Design Methodology for the Modified Beam-in-Slab Bridge System

Travis F. Konda
Iowa State University
494 Town Engineering Building
Ames, IA 50011
tkonda@iastate.edu

F. Wayne Klaiber
Bridge Engineering Center
418 Town Engineering Building.
Iowa State University
Ames, IA 50011
klaiber@iastate.edu

Terry J. Wipf
Bridge Engineering Center
Iowa State University
2901 South Loop Drive, Suite 3100
Ames, IA 50010
tjwipf@iastate.edu

ABSTRACT

Researchers at the Iowa State University (ISU) Bridge Engineering Center have developed the modified beam-in-slab bridge (MBISB) system as an alternative replacement for use on low-volume roads. The system consists of longitudinal steel girders with a concrete arched deck cast between the girders. Composite action between the concrete and steel is obtained by using an alternative shear connector, also developed at ISU. Other than the nominal transverse reinforcement required for the ASC, the MBISB requires only minimal additional reinforcement.

After an extensive laboratory testing phase, two demonstration bridges were constructed and field tested to determine the properties of the design. The demonstration bridges, MBISB 1 (L=50 ft, W=31 ft) and MBISB 2 (L=70 ft, W=32 ft), were constructed by in-house forces using standard construction equipment. The resulting structures saved the bridge owner slightly more than 20% over the costs of conventional designs. Test results indicated that the MBISB design exceeded strength and serviceability requirements, and the experimental lateral load distribution factors were comparable to AASHTO LRFD values.

The experimental data were corroborated through analytical modeling, leading to the development of a design methodology for MBISBs ranging in length from 40 ft to 80 ft. An HS-20 truck was used as the design vehicle and all applicable AASHTO LRFD design criteria for steel girder/concrete deck bridges were satisfied. Design tables based on the desired bridge length, width, material strengths, and deck thickness were developed, along with design aids that include a sample design and standard plan sheets. The design tables are included in Volume 2 of the final report for Iowa DOT Project TR-467.

Key words: beam-in-slab bridge— composite action—low-volume road bridge—replacement alternative

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PROBLEM STATEMENT

In Iowa, county governments are charged with maintaining and replacing off-system bridge structures, a majority of which are found on low-volume roads (LVRs). Approximately 30% of the more than 19,700 bridge structures located on Iowa's off-system roads are either structurally deficient or functionally obsolete (FHWA 2004). Due to limited resources and the costs associated with maintaining an aging and deteriorating bridge population, county engineers have expressed an interest in innovative methods to extend available replacement funds.

Many counties in Iowa employ full-time bridge crews to maintain and repair deficient structures. Due to the advanced levels of deterioration and functional obsolescence, replacement of the deficient structure is often the most cost-effective solution. The MBISB system is an alternative replacement design developed by the Iowa State University Bridge Engineering Center (BEC) to address the replacement of LVR bridges with spans of 40 ft to 80 ft.

RESEARCH OBJECTIVE

The objective of the MBISB research was to develop an alternative, low-cost design for use on LVRs when replacing deficient bridges with spans in excess of 40 ft. The resulting MBISB design is constructible by in-house forces, requires no specialized equipment, and reduces costs by approximately 20% when compared with conventional systems.

DEVELOPMENT OF THE MBISB SYSTEM

The development of the MBISB system grew from the beam-in-slab bridge (BISB) system, a successful alternative design limited to spans of approximately 50 ft due to prohibitive stresses and deflections resulting from the bridge's own weight. The original BISB design consists of simply supported W12x79 girders spaced 24 in. on center. Steel confining straps are welded to the bottom flanges at the longitudinal quarter points to provide confinement during the placement of the concrete. A stay-in-place plywood formwork floor rests on the top of the bottom flanges, leaving space between the web and the formwork to allow for the concrete to be in contact with the bottom flange. The void between the girders is filled with unreinforced concrete, which is struck off even with the top flange; a typical cross-section of a BISB is presented in Figure 1.

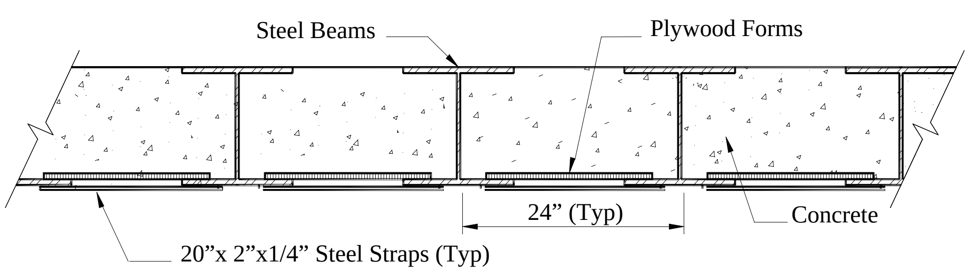

Figure 1. Typical cross-section of a BISB

To span longer distances, the self weight of the original BISB needed to be reduced and the structural efficiency of the system needed improvement. Two modifications were incorporated into the MBISB to achieve the desired effects: an alternative shear connector (ASC) to create composite action between the concrete and steel and a transverse arch to remove ineffective concrete from the cross-section.

Alternative Shear Connector

The ASC, a mechanism for developing composite action, consists of 1 1/4-in.–diameter holes that are either torched or cored in the web of the longitudinal W sections on 3-in. centers, 1 1/2 diameters below the bottom surface of the top flange for the length of the member. Reinforcement (#4 or #5, f_y = 60 ksi) is installed transversely through every fifth hole to provide confinement of the concrete shear dowels, which are formed in the holes when the deck concrete is placed. A series of static and cyclic push-out and flexural beam tests were performed in the laboratory to validate the ASC design; results indicated strength and fatigue properties that meet or exceed the performance of traditional shear studs (Klaiber et al. 1997; 2000).

Transverse Arch

As noted, the self weight of the cross section needed to be reduced so longer spans could be obtained. Recognizing that the concrete below the neutral axis is structurally ineffective and that its removal has minimal effect on the flexural rigidity of the system, a transverse arch cross-section was developed to remove a majority of the ineffective concrete. Two formwork designs, a removable/reusable system and a stay-in-place system, were developed with the arched formwork resting upon the bottom flanges of the longitudinal girders, similar to the original BISB system.

Due to the transverse arch, the mode of resistance of the deck was expected to change from flexure to punching shear, provided the arch was adequately confined. Previous research has indicated that when a bridge deck is adequately confined, the applied loads are resisted by internal arching action, which significantly reduces the amount of internal reinforcement required (Mufti et al. 1993).

RESEARCH METHODOLOGY

Laboratory Phase

Single Bay Specimens

A series of laboratory tests were performed to evaluate the applicability of the two modifications in a full-scale bridge. Continuing the preliminary research performed by Klaiber et al. (2000), two 14.5-ft–long single bay specimens (Specimens 1 and 2) were constructed; a typical cross-section is shown in Figure 2. The purpose of the single bay specimens was to determine the capacity and failure mode of the transverse arch. Each specimen was subjected to a single simulated wheel load, resulting in splitting/punching shear failures of the arched deck at loads 3.5 and 5.8 times greater than a factored 45-kip HS-20 wheel load (Konda 2004). A photograph of the typical testing set up as well as the geometric configuration of the single bay specimens is shown in Figure 3.

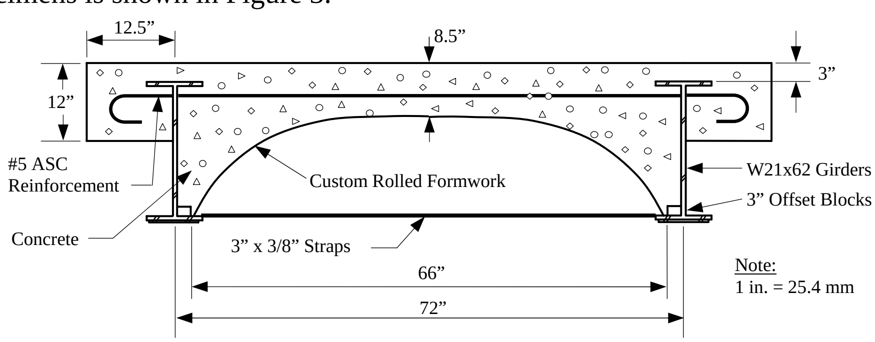

Figure 2. Typical cross-section of a single bay arched specimen

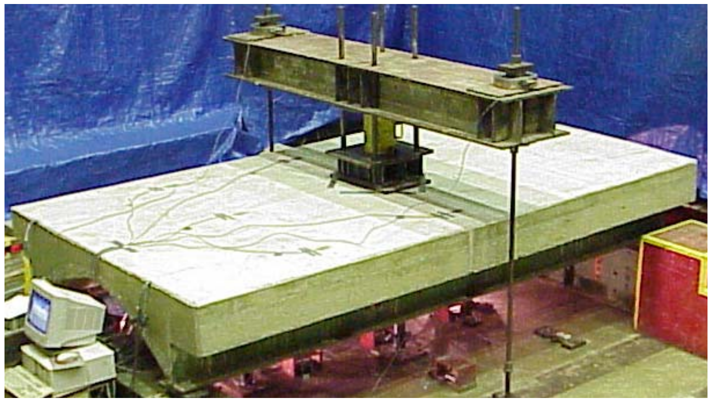

Figure 3. Geometric and loading configuration of Specimen 1

Model Bridge

The single bay specimens provided information on the ultimate capacity of the transverse arched deck, but did not describe the structural behavior of the modified system when integrated into a full-scale bridge. A three-bay model bridge was constructed to determine the lateral load distribution characteristics, flexural capacity, and the deck punching shear capacity. The 30.5-ft–long by 20-ft–wide model bridge was constructed using four W21x62 girders spaced on 72in. centers; the resulting cross-section is shown in Figure 4.

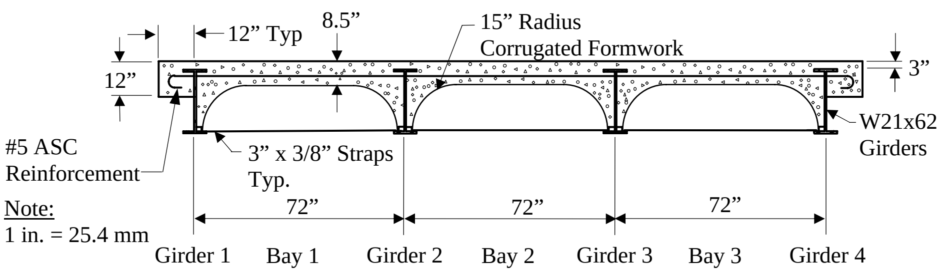

Figure 4. Cross-section of the model bridge

Due to the arching action developed within the deck, the reinforcement was reduced to that required to complete the ASC and stiffen the free ends of the bridge. Crack control reinforcement (a transverse layer of #3, f_y = 60 ksi reinforcement on 15-in. centers) was placed in a portion of the deck to determine whether it was necessary to prevent spalling or excessive cracking. A custom-rolled formwork system, which removed approximately 52% of the concrete from the cross-section, was used to form the transverse arched deck. After adequate curing of the concrete, the formwork was removed and cleaned for future use. The formwork and reinforcement used in the model bridge are shown in Figure 5.

Figure 5. Installed modifications prior to the deck placement

After adequate curing, the model bridge was subjected to both service and then ultimate loadings. The load response was quantified by strain and deflection instrumentation installed at the quarter, middle, and three-quarter span locations on the girders and the concrete deck. Due to equipment constraints, all loads were applied through 12-in. x 12-in. load pads, simulating wheel loads. A total of fifteen 45-kip service-level loads were applied at the quarter, middle, and three-quarter span locations to determine the lateral load distribution characteristics of the bridge. After completion of the service-level tests, an ultimate flexural test was completed, which was followed by five deck punching shear tests.

Lateral load distribution factors were determined by calculating the percentage of the total applied service-level moment that was carried by each effective section; the experimental distribution factors were determined based on the recorded girder strains at the midspan of the model. Since the cross-section of the model was similar to a slab/girder bridge, AASHTO LRFD bridge specification lateral load distribution design values for a single-lane loading were compared to the experimental results (AASHTO 1994). For the interior girders, the maximum experimental value of 45% matched the AASHTO design value. The experimental value for the exterior girder was slightly larger (56% vs. 54%). Due to the severity of the single-point load placed directly over the exterior girder, a conservative experimental value was not expected. Based on the comparative results, it was decided that AASHTO LRFD lateral load distribution values for a slab/girder bridge are applicable for the design of similar MBISBs.

The ultimate flexural loading mentioned earlier was applied about the midspan of the model via a four-point loading configuration. Three of the four girders yielded under a total loading of 302 kips, causing a maximum midspan deflection of 4.01 in. The eccentric positioning of the load pads caused a torsional effect, resulting in cracking of the deck, backwall, and both overhangs. In spite of the cracking and large displacements, the deck gave no indication of an impending catastrophic failure; thus, internal arching action was maintained within the deck slab while the longitudinal members experienced a ductile failure.

Five punching shear tests were completed on the distressed deck, three in the interior of the specimen and two near the free end, to determine the localized capacity and failure mode of the distressed deck. Load was applied through 12-in. x 12-in. load pads for all the tests, with punching shear failure occurring in all cases. The failure loads ranged from 117 kips to 158 kips, which are at least 2.6 times greater than a factored HS-20 wheel load (AASHTO 1994). The mode of failure and magnitude of load necessary to cause failure indicated that there was adequate lateral confinement for internal arching to develop, which resists the applied simulated wheel loading.

Demonstration Bridges

Results from the single bay specimens and the model bridge testing indicate the modifications have more than adequate strength to resist service-level loads. Two demonstration bridges utilizing the previously described modifications were designed, constructed, and field tested to further quantify the structural behavior of the MBISB system and verify the design process. The design and construction of the demonstration bridges is described in the following sections.

Construction of Demonstration Bridges

MBISB 1

The first demonstration bridge, MBISB 1, (L = 50 ft, W = 31 ft) followed closely the original BISB design and consisted of 16 W12x79, f_y = 50 ksi steel sections spaced on 24-in. centers. Holes for the ASC were torched in the webs of the sections, as previously described. Reinforcement (#4, f_y = 60 ksi) was placed through every fifth ASC hole to confine the concrete shear dowels. The transverse arch was formed by a stay-in-place formwork system fabricated from a section of 24-in. diameter, 16-gage corrugated metal pipe (CMP); the transverse arch reduced the self weight of the cross-section by approximately 20%. As in the original BISB system, the concrete deck was placed and struck off evenly with the top flanges.

MBISB 2

The second demonstration bridge, MBISB 2, (L = 70 ft, W = 32 ft), was designed using the AASHTO LRFD bridge specifications and has the outward appearance of a typical slab/girder bridge with six W27x129, f_y = 50 ksi steel girders (AASHTO 1994). However, by including the two modifications, the structure was readily constructed by in-house county forces without the need for specialized equipment. Prior to arriving at the construction site, the girders were cambered and the ASC holes cored in a fabrication shop. After the girders were placed, they were laterally aligned with threaded transverse tension rods, which were attached to the bottom flanges with friction clips. Two lines of diaphragms were installed to provide compression flange bracing, and 12-in.–thick reinforced concrete backwalls were placed at each end of the structure.

The transverse arches between the longitudinal girders were formed using a recoverable/reusable custom-rolled formwork system that was developed during the laboratory phase. The custom-rolled formwork consists of two individual pieces constructed from the same material as CMP, which were bolted together to form a single 24-in.–wide (nominal) section. The individual sections were bolted together into groups of four and five sections off-site and then installed in the bridge as shown in Figure 6.

When compared to traditional Iowa DOT bridge decks, the reinforcement required in MBISB 2 was reduced by approximately 70%. The deck reinforcement was limited to the transverse ASC reinforcement (#5's on 15-in. centers, f_y = 60 ksi) and temperature and shrinkage reinforcement (#4's on 12-in. centers, longitudinal, and #3's on 15-in. centers, transverse). An overhang (12 in. x 12 in.) was necessary on each edge of the bridge to develop the ASC reinforcement; the overhang formwork was also used to set the final deck elevation. Contrary to the BISB design and MBISB 1, the longitudinal girders were embedded with a minimum of 3 in. of concrete placed over the top flanges.

Figure 6. Installed custom-rolled formwork in MBISB 2

Construction Costs

MBISB 1 and MBISB 2 cost approximately \$50/ft^2 and \$52/ft^2 to construct, respectively, including the costs of the sub- and superstructure, labor, and equipment. When compared to conventional designs for these sites, the county realized a cost savings of approximately 20%.

Field Testing of the Demonstration Bridges

Both demonstration bridges were field tested to determine the lateral load distribution, service level stresses, and overall flexural stiffness when subjected to service-level truck loadings. The trucks, weighting between 50 kips and 56 kips, traveled across the bridge at approximately 2 mph (i.e., quasi-static tests) in the five lanes indicated in Figure 7 to create maximum effects in the interior and exterior girders. Prior to the quasi-static tests, static load tests were performed, in which two trucks were placed on the bridge simultaneously.

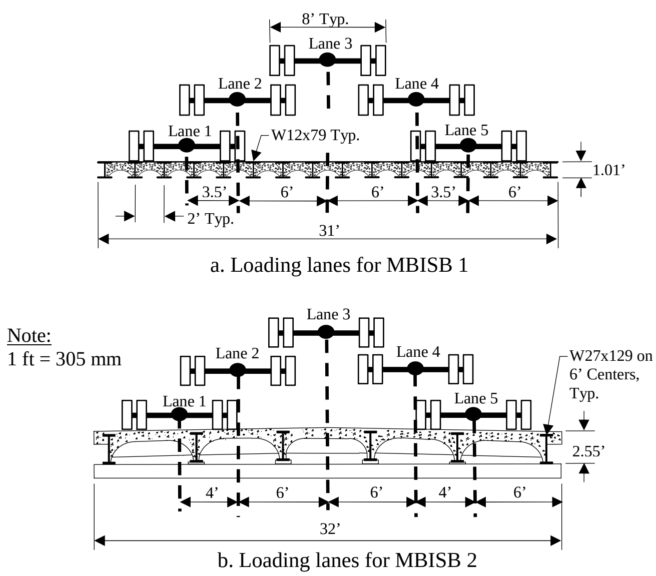

Figure 7. Layout of the loading lanes for field testing

Structural response was measured by strain and deflection instrumentation installed at the quarter and midspan locations to develop response profiles at the two sections. Additional strain instrumentation was installed near the abutments to determine the level of restraint present in the connections between the abutments and the girders.

Field Test Results

MBISB 1

A maximum midspan deflection of 0.73 in. was recorded during the static load test when the two trucks occupied Lanes 2 and 4 (see Figure 7). The maximum recorded deflection was slightly less than the suggested AASHTO service-level live load deflection limit (i.e., L/800) which is 0.74 in. A maximum midspan tensile strain of 154 microstrain (4.5 ksi, assuming Young's Modulus = 29,000 ksi) occurred when two trucks (in Lanes 2 and 4) occupied the bridge. When combining the measured live load and self weight stresses, a maximum tensile stress of 11.5 ksi resulted, a value which is less than 1/4 of the steel yield stress. Based on the strains measured close to the abutments, there was minimal end restraint present (i.e., the bridge is simply supported). Lateral load distribution factors for MBISB 1 were determined by applying the techniques similar to those used in the laboratory model bridge. Based on the experimental results, the lateral load moment distribution design factors for a single lane loading were determined to be 12% for both the interior and exterior girders. This value was not directly compared to the AASHTO LRFD design values because the cross-section of MBISB 1 does not correlate with the presented design information (AASHTO 1994).

MBISB 2

A maximum midspan deflection of 0.5 in. was recorded for both the interior and exterior girders during the static load tests when the trucks first occupied Lanes 2 and 4 and then Lanes 1 and 5, respectively. The maximum deflection measured during the rolling tests was 0.47 in., which occurred in the exterior girder during the Lane 1 loading. All measured deflections were less than 50% of the suggested 1.04-in. (L/800) limit (AASHTO 2004).

Strain profiles were developed at the quarter and midspans and were used to confirm composite action and develop the lateral load distribution factors; a typical midspan strain profile when the truck is in Lane 1 is presented in Figure 8. A maximum midspan tensile stress of 5.54 ksi was determined during the static loading. A total tensile stress of 26.2 ksi results when the effects of the live load and the self weight are combined. Similar to MBISB 1, there was no significant evidence of restraint at the abutments, which indicated a simply supported structure. The resulting experimental values for both single and multiple lane loadings were calculated and compared to AASHTO LRFD design values for steel girder/concrete slab bridges; the results, presented in Table 1, have been adjusted to reflect similar multi-presence and traffic use factors. When compared to the field test values, the AASHTO distribution factors are conservative and thus are considered applicable to the MBISB.

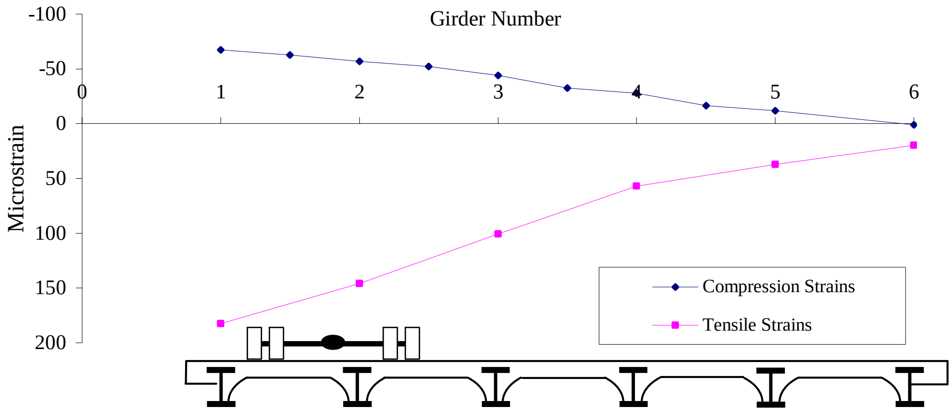

Figure 8. Midspan strain profile for MBISB 2, Lane 1 loading

Table 1. 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

MBISB DESIGN METHODOLOGY

When the structural behavior of the laboratory model bridge and MBISB 2 under service-level loads were compared to AASHTO LRFD design values for a slab/girder bridge, the specification values were conservative, but deemed applicable for future designs. Thus a design methodology, similar to that of a standard composite slab/girder bridge was developed for the MBISB system. A detailed discussion of the addressed limit states and design parameters is provided in Volume 3 of the final report for Iowa DOT project TR-467 (Wipf et al. 2004a; AASHTO 1994).

To facilitate the use of the MBISB system, a series of designs were created using the methodology developed for use by county engineers and consultants in need of a replacement LVR bridge with a span of up to 80 ft. The designs provide the engineer with design information for the MBIBS system based on the following six parameters:

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

Designs were created for nine different lengths, increasing in 5-ft increments from an initial length of 40 ft. Two overall bridge widths, 26 ft and 32 ft, were selected so that two lanes of traffic or large agricultural equipment could be accommodated. Girder spacing is determined by dividing the predetermined bridge width by the number of girders used. For each girder spacing, the five lightest W sections that satisfied the strength limit state for an HS-20 design truck and lane load are included so that recycled girders, if available, can be readily evaluated for a variety of design configurations. A concrete compressive strength of 4 ksi is conservatively specified for the MBISB deck. Yield strengths of the longitudinal girders are specified as either 36 ksi or 50 ksi, which are readily available structural steels. A yield strength of 60 ksi is specified for the deformed reinforcing bars used in the concrete deck.

With the establishment of the previously stated boundary conditions, a series of designs were evaluated and the output arranged in a tabular format. An example of such an output for a 65-ft–long, 32-ft–wide MBISB, with a steel compressive strength of 50 ksi and 3 in. of concrete placed above the top flanges of the longitudinal members, is presented in Table 2. A complete listing of the designs available is presented in Volume 2 of the final report for Iowa DOT project TR-467 (Wipf et al. 2004b). In addition to the design outputs, a detailed example and explanation of the use of the design information is included. With the information provided in the design output for the selected bridge geometry, the user can readily compare gross material quantities for estimation purposes and determine the design option that best address a given site.

Table 2. Design output example for a 65 ft long, 32 ft wide MBISB

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

Material and section properties: f_y = 50 ksi, f'_c = 4 ksi, Cover = 3 in., L/800 = 0.975 in.

CONCLUSION

Results from the laboratory and field evaluations verified that the modifications, the ASC and the transverse arch, can be successfully incorporated into a full--scale bridge, resulting in the MBISB system. Laboratory testing of the MBISB system provided evidence that the deck resisted the simulated wheel loads through arching action and provided a minimum reserve capacity in excess of 70 kips when subjected to a simulated HS-20 wheel loading. Results from the service loadings indicated that the experimental lateral load distribution factors present in the model bridge were similar to the AASHTO

LRFD design values. When subjected to ultimate flexural loads, the girders yielded prior to the occurrence of a localized punching shear failure, resulting in a ductile failure.

The two demonstration bridges provided verification of the applicability of the MBISB design by exceeding performance standards while being constructible by county forces at costs less than conventional designs. The structural behavior of MBISB 2 was determined and compared to the AASHTO LRFD design values for a slab/girder bridge, which were both conservative but applicable for the design of the MBISB system.

A design methodology for future MBISBs was then developed by applying the AASHTO LRFD limit states for a slab/girder bridge while incorporating the modifications for composite action and formwork for the deck. A series of designs were developed and included in a design manual to assist engineers in the design of future MBISBs.

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