INVESTIGATION OF PREFABRICATED STEEL-TRUSS BRIDGE DECK SYSTEMS

FHWA/MT-17-009/8226-001

Final Report

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THE STATE OF MONTANA
DEPARTMENT OF TRANSPORTATION

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FEDERAL HIGHWAY ADMINISTRATION

November 2017

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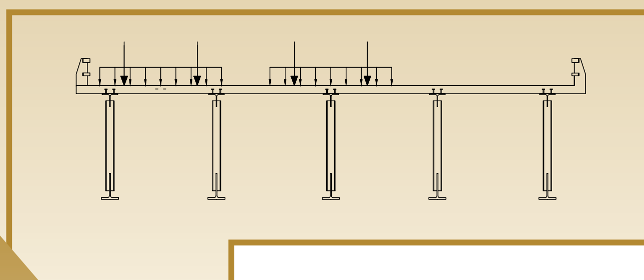

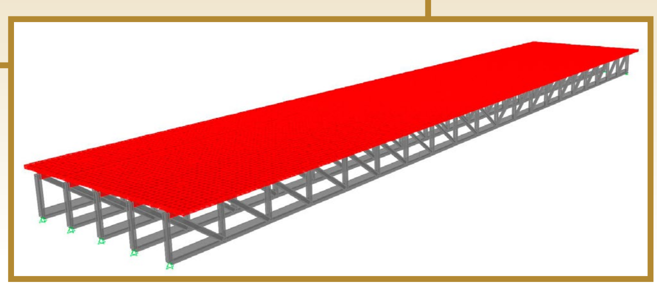

RESEARCH PROGRAMS

MDT

Investigation of Prefabricated Steel-Truss Bridge Deck Systems

Final Report

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16. Abstract
Steel truss bridges are an efficient and aesthetic option for highway crossings. They are relatively light weight compared with plate girder systems make them a desirable alternative for both material savings and constructability. A prototype of a welded steel truss 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 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 trusses, bolted connections between the diagonal tension members and the top and bottom chords of the truss were evaluated. In this research, both a conventional cast in place deck system and an accelerated bridge deck system (cast integral with the truss) were evaluated for the bolted/welded steel truss bridge. A 3D finite element model was used to more accurately calculate the distribution of lane and truck loads to the individual trusses. Truss members and connections for both construction alternatives were designed using loads from AASHTO Strength I, Fatigue I, and Service II load combinations. A comparison was made between the two truss configurations and a 205 ft. plate girder used in a previously designed bridge over the Swan River. Materials and fabrication estimates suggest the cost of the conventional and accelerated construction methods is 10% and 26% less, respectively, than the plate girders designed for the Swan River crossing.

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1. Introduction

This final report summarizes the literature review, analytical evaluation, and analysis of results for the Prefabricated Steel Truss Bridge Deck Systems project. 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 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.

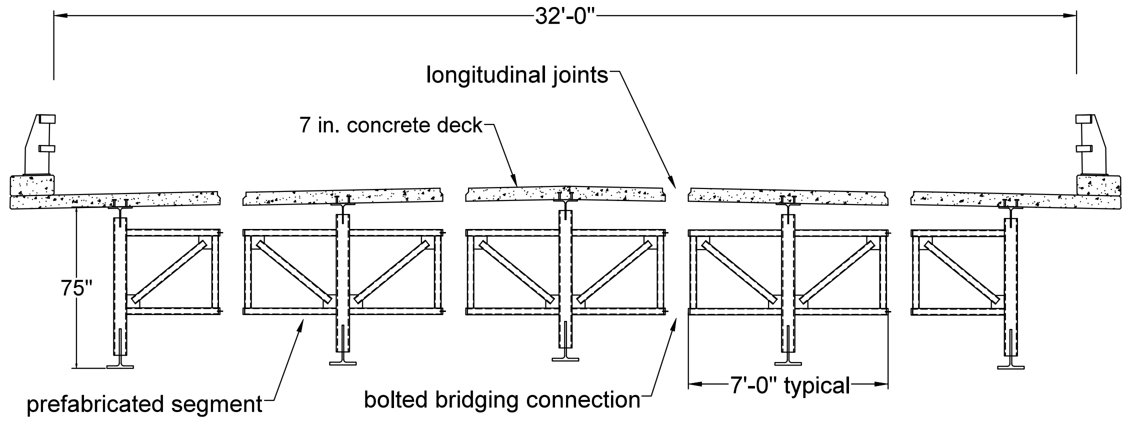
1.1. Description of Proposed Prefabricated Bridge System

Preliminary designs were completed by Allied Steel for three different prefabricated steel truss/integral concrete deck bridge systems intended 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). The prefabricated elements for these systems consist of a single truss 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.

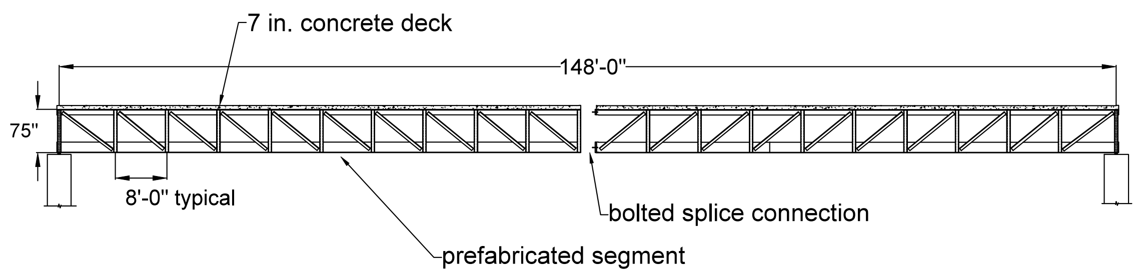
Table 1: Prototype Bridge Systems

Option	**Span**	**Deck Thickness**	**Top Chord Member**	**Bottom Chord Member**	**Vertical Member**	**Diagonal Member**	**Steel Weight**
1	148 ft.	7 in.	WT12x38	WT18x97 / WT20x147	HSS6x6 / HSS5x5	LL5x3 / LL6x3 / LL7x4	29,100 lbs.
2	148 ft.	7 in.	WT12x38	WT18x97 / WT20x147	W8x15-31	W6x16 / W8x21-28	28,000 lbs.
3	108 ft.	8-1/4 in.	PL3/4x12	PL1-3/4x12 / PL2x6	W8x18-24	PL1x6	18,080 lbs.

In all cases, the vertical and diagonal truss members are welded to the top and bottom chords of the steel truss. 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 is shown in Figure 1.



(a) Cross-Section



(b) Elevation

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

1.2. Summary of Work

The literature review 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, and 4) full-scale experimental studies.

The objectives of the analytical evaluation were to 1) identify any impacts on the projected service life of the prototype truss 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.

A bolted/welded prefabricated steel truss bridge was investigated as an alternative to the welded truss bridge. Use of bolted connections at selected locations in the trusses 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. Work completed includes 1) development of a 3D finite element model used to more accurately calculate the distribution of lane and truck loads to the truss members, 2) determination of member sizes and connection geometry to satisfy AASHTO Strength I, Fatigue I, and Service II load combinations for both conventional and accelerated construction methods, and 3) estimation of potential cost savings related to materials, fabrication, and construction of these alternatives compared with the 205 ft. Swan River plate girders.

2. Literature Review

In reviewing prefabricated bridge systems with a view toward investigating their deployment, four subject areas of interest were identified and researched in the literature: 1) modular steel systems, 2) concrete decks, 3) welded connections subjected to fatigue, and 4) full-scale experimental studies. 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 22 sources (journal publications, trade journal articles, and state, federal, and private reports) as the most relevant to the proposed prefabricated steel truss bridge.

2.1. 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.

2.1.1. 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.

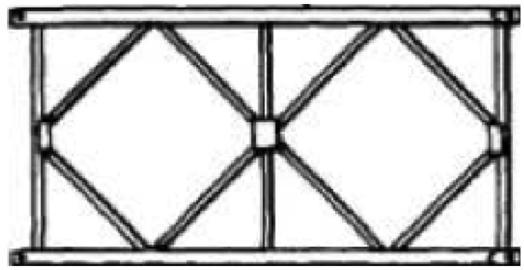


Figure 2: Detail of a Bailey Bridge Panel (Klaiber and Wipf 2004)

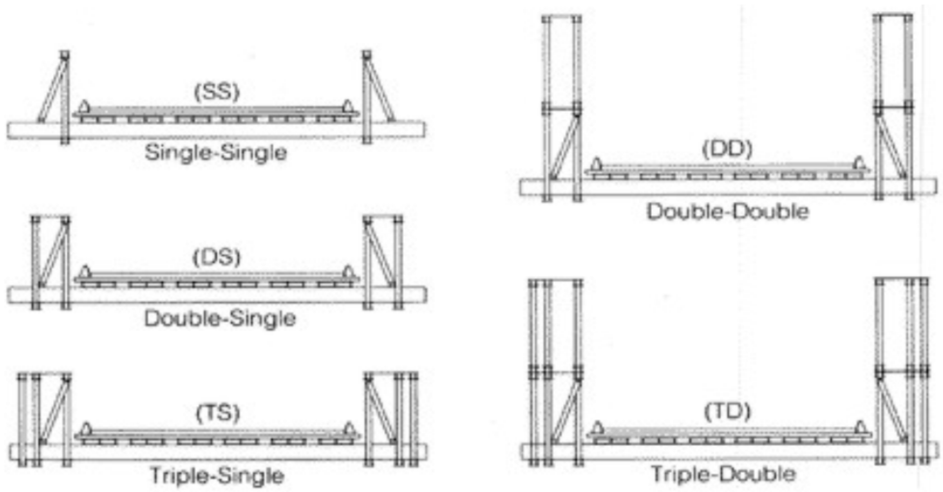


Figure 3: Bailey Configurations (SDR Engineering Consultants 2005)

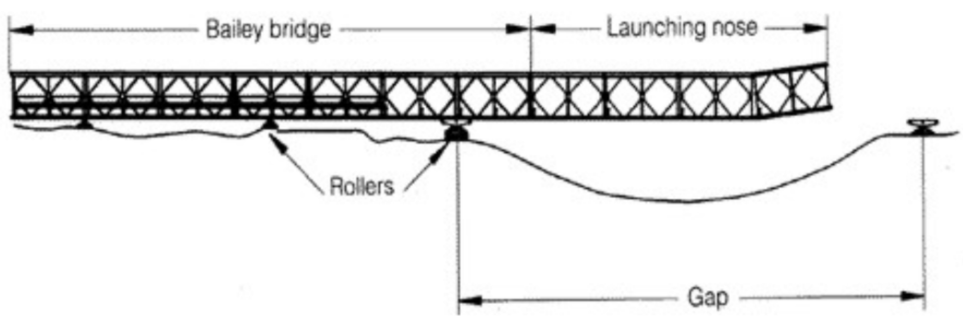


Figure 4: Bailey Bridge Launching Diagram (SDR Engineering Consultants 2005)

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.

The Acrow Panel Bridge is made up of three different stock items that are assembled to form the desired configuration. A photo of an 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 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.

Figure 7: Crosier Bottom Crossing (McConahy 2004)

2.1.2. 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 8: Prefabricated Wide-Flange Beams topped with a Composite Concrete Deck

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 span and width of the prefabricated segments for this specific case was not provided. Keys cast in the overhanging slabs are grouted together with non-shrink grout during construction. A similar joint system was investigated by Au et al. (2008) and is discussed in the following section of this report.

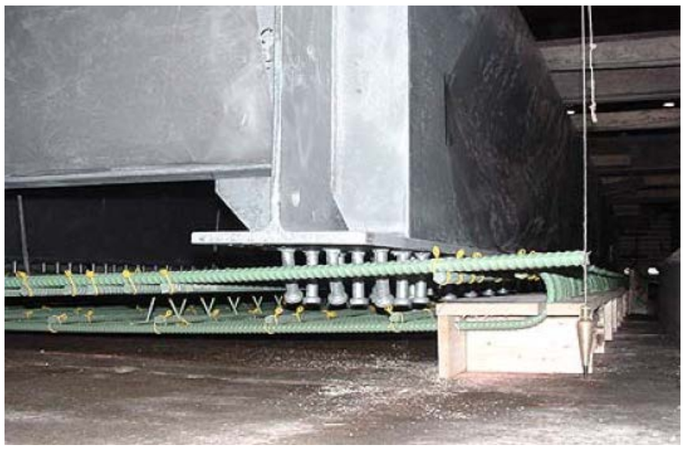


Figure 9: Prefabricated Bridge Units cast Upside-Down (Fort Miller Company 2016)

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. of Longitudinal Segments	Length
Garden State Parkway, NJ	April 2016	4	53 ft.
Route 28, MA	April 2016	4	90 ft.

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

2.1.3. 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.

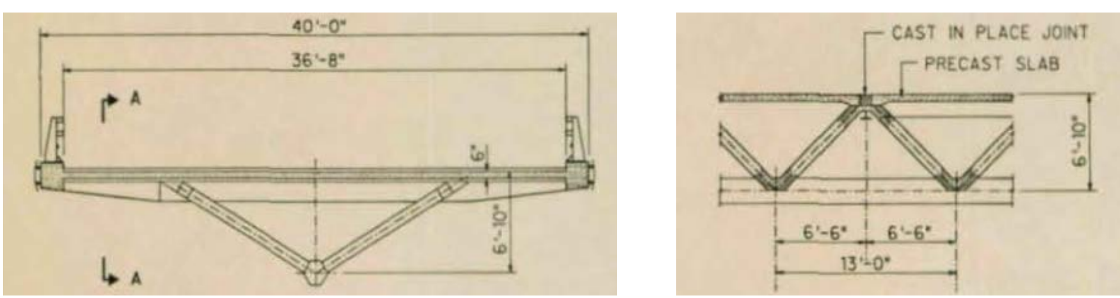


Figure 11: Roize Bridge Cross-Section and Elevation View (Muller 1993)

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 the 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.

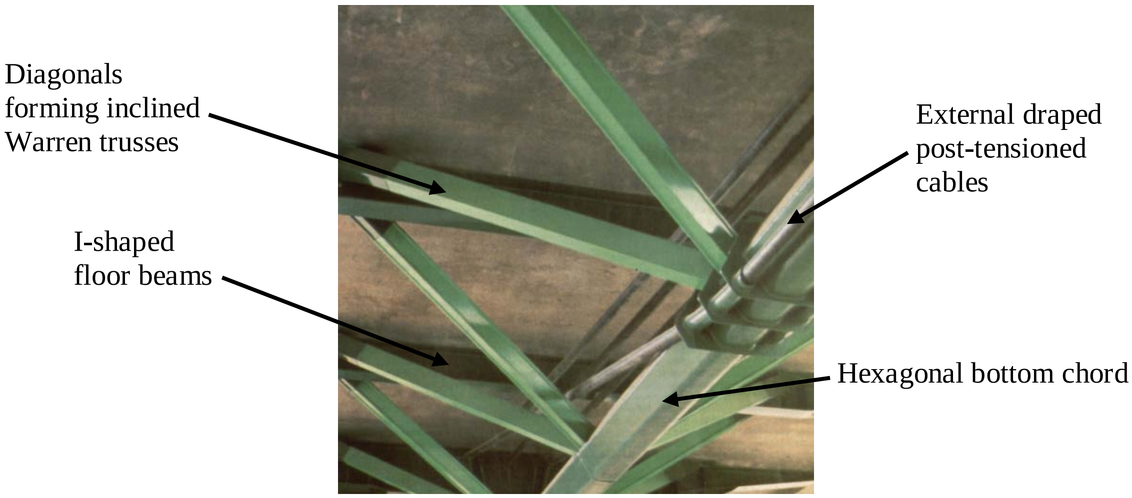


Figure 12: Space Truss Superstructure of the Roize Bridge (Muller 1993)

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).

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. Photos of the completed structure are shown in Figure 14.

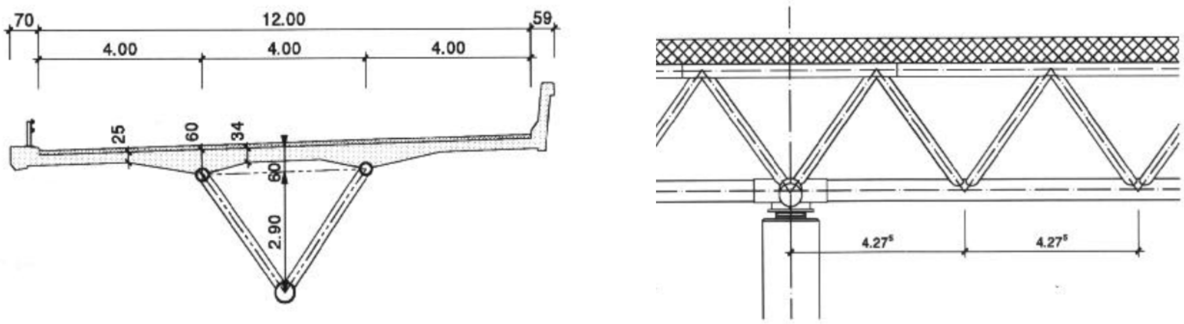


Figure 13: Lully Viaduct Cross-Section and Elevation View, SI Dimensions (Dauner et al. 1998)

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

2.1.4. Modular System Comparison

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 system 3, composite space truss, and system 4, steel girders and concrete deck, which ranked 1st and 3rd, respectively, for the bridge systems considered by SDR. Unlike the proposed system where the bridge is supported by the bottom chord, the under-slung truss (System 5) evaluated by SDR was supported by the top chord and was not as modular as the other bridge types considered.

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	13	70
6	Cold-Formed Steel Plate Box	23	16	22	11	72

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.

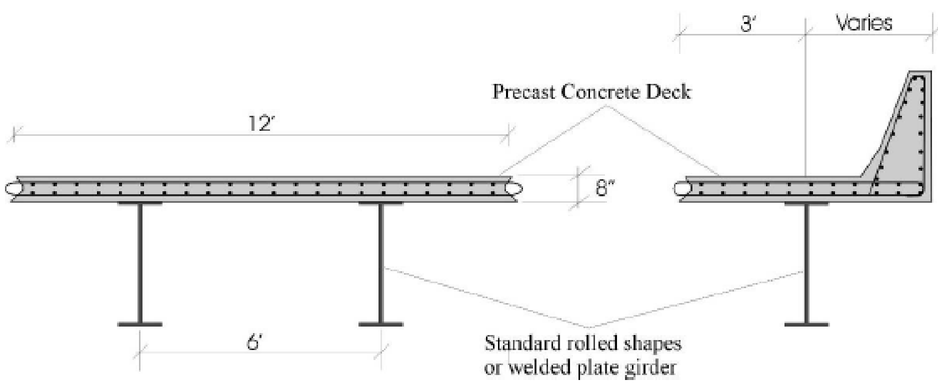


Figure 15: Modular Precast Concrete Bridge Concept (SDR Engineering Consultants 2005)

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 that 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.

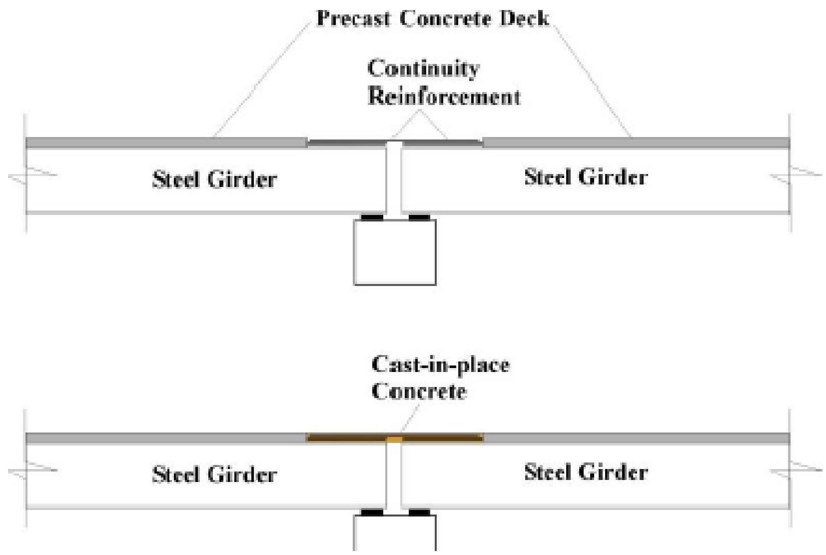


Figure 16: Continuous Precast Modular Bridge Concept (SDR Engineering Consultants 2005)

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.

2.2. 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.

2.2.1. 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 ft. 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.

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

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.

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.

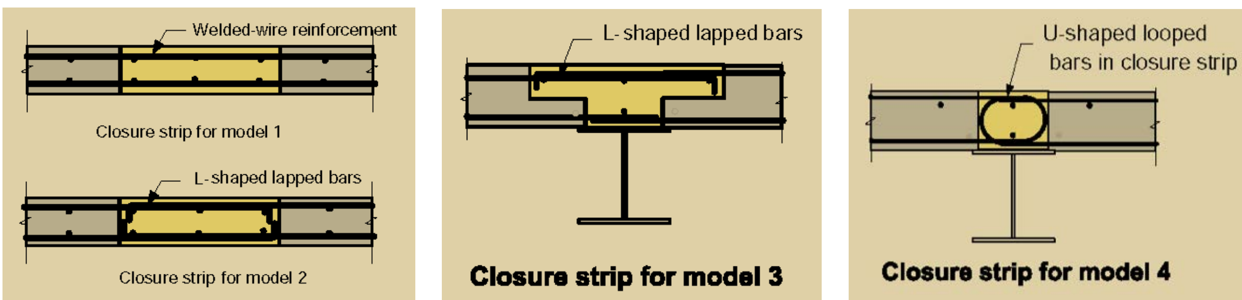


Figure 18: Closure Strip Details for Four Configurations Considered (Au et al. 2008)

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 loading and unloading cycles were completed before the maximum failure load was reached.

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 applications 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 is due in 2017 (Montana Department of Transportation 2012).

2.2.2. 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.

2.2.3. 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 (System No. 4 in Table 3).

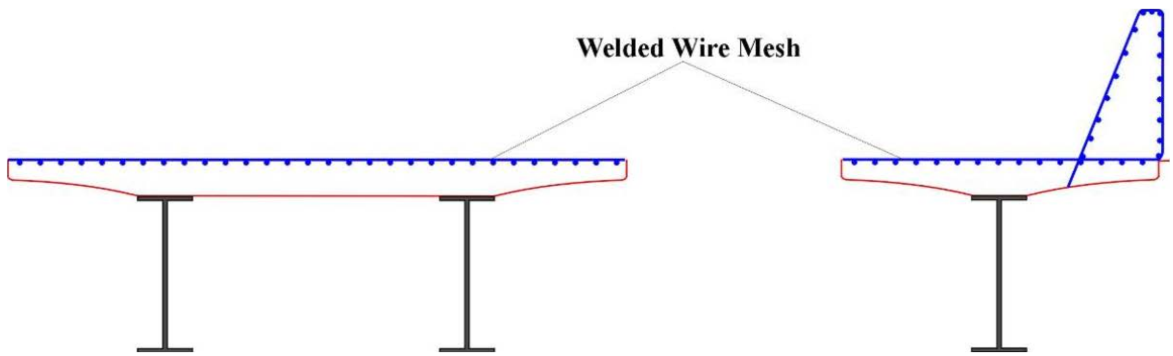


Figure 19: Proposed Cross-Section for a Cast-In-Place Concrete Deck without Formwork (SDR Engineering Consultants 2005)

2.3. 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.

2.3.1. 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 connection fatigue requirement (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.

2.3.2. 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 conclusions of the study noted 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 relationship of 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.

2.4. 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 2) 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 and experimental setup are shown in Figure 22. Lateral buckling was observed in the top chord members adjacent to the central node at a load of 500 kN and 507 kN 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 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.

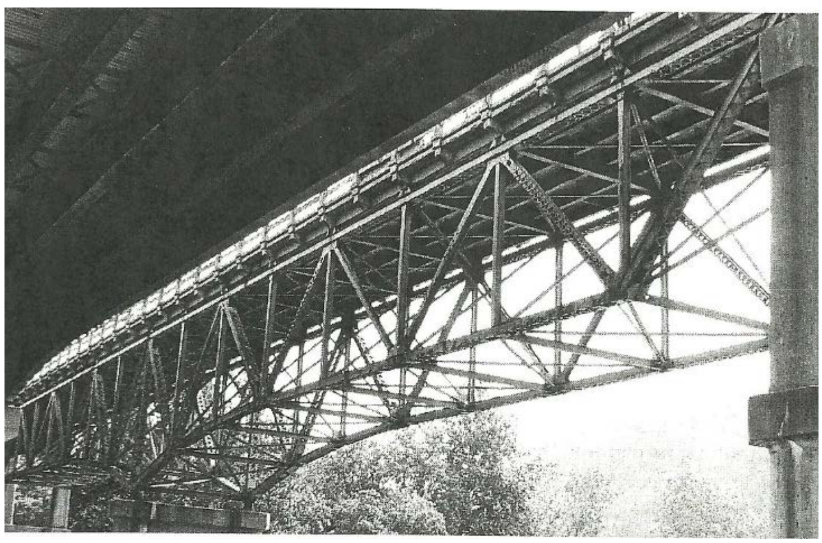


Figure 23: Hillsville Truss (Hickey et al. 2009)

Loaded trucks with known dimensions and weights were parked along the bridge, and strain gauges were strategically placed to record various member strains. 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 moments could be included. 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 in the riveted gusset plate connections of 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 will be included in the analysis of the proposed prefabricated system.

2.5. Summary

The proposed prototype bridge structure consists of a prefabricated welded steel truss 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.

With these observations in mind, the service life, fatigue strength, and joint restraint of the proposed welded steel trusses were included in the following analytical evaluation.

3. Analytical Evaluation

The analytical evaluation was performed to 1) identify any impacts on the projected service life of the prototype truss 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) 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.

3.1. Projected Fatigue Impacts of the Welded Member-to-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 spans 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.

3.1.1. 2D Finite Element Model

A two-dimensional model shown in Figure 24 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. The 7 ft. wide concrete deck was connected to the top chord of the steel truss with link elements at the panel points to generate composite action of the deck and steel truss below. 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 24. 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).

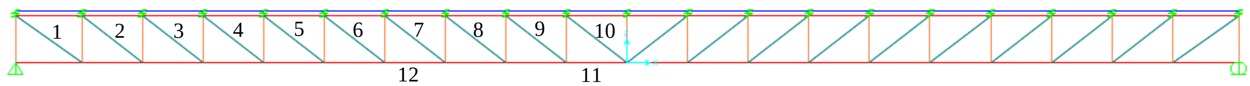


Figure 24: SAP2000 Model with Diagonal and Bottom Chord Tension Member Labels

3.1.2. 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 trusses. The loading diagrams used for an interior truss are shown in Figure 25 and Figure 26. 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.

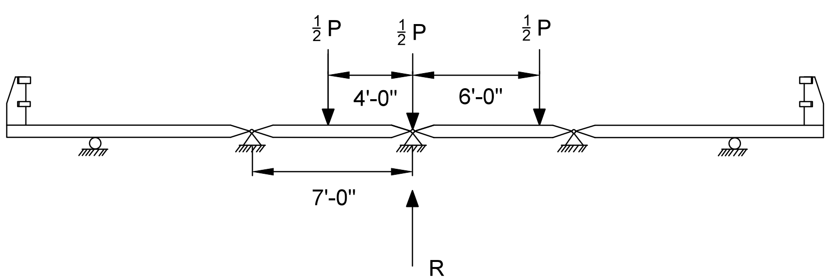


Figure 25: AASHTO Lever Rule Loading Diagram for Strength I Load Combination with Two Lanes Loaded

3.1.3. 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. Considering first the fatigue susceptibility of the basic connection to be used in this truss system, the situations of interest both fall in AASHTO (2014) Detail Category E’. A typical welded connection detail in the proposed steel truss is shown in Figure 27.

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. Illustrative examples of the relevant detail categories for these members from AASHTO Table 6.6.1.2.3 are shown in Figure 28a for the bottom chord member and Figure 28b for the diagonal members.

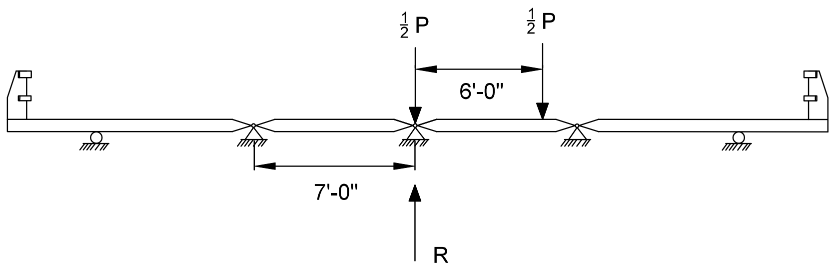


Figure 26: AASHTO Lever Rule Loading Diagram for Fatigue Load Combination with One Lane Loaded

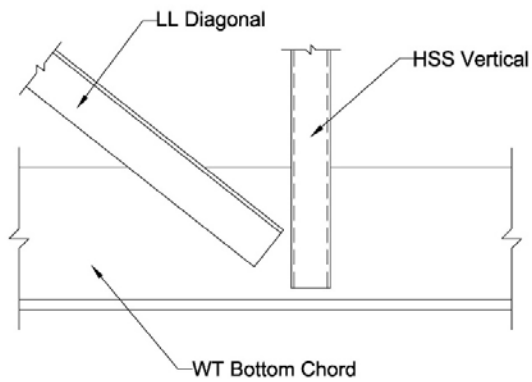
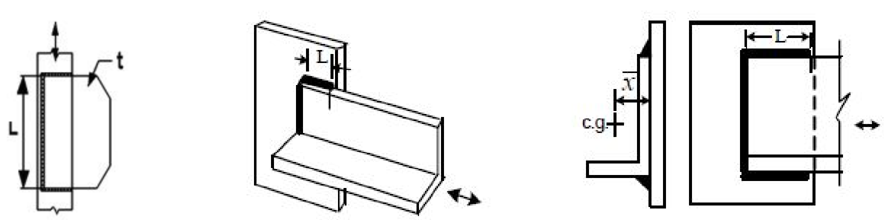


Figure 27: Proposed Connection Detail



(a) (b)

Figure 28: Connection Examples of Detail Category E' for Longitudinally Loaded Welded Attachments (AASHTO, 2014 Table 6.6.1.2.3-1 Description 7.1-7.2)

The cross-section geometry of the truss members and the required weld lengths result in a Detail Category E' designation for both the bottom chord and diagonal members shown in Figure 27. 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. The stress threshold for an infinite-life design for Detail Category E' is 2.6 ksi using the Fatigue I load combination (AASHTO Table 6.6.1.2.3-1).

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/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 was determined (AASHTO Section 3.6.1.4), which is approximately 1.8 times higher than the infinite design life stress threshold of 2.6 ksi determined above.

3.1.4. Calculated Stresses Versus 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 4.

Table 4: 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

3.1.4.1. Strength I Load Combination

The Strength I load combination results for the diagonal members are shown in Figure 29. Member labels on the x-axis of this figure correspond with the member numbers shown in Figure 24 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. Four diagonal members may require slightly larger cross-sections.

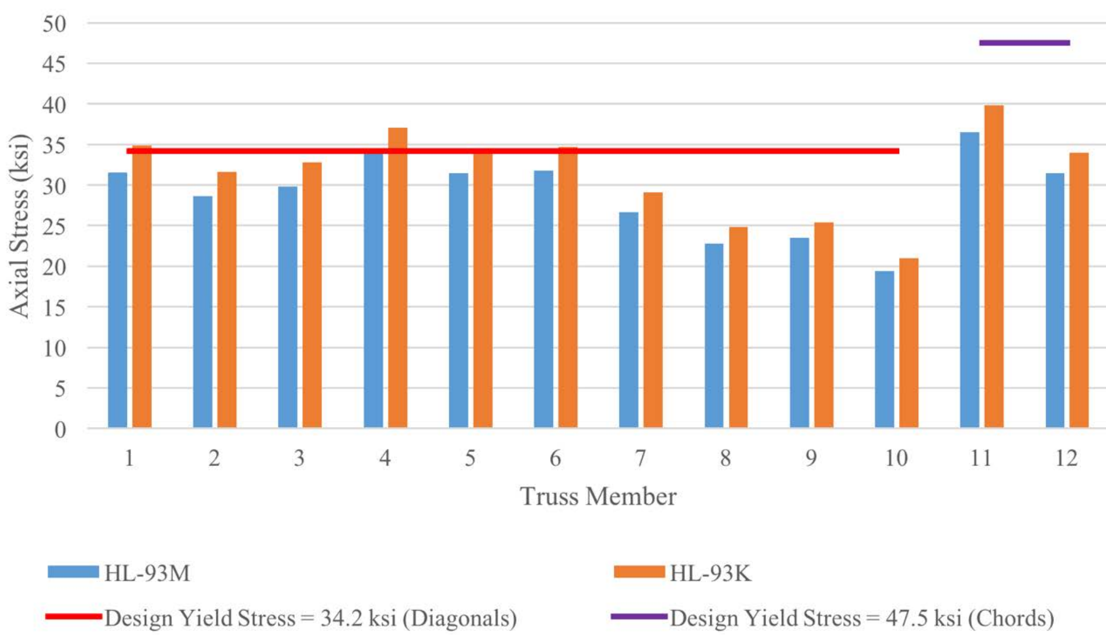


Figure 29: Axial Stress in the Diagonal and Bottom Chord Members for the Strength I Load Combination

3.1.4.2. Fatigue 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. The effective stresses calculated with the Fatigue I load combination for the diagonal and bottom chord members are shown in Figure 30. This preliminary analysis suggests that diagonal and bottom chord members are inadequate for an infinite-life design using the Fatigue I load combination threshold of 2.6 ksi for Detail Category E'.

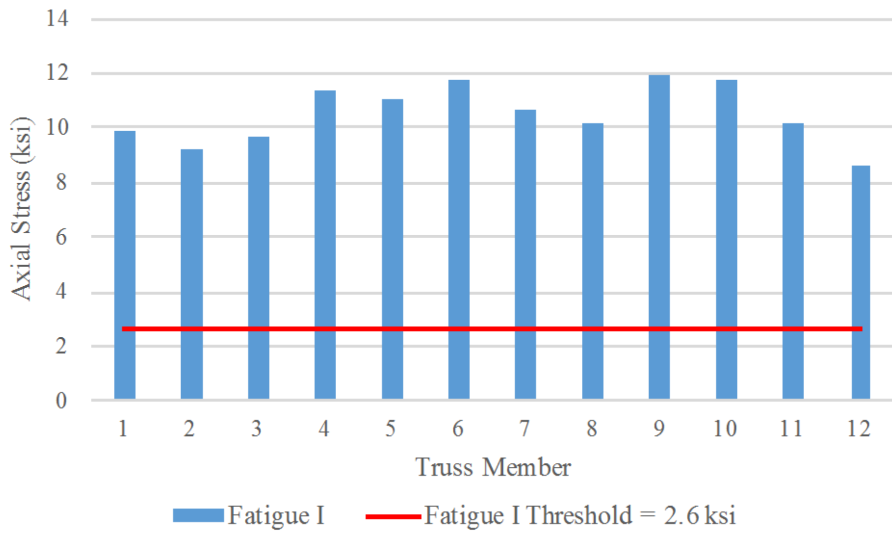


Figure 30: Axial Stress in the Diagonal and Bottom Chord Members for the Fatigue I Load Combination

3.1.4.3. Fatigue II Load Combination

Calculated effective stresses using the Fatigue II load combination for the diagonal and bottom chord members are shown in Figure 31. The results suggest that 9 of the 10 diagonals and one bottom chord member are not adequate for a finite-life design of 75-years using the Fatigue II load combination threshold of 4.6 ksi.

3.2. Materials and Fabrication Costs

Before further pursuing the prefabricated welded steel truss 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 truss 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 32.

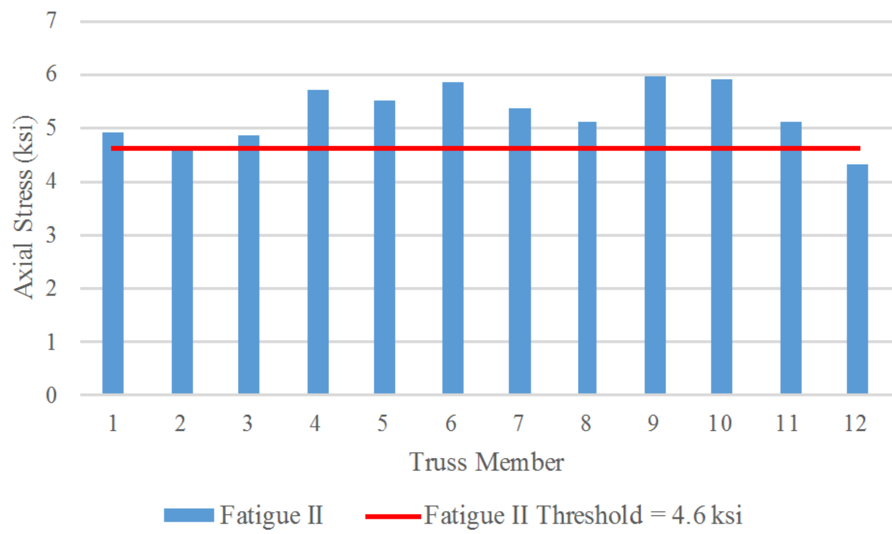


Figure 31: Axial Stress in the Diagonal and Bottom Chord Members for the Fatigue II Load Combination

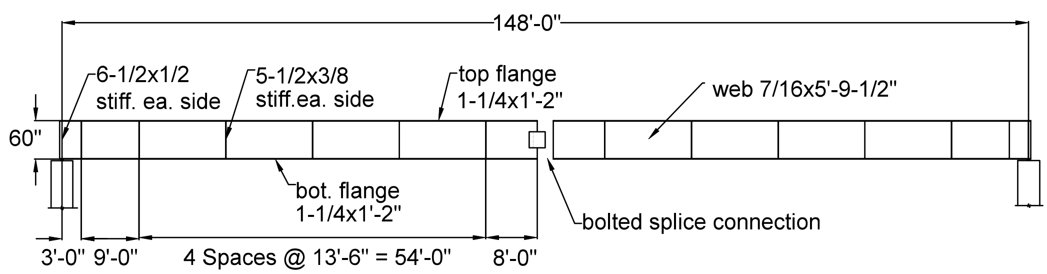


Figure 32: Elevation View of Plate Girder

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 truss configurations. For this reason, Option 3 is not included in the cost comparison described below.

3.2.1. AVEVA

AVEVA provided the most detailed cost estimate for the two truss and 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 truss and girder options provided by AVEVA are summarized in Table 5.

Table 5: AVEVA Price Estimates

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

3.2.2. 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 6.

Table 6: RTI Fabrication Price Estimates

	Option 1	Option 2	Plate Girder
Total Weight	29,100 lbs.	28,800 lbs.	36,560 lbs.
RTI Fabrication	$40,740	$40,320	$51,190

3.2.3. Allied Steel

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

3.2.4. Price 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 can be seen in Table 7.

Table 7: Steel Price Estimates

	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

It is important to recognize the potential variation of the cost estimates shown in Table 7. 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. Despite the potential sources for variation, the prices shown in Table 7 suggest the two steel trusses range from approximately 5% to 20% less than a comparable plate girder.

3.3. Alternative Truss Configurations

Based on further discussion with Allied Steel and AVEVA and the desire to improve the fatigue performance, revisions were made to the proposed truss members and their connections. Allied Steel suggested that a truss 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 33. The stress threshold for the Fatigue I load combination for Detail Category B is 16 ksi and is a significant improvement over the 2.6 ksi threshold for the welded connection with a Detail Category E'.

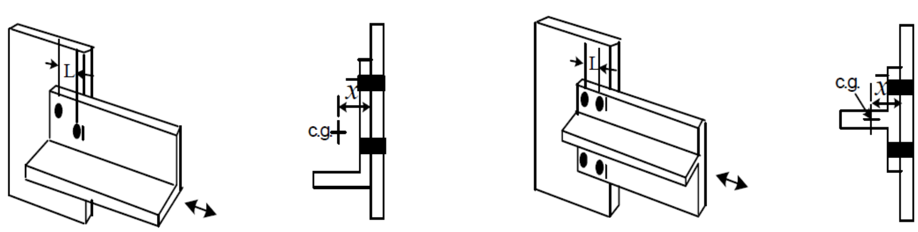


Figure 33: Diagonal Member Connection Examples of Detail Category B for Longitudinally Loaded Bolted Attachments (AASHTO 2014 Table 6.6.1.2.3-1 Description 2.5)

The welded knife-plate connection between the wide-flange vertical member and the web of the top and bottom chord WT-sections is most closely represented by AASHTO (2014) Detail Category C’ shown in Figure 34. The stress threshold for the Fatigue I load combination is 12 ksi.

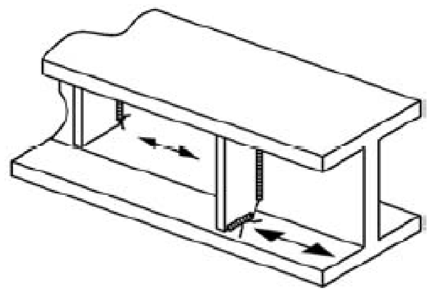


Figure 34: Example of Detail Category C’ for Longitudinally Loaded Bottom Chord with Transverse Welded Attachments (AASHTO, 2014 Table 6.6.1.2.3-1 Description 4.1)

A drawing of a single truss panel showing the wide flange vertical members for this new option is shown in Figure 35.

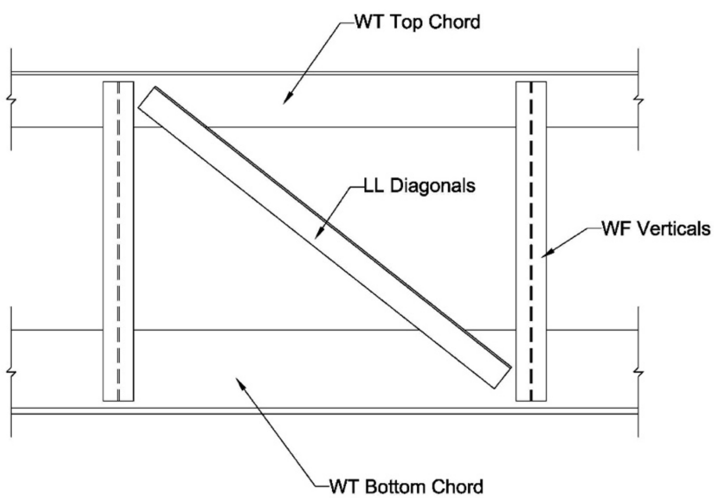


Figure 35: Typical Panel Layout of Option 4

To further explore this new truss configuration (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 truss options and the plate girder are shown in Table 8.

Table 8: Weight Comparison

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

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 truss.

Before continuing with the fatigue analysis for the new truss configuration, three additional bridge spans were analyzed and compared with the plate girder 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 used. A plot of steel weight vs. span length is shown in Figure 36. The difference between the total weight of steel for the two systems increases for larger spans.

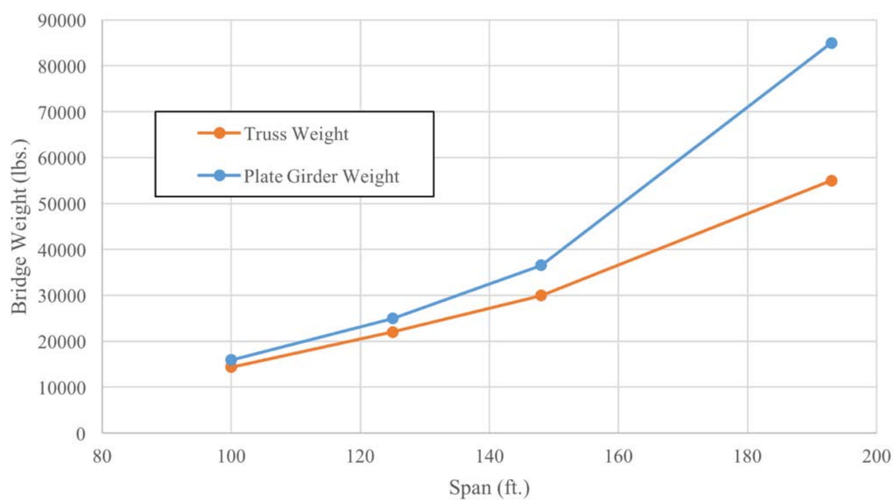


Figure 36: Comparison of Truss and Plate Girder Weight as Span Changes

3.4. 205 ft. Truss Design

A 205 ft. steel truss span was selected for further consideration in this study, as MDT is currently designing a 205 ft. plate girder bridge for the Swan River crossing. To improve the fatigue response of the steel truss, 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 trusses 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 9. The finite element program SAP2000 was again used for the analysis of this new truss system with the same modeling parameters as the 148 ft. model (Section 3.1). An elevation view of the bridge is shown in Figure 37. 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 38. 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 25).

Table 9: 205 ft. Bolted/Welded Steel Truss Properties

Span	Deck Thickness	Top Chord Member	Bottom Chord Member	Vertical Member	Diagonal Member	Steel Weight
205 ft.	8 in.	WT16.5x65	WT20x162 / WT16.5x193.5	W10x39	MC10x33.6 / MC10x25 / MC8x18.7	69,000 lbs.

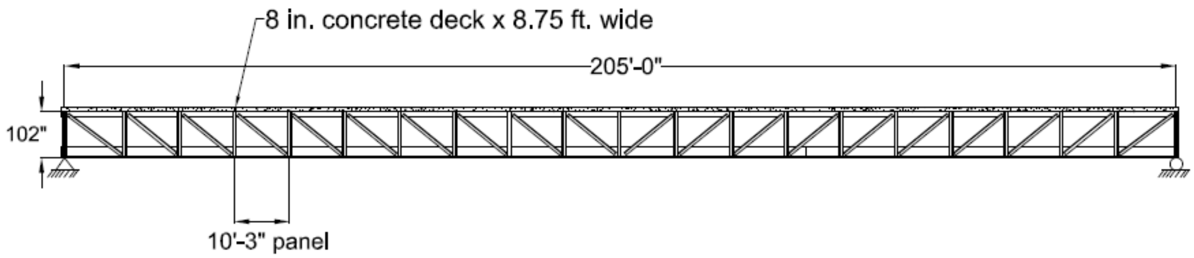


Figure 37: 205 ft. Bolted/Welded Steel Truss Elevation View

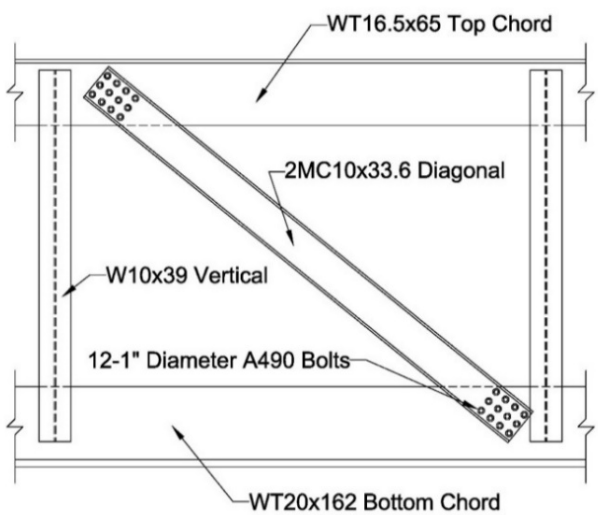


Figure 38: Bolted Connection Detail

Results indicate that the new truss members and bolted connection configuration satisfy strength and fatigue requirements for an infinite-life design. Tensile stresses in the diagonal members and bottom chord members are shown in Figure 39 relative to 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 are shown in Figure 40 relative to the 16 ksi and 12 ksi thresholds for the diagonal and bottom chord tension members using the Fatigue I load combination.

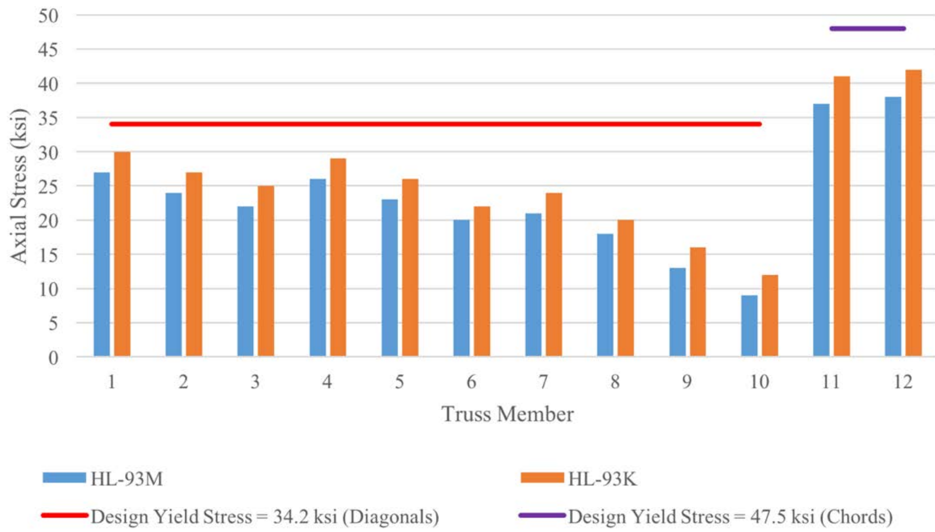


Figure 39: Axial Stress in the Diagonal and Bottom Chord Members with the Bolted Connection for the Strength I Load Combination

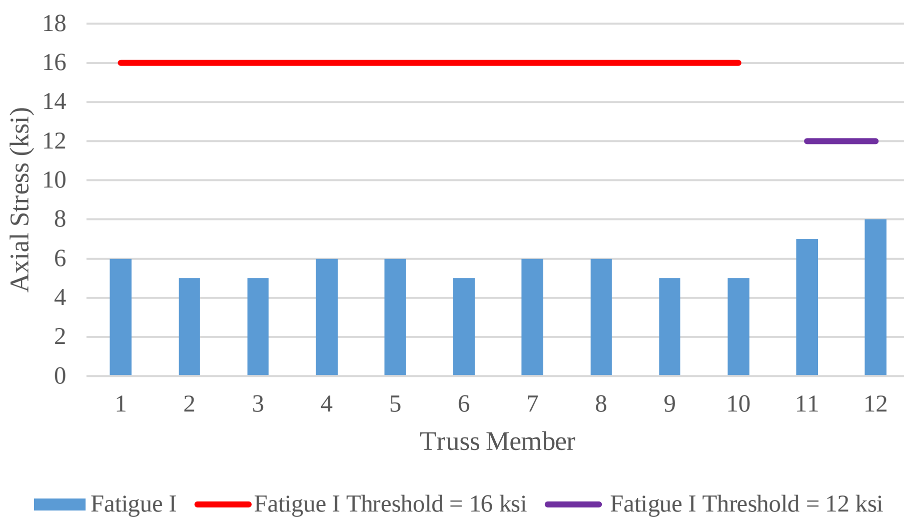


Figure 40: Axial Stress in the Diagonal and Bottom Chord Members with the Bolted Connection for the Fatigue I Load Combination

3.5. Summary

A preliminary analysis of a 148 ft. span prefabricated steel truss 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. truss configurations and a comparable plate girder. The estimates suggest the welded steel truss options cost approximately 5% to 20% less than a comparable plate girder.

Based on discussions with Allied Steel and AVEVA, and based on the projected fatigue performance of the initial truss options, a new truss configuration was identified. The new configuration includes more economical wide flange vertical members and bolted diagonal member connections to improve fatigue performance. The bolted 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'. A preliminary design of a 205 ft. steel truss was compared with a comparable plate girder designed by MDT for the Swan River crossing. Results indicate the bolted/welded steel truss is approximately 24% lighter than the plate girder.

4. Analysis of Results

The preliminary analysis and design of a 205 ft. steel truss bridge using the geometry of the Swan River plate girder bridge indicate the prefabricated truss alternative with bolted connections between the diagonal members and chords satisfies AASHTO fatigue requirements for an infinite life design. To further investigate the potential material and fabrication cost savings for the lighter truss system, a three-dimensional finite element model was created to more accurately estimate the distribution of multiple lane and axle loads to the trusses in the system and attendant individual truss members. 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 truss 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 truss configurations, and selected connection details determined. Updated materials and fabrication costs were subsequently obtained from Allied Steel, AVEVA, and RTI, Inc. Potential construction and erection advantages for the two truss configurations are compared with the planned plate girder construction for the Swan River project.

4.1. Refined Analysis Approach

SAP2000 was used to create a 3D finite element model of the Swan River Bridge that consisted of a 205 ft. span and a roadway width of 40 ft (see Figure 41). Grade 50 steel was used for the WT and wide flange cross sections, and Grade 36 steel was used for the diagonal channel members. The 8 in. concrete deck was modeled with approximately 1 ft. by ft. shell elements. Concrete strength was 4000 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 uncracked cross-section in compression). Similar to the 2D model used in the preliminary analysis, the bottom chord, diagonal and vertical members were pin-connected at the panel points and a continuous member was used for the top chord.

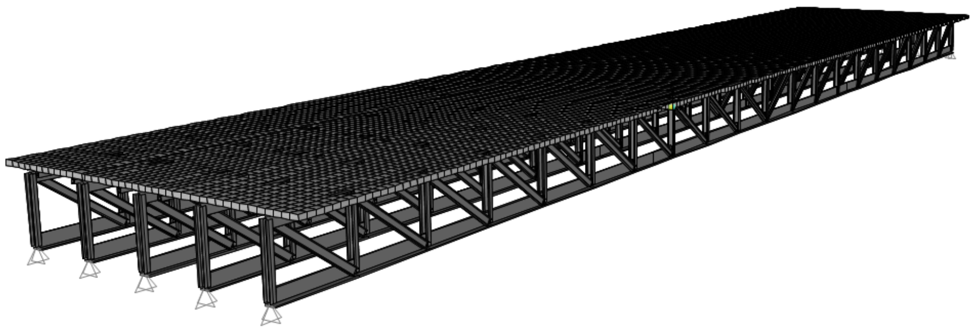


Figure 41: 3D Finite Element Model

4.1.1. Loading

The clear roadway width of 40 ft. for the proposed steel truss 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 HL93 design truck are shown in Figure 42. The HL-93 truck loads 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.

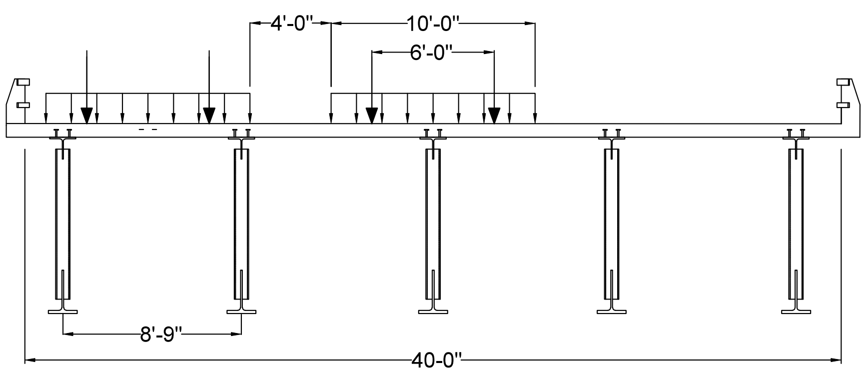


Figure 42: Location of Uniform Lane Loads and Concentrated Design Truck Loads for a Two-Lane Condition

4.1.2. Load Distribution Analysis

A 3D model calculates controlling forces in the individual truss members 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 42, 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 trusses 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 comparison of the maximum tension and compression forces are shown in Table 10. The ratios indicate the 3D model results in reduced vertical, diagonal, and bottom chord forces of approximately 50%.

Table 10: 2D Distribution Factor Versus 3D Finite Element Model Results for the Proposed Truss Geometry using SAP2000

Loading	Maximum Tension (+) / Compression (-) Forces (kips)					
	2D Model			3D Model		
	Vertical	Diagonal	Bot. Chord	Vertical	Diagonal	Bot. Chord
Lane	-66	104	431	-37	56	273
Truck	-66	107	437	-36	52	172
Lane + 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 11. 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 were considered. Referring to Tables 10 and 11, the 3D / 2D ratios for the steel truss 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.

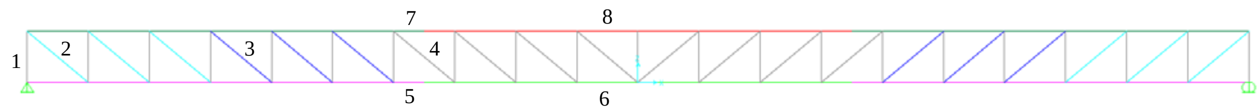
Moving forward in these analyses, the decision was made to proceed with a distribution factor of 0.75 for the proposed steel truss system. This value is generally centered between the distribution factor of 0.93 calculated for the trusses using the relatively simple and typically conservative lever rule, and the much smaller value indicated by the more complex 3D finite element analysis (which did only consider a single load case). Further, this value of 0.75 is generally centered between the distribution factors determined for the Swan River plate girder system (0.67 for moment and 0.87 for shear) calculated using the AASHTO distribution factor equations in Section 4.6.2.2. Thus, the truss system design subsequently generated below is directly comparable with the existing plate girder design.

Table 11: 2D Distribution Factor Versus 3D Finite Element Model Results for the Swan River Plate Girder using AASHTOWare

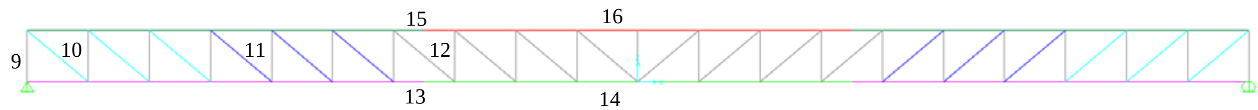
Loading	**Mid-span Bending Moment (kip-ft.)**	
	2D Model	**3D Model**
Lane	3364	1716
Truck	4537	2428
Lane + Truck	7901	4144
	3D / 2D Ratio	0.52

4.1.3. Results

The 2D SAP2000 model with a distribution factor of 0.75 was used to calculate truss member forces for two truss 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 designed are shown in Figure 43. 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 (truss plus deck) in service will 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 truss and deck is carried just by the steel truss, with due consideration of all incidental loads that have to be supported by the trusses during deck construction. The top chord design for conventional construction was controlled by the depth required for the bolted diagonal connection.



(a) Truss 1



(b) Truss 2

Figure 43: Location and Designation of Truss Members Designed for (a) Truss 1 using Conventional Construction and (b) Truss 2 using Accelerated Construction

Calculated service-level forces from SAP2000 are shown in Table 12 and Table 13 for Truss 1 and Truss 2, respectively. Factored load combinations used for member and connection design are shown in Table 14 and Table 15, again for Truss 1 and Truss 2, respectively. Referring to Tables 12 and 13, as would be expected, the live load demands in individual truss members (with the exception of the construction live load demands) in the two truss configurations are effectively identical, as these demands are carried in both systems by the identical composite steel truss/concrete deck system. The truss member forces are different in the two configurations for the demands from the dead load of the truss and deck, as this demand is carried by just the truss in conventional construction (Truss 1) scenario. The member's forces are approximately 10 percent lower in the Truss 2 compared to the Truss 1 scenario. Correspondingly, and as is seen in Tables 14 and 15, in load cases dominated by dead load demands (i.e., Strength I and Service II), design forces are similarly smaller in the Truss 2 compared to the Truss 1 scenario. Selected member sizes and the total steel weight for the two truss configurations are shown for Truss 1 and Truss 2 in Table 16 and Table 17, respectively. The only difference between the member designs for the two trusses 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 truss, 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 truck load with the design lane load (AASHTO 3.6.1.3.2) for both configurations. Final member sizes and the total steel weight for the two truss configurations are shown for Truss 1 and Truss 2 in Table 16 and Table 17, respectively.

Table 12: 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	18	27	97	73	100	95
3	129	14	18	65	60	82	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 13: 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 14: Factored Load Combinations Considered for Truss 1

Member Number	Axial Tension (+) / Compression (-) (kips)		
	Strength I	Service II	Fatigue I
1	-407	-308	-85
2	611	463	123
3	436	330	99
4	272	205	78
5	2066	1563	408
6	2431	1839	471
7	-2234	-1690	-440
8	-2457	-1859	-475

Table 15: Factored Load Combinations Considered for Truss 2

Member Number	Axial Tension (+) / Compression (-) (kips)		
	Strength I	**Service II**	**Fatigue I**
9	-346	-264	-85
10	522	400	125
11	378	289	102
12	243	185	80
13	1761	1350	411
14	2071	1588	475
15	-1904	-1460	-443
16	-2096	-1607	-477

Table 16: 205 ft. Bolted/Welded Truss 1 Properties

Span	Deck Thickness	Top Chord Member	Bottom Chord Member	Vertical Member	Diagonal Member	Steel Weight
205 ft.	8 in.	WT18x116 / WT18x128	WT20x162 / WT18x181	W10x39	MC10x28.5 / MC10x22 / MC8x18.7	80 kips

Table 17: 205 ft. Bolted/Welded Truss 2 Properties

Span	Deck Thickness	Top Chord Member	Bottom Chord Member	Vertical Member	Diagonal Member	Steel Weight
205 ft.	8 in.	WT16.5x65	WT20x162 / WT18x181	W10x39	MC10x28.5 / MC10x22 / MC8x18.7	68 kips

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

4.2. Connection Design

Using the factored loads shown in Table 14 and Table 15 and the refined member sizes shown in Table 16 and Table 17, connection designs were completed at the joints of three different truss panels (see Figure 44). 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 45 through Figure 47.

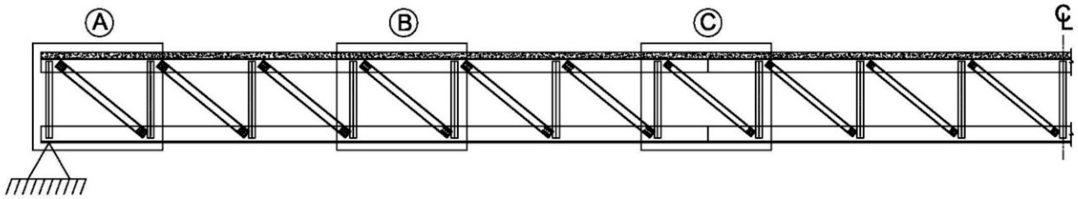


Figure 44: Connection Detail Locations

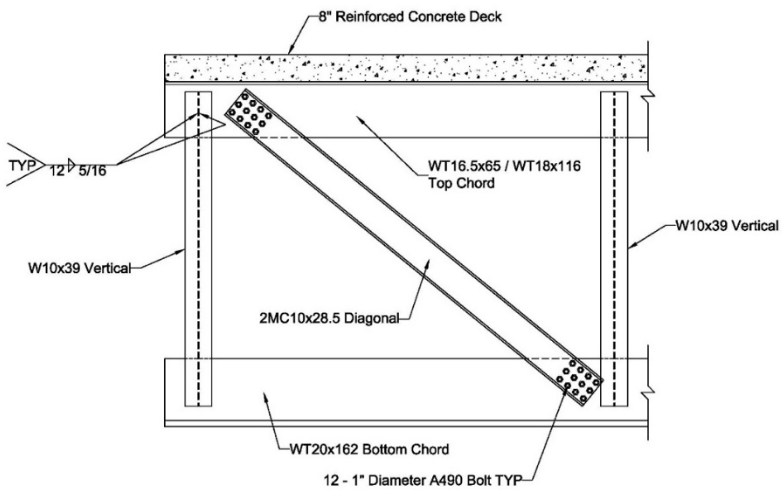


Figure 45: Connection Detail A (12-bolt connection)

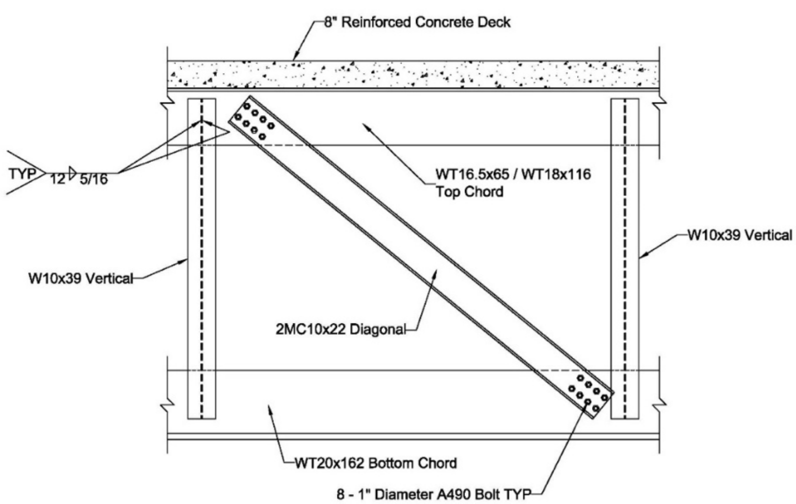


Figure 46: Connection Detail B (8-bolt connection)

4.3. Splice Locations

Based on shipping regulations and construction considerations related to member weight and length, two different splice locations are proposed for this welded/bolted steel truss bridge. A single splice at the truss mid-span was selected for a conventional concrete deck cast after erection of the steel trusses (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 48.

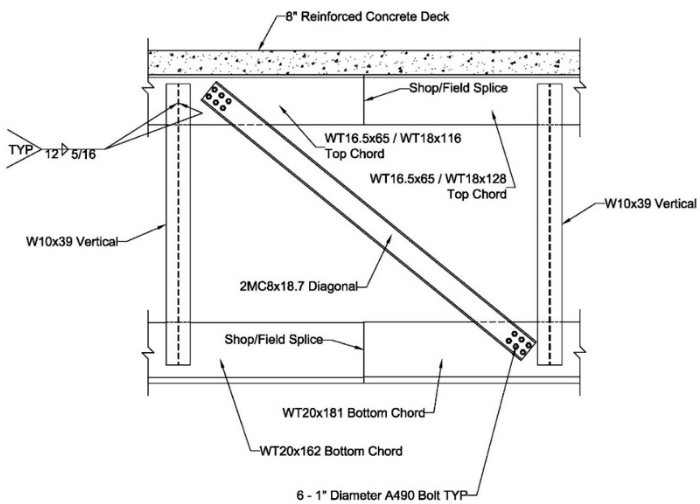
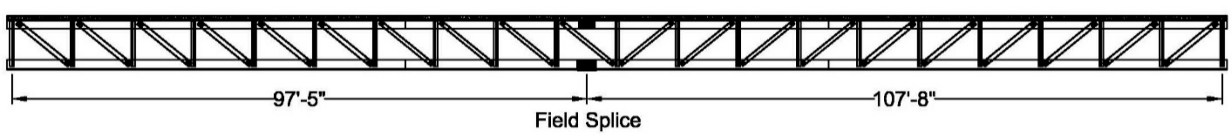
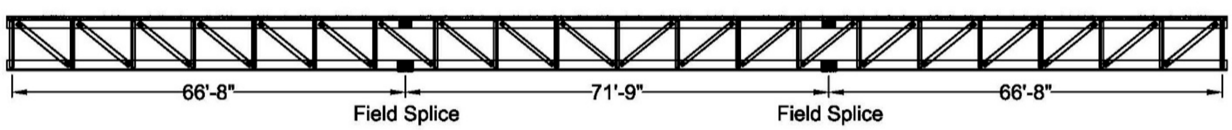


Figure 47: Connection Detail C (6-bolt connection)



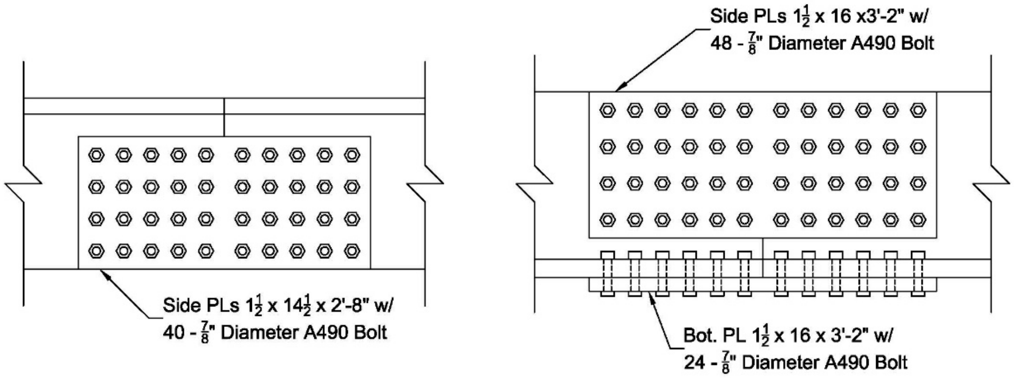
(a) Truss 1



(b) Truss 2

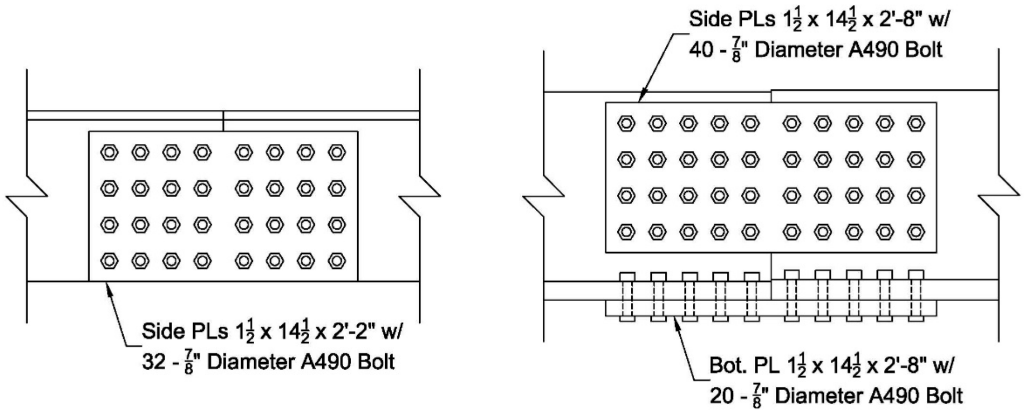
Figure 48: Proposed Truss Elevation with (a) Single-Splice and (b) Two-Splice Condition

Details for the two splice configurations are shown in Figure 49 and Figure 50. 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 yield, and rupture).



(a) Top Chord (b) Bottom Chord

Figure 49: Splice Connection Details for the Single-Splice in Truss 1



(a) Top Chord (b) Bottom Chord

Figure 50: Splice Connection Details for the Two-Splices in Truss 2

4.4. Cost and Other Considerations

A second cost comparison was made for the materials and fabrication of the refined bolted/welded steel truss to assess the impact of the bolted connections and the refined member design for the 205 ft. span. The impact of the two splice configurations, and of the member weights and lengths in the two systems was assessed relative to the Swan River plate girders. General advantages and disadvantages of the bridge decks used for conventional and accelerated construction were evaluated with input from Sletten Construction (Great Falls, MT) and Dick Anderson Construction (Missoula, MT), two companies active in bridge construction in Montana.

4.4.1. Materials and Fabrication Costs

Estimated prices for a bolted/welded steel truss and a plate girder for the Swan River crossing, obtained from the same sources used previously, are shown in Table 18. Costs of the splice connections were not included in the estimates.

Table 18: Final Steel Price Estimates

	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

The variation in estimates shown in Table 18 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 fabrication rates for fabricators located across the country. The estimates from RTI were based on an approximate cost of $1.40/lb. of steel and was the same value used for the cost estimate of the all-welded steel truss discussed above. Because the three cost estimates have included different assumptions in their labor, materials, and fabrication process, an average value was selected to represent the potential cost savings for the two steel truss alternatives. The average values shown in Table 18 result in an estimated materials and fabrication cost savings for Truss 1 and Truss 2 of 10% and 26%, respectively.

Allied Steel indicated that the bolted connections between the diagonal and bottom chords 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 truss 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.

4.4.2. Shipping Considerations

The structural elements being considered for this 205 ft. 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 19.

Table 19: Shipping Guidelines for Montana

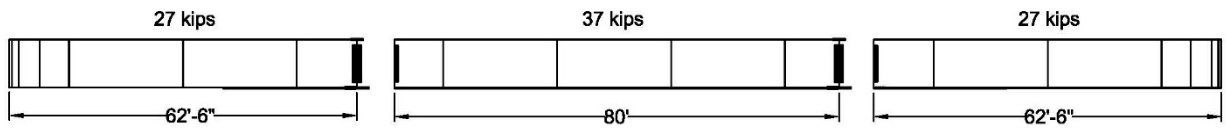
Gross Legal Load	Up to 120,000 lbs., depending on trailer/axle combination
Flag Vehicle Requirements	One flag vehicle for loads > 120 ft. on interstate One flag vehicle for loads > 110 ft. on non-interstate
Permit Requirements	Lengths over 75 ft.

The weights of the steel trusses and plate girders for the single and two-splice configuration are shown in Table 20. An elevation view with the weight of each splice section for the plate girder and both truss alternatives are shown in Figure 51 with the weight of the concrete deck being included in the total weight of each splice section for Truss 2.

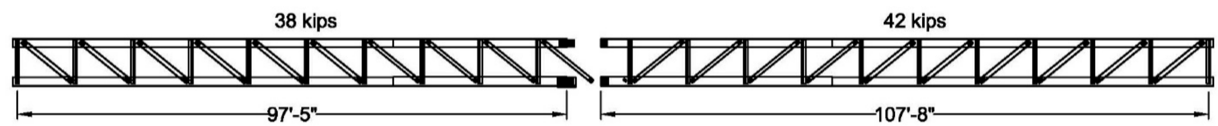
Table 20: Length and Weight of Plate Girder and Truss Construction Alternatives

	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

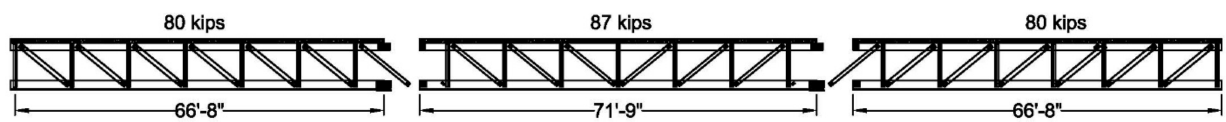
For the 205 ft. bridge span under consideration, True North Steel indicated a preference to ship steel trusses with a single splice configuration. The maximum member length for this condition is approximately 108 ft. (Figure 48) and would require a permit (Table 19). The bare-steel weight of 40 kips would enable up to 3 trusses to be delivered on a single truck without exceeding the gross legal load. The two-splice steel truss with a cast-in-place deck has a length of approximately 71 ft. and a total weight of 85 kips. A single truss with concrete deck could be shipped without exceeding legal load requirements or requiring a permit.



(a) Plate Girder



(b) Truss 1



(c) Truss 2

Figure 51: Weight of each Splice Section for the (a) Plate Girder, (b) Truss 1 and (c) Truss 2

4.4.3. Erection

Potential erection issues were also considered with the truss and 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 truss or plate girders 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 trusses 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. Decked bulb tee systems are capable of spanning up to 160 ft, however 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 truss 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 result 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).

4.5. Summary

A 3D finite element model was created to more accurately distribute the loads to the bolted and welded steel trusses and associated truss members in the 205 ft. Swan River crossings being considered in this analysis. Based on further consideration of the load distribution to the individual trusses in the bridge system proposed in this study, including more refined 3D finite element analyses of this system, the decision was made to move forward with a distribution factor of 0.75. The factor is also 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 truss erection. The steel weight of the truss increased by 18% using the larger top chord members.

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.

5. Summary and Conclusions

Steel truss bridges are an efficient and aesthetically pleasing option for highway crossings. Their lightweight compared with plate girder systems make them a desirable alternative for both material savings and constructability. A prototype 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 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/integral concrete deck bridge systems intended 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. truss and a comparable plate girder suggest a welded steel truss would cost approximately 5% to 20% less than a comparable plate girder. Based on discussions with steel fabricators and the projected fatigue performance of the welded connections, a new truss configuration was designed with more economical wide flange vertical members and bolted diagonal member connections.

A 3D finite element model of the new truss configuration was created to more accurately distribute the loads to bolted and welded trusses and their attendant members using the geometry of the 205 ft. Swan River crossing. Conventional and accelerated construction scenarios were considered in the design of the truss members, connections, and splices. The conventional construction scenario assumed a single splice at mid-span with a concrete deck cast after the truss was erected. For the accelerated construction scenario, the assumption was made that the truss elements with integral concrete deck would bridge the span in three segments (resulting in two splices). The refined truss design and input from fabricating and construction professionals was used to assess the potential of a 205 ft. bolted/welded steel truss bridge constructed using conventional or accelerated methods. The final truss designs were compared with an equivalent plate girder design.

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

- The bolted member end 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 using geometry from the plate girder bridge over the Swan River reduced the loads to the truss members by approximately 50% compared with a 2D model using a distribution factor of 1.0. For the bridge geometry and loading considered, a distribution factor of 0.75 was selected as a representative value between the conservative lever rule and more sophisticated 3D analysis.
- 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 truss using the larger top chord member increased by 18% (80k for conventional construction, 68k for accelerated (precast deck)).
- 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. Materials and fabrication prices suggest a reduction in cost of up to 10% and 26% for the two construction alternatives, respectively.
- 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 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.

5.1 Implementation Recommendations

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

- Discuss potential bridge crossing sites and geometries with steel fabricators and local contractors to receive more specific suggestions for successfully implementing a steel truss bridge system built using conventional or accelerated construction methods.
- Evaluate the joint and concrete deck performance of the Maxwell Coulee bridge that utilized a rolled wide-flange section with an integral concrete deck.
- Investigate alternative contracting methods for a steel truss bridge constructed with an integral concrete deck. The Construction Manager/General Contractor method could provide a more efficient and economical delivery.
- Complete a final design of a steel truss for a selected bridge crossing with input from erector, fabricator, and Maxwell Coulee observations.
- Implement a monitoring and evaluation program, including instrumentation and remote data acquisition, for the constructed steel truss bridge.

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