T. J. Wipf, F. W. Klaiber, L. W. Brehm, T. F. Konda

Investigation of the Modified Beam-in-Slab Bridge System

Design Guide
Volume 3 of 3

November 2004

Sponsored by the
Highway Division of the Iowa
Department of Transportation and the
Iowa Highway Research Board

Iowa DOT Project TR-467

Final

REPORT

IOWA STATE UNIVERSITY
OF SCIENCE AND TECHNOLOGY

Department of Civil, Construction and Environmental Engineering

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ABSTRACT

This project (Phase 3 of the Investigation of Two Bridge Alternatives for Low Volume Roads) is a continuation of research which addresses some of the numerous bridge problems on Iowa's secondary road system. In the previous phases, Iowa DOT projects HR-382 (Phase 1) and TR-410 (Phase 2), alternative designs for replacing bridges on low volume roads (LVRs) were investigated. Phase 1 investigated two replacement concepts, the first being the development of Steel Beam Pre-cast Units and the second the modification of the original Benton County Beam-in-Slab Bridge (BISB) design. Phase 2 continued with the development of an alternative shear connector (ASC) for obtaining composite action in the BISB system and an arch formwork between the girders to reduce the self weight of the system.

Results from the first two phases of investigation supported the continued refinement of the Modified Beam-in-Slab Bridge (MBISB) design. This final phase of the investigation was undertaken to develop a competitive alternative bridge replacement for longer spans (i.e. greater than 50 ft). Building on the previous investigations, laboratory tests were undertaken to evaluate the combined behavior of the ASC and the transverse arch.

Three laboratory specimens were designed, constructed and tested to determine the strength, stiffness and constructability of the proposed MBISB design. The results from the laboratory tests indicated that the proposed MBISB system met all applicable design requirements for use on LVRs. Two demonstration bridges were designed and constructed; the construction process was monitored and documented to aid in the implementation of future MBISB designs. The demonstration bridges were field tested to quantify their behavior and to confirm design assumptions. Results from the field tests were used to assist in the development of design criteria for the MBISB system.

The final report for this investigation consists of three volumes; this volume (Volume 3) is a design guide which explains the development of the MBISB design criteria. Volume 1 focuses on the design, evaluation and interpretation of the results obtained from the laboratory and field tests which verified the applicability of the MBISB system. Volume 2, the Design Manual, provides a detailed example problem illustrating the steps required to complete a MBISB design. The Design Manual also includes tabulated data on other MBISB designs and a PowerPoint slide show describing the methodology and the techniques used in the construction of MBSIB 2.

The Design Guide, (Volume 3) is meant to complement the Design Manual (Volume 2), by providing background information on the development of the MBISB design criteria. The Design Guide contains an overview of the laboratory and field tests that were performed and the experimental results obtained. The MBISB design methodology was developed based on the test results and the applicable American Association of State Highway and Transportation Officials (AASHTO) Load Resistance Factored Design (LRFD) Bridge Design Specifications strength and serviceability requirements. An explanation of the resulting MBISB design methodology and criteria which include the design requirements that must be satisfied are also presented.

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1. INTRODUCTION AND BACKGROUND INFORMATION

The following document is the third volume (Volume 3 of 3) of Iowa Department of Transportation (Iowa DOT) report TR-467 "Investigation of the Modified Beam-in-Slab Bridge System, Volumes 1-3" and focuses on the design and construction methods developed for the Modified Beam-in-Slab Bridge (MBISB) system. This alternative bridge replacement design, specifically for low volume roads (LVRs), is based on the original Beam-in-Slab Bridge (BISB) system (*1*). The MBISB system is designed to be constructible by county crews without the use of specialized equipment. The design criteria are based on laboratory and field testing and the American Association of State Highway Transportation Officials (AASHTO) Load Resistance Factor Design (LRFD) Bridge Design Specifications (*2*). AASHTO LRFD Bridge Specifications require that a structure meet both strength and serviceability criteria addressing different aspects of the design. Basic background information explaining the design criteria and the assumptions made are provided to assure the designer of the applicability of the system.

1.1 Purpose

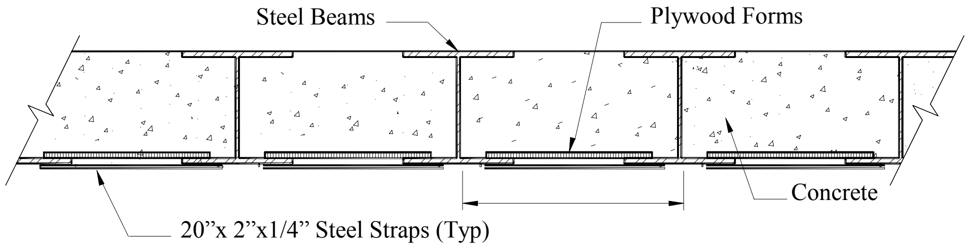
In Iowa, county governments, and more specifically, county engineers are faced with the challenge of upgrading or replacing deficient off system road structures, a majority which are found on LVRs with an Average Daily Traffic (ADT) count significantly less than 400. As reported by the National Bridge Inventory, over 31% of the 19,659 bridge structures owned and maintained by Iowa Counties are either structural deficient or functionally obsolete (*3*). Due to limited resources and the costs associated with maintaining and upgrading this deteriorated bridge population, county engineers have expressed an interest in alternative replacements for the purpose of extending available replacement funds (*4*).

Through funding by the Highway Division of the Iowa DOT and the Iowa Highway Research Board, the Bridge Engineering Center (BEC) of Iowa State University (ISU) has developed various alternative bridge replacements specifically for LVRs. The alternative designs must first be cost competitive when compared to traditional designs; to obtain this goal, alternative designs have focused on reducing the complexity of the system and the design process. In addition, since many counties in Iowa have bridge crews, the alternative structures are designed to be readily constructible by these in-house forces without requiring the use of specialized equipment or complex construction processes.

1.2 Previously Developed Bridge Alternatives

Several successful alternatives have been developed, evaluated and implemented for use on the Iowa off system road system. Alternative designs include railroad flatcar superstructures and the original BISB system developed in Benton County, IA during the mid-1970's. Approximately 80 of these structures are in service today and additional structures are still being constructed, (Lyle Brehm, former Assistant Benton County Engineer, personal communication). The original BISB design typically consists of simply supported W12x79 longitudinal girders spaced 24 in. on center. Steel confining straps are welded to the bottom flanges at the longitudinal quarter points to provide restraint during the placement of the concrete. Plywood formwork rests on the top of the bottom flanges, leaving space between the web and the formwork, allowing for the concrete to be in contact with the bottom flange. Unreinforced concrete is placed to fill the void between the girders and is struck off even with the top flanges to complete the BISB system; a typical BISB cross-section is presented in Figure 1.1a and a photograph of an in service BISB with guardrails is shown in Figure 1.1b.

Field and laboratory tests were performed to evaluate the strength and load distribution characteristics of the original design. Detailed results of the field and laboratory testing were reported in the final report of Iowa DOT Project HR-382 "Investigation of Two Bridge Alternatives for Low Volume Roads, Volume 2 of 2, Concept 2: Beam-in-Slab Bridge" (*1*). For both the field and laboratory tests, the BISB design displayed excellent load distribution properties and minimal live load flexural stresses. Based on the experimental results, the



a. Typical cross section

b. In service BISB with guardrails

Figure 1.1. Original BISB design.

BISB design was determined to have more than adequate strength to resist typical loads associated with LVR systems. However, the maximum span length of the BISB design is limited to approximately 50 ft due to large deflections resulting from the self weight of the structure. In addition, the structural efficiency of the design can be improved if there is composite action between the steel beams and the unreinforced concrete.

1.3 Modifications

Recognizing the benefits of the original BISB design, namely the simplicity and speed of construction, researchers of the ISU BEC proposed modifying the original design to increase the structural efficiency and range of applicability. The proposed modifications resulted in a reduction in self weight and the development of composite action between the steel girders and the surrounding concrete.

1.3.1 Alternative Shear Connector (ASC)

Composite action in most concrete slab/steel girder bridges is normally developed through the use of shear stud connectors that require special equipment to install. The Alternative Shear Connector (ASC) was developed by ISU researchers as a means to develop composite action while maintaining the simplicity of the construction process. The final design for the ASC consists of 1 1/4 in. diameter holes that are either cored or torched through the girder web. The holes are centered one diameter below the web/top flange juncture and are positioned on 3 in. centers longitudinally for the length of the girder. When plastic concrete flows through the holes and cures, a shear dowel is created and mechanically joins the steel girder to the concrete. Reinforcing steel (#4 or #5 reinforcing bar) is placed transversely through every fifth hole to provide lateral confinement of the concrete dowels.

An extensive laboratory testing program was undertaken to evaluate the strength, stiffness, and fatigue characteristics of the ASC due to direct shear as well as shear due to flexure. Push out tests were performed first to develop the hole pattern, followed by full scale test beams subjected to flexural loads. In all cases, the ASC surpassed the required limits while providing full composite action. More details on the ASCs can be found in the final reports for Iowa DOT Projects HR-382 and TR-410 (*1, 5*).

1.3.2 Transverse Arch

In the original BISB design, the void between the longitudinal girders was filled completely with non-composite, unreinforced concrete. Since essentially all the concrete below the neutral axis is in tension when subjected to flexure, it is structurally ineffective. If a portion of the ineffective concrete were removed, the self weight of the structure would be greatly reduced by decreasing the total volume of concrete present. Such a reduction in self weight would allow for longer spans with girders set at a wider spacing (> 2 ft). A transverse arch was proposed and investigated as a possible means of reducing the ineffective concrete. The arched formwork rests on the top of the bottom flanges and spans between the longitudinal girders resulting in a cross section similar to that presented in Figure 1.2.

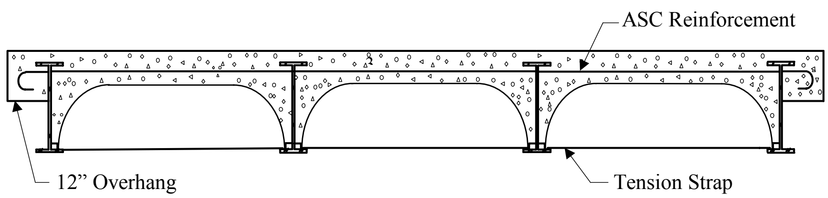


Figure 1.2. Cross section with the transverse arch implemented.

In addition to removing a majority of the ineffective concrete from the cross section, the hypothesis that the transverse arch would change the mode of structural resistance within the deck from flexure in the transverse direction to one of arching action was introduced. The results of such a change would be a considerable reduction in the reinforcement required for the deck; thus reducing the cost, the corrosion potential, as well as simplifying the construction process. The hypothesis was based on results obtained by Canadian researchers who demonstrated that an adequately confined bridge deck slab will resist loadings through internal arching action, failing in a punching shear mode at loads several times greater than design loads (*6, 7*).

2. LABORATORY INVESTIGATION

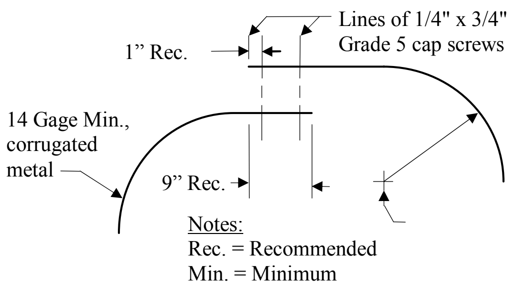
Further research was undertaken to test the applicability of combining the two modifications into an alternative bridge replacement design. The areas of concern that were addressed in the laboratory phase included constructability, ultimate strength, and the integrated behavior of the two modifications. Separating the areas of concern for individual study was impossible since they are interrelated; thus, they were addressed simultaneously.

2.1 Constructability

Constructing the transverse arch in a practical and expedient manner was a necessary requirement if the concept were to remain viable. A variety of materials and methods of forming the transverse arch were investigated with the most desirable solution being a custom rolled corrugated metal section made from steel normally used to construct corrugated metal pipe (CMP). The formwork sections are designed to be removable and reusable as well as modular to allow for offsite construction. A typical arched formwork section is presented in Figure 2.1a and 2.1b. An alternative means of forming the circular arched section for smaller girder spacing is achieved by using a section of CMP resulting in a stay-in-place formwork system. Stay-in-place formwork constructed from sections of CMP has been used in a demonstration bridge a well as several BISB/MBISB derivatives.

2.2 Ultimate Strength of Arched Deck

With the use of the arched section, the mode of failure was expected to change from transverse flexure to punching shear. Due to the change in the mode of failure to one dominated by compression, the amount of slab reinforcement necessary can be greatly reduced. Two additional 2 beam, single bay, transverse arch specimens were constructed to test the punching shear hypothesis and verify the results of the preliminary specimen constructed during Project TR-410 (*5*). The deck reinforcement was reduced to include only the transverse steel needed to complete the ASC. The ASC was included to provide composite action and hence longitudinal confinement of the arched section. Lateral restraint was provided by steel straps welded to



a. Exploded view of circular arched formwork section

b. Completed arched formwork section

Figure 2.1. Typical custom rolled arch formwork sections.

the bottom flanges of the longitudinal girders at the 1/5 and 4/5 pts in addition to the transverse ASC reinforcing steel.

Each of the single bay specimens were subjected an ultimate point load applied over a 20 in. x 16 in. area, simulating the contact area of a single truck wheel. A typical loading setup for the punching shear tests is presented in Figure 2.2. The resulting failure modes were influenced by the amount of transverse restrain provided by the transverse straps welded to the bottom flanges, i.e., the larger the straps, the greater restraint. A combination of transverse flexure/punching shear occurred in the specimens that had 2 in. x 1/4 in. straps while a pure punching shear failure occurred in the specimen that had 3 in. x 3/8 in. straps. The applied load at failure ranged from 155 kips to 260 kips with the higher values being attributed to the additional lateral restraint. The resulting failure loads are considerably higher than a factored wheel load of 45 kips indicating the transverse arch system, even with a majority of the reinforcement normally associated with a concrete bridge deck removed, possessed significant reserve capacity.

2.3 Integrated Behavior

Since the modifications were to be combined for use in an alternative bridge design, a full scale bridge model was constructed in the laboratory and tested under service and ultimate loadings to evaluate the MBISB system. The model was 31 ft long by 20 ft wide, and was constructed using

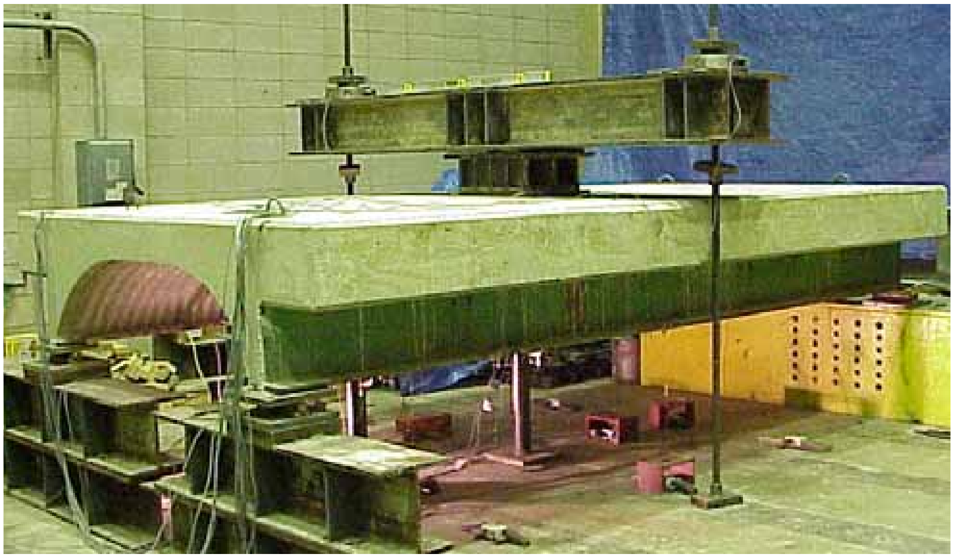


Figure 2.2. Typical setup of the ultimate punching shear test for a single bay specimen.

four W21x62 A36 girders set on 6 ft centers. A typical cross section of the model bridge is presented in Figure 2.3a. Holes were torched in the webs of the girders to form the ASC with #5 reinforcing steel completing the composite action system. Transverse arches were formed using recoverable custom rolled corrugated metal formwork, resulting in a minimum deck thickness of 8 1/2 in.

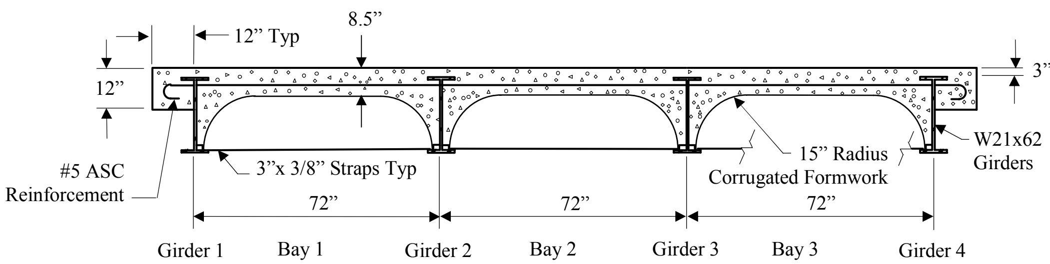
Structural reinforcement of the concrete consisted of only the transverse steel necessary to complete the ASC. Additional #3 reinforcing steel was placed transversely over 1/2 of the model to arrest cracking due to temperature and shrinkage effects as well as possible spalling of the concrete directly over the girders. As one would expect, temperature and shrinkage reinforcement was found to have minimal effect on the structural behavior. The girders, reinforcement, and the formwork of the model bridge prior to concrete placement are shown in Figure 2.3b.

In addition to the ASC reinforcement, transverse restraint was provided by two 3 in. x 3/8 in. steel straps welded to the bottom flanges of the longitudinal girders at the 1/3 and 2/3 points.

Diaphragms constructed from W6x9 sections were placed at the midspan to provide compression flange bracing. An overall view of the completed model taken during the ultimate load test is presented in Figure 2.3c.

2.3.1 Service Level Evaluations

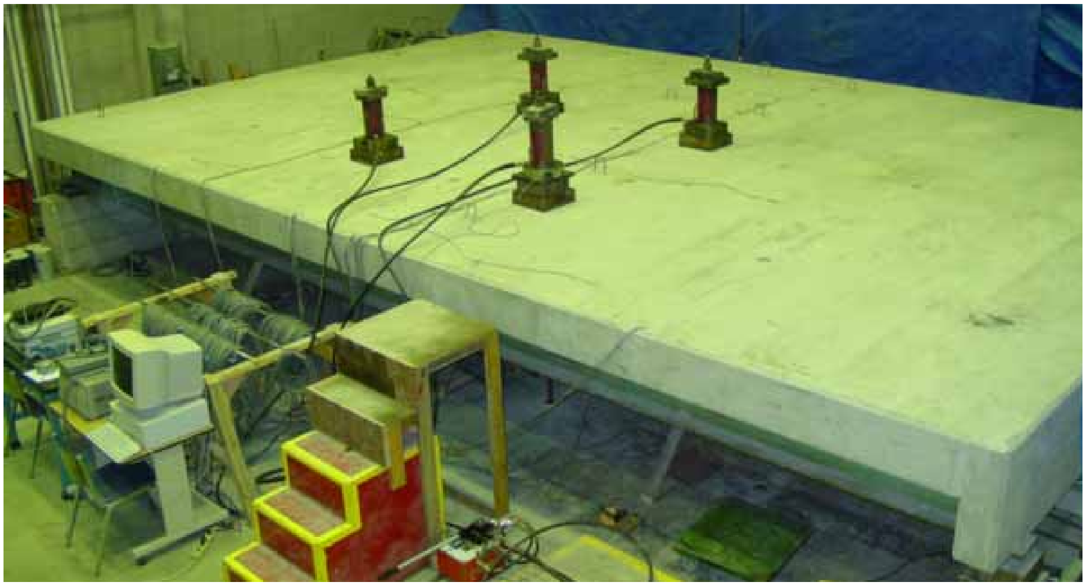
Previous tests on the combined modifications focused on a single mode of behavior such as the deck resistance to a punching shear failure. The full scale model permitted the application of numerous combinations of loading and thus the evaluation of a variety of behaviors. The response of the model bridge to the applied loads was recorded with strain gages and deflection



a. Typical transverse cross section of the model bridge

b. Formwork and reinforcement in the model bridge

Figure 2.3. Layout of the model bridge.



c. Over view of the model bridge during the ultimate flexural loading

Figure 2.3. Continued.

instrumentation placed at critical sections. The load distribution and flexural response of the MBISB structure was determined by applying a series of 45 kip service level point loads at the quarter, mid and three quarter span locations. The positioning of the applied service level loads is illustrated in Figure 2.4. The maximum tensile stress (11 ksi) and maximum deflection (0.29 in.) that result from the service level live loading are well below critical values and occur when the point load was placed at the extreme edge position (Position S4).

Lateral load distribution factors for a single lane loading were also calculated based on the measured strains and were compared to AASHTO LRFD Bridge Specification lateral load distribution factors for a concrete slab/steel girder bridge (*2*). The experimental load distribution factors result from a loading condition (single point load) more severe than that used to develop specification values, which were developed with a full truck/s on a bridge. Therefore, the results from the experimental data represent an extreme loading condition. The maximum experimental load distribution for an interior girder was 45% which matched the AASHTO value for a single lane loading. The experimental distribution factor for the exterior girder was slightly larger

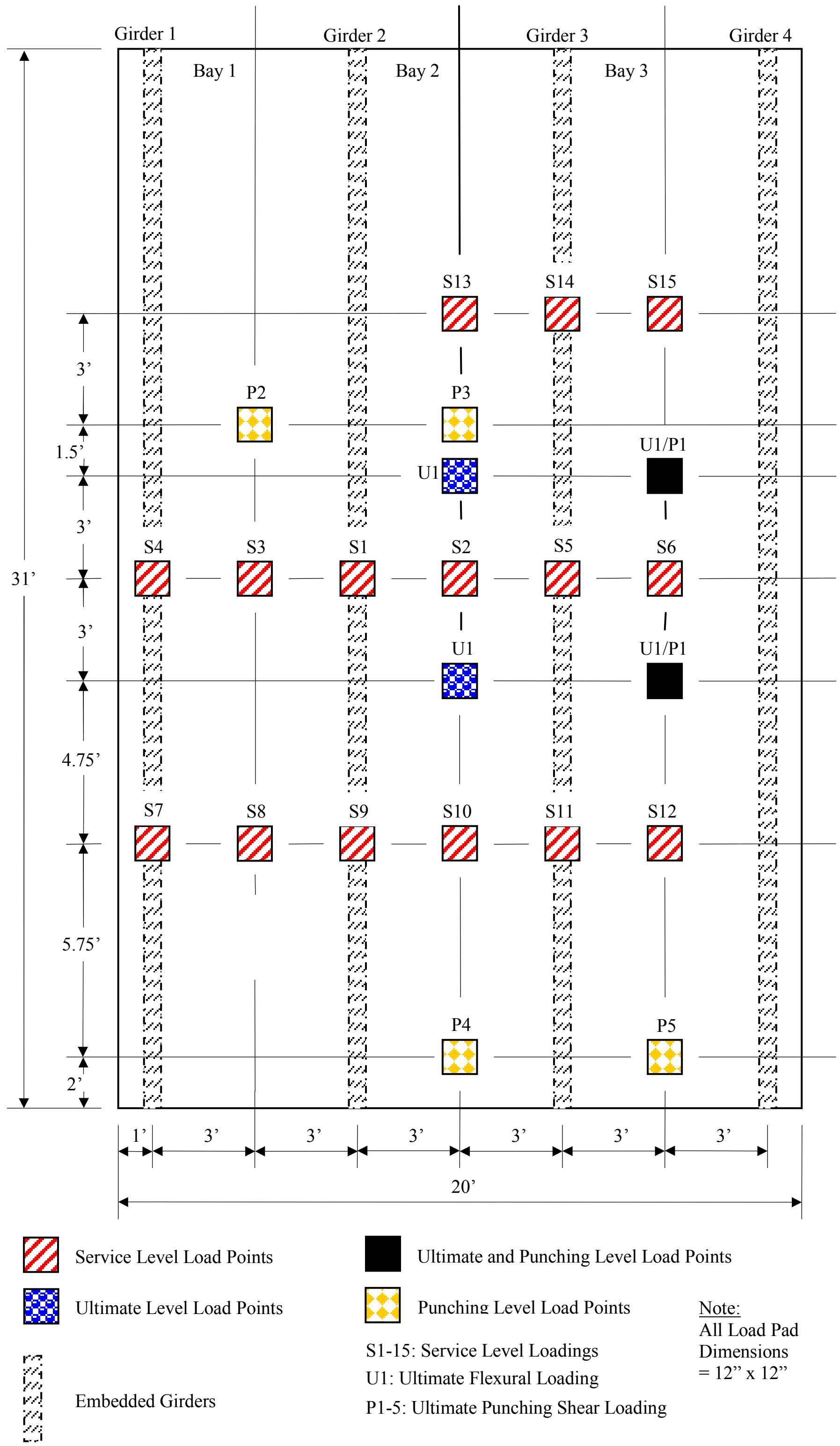


Figure 2.4. Plan view of model bridge illustrating various load points.

(56% vs. 54%) than the single lane value calculated by the methods outlined in the AASHTO specifications. However, because of the severity of a single point load, as opposed to a truck, placed directly over the girder, a conservative result was not expected. The experimental distribution factors were calculated based on a cracked moment of inertia; the applicable multi-presence and Average Daily Truck Traffic (ADTT) factors are applied to both the specification based and experimental distribution factors for consistency when comparing the results (*2*).

2.3.2 Ultimate Flexure Test

After the service level tests were completed, the model bridge was subjected to a four point ultimate flexure test; the positioning of the load points are presented in Figure 2.3c and Figure 2.4. The specimen resisted 302 kips while undergoing over 4 in. of deflection. Three of the four longitudinal girders permanently yielded and the deck experienced considerable cracking but did not experience a punching failure. Through out the test, the ASC provided full composite action adding to the existing experimental evidence of the applicability and capacity of the ASC.

2.3.3 Ultimate Punching Tests

Additional tests were performed on the yielded model bridge; a series of 5 individual point loads were applied to investigate the punching characteristics of the distressed deck. Similar to the original punching tests performed on the single bay specimens, a 12 in. x 12 in. load pad was used to simulate a wheel load in the tests to determine the remaining capacity of the deck. Punching shear failures occurred in all cases with the failure load ranging from 117 kips to 158 kips, which is several times larger than a factored wheel load. The 12 in. x 12 in. load pad is smaller than the contact area of a typical tire thus creating a more critical loading case.

2.4 Summary of Laboratory Testing

The laboratory testing phase provided results to support the implementation of the two proposed modifications. The ASC provided both full composite action and longitudinal confinement which changed the deck failure mechanism from one of transverse flexure to punching shear. The implementation of the transverse arch reduced the amount of concrete needed when compared to the original BISB design, which thus improved the structural efficiency. As a result of the reduced self weight, longer spans with the girders set at a wider spacing can be implemented, furthering the efficiency of the design. At the same time, the use of the custom rolled corrugated formwork sections maintained in field construction simplicity; after the concrete cured, the formwork was removed and prepared for reuse.

The transverse arch, when adequately restrained in both the longitudinal and transverse direction, changed the mode of structural behavior by resisting simulated wheel loads though an arching action rather than flexure. The strength of the deck due to the arching action was considerably larger than a comparable deck resisting wheel loads in flexure, even with a majority of the typical deck reinforcement removed. Experimental lateral load distribution factors for the MBISB system are comparable to AASHTO LRFD Bridge Specification values for a single lane loading (*2*). Through out the testing sequence, the laboratory model bridge resisted loads significantly greater than design loads even after the longitudinal girders were yielded and the deck heavily cracked.

2.5 Analytical Modeling

A grillage model using ANSYS finite element software was developed and calibrated based on the experimental service level load results. The longitudinal girders were represented by Beam 4 elements while the transverse elements consisted of tapered Beam 44 elements to account for the increased deck thickness due to the transverse arched deck (*8*). Gross section properties were assigned to the elements following the methodology presented in Hambly (*9*). The analytical model was then used in the development of the MBISB design criteria

3. DEMONSTRATION BRIDGES

Based on the presented results of the laboratory testing, two demonstration bridges with the investigated modifications were designed, constructed and field tested to further verify the applicability of the MBISB system. The design methodology is explained in greater detail in Chapters 4 and 5; the construction techniques used are presented in the Design Manual, Volume 2 (*10*). A more detailed description of the laboratory and field testing as well as a more in depth presentation of the resulting data is presented in Volume 1 (*11*).

3.1 MBISB 1

The design of the first demonstration bridge, referred to as MBISB 1, closely followed the original BISB design with the addition of the ASC and the transverse arch.

3.1.1 MBISB 1 Design

The initial demonstration bridge (MBISB 1) is 50 ft long by 31 ft wide with 16-W12x79 Grade 50 sections spaced on 2 ft centers for the longitudinal members. Composite action is developed by the ASC which was formed by torching holes through the web at the prescribed size and spacing. Sections of 24 in. diameter 16 gauge CMP were used to form the transverse arch between the girders, reducing the self weight by approximately 20% when compared to the BISB design. Transverse confinement of the concrete dowels completing the ASC was provided by #4 Grade 60 reinforcing steel. The girders were restrained in the transverse direction during the concrete placement by 4 lines of 1 1/2 in. x 1/4 in. steel straps welded to the bottom flanges at the 1/5 points of the span.

3.1.2 MBISB 1 Construction

Concrete was placed and struck off even with the top flanges of the girders, as shown in Figure 3.1a. Special attention was given to ensure the concrete flowed through the ASC holes to form the shear dowels as shown in Figure 3.1b. After adequate curing, a guardrail system was installed, backfilling completed and the bridge was opened to traffic.

a. Concrete struck off even with the top flange

b. Concrete flowing through the ASC holes forming shear dowels

Figure 3.1. Concrete placement for MBISB 1.

The cost for the MBISB 1 structure was approximately $50 psf making this a competitive alternative design when compared to traditional replacement designs (costs compiled by Tama County Engineer's Office, unpublished data). MBISB 1 was constructed by the Tama County Bridge Crew without the need for extra equipment or expertise. The costs presented are reflective of the site and market conditions and may vary for other cases.

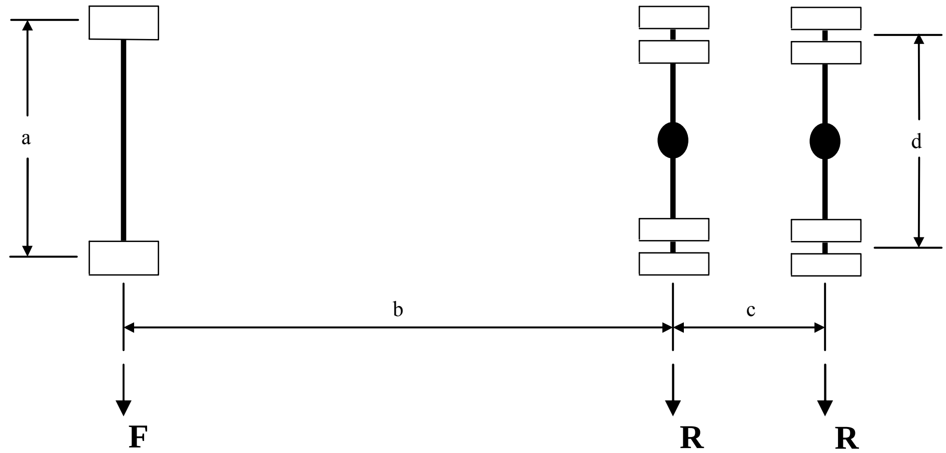
3.1.3 Permanent Deflections

Camber was not introduced to the girders for MBISB 1 but a 1.1% transverse crown was put in place to provide for drainage. However, full drainage of the deck surface was not achieved due to the final girder elevations, resulting in surface ponding. The ponding was due to a concave abutment cap profile that resulted from bending caused by residual welding stresses and differential deflections between the interior and exterior girders due to flexural and torsional effects. By implementing the following criteria, the ponding of water on future MBISB structures can be avoided:

- Camber the girders to counteract anticipated dead load deflections.
- Prevent concave curvature in the abutment cap.
- Increase the cross slope to 2% crown as specified by AASHTO (*12*).

3.1.4 Field Testing

A field test was performed on MBISB 1 to quantify the structural behavior with particular interest in the resulting stresses, deflections and lateral load distribution in the system. The bridge was instrumented to measure strain and deflection at critical locations (the abutments, quarter, mid, and three quarter span). The bridge was divided into loading lanes to create maximum effects at the interior and exterior locations as well as testing for transverse symmetry. Load was applied to the bridge by two loaded tandem axle dump trucks provided by Tama County. The axle spacing and weights for each test vehicle are presented in Figure 3.2, an assumption is made that the measured tandem weight is evenly distributed between the two axles. A detailed layout of the instrumentation, the loading lanes, and the test vehicle position with respect to said lane is presented in Figure 3.3.



Truck Number	a (in.)	b (in.)	c (in.)	d (in.)	F (kips)	R (kips)	Total Load (kips)
927	80	191	53.5	72	17.52	16.74	51.00
929	80	191	53.5	72	15.62	17.41	50.44

Figure 3.2. Wheel and load configurations for the MBISB 1 test vehicles.

The bridge was subjected to a series of static and rolling tests to maximize desired effects. For the static testing case, both test vehicles were positioned on the structure to create a maximum moment effect at the midspan. The rolling tests consisted of one test vehicle crossing the bridge following one of the test lanes at approximately 2 mph. The positioning of the test vehicles for the static load is depicted in Figure 3.4a, and a typical Lane 4 rolling test is presented in Figure 3.4b.

3.1.5 Data Analysis

Using the measured data, the following structural behavior characteristics were investigated and quantified: deflections, flexural stresses and lateral load distribution. The maximum live load deflection and stress were attributed to the static load case which was expected since both test vehicles were on the structure simultaneously thus doubling the load. The maximum deflection that resulted from a single test vehicle performing the rolling tests was found in the exterior girder. However, all measured deflections were less than the recommended

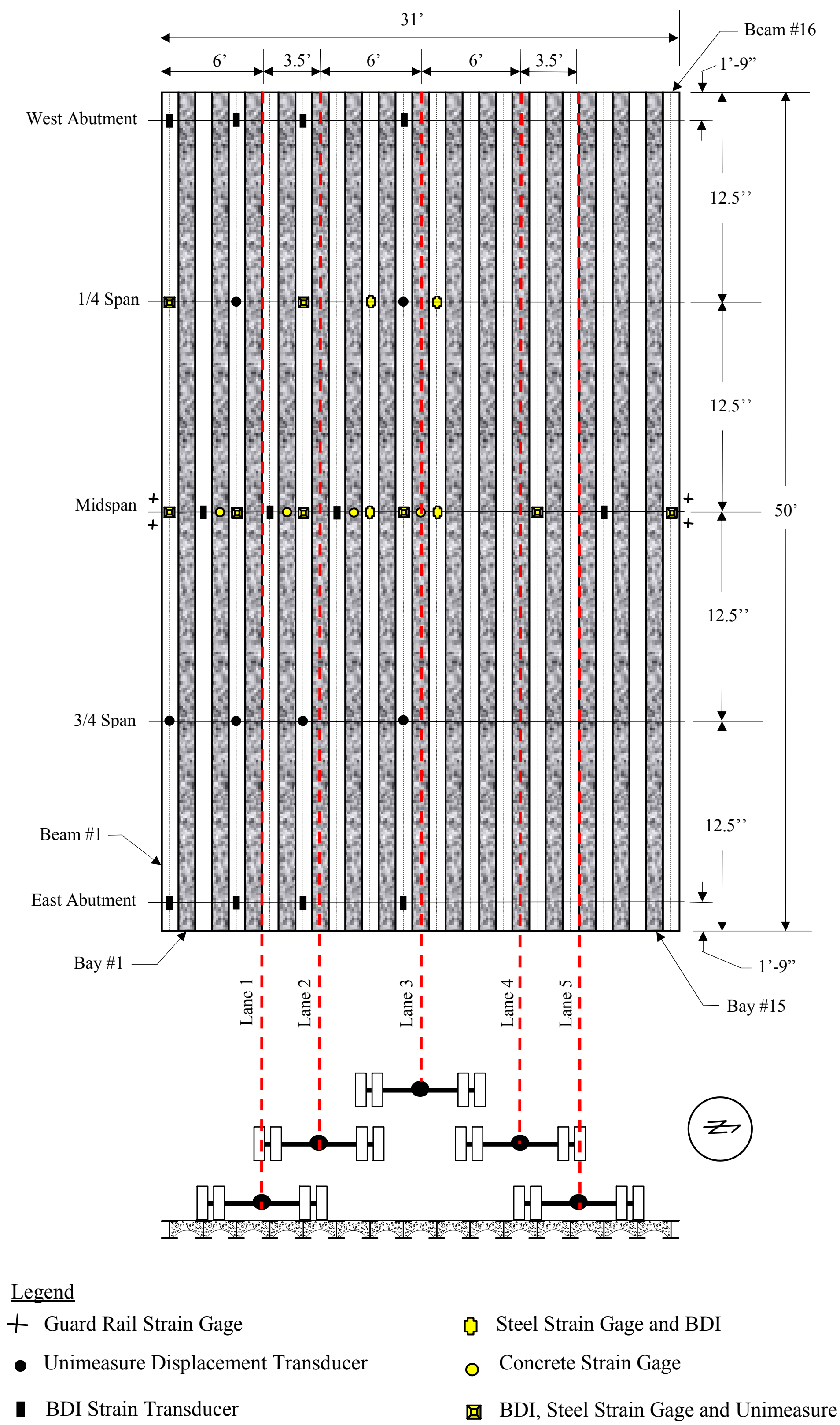


Figure 3.3. Instrumentation and loading lane layout for MBISB 1.

a. Static load test

b. Rolling test with test vehicle in Lane 4

Figure 3.4. Test vehicles on MBISB 1.

AASHTO L/800 deflection criteria which is 0.735 in. for MBISB 1 (*2*). The resulting maximum deflections are presented in Table 3.1. Maximum midspan stresses that were calculated from the measured strains (assuming E_s = 29,000 ksi and E_c = 4,230 ksi) are presented in Table 3.2. The locations of the maximum effects are noted and shown in Figure 3.3. The measured stresses fall well below maximum service level stress values due to live loads as specified by the AASHTO LRFD Bridge Specification (*2*).

Table 3.1. Maximum deflections in MBISB 1 during load testing.

Maximum Deflection	Interior Girder (Girder 8)	Exterior Girder (Girder 16)
Static Load Test	0.73 in.	0.68 in.
Rolling Load Test	0.34 in.	0.56 in.

The experimental distribution factors were not compared to design specifications since the 2 ft girder spacing can not be directly correlated to the AASHTO LRFD Specification (*2*). The experimental distribution factors were calculated without accounting for the structural contribution of the guardrail. The resulting experimentally determined moment distribution factor for the exterior girder is 11%, and the interior distribution factor is 12%. Since the maximum single lane distribution factors for the interior and exterior girders are quite similar, for design, all girders for this structure can be considered to have a maximum distribution factor of 12%. The distribution factors are calculated using the gross composite section and applying appropriate multi-presence and ADTT factors.

Table 3.2. Maximum midspan flexural stresses experienced by MBISB 1 during load testing.

	Maximum Effects		
	Stress (ksi)	Location	
Static Test	Maximum Value	Maximum Effect Location	Load Location
Concrete	- 0.46	Bay 6	Lane 2 and 4
Girder	- 3.72	Girder 6	Lane 2 and 4
Girder	+ 4.47	Girder 6	Lane 2 and 4
Rolling Test			
Concrete	- 0.37	Bay 2	Lane 1
Girder	- 2.05	Girder 1	Lane 1
Girder	+ 3.75	Girder 2	Lane 1

Note: + = tensile stress; - = compressive stress

3.1.6 Analytical Modeling

Using ANSYS finite element software, a grillage model was developed and calibrated based on the experimental service level load data. A parametric study was completed to quantify the effect the guardrail had on the global behavior of the structure. Results indicate that the guardrail increases the total longitudinal flexural rigidity of the structure by approximately 15% to 20% depending on the considered loading condition. In spite of the contribution of the guardrail to the flexural rigidity of the bridge, the resulting increased stiffness is not included in the MBISB design methodology developed.

3.1.7 Summary, MBISB 1

MBISB 1 was the first bridge constructed combining the ASC to develop composite action and the transverse arch to reduce the self weight of the structure. Construction followed the same format as the original BISB design with the concrete being struck off even with the top flanges of the girders (*1*). The final structure is cost competitive with conventional designs of similar length and fully constructible by in-house forces. The small amount of ponding that occurred on this bridge can be avoided in the future by introducing camber, increasing the cross slope, and leveling the abutment cap.

MBISB 1 was field tested to quantify the deflections, strains, (stresses), and lateral load distribution factors of the completed structure. Both one and two lane loadings were applied and the deflections and strains (stresses) that resulted were below specified limits. Lateral distribution factors were evaluated based on the measured strains and compared to analytical values determined by the grillage analysis; a single lane loading distribution factor of 12% was determined to govern. Overall, MBISB 1 met strength and serviceability requirements as well as being cost competitive and readily constructible.

3.2 MBISB 2

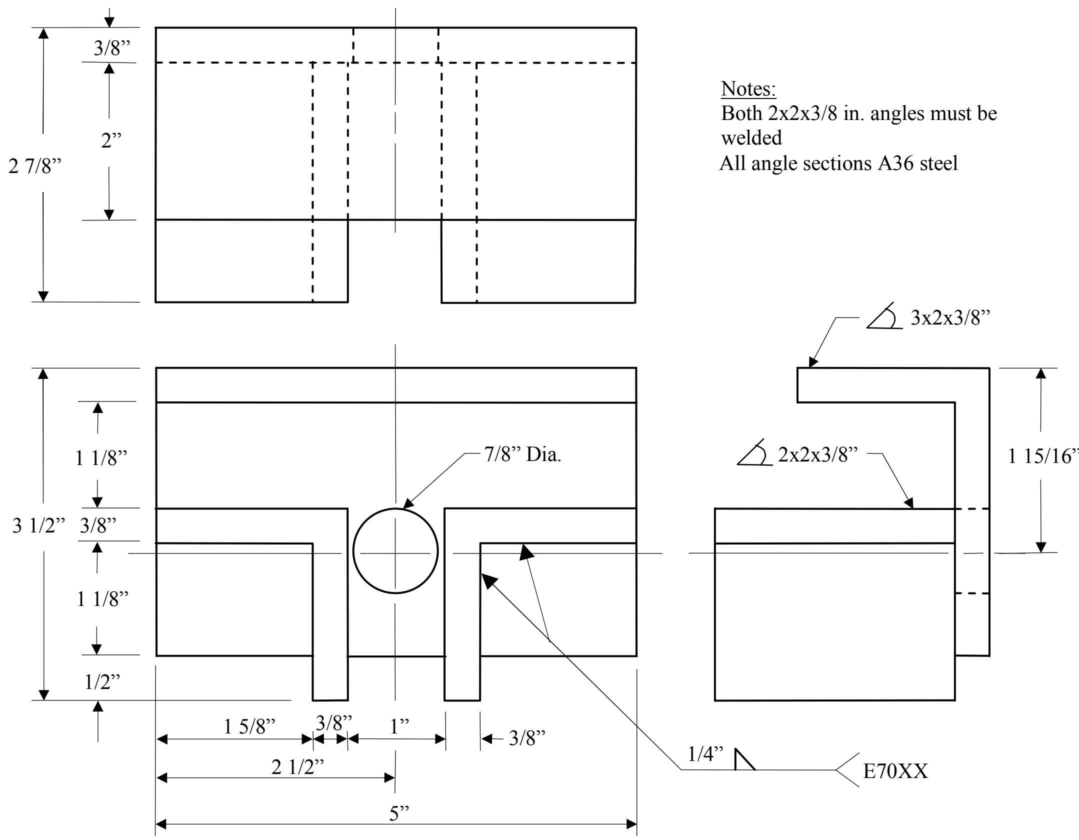
A second demonstration bridge, referred to as MBISB 2, was designed and constructed based upon AASHTO LRFD Bridge Specification and data from the laboratory tests (*2*). The ASC and the transverse arch were again implemented to improve the efficiency of the design when compared to the original BISB design.

3.2.1 MBISB 2 Design

The second demonstration bridge is 70 ft long by 32 ft wide with six W27x129 Grade 50 girders spaced on 72 in. centers acting as the main longitudinal members. All the girders were cambered to counteract the self weight deflection with the two exterior girders cambered to 2 1/2 in. at the midspan and the four interior girders cambered to 4 in. at the midspan. While all the girders were the same size, the two camber values were necessary to account for the differing dead load carried by the interior and exterior girders. Holes to develop composite action were cored through the longitudinal girder webs at the prescribed size and spacing, forming the ASC. The transverse arch between the girders was formed by 14 gage custom rolled corrugated metal arched formwork which rested on the bottom flanges of the longitudinal girders. Reinforcing steel (#5 Grade 60 deformed bar) was placed through every fifth ASC hole to provide transverse confinement for the shear dowels. Two lines of diaphragms, consisting of recycled S18x54.7 sections, were installed 24 ft from the center line of each abutment to provide adequate compression flange bracing during the construction phase. Three lines of threaded 3/4 in. diameter rods were centered between each diaphragm line and were attached to the bottom flanges with tension clips to provide transverse restraint during the concrete placement. The tension rods can be removed and reused after the curing of the concrete. A schematic of a typical tension clip and a photograph of an installed tension clip on MBISB 2 are presented in Figure 3.5.

3.2.2 MBISB 2 Construction

Construction of the MBISB superstructure commenced with placing the fabricated girders at the desired locations on the abutments. The diaphragms were installed and 12 in. thick concrete backwalls were placed at each end of the girders. The custom rolled corrugated arched formwork sections, exterior formwork, and the necessary reinforcement were installed completing the deck placement preparations. Following the original BISB design, the deck concrete was placed between the girders; however, the top flange of the girders was embedded by



a. Schematic of tension clip

b. Installed tension clip

Figure 3.5. Tension clip for restraining the bottom flanges of the girders of MBISB 2.

3 in., increasing the flexural rigidity while decreasing the potential for corrosion. A typical cross section of MBISB 2 is presented in Figure 3.6. The different diameters in the custom rolled formwork used in MBISB 2 resulted from the reuse of the sections previously used in the laboratory model bridge (15 in. radius sections).

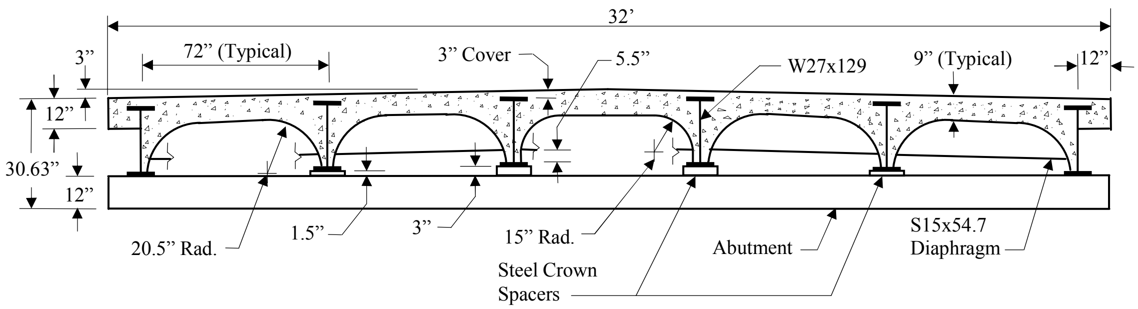


Figure 3.6. Cross section of MBISB 2.

Due to the 3 in. cover over the girders, a power screed supported by the exterior formwork was used to establish the final deck elevation. The deck pour is shown in Figure 3.7a; a conveyor truck was used to assist in the concrete placement. After the concrete had adequately cured; a Thrie Beam guardrail system was installed, backfilling completed, and the bridge opened to traffic. The custom rolled arched formwork sections were removed (see Figure 3.7b) during the construction off season and stored for future use.

Similar to MBISB 1, the construction of MBISB 2 was completed by Tama County forces with the exception of the ASC holes which were cored by the steel fabricator. MBISB 2 cost approximately \$52/ft^2, (costs compiled by Tama County Engineer’s Office and ISU BEC, unpublished data) making the design a competitive alternative for long spans (>50 ft). The costs presented are reflective of the site, market conditions and the reuse of the formwork system and may vary for other cases.

a. Deck placement

b. Formwork removal

Figure 3.7. MBISB 2 construction.

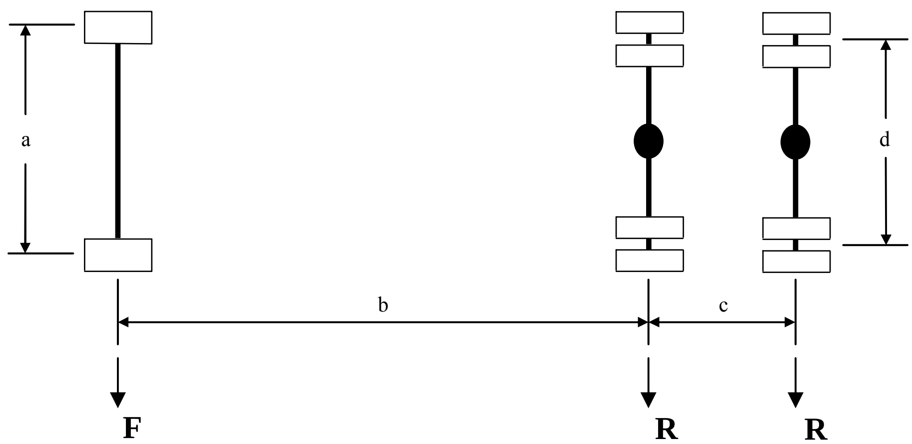
3.2.3 Permanent Deflections

Deck ponding was avoided due to the combined effect of the 2.0% cross slope, girder camber, and the final deck elevation set by the power screed resting on the exterior formwork. The self weight of the cross section completely removed the 4 in. camber in the interior girders. Thus, the resulting interior girder profile was then essentially “flat” after the placement of the concrete. The exterior girders underwent a larger amount of deflection than initially estimated resulting in a concave girder profile with a maximum midspan deflection due to the self weight equal to 1.2 in. below the horizontal profile. The concave deflection resulted from rounding down the required camber during the design phase and an increase in the cross sectional area of the exterior girder section after the girders had been ordered. The concave deflection was counteracted by adding a tapered rail to the exterior formwork, increasing the concrete depth over the exterior girder which in turn resulted in a level longitudinal deck elevation.

3.2.4 Field Testing

Similar to MBISB 1, a field test was performed on MBISB 2 to quantify the structural behavior with particular interest in the resulting strains (stresses), deflections, and lateral load distribution of the system. The bridge was instrumented at critical locations (near the abutments, the midspan, the quarter span, and the three quarter span) to measure the longitudinal flexural strains and vertical deflections. Additional instrumentation was placed on the diaphragms and the transverse tension rods to measure secondary effects. The deck surface was divided into five loading lanes to establish maximum effects at both the interior and exterior locations as well to evaluate the presence of transverse symmetry.

Load was applied to the bridge using two loaded tandem axle dump trucks provided by Tama County. The axle spacing and weights are presented in Figure 3.8; an assumption is made that the measured tandem weight is evenly distributed between the two rear axles. A detailed



Truck Number	a (in.)	b (in.)	c (in.)	d (in.)	F (kips)	R (kips)	Total Load (kips)
917	80	192	54	72	16.94	16.95	50.84
918	80	192	54	72	17.84	19.08	56.00

Figure 3.8. Wheel and load configurations of the MBISB 2 test vehicles.

layout of the instrumentation, the loading lanes and the test vehicle position is presented in Figure 3.9.

The bridge was subjected to two static and five rolling tests to quantify the desired maximum effects. For the static tests, both test vehicles were simultaneously placed on the structure to create a maximum moment effect about the midspan. The test vehicles occupied Lanes 2 and 4 for the first static test and Lanes 1 and 5 for the second. The rolling tests consisted of a single test vehicle crossing the bridge in one of the five test lanes at approximately 2 mph. This procedure was repeated for each of the five lanes, completing the rolling test sequence. Photographs of the vehicles used in the static load tests and the vehicle used in a typical lane rolling test are presented in Figure 3.10a and Figure 3.10b, respectively.

3.2.5 Data Analysis

The collected data were used to quantify the following structural behavior characteristics: deflections, flexural stresses, and lateral load distribution. The maximum live load deflections and strains (stresses) were attributed to the static loading cases, which were expected since both

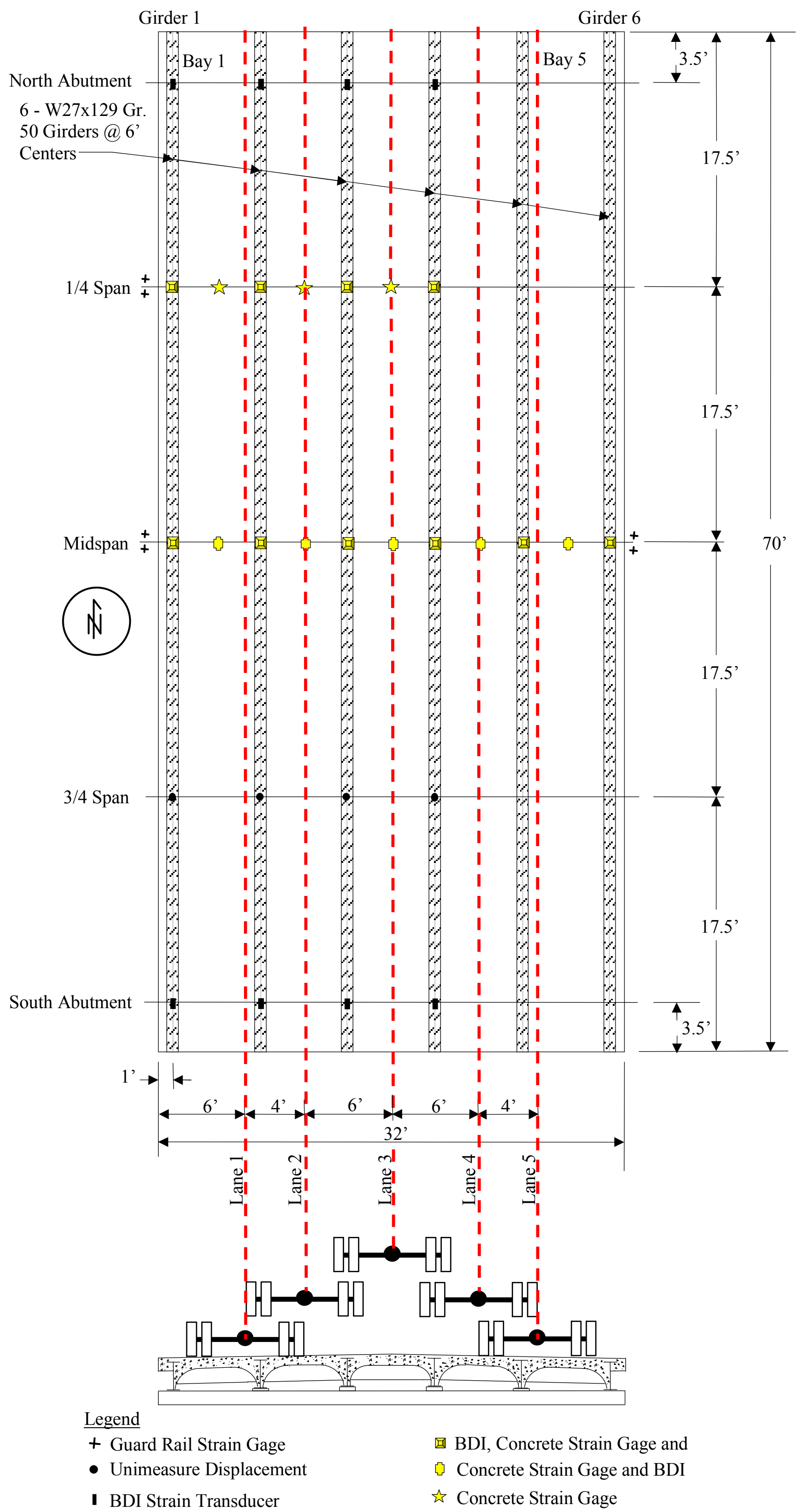


Figure 3.9. Layout of main flexural instrumentation and test vehicle placement for MBISB 2.

a. Static test 1, test vehicles in Lanes 2 and 4

b. Rolling test with test vehicle in Lane 2

Figure 3.10. Test vehicles on MBISB 2.

test vehicles were simultaneously on the structure thus doubling the load effect. The largest deflection that resulted during a rolling test occurred in an exterior girder with the vehicle occupying Lane 1. All measured deflections were less than the recommended AASHTO L/800 deflection criteria, which is 1.04 in. for this bridge (*2*). The resulting maximum deflections for the field test are presented in Table 3.3.; the locations of the maximum deflections are shown in Figure 3.9.

Table 3.3. Maximum midspan deflections experienced by MBISB 2 during load testing.

Maximum Deflection	Interior Girder (Girder 3)	Exterior Girder (Girder 1)
Static Load Tests (two test vehicles)	0.50 in.	0.50 in.
Rolling Load Tests (one test vehicle)	0.29 in.	0.48 in.

The maximum flexural stresses for both the static and rolling loads were calculated from the measured strain values. As expected, the maximum stresses resulted from the static load tests; however, neither the compression nor tension stresses approach AASHTO serviceability limits. The maximum calculated stresses, assuming elastic conditions, are presented in Table 3.4.

Table 3.4. Maximum midspan flexural stresses experienced by MBISB 2 during load testing.

	Maximum Effects		
	Stress (ksi)	Location	
Static Test	Maximum Value	Maximum Effect Location	Load Location
Concrete	- 0.41	Girder 4	Lane 2 and 4
Girder	+ 5.54	Girder 1	Lane 1 and 5
Rolling Test			
Concrete	- 0.33	Girder 6	Lane 5
Girder	+ 4.85	Girder 6	Lane 5

Note: + = tensile stress; - = compressive stress

Lateral load distribution factors for the girders were calculated based on the measured midspan girder tension strains for both the static and rolling loads. Cracked section properties were used to calculate the exterior distribution factors and the uncracked (gross section) properties were used to calculate the interior distribution values. The reason for this assumption was to provide the most conservative distribution factor although the difference in the calculated distribution factors between the cracked and uncracked assumption was small (approximately five percent).

The experimental distribution values were compared to the AASHTO LRFD Bridge Specification lateral load distribution factors for slab/girder bridges. The rolling tests accounting for a one lane loading and the static tests accounting for a two lane loading. The experimental distribution factors were calculated without accounting for the additional flexural rigidity attributed to the guardrail. The controlling experimental and the AASHTO LRFD specified lateral load distribution factors are presented in Table 3.5; all values are multiplied by applicable multi-presence and ADTT reduction factors. When comparing the experimental and AASHTO lateral load distribution factors, one observes the AASHTO values provide conservative values. Since the AASHTO distribution values are conservative, for simplicity the MBISB design methodology uses the AASHTO LRFD specified distribution factors for slab/girder bridges (*2*).

Table 3.5. Comparison of controlling experimental and AASHTO moment distribution factors for MBISB 2.

	DISTRIBUTION FACTORS			
Girder	One Lane (%)		Two Lane (%)	
	Experimental	AASHTO LRFD Specification	Experimental	AASHTO LRFD Specification
Interior	32	36	43	49
Exterior	36	46	41	56

3.2.6 Analytical Modeling

A grillage model using ANSYS finite element software was developed and calibrated using the field test data (*8*). A parametric study was undertaken to quantify the effect of the guardrail on the global behavior of the structure. Analysis results indicate that the guardrail has an influence on the total longitudinal flexural rigidity of the structure but not as significant as in the case of MBISB 1. The calibrated analytical model was then used to assist in the development of the MBISB design criteria.

3.2.7 Summary, MBISB 2

MBISB 2 was the second bridge constructed implementing the ASC to develop composite action and the transverse arch to reduce the self weight of the structure. The final design is similar to a slab/girder bridge with the deck encasing the girders; this differed from the original BISB design where the concrete was struck off even with the top flange. The final structure is cost competitive with conventional designs and fully constructible by county forces.

The MBISB 2 was subjected to a field test to quantify the deflections, strains (stresses), and lateral load distribution factors for the completed structure. Both one and two lane loadings were applied and the resulting deflections and strains (stresses) due to the live loadings were below specified limits. Lateral load distribution factors were evaluated based on the measured strains and compared to AASHTO LRFD Bridge Specification factors (*2*). The AASHTO lateral load distribution factors were determined to be conservative and are taken as the governing factors for future design criteria. Overall, MBISB 2 met strength and serviceability requirements as well as being cost competitive and readily constructible.

4. MBISB DESIGN CRITERIA

The design criteria and methods applied to the MBISB system is an amalgamation of AASHTO LRFD Bridge Specifications, the data obtained from the laboratory tests and the field testing of the demonstration bridges (*2*). The resulting MBISB design criteria is specifically for LVR applications accommodating structures spanning 40 ft to 80 ft with widths of 26 ft and 32 ft, being considered (i.e. accommodating two lanes of traffic). A discussion of the assumptions and steps taken to develop the MBISB design criteria are presented in this chapter. A series of MBISB designs were then developed addressing the following variables: length, width, girder spacing, steel yield strength, concrete compressive strength and concrete cover. The resulting designs are presented in tabular format in the Design Manual "Investigation of the Modified Beam-in-Slab Bridge System, Volume 2 of 3" and are referred to as the design output (*10*).

4.1 Design Loads and Distribution

4.1.1 Design Loads

The flexural and shear requirements are based on the AASHTO LRFD HS-20 truck or tandem loading and the required 0.64 k/ft lane load. The vehicular and lane loads are factored to account for several strength and serviceability conditions as specified by the AASHTO LRFD Bridge Specification (*2*).

4.1.2 Distribution Factors

The distribution of moment and shear to the longitudinal girders is governed by AASHTO LRFD Bridge Specification distribution factors for a concrete slab/steel girder bridge. When compared to field and laboratory test data and analytical results, the AASHTO distribution factors are conservative, but are still considered to be applicable for the MBISB design. Critical distribution factors for both the interior and exterior girders were calculated for single and multiple lane loadings for both shear and moment (*2*).

4.1.2.1 Moment Distribution Factors

The interior girder distribution factors are calculated following the relationships presented in the AASHTO LRFD specifications for both single and multiple lane loadings. Single lane moment distribution factors for the exterior girders are calculated by the two indicated methods and compared for maxima as required by AASHTO LRFD specifications. For slab girder bridges with diaphragms or cross frames, a linear rotation is applied about the center of gravity (CG) of the cross section and the resulting exterior girder reaction, which is taken as the distribution factor, is calculated. The second method for the single lane loading involves the application of the lever rule about the exterior girder.

The controlling exterior distribution factor for a multiple lane loading is also calculated by applying two criteria. For the first, the interior multiple lane value is modified by the specified factor which is influenced by the distance between the exterior girder and the edge of the curb. The linear rotation method is also applied with two design trucks occupying the bridge deck; the larger value controls the exterior girder design.

All distribution factors have been reduced by a factor of 0.9 per AASHTO LRFD specifications due to the low ADTT that LVR bridges experience. A multi-presences factor of 1.2 is applied to the single lane loading cases for the lever rule and the linear rotation while a factor of one is applied for the two lane loading. The multi-presence factors account for the statistical probability that all lanes would be experience a maximum loading at the same time *(2)*.

4.1.2.2 Shear Distribution Factors

The shear distribution factors are determined using methods similar to the moment distribution. Interior girder distribution factors for both single and multiple lane loadings are calculated by applying relationships directly from the Specification. Similar to the moment distribution factors, the exterior values are calculated by applying a linear rotation and the lever rule for the single lane loading case. For a two lane loading, the controlling shear distribution factor results from either the linear rotation or the modified interior multiple lane distribution value. The appropriate multi-presence and ADTT factors are also applied to the shear distribution values (*2*).

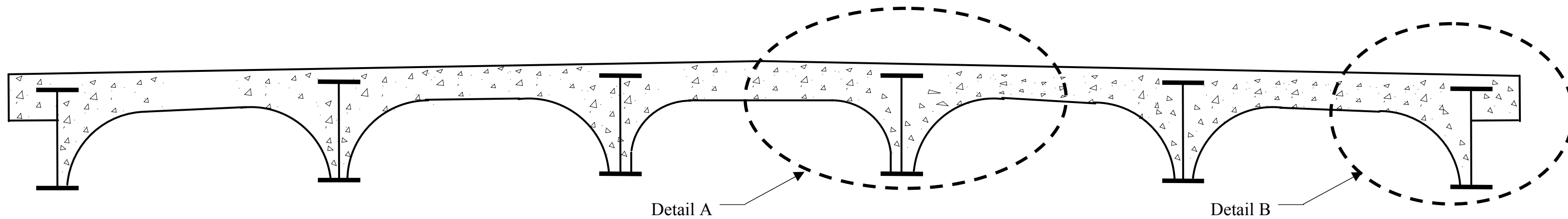
4.2 Section Property Calculations

The MBISB design utilizes a transverse arch with a circular radius between the girders to reduce the self weight of the section and introduce arching action as the mode of structural resistance in the deck. To simplify the calculation of the sectional properties, the geometry of the circular section is estimated by a series of rectangles and triangles. The concrete self weight is calculated based on the estimated cross sectional area; the tributary width of a respective cross section area is set equal to the girder spacing (S). The effective flange width of the composite section, assumed to be equal to the girder spacing, is applied for the moment of inertia calculations. The effective flange width assumption was verified by the approximately linear transverse strain profile measured during both the laboratory and field testing.

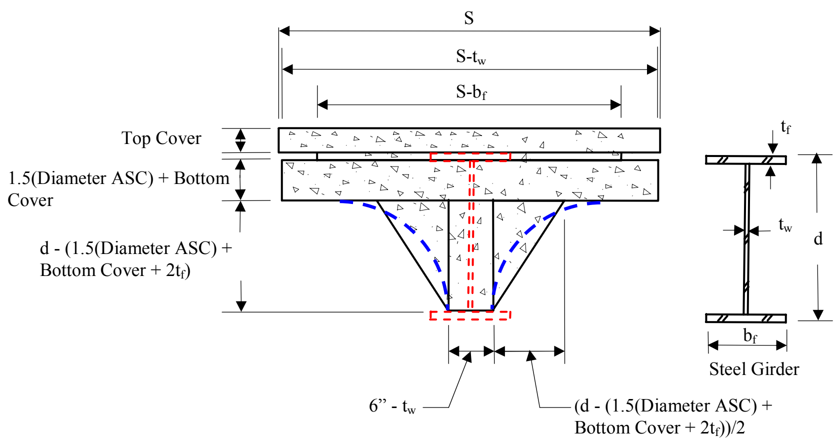
4.2.1 Cover and Slab Depth

Two cases of top cover, zero and 3 in., are considered for the design criteria. The 3 in. top cover is in reference to the depth of concrete placed over the top flanges of the girders. The 3 in. cover depth is to protect the reinforcement and the girders from corrosion while increasing the flexural rigidity of the section. Implementing a design with the 3 in. cover results in a more involved construction process as opposed to the zero cover case where the concrete is struck off even with the top flanges of the girders as in the original BISB design (*1*). Typical approximated cross sections of the two cover depths (3 in. cover and zero cover) for both the interior and exterior sections are presented in Figures 4.1 and Figure 4.2 The sections are divided into approximate individual shapes that are used to calculate the cross sectional area and gross flexural properties.

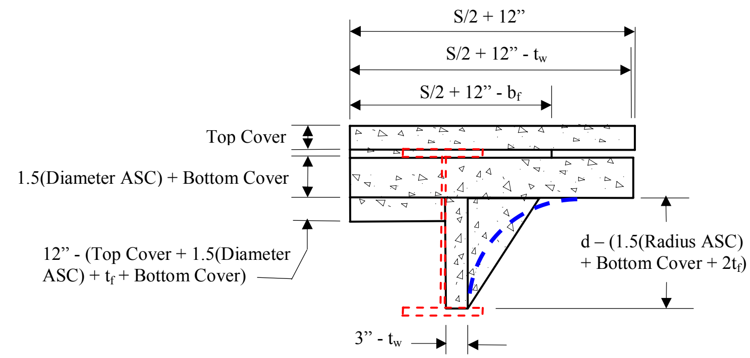
The minimum slab depth is determined by adding the top cover depth, the flange thickness of the longitudinal girders, 1 1/2 times the diameter of the ASC holes, and the bottom cover as listed in Equation 1.



a. Actual cross section of a MBISB design

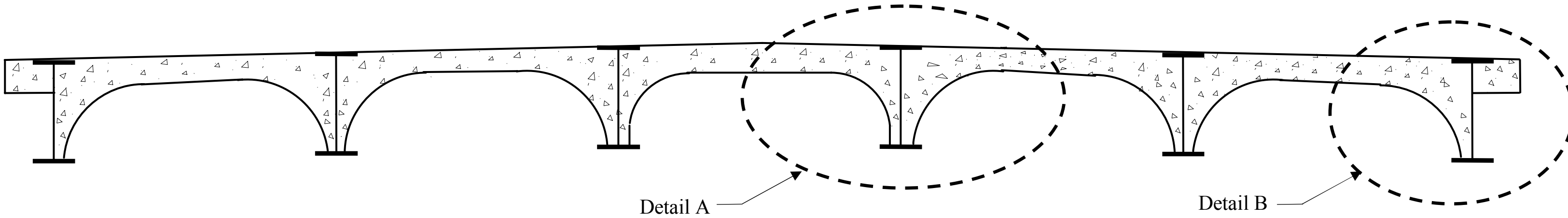


b. Detail A: Approximated interior cross section

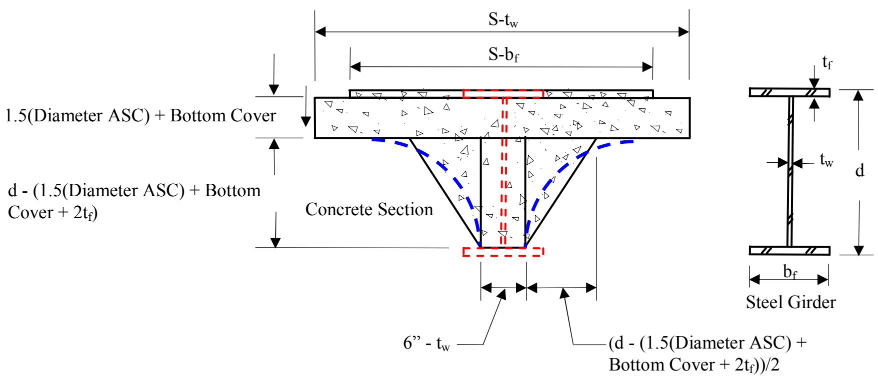


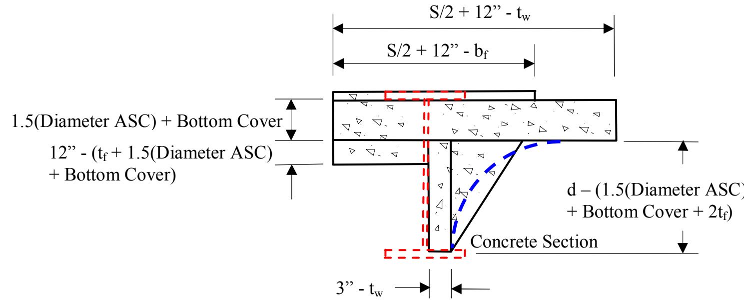
c. Detail B: Approximated exterior cross section

Figure 4.1. Approximated section properties of the MBISB with 3 in. of cover over the top flange of the girder.



(a) Actual cross section of a MBISB design





(b) Detail A: Approximated interior cross section

(c) Detail B: Approximated exterior cross section

Figure 4.2. Approximated section properties of the MBISB with no cover over the top flange of the girder.

$$\text{Minimum Slab Depth} = (t_c + t_f + 1.5d_{ASC} + b_c) \qquad \text{(Eqn 1)}$$

Where :
t_c = top cover, in.
t_f = top flange thickness , in.
d_{ASC} = diameter of ASC hole, in.
b_c = bottom cover, in.

The concrete flange portion is divided into 3 horizontal sections if there is top cover and 2 sections if there is no cover to account for the concrete displaced by the steel girder.

4.2.2 Arch Radius

The radius of the arched section (Equation 2) is approximated by calculating the remaining depth of the concrete section which is equal to the total deck thickness plus the thickness of one steel section flange subtracted from the cover plus the steel section depth. The resulting radius value is rounded down to the nearest 1/2 in. for construction purposes.

$$\text{Arch Radius} = (t_c + d) - (t_d + t_f) \qquad \text{(Eqn 2)}$$

Where:
d = depth of steel section, in.
t_c = top cover, in.
t_d = total deck thickness, in.
t_f = bottom flange thickness, in.

The individual triangles that approximate the circular sections are assumed to project horizontally a distance equal to 1/2 the arch radius. The area and second moment of area of the triangles are calculated as an estimate of the circular section properties. A series of calculations were performed to compare both the area and second moment of area with those of the actual circular arch and were found to be within +/ - 5% of the values found by the approximate method.

The circular arched formwork sections are constructed from corrugated steel normally used in the fabrication of CMP. The radius of the arch for a given section is calculated and listed in the tabular output of each selected bridge design and can be found in the Design Manual, (Volume 2) (*10*). The final segment of the cross section is taken as a rectangle with a height equal to the arch radius. The width of the rectangle accounts for the concrete cover over the web and is approximated as 3 in. minus the web thickness.

4.2.3 Section Property Calculations (Flexural Rigidity)

Gross transformed composite moment of inertia values were calculated for both short and long term flexural properties by transforming the concrete in the section to steel properties through an integer modular ratio calculated using Equation 3.

$$n = \frac{E_s}{E_c} \qquad \text{(Eqn 3)}$$

Where:
n = modular ratio
E_s = Steel Modulus of Elasticity
E_c = Concrete Modulus of Elasticity

The long term reduction in concrete stiffness due to shrinkage effects was estimated by increasing the modular ratio by a factor of three. The top and bottom flexural section modulus for the interior and exterior sections were calculated for both the short and long term sections as well.

4.2.4 Self Weight Calculations

The estimated cross sectional areas are applied to calculate the self weight of the designed structure; each girder is assumed to carry a tributary width equal to the girder spacing. Additional assumptions are made to account for possible future overlays and the guardrail system. The future overlay is estimated as a 1 1/2 in. thick concrete overlay over the whole bridge resulting in an additional 18.75 psf uniform loading. The guardrail system is estimated as 100 plf on each side of the bridge for a total loading of 200 plf distributed evenly amongst all the girders. The guardrail loading is based on the assumption that a steel guardrail similar to the demonstration bridges will be used. A steel Thrie Beam guardrail system was used on the MBISB demonstration bridges resulting in a self weight below the assumed 100 plf. Alternative guardrails employed on LVR bridges have included channels and W sections which are also less than the 100 plf estimate. The use of concrete crash barriers (Jersey type barriers) has not been considered in the analysis or design of the MBISB system; if it is desired to use such a guardrail system, an updated analysis is required.

4.3 Strength I Design Conditions

AASHTO LRFD Bridge Specification requires the longitudinal girders to resist the factored Strength I loading condition for both shear and moment. The factored loading includes the self weight, the distributed lane load and the distributed truck/tandem live load moment increased by the impact factor. The interior design moment will control the flexural and shear design in all cases due to the larger dead loads (overlay and self weight) when combined with the controlling interior lateral load distribution factors. The resulting design value for both shear and moment is reduced by a 0.95 load modification factor accounting for the redundancy, ductility and the operational importance of the structure (*2*).

4.3.1 Section Yielding

The maximum flexural resistance of a section for the Strength I condition is taken as the full plastic moment per AASHTO LRFD Bridge Specification (*2*). The considered section must first undergo yielding before reaching the plastic resistance mode; therefore, the maximum stress in the top and bottom flanges of the steel section resulting from the applied Strength I moment were calculated using the respective gross section moduli. If the section yields, the plastic neutral axis (PNA) is calculated by balancing the compression and tension forces in the composite section assuming full plastic behavior. All concrete below the neutral axis (tension side) is assumed to be cracked and thus does not contribute to the plastic flexural capacity. The plastic moment is calculated by multiplying the resulting plastic forces by the corresponding distances to the PNA. The resulting plastic moment is then compared to the Strength I moment demand. If the resulting plastic moment is larger than the Strength I moment demand, the Strength I flexural moment condition is satisfied, if not, a larger section is required.

4.3.2 Flexural Ductility

The AASHTO LRFD Bridge Specification requires a check to ensure adequate ductility of the sections in flexure (*2*). The check is to ensure the tension flange of the steel girder will reach strain hardening prior to the extreme compression concrete reaching crushing strain. The values are based on a linear strain profile about the plastic neutral axis with the tension flange strain being set equal to 0.012 which is taken as the onset of strain hardening. The corresponding concrete strain is then calculated and compared to a limiting factored strain value of 0.0018. If the specified criterion is not met, a more detailed analysis is performed to ensure that the concrete strain remains below the crushing strain of 0.003.

4.4 Constructability

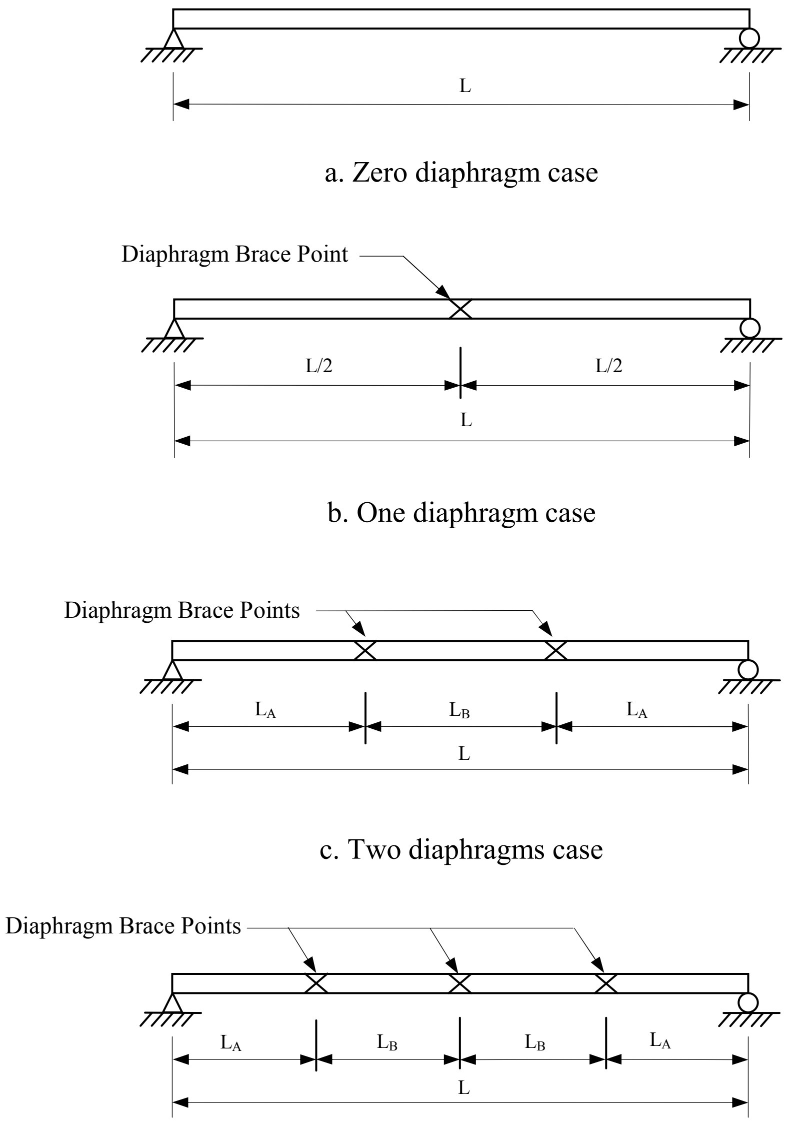
4.4.1 Construction Loads

Both strength and serviceability criteria must be met during the construction phase. Factored loads that must be considered for the construction phase are the self weight of the concrete, the steel girders and a uniformly distributed construction load of 25 psf to account for the formwork, workers and machinery that add temporary load to the structure. The construction load calculation assumes the custom rolled arched formwork system will be used to create the transverse arch. The uniformly distributed construction load is multiplied by the full spacing for all girders to account for the increased load on the exterior girder due to the exterior formwork and the weight of the power screed used to strike off the concrete which is supported by the exterior girder. The self weight of the structure is factored by 1.25 and the uniform construction loading is increased by a 1.5 factor as per AASHTO LRFD Bridge Specification (*2*).

4.4.2 Compression Flange Bracing (Lateral Torsional Buckling)

Proper compression flange bracing to prevent lateral torsional buckling (LTB) of the compression flange is extremely important during the deck placement. At this stage of construction, the compression flange of the girder must be appropriately braced to prevent a catastrophic failure. AASHTO LRFD design criteria ignoring the St. Venant's torsional contribution to resisting LTB is applied (AASHTO LRFD Bridge Specification [A6.10.6.4.1]) (*2*). The moment gradient factor (C_b) is included in the analysis, increasing the allowable unbraced length (l_b) depending on the applied moment and bracing condition. Steel diaphragms with a depth ≥ 1/2 the depth of the longitudinal girders and a kl/r ratio of < 140 are required at the designated brace points to prevent LTB. The end condition (k) is conservatively taken as one and the length (l) is set equal to the girder spacing. The radius of gyration (r) about the plane of buckling is an inherent section property.

The developed design methodology considers four possible bracing configurations with either zero, one, two or three diaphragms present. The positioning of the four bracing conditions and the corresponding unbraced lengths, which are listed in the Design Manual, (Volume 2) (*10*), are presented in Figure 4.3. For designs where one diaphragm is required, the steel diaphragm section is assumed to be placed at the midspan of the bridge, reducing the unbraced length (l_b) to L/2. For design cases where two diaphragms are required to meet the compression flange bracing criteria, the span is divided into three approximately equal sections with the unbraced length of the middle section (L_B) determined first and rounded down to the nearest whole foot. The remaining distance ($2*L_A$) is then equivalently divided (see Figure 4.3c). When three diaphragms are required, the two interior lengths are set equal to L_B and the remaining length is equally divided resulting in the L_A distances. In the design output, only the unbraced length L_B is listed, leaving the remaining lengths to be calculated by the designer.



a. Zero diaphragm case

b. One diaphragm case

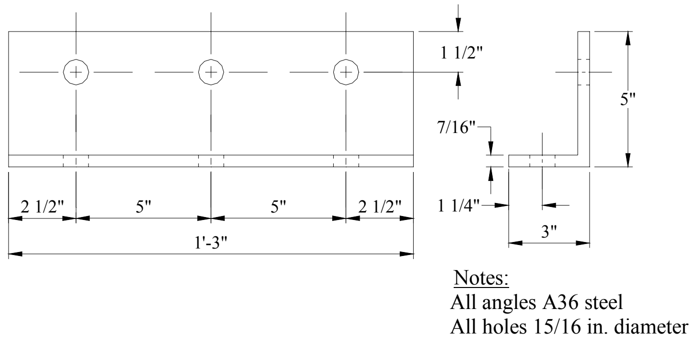
c. Two diaphragms case

d. Three diaphragms case

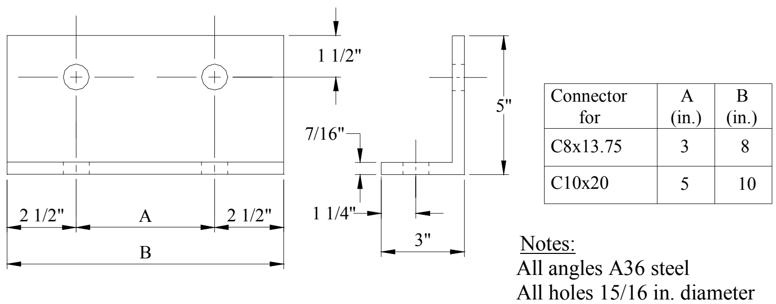
Figure 4.3. Possible diaphragm layouts for the MBISB.

4.5 Diaphragms

The diaphragms (depth of which are based on the depth of the longitudinal girders) are assumed to be compression members and therefore must meet the previously stated kl/r ratios. Since the diaphragms are to be bolted to the longitudinal girders, a suggested angle connector is presented in Figure 4.4. All the connectors are constructed from 5x3x7/16 in. A36 angle sections



a. Diaphragm connector for C15x33.9 section



Connector for	A (in.)	B (in.)
C8x13.75	3	8
C10x20	5	10

b. Diaphragm connector for C10x20 and C8x13.75 sections

Figure 4.4. Diaphragm connection details.

and are attached with 7/8 in. diameter A325 structural bolts; washers must be installed under the nuts.

If the design is not skewed, the clip angles can be bolted “back to back” reducing the number of bolts and holes required. To accommodate the elevation difference between the longitudinal girders, the diaphragms are sloped and bolted in as a ‘construct-to-fit’ detail. The positioning of the sloped diaphragms is presented in Figure 4.5; the specified diaphragm and 5x3x7/16 in. angle connector sections based on the depth of the longitudinal girders are listed in Table 4.1. The designer has the option to select alternative sections and connections that are structurally equivalent to those specified. This option was exercised in the case of the second demonstration bridge (MBISB 2) where S18x54.7 sections were used for the diaphragms and WT10.5x31 sections were used for the connectors due to their availability.

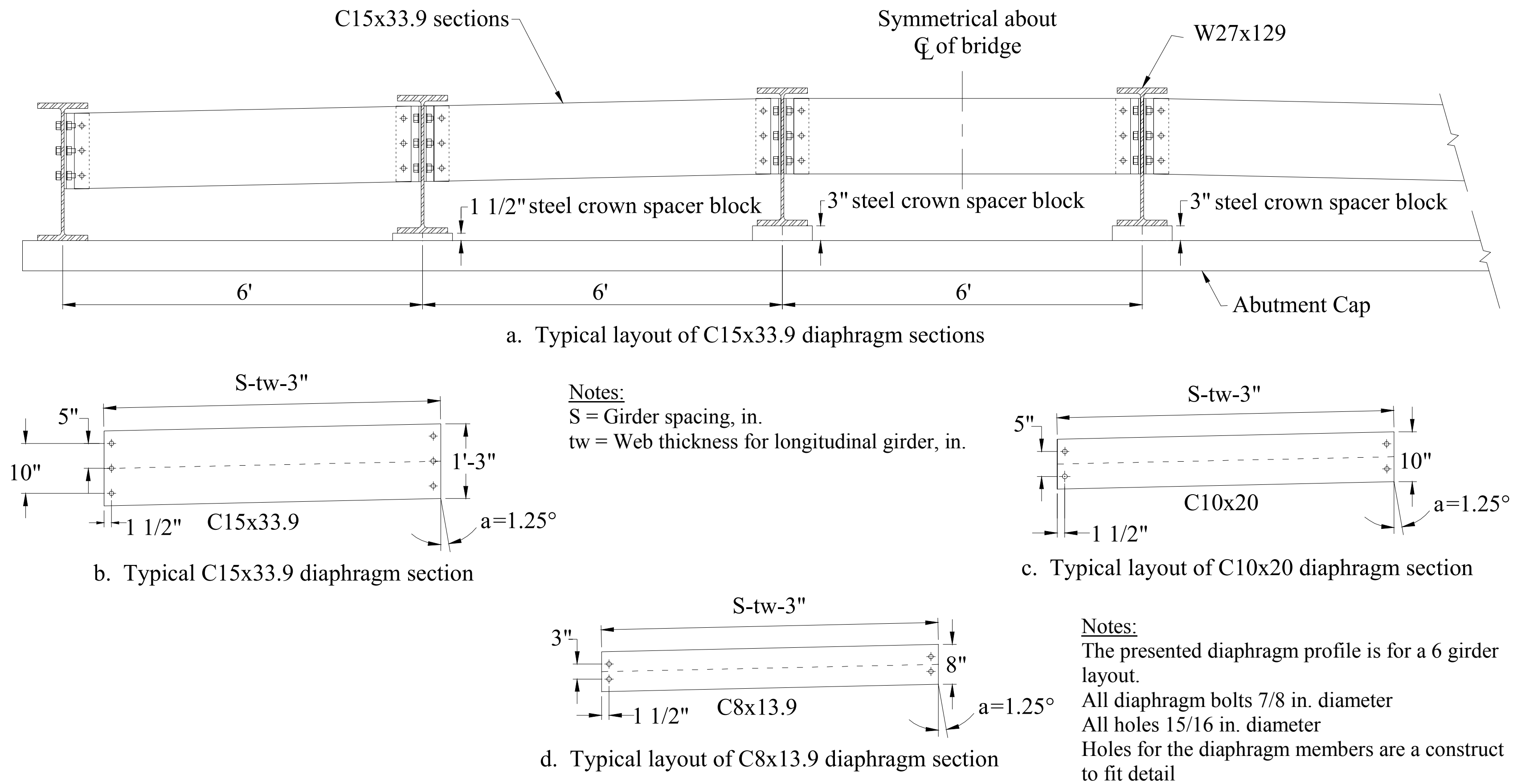


Figure 4.5. Profile of the suggested diaphragm system for the MBISB design.

Table 4.1. Specified diaphragm sections and connectors based on longitudinal girder depth (A36 steel).

Longitudinal Girders	Diaphragm Sections	Diaphragm Connector Length (in.)
W24 and Up	C15x33.9	15
W18 and W21	C10x20	10
W12 and W14	C8x13.75	8

4.6 Lateral Loading

4.6.1 Wind Loading

A lateral wind loading is estimated for a 100 mile per hour wind acting on a bridge structure 20 ft above the surface of the water. Following AASHTO LRFD Bridge Specifications the lateral force acting on the side of the girder bridge is determined (*2*). The resulting estimated sliding force transferred laterally through the deck to the abutments. The wind pressure is factored by 1.4 to evaluate the Strength III limit state.

4.6.2 Water Loading

Many LVR bridges span intermittent streams whose flow can fluctuate drastically to the point of inundating the structure. For this reason, calculations are performed to estimate a lateral water force if the superstructure were submerged by 6 in. with a 10 mph flow striking the bridge. The wind loading is not included since if the bridge is under water, the majority of the area the wind can strike is removed. Similar to the wind loading, the sliding force due to the combined static and kinematic water pressure is calculated and transferred through the deck to each abutment. The resulting sliding force due to the water will always be larger than the wind loading and must be resisted to prevent the bridge superstructure from being pushed from the substructure.

4.7 Optional Deflection Control

AASHTO LRFD Bridge Design criteria do not provide a limiting live load deflection criterion but instead recommends a maximum allowable live load deflection of L/800 (*2*). An estimated midspan deflection is calculated with the design truck centered about the bridge midspan and applying the short term composite section flexural rigidity. A given section need not be immediately ruled out if the deflection criterion is not met if the designer can accept a possible larger live load deflection.

4.8 Service Limit State Control of Permanent Deflection

AASHTO LRFD Bridge Specifications require the compliance of not only strength limit states but serviceability limit states as well (*2*). To ensure the structure does not undergo excessive long term permanent deflections, the combined stress effects due to factored service level loadings are investigated. The Service Level I limit state requires the steel stresses in the extreme tension fibers to be less than 95 percent of the steel yield stress; the proposed section is rejected during the design if the service level stress is exceeded.

4.9 Dead Load Camber

Camber is introduced to the girders to counteract the significant deflections resulting from the structure's self weight. The deflection is based on the self weight, additional overlay, and the distributed exterior guardrail loads. Deflections for both the interior and exterior girders are calculated, then rounded up to the nearest 1/4 in. The total exterior girder camber is set equal to the rounded interior girder camber and the camber of the interior girders is then increased by 1/2 in. resulting in a larger camber when compared to the exterior girders. The extra 1/2 in. of camber is added to the interior girders to provide a factor of safety against a final concave transverse profile. The diaphragms are expected to be bolted tight to a slip critical condition to force a uniform deflection between the girders when the plastic concrete is placed.

4.10 Buoyancy Check

Calculations are performed to ensure the superstructure will not float if the water depth reaches an appropriate level. A buoyancy check is performed by assuming a worst case scenario with all the transverse arches between the longitudinal girders filled with air and the water level set equal to the driving surface of the bridge. In all cases, (i.e. various bridge lengths and widths) the weight of the structure is sufficient to prevent the bridge from floating off the abutments. However, the designer must keep in mind that the buoyancy force will greatly reduce vertical reactions due to the self weight, thus reducing frictional resistance of the support.

4.11 Backwall Design

Concrete backwalls were constructed at each end of MBISB 2 to retain the approach soil. A description of the design utilized for the backwall system in the MBISB 2 bridge follows. The backwall design chosen for use in other MBISBs is left to the discretion of the designer; a suggested 12 in. thick backwall cast between the longitudinal girders is shown in Figure 4.6. For the backwall design used in MBISB 2, closed loop stirrups (#3 Grade 60 deformed reinforcing bar) were spaced at 12 in. centers to provide confinement and shear resistance. A complete

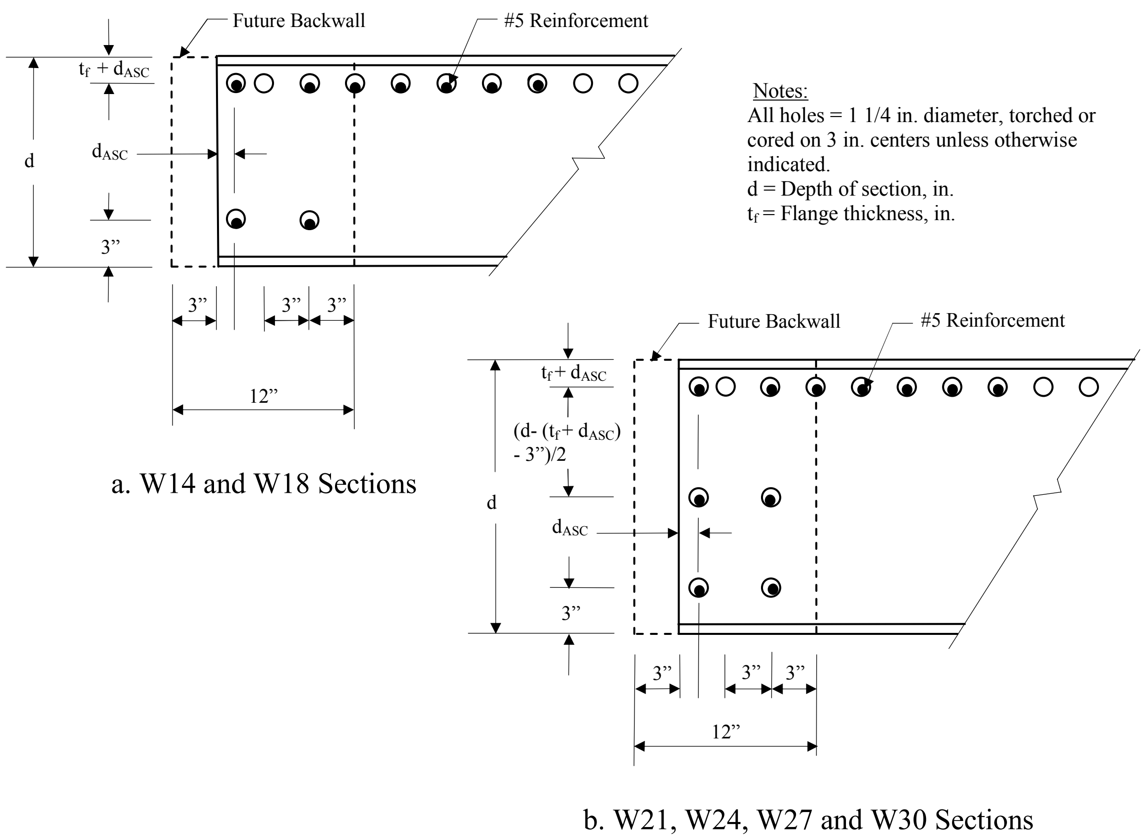


Figure 4.6. Layout of backwall reinforcement for future MBISBs.

listing of the needed reinforcement for a selected future design can be calculated following the information presented in the Design Manual, (Volume 2) (*10*). Typical wooded formwork panels and standard 12 in. snap ties were used to form the backwall. The formwork panels on the soil side of the backwall were constructed high enough to accommodate the final deck elevation.

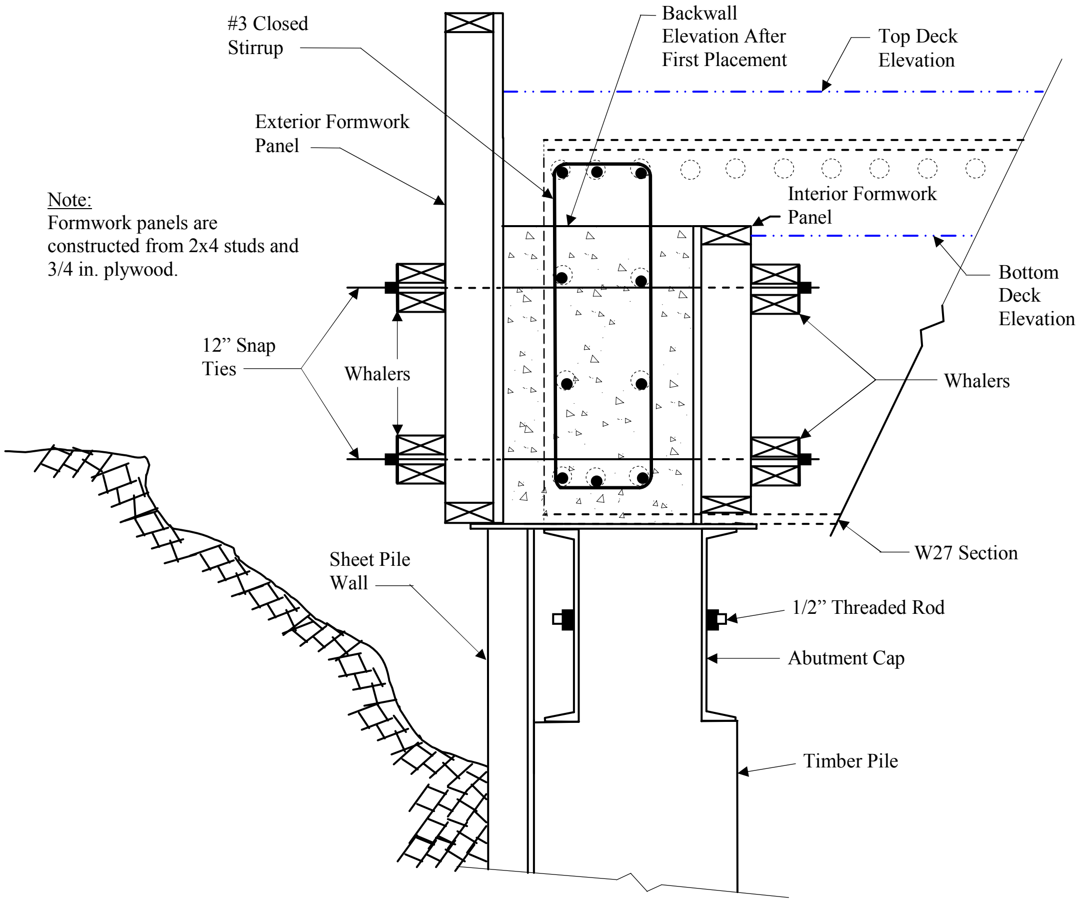
During construction, the interior side of the backwall served as a vertical surface to restrain the interior arched formwork. Thus, the backwall was placed in two stages; the first placement raised the backwall elevation to approximately 1 in. above the bottom of the future deck. The interior backwall formwork was removed after the concrete had adequately cured and the custom rolled arched formwork was matched to the backwall. The remaining portion (second placement) of the backwall was completed when the deck concrete was placed. The backwall formwork setup used in MBISB 2 is presented in Figure 4.7a and the abutted custom rolled formwork is presented in Figure 4.7b. The backwall reinforcement presented in Figure 4.7 is greater than the reinforcement in Figure 4.6; after reviewing the conservative backwall design for MBISB 2, the amount of reinforcement required was reduced.

4.12 Interior Arched Formwork Design

Both demonstration bridges utilized the transverse arched formwork constructed from galvanized corrugated metal removing a majority of the ineffective concrete. A stay-in-place formwork system was used for MBISB 1 as opposed to the removable/reusable custom rolled arched formwork used in MBISB 2.

4.12.1 Stay-in-Place MBISB 1 Formwork

As previously stated, the arched formwork for MBISB 1 was cut from 24 in. diameter, 16 gauge CMP; the section of the CMP used is shown in Figure 4.8. The depth and angle for the arched section can be laid out on a plywood jig to facilitate marking and cutting the arch from the CMP. The arched sections rested on the bottom flange of the longitudinal girders and were held in place by 1 in. angle clips welded to the top of the bottom flange. Concrete is placed over the



a. Cross section of backwall formwork

b. Custom rolled arched formwork abutted to the backwall

Figure 4.7. Backwall construction for MBISB 2.

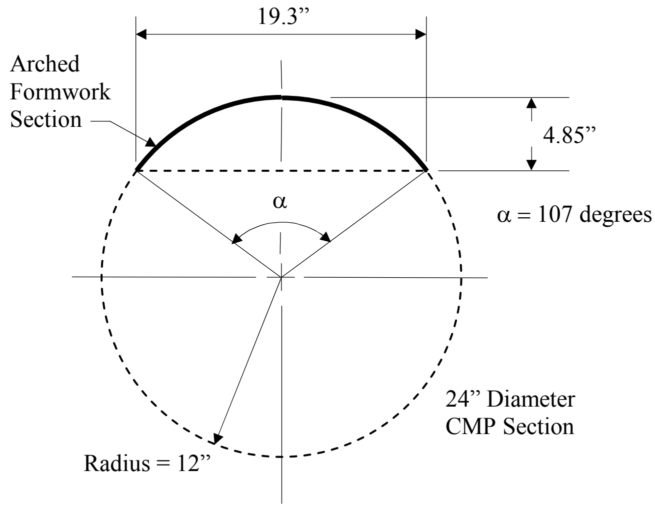


Figure 4.8. Stay-in-place formwork section cut from the 24 in. diameter CMP used in MBISB 1.

formwork, permanently encasing the CMP. The use of stay-in-place formwork in Iowa is limited due to concerns relating to the inspection of the deck undersurface.

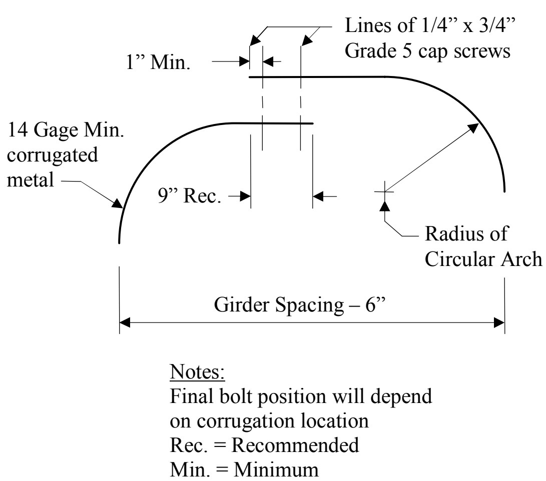
Using a stay-in-place formwork has the benefit of reducing construction time; but readily applicable girder depths and spacing are limited. Based on the developed design criteria, two girder depths and spacing are readily applicable to using a 1/2 section of CMP resulting in a stay-in-place formwork system. A stay-in-place formwork system for W21 sections spaced at 3 ft and W27 sections spaced at 4 ft can readily be formed from 1/2 sections of 14 gauge CMP with 30 in. and 42 in. diameter sections, respectively.

4.12.2 Custom Rolled MBISB 2 Formwork

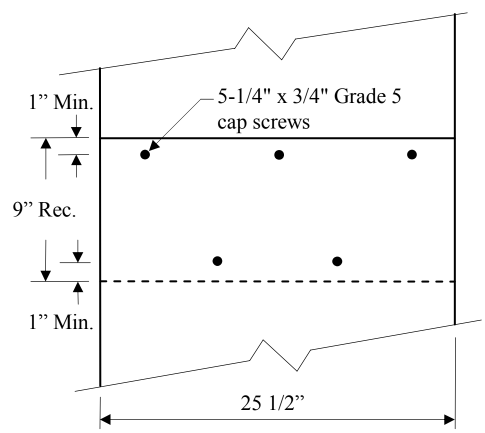
The transverse arched section between the longitudinal girders was developed to remove a majority of the ineffective (tension) concrete from the cross section and change the mode of structural resistance. A removable/reusable formwork system which maximized the amount of concrete removed from the section while forming the arch was desired. The custom rolled arch formwork section was developed and constructed from the same galvanized corrugated material as CMP. The formwork was constructed by first assembling two individual section components and then joining the resulting arches to form batteries of formwork.

4.12.2.1 Individual Arch Sections

The two individual components of an arched section were formed by rotating a 24 in. wide (nominal) corrugated metal blank through a 90 degree turn in a rolling machine set to the radius of the arch. The manufactures that rolled the custom sections for this project are listed in Appendix A. A jig was constructed to aid in the assembly of the individual arched sections which were bolted together with 5 – 3/4 x 1/4 in. Grade 5 cap screws. The cap screws were placed with the nut backed by a washer on the concrete side; after the deck concrete had cured, the bolts were undone and the individual components removed. The individual pieces were assembled at the rate of 6 per hour by one worker. The jig and the assembly sequence of the individual components can be viewed in Figure 4.9.



a. Exploded view of circular arch formwork section



b. Layout of cap screws for the individual formwork sections

Figure 4.9. Assembly of the individual arched sections.

c. Jig for assembling individual components

d. Jig with first component in position

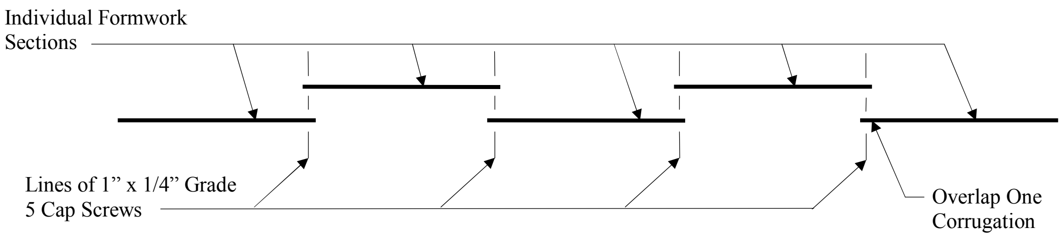
e. Individual components clamped in preparation of drilling the bolt holes

f. Completed individual arched section

Figure 4.9. Continued.

4.12.2.2 Arched Formwork Batteries

To reduce in field construction time, the individual arch sections were assembled into batteries of 8 ft or 10 ft (4 or 5 individual sections). The individual sections were joined together with four 1 in. x 1/4 in. Grade 5 bolts per joint and were positioned together in an 'over/under' configuration as shown in Figure 4.10a to prevent the entrapment of the individual sections. Each battery had four #9 wire lift loops installed to facilitate their placement. The batteries were then prepared for transportation and stored. The assembly process required 75 minutes with two workers to per battery complete and is documented in Figure 4.10b-e.



a. Typical 'over/under' battery layout configuration

b. Individual arch section fitted in the jig

c. Fitting individual sections in position

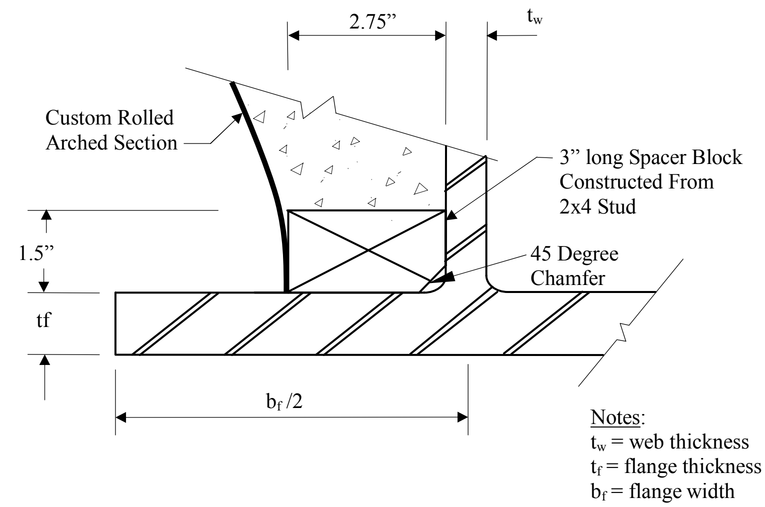
d. Bolting the individual sections together

e. Moving completed battery to storage

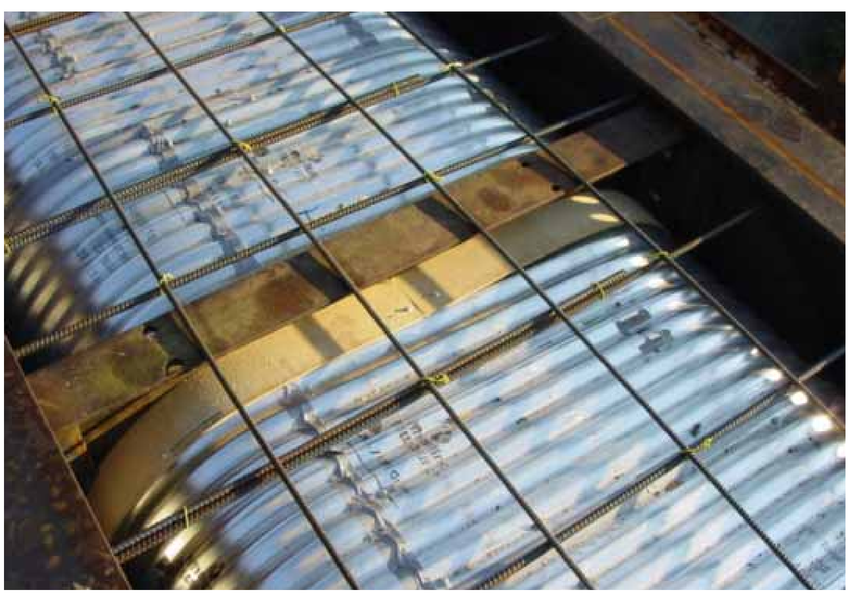
Figure 4.10. Assembly of the formwork batteries.

4.12.2.3 In Field Construction Sequence

The batteries were placed directly into the bridge from the transport vehicle and held in place with spacer blocks set on 2 ft centers (see Figure 4.11a). The batteries were placed with a 2 ft space between the adjacent batteries which allowed worker access to adjust the battery's position and prevented the entrapment of a section once the concrete was placed; this space was eventually covered with an individual arched section. Gaps at the diaphragms and backwalls were covered with 1/4 in. thick plywood held in place with self-tapping screws; the shimmed gaps can be viewed in Figure 4.11b.



a. Installed spacer block



b. Shims at the diaphragm

Figure 4.11. Completing the interior formwork for MBISB 2.

4.12.3 Concrete Placement Sequence

Formwork deflections under the weight of the plastic concrete were estimated using a structural model created in STAAD Pro, a structural analysis software program (*13*). Of particular concern are the deflections of the arch leg due to the lateral pressure of the plastic concrete causing a deflection large enough that a leg of the formwork could fall off the supporting bottom flange resulting in a catastrophic failure.

Measures to prevent such a failure include eliminating girders with a flange width of less than 8 in. from consideration in the design methodology. In addition, the legs of the arched formwork must not be curled towards the edge of the bottom flange; this curling may result during the rolling process and must be corrected prior to installation.

A specific concrete placement sequence similar to that used for MBISB 2 is also specified and consists of first placing a windrow of concrete on the top portion of the formwork to engage the arching action by spreading the legs of the arch so that they are against the spacer blocks. A 12 in. lift of concrete is then placed on each side of the arch to balance the lateral pressure of the plastic concrete. After adequate vibration, a second lift is placed in the same fashion, raising the concrete elevation to the bottom of the ASC holes. The concrete is vibrated and a final lift added bringing the concrete up to the deck surface elevation.

4.13 Concrete and Steel Quantities

The volume of concrete required is estimated by multiplying the number of girder sections by the applicable previously approximated cross sectional area by the bridge length (centerline to centerline of abutments). The resulting volume is rounded up to the nearest integer and an additional cubic yard is added to account the amount of concrete “lost” in construction. The concrete needed to perform quality control tests is not included in the design output and must be included by the designer based on the number and type of tests performed. Additional concrete will also be needed to construct the required backwalls. Structural steel quantities for the girders are calculated based on the full length of the girders (assumed to be 1 ft greater than the centerline to centerline span length) and the weight of the W sections.

4.14 Deck Reinforcement

Due to the transverse arched deck, the mode of resistance to wheel loadings is an arching action as opposed to flexure. The arching action places the concrete in compression reducing the amount of deck reinforcement required. As reported in Chapter 2, laboratory specimens were subjected to simulated wheel loads with the only reinforcement in the deck being the transverse reinforcement needed to complete the ASC. The force needed to fail the transverse arched deck was found to be several times larger than a factored wheel load which indicated that the deck reinforcement, compared to that required in traditional reinforced concrete decks, can be greatly reduced. Therefore, the structural reinforcement in MBISB 2 was limited to the transverse reinforcement necessary to complete the ASC. When compared to similar traditional concrete decks, the transverse arch reduces the amount of structural reinforcement required by approximately 70 percent.

The transverse deck reinforcement is spaced on 15 in. centers (every 5^{th} ASC hole) for the entire length of the girders. At the ends of the bridge, a total of seven lines of reinforcement are required to provide a stiffened edge of the arched section. Canadian researchers have determined that the ends of a deck slab relying on arching action require additional in plane stiffening to provide longitudinal confinement of the arch (*14*). A typical layout of the structural reinforcement is shown in Figure 4.12.

Additional temperature and shrinkage reinforcement was added for crack control in both the longitudinal and transverse directions. Longitudinal temperature and shrinkage steel (#4 Grade 60 deformed reinforcement on 12 in. centers) was added in the five bays of MBISB 2. The required longitudinal temperature reinforcement varies for the different girders spacing and is discussed in detail in the Design Manual, (Volume 2) (*10*).

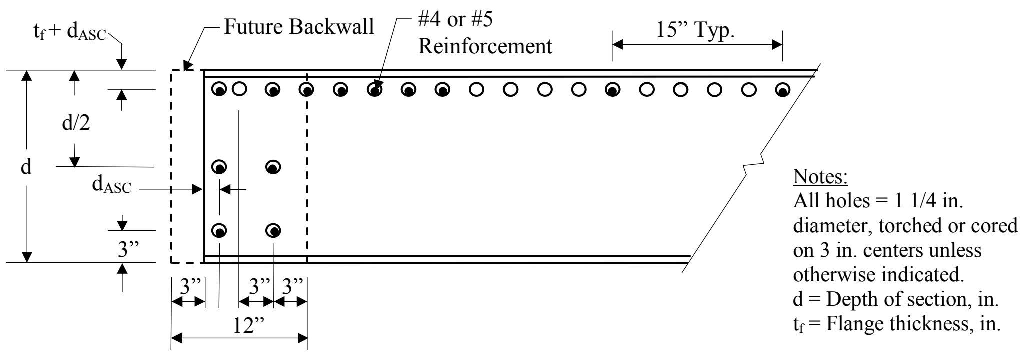


Figure 4.12. Layout of deck reinforcement.

Additional transverse temperature and shrinkage steel is required if the 3 in. cover option is utilized. For the demonstration bridge, #3 Grade 60 deformed reinforcement on 15 in. centers (spaced between the transverse ASC reinforcement) was supported on 3/4 in. chairs placed on the top flange of the longitudinal girders. The purpose of the additional transverse reinforcement is to control cracking due to temperature and shrinkage effects as well as to confine the concrete over the top flange of the longitudinal girders. The installed reinforcement in MBISB 2, both structural and crack control, prior to the concrete placement, can be viewed in Figure 4.13.

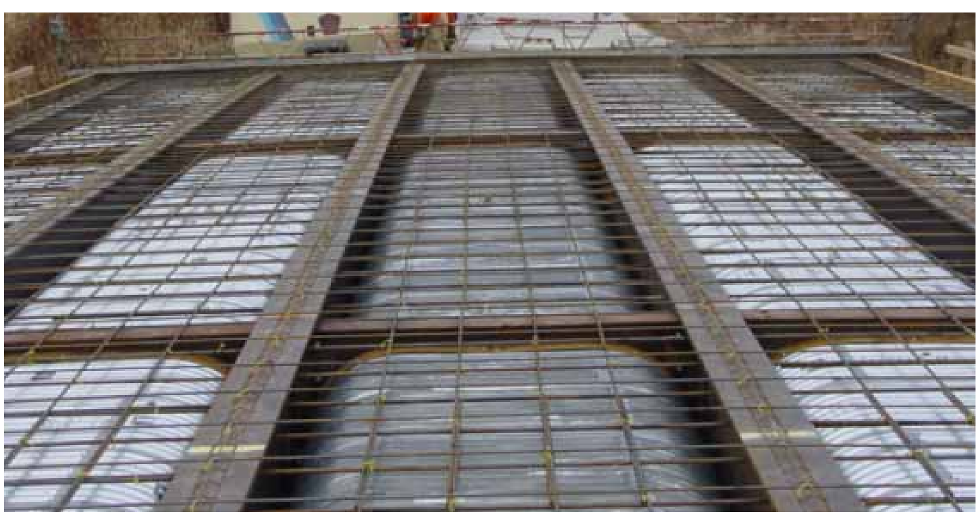


Figure 4.13. ASC and crack control reinforcement prior to concrete placement.

4.15 Exterior Formwork Design

The developed design criteria rely on composite action for all longitudinal girders; therefore, the exterior girders must also have the ASC. In order to get full development of the transverse reinforcement in the ASC, a concrete overhang (12 in. x 12 in.) is needed on the outside of the exterior girders. As previously noted the exterior formwork sets the deck elevation as well as forms the exterior overhang needed to complete the ASC. Additional information on the construction of the exterior formwork and the exterior formwork supports is included in the Design Manual, (Volume 2) (*10*).

4.16 Guardrail Design

The MBISB design methodology does not include the design of the guardrail system; the implemented guardrail system is left to the designer. As previously stated in section 4.2.4, the weight of the implemented guardrail system is assumed to be equal to 100 plf per side for design purposes; the use of a heavier guardrail system will require an updated design. The guardrail systems for the demonstration bridges consisted of rolled galvanized W-Beams and Thrie Beams supported on posts set on 6 ft centers. MBISB 1 utilized a rolled W-Beam crash rail section supported by C8x11.5 channels that were attached to the exterior girder by welding and bolting. MBISB 2 utilized a rolled Thrie Beam crash rail section supported by ST6x3x1/4 tubes held in place by anchor rods embedded in the deck concrete. The respective guardrail systems are shown in Figure 4.14.

a. MBISB 1 guardrail system

b. MBISB 2 guardrail attachment bolts

c. MBISB 2 post and rail

d. MBISB 2 guardrail system

Figure 4.14. Guardrail systems used on the MBISB demonstration bridges.

5. DESIGN

This chapter discusses, with reference to the demonstration bridges, the design of a generic MBISB that meets the criteria specified in the previous chapter. A design methodology was developed to address the specific requirements and with the aid of computer programs, MBISB designs were produced that include the following variables:

- Bridge length
- Bridge width
- Girder spacing
- Steel yield strength
- Concrete compressive strength
- Depth of cover

For a given set of variables that satisfy the requirements presented in Chapter 4, basic design criteria are calculated and summarized in a tabular format (See Table 5.1) for numerous design configurations. From the information listed in an output table, a specific design is selected. Additional steps are then performed using the information provided for the remaining necessary components.

A series of selected design outputs as well as a design example and a Power Point slide show describing the sequence and methodology employed for the construction of MBISB 2 are included in the Design Manual, (Volume 2) (*10*). The Design Manual contains information similar to this chapter with the addition of a numerical design example. This Design Guide (Volume 3), is meant to be a reference to Volume 2 providing information on the development of the MBISB designs presented.

5.1 Organization of the Design Output

Table 5.1 is an example of the generated design output used to specify materials and quantities required for a particular width and length of MBISB. The output was created to provide the designer with several combinations that are applicable for the desired span length.

Table 5.1. Design output example for a 65 ft long, 32 ft wide MBISB.

a. Material and section properties: f_y = 50 ksi, f'_c = 4 ksi, Cover = 3 in.

Number of girders @ spacing	Section	Radius of formwork (in.)	Volume of concrete (yd^3)	Weight of steel (kips)	Interior camber (in.)	Exterior camber (in.)	Number of diaphragms	Diaphragm spacing B (ft)	Service level deflection (in.)	Optional defl. control	Water sliding force (kips)
6 @ 6 ft	W30X116	23.5	91	45.94	4.25	3.75	2	21	0.642	Y	29.6
	W30X124	23.5	91	49.1	4	3.5	2	21	0.615	Y	29.8
	W27X129	20.5	86	51.08	4.25	3.75	2	21	0.752	Y	26.9
	W24X131	17.5	79	51.88	4.5	4	1	0	0.964	Y	23.6
	W30X132	23.5	92	52.27	3.75	3.25	2	21	0.594	Y	29.9
7 @ 5 ft	W30X108	23.5	97	49.9	4	3.5	2	21	0.61	Y	29.4
	W27X114	20.5	90	52.67	4.25	3.75	2	21	0.737	Y	26.6
	W30X116	23.5	98	53.59	3.75	3.25	2	21	0.582	Y	29.6
	W24X117	17.5	83	54.05	4.5	4	1	0	0.935	Y	23.4
	W30X124	23.5	98	57.29	3.5	3	2	21	0.557	Y	29.8
8 @ 4.29 ft	W27X102	20.5	95	53.86	4.25	3.75	2	21	0.718	Y	26.4
	W27X114	20.5	95	60.19	4	3.5	2	21	0.676	Y	26.6
	W24X117	17.5	87	61.78	4.25	3.75	1	0	0.855	Y	23.4
	W21X122	15	80	64.42	4.5	4	1	0	1.09	N	20.7
	W27X129	20.5	96	68.11	3.5	3	2	21	0.623	Y	26.9
9 @ 3.75 ft	W24X103	17.5	91	61.18	4.5	4	2	21	0.85	Y	23.6
	W24X104	17.5	90	61.78	4.25	3.75	1	0	0.851	Y	23.2
	W21X111	15	82	65.93	4.5	4	1	0	1.073	N	20.5
	W24X117	17.5	91	69.5	4	3.5	1	0	0.791	Y	23.4
	W21X122	15	83	72.47	4.25	3.75	1	0	1.007	N	20.7
11 @ 3 ft	W21X93	15	89	67.52	5	4.5	2	19	1.06	N	20.6
	W21X101	15	88	73.33	4.5	4	1	0	0.991	N	20.4
	W18X106	12	80	76.96	5	4.5	1	0	1.33	N	17.7
	W21X111	15	88	80.59	4.25	3.75	1	0	0.938	Y	20.5
	W18X119	12	80	86.39	4.75	4.25	1	0	1.213	N	18

L/800 = 0.975 in.

b. Material and section properties: f_y = 50 ksi, f'_c = 4 ksi, Cover = 0 in.

Number of girders @ spacing	Section	Radius of formwork (in.)	Volume of concrete (yd^3)	Weight of steel (kips)	Interior camber (in.)	Exterior camber (in.)	Number of diaphragms	Diaphragm spacing B (ft)	Service level deflection (in.)	Optional defl. control	Water sliding force (kips)
6 @ 6 ft	W30X116	23.5	73	45.94	3.5	3	2	21	0.822	Y	26.3
	W30X124	23.5	73	49.1	3.25	2.75	2	21	0.783	Y	26.5
	W27X129	20.5	68	51.08	3.5	3	2	21	0.964	Y	23.7
	W30X132	23.5	74	52.27	3.25	2.75	2	21	0.753	Y	26.6
	W27X146	20.5	67	57.82	3	2.5	1	0	0.884	Y	23.5
7 @ 5 ft	W30X108	23.5	79	49.9	3.5	3	2	21	0.78	Y	26.1
	W27X114	20.5	72	52.67	3.75	3.25	2	21	0.948	Y	23.4
	W30X116	23.5	79	53.59	3.25	2.75	2	21	0.739	Y	26.3
	W30X124	23.5	80	57.29	3.25	2.75	2	21	0.704	Y	26.5
	W27X129	20.5	73	59.6	3.25	2.75	2	21	0.865	Y	23.7
8 @ 4.29 ft	W27X102	20.5	77	53.86	3.75	3.25	2	21	0.925	Y	23.2
	W27X114	20.5	77	60.19	3.5	3	2	21	0.864	Y	23.4
	W24X117	17.5	68	61.78	3.5	3	1	0	1.102	N	20.3
	W21X122	15	62	64.42	4	3.5	1	0	1.422	N	17.7
	W27X129	20.5	78	68.11	3	2.5	2	21	0.787	Y	23.7
9 @ 3.75 ft	W24X103	17.5	73	61.18	3.75	3.25	2	21	1.102	N	20.5
	W24X104	17.5	72	61.78	3.75	3.25	1	0	1.099	N	20.1
	W21X111	15	64	65.93	4	3.5	1	0	1.401	N	17.6
	W24X117	17.5	72	69.5	3.5	3	1	0	1.011	N	20.3
	W21X122	15	65	72.47	3.75	3.25	1	0	1.303	N	17.7
11 @ 3 ft	W21X101	15	70	73.33	3.75	3.25	1	0	1.285	N	17.5
	W21X111	15	70	80.59	3.5	3	1	0	1.206	N	17.6
	W18X119	12	62	86.39	4	3.5	1	0	1.583	N	15.2
	W21X122	15	71	88.57	3.25	2.75	1	0	1.121	N	17.7
	W18X130	12	62	94.38	3.75	3.25	1	0	1.446	N	15.5

L/800 = 0.975 in.

The listed designs are meant to be a tool for bridge designers to expediently design a particular bridge with minimal effort. Cost variables including the speed of construction, girder availability, the selected formwork system, etc., can all be considered for the numerous combinations so that the most cost effective design can be obtained.

5.2 Description of the Design Program Output

The output in Table 5.1 contains the information necessary to specify and compare quantities of materials and components needed for a given bridge design. A brief description of the design output presented in Table 5.1 follows:

Length of Bridge: Ranges from 40 ft to 80 ft in 5 ft increments; the length is taken from centerline to centerline of the simple support. If a desired span length falls between the evaluated lengths (i.e. 62 ft), the next available longer span length (65 ft) is to be used for the design.

Width of Bridge: Two deck widths are considered in the design methodology, 26 ft and 32 ft.

Cover: Two cover cases are provided; either a 3 in. cover over the girders or no cover where the concrete is struck off evenly with the top flanges of the girders similar to the original BISB design.

Structural Steel Yield Strength: Two steel yield strengths, 36 ksi and 50 ksi, are provided.

Concrete Compressive Strength: A concrete compressive strength of 4 ksi is assumed for all design configurations.

Number of Girders for Given Spacing: For the given bridge width, the number of girders desired is selected and the corresponding spacing is calculated.

W Section: For a given girder spacing, the first five W sections that satisfy the design criteria are listed by order of weight with the lightest listed first. All girders have at least an 8 in. wide flange and all sections must have a web and flange thickness of at least 0.25 in. The girder depths considered range in depth from 12 in. to 30 in.

Radius of Formwork: The MBISB design methodology was created with the assumption that custom rolled arched sections will be used for the interior formwork. The radius of the arched formwork section is calculated and rounded down to the nearest 1/2 in. for constructability considerations.

Volume of Concrete: The concrete volume needed for the deck is calculated based on the approximated cross sectional area (see Section 4.2); the final volume of concrete is determined by rounding up to the nearest yard and adding an additional yard to account for losses attributed to construction. The listed concrete volume does not consider the concrete needed for the backwalls or quality control tests.

Weight of Steel: The steel weight accounts only for the specified girders and not any reinforcing steel. The weight is calculated for a length 1 ft larger than the centerline to centerline of supports.

Interior Girder Camber: Camber for the interior girders is specified to counteract the deflections due to self weight, overlay, and barrier rail effects. The corresponding deflection is calculated by applying the appropriate section moduli corresponding to the steel section and long term composite section, respectively. The resulting deflection is then rounded up to the nearest 1/4 in. and an additional 1/2 in. of camber is added to ensure a final transverse convex profile.

Exterior Girder Camber: The camber of the exterior girders is set equal to the interior girder camber minus the additional 1/2 in. that is added to the interior girders; therefore, the specified camber for the exterior girders is always 1/2 in. less than the interior girders. The camber criteria were developed based on the assumption that the diaphragms are bolted to a slip critical condition holding the difference in the camber constant.

Number of Diaphragms: The number of diaphragms required to provide adequate compression flange bracing for a given section during the deck placement is listed. If one diaphragm is indicated, the diaphragm is placed at the midspan, reducing the unbraced length to L/2. If two or three diaphragms are needed, the positioning of the diaphragms is provided in the Diaphragm Length B column.

Diaphragm Length B: When two or three diaphragms are required, the girder is divided into three or four sections respectively; the middle section(s) length is/are designated as Length B (L_B). The remaining two lengths, (L_A) are set equal and can be longer than L_B due to an increased moment gradient factor (C_b). The diaphragm layout for the four possible configurations is presented in Figure 4.3.

Service Level Deflection: AASHTO LRFD Bridge Specification suggests the live load deflection be limited to the ratio of L/800. The theoretical midspan deflection is calculated by placing the HS-20 truck about the midspan with the load being equally resisted by the short term composite flexural rigidity of all the girders. The resulting deflection is then compared to the previously calculated live load deflection limit (*2*).

Optional Deflection Control: The limiting deflection criterion is then evaluated by comparing the Service Level Deflection to the calculated AASHTO value. A response of “true” is indicated in the Optional Deflection Control column if the criterion is met. A response of “false” indicates the estimated deflection is larger than the recommended value; the designer can then compare the estimated deflection and the L/800 criterion and apply engineering judgment as to whether to accept or reject the design.

Water Sliding Force: The water sliding force is approximated by assuming 6 in. of water is passing over the bridge at 10 mph. The calculated sliding force is per abutment; the girder/abutment connection should be designed to resist the lateral force.

Once a specific design has been chosen, additional steps must be completed by the designer to complete the design. The following is a discussion of the main action points and some of the lessons learned from the design/construction of the demonstration bridges. This discussion is meant as a guide only and specific construction steps/methods for completing a MBISB design are left to the designer.

5.3 Girder Fabrication

The girders for the design selected from the output listed in Table 5.1 require the following fabrication: cambering and the installation of holes, including those for the ASC, the diaphragm connectors and the backwall reinforcement. Cambering will most likely be performed in a steel fabrication shop. If the designer selects to forego cambering the girders or if recycled girders are utilized, two courses of action can be pursued. If recycled girders are used and/or camber is not introduced to the girders the designer may chose to simply allow the girders to deflect resulting in a concave longitudinal deck surface. If camber is not introduced and the resulting dead load deflections are large, the girders may be shored during construction, which will significantly reduce the dead load deflection and stresses. If the girders are shored, the designer must ensure the compression flanges are adequately braced in both the positive and negative moment regions and the shoring has adequately capacity to resist the construction loads.

The ASC holes (1 1/4 in. diameter on 3 in. centers for the length of the girder) can be either torched or cored with little difference in performance. The typical layout of the ASC holes in a longitudinal girder, which can be installed by either in-house forces or by a steel fabrication shop, are illustrated in Figure 4.12. Installing the holes for the diaphragm connectors and the backwall reinforcement may be performed by the steel fabricator if the quality control of the substructure ensures proper alignment. The installation of the holes for the diaphragm connectors and the backwall reinforcement were carried out in the field for MBISB 2. This proved to be a prudent procedure because of unforeseen abutment alignment issues.

5.4 Diaphragm Design/Construction

The required channel diaphragm sections based on the selected longitudinal girders are listed in Section 4.5. However, the designer has the option of using a structurally equivalent section for the diaphragm, as was the case in MBISB 2. A detailed description of a channel diaphragm design is presented in the Design Manual (Volume 2) (*10*). The diaphragms for MBISB 2 were constructed from recycled S18x54.7 sections, chosen for their availability and excess capacity. The diaphragms were cut to length (66 in.) in the field accounting for the angle resulting from the difference in elevation between the girders due to the effects of the crown and camber.

The diaphragm connections were constructed from WT10.5x31 sections cut from a recycled W21x62 girder; the diaphragm connection used for MBISB 2 is presented in Figure 5.1. The T sections were used due to availability and capacity and were attached in a fashion similar to an angle section with only one leg of the flange and the stem being bolted. Three 3/4 in. diameter structural grade bolts per leg were used to fasten the components together. The diaphragm connectors were fabricated by in-house forces and were installed before the girders were set in place. Fabrication of the diaphragm sections was completed in the field with the connection to the longitudinal girders being treated as a ‘build to fit’ condition.

Diaphragm sections are to be bolted into place to avoid fatigue and unknown stress concentrations associated with welding.

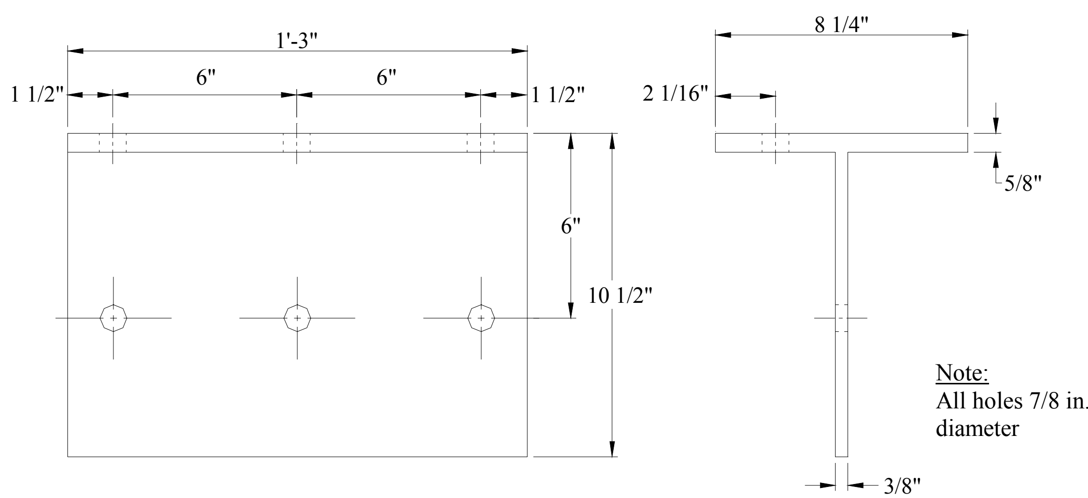


Figure 5.1. Diaphragm connection used in the construction of MBISB 2.

5.5 Formwork Fabrication

The MBISB utilizes formwork that is unique to the design. The formwork can be classified as interior and exterior systems with a majority of both systems being modular. Modular formwork construction is another benefit of the MBISB design, allowing for the preparation of the system during the construction off season and decreasing the in field construction time.

5.5.1 Interior Formwork System

The MBISB deck design differs from traditional slab/girder bridge designs by including a transverse arch spanning between the longitudinal girders. The transverse arch was introduced to remove a considerable amount of ineffective concrete (concrete in tension) from the original BISB cross section. An additional benefit of the arch configuration was the mode of resistance in the deck changed from transverse flexure to an arching action allowing for a significant reduction in the deck reinforcement. Two formwork systems were employed in the forming of the transverse arch; the first system was a stay-in-place formwork while the second was a removable/reusable system referred to as the custom rolled arch formwork.

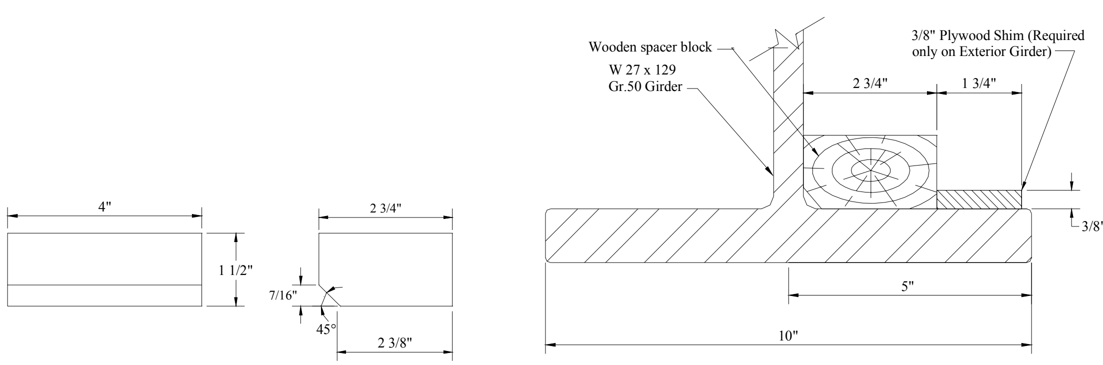
5.5.1.1 Stay-in-place formwork

A stay-in-place formwork system was employed for the first demonstration bridge (MBISB 1). The formwork consists of a section (presented in Figure 4.8) cut from a CMP and is described in greater detail in section 4.12.1. The use of stay-in-place formwork is not an Iowa DOT standard practice and is limited in application for the MBISB design to a few girder spacing and depths due to geometric constraints. Based on the developed design criteria, W21 sections spaced at 3 ft and W27 sections spaced at 4 ft readily accept a stay-in-place formwork system constructed from 1/2 sections of 14 gage 30 in. and 42 in. diameter, 2 2/3 in. x 1/2 in. CMP, respectively.

5.5.1.2 Removable custom rolled formwork

The removable and reusable custom rolled formwork was developed to efficiently form an arch between the girders and was constructed from the galvanized steel (2 2/3 in. x 1/2 in. corrugation) normally used to construct CMP. An individual arch formwork section consists of two 24 in. wide components (nominal width) rolled to the radius specified in the design output, which sets the depth of the concrete deck. Quality control measures must be instigated during the rolling process so the individual components and sections will fit together and ensure the proper deck depth. The individual components are bolted together as described in Section 4.12.2.1 forming the individual arch sections.

To expedite the in field construction process, the individual sections were positioned in batteries consisting of 4 or 5 individual sections with the aid of a jig as shown in Figure 4.10 and discussed in Section 4.12.2. A schematic describing the wooden spacer blocks that are set on 24 in. centers to secure the batteries and individual sections prior to the concrete placement are presented in Figure 5.2.



a. Wooden Spacer Block

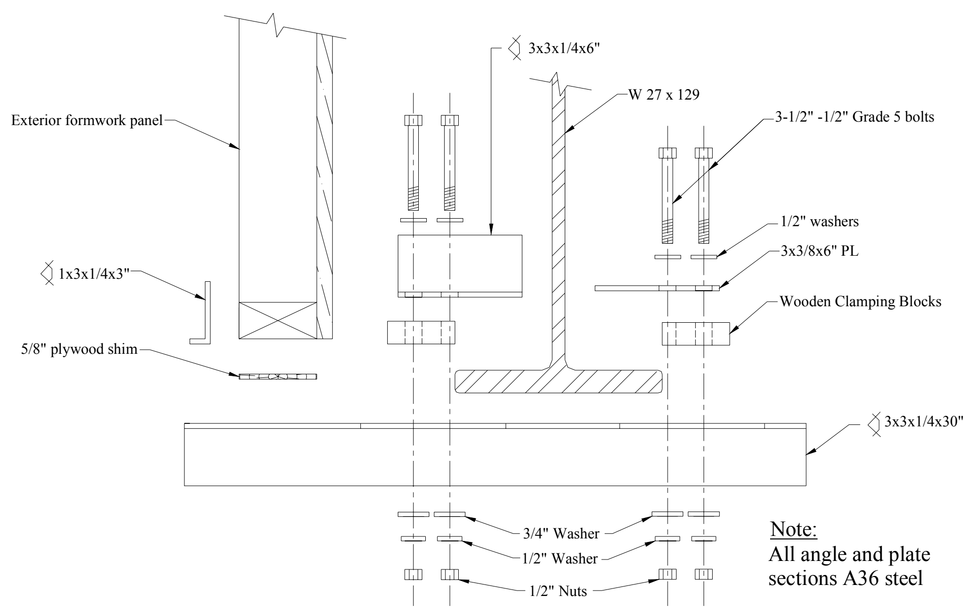
b. Installed wooden spacer block on a typical exterior girder

Figure 5.2. Wooden spacer block for securing the custom rolled arched formwork.

5.5.2 Exterior Formwork System

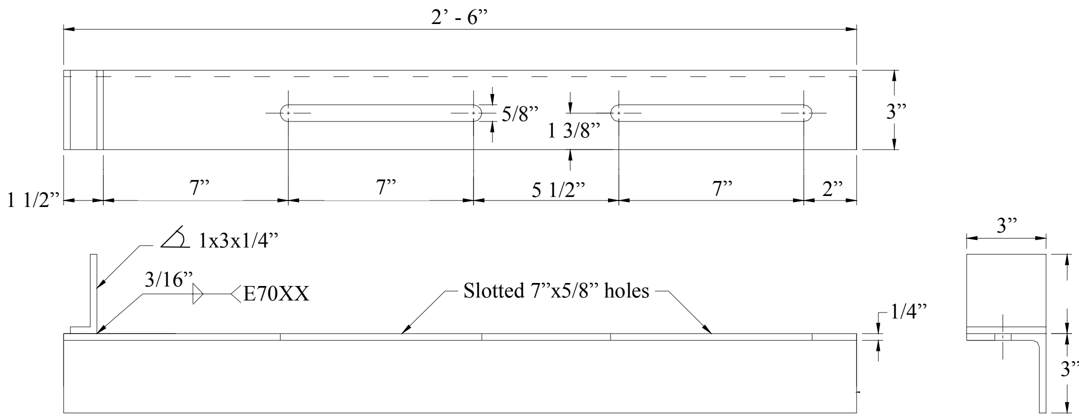
The developed design criteria relies on composite action for all longitudinal girders, therefore, the exterior girders must have the completed ASC requiring full development of the transverse structural reinforcement. A 12 in. x 12 in. deck overhang cast on the exterior side of each exterior girder provides for the necessary concrete development. The exterior overhang for MBISB 2 was constructed using the system described in the following paragraphs.

The main components of the exterior formwork supports are constructed from 3x3x1/4 in. A36 angles which are clamped to the bottom flange of the exterior girders on 3 ft centers. An “exploded” view of the exterior support and the individual components is presented in Figure 5.3. The exterior formwork supports are adjustable for use with other sized girders thus allowing for multiple applications.

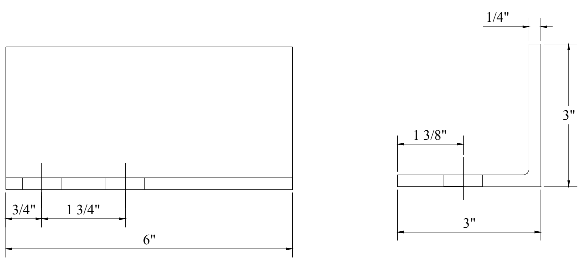


a. Exploded view of the exterior formwork support

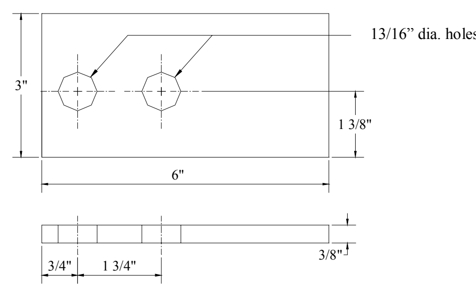
Figure 5.3. Exterior formwork support and its corresponding components.



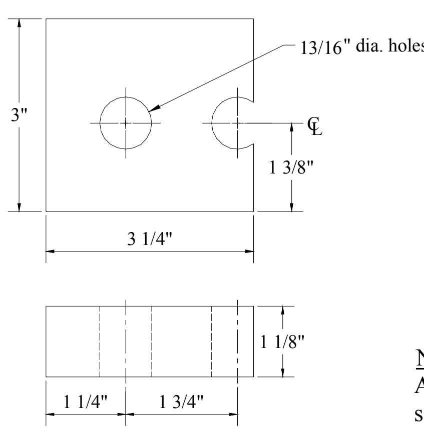
b. Angle section for the exterior formwork support



c. Angle clamp component



d. Plate clamp component



Note:
All angle and plate sections A36 steel
dia. = diameter

e. Wooden clamping block

Figure 5.3. Continued.

The exterior formwork panels were constructed from 2x4 studs and 3/4 in. plywood. Since the exterior formwork also sets the final elevation of the deck, extensions and shims were used to adjust the formwork. A typical completed exterior formwork panel is shown in Figure 5.4, and an installed exterior formwork panel is shown in Figure 5.5.

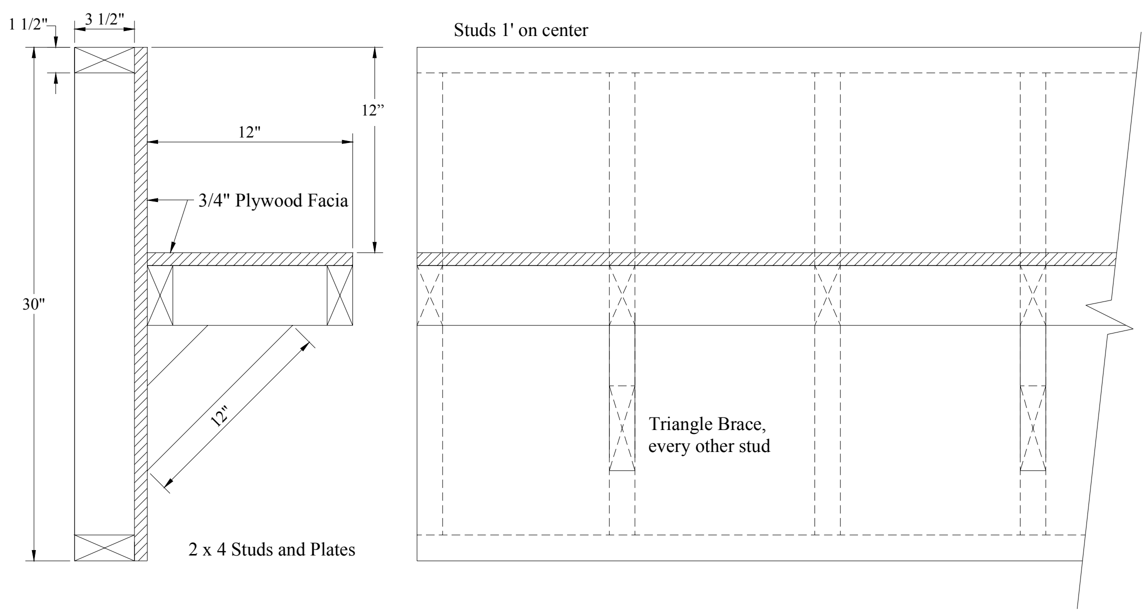


Figure 5.4. Typical exterior formwork panel used for MBISB 2.

5.6 Reinforcement

The design output does not list the reinforcement required for a selected bridge due to the large number of design combinations. Given a selected design, the amount of reinforcement and temperature and shrinkage (T & S) reinforcement can be determined by applying the following criteria.

5.6.1 Transverse Reinforcement

Due to the arched deck system employed, the only reinforcement required within the deck of a MBISB is the transverse ASC reinforcement. The MBISB design methodology

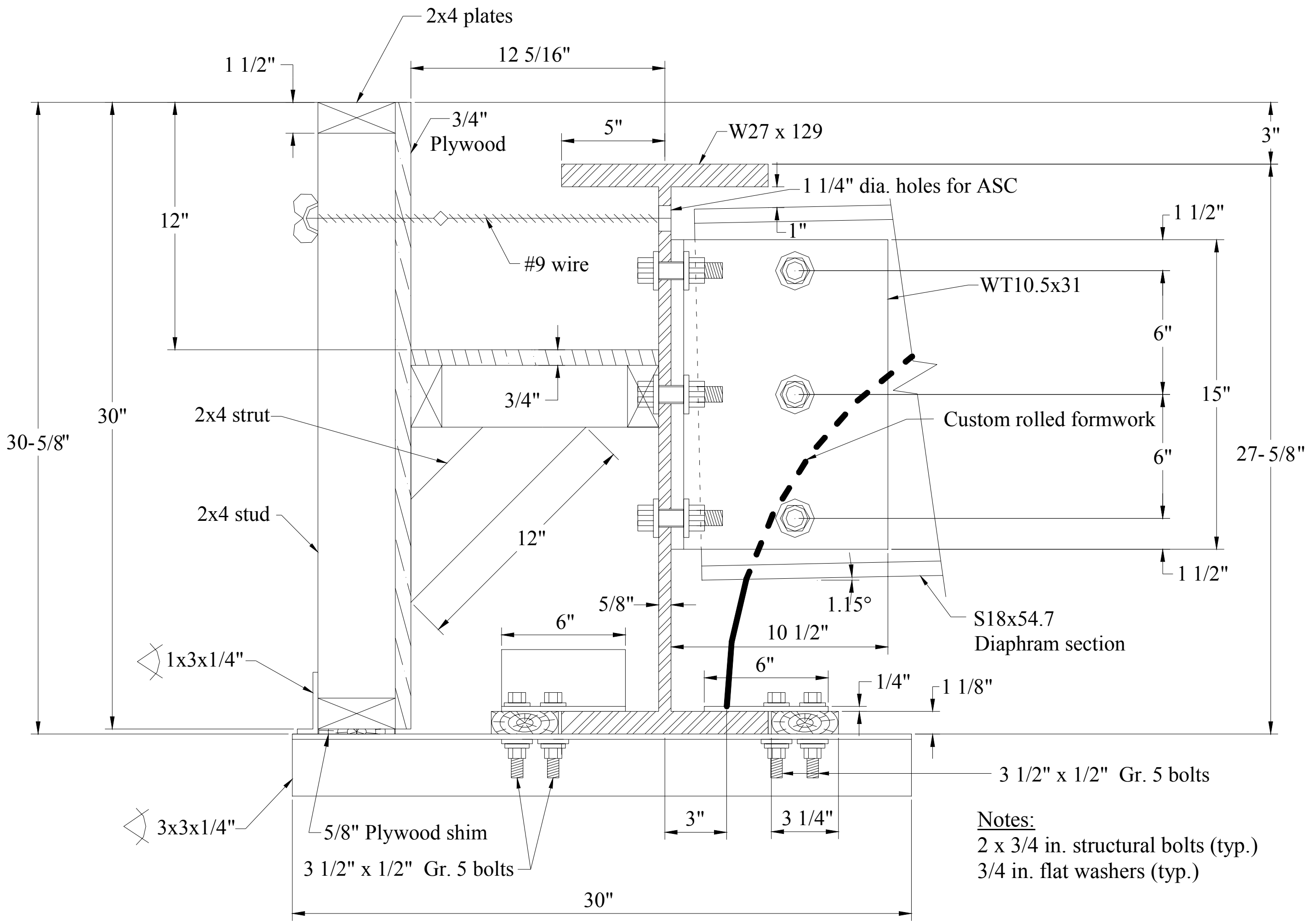


Figure 5.5. Installed exterior formwork.

requires all reinforcement to be #5 Grade 60 deformed reinforcing bar; the geometric configuration of the transverse reinforcement is described in greater detail in the following sections.

5.6.1.1 Backwall reinforcement

The amount of reinforcement needed (located in Table 5.2) in the backwall is a function of the depth of the girder selected for a particular bridge. The reinforcement passes through the holes in Figure 4.6. Closed loop stirrups consisting of #3 Grade 60 reinforcing bars spaced on 12 in. centers to provide confinement and shear reinforcement for the backwall. Four additional stirrups are provided in the portions of the backwall outside of the exterior girders. The number of stirrups per backwall (a function of the bridge width and girder spacing) is listed in Table 5.3.

5.6.1.2 End stiffening reinforcement

As previously discussed in Section 4.14, the ends of the bridge deck must be stiffened in the plane of the deck; this is accomplished by placing 5 lines of reinforcement inside each

Table 5.2. Number of lines #5 Grade 60 backwall reinforcement (per backwall).

Selected Girder	W14	W18	W21	W24	W27	W30
Lines of Reinforcement	4	4	6	6	6	6

Table 5.3. Number of #3 Grade 60 closed loop stirrups (per backwall).

Girder Spacing	Closed Loop Stirrups	
	32 ft Wide Bridge	26 ft Wide Bridge
6 ft	29	24
5 ft	28	-
4 ft – 9 in.	-	24
4 ft – 3 in.	25	-
4 ft	-	22
3 ft – 9 in.	28	-
3 ft	28	20

backwall. The five lines of reinforcement are passed through the first five ASC holes inside of the backwall, similar to the previously described backwall reinforcement. When determining the total number of transverse lines of reinforcement, the fifth line of end stiffening reinforcement is considered as the first line of the regularly spaced (15 in. centers) deck reinforcement. Therefore, four lines of reinforcement are required to stiffen each end of the bridge.

As previously noted, the remaining length of the deck requires the reinforcement necessary to complete the ASC. Therefore, the #5 Grade 60 reinforcement is placed through every fifth ASC hole (15 in. spacing) as indicated in Figure 4.12.

5.6.1.3 Total backwall/ASC reinforcement

The total amount of reinforcement required in a particular bridge is determined by applying Equation 4. Note: the bridge length in the following equation is taken as the out to out length, not the centerline to centerline of support length used in the previous design calculations.

$$\mathrm{TR} = ((\text{Lines of backwall reinforcement}) \text{ x } 2) + \mathrm{ES} + (\text{Length} - (3\ 1/2)) \text{ x } \mathrm{BS} \qquad \text{(Eqn 4)}$$

Where:

TR = Total number of lines of transverse reinforcement

ES = Lines of End Stiffening reinforcement = 8

Length = Out to out bridge length, ft

BS = Bar spacing coefficient = 12"/15" = 0.8

The layout and dimensions of the reinforcement are presented in Figure 5.6 and the lengths of the reinforcement prior to the bending of a standard 180 degree hook are listed in Table 5.4.

5.6.2 Longitudinal Temperature and Shrinkage Reinforcement

As previously discussed, longitudinal reinforcement is needed to prevent cracking due to T & S effects. The lines of #4 Grade 60 reinforcing bars specified for this purpose are a function of the number of bays and the bridge width which is either 26 ft or 32 ft. The reinforcement in a

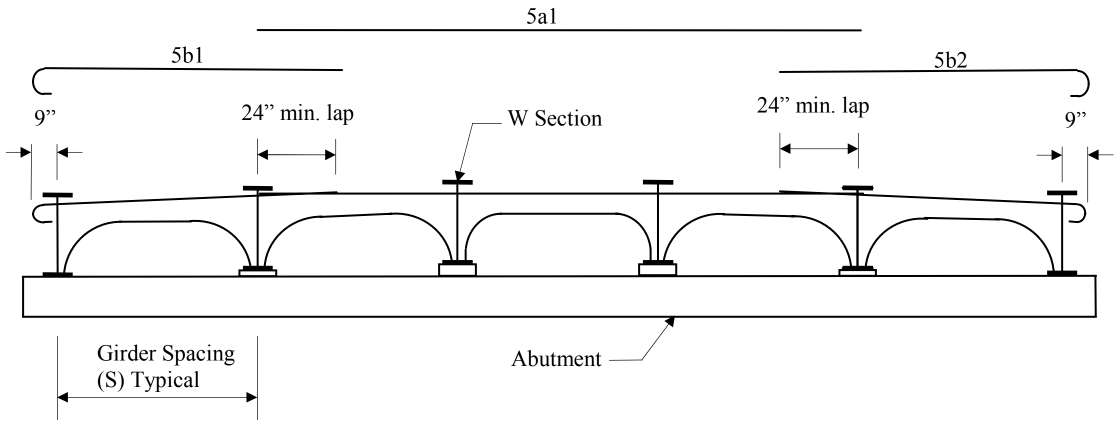


Figure 5.6. Transverse reinforcement for the MBISB design.

Table 5.4. Length of transverse reinforcement.

	Bar/Length (in.)		
Width of Bridge (ft)	5a1	5b1	5b2
26	-	120	240
32	240	86	154

given bay is distributed in the following manner: A line of T & S reinforcement is placed 18 in. from each girder web. The distance between these reinforcing bars is divided equally for the remaining T & S reinforcement; the maximum spacing between the T & S longitudinal reinforcement is 12 in.

Three additional lines of T & S reinforcement are needed in each overhang. A typical layout of the longitudinal T & S reinforcement for MBISB 2 (girder spacing = 6 ft) is shown in Figure 5.7. The number of lines of longitudinal T & S reinforcement required (based on the bridge width and girder spacing, including the overhang reinforcement) is listed in Table 5.5. The longitudinal layout and lengths of the T & S reinforcement for a given bridge is presented in Figure 5.8 and Table 5.6 respectively.

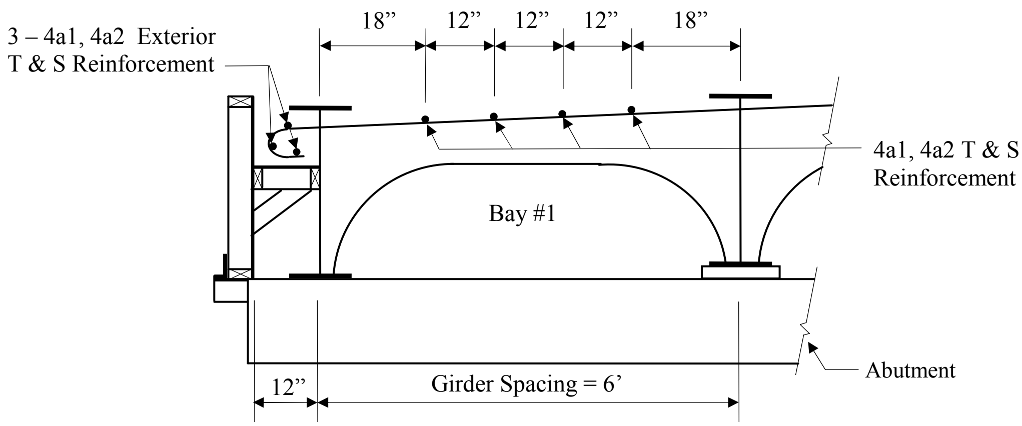


Figure 5.7. Typical layout of the longitudinal T & S reinforcement.

Table 5.5. Number of lines of #4 Grade 60 longitudinal T & S reinforcement.

Girder Spacing	Lines of Reinforcement	
	32 ft Wide Bridge	26 ft Wide Bridge
6 ft	26	22
5 ft	24	-
4 ft – 9 in.	-	21
4 ft – 3 in.	27	-
4 ft	-	18
3 ft – 9 in.	22	-
3 ft	16	14

5.6.3 Transverse Temperature and Shrinkage Reinforcement

For bridges that have the 3 in. of concrete cover over the longitudinal girders, transverse T & S reinforcement (#3 Grade 60 deformed reinforcing bar) is required over the girders. The transverse T & S reinforcement rests on 3/4 in. chairs placed on the longitudinal girders and

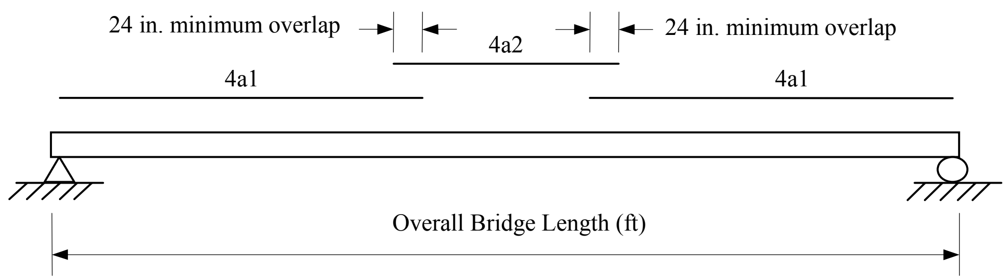


Figure 5.8. Typical longitudinal layout of T & S reinforcement.

Table 5.6. Number and length of the #4 Grade 60 T & S reinforcing bars per line.

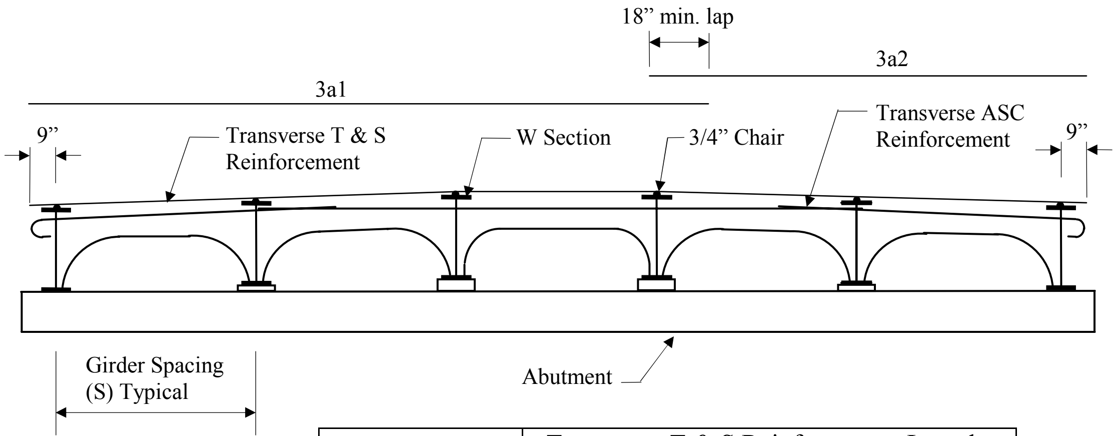
Bridge Length (ft)	40	45	50	55	60	65	70	75	80
	Number of and length of T & S reinforcement (Assuming 20 ft bars)								
4a1	2-20	2-20	2-20	2-20	3-20	3-20	3-20	4-20	4-20
4a2	1-4	1-9	1-14	1-19	1-6	1-11	1-16	1-3	1-8

Note: X-Y, X = number of bars per line Y = bar length in ft

is spaced on 15 in. centers, approximately half way between the lower ASC transverse reinforcement. The layout of the reinforcement for a 26 ft and 32 ft wide bridge is shown in Figure 5.9. The number of lines of transverse temperature and shrinkage steel based on the overall bridge length is listed in Table 5.7.

5.7. Tension Rods and Clips

During the deck concrete placement, significant lateral forces are applied to the girders due to the plastic concrete. Threaded tension rods are installed to restrain the bottom girder flange thus maintaining their spacing. Clips that 'cup' the bottom flange are used to attach the tension rods to avoid welding on a fraction critical section. Details of the tension clip used for the MBISB 2 are presented in Figure 3.5a and an installed clip and tension rod is shown in Figure 3.5b. As an added benefit, the threaded tension rods can be used to remove sweep (if present) in the girders.



Bar Mark	Transverse T & S Reinforcement Lengths (in.)	
	Bridge Width	
	26 ft	32 ft
3a1	240	240
3a2	84	156

Figure 5.9. Layout of transverse T & S reinforcement.

Table 5.7. Number of #3 Grade 60 transverse T & S reinforcing bars required.

Bridge Length (ft)								
40	45	50	55	60	65	70	75	80
Lines of #3 T & S reinforcement								
32	36	40	44	48	52	56	60	64

For MBISB designs with no diaphragms or one line of diaphragms, two evenly spaced lines of tension rods are required. When a greater number of diaphragm lines (two or three) are required, a line of tension rods is required midway between each longitudinal segment defined by the diaphragms and the abutments. The length of an individual tension rod is a function of the girder spacing and the flange width; a schematic of an installed tension rod is shown in Figure 5.10.

The tension rods were left in place on MBISB 2 to determine their effect on the performance of the structure. Based on the low strains measured during the field testing, the tension rods may be removed and reused in another structure after the deck concrete has adequately cured.

5.8. Conclusion of Design

Using the results of the experimental testing and the AASHTO LRFD Bridge Specification, a design methodology for the MBISB system has been developed (*2*). A design example which presents more details on determining the quantities required for a particular MBISB are presented in the Design Manual (Volume 2) (*10*). In addition, Volume 2 includes a PowerPoint slide show which describes the construction sequence and procedures used in the construction of the second demonstration bridge, MBISB 2.

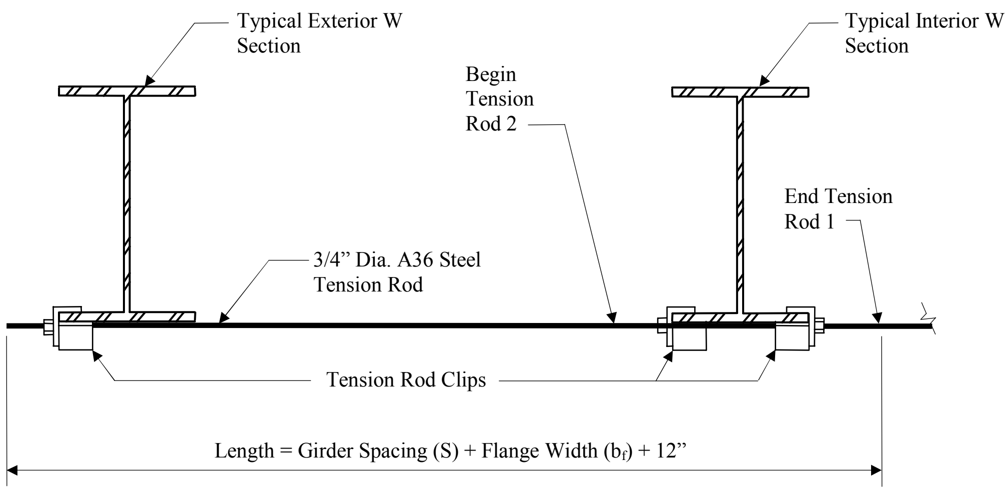


Figure 5.10. Typical layout of the tension rods and clips.

6. SUMMARY AND CONCLUSIONS

The purpose of this Design Guide, (Volume 3) is to provide the engineer with a basic background of the testing and analysis that was completed for the development of the MBISB system. Pertinent design criteria are presented along with the developed design methodology. The results of the testing portion of the project are discussed in greater detail in Volume 1 (*11*). Volume 2, the Design Manual, provides the designer with member sizes, spacing, etc., as well as step by step instructions and examples on how to complete a particular design (*10*). In addition, Volume 2 includes a PowerPoint presentation that describes the construction of MBISB 2.

6.1 Summary

The summary for the information discussed in this volume will be presented in three sections: laboratory and field testing, design criteria and developed design methodology.

6.1.1 Laboratory and Field Testing

Prior research at Iowa State University has provided information on the development of the ASC, the BISB system and the desire of Iowa's county engineers to have alternative bridge replacement designs (*1, 4, 5*). The MBISB system, which had less self weight and composite action, was proposed so that the original BISB system could be used on longer spans. The combination of the ASC which developed the desired composite action and the transverse arch formwork system which reduced the self weight of the MBISB system was initially investigated in the ISU Structures Laboratory.

6.1.1.1 Laboratory Evaluation

Two single bay specimens were constructed and tested to investigate the failure mode of the transverse arched deck. The mode of failure was influenced by the size of the transverse straps that were welded to the bottom flanges of the girders, however, both specimens failed at magnitudes significantly larger than a 45 kip factored wheel load: Specimen 1 = 155 kips and Specimen 2 = 260 kips.

The single bay specimens provided information about the mode of deck failure; however the behavior of the modifications in a bridge system remained unknown. Thus, a full scale, three bay model bridge (L = 31 ft, W = 20 ft) was constructed to determine the structural performance of the MBISB system. Deck reinforcement was limited to that necessary to complete the ASC and the transverse arch was formed using a custom rolled formwork system.

The model bridge was subjected to a series of 45 kip point loads to determine the lateral load distribution and service level stresses in the system. The measured stresses due to the live load were significantly smaller than the yield stress of the girders (11 ksi vs. 36 ksi) and the lateral load distribution values compared with the AASHTO design values. When 302 kips of live load were applied to the model bridge, it failed in flexure with a midspan deflection of approximately 4 in. Additional punching tests were then completed on the distressed deck to obtain data on the behavior of the transverse arched deck.

Based on the measured strain profiles and deflections, the ASC provided composite action between the concrete deck and the steel girders for all the specimens which verified its effectiveness. When the ASC was combined with the transverse arch, which significantly reduced the self weight of the structure, arching action became the controlling mode of structural resistance, allowing for a significant reduction in the reinforcement required. Additional data and information on the MBISB was obtained from the construction and field testing of the two demonstration bridges.

6.1.1.2 Field Evaluation

The design and construction of MBISB 1 (L = 50 ft, W = 31 ft) was similar to the original BISB system except for the addition of the two modifications. Test vehicles were applied in 5 different lanes to quantify the service level stresses, deflections and lateral load distribution of the structure. The maximum measured service level stress and deflection were 4.47 ksi and 0.73 in. respectively which were less than established limits. The lateral load distribution factors (determined from the field test data) for design were determined to be 12% for both the interior and exterior girders. Due to the geometric configuration of MBISB 1, the lateral load distribution factors could not be directly compared to any AASHTO design values.

MBISB 2 (L = 70 ft, W = 32 ft) was constructed to demonstrate the applicability of the MBISB design for spans in excess of 50 ft. The completed structure was also subjected to a series of test vehicles applied in 5 different lanes to quantify the same parameters measured on MBISB 1. The maximum measured service level stress and deflection were 5.54 ksi and 0.50 in., respectively; this is less than established limits. The experimentally determined lateral load distribution factors for single and two lane loadings were calculated and compared to the AASHTO design values for a slab/girder bridge. All four experimental values (one lane case: 32% interior, 36% exterior, two lane case: 43% interior, 41% exterior) were smaller than the AASHTO design values; therefore, the AASHTO design values were selected for use in future MBISB designs.

For both demonstration bridges, the ASC provided composite action and the transverse arch decreased the self weight of the structure. The cost of construction for MBISB 1 and 2 was determined to be $50 psf and $52 psf, respectively, which makes the system competitive with conventional bridge designs. Analytical analyses were performed to validate the contribution of the guardrail to both bridges, since the guardrail provided additional flexural resistance to the system.

6.1.2 Design Criteria

The design criteria developed considered both the experimental results from the field and laboratory tests as well as the applicable design requirements in the AASHTO LRFD Bridge Specification (*2*). The design criteria are based on an AASHTO HS-20 truck and lane load with appropriate strength and serviceability factors applied. The lateral load distribution factors are calculated using the AASHTO LRFD values for a slab/girder bridge which through the laboratory and field testing have been shown to be applicable. The section properties for a specific MBISB design are based on the assumption that the custom rolled arched formwork will be used to form the transverse arch. The plastic moment of a specific design is compared to the factored and distributed moment resulting from the AASHTO Strength I condition to ensure a structure with adequate capacity.

Serviceability and constructability requirements have also been addressed in the design criteria developed. Providing adequate compression flange bracing during construction is extremely important to prevent lateral torsional buckling failures. Therefore, diaphragms are installed to reduce the unbraced length of the longitudinal girders and thus eliminate such failures. A live load deflection is calculated and compared to the recommended AASHTO L/800 deflection limit; service level stresses are also calculated to determine if the permanent deflection criteria are satisfy.

Dead load cambers are specified as well as quantities of structural steel and concrete required for a particular MBISB. Due to the combination of the ASC and the transverse arch, the resulting deck arching action reduces the reinforcement required; the only reinforcement required is that in the ASC and a minimal amount of temperature and shrinkage steel. The guardrail system (assumed to weigh less than 100 plf) is left to the designer.

6.1.3 Design Methodology

A design program was developed which evaluated the numerous design criteria and specified the components necessary to construct a MBISB for a desired span length and bridge width. The following design variables are generated and presented in a tabular format:

- Bridge length
- Bridge width
- Steel yield strength
- Concrete compressive strength
- Number of girders required
- Girder spacing
- Selected W section
- Radius of formwork
- Volume of concrete
- Weight of structural steel

- Interior girder camber
- Exterior girder camber
- Number of diaphragms
- Diaphragm spacing
- Service level deflection
- Optional deflection control
- Water sliding force

Using the information listed in the design tables, an engineer can complete the specifications for the desired MBISB including number, size and spacing of the girders, diaphragm size, required formwork system, the reinforcement required, etc. A detailed example of the design methodology is presented in the Design Manual, (Volume 2); also presented in Volume 2 is a PowerPoint presentation describing the construction for MBISB 2 (*10*).

6.2 Conclusions

Based on the laboratory investigation and the field demonstration bridges, the MBISB system, which incorporated ASC and the transverse arch, is a viable alternative design for spans between 40 ft and 80 ft on LVRs. The design meets the project goals of being lower in cost than conventional systems and being constructible by in-house forces. By applying the developed design methodology, an engineer can specify the necessary components to complete a MBISB providing the end user with a low cost structure that satisfies strength and serviceability criteria.

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APPENDIX A

The following manufactures supplied the custom rolled arched sections for the laboratory specimen (Contech) and MBISB 2 (Midwest Culvert, LTD.)

Contech Construction Products Inc.
5335 Merle Hay Road, Suite 4
Johnston, Iowa 50131
Phone: (515) – 331 – 2517
Fax: (515) – 331 – 2518

Midwestern Culvert, LTD.
1114 S.E. Lorenz Drive
Ankeny, Iowa 50021
Phone: (515) – 964 – 0497
Fax: (515) – 964 – 0545