A MODULAR HYBRID STEEL TRUSS WITH COMPOSITE DECK FOR ACCELERATED BRIDGE CONSTRUCTION

Presenting Author: Damon Fick, Ph.D., P.E., Assistant Professor, Montana State University, 406-994-6123, damon.fick@montana.edu

Co-Author 1: Tyler Kuehl, F.E., Graduate Research Assistant, Montana State University, 603-630-7639, tylerwkuehl@gmail.com

ABSTRACT

Steel truss bridges are an efficient and aesthetic option for highway crossings. Their lightweight compared with plate girder systems make them a desirable alternative for both material savings and constructability. The documented use of modular prefabricated steel trusses that include an integral cast-in-place deck for permanent bridge replacement projects constructed on an accelerated schedule is limited. Similar prefabricated systems using wide-flange beams or plate girders instead of trusses are more common and have been successfully implemented. The objectives of the current study were to 1) Identify the limitations of welded connections for a modular truss configuration using a prototype bridge span of 148 ft., 2) Investigate a bolted and welded hybrid steel truss alternative to improve the fatigue performance of the truss connections, and 3) Perform a cost comparison and evaluation of potential construction efficiencies for both the hybrid truss with integral concrete deck and plate girder alternatives. Preliminary analysis of the all-welded truss spanning 148 ft. indicates that fatigue stresses for the welded connections exceed threshold values by approximately 2.5 times for an infinite-life design. The hybrid steel truss was designed for a 205 ft span to compare with a recent plate girder bridge project designed by the Montana Department of Transportation. Changing the diagonal truss member connections to a bolted configuration satisfied fatigue threshold stresses for both Fatigue I and Fatigue II load combinations. The hybrid truss steel weight was 28% less than the plate girder and results in estimated materials and fabrication savings of 26%. For the two-splice configuration considered, the more efficient connection between the hybrid truss members enabled fewer bolts and a shorter center span that could contribute to the overall efficiency of constructing a prefabricated steel truss with integral concrete deck.

INTRODUCTION

A prototype bridge structure has been proposed by a steel fabricator in central Montana 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 system proposed consists of a prefabricated welded steel truss topped with an integral 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.

Proposed Prefabricated Bridge System

A preliminary design for an all-welded prefabricated steel truss/integral concrete deck bridge system intended for a 148 ft span in Montana is shown in Figure 1. The prefabricated elements for this system consists of a single truss supporting a 7 ft wide concrete deck cast at the steel fabrication facility. Member shapes for this preliminary design consisted of WT sections for the top and bottom chords, HSS verticals and double angle diagonals. Member sizes selected for the preliminary design are shown in Table 1.

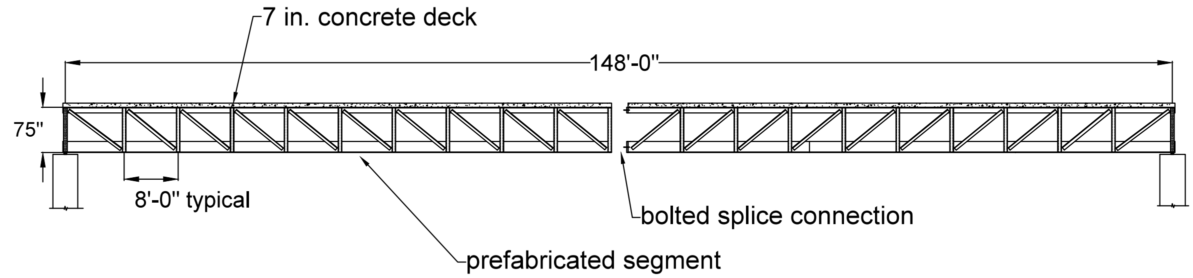


Figure 1: Typical 148 ft. Truss Elevation View

Table 1: 148 ft. Prototype Bridge Truss Members

Span	Deck Thickness	Top Chord Member	Bottom Chord Member	Vertical Member	Diagonal Member
148 ft.	7 in.	WT12x38	WT18x97 / WT20x147	HSS6x6 / HSS5x5	LL5x3 / LL6x3 / LL7x4

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 created continuity between the segments.

PROJECTED FATIGUE IMPACTS OF WELDED CONNECTIONS

The truss configuration (Figure 1) with member sizes shown in Table 1 was used 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, identifying fatigue stress thresholds, and comparing predicted stress levels at various locations in the system as determined from a 2D finite element model with these stress thresholds.

2D Finite Element Model

A two-dimensional finite element model was created using the program SAP2000 [1]. The restraints at the ends of the diagonal and vertical truss members were released to create pinned connections. The top and bottom chords were modeled as both pinned and fixed connections to evaluate the effects of the continuous members. Forces were found to be within 5% and pinned connections were subsequently used for the bottom chord, vertical, and diagonal members. The top chord member was modeled as continuous. To simplify modeling and appropriately generate composite action, the 7 ft. wide concrete deck and top chord elements were coincidently located at the composite neutral axis. The diagonal and bottom chord tension members that were the focus of this preliminary analysis are labeled in Figure 2.

Fatigue Thresholds

The threshold stress a member can experience is significantly affected by the connection configuration and the number of load cycles it will experience over its design life. The fatigue susceptibility of the connection between the diagonal members and top and bottom chords is AASHTO [2] Detail Category E. 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 (member #1 in Figure 2) and at midspan for the bottom chord (member #12 in Figure 2).

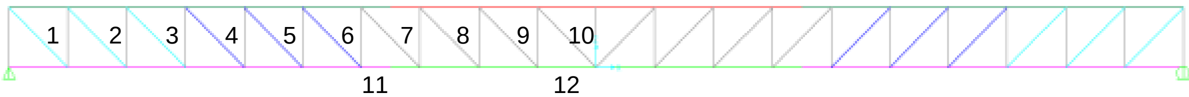


Figure 2: SAP2000 Model with Diagonal and Bottom Chord Tension Member Labels

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 4.5 ksi using the Fatigue I load combination.

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 two lane, 100 ft. span bridge in central Montana 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 could be used. The calculated 75-year design life threshold fatigue stress was 6.4 ksi.

Fatigue Analysis Results

AASHTO's Fatigue I and Fatigue II load combinations were used in the preliminary analysis of the proposed prefabricated bridge. A distribution factor of 0.57 was calculated using the lever rule for both load combinations.

The effective stresses calculated with the Fatigue I load combination for the diagonal and bottom chord members are shown in Figure 3. This preliminary analysis indicates that diagonal and bottom chord members are inadequate for an infinite-life design using the Fatigue I load combination threshold of 4.5 ksi for Detail Category E.

Calculated effective stresses using the Fatigue II load combination for the diagonal and bottom chord members are shown in Figure 4. The results suggest the diagonal and two bottom chords members are adequate for a finite-life design of 75-years using the Fatigue II load combination threshold of 6.4 ksi.

ALTERNATIVE 205 FT. TRUSS WITH BOLTED CONNECTIONS

Based on further discussions with steel fabricators and the desire to improve fatigue performance, revisions were made to the 148 ft. proposed truss members and connections. Because of the relatively higher price per pound of HSS material, wide-flange sections were selected for the vertical members. In addition, a bolted connection between the diagonal member and top and bottom chord was investigated. Welded connections were still used between the vertical wide-flanges and the top and bottom chords. Double-channel sections were selected as the diagonal members to improve the connection geometry for the bolted connections. The span of the alternative truss configuration was increased to 205 ft. to compare with a recent 205 ft. plate girder bridge project designed by MDT. 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. An elevation view of the alternative 205 ft. truss is shown in Figure 5.

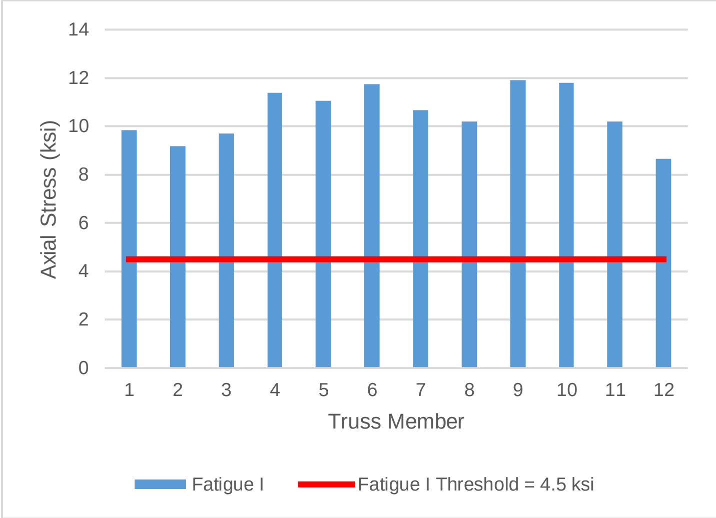


Figure 3: Axial Stress in the Diagonal and Bottom Chord Members with the Welded Connection for the Fatigue I Load Combination

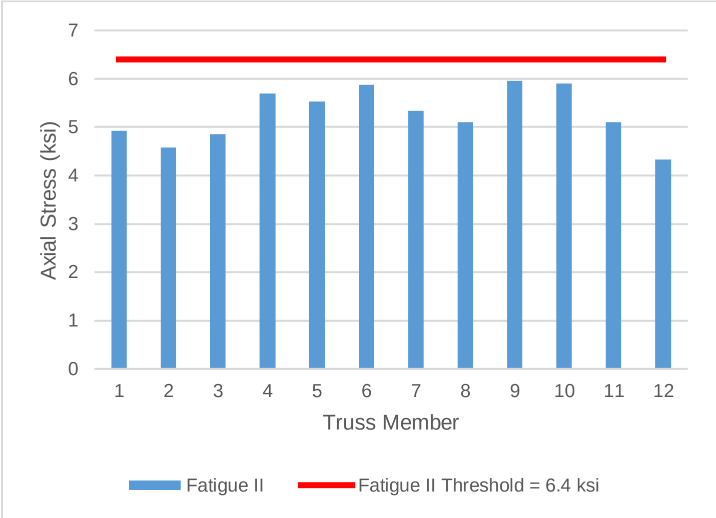


Figure 4: Axial Stress in the Diagonal and Bottom Chord Members with the Welded Connection for the Fatigue II Load Combination

The distribution factor calculated using the lever rule for the truss configuration shown in Figure 5 was 0.93. To investigate the accuracy of this distribution factor, a 3D finite element model (SAP2000) was used to distribute the live loads to the truss members through the concrete deck. The steel channel and T-section truss members were modeled as frames and the concrete deck was represented by approximately 1 ft. by 1 ft. shell elements. An effective moment of inertia of one-half of the gross moment of inertia was used for the slab in the transverse direction and gross section properties were assumed in the longitudinal direction. 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. Composite action was again modeled by locating the 8.75 ft. wide concrete deck and top chord at the composite neutral axis.

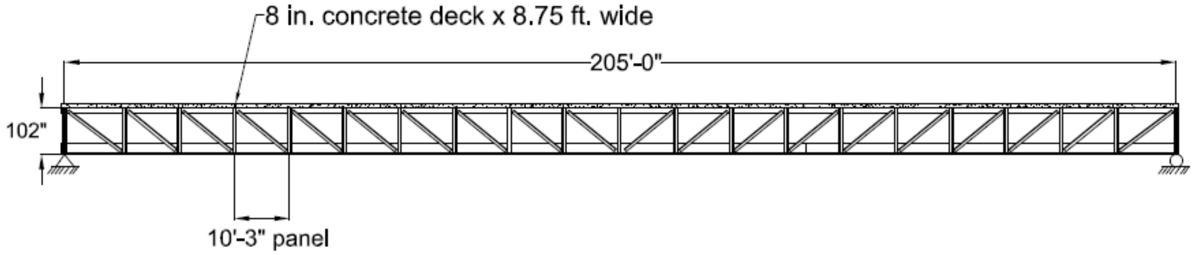


Figure 5: Typical 205 ft. Truss Elevation View

To match the loading used to calculate the distribution factors with the lever rule for the 2D model, two loaded lanes were considered with a multiple presence factor of 1.0. The locations of the distributed lane load and concentrated HL-93 design truck are shown in Figure 6. The HL-93 truck loads were applied as moving loads along the length of the bridge in the SAP2000 model, thereby producing an envelope of tension and compression forces in the steel truss.

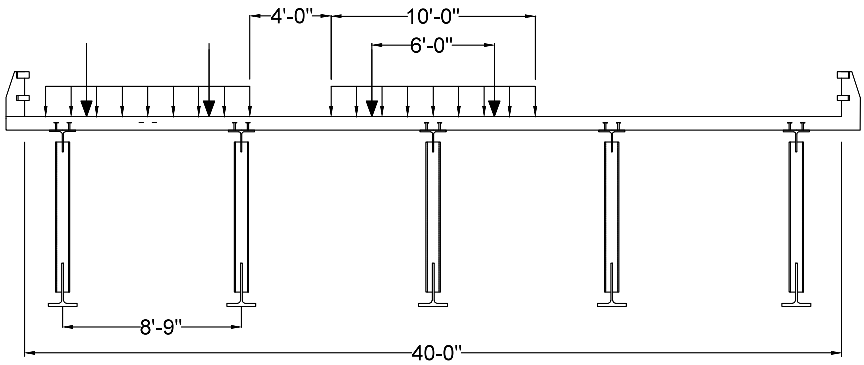


Figure 6: Location of Uniform Land Loads and Concentrated Design Truck Loads for a Two-Lane Condition

To evaluate the magnitude of the tension and compression forces from the 3D analysis using the location of loads shown in Figure 6, 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. The comparison indicates the 3D model results in reductions of vertical, diagonal, and bottom chord forces by approximately 50%.

Moving forward with this analysis, 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 approximately centered between the distribution factors determined for the plate girder system (0.67 for moment and 0.87 for shear) calculated using the AASHTO distribution factor equations. Thus, the truss system design subsequently generated below is directly comparable with the existing plate girder design.

Using a distribution factor of 0.75 with the 2D model, preliminary member sizes selected for the 205 ft hybrid truss are shown in Table 2. The configuration of the bolted connection is shown in Figure 7.

Table 2: 205 ft. Truss Member Sizes

Top Chord Member	Bottom Chord Member	Vertical Member	Diagonal Member
WT16.5x65	WT20x162 / WT20x181	W10x39	MC10x28.5 / MC10x22 / MC8x18.7

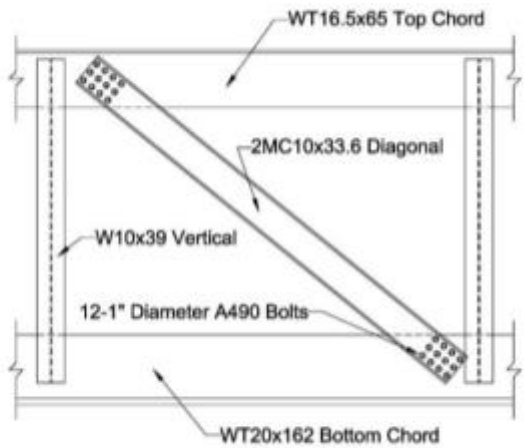


Figure 7: Bolted Connection Detail

Fatigue Analysis Results for the Bolted Connection

The Fatigue I and II load combinations were considered for the bolted connections of the 205 ft. hybrid truss. The Fatigue I threshold increased to 16.0 ksi for Detail Category B and the Fatigue II threshold increased to 14.3 ksi calculated using the same procedure as described above.

Results indicate that the new hybrid truss members satisfy fatigue requirements for an infinite- and finite-life design. Axial stresses in the diagonal and bottom chord members are shown in Figure 8 and Figure 9 relative to the 16.0 ksi and 14.3 ksi thresholds for the Fatigue I and Fatigue II load combinations, respectively.

MATERIALS AND FABRICATION COSTS

Estimated materials and fabrication costs for the prefabricated steel truss and the plate girder design by MDT were obtained from three different sources. Company 1 is the steel fabricator that designed the original all-welded 148 ft. steel truss. Company 2 is a steel fabrication company located in West-Central Montana. Company 3 provides software solutions related to the steel fabrication industry, including cost estimating products.

The weight of steel for both bridge options is shown in Table 3 along with the estimated price from the three sources. The bare steel truss weight of 68 kips was 28% lighter than the plate girder weight of 94 kips. The maximum percent price reduction from the three sources was 33% and the average was 26%.

The variation in estimates shown in Table 3 reflect many different fabrication aspects. Company 1 provided a detailed quotation for the two alternatives that included labor estimates for the welded and bolted connections. The estimates from Company 2 were based on an approximate cost of $1.40/lb. of steel. The labor rates used by Company 3 represent approximate nation-wide fabrication rates including large companies that specialize in plate girder fabrication. Because the three cost estimates include different assumptions in labor, materials, and fabrication processes, the average value of 26% is considered a reasonable approximation to the potential cost savings for the hybrid truss alternative.

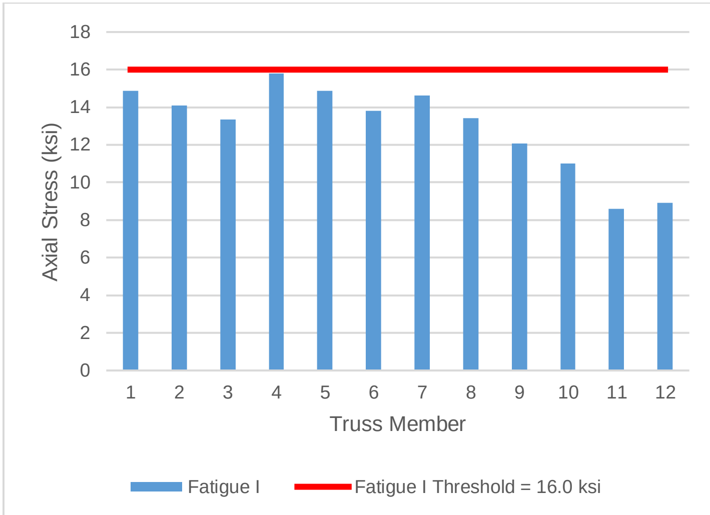


Figure 8: Axial Stress in the Diagonal and Bottom Chord Members with the Bolted Connection for the Fatigue I Load Combination

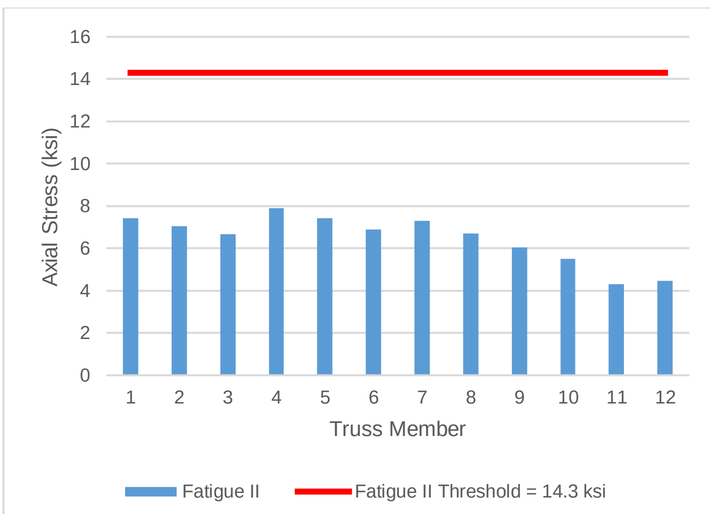


Figure 9: Axial Stress in the Diagonal and Bottom Chord Members with the Bolted Connection for the Fatigue II Load Combination

Additional advantages noted by the steel fabricator sources for the hybrid steel truss were the ability to build camber into the bridge, thus eliminating 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 requirements would be less stringent for the fillet welds between the vertical and top and bottom chords.

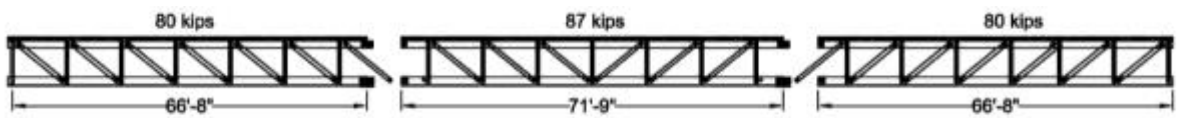
Table 3: Steel Price Estimates

	Prefabricated Steel Truss Wt. = 68 kips	**Plate Girder Wt. = 94 kips**	**Difference**
Company 1	$94,000	$135,000	30%
Company 2	$84,000	$126,000	33%
Company 3	$85,000	$95,000	11%
Average	$88,000	$119,000	26%

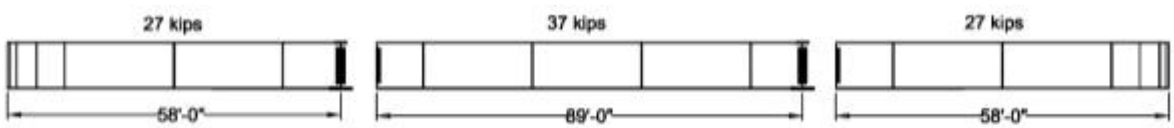
SHIPPING AND ERECTION CONSIDERATIONS

The plate girder was designed with two splices between member lengths of 58 ft., 89 ft., and 58 ft. The reduction in moment from mid-span to 58 ft. from the support was approximately 30%, resulting in significantly smaller splices that did not require increasing the top flange thickness to account for tension rupture through the bolt holes.

Because of the increased weight of the hybrid steel truss with integral concrete deck and for a better comparison with the plate girder, two splices were used between member lengths of 66.6 ft., 71.8 ft., and 66.6 ft. An elevation view with the weight of each splice section for the hybrid truss and plate girder is shown in Figure 10. The weight of the concrete deck is included in the total weight of each splice section for the hybrid truss.



(a) Hybrid Truss with Integral Concrete Deck



(b) Plate Girder

Figure 10: Weight of Each Splice Section for the (a) Hybrid Truss with Integral Concrete Deck and (b) Plate Girder

Shipping Considerations

A summary of relevant general shipping requirements in Montana [3] is provided in Table 4. For the 205 ft bridge span under consideration, the steel truss with a cast-in-place deck has a length of approximately 72 ft. and a total weight of 85 kips. A single truss with a concrete deck could be shipped without exceeding legal load requirements or requiring a permit. For the longest center span of the plate girder, a permit would be required for a length over 75 ft., however more than one girder could be shipped, depending on the trailer/axle combination used.

Table 4: 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.

Erection Considerations

Input from two companies in Montana, both active in heavy civil/bridge construction, was gathered for potential erection advantages and disadvantages of a prefabricated composite hybrid truss and plate girder.

The first construction company preferred the composite hybrid truss to the plate girder. They also were partial to three sections instead of two because shorter member lengths provide easier transportation, site access, unloading, and staging than longer members. This company also suggested additional flexibility is available with the shorter member lengths that is suitable for different construction site conditions. Other advantages noted were 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 is challenging. The ability to field splice the steel truss 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 almost double (552/224) compared with the total number of bolts used in the two splices of the truss. 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.

The second construction company, however, preferred the plate girder to the composite truss. Their concern was related to the heavier cranes and temporary support structures that may be required for the additional weight from the precast integral deck.

CONCLUSIONS

Preliminary designs were completed for a prefabricated all-welded steel truss/integral concrete deck bridge systems intended for a 148 ft bridge span in Montana. A fatigue analysis was performed for the truss members using AASHTO's Fatigue I and II load combinations.

To improve the fatigue response of the welded connections, a hybrid steel truss with bolted and welded connections was designed for a 205 ft span to compare with a recent plate girder bridge project designed by the Montana Department of Transportation. Bolted connections were used for the diagonal truss members and fillet welds for the vertical truss members. The Fatigue I and II load combination results indicate that the hybrid truss with bolted connections satisfy fatigue requirements for both infinite- and finite-life designs.

The following conclusions were made from this investigation of all-welded and hybrid prefabricated steel truss bridge members with an integral concrete deck cast at the fabrication facility:

- Calculated tension stresses for an all-welded, 148 ft span steel truss were approximately 2.5 times threshold stresses for Detail Category E using the Fatigue I (infinite life) load combination. The welded connection does, however, meet a 75-year design life using projected AADT for a bridge crossing where the proposed steel truss/composite deck system could be used.
- A revised preliminary hybrid truss design for a 205 ft bridge with bolted diagonal member connections meets threshold stresses for Detail Category B using both Fatigue I and Fatigue II load combinations.
- The distribution factor calculated using the lever rule was 0.93 for the 205 ft span with trusses space at 8.75 ft. An equivalent distribution of approximately 0.5 was calculated using a 3D finite element model and indicates less conservative distribution factors may be appropriate as truss spacing increases.
- The steel weight of the bolted and welded steel trusses was 28% less than the steel weight of the plate girders. Material and fabrication cost estimates from three sources suggest a bolted/welded steel truss would cost approximately 26% less than a comparable plate girder.
- Despite the lighter weight of the plate girder members, the erection sequence and efficiency is similar to the heavier prefabricated steel truss segments because of the similar member lengths and number of splices. The shorter truss segments would not require special shipping permits in the state of Montana.

  The more efficient bolted splice for tension and compression forces in the hybrid steel truss require approximately one half of the bolts used in the plate girder splices. Fewer bolts and a shorter center span could contribute to the overall efficiency of constructing a prefabricated hybrid steel truss with integral concrete deck.

Based on this investigation, the steel truss configurations for 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.

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