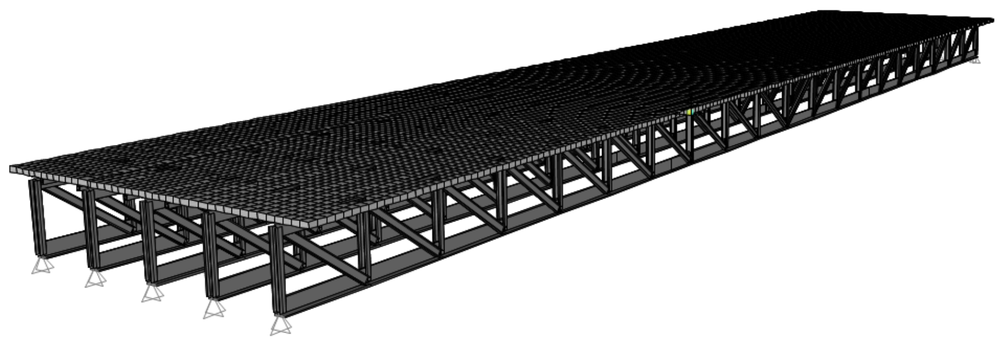


Figure 43. 3D Finite Element Model

3D Loading

The clear roadway width of 40 ft. for the proposed steel truss girder bridge requires up to three design lanes of traffic to be considered in the analysis (AASHTO 3.6.1.1). A multiple presence factor (*m*) is applied to the loads to account for the probability of simultaneous lane occupation by the full design load. To match the loading used to calculate the distribution factors with the lever rule for the 2D model, two loaded lanes were considered with a multiple presence factor of 1.0. The locations of the distributed lane load and concentrated HL-93 design truck are shown in Figure 44. The HL-93 truckloads were applied as moving loads along the length of the bridge in the SAP2000 model and resulted in an envelope of tension and compression forces in the steel truss girder.

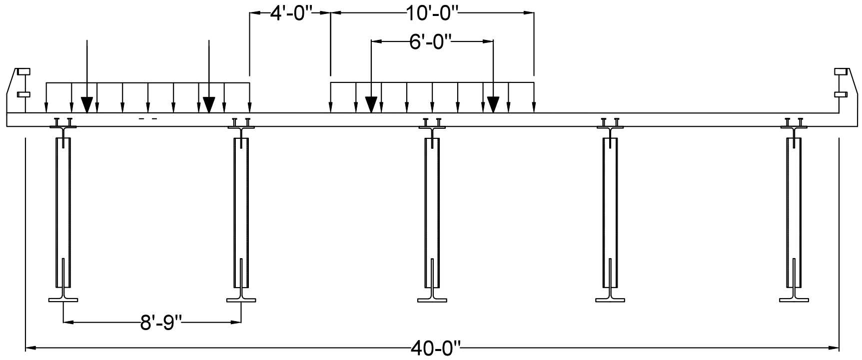


Figure 44. Location of Uniform Lane Loads and Concentrated Design Truck Loads for a Two-Lane Condition

<u>3D Analysis</u>

Controlling forces in the individual truss members are calculated in a 3D model by varying the locations of the loads and number of lanes along the continuous concrete deck. To evaluate the magnitude of the tension and compression forces from the 3D analysis using the location of loads shown in Figure 44, forces in the individual truss members were compared with those from a 2D model using a distribution factor of 1.0. The ratio of the 3D to 2D forces represents the reduction in truss member forces achieved by distributing the applied loads to the steel truss girder through an explicit model of the concrete deck, rather than relying on simplified distribution factors available for this purpose in AASHTO Section 4.6.2.2.2. A similar procedure was used in the study conducted by Eom and Nowak (2001). A comparison of the maximum tension and compression forces from the two models are shown in Table 11. The 3D to 2D ratios indicate a live load distribution factor of 0.55 could be used to estimate vertical, diagonal, and bottom chord forces in a simplified 2D model.

Table 11. 2D Distribution Factor vs. 3D Finite Element Model Results for the Proposed Steel Truss Girder Geometry using SAP2000

Loading	Maximum Tension (+) / Compression (-) Forces (kips)					
	2D Model			3D Model		
	Vertical	Diagonal	Bottom Chord	Vertical	Diagonal	Bottom Chord
Design Lane	-66	104	431	-37	56	273
Design Truck	-66	107	437	-36	52	172
Design Lane + Design Truck	-132	211	868	-73	108	445
3D / 2D Ratio				0.55	0.51	0.51

A similar comparison was made for the Swan River plate girder bridge. Calculated bending moments for the middle girder using AASHTOWare Bridge Design/Rating software were provided by MDT and the results from the 2D and 3D analyses are shown in Table 12. Note that the AASHTOWare software is programmed to evaluate multiple locations of the HL93 vehicle load, while in the analysis done above using the more general purpose SAP2000 program, only one position for these loads was considered. Referring to Table 11 and Table 12, the 3D / 2D ratios for the steel truss girder using the SAP2000 model with a single load configuration is comparable with a similar 3D to 2D analysis for the Swan River plate girder bridge using AASHTOWare and multiple load positions.

Table 12. 2D Distribution Factor vs. 3D Finite Element Model Results for the Swan River Plate Girder using AASHTOWare

Loading	Mid-span Bending Moment (kip-ft.)	
	2D Model	3D Model
Design Lane	3364	1716
Design Truck	4537	2428
Design Lane + Design Truck	7901	4144
3D / 2D Ratio		0.52

Approximate Live Load Distribution Factor

To apply the current LDF equations in AASHTO (2014) and the proposed expression by Cai (2005), a moment inertia for the girder is needed to represent the relative stiffness between the longitudinal and transverse bridge directions. Using the equivalent distribution factor of 0.55 calculated from the 3D finite element model, truss member sections were proportioned, and deflections were calculated at each of the steel truss girder nodes using the 2D model. These displacements were used to back-calculate moments of inertia at each location and resulted in an average equivalent moment of inertia of 121,245 in.^{4}.

A recent investigation by Siekierski (2015) proposed a simplified equation for computing the moment of inertia of a steel truss by combining average individual moments of inertia (I_t and I_b) and areas (A_t and A_b) for the top and bottom chord and the distances from the truss center of gravity to the center of gravity of the top and bottom chords (z_t, z_b). This expression results in an equivalent moment of inertia of 112,248 in.^{4}, which is close in magnitude (nine percent lower) then the value obtained above.

$$I_{eqv} = I_t + A_t z_t^2 + I_b + A_b z_b^2$$

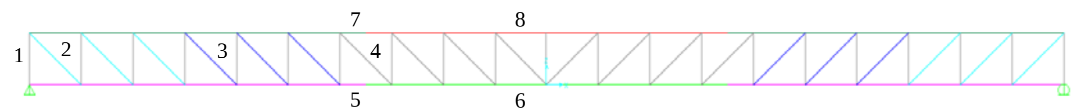
Using the analytical moment of inertia obtained from the 2D model displacements and the simplified approach from Siekierski (2015), distribution factors were calculated using the three LDF equations described above and are shown in Table 13. Based on the increased range of bridge configurations where the AASHTO equations are applicable (Table 4) and the negligible influence of the moment of inertia (Table 13), a distribution factor of 0.63 was selected for the Strength I load combination with two loaded lanes and a multiple presence factor of 1.0. This distribution factor was used to proportion the steel truss girder member sizes for the subsequent evaluation of the fatigue response of the connections for the 205 ft. steel truss girder considered in this investigation.

Table 13. Distribution Factor Comparison

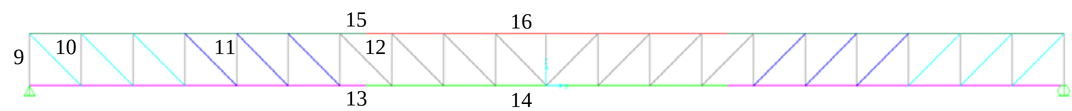
Equivalent Moment of Inertia (in.^{4})	Live Load Distribution Factor			
	AASHTO	Cai	Huo	Lever Rule
121,245	0.63	0.64	0.66	0.93
112,248	0.63	0.64	0.66	0.93

Refined 205 ft. Steel Truss Girder Design

The 2D SAP2000 model with a distribution factor of 0.63 was used to calculate truss member forces for two steel truss girder configurations. The first configuration (Truss 1) assumed conventional construction methods where the concrete deck would be cast in place after steel erection at the site. The second configuration (Truss 2) is an accelerated construction method where the concrete deck would be cast prior to shipping the prefabricated composite assembly to the bridge site. The location of the members selected for design are shown in Figure 45. The difference between the two configurations is the larger top chord required for the conventional construction method (Truss 1). For the accelerated construction scenario, it was assumed that based on the construction method, the self-weight of the structure (steel truss girder + deck) in service would be carried by the composite cross-section. Conversely, for the conventional construction scenario, assuming no shoring is used in the construction process, the self-weight of the steel truss girder and deck is carried just by the steel truss girder, with due consideration of all incidental loads that have to be supported by the steel truss girders during deck construction. The depth required for the bolted diagonal connection controlled the top chord design for conventional construction.



(a) Truss 1



(b) Truss 2

Figure 45. Location and Designation of Truss Members Designed for (a) Truss 1 using Conventional Construction and (b) Truss 2 using Accelerated Construction

Service and Design Forces

Calculated service-level forces from SAP2000 are shown in Table 14 and Table 15 for Truss 1 and Truss 2, respectively. Factored load combinations used for member and connection design are shown in Table 16 and Table 17, again for Truss 1 and Truss 2, respectively. Referring to Table 14 and Table 15, as would be expected, the live load demands in individual truss members (with the exception of the construction live load demands) in the two steel truss girder configurations are effectively identical, as these demands are carried in both systems by the identical composite steel truss girder/concrete deck system. The truss member forces are different in the two configurations for the demands from the dead load of the steel truss girder and deck, as this demand is carried by just the steel truss girder in conventional construction (Truss 1) scenario.

The member’s forces are approximately 10% lower in the Truss 2 compared to the Truss 1 scenario. Correspondingly, and as is seen in Table 16 and Table 17, in load cases dominated by dead load demands (i.e., Strength I and Service II), design forces are similarly smaller in Truss 2 compared to Truss 1. Selected member sizes and the total steel weight for the two steel truss girder configurations are presented for Truss 1 and Truss 2 in Table 18 and Table 19, respectively. The only difference between the member designs for the two steel truss girders is for the top chord members, with heavier members being used for Truss 1. In general, two different member sizes were used across the top and bottom chords in each steel truss girder, with three different member sizes for the diagonals.

While not reported in detail, there was relatively small variation in factored loads for the vertical members, and a single member size was selected for fabrication efficiency. The calculated mid-span deflection was 2.8 in. (L/880) using the controlling load from 25% of the design truckload with the design lane load (AASHTO 3.6.1.3.2) for both configurations.

The steel weight for the accelerated construction alternative (Truss 2) using the refined LDF design is 28% less than the Swan River plate girder (68k vs. 94k). Following the conventional construction alternative (Truss 1), the additional steel weight in the top chord results in only a 15% reduction in steel weight from the Swan River plate girder (80k vs. 94k).

Table 14. Calculated Service Level Forces for Truss 1

Member Number	Axial Tension (+) / Compression (-) Force (kips)						
	Steel/Concrete Weight	Formwork Weight	Construction Live Load	Design Lane Load	Design Tandem	Design Truck	Design Truck (Fatigue)
1	-128	-13	-18	-66	-50	-66	-66
2	191	20	27	97	73	100	95
3	129	14	18	65	57	77	77
4	70	7	10	35	48	65	60
5	649	68	91	330	245	335	315
6	765	80	107	388	288	393	364
7	-703	-73	-98	-357	-264	-361	-340
8	-772	-81	-108	-393	-290	-397	-367

Table 15. Calculated Service Level Forces for Truss 2

Member Number	Axial Tension (+) / Compression (-) Force (kips)				
	Steel/Concrete Weight	Design Lane Load	Design Tandem	Design Truck	Design Truck (Fatigue)
9	-116	-66	-50	-66	-66
10	174	98	74	101	96
11	117	66	62	83	79
12	64	36	50	67	62
13	590	331	247	337	318
14	696	390	291	395	367
15	-639	-358	-267	-364	-343
16	-704	-395	-293	-400	-369

Table 16. Factored Load Combinations Considered for Truss 1

Member Number	Axial Tension (+) / Compression (-) Force (kips)			
	Strength I	Service II	Fatigue I	Fatigue II
1	-328	-277	-53	-26
2	493	416	77	38
3	353	298	62	31
4	221	188	48	24
5	1665	1404	254	127
6	1960	1653	293	147
7	-1801	-1513	-274	-137
8	-1981	-1671	-296	-148

Table 17. Factored Load Combinations Considered for Truss 2

Member Number	Axial Tension (+) / Compression (-) Force (kips)			
	Strength I	Service II	Fatigue I	Fatigue II
9	-319	-246	-53	-26
10	482	370	78	39
11	347	266	63	32
12	221	169	50	25
13	1624	1249	256	128
14	1911	1469	296	148
15	-1757	-1351	-276	-138
16	-1934	-1487	-297	-149

Table 18. 205 ft. Bolted/Welded Truss 1 Properties

Span (ft.)	Deck Thickness (in.)	Top Chord Member	Bottom Chord Member	Vertical Member	Diagonal Member	Steel Weight (kips)
205	8	WT18x116 / WT18x128	WT20x162 / WT20x181	W10x39	2MC10x28.5 / 2MC10x22 / 2MC8x18.7	80

Table 19. 205 ft. Bolted/Welded Truss 2 Properties

Span (ft.)	Deck Thickness (in.)	Top Chord Member	Bottom Chord Member	Vertical Member	Diagonal Member	Steel Weight (kips)
205	8	WT16.5x65	WT20x162 / WT20x181	W10x39	2MC10x28.5 / 2MC10x22 / 2MC8x18.7	68

Fatigue Analysis Results for the Bolted and Welded Connections

The Fatigue I and II load combinations were considered for the bolted and welded connections of the 205 ft. steel truss girder. Using the member sizes proportioned using a distribution factor of 0.63 (Table 18 and Table 19), the moment of inertia calculated from the 2D model displacements was 195,390 in4 (Truss 1) and 130,026 in4 (Truss 2). These values were used to calculate appropriate distribution factors for the Fatigue I and Fatigue II load combinations. Both values gave a distribution factor of 0.56. The distribution factor and stress thresholds of the relevant Detail Categories for the connections are shown in Table 20.

Table 20. Threshold Stresses and Distribution Factors used for the Fatigue I and Fatigue II Load Combinations

Load Combination	Fatigue Threshold (ksi) Detail Category		Distribution Factor
	B	C’	
Fatigue I	16.0	12.0	0.56
Fatigue II	14.3	10.2	

The refined truss members for Truss 1 and Truss 2 were found to satisfy fatigue requirements for an infinite and finite-life design. The same members identified in Figure 26 were evaluated for fatigue. Axial stresses in these members are shown in Figure 46 and Figure 47 relative to the Fatigue I and II thresholds for Detail Category B and C’.

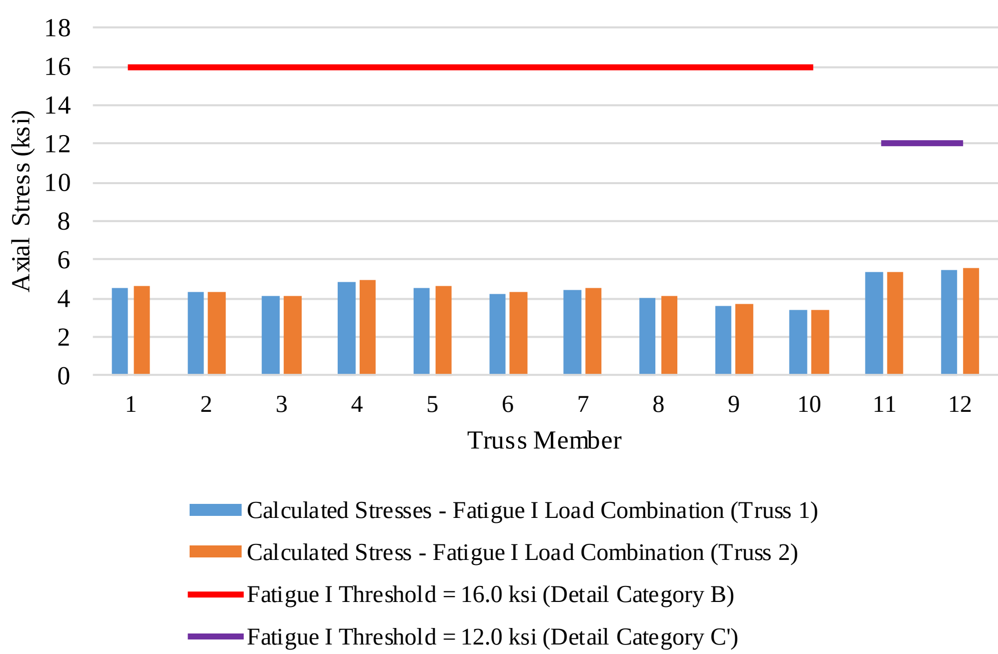


Figure 46. Axial Stress in the Diagonal and Bottom Chord Members of Truss 1 and Truss 2 with the Bolted/Welded Connection for the Fatigue I Load Combination

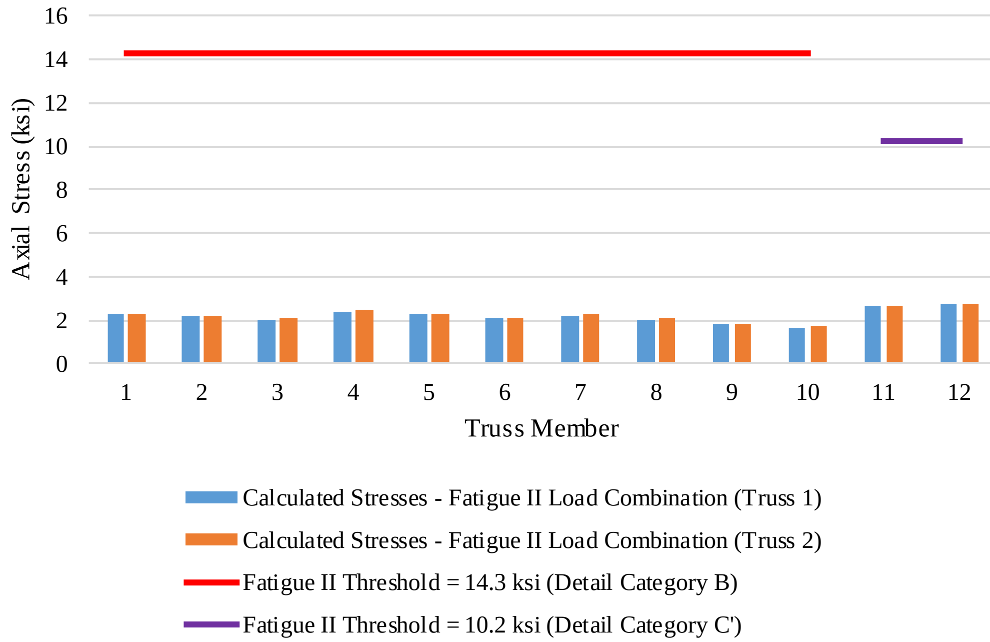


Figure 47. Axial Stress in the Diagonal and Bottom Chord Members of Truss 1 and Truss 2 with the Bolted/Welded Connection for the Fatigue II Load Combination

Connection Design

Using the factored loads shown in Table 16 and Table 17 and the refined member sizes shown in Table 18 and Table 19, connection designs were completed at the joints of three different steel truss girder panels (see Figure 48). Limit states considered in the connection design include bolt shear, tension rupture, and tension yielding using loads from the Strength I load combination. The slip critical connections were designed using the Service II load combination. The connection details are shown in Figure 49, Figure 50, and Figure 51 for each selected panel.

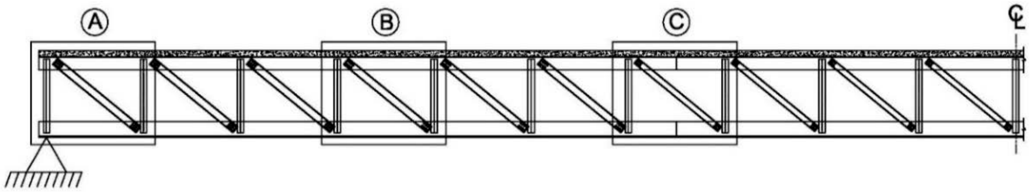


Figure 48. Connection Detail Locations

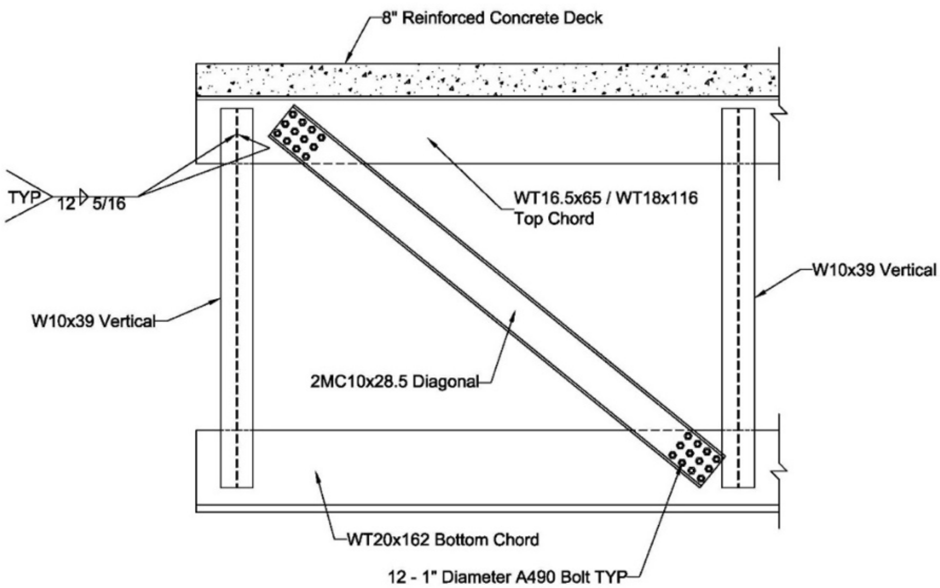


Figure 49. Connection Detail A (12-bolt connection)

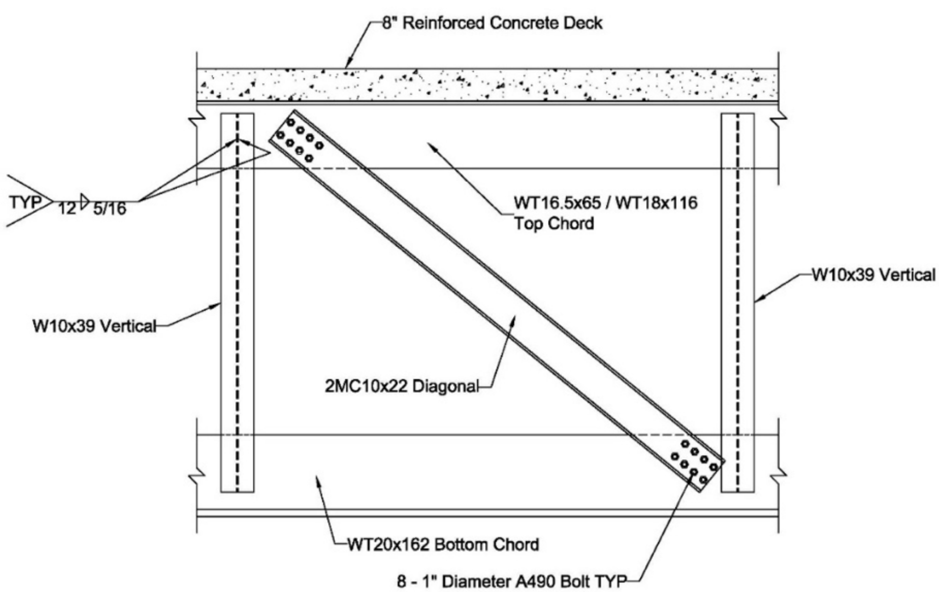


Figure 50. Connection Detail B (8-bolt connection)

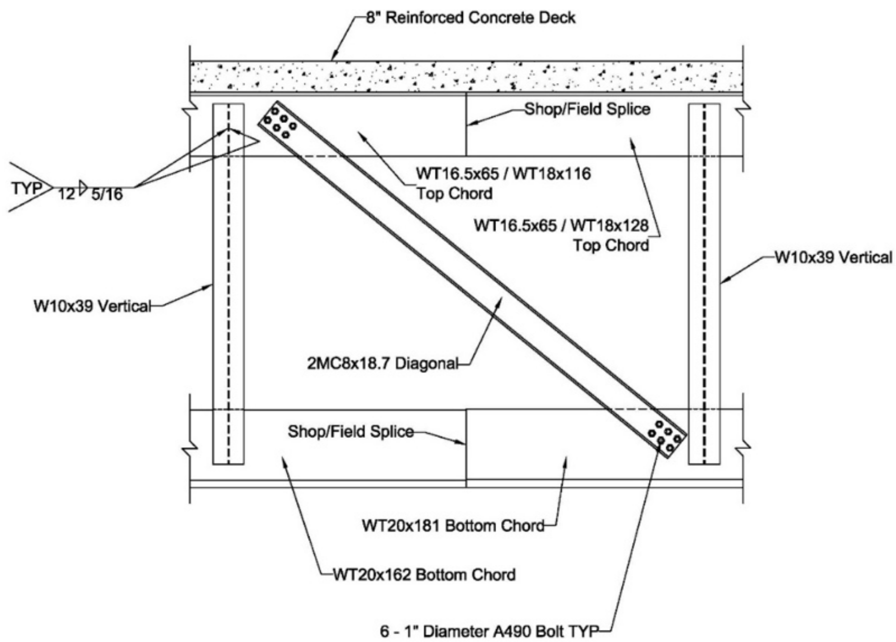
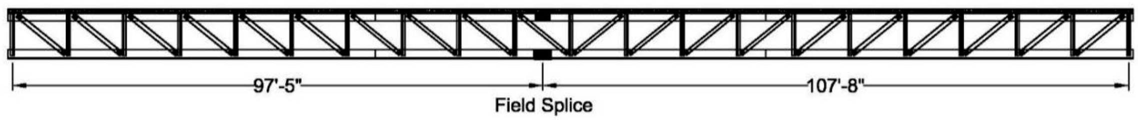


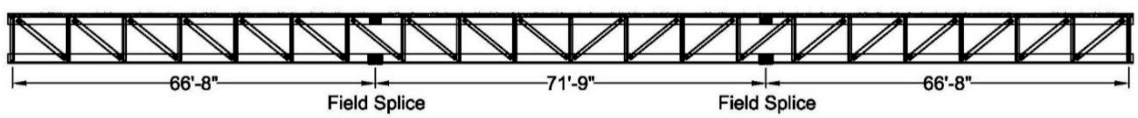
Figure 51. Connection Detail C (6-bolt connection)

Splice Locations

Based on shipping regulations and construction considerations related to component weight and length, two different splice locations are proposed for this welded/bolted steel truss girder bridge. A single splice at the steel truss girder mid-span was selected for a conventional concrete deck cast after erection of the steel truss girders (Truss 1). Two splices, each located at approximately 1/3 points of the 205 ft. span, were selected for the accelerated construction method in which the concrete deck would be cast prior to erection (Truss 2). Locations of the splices for the two configurations are shown in Figure 52. Details for the two splice configurations are shown in Figure 53 and Figure 54. Limit states considered in the design of both splice connections were the same as those considered in designing the truss member connections (bolt shear, tension yielding, and rupture).

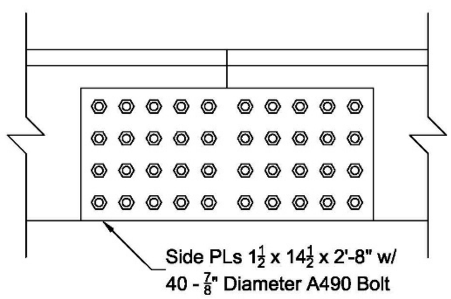


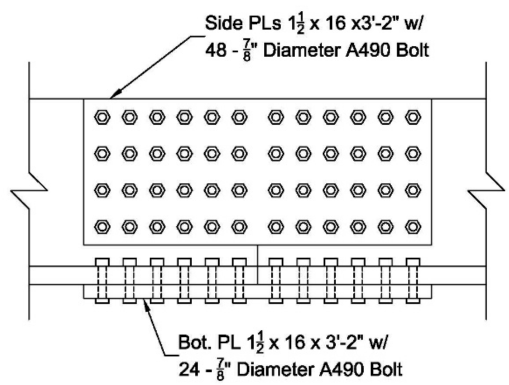
(a) Truss 1



(b) Truss 2

Figure 52. Proposed Steel Truss Girder Elevation with (a) Single-Splice and (b) Two-Splice Condition



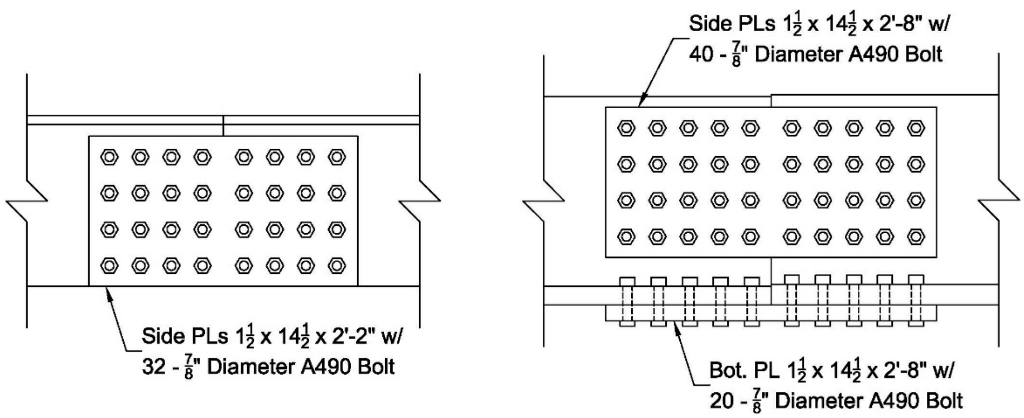


(a) Top Chord (b) Bottom Chord

Figure 53. Splice Connection Details for the Single-Splice in Truss 1

Summary

A 3D finite element model was created to more accurately distribute the loads to the bolted and welded steel truss girders and associated truss members in the 205 ft. Swan River crossings being considered in this analysis. Based the more refined 3D finite element analyses of this system and an approximate load factor calculated using an equivalent moment of inertia, the decision was made to move forward with a distribution factor of 0.63 compared to 0.93 used in the earlier analysis. This factor is relatively consistent with the factor calculated by AASHTO formula for the plate girders in the Swan River crossing, which is being used for comparative purposes in this investigation. Member forces from the refined 2D analysis were used to design selected truss members, connections, and splices for two scenarios, namely, use of conventional and accelerated construction methods. Significantly larger top chord members were required for the conventional construction scenario to support construction loads during casting the deck after steel truss girder erection. The steel weight of the steel truss girder increased by 18% using the larger top chord members. Fatigue was not an issue in any of the redesigned truss members and end connections.



(a) Top Chord (b) Bottom Chord

Figure 54. Splice Connection Details for the Two-Splices in Truss 2

CHAPTER FIVE

COST AND CONSTRUCTION CONSIDERATIONS

Updated materials and fabrication costs were obtained from Allied Steel, AVEVA, and RTI, Inc. A cost comparison was made for both the conventional and accelerated construction alternatives to assess the impact of the bolted connections and the refined member design for the 205 ft. span. Potential construction and erection advantages for these configurations were compared with the Swan River plate girder with input from Sletten Construction (Great Falls, MT) and Dick Anderson Construction (Missoula, MT). The impact of the two splice configurations and of the member weights and lengths in the two systems were assessed relative to steel plate girders.

Material and Fabrication Costs

Estimated prices for a bolted/welded steel truss girder and a plate girder for the Swan River crossing, obtained from the same sources used previously, are shown in Table 21. Costs of the splice connections were not included in the estimates.

The variation in estimates shown in Table 21 reflect many different fabrication aspects. Allied Steel provided a quotation for the three alternatives that included labor estimates for the welded and bolted connections. The labor rates used by AVEVA are representative of approximate rates for fabricators located across the country. The estimates from RTI were based on an approximate cost of \$1.40/lb. of steel, which was the same value used for the cost estimate of the all-welded steel truss girder discussed above. Because the three cost estimates included these different assumptions an average value was selected to represent the potential cost savings for the two steel truss girder alternatives. Relative to the plate girder alternative, the average values shown in Table 21 result in an estimated materials and fabrication cost savings for Truss 1 and Truss 2 of 10% and 26%, respectively, compared to the plate girder.

Table 21. Final Steel Price Estimates

Company	Plate Girder	Truss 1	Truss 2
Allied Steel	$135,000	$105,000	$94,000
AVEVA	$95,000	$103,000	$85,000
RTI Fabrication	$126,000	$112,000	$84,000
Average	$119,000	$107,000	$88,000

Allied Steel indicated that the bolted connections between the diagonal members and bottom chord would be less expensive than the welded connections considered previously in the preliminary evaluation. Allied Steel also pointed out that camber could be built in to the bolted and welded steel truss girder connections during fabrication and would eliminate the need for heat curving, a practice commonly done for large plate girders. Another additional cost associated with the plate girder is the required weld inspections for the full penetration welds between shop splices in the flange and the web. Inspection of the fillet welds used for the vertical truss members would not be required according to Allied Steel.

Shipping

The structural elements being considered for this 205 ft. bridge are large enough that issues could be encountered in shipping them to the job site. A summary of some general shipping requirements in Montana (Montana Department of Transportation 2006) were provided by True North Steel (Billings, MT) and are shown in Table 22.

Table 22. Shipping Guidelines for Montana

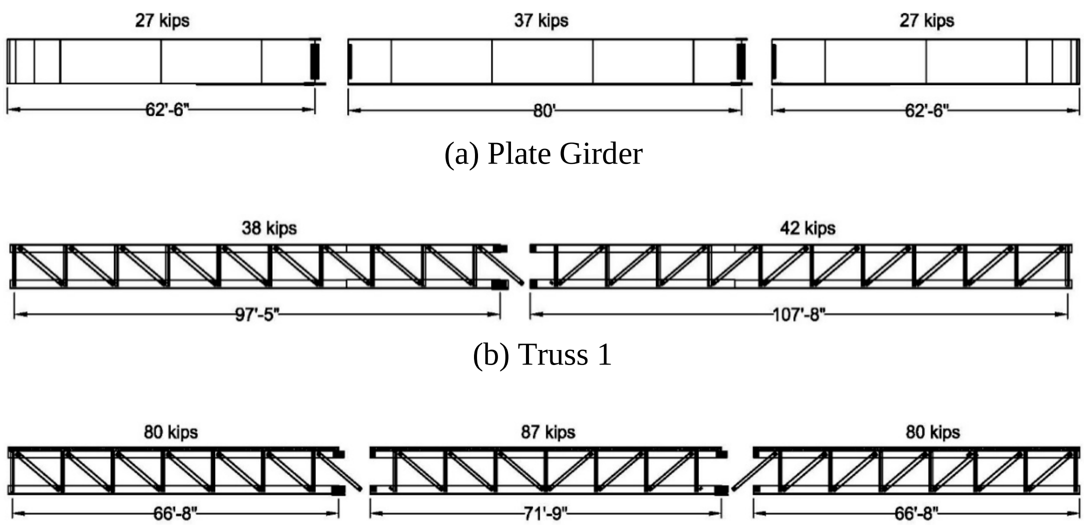
Gross Legal Load	Up to 120,000 lbs., depending on trailer/axle combination
Flag Vehicle Requirements	One flag vehicle for loads > 120 ft. on interstate One flag vehicle for loads > 110 ft. on non-interstate
Permit Requirements	Lengths over 75 ft.

The weights of the steel truss girders and plate girders for the single and two-splice configuration are shown in Table 23. An elevation view with the weight of each splice section for the plate girder and both steel truss girder alternatives are shown in Figure 55 with the weight of the concrete deck being included in the total weight of each splice section for Truss 2.

Table 23. Length and Weight of Steel Plate Girder and Steel Truss Girder Construction Alternatives

Bridge Type	Member Lengths (ft.)	Approximate Weight (kips)		
		Steel	Concrete Deck	Total Lift Weight
Plate Girder (2 splices)	62.5 / 80 / 62.5	27 / 37 / 27	-	27 / 37 / 27
Truss 1 (conventional construction, 1 splice)	108 / 97	42 / 38	-	42 / 38
Truss 2 (accelerated construction, 2 splices)	66.7 / 71.8 / 66.7	22 / 24 / 22	58 / 63 / 58	80 / 87 / 80

For the 205 ft. bridge span under consideration, True North Steel indicated a preference to ship steel truss girders with a single splice configuration. The maximum member length for this condition is approximately 108 ft. (Figure 55) and would require a permit (Table 22). The bare-steel weight of 40 kips would enable up to three steel truss girders to be delivered on a single truck without exceeding the gross legal load. The two-splice steel truss girder with a cast-in-place deck has a length of approximately 71 ft. and a total weight of 85 kips. A single steel truss girder with concrete deck could be shipped without exceeding legal load requirements but would require a permit due to the total length with the towing unit.



(a) Plate Girder

(b) Truss 1

(c) Truss 2

Figure 55. Weight of each Splice Section for the (a) Plate Girder, (b) Truss 1 and (c) Truss 2

Erection

Potential erection issues were also considered with the steel truss girder and steel plate girder systems through consultation with Sletten Construction Company (Great Falls, MT) and Dick Anderson Construction (Missoula, MT). Sletten indicated that the one- and two-splice configurations for Truss 1 and the plate girder would be approximately equivalent if the existing bridge is available to use for construction. In this case, the steel truss girder or steel plate girder would be connected on the ground using two cranes, rolled on to the existing bridge and then set in place using two cranes. This construction method with Truss 2 was not recommended by Sletten because of the additional weight from the precast integral deck. Without access to the existing bridge, Sletten preferred the single splice configuration of Truss 1 because only one temporary support structure would be required to set one half of the bridge while the second member is lifted by the crane to make the splice connection. Potential lifting methods and rigging pick-points for the light and slender bare steel truss girders were not evaluated.

Dick Anderson Construction preferred the Truss 2 alternative, built using either conventional or accelerated construction methods. The shorter member lengths provide easier transportation, site access, unloading, and staging than longer members. Dick Anderson Construction also suggested additional flexibility is available with the shorter member lengths and would be suitable for different construction site conditions. Advantages of Truss 2 built with accelerated construction methods (integral precast deck) would be faster construction time and a potential alternative to precast decked bulb tee systems. While decked bulb tee systems are capable of spanning up to 160 ft., at these longer lengths, transportation and site access could limit their use. The ability to field splice Truss 2 with a concrete deck would create lighter members and potentially more efficient construction.

The total number of bolts used in the two plate girder splices is 552 compared with only 224 bolts for the two splices used in Truss 2. The fewer bolts required for resisting tension and compression forces (as opposed to moment and shear in the plate girder) suggests the field splice connection may be more efficient for the steel truss girder alternatives. Dick Anderson Construction indicated that significant savings would not be realized for the smaller numbers of bolts used in a splice connection. However, reducing the number of splices from two to one results in reduced construction costs. Note also that a total of 560 bolts are used for the two field splices and the diagonal member connections for Truss 2, which is approximately the same as the number of bolts used in the two plate girder splices (552 bolts).

Summary

The steel weight of the bolted and welded steel trusses assuming conventional and accelerated construction were 15% and 28% less than the steel weight of the Swan River plate girders. Using an average of the materials and fabrication estimates from Allied Steel, AVEVA, and RTI Fabrication suggests a reduction in cost of 10% and 26% for the two construction alternatives, respectively.

Single-splice and two-splice erection alternatives were considered with input from Sletten Construction and Dick Anderson Construction. A single-splice member is the preferable alternative if the existing bridge is not available for use during construction, as then only one temporary support is required. With access to the existing bridge during construction, both splice configurations would be approximately the same in construction efficiency. Concern was expressed from a construction professional about the weight of Truss 2 with an integral precast concrete deck for use with a 205 ft. span bridge. A potential advantage for Truss 2 with an integral concrete deck, however, is a potential alternative to decked bulb tee systems with the capability of splicing two or more members together to achieve longer spans with lighter members.

CHAPTER SIX

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

Steel truss girder bridges are an efficient and aesthetically pleasing option for highway crossings. Their light weight compared with plate girder systems make them a desirable alternative for both material savings and constructability. A prototype steel truss girder bridge structure has been proposed as a potential alternative for accelerated bridge construction (ABC) projects in Montana. The proposed system consists of a prefabricated welded steel truss girder topped with a composite concrete deck cast-in-place at the fabrication facility. These composite members are transported to the site, where they are set next to each other on a prepared foundation to create the bridge. This specific bridge and prefabricated construction technique are not well represented in the literature, and thus there is a need to identify potential bridge spans and traffic volumes where the proposed system is viable and economical.

Preliminary designs were completed by Allied Steel for three different prefabricated steel truss girder/integral concrete deck bridge systems, specifically for a 108 ft. bridge over Big Dry Creek (Jordan, MT) and two configurations of a 148 ft. bridge over Cooper Creek (Thompson Falls, MT). A preliminary analysis of the 148 ft. span was completed using AASHTO’s Strength I, Fatigue I, and Fatigue II load combinations. Results indicate that load-induced fatigue stresses for the Fatigue I load combination exceed threshold values by a factor of approximately 4.0 for an infinite-life design. For a 75-year design life using Fatigue II load combinations, estimated fatigue stresses are approximately 18% higher than design requirements based on measured traffic on Hwy 200 East of Jordan, MT. Material and fabrication cost estimates from three sources for the 148 ft. steel truss girder and a comparable plate girder suggest a welded steel truss girder would cost approximately 5% to 20% less than a comparable steel plate girder. Based on discussions with steel fabricators and the projected fatigue performance of the welded connections, a new steel truss girder configuration was designed with more economical wide-flange vertical members and bolted diagonal member connections.

A 3D finite element model of the new steel truss girder configuration was created to more accurately distribute the loads to the steel truss girders and their attendant members using the geometry of the 205 ft. Swan River crossing. The resulting load distribution factor favorably compared to an approximate factor calculated using an equivalent moment of inertia with expressions for steel plate girders from AASHTO.

Conventional and accelerated construction scenarios were then considered in the design of the truss members, connections, and splices of the new system. The completed design offered an infinite fatigue life. The conventional construction scenario assumed a single splice at mid-span with a concrete deck cast after the steel truss girder was erected. For the accelerated construction scenario, the assumption was made that the steel truss girder elements with integral concrete deck would bridge the span in three segments (resulting in two splices). Using the new steel truss girder design and based on input from fabricating and construction professionals, the potential cost was determined for a 205 ft. bolted and welded steel truss girder bridge constructed using conventional or accelerated methods. The final steel truss girder designs were compared with an equivalent steel plate girder design.

The following conclusions were made from this investigation of prefabricated steel truss girder bridge deck systems project:

- The steel weight of the final bolted and welded steel truss girders assuming conventional and accelerated construction were 15% and 28% less than the steel weight of the Swan River plate girders. Materials and fabrication prices suggest a reduction in cost of up to 10% and 26% for the two construction alternatives, respectively.
- The bolted diagonal member connections meet Detail Category B requirements from AASHTO and have a threshold fatigue stress that is approximately 6.0 times greater than the welded connection Detail Category E'. The bolted connections are able to meet design requirements for an infinite life design using the Fatigue I load combination.
- A 3D analysis of the steel truss girder using geometry from the plate girder bridge over the Swan River reduced the loads to the truss members by approximately 45% compared with a 2D model using a distribution factor of 1.0.
- An approximate distribution factor was calculated using AASHTO equations with an equivalent moment of inertia. This value resulted in a factor that is simpler to calculate than the 3D model yet produces reasonable distributions that are less conservative than the lever rule.

- Significantly larger top chord members were required for the conventional construction method to support the construction loads required for casting the deck after erection. The total steel weight of the steel truss girder using the larger top chord member increased by 18% (80k for conventional construction, 68k for accelerated construction (precast deck)).
- A single splice across the bridge span and two splices for accelerated construction methods were considered. Input from erection and construction professionals indicate a single splice is preferred if a temporary support structure is required during erection.

Based on this investigation, the steel truss girder configurations for both conventional and accelerated construction methods are attractive alternatives for bridges using bolted and welded connections. More specific materials, fabrication, and construction savings from these systems could be identified with a completed final design and a specific construction site to consider.

Recommendations for Future Work

The following implementation recommendations are made based on the results of the Prefabricated Steel Truss Bridge Deck Systems project:

- Potential bridge crossing sites and geometries should be discussed with steel fabricators and local contractors to receive more specific suggestions for successfully implementing a steel truss girder bridge system built using conventional or accelerated construction methods.

- The joint and concrete deck performance of the Maxwell Coulee bridge that utilized a rolled wide-flange section with an integral concrete deck should be monitored.
- Alternative contracting methods for a steel truss girder bridge constructed with an integral concrete deck should be investigated. The Construction Manager/General Contractor method could provide a more efficient and economical delivery.
- A final design should be completed of a steel truss girder for a selected bridge crossing with input from an erector and fabricator combined with Maxwell Coulee observations.
- A monitoring and evaluation program should be conducted on any such bridge, including instrumentation and remote data acquisition.

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