

2004

# Investigation of the modified beam-in-slab bridge system

Travis Frank Konda  
*Iowa State University*

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**Investigation of the modified beam-in-slab bridge system**

by

**Travis Frank Konda**

A dissertation submitted to the graduate faculty  
in partial fulfillment of the requirements for the degree of

**DOCTOR OF PHILOSOPHY**

Major: Civil Engineering (Structural Engineering)

Program of Study Committee:  
F. Wayne Klaiber, Co-major Professor  
Terry J. Wipf, Co-major Professor  
Fouad Fanous  
Vernon Schaefer  
Loren Zachary

Iowa State University

Ames, Iowa

2004

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**Co-major Professor**

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**For the Major Program**

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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 Precast 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 spans greater than 50 ft that is lower in cost than conventional systems and relatively easy to construct.

This dissertation presents the development of the MBISB system in the form of three technical papers beginning with a summary of previous research and an overview of the laboratory testing. The results from the laboratory phase, presented in detail in the second technical paper, indicated the modifications were applicable to full scale LVR structures; thus, two demonstration bridges were designed and constructed. The first bridge, MBISB 1 (L = 50 ft, W = 31 ft), consists of 16-W12x79 girders on 2 ft centers; the second bridge, MBISB 2 (L = 70 ft, W = 32 ft), consists of 6-W27x129 girders on 6 ft centers. The resulting structures were constructed by in-house forces and cost approximately 20 percent less than conventional designs.

The demonstration bridges were field tested to determine the structural behavior; instrumentation was installed at critical sections to measure strains and deflections. The resulting data confirmed compliance with strength and serviceability requirements. Based on the field data and subsequent analysis, the demonstration bridges were found to exceed design requirements and possess

considerable reserve capacity. The results of the field testing are presented in the third technical paper.

A MBISB design methodology was then developed based on the test results and applicable American Association of State Highway and Transportation Officials (AASHTO) Load Resistance Factored Design (LRFD) Bridge Design Specification requirements for future use.

## 1. INTRODUCTION AND PROBLEM DESCRIPTION

### THESIS ORGANIZATION

This dissertation documents the investigation and development of the Modified Beam-in-Slab Bridge (MBISB) system (Iowa Department of Transportation (Iowa DOT) Project TR-467). This text deviates from traditional format in that it consists of a background chapter (Chapter 1) that outlines the basic course of the research and three other chapters (Chapters 2-4) which are technical papers that describe the research undertaken and the results from the field testing of the two MBISB demonstration bridges. The general conclusions of the project including an overview of the developed design methodology are presented in Chapter 5 and recommendations for research are proposed in Chapter 6.

The first paper (Chapter 2), "Beam-in-Slab Low Volume Road Bridge System", details the background of the MBISB system and provides a description of the laboratory testing conducted to evaluate the two modifications that were applied to the original BISB design. The development and implementation of the Alternative Shear Connector (ASC) and the transverse arch are discussed in detail. Results from the previous laboratory testing required for the development of the final ASC design are presented. Various methods of constructing the transverse arch were investigated resulting in the custom rolled arched formwork system that is removable and reusable. An overview of results from three single bay specimens which were constructed to investigate the mode of structural resistance are presented along with a description of a three bay laboratory model bridge. This paper was presented at the Eighth International Conference on Low Volume Roads held in Reno, Nevada in June of 2003, sponsored by the Transportation Research Board (TRB).

The second paper (Chapter 3), "Laboratory Investigation of the Modified Beam-in-Slab Bridge System, an Alternative Bridge Replacement for Low Volume Roads", details the results from the laboratory evaluation of the proposed modifications. The strength and failure modes of all the

tested specimens are presented. In addition, the lateral load distribution characteristics for the laboratory model bridge are presented. This paper will be submitted for publication in the Canadian Journal of Civil Engineering.

The design, construction and structural performance of the two demonstration bridges is presented in the final paper (Chapter 4), "In-field Performance of the Modified Beam-in-Slab Bridge, a Low Volume Bridge Replacement Option". This paper is currently under peer review for publication and has been accepted for presentation at the 6<sup>th</sup> International TRB Bridge Engineering Conference (to be held July, 2005 in Boston, Massachusetts). This paper focuses on the construction of and the results from field testing two demonstration bridges; also the results of the field testing are compared to design values and analytical results.

A full description of the development and implementation of the design methodology is not included in this dissertation. This information is included in two additional volumes (Volume 2 and 3) which complete the final report for Iowa DOT project TR-467. These volumes provide an overview of the research conducted and focus more on the design and construction of future MBISBs and are meant to complement each other (1, 2). The purpose of this dissertation is to present the investigation of the proposed MBISB design and report the ensuing results. Thus, the design information has been separated from the dissertation since the design methodology was developed and is supported by the results presented within this document.

## **BACKGROUND**

The State of Iowa ranks 5<sup>th</sup> in the nation for the total number of bridges with approximately 25,000 structures (3). A bridge structure, as defined by the Federal Highway Administration (FHWA), is a permanent traffic carrying structure with a minimum total span of 20 ft (4). Based on published National Bridge Inventory (NBI) data, approximately 78% (19,659) of the bridges in Iowa are owned by the 99 counties and are considered to be on off system roads (5). An off system road is

defined as a road and related structures whose maintenance is the responsibility of local county governments. A majority of the off system roads can be considered low volume roads (LVR) with Average Daily Traffic (ADT) counts less than 400 vehicles. Roads with such traffic counts are defined as very low volume roads by the American Association of State and Highway Transportation Officials (AASHTO) (6); however, in this project, they are referred to as LVRs.

Nationally, the health of bridges is a concern due to age and continued deterioration; approximately 27% of bridge structures are classified as either structurally deficient, functionally obsolete, or both (3). By definition, a structurally deficient bridge is one that has been “restricted to light vehicles only, is closed, or requires immediate rehabilitation to remain open”. A functionally obsolete bridge is one in which the “deck geometry, load carrying capacity (in comparison to the original design to the current State legal loads), clearance, or approach roadway alignment no longer meets the usual criteria for the system of which it is an integral part” (7, 8).

The national trend of deteriorated bridges in need of repair or replacement is reflected in Iowa where 28% of the total bridge population is classified as either structurally deficient (21%), functionally obsolete (7%) or both (4). The percentage increases to 31 for off system structures, which is attributed to an aging bridge population (average Iowa off system structure age = 44.5 years) accompanied by limited funds for maintenance (9).

Maintaining the off system bridge population is a major task for county governments. The challenge of remedying deficient bridge structures is one of scale; there are many more bridges that are structurally deficient or functionally obsolete than funds available to repair or replace them. Due to advanced levels of deterioration and antiquated designs, the replacement of deficient off system bridges is frequently more economical than repairing them. Therefore, when polled in 1994, almost 70% of Iowa County Engineers expressed an interest in implementing alternative designs developed specifically for use on LVR systems, especially if the designs were easy to design and construct (10).

LVR bridges, while normally carrying a significantly smaller number of vehicles than on system structures, must still be designed to carry legal loads plus wide, heavy agricultural and service vehicles. However, some counties often lack sufficient resources to design and construct traditional bridges.

Developing a lower cost structure, when compared to conventional designs, while still meeting legal load requirements is the main reason for researching alternative designs. Many counties have in-house forces for the construction and maintenance of LVR bridges and thus are interested in such alternatives designs. The Iowa State University (ISU) Bridge Engineering Center (BEC), through research sponsored by the Iowa Highway Research Board, has developed, implemented and evaluated various alternative designs for the purpose of replacing some of Iowa's deficient LVR bridges. One of the more successful alternatives for replacing deficient structures was developed in Benton County, Iowa, almost 30 years ago. Remarkable in its simplicity, the system, referred to as the Beam-in-Slab Bridge (BISB), has proven, through both in-service use and laboratory and field testing, to be an effective replacement alternative for spans of up to 50 ft (11).

The original BISB system consists of longitudinal W12 sections spaced on 2 ft centers that serve as the main structural elements. The girders are restrained during the construction phase by steel straps welded to the bottom flanges of the beams. A plywood stay-in-place formwork 'floor' rests on the bottom flanges. A 3 in. gap is left between the plywood and the web to allow for contact of the concrete with the bottom flange. To complete the structure, unreinforced concrete is placed between the steel sections and struck off even with the top flanges. A cross section of the original BISB design is presented in Figure 1.1.

The original BISB system has the advantages of simple design, ease of construction and excellent structural performance, based upon the results from the laboratory and field testing. Two specimens, a two beam and a four beam test specimen, simulating the in-field BISB were constructed

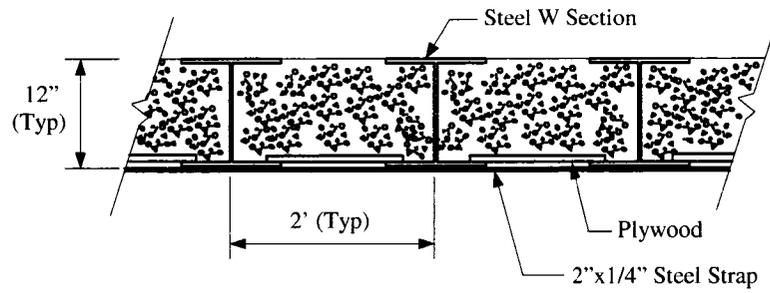


Figure 1.1 Typical Beam-in-Slab Bridge cross section.

in the laboratory and subsequently tested at service and ultimate load levels. A field test was performed on an in-service BISB located in Benton County, Iowa in 1996 to evaluate the structural behavior of the bridge under service loads. A photograph of the load vehicles on the tested BISB is presented in Figure 1.2. Both the laboratory specimens and the in-service bridge exhibited excellent lateral load distribution and significant reserve strength (11).

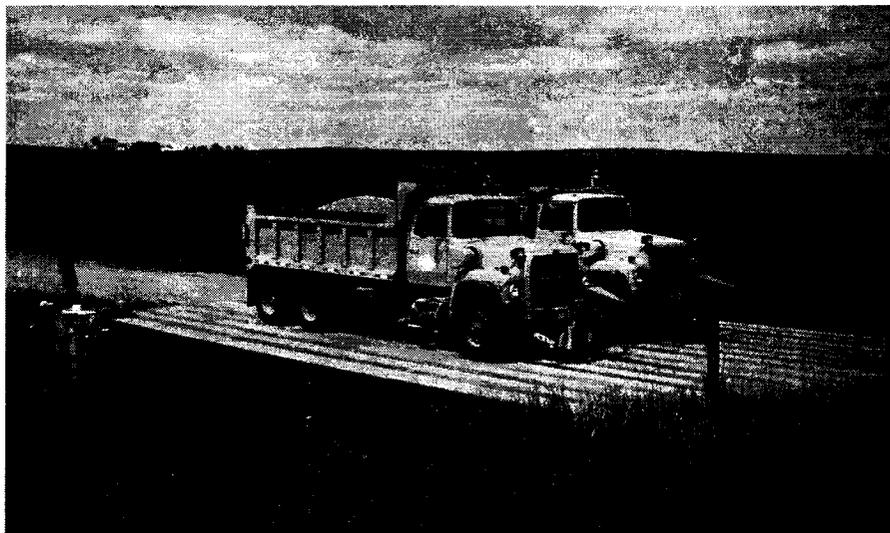


Figure 1.2 Field testing of the original Beam-in-Slab Bridge system.

While the original BISB design is cost competitive (approximately \$50 psf) and readily constructible by county forces, spans are limited to approximately 50 ft due to the large deflections and stresses that result from the self weight of the structure. Since the unreinforced concrete does not develop composite action with the steel girders, it does not contribute to the flexural rigidity of a section. The girder depth and spacing are also limited by the self weight, resulting in relative shallow sections (typically W12's) at small spacings (typically 2 ft). The section size and spacing are generally held constant for various span lengths, placing an upper bound on the applicable length as previously noted while resulting in an over designed structure for shorter spans which further reduces the overall efficiency of the BISB design.

## **OBJECTIVE**

The objective of this project was to increase the applicability of the BISB concept as an alternative bridge replacement on Iowa's LVRs by increasing the structural efficiency of the design while maintaining the ease of construction found in the original system. Two modifications to the original BISB system were proposed, implemented, and then evaluated to improve the structural efficiency.

### **Modification 1: Composite Action**

By developing composite action between the concrete deck and the steel girders, a reduction of 20 to 30 percent in the steel required for the longitudinal flexural members is attainable (12). For traditional steel girder/concrete slab bridge designs, composite action is generally developed through the use of shear studs which require special equipment for installation which most Iowa counties do not have. Therefore, an Alternative Shear Connector (ASC) that can be fabricated by in-house forces was developed at ISU through a two phase laboratory study. The behavior of the ASC is similar to the "Perfobond Rib" developed by German researchers (11, 13).

The final design of the ASC consists of 1 1/4 in. diameter holes that are either torched or cored through the web of the longitudinal girder. The holes are spaced on 3 in. centers along the length of the girder and centered one diameter below the top flange. Composite action is developed when the plastic concrete flows through the holes, which after the concrete cures, becomes a mechanical connection (i.e. shear dowels) between the deck concrete and the steel girders. Reinforcement (#4 or #5 Grade 60) is placed through every fifth hole resulting in transverse reinforcement on 15 in. centers. The purpose of the transverse reinforcement is to provide lateral confinement to the concrete shear dowels (13).

Based on the results of two investigations, the ASC was determined to be an effective method of developing composite action that can be installed by county forces without the use of special equipment. Implementing the ASC requires the concrete of the bridge deck to be below the bottom flange of the longitudinal girder. This requirement is readily met when the ASC is combined with the original BISB concept where the deck concrete encases the webs of the girders.

### **Modification 2: Reducing the Self Weight**

As previously noted, the applicable length of the original BISB design is limited due to the self weight of the system. Since the concrete on the tension side of the neutral axis is structurally ineffective, removing a majority of this concrete reduces the self weight of the system while causing minimal change in the overall behavior. A large portion of the ineffective concrete can be removed by forming an arch transverse to the longitudinal girders. The transverse arch maintains the concept of the original BISB design where the deck concrete encases the webs which in turn readily accommodates the ASCs for developing composite action. Like the plywood 'floor' formwork used in the original BISB design, the transverse arch rests on the top surface of the bottom flanges of the longitudinal girders.

In research conducted at ISU, a specimen that implemented both the ASC and the transverse arch was constructed and evaluated. The results from this preliminary test indicated the combination of the two modifications had potential as an alternative bridge design; however, more experimentation and analyses were needed before the modifications could be implemented in the field (13). Specimens and bridges that incorporate the proposed modifications to the original BISB design are referred to as Modified Beam-in-Slab Bridge (MBISB) designs.

### **Additional Benefits of the Modifications**

#### *Increased Girder Depth and Spacing*

The adaptation of the two modifications to the original BISB design results in a more efficient system that can be used for longer spans. Due to the removal of a majority of the ineffective concrete from the cross section by the transverse arch a reduction in self weight results. Transverse arch formwork also permits larger girder spacings. Although the self weight has been reduced, the fact that there are greater girder spacing and greater span lengths required larger girders to be used. The wider girder spacing reduces the number of girders required and thus the number of ASC holes that must be fabricated. Field construction time is also reduced (which improves the efficiency of the system) since there are fewer girders to place.

#### *Increased Composite Flexural Rigidity*

Since the original BISB design did not have composite action, no benefit was gained by placing concrete above the top flange. Rather, the top flanges of the longitudinal sections were used as guides for striking off the deck concrete, thus simplifying the construction process. With the implementation of the ASC, the flexural rigidity of the system is increased; placement of concrete over the top flanges also increases the flexural rigidity a significant additional amount. However, embedding the top flanges results in more complex construction, requiring the use of a power screed rather than finishing the concrete even with the top flanges as was done in the BISB.

### *Reduced Corrosion/Skidding Potential*

The susceptibility to corrosion is reduced by embedding the top flanges of the girders in the deck. Some original BISB structures with the top flanges of the girders exposed have been in service for almost 30 years with minimal corrosion. Covering the top flanges also reduces skidding on the deck surface since there are no steel surfaces exposed. However, skidding has not been a reported problem in BISB structures. This investigation, thus, considered two possibilities: finishing the deck concrete even with the top flange to maintain the simplicity of construction and encasing the top flanges in concrete (more protection and greater flexural rigidity, but more complex construction).

### *Reduction of Deck Reinforcement*

Implementing the transverse arch between the longitudinal girders not only reduced the self weight of the structure but introduced the possibility of the deck resisting wheel loads through arching action rather than flexure. It was hypothesized that if internal arching action between the longitudinal girders were developed, the reinforcement required for deck could be greatly reduced. The hypothesis is based upon work performed by Canadian researchers who have successfully demonstrated the presence of internal arch action in bridge decks that are adequately confined in both the transverse and longitudinal direction. When such confinement is present, the mode of resistance within the deck is changed from flexure to internal arching (14).

In the Canadian system, the in-plane arching action is developed in the deck by transversely restraining the longitudinal girders with a series of transverse steel straps that are welded to the top flanges of the girders prior to the placement of the concrete deck. Longitudinal restraint is provided by the composite longitudinal girder. Since the resulting arching action resists the wheel loadings, there is no need for traditional reinforcement in the deck slab itself, which greatly reduces the potential for corrosion in the deck. A small amount of fiber reinforcement is added to the deck concrete to control temperature and shrinkage cracking (15). For the first bridge, extensive laboratory

testing was undertaken to provide evidence that the 'steel free' system was applicable; once in place, the bridge was monitored with a variety of instrumentation to validate the design assumptions (16, 17). Several additional bridges have been constructed in Canada where no reinforcing steel is included within the deck slab (18).

The 'steel free' deck system developed by the Canadian researchers is very similar to a traditional steel girder/composite concrete deck design with shear studs to develop composite action; however, there is minimal deck reinforcement (14). The MBISB system is unlike typical steel girder/composite concrete deck designs since there is a transverse arched deck between the girders. Transverse restraint of the longitudinal girders is provided by the ASC reinforcement and the continuity of adjacent arched sections. Longitudinal confinement of the arched deck is provided by the composite girders. To control temperature and shrinkage cracking, a minimal amount of Grade 60 steel reinforcement is used. By reducing the amount of reinforcement required in the deck, the cost of the structure and the construction time are reduced, as well as the potential for deck deterioration resulting from corrosion.

## **RESEARCH TASKS**

A research plan was developed to evaluate the applicability of the two modifications (adding composite action and reducing the self weight) to the original BISB concept. As previously noted, the reason for implementing the modifications is to increase the applicable span length of the original BISB design. The research plan included an initial laboratory investigation phase and a field demonstration phase where two demonstration bridges that included the modifications (i.e. MBISB) were designed, constructed and evaluated.

### **Laboratory Tasks: Single Bay Specimens**

Three single bay laboratory specimens were designed, constructed and evaluated for the

purpose of investigating the integrated behavior of the ASC and the transverse arch. Continuing the research begun by Klaiber et al. (13), two full scale single bay specimens were constructed and tested to investigate the arching action that developed in the deck, which was confined by the transverse ASC reinforcement and the transverse straps which were welded to the bottom flanges of the girders. Both specimens (clear span = 14.5 ft) were constructed with two W21x62 girders spaced 6 ft apart; the girders were embedded in the slab with 3 in. of concrete placed over the top flanges to increase the flexural rigidity of the composite section and reduce the potential for corrosion and skidding. A different formwork system was used on each specimen to evaluate the ease of construction.

A single concentrated static load, simulating the effect of a wheel load, was applied to the center of each specimen. By observing the mode of failure, internal arching was determined to be the main mode of resistance for the deck. The overall applicability of the modified system to an alternative bridge design was also evaluated based upon the ultimate strength of the specimens.

#### **Laboratory Tasks: Three Bay Specimen**

A model bridge (L = 31 ft, W = 20 ft) consisting of 4-W21x62 girders on 6 ft centers, was designed and constructed to investigate load distribution and strength characteristics of a bridge system utilizing both modifications. The design was based upon analytical modeling and the data obtained from the laboratory evaluation of the previous single bay specimens. Concentrated static loads, simulating a service level wheel load, were placed at numerous locations on the structure to quantify lateral load distribution. The specimen was then tested to failure to observe the load redistribution, mode of failure, and the ultimate strength of the specimen.

#### **Field Demonstration Bridges**

Incorporating the information obtained in the laboratory testing, two demonstration bridges were designed, constructed and field tested to evaluate the performance of the modifications. The

performance and durability of the demonstration bridges will continue to be monitored to ensure the resulting MBISB is a low maintenance, long lasting design.

#### *MBISB 1*

The first demonstration bridge (MBISB 1), constructed in Tama County, Iowa by in-house forces incorporated both previously discussed modifications. The purpose of this structure was to demonstrate the ability of the ASC to develop composite action and the transverse arch, which reduced the self-weight of the structure by approximately 20 percent. The transverse arch formwork was constructed from a 24 in. diameter corrugated metal pipe (CMP); formwork in this bridge was left in place. MBISB 1 (L = 50 ft, W = 31 ft) which consists of 16-W12x79 girders on 2 ft centers was field tested and found to have excellent lateral load distribution characteristics in addition to satisfying stress and deflection limitations.

#### *MBISB 2*

The second demonstration bridge (MBISB 2) was also constructed in Tama County, Iowa by in-house forces resulting in a 70 ft long, 32 ft wide bridge structure that is supported by 6-W27x129 girders on 6 ft centers. The purpose of MBISB 2 was to demonstrate that with the implementation of the two modifications, the system can be used in longer spans (i.e. > 50 ft) with an increased efficiency. Composite action was developed through the use of the ASC and the transverse arch was constructed with a reusable custom rolled steel formwork. This bridge was also field tested, and similar to MBISB 1, the structure was found to surpass limiting performance values.

Grillage models were developed for both demonstration bridges to validate the behavior and determine the contribution of the guardrail to the total flexural resistance of the system. Through the use of the grillage models the contribution of the guardrails to the total flexural resistance was validated for both bridges. The development and results of the grillage analysis is presented in greater detail in Chapter 4 and Appendix A.

## **Design Methodology**

A MBISB design methodology was developed by combining the data obtained from the laboratory and field tests with the requirements of the American Association of State and Highway Transportation Officials (AASHTO) Load Resistance Factored Design (LRFD) Bridge Specification for a steel girder/composite concrete deck bridge (19). The design methodology addresses the applicable strength and serviceability limit states pertinent to a MBISB structure constructed on a LVR.

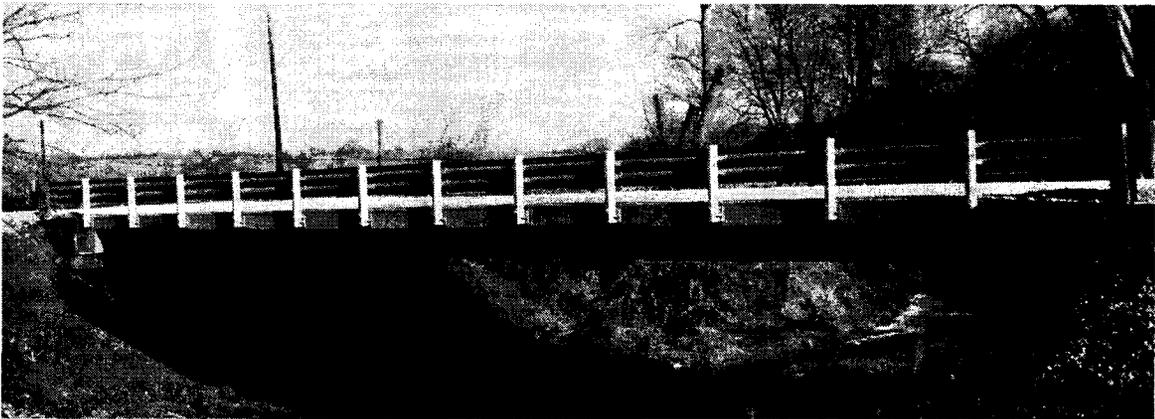
The design methodology was applied to create a data base of MBISB design requirements for a variety of geometries (i.e. bridge lengths and widths) and material strengths; this data base is presented in Volume 2 of the final report for Iowa DOT project TR-467 (2). In addition to the data base, a PowerPoint slide show documenting the construction of MBISB 2 is included to provide a guide for designers and construction managers. By applying the presented materials, additional MBISBs similar to the completed demonstration bridges shown in Figure 1.3, can be designed and constructed at a significant cost savings when compared to conventional designs.

## **Pre-Cast Modified Beam-in-Slab Bridge**

A derivative of the MBISB design has been developed by county engineers in Blackhawk County, Iowa that utilizes both the MBISB design concepts combined with pre-cast concrete technologies. The design consists of four pre-cast panels which incorporate the ASC to develop composite action and the transverse arch to reduce the self weight of the system. The four panels which make up the deck are constructed in the off-season and then transported to the construction site at a later date and connected by an in-field concrete pour. The resulting structure has a total cost savings of approximately 15% when compared to conventional designs for similar application. The ISU BEC field tested a pre-cast MBISB and determined it to exceed design requirements (20). A more detailed description of the design and the field test results are presented in Appendix B.



a. MBISB 1



b. MBISB 2

Figure 1.3 Completed MBISB demonstration bridges.

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## 2. BEAM-IN-SLAB LOW VOLUME ROAD BRIDGE SYSTEM

A paper accepted by and presented at the Eighth International Conference on Low Volume Roads

sponsored by:

Transportation Research Board

Travis F. Konda<sup>1,2</sup>, F. Wayne Klaiber<sup>1,3</sup>, Terry J. Wipf,<sup>1,4</sup>

### ABSTRACT

Funding for the repair and replacement of structurally deficient and functionally obsolete bridges is a nationwide problem. This problem is magnified for the managers of low volume road (LVR) systems with limited budgets; thus innovative replacement alternatives are always being sought. The beam-in-slab-bridge (BISB) is an alternative replacement bridge developed in Iowa. The bridge consists of W sections spaced 24 in. (610 mm) on center, with concrete filling the void space between them. Spans of up to 50 ft (15.24 m) can be constructed using this low cost alternative. Field and laboratory testing confirmed the system is capable of handling legal loads. Modifications to the BISB are being investigated in this study to improve its structural efficiency, making it possible to use the system in longer spans. An alternative shear connector (ASC) has been developed to provide composite action in the BISB. The ASC consists of holes, either torched or drilled, through the web of the girder forming concrete dowels that provide for composite action.

Ongoing research promises more structural efficiency with the development of an arched formwork system which makes possible wider girder spacing and reduced self weight. Combining the ASC with the arched formwork should result in an essentially steel free system that is relatively

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<sup>1</sup>Graduate Student, Distinguished Professor, and Professor respectively, Department of Civil, Construction and Environmental Engineering, Iowa State University

<sup>2</sup>Primary researcher and author

<sup>3</sup>Author for correspondence, researcher and author

<sup>4</sup>Researcher and author

easy to construct. Based on initial results it should be possible to construct spans of up to 75 ft (22.86 m) to carry legal loads in Iowa with these modifications.

## INTRODUCTION

It is a well known fact that America's transportation infrastructure is in constant need of maintenance and modernization to meet expected levels of service. The 2001 Infrastructure Report Card, published by the American Society of Civil Engineers (ASCE), rated 29% of the bridges in the United States as either structurally deficient or functionally obsolete (1). ASCE defines a structurally deficient bridge as one that "is closed or restricted to light vehicles because of its deteriorated structural components, these bridges must be posted for limits of speed and weight." Functionally obsolete bridges have older design features that make them unsafe for some vehicles and unable to handle current traffic volumes (1).

The numbers presented by ASCE represents all bridges, however, since agencies responsible for low volume road (LVR) bridges have very limited budgets, more than likely a greater percentage of low volume bridges (LVB) are either structurally deficient or functionally obsolete. The definition of a LVR is a matter of perception, but gravel roads in rural agricultural areas are what come to mind when LVRs are mentioned. While these systems don't have large volumes of traffic, they are subjected to heavy loads due to agricultural and off road equipment (2). LVBs on the LVR systems are often older, deteriorating structures that were not designed for current loads or vehicle widths. Repair and replacement of these inadequate structures is a costly proposition, leaving the managers of LVRs searching for alternatives. This is a national problem which is especially acute in the state of Iowa, which ranks 5<sup>th</sup> in the nation for the number of road structures (3) but only 30<sup>th</sup> in total population (4).

Iowa is a rural agricultural state with 89,200 miles of secondary roads and 20,855 off system structures. Approximately 82% of Iowa's bridges are on the secondary road system (3). Bridges on

secondary roads are considered to be off-system; referring to those roads and structures not cared for by state or federal forces, leaving the maintenance and replacement costs to the county governments.

### **Need for Bridge Alternatives in the Rural Sector**

Most Iowa counties have full time bridge crews with the capability to repair and construct short span bridges of 100 ft (30.48 m) or less. However, they lack the resources to design and construct traditional bridge systems. Thus, there is a need for a simple to construct, standard design, lower cost bridge replacement alternative for off system roads.

The Iowa Highway Research Board in conjunction with Iowa State University's Bridge Engineering Center joined resources to develop a LVB alternative. Alternative designs need to fulfill the following criteria:

- Support legal loads
- Be constructible by county forces
- Span a minimum of 50 ft (15.24 m) with the possibility of obtaining spans of at least 75 ft (21.34 m)
- Have a service life of at least 50 years
- Have the option of using recycled girders

### **ORIGINAL BEAM-IN-SLAB-BRIDGE CONCEPT**

The beam-in-slab-bridge (BISB) system, developed in Benton County, Iowa more than twenty years ago, is a successful LVB alternative. The original BISB system consists of W sections set on 24 in. (610 mm) center-to-center spacing. Plywood is placed on the top surface of the bottom flange to form a "floor" between the beams. The edges of the plywood are positioned approximately 3 in. (76 mm) from the beam webs to allow direct contact of the concrete with the bottom flange (Figure 1 (a)). Transverse steel straps are welded to the bottom flanges to support the steel members during the concrete placement. Forms are placed at the ends of the steel sections; the void between

the steel beams is filled with unreinforced concrete and struck off at the top flange of the steel beams leaving their top flanges exposed. The largest BISB in service is 30 ft (9.14 m) wide and 50 ft (15.24 m) long.

### **Performance of the Beam-in-Slab-Bridge**

To the authors' knowledge, the BISB is unique to Iowa; several extensive literature searches were conducted yielding no additional information on the BISB concept. The original design was in service in rural Iowa with minimal engineering analysis for over 15 years before the behavior of the structures was investigated. A majority of the 80+ BISBs in Iowa are in Benton County; they have excellent performance records and have required minimal maintenance (Lyle Brehm, former Assistant Benton County Engineer, unpublished data). Although the structures were extremely stiff, the BISB system was not field tested until 1996.

#### *Field Testing*

To quantify the behavior of the BISB system, an existing BISB [L = 47.5 ft (14.48 m), W = 30 ft (9.14 m), beams = 16 - W12x79, no guardrails] was field load tested. The bridge was instrumented with strain gages and displacement transducers. Two loaded rear tandem axle trucks with a combined gross weight of 102 kips (454 kN) were used as test vehicles. They were positioned on the bridge to produce maximum loading effects at critical locations and other various locations to determine load distribution.

The field test results indicated an extremely stiff structure in both the longitudinal and transverse directions. While numerous tests were run, only the maximum results are presented (5). The maximum resulting vertical deflection at the centerline was approximately 0.275 in. (7.0 mm); corresponding to a 1:2,180 deflection ratio, well within the recommended American Association of State Highway and Traffic Officials (AASHTO) Load Factored Resistance Design (LRFD) Bridge Design Specification (6) limit of 1:800 indicating the bridge is significantly over designed from a serviceability aspect.

### *Bridge Rating*

The tested BISB was rated using the 1994 AASHTO Manual for Condition Evaluation of Bridges (7) and the 1992 AASHTO Standard Specifications for Highway Bridges (8). Based on an arbitrary interior beam loaded with an HS-20-44 truck and AASHTO load distribution factors, the inventory rating for the field tested bridge was slightly over 40 tons (356 kN) (9).

### *Laboratory Testing*

To better understand the strength and load distribution behavior of the BISB, two representative specimens were constructed using W12x79 steel sections and tested in the laboratory; details of the specimens are presented below.

**Specimen #1 – Two Beam Specimen** Specimen #1, [L = 28.5 ft (8.69 m), W = 24 in. (61 mm)] was constructed following field procedures and was tested in four point bending to evaluate the amount of composite action and overall strength of a two beam section. The load was applied directly to the concrete portion of the cross section in an effort to punch through the unreinforced concrete (Figure 1 (b), (c)).

No composite action was developed; this was confirmed by measuring the slip between the concrete and the steel girders. The beams yielded at a total load of 120 kips (533 kN), and a punching failure did not occur in the concrete (5).

**Specimen #2 – Four Beam Specimen** The four beam specimen [Specimen #2: L = 28.5 ft (8.69 m), W = 72 in. (1830 mm)] was constructed and tested to determine the lateral load distribution characteristics of the BISB system (Figure 1 (d)). A nearly linear deflection distribution was found regardless of the load position indicating excellent lateral distribution.

The specimen was tested to failure by applying four point loading similar to Specimen #1. A total load of 300 kips (1,334 kN) was applied before yielding began. A maximum load of 370 kips (1,644.5 kN) with a corresponding deflection of 4.06 in. (103 mm) was reached before the test was terminated. Similar to the first specimen, the concrete between the girders did not fail (5).

### **Conclusions From Testing the BISB**

Based on the field and laboratory test results, the BISB was determined to be an effective, though not highly efficient, system for replacing deficient and obsolete bridges. Advantages of the BISB include simple design, speed and ease of construction and cost competitiveness. When compared to two traditional replacement systems, a pre-stressed, pre-cast quad tee product and a cast-in-place slab deck bridge, the BISB system reduced costs by up to 24%. This comparison is based on 2001 and 2002 construction data from Tama County, Iowa and Blackhawk County, Iowa. The above comparison includes the removal the existing structure and the construction of the new substructure and superstructure. This makes the BISB system a highly attractive replacement option for LVB with spans of up to 50 ft (15.24 m).

The main drawback of the original BISB system is structural inefficiency, which limits the span length. Also, the girder spacing is based primarily on constructability and not load demand, resulting in an over designed structure.

### **MODIFICATIONS TO THE BISB TO INCREASE EFFICIENCY**

Two modifications were proposed to improve the structural efficiency of the BISB. The first modification was the development of an Alternative Shear Connector (ASC) to create composite action in the section. The second modification involved using an arched formwork between the girders to removing a majority of the ineffective concrete on the tension side of the section. Removal of this concrete reduces self weight while facilitating wider girder spacing and longer spans.

#### **Alternative Shear Connector**

Since most counties lack the equipment for attaching shear studs to a girder, a simpler system was desired to obtain composite action. Although usually not a problem on LVRs, traditional shear studs are susceptible to fatigue failures. In hopes of overcoming potential fatigue problems, Leonhardt et al. developed the Perfobond Rib (5), consisting of a steel plate (typically 15 in. (380 mm) long)

perforated with holes welded on edge to the top flange of a girder. The concrete dowels that are formed in the holes develop composite action by resisting the horizontal shear and prevent material separation. Transverse reinforcement is necessary to prevent the concrete dowels from “popping out” at high loads (5).

Roberts and Heywood modified the Perfobond concept by removing the top flange of the steel section and drilled holes directly through the web of the resulting T section (5). This modification was evaluated in both shear box tests and push out tests; in addition, a full-scale bridge was designed and constructed. The bridge was subjected to both cyclic and ultimate static loading to quantify the behavior of the ASC in an actual bridge. No measurable deterioration or slip was recorded in either test providing evidence that the ASC could be used as an inexpensive method of obtaining composite action.

At approximately the same time as Roberts and Heywood’s work, Iowa State University researchers were conducting similar research to develop an ASC. The objective was to develop an ASC that involved either drilling or torching holes through the web of a rolled section with the top flange either removed or intact.

#### *Push Out Specimens*

To ensure the proposed ASC was applicable, a series of push out tests were performed. The testing program was divided into a static testing and a cyclic testing portion.

**Static Push Out Specimens** Thirty six push-out specimens (11 series), were statically tested in an effort to evaluate the following variables:

- Size of shear holes
- Spacing of shear holes
- Alignment of shear holes
- Inclusion of steel reinforcement through a shear hole
- Size of steel reinforcement through shear hole

- Effects of “sloppy” hole craftsmanship

A typical push out specimen is shown in Figure 2 (5).

Each specimen consisted of a stiffened steel plate partially encased in two concrete slabs. The thickness of the steel plate was selected to simulate the least beam web thickness encountered in the field. The specimens were cured and then statically loaded until the shear connectors failed. Test results indicated that the ASC has both excellent strength and ductile failure characteristics. The holes that were torched had slightly better performance than drilled holes, probably due to being slightly oversized. Results from the static testing indicated the strength of the ASC is primarily influenced by the following four factors (5):

- Concrete compressive strength
- Size of the shear holes
- Number of shear holes
- The amount of transverse slab reinforcement

Using regression analysis techniques, a design equation was developed to predict the strength of the ASC by accounting for the effects of the four listed variables (5).

**Cyclic Loaded Push Out Specimens** Once the static portion of the testing was completed, an additional 27 push-out specimens were constructed and subjected to 500,000 to 1,500,000 load cycles at varying percentages of the ultimate load to quantify the fatigue effect of both the load magnitude and the number of load repetitions. Results confirmed that while strength degradation of the ASC does occur under cyclic loading, the capacity is not reduced to an unacceptable level. Using a regression analysis model developed from the fatigue test data, the ASC was determined to have 69% of the ultimate static strength at 500,000 load cycles, 16% higher than the horizontal shear in assumed design conditions (10).

#### *Composite Section Beam Tests*

Push out tests, while providing information on the ultimate strength and slip characteristics of a shear

connector, only simulate the behavior that occurs within a composite beam. Seven full scale composite beam specimens were constructed in the laboratory from recycled W21x62 sections and subsequently tested to further quantify the behavior of the ASC.

**Static Beam Tests** Six beam specimens were tested under service level and ultimate static load conditions. Three of the six specimens had the top flange of the rolled steel section removed, while the other three specimens had the top flange intact. Four different ASC configurations were tested and compared with a specimen using traditional shear studs. Figure 3 (a), (b) illustrates a typical cross section and profile view of a specimen with the top flange intact.

All the specimens maintained full composite action at service level loads. The specimens with the top flange intact experienced less slip and had higher ultimate capacities than the specimens with the top flange removed. The increased capacity of the specimens with the top flange in place is due to the better confinement of the concrete dowels forming the shear connection (5), (10).

**Cyclic Loaded Beam Tests** One beam specimen with the top flange removed was subjected to cyclic loading to investigate the effects of fatigue on the ASC. The specimen failed at 464,000 cycles when the recycled steel beam failed due to fatigue. The failure occurred through a series of holes in the beam web where diaphragms had been bolted. The ASC was providing composite action at this level and had given no indication of deteriorating. It's likely the ASC would have performed adequately beyond the 500,000 load cycles recommended by AASHTO (10).

#### *Final ASC Design*

The final shear connector design is presented in Figure 3 (b). The ASC design consists of 1-1/4 in. (32 mm) diameter holes spaced 3 in. (76 mm) center to center with either a #4 or #5 reinforcement bar, depending on strength requirements, placed transversely in every fifth hole. The holes can be either torched or drilled with minimal difference in performance.

Three demonstration bridges have been constructed utilizing the ASC system and are

scheduled for field load testing during the spring of 2003. Results will be compared to the performance of the original BISB that does not have composite action.

### **Reducing the Concrete Area With Arch Formwork**

To improve the efficiency of the BISB, the girder spacing was increased to a maximum of 72 in. (1,830 mm) and the depth of the steel sections was also increased. The 72 in. (1,830 mm) girder spacing was based on geometric constraints. These modifications reduced the number of steel members required.

The main drawback of the original BISB design is the large dead load of the concrete. In order to improve the structural efficiency, a majority of the concrete on the tension side of the neutral axis needed to be removed. Therefore, the new formwork system needed to span at least 66 in. (1,676 mm), to allow for a significant reduction in concrete below the neutral axis.

#### *Arched Formwork*

The most logical solution to the previously stated problem was an arched formwork scheme that removed most of the unwanted concrete while remaining self-supporting and allowing for the use of the ASC. Several different materials and geometric configurations were investigated.

**Polyethylene Pipe** Polyethylene drainage pipe was used as the formwork for the first arched laboratory test specimen (Figure 4 (a)). Since the girder spacing was 45 in. (1,143 mm), the formwork was made from a section of 42 in. (1,067 mm) diameter pipe. The circular section reduced the amount of concrete needed by 36% and was freestanding. Similar to the plywood in the original BISB, the polyethylene pipe was a single use, stay in place formwork system. For larger girder spacings, polyethylene pipe is not an effective option due to increased material costs and limited geometries.

**Arched Plywood** Arching plywood between the girders was investigated as a possible formwork solution. This was attempted with W21x62 girders spaced 72 in. (1,830 mm) apart with minimal

success (Figure 4 (b)). Two layers of plywood with a thickness of 1/4 in. (6.4 mm) each were tested to see if a structurally adequate arch shape could be obtained.

In spite of previous testing and analytical modeling, when filled with concrete, reverse curvature snap through buckling began to develop. This required the formwork to be shored ruling out the arched plywood as a possible formwork solution.

**Culverts** Corrugated Metal Pipe (CMP) was used successfully as the arched formwork for one demonstration bridge, but the girders were only spaced 24 in. (610 mm) on center requiring a minimal span of 18 in. (457 mm). A 24 in. (610 mm) diameter, 16 gage CMP was cut into thirds and placed between the girders (Figure 4 (c)), removing 17% of the concrete needed between the girders.

**Custom Rolled Steel Sections** Circular sections, while readily available, are not the most efficient shape especially at larger girder spacings; they are also limited to standard sizes. Thus, an alternative to the circular section was sought.

CMP is rolled from 25 1/2 in. (648 mm) wide steel sections and then riveted together forming the pipe. Based this procedure, the concept of custom rolled arched steel formwork sections was developed. Two designs were chosen (small radius specimen and large radius specimen) and test specimens of each configuration were constructed to confirm the analytical analysis of the sections.

The small radius (15 in. (380mm)) formwork for W21x62 girders spaced at 72 in. (1,830 mm) was constructed from 14 gage galvanized steel with a 2 2/3 in. (67.5 mm) x 1/2 in. (12.7 mm) corrugation pattern. The large radius (27 in. (661 mm)) formwork specimen for W21x62 girders spaced at 72 in. (1,830 mm) were also made from 14 gage galvanized steel with the same corrugation pattern.

The large radius formwork removed 45% of the concrete needed to fill the section while the small radius formwork removed over 52% of the concrete volume resulting in a significant reduction of concrete. Since metal stay in place formwork is not a standard practice in Iowa, the sections are recoverable and available for use in other bridges.

### **Arching Action and the “Steel Free” Deck**

The original BISB had no steel reinforcement, relying solely on the punching shear strength of the concrete between the girders. This works well for girders spaced 24 in. (610 mm) on center where the concrete is significantly confined between the girder flanges. However, when the girder spacing is increased and the deck thickness is reduced by the arched formwork, a different mechanism is needed to resist the wheel loads on the deck.

#### *Internal Arching*

It is well researched and documented that internal arching occurs in a bridge deck slab when properly restrained. The AASHTO LRFD Bridge Design Specification (6) allows for the empirical design of deck slabs utilizing the internal arching action in the deck to resist the wheel loads. Failures in this system are not flexural in nature but rather are punching shear failures.

Canadian researchers have taken this concept a step further and constructed a steel free bridge deck. The steel free deck consists of steel girders with shear studs to provide composite action. A steel tie strap restrains the top flanges of the girders; research has shown the need to confine the top flange to prevent a premature flexural failure (11).

Iowa State University researchers are currently investigating the applicability of combining the ASC and the internal arching with an essentially steel free deck concept. The only reinforcing steel that will be present in the section is the transverse steel needed for the ASC and the necessary temperature and shrinkage steel to prevent excessive cracking. This portion of the research is currently underway; thus results will be available upon the completion of the project.

#### *Arched Specimens*

Four arched specimens have been constructed and tested to date. The following main objective of testing the specimens was to investigate and quantify the following:

- Ultimate strength of the section
- Failure mode of the section

- Ease and applicability of construction techniques
- Load distribution

All the specimens were constructed using recycled W21x62 girders and the ASC. The transverse ASC reinforcement serves a dual purpose in all cases, confining both the concrete dowels and allowing for the development of the internal arching action. The transverse steel was the only reinforcement present in the first three specimens. The fourth specimen had #3 reinforcing steel placed transversely across the girders to prevent possible spalling over the girders. Transverse steel straps were welded to the bottom flanges of the girders in all the specimens to restrain the girders during concrete placement and loading.

**First Arch Specimen** A cross section of the first specimen is presented in Figure 4 (a). Following the original BISB design, the top flanges of the girder were exposed and used as guides to strike off the concrete. The first specimen had the following properties: 2 girders set on 45 in. (1,143 mm) centers, a length of 33.5 ft (10.21 m), and polyethylene pipe formwork.

At service level loads, the specimen showed no signs of distress. The steel girders began to yield at a load of 126 kips (560 kN), indicating that the arched deck had a higher capacity than the girders in flexure. In order to investigate the punching shear capacity of the arched deck, the girders were blocked up and the specimen was reloaded. After blocking, the specimen failed in a splitting mode when the bottom flange straps failed at a load of 177 kips (787 kN).

**Second Arch Specimen** A cross section of the second specimen is presented in Figure 4 (b). The girders were fully embedded with 3 in. (76 mm) of cover over the top flanges of the girder. This modification was made to reduce the transverse steel, increase the moment of inertia of the section and provide for a more skid resistant deck. The second specimen had the following properties: 2 girders set on 72 in. (1,830 mm) centers, a length of 14.5 ft (4.42 m), and arched plywood formwork.

The clear span was selected so the arched deck would fail in a punching mode before the girders would fail in flexure. The second specimen was tested similarly to the first specimen and no

distress was present under service level loads. At an ultimate load of 155 kips (690 kN) a splitting/punching shear failure occurred in the deck when a bottom flange strap failed.

**Third Arch Specimen** The third specimen was constructed to investigate possible improvements to the second specimen. The large radius custom rolled corrugated steel sections were used as the formwork since the arched plywood did not perform adequately. A photograph of the third specimen set up for testing is presented in Figure 5 (a). The third specimen had the following properties: 2 girders set on 72 in. (1,830 mm) centers, and a length of 14.5 ft (4.42 m).

The corrugated steel formwork, which was removed prior to testing, performed flawlessly and became the preferred formwork system. As with the second specimen the selected clear span forced a punching failure in the deck. Larger confining straps were used on the third specimen and an ultimate load of 260 kips (1,157 kN) was reached. This test provided the assurance that the arch had sufficient strength to handle AASHTO HS-20 loadings.

**Fourth Arch Specimen** A fourth specimen, consisting of three monolithic bays was constructed to quantify the load distribution characteristics of the arch system. A photograph of the fourth specimen is presented in Figure 5 (b). The fourth specimen had the following properties: 4 girders set on 72 in. (1,830 mm) centers, a length of 30.5 ft (9.3 m), and small radius custom rolled steel sections were used for the formwork.

The specimen exhibited excellent load distribution under both service and ultimate loading conditions indicating that the transverse arch could be used in a demonstration bridge.

#### *Demonstration Bridge*

A 70 ft (21.34 m) long, 32 ft (9.75 m) wide, simply supported demonstration bridge utilizing the ASC and arched section has been constructed by county forces in Tama County, Iowa. The bridge deck was cast on November 7, 2002 and is not completely finished at this time. A photo of the construction of the bridge is presented in Figure 6. The bridge consists of 6 - W27X129 Grade 50 girders with 5 arched bays formed by custom rolled corrugated steel formwork sections. A side

mounted three-beam bridge railing was chosen for the guardrail system. The bridge will be load tested in the spring of 2003. Initial estimates suggest a total bridge cost savings of approximately 10% relative to a conventional bridge system.

#### *Future Research*

Using the information from the planned load tests in combination with the existing laboratory results, design specifications complete with interactive software and standard drawings will be developed for distribution to Iowa county engineers. While this project has focused only on the superstructure, Iowa State University researchers have recently undertaken an Iowa Department of Transportation sponsored project to investigate and develop numerous standard substructures for use with LVBs. The resulting designs will be governed by the same parameters as the superstructure systems, especially the constructability aspect since many Iowa county crews have the capability to drive piles.

## **CONCLUSIONS**

The repair and replacement of structurally deficient and functionally obsolete bridges on low volume roads presents a serious challenge to the agencies charged with maintaining them. While this is a nation wide problem, it is magnified in the state of Iowa due to the large number of structures on the LVR system. In an effort to find more economical solutions to replace these structures, the BISB concept was developed.

The original BISB is a practical design that is an adequate replacement alternative for structures up to 50 ft (15.4 m) in length. Simple to construct and maintain, the structure has been proven to be more than adequate by use, field and laboratory testing, and analytical analysis. The main disadvantage to the system is a low efficiency due to excessive self weight and a lack of composite action.

To make the BISB more efficient, the ASC was developed. This simple system of cutting holes in the webs of the steel girders to provide composite action has been proven to be highly

effective in the laboratory and with sufficiently strength for both service level and fatigue loads. Future field tests and continued use will help to confirm the ASC as an applicable method of obtaining composite action, making the BISB system more efficient.

The arched steel free deck system also increases the efficiency of the original BISB design by allowing for greater girder spacings and less concrete self weight. Laboratory tests and analysis confirmed that the system is an applicable superstructure that can be used for bridges up to 75 ft (22.86 m) in length, an increase of 50% over the original BISB design.

A 70 ft (21.34 m) long demonstration bridge combining the ASC and the transverse arch section has been successfully constructed in Tama County, Ia. and will be service load tested to further investigate the behavior of this system in the spring of 2003.

#### **ACKNOWLEDGEMENTS**

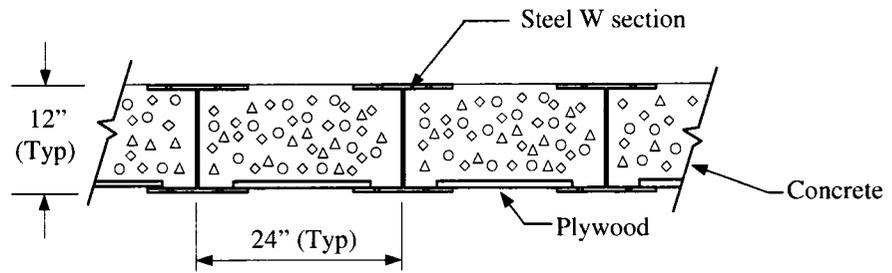
The study presented in this paper was conducted by the Bridge Engineering Center at Iowa State University through funding provided by the Iowa Department of Transportation, Highway Division and the Iowa Highway Research Board. The authors wish to thank the county engineers and Doug Wood, ISU Research Laboratory Manager for their help with the construction, instrumentation, and field testing of the demonstration bridges.

The opinions, findings, and conclusions expressed in this publication are those of the authors and not necessarily those of the Iowa Department of Transportation.

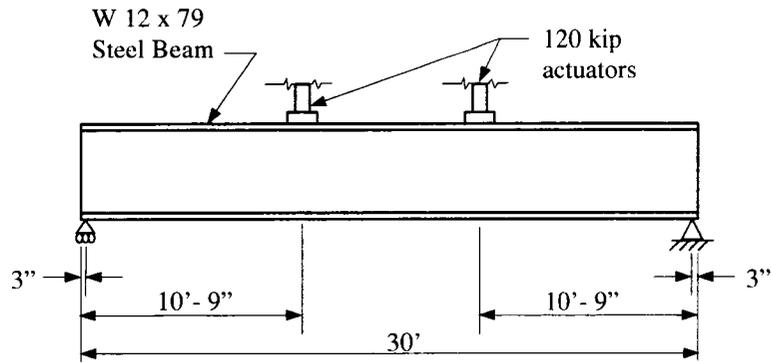
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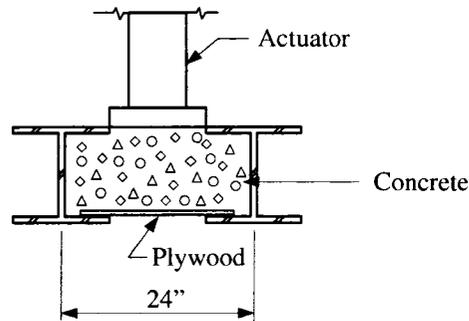
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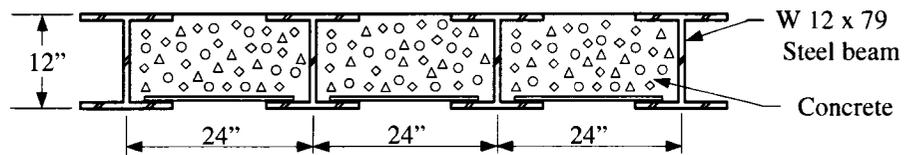
(a) Typical section of the original BISB



(b) Loading set up of 2 beam specimen



(c) Cross Section of the 2 beam specimen



(d) Cross section of the 4 beam specimen

Figure 1 Original beam-in-slab-bridge (BISB).

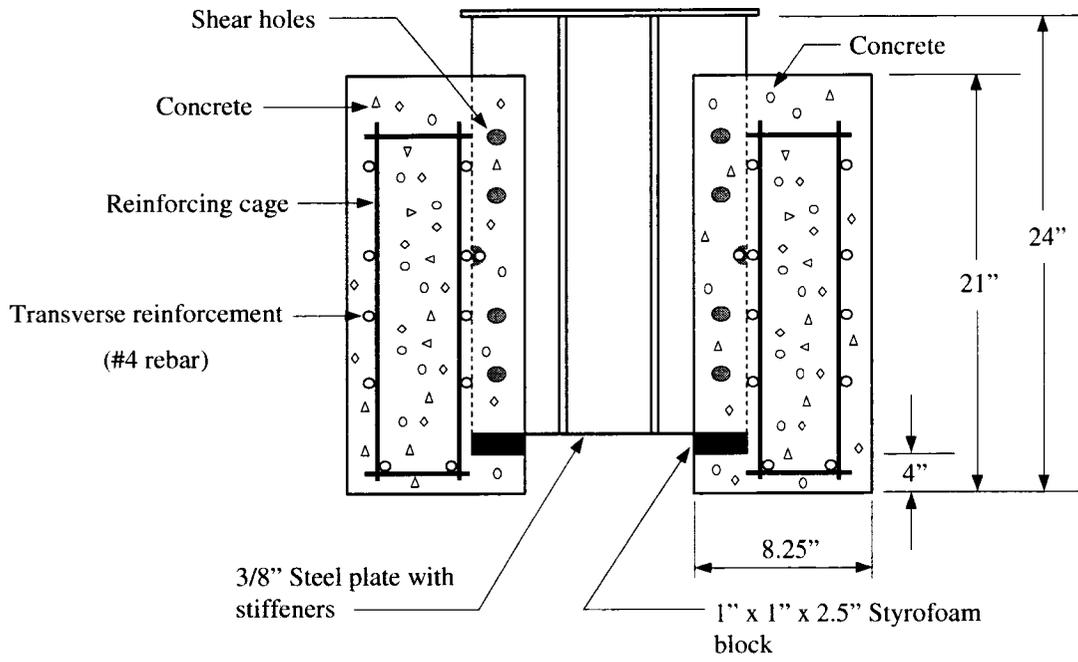
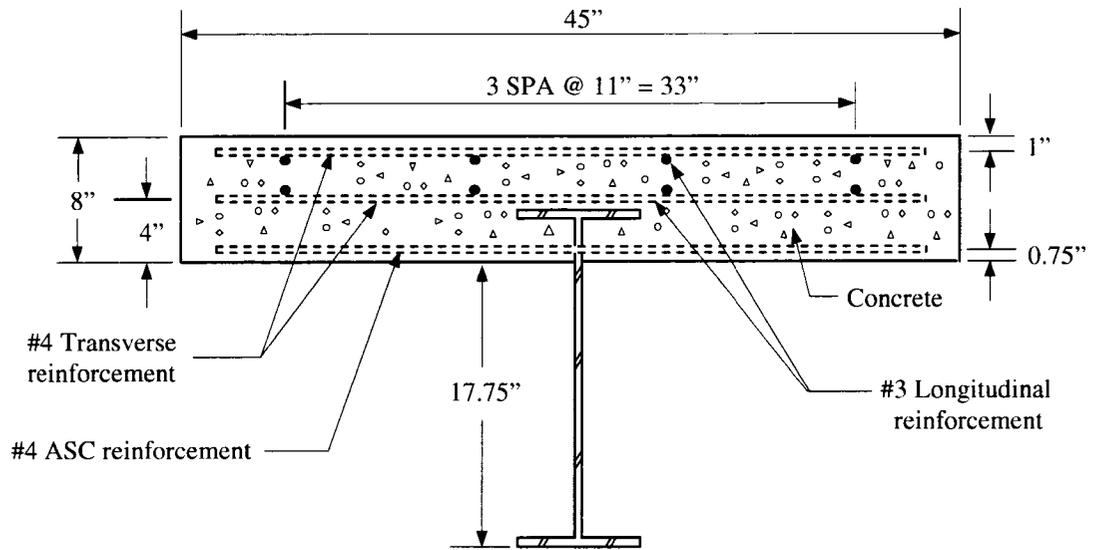
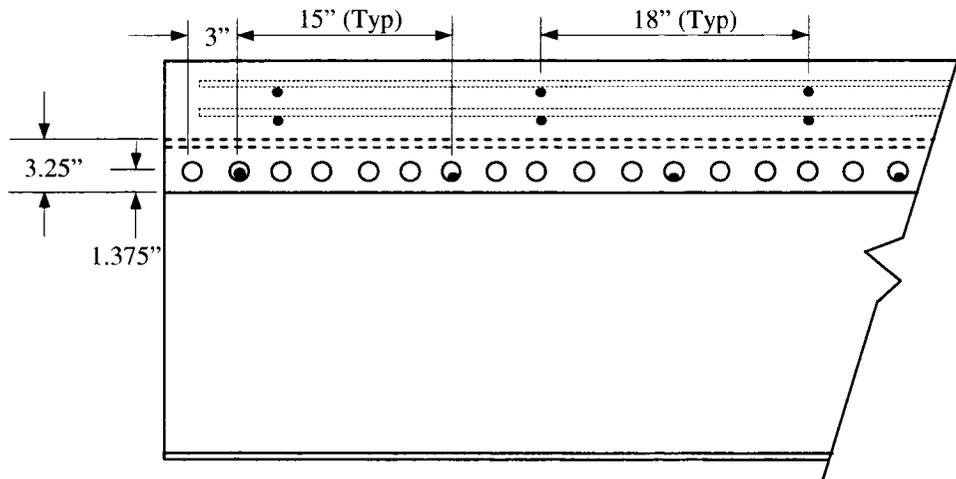


Figure 2 Push out specimen setup.

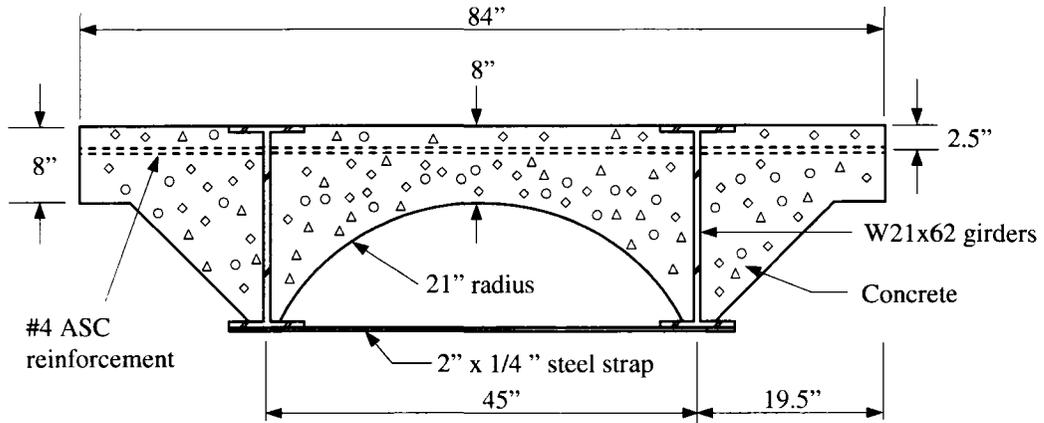


(a) Cross section of composite beam for the ASC tests

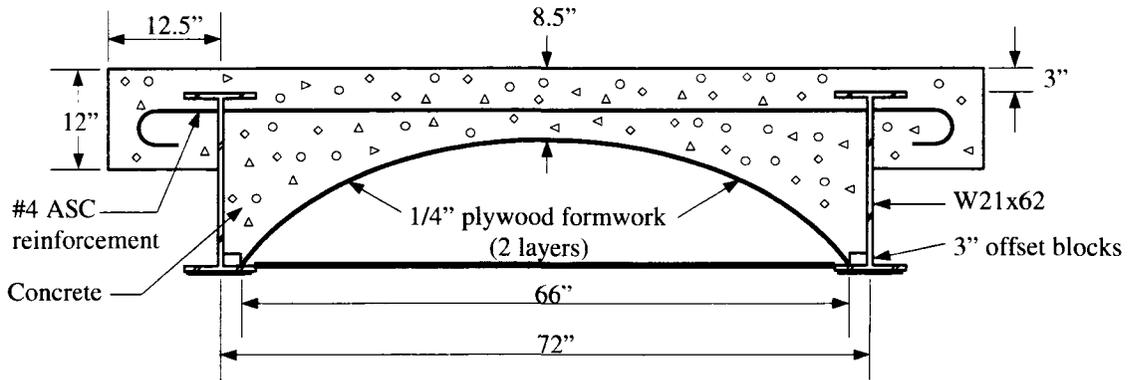


(b) Profile view of the ASC

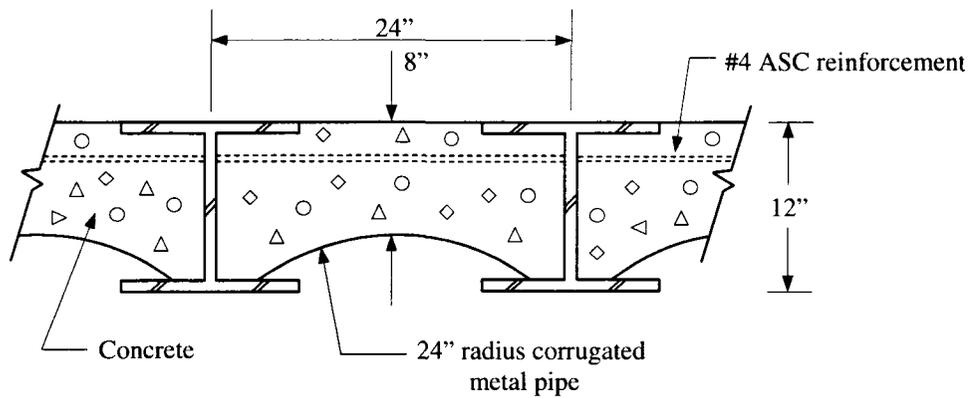
Figure 3 Typical composite beam used in the alternative shear connector (ASC) tests.



(a) Cross section of the first arched specimen, polyethylene pipe formwork

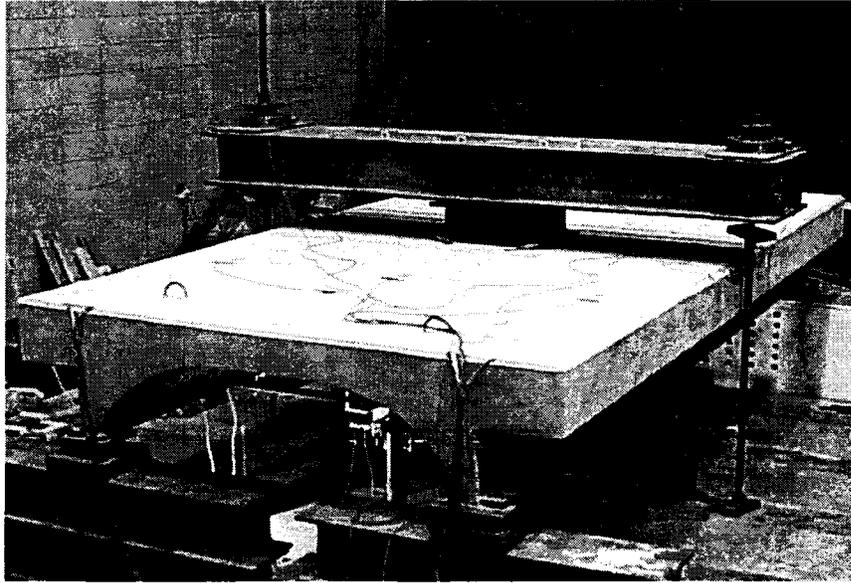


(b) Cross section of the second arched specimen, arched plywood formwork

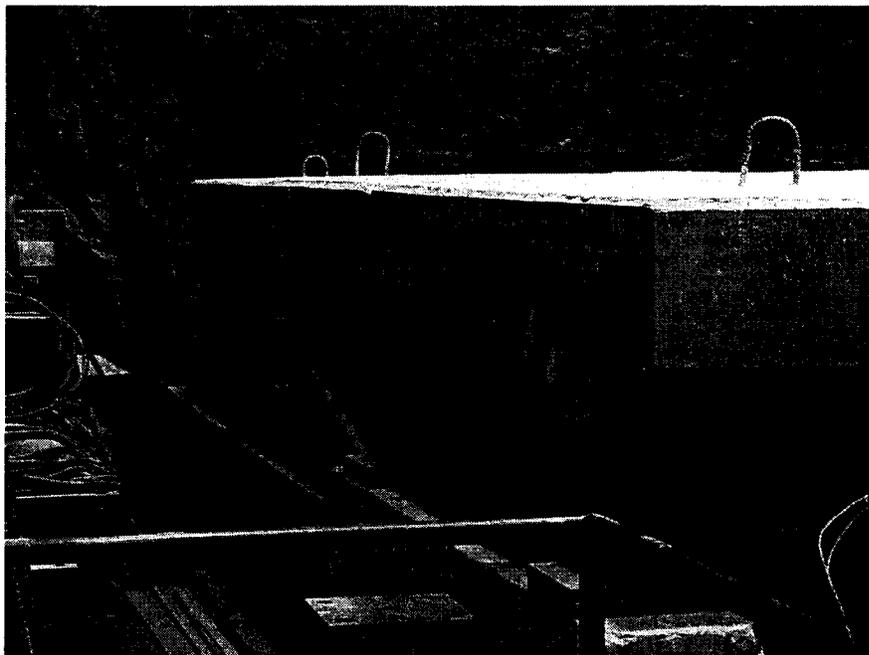


(c) Typical cross section of arched culvert formwork

Figure 4 Arched deck specimens.



(a) Testing set up for the third arched specimen



(b) End view of the fourth arched specimen

Figure 5 Arched deck specimens #3 and #4.



Figure 6 Photograph of the construction of the demonstration bridge.

### 3. LABORATORY INVESTIGATION OF THE MODIFIED BEAM-IN-SLAB BRIDGE SYSTEM, AN ALTERNATIVE BRIDGE REPLACEMENT FOR LOW VOLUME ROADS

A paper to be submitted to the Canadian Journal of Civil Engineering

Travis F. Konda<sup>1,2</sup>, Terry J. Wipf<sup>1,3</sup>, F. Wayne Klaiber<sup>1,3</sup>

#### ABSTRACT

To extend available resources, managers of Iowa's Low Volume Roads (LVRs) and bridges have expressed interest in developing low cost alternative bridge designs for replacing deficient structures. The Beam-in-Slab Bridge is one such alternative design that consists of W12 sections spaced on 24 in. (610 mm) centers. Unreinforced concrete is placed between the sections to form the deck. While simple to construct and lower in cost than comparable designs, spans are limited to 50 ft (15.24 m) due to self weight stresses and deflections. Two modifications (developing composite action and reducing self weight) were proposed so that the system could be used in larger spans.

An Alternative Shear Connector (ASC), which requires no specialized equipment to install, was developed; the ASC consists of holes in the webs of the girders through which plastic concrete passes. After the concrete cures, shear dowels are formed. The self weight is reduced by introducing a transverse arch between the longitudinal girders to remove the majority of concrete in tension. The modifications were evaluated through laboratory testing of three single bay specimens and a three bay model bridge to determine the strength, mode of failure and global behavior of the structures with the modifications. Results indicated that the system, referred to as the Modified

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<sup>1</sup>Graduate Student, Distinguished Professor, and Professor respectively, Department of Civil, Construction and Environmental Engineering, Iowa State University

<sup>2</sup>Author for correspondence, primary researcher and author

<sup>3</sup>Researcher and author

Beam-in-Slab Bridge, had significantly more strength than that required to resist legal loads. The modifications improved the efficiency, thus reducing the self weight and increasing the strength so that the system can be used in spans up to 80 ft (24.38 m).

**Keywords:** composite action, deck slabs, girder bridge, internal arching, low volume roads, punching shear

## INTRODUCTION

According to data from the Federal Highway Administration (FHWA), 28% of the 25,000 plus bridges in the State of Iowa are either structurally deficient or functionally obsolete (2004a). This percentage increases to 31% when only the 19,665 structures owned by Iowa Counties (referred to as off-system structures) are considered (FHWA 2004b). Many of the off-system structures are located on roads that have an Average Daily Traffic (ADT) less than 400 vehicles; such roads are designated as low volume roads (LVRs) (American Association of State Highway Transportation Officials (AASHTO) 2001).

Maintaining this population of aging structures (average age of a county bridge in Iowa is 45.5 years (FHWA 1995, 2002)) is an economic challenge with replacement often being the most cost effective solution. One method of extending replacement funds is through the use of alternatives which cost less than comparable conventional designs.

The Beam-in-Slab Bridge (BISB) is one such alternative; presently there are over 80 of these bridges in service. The BISB is very simple, consisting of longitudinal W12 beams on 24 in. (610 mm) centers with an unreinforced concrete “deck” placed between them. Formwork consists of stay-in-place plywood that rests on the top surface of the bottom flanges of the longitudinal beams. After placement, the unreinforced concrete is struck off even with the top flanges. A typical BISB cross section is shown in Figure 1.

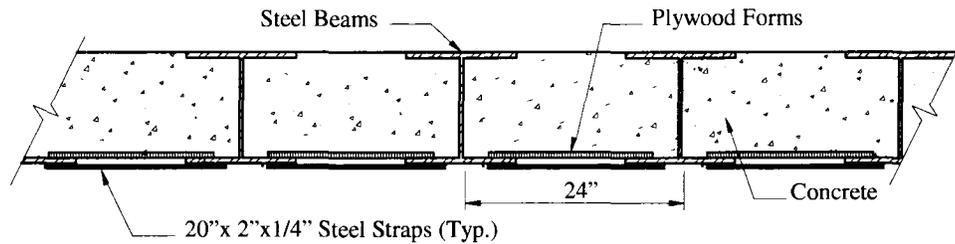


Figure 1 Cross section of a BISB.

An existing BISB [ $L = 47.5$  ft (14.48 m),  $W = 30$  ft (9.14 m)] was field tested; deflections measured in these tests revealed a very stiff structure with a maximum deflection ratio of  $L:2,180$ , which is well within serviceability limits. Two laboratory BISB models were also constructed and evaluated under service and ultimate loads; results from the laboratory tests verified that the BISB system meets strength and serviceability criteria (Klaiber et al. 1997).

The BISB has many advantages: simplistic design, can be constructed by in house forces, and is cost competitive. According to county engineers in Blackhawk and Tama Counties, Iowa, the BISB system costs approximately three-fourths that of a conventional bridge. Such attributes make the BISB system an attractive alternative; however, the system is limited to spans of 50 ft (15.24 m) or less due to large self weight deflections and stresses. The BISB design also could be more efficient if composite action were developed.

## PROPOSED MODIFICATIONS

Two modifications to the original BISB design were proposed with the objective of increasing the applicable span length. One modification was to reduce the self weight by removing a portion of the concrete from the cross section. The other was to develop composite action thus increasing the flexural stiffness and improving the structural efficiency. Any modifications made had

to be simple enough so that the resulting system could be constructed by in house forces. An increase in the applicable span length to at least 75 ft (22.86 m) was desired by Iowa County Engineers.

### **Alternative Shear Connector**

In most composite concrete/steel girder bridges constructed today, shear studs are used to develop composite action between the steel girders and the concrete deck. However, the installation of the studs requires special equipment which is not readily available in most counties. Therefore, an Alternative Shear Connector (ASC) that could be fabricated by county crews with readily available equipment was developed.

Expanding upon the work of Leonhardt et al. (1987), and Roberts and Haywood (1994), an ASC was developed that can be fabricated with either an oxyacetylene torch or a drill. The ASC consists of 1 1/4 in. (32 mm) diameter holes torched or cored through the web of each longitudinal girder. The holes are spaced on 3 in. (76 mm) centers for the length of the girder and are positioned one diameter below the bottom of the top flange. Reinforcement (#13 or #16,  $f_y = 410$  MPa) is placed through every fifth hole (15 in. (380 mm) spacing) to confine the concrete shear dowels which are formed when plastic concrete flows through the holes and cures. The ASC is readily incorporated in the original BISB design since the concrete fills the void between the girders and is in direct contact with the girder webs. The strength and behavior of the ASC were determined from laboratory tests which included static and cyclic push out tests as well as flexural beam tests (Klaiber et al. 1997) and (Klaiber et al. 2000).

### **Transverse Arch**

As previously noted, to increase the span length of the BISB, a reduction in its self weight was required. Recognizing the unreinforced concrete on the tension side of the neutral axis is ineffective and does not contribute to the flexural strength, this concrete can be removed, thus reducing the self weight, while not significantly altering the behavior. A transverse arch supported on

the bottom flanges of the longitudinal girders was selected as a means of removing a majority of the ineffective concrete.

By incorporating the ASC and the transverse arch in the original BISB design, a more efficient replacement bridge (referred to as the Modified Beam-in-Slab Bridge (MBSIB)) that can span 80 ft (24.38 m) resulted. In the previously indicated investigations, the ASC developed composite action that in some cases out performed conventional shear studs. However, the combination of the ASC and the transverse arch had yet to be evaluated; likewise, procedures for constructing the transverse arch were also unknown.

## **SPECIMEN FABRICATION/TESTING**

A laboratory investigation was undertaken to determine the applicability of the MBISB for use on LVRs. The service level response, ultimate strength and modes of failure of the combined modifications were evaluated by designing, constructing and testing three single bay specimens and a three bay model bridge.

### **Single Bay Specimens**

The single bay specimens were tested to determine the mode of structural resistance and the capacity of the transverse arched deck. In previous research performed by Mufti and Bakht (e.g. 1993, 1996), it was determined that when a bridge deck is adequately confined, the applied loads are resisted by internal arching action. In their research, bridge decks were confined in the longitudinal direction by composite girders and by external straps or internal reinforcement in the transverse direction. In-plane arching action places the deck concrete in compression and if it is overloaded, a punching shear failure rather than a transverse flexural failure will occur. By changing the mode of failure, the amount of reinforcement required can be significantly reduced; if external transverse straps are used, such reinforcement in the deck slab can be completely eliminated (Newhook and Mufti 1996).

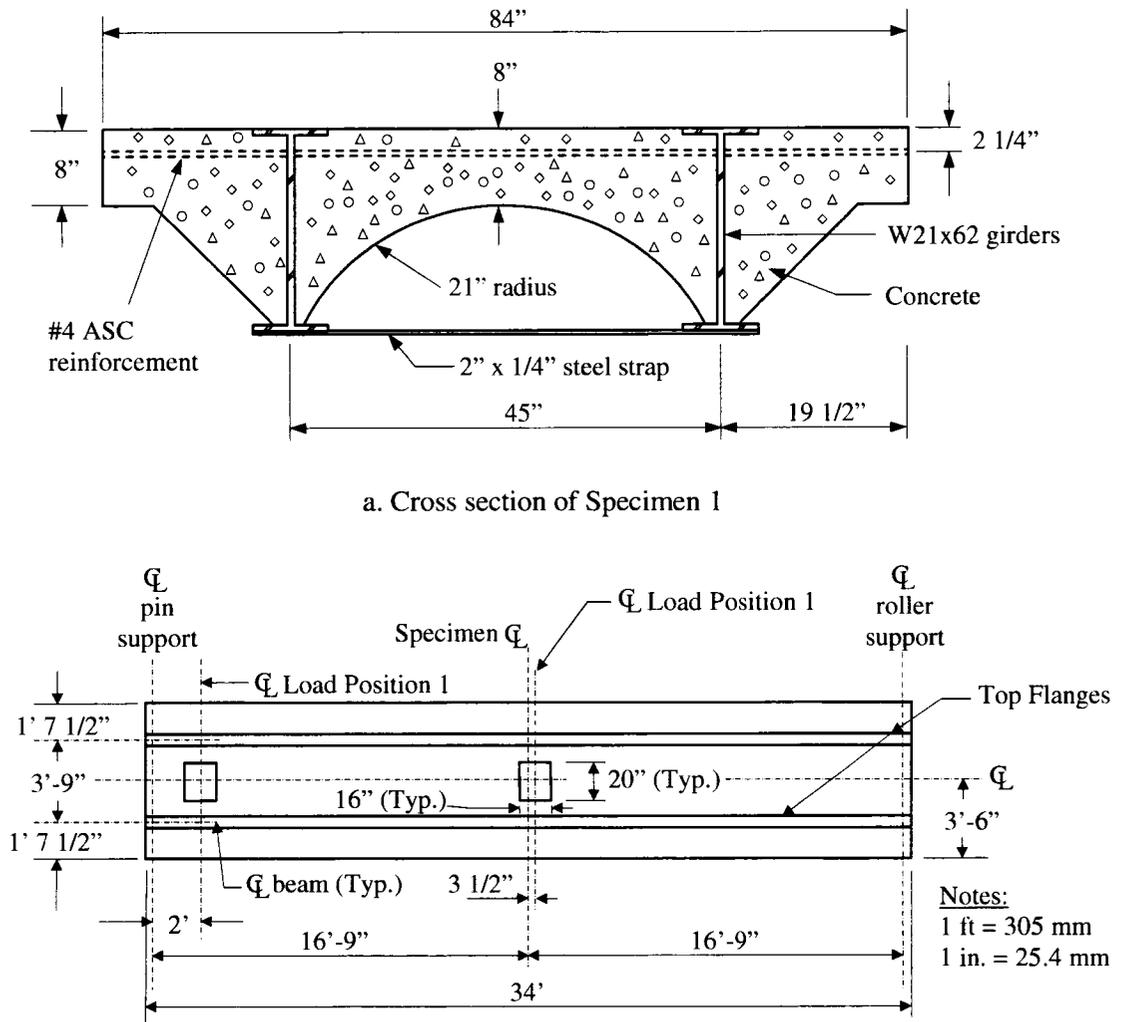
A key aspect of the laboratory investigation was the development of arching action for structural resistance in the transverse arched deck. By implementing the ASC, the arched deck would be confined in the longitudinal direction by the composite girders. Transverse confinement would be provided by the combination of the ASC reinforcement and transverse straps welded to the bottom flanges of the longitudinal girders.

Developing a low cost, simple to construct, self supporting arched formwork system that removed a majority of the ineffective concrete was a significant portion of the laboratory investigation. Numerous systems were evaluated including polystyrene foam blocks, air bladders and prefabricated arched concrete sections but all were cost prohibitive. Three arched formworks, consisting of a system constructed from sections of polyethylene pipe, an arched plywood system and a custom rolled steel pan system were investigated.

#### *Specimen 1*

With the implementation of the transverse arch, the use of deeper girders set at a wider spacing was possible, resulting in a more efficient flexural section. The first single bay specimen was constructed using 2-W21x62 A36 girders set on 45 in. (1143 mm) centers and spanned 33 1/2 ft (10.21 m). The ASC was incorporated to develop composite action by torching holes in the webs of the girders at the previously described spacing. Reinforcement (#13,  $f_y = 410$  MPa) was placed through every fifth hole completing the ASC. The deck was stiffened at the free ends by placing four additional lines of ASC reinforcement to maintain the in-plane arching action (Bakht and Agarwal 1995). Due to the expected arching action within the deck, the total reinforcement was limited to that required to complete the ASC. This configuration of the ASC holes and reinforcement was used in all three single bay specimens.

The transverse arch was formed from a section of 42 in. (1,066 mm) diameter polyethylene pipe placed between the two girders removing approximately 36% of the concrete that is in the BISB. A cross section of Specimen 1 is presented in Figure 2a. Similar to the BISB system, 2 in. x 1/4 in.



a. Cross section of Specimen 1

b. Load positions on Specimen 1

Figure 2 Configuration of Specimen 1.

(51 mm x 6.4 mm) steel straps were welded to the bottom flanges of the longitudinal girders at the 1/3 points to prevent the girders from spreading apart during the concrete placement; the straps were left in place during the testing of the specimen. The deck concrete was placed and struck off even with the top flange of the girders following the original BISB design (Klaiber et al. 1997). The specimen was instrumented to measure strains and deflections. Strain gages were placed at the quarter, mid and three quarter spans on the top and bottom flanges of the girders to quantify the

flexural behavior. Deflection transducers were installed under the girders at the previous locations as well as directly under the two loading points, Load Point 1 (LP1) and Load Point 2 (LP2) which are shown in Figure 2b. Load was applied through a 20 in. by 16 in. (610 mm x 406 mm) pad, whose dimension were based on AASHTO LRFD Bridge Specifications, simulating a typical tire contact area. The load pad was offset 3 1/2 in. (89 mm) from the midspan at LP1 so that it was between the adjacent transverse reinforcement, thus creating a worst case loading.

After a “shake down” loading (applied to all specimens), a service load of 45 kips (200 kN), simulating a factored HS-20 truck wheel load, was applied at LP1. With the completion for the service level test at LP1, an ultimate load was then applied; the entire sequence was then repeated with the load applied at LP2.

#### *Specimen 2 and 3*

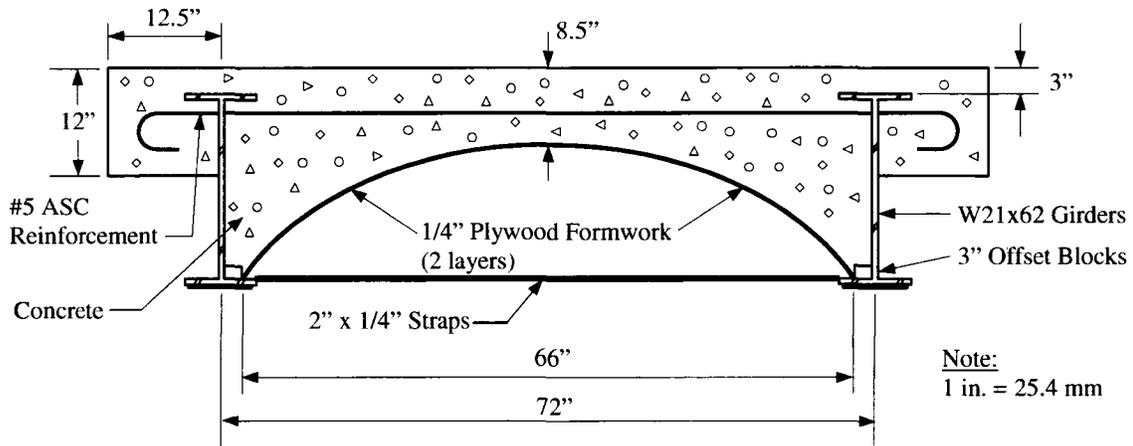
Two additional single bay specimens (Specimen 2 and 3) were constructed using 2-W21x62 A36 girders with 6 in. x 3/8 in. (152 mm x 10 mm) A36 cover plates welded to the bottom flanges. For both specimens, the girder spacing was increased to 72 in. (1,830 mm) to investigate the applicable range of the modifications. The specimen span length was limited to 14.5 ft (4.42 m), significantly increasing the yielding moment to make a deck failure the critical mode of failure.

In the BISB design, the top flanges of the girders are exposed to the elements and while problems with corrosion and skidding have not been reported, the option of encasing the girders to reduce such possibilities was desired. Therefore, the girders for Specimens 2 and 3 were embedded with 3 in. of concrete cast over the top flanges, increasing the longitudinal flexural stiffness by approximately 34%. Deck reinforcement (#16,  $f_y = 410$  MPa) was limited to that required for the ASC and to stiffen the free ends of the arched sections.

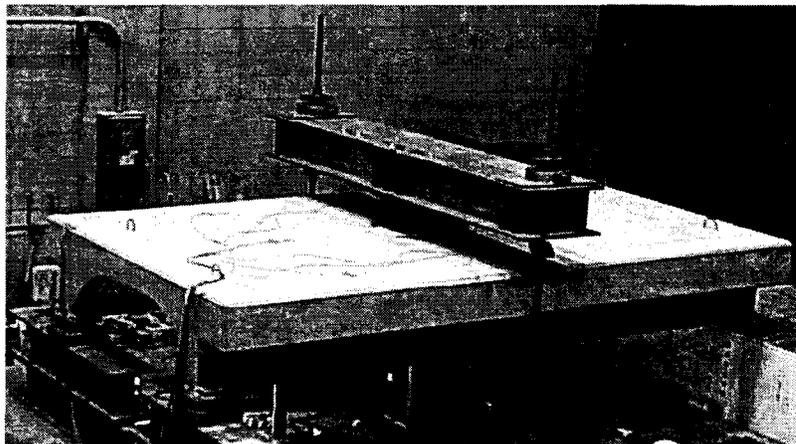
Due to the wider girder spacing, circular arched formwork cut from a section of pipe was no longer applicable. Thus, a formwork system consisting of arched plywood strips was implemented for Specimen 2; two layers of 1/4 in. (6 mm) thick plywood, 75 in. (1,905 mm) long x 24 in.

(310 mm) wide were soaked in water and wedged into place between the girders. This single use system removed approximately 40% of the concrete but was difficult to construct and structurally inadequate; thus, this formwork system was not a feasible option in future construction. The cross section of Specimen 2 is shown in Figure 3a.

Custom rolled formwork, constructed from 14 gage (2 2/3 in. x 1/2 in. (68 mm x 13 mm)) corrugated metal, normally used for corrugated metal pipe, was developed for Specimen 3. The removable/reusable arch sections were formed by bolting together two individual components. A



a. Cross section of Specimen 2



b. Loading configuration

Figure 3 Geometric configuration and load placement on Specimens 2 and 3.

removable formwork system was preferred, since stay-in-place formwork is contrary to Iowa Department of Transportation standard practice. The self-supporting custom rolled system removed 45% of the concrete from the section and became the recommended formwork system.

Transverse straps were welded to the bottom flanges at the 1/5 and 4/5 points of the span in both specimens. Both straps had fully tensioned double shear bolted connections incorporated to isolate the middle portion of the strap, creating a fracture critical section at the bolt holes. The straps for Specimen 2 consisted of 2 in. x 1/4 in. (51 mm x 6 mm) A36 steel sections, while those used in Specimen 3 were 3 in. x 3/8 in. (76 mm x 10 mm) A36 steel.

The instrumentation used in the testing of Specimens 2 and 3 was similar to that used in the testing of Specimen 1 with strain gages and deflection transducers being placed at the mid and quarter span points. The transverse straps were also instrumented with strain gages to quantify the amount of lateral confinement provided. Service and ultimate loads were applied to both specimens using the load pad that was used on Specimen 1. The load was positioned about the centerline of the respective specimens and was offset from the midspan by 3 in. (76 mm) so that it was between the adjacent transverse reinforcement. The position of the load on Specimen 3 is shown in Figure 3b. Both specimens were subjected to a 45 kip (200 kN) service load followed by ultimate load tests that resulted in the failure of the respective specimens.

### **Model Bridge**

The single bay specimens, while providing information on the capacity of the combined modifications, do not provide information on their behavior when implemented in a full scale bridge. A three bay model bridge, 30.5 ft (9.3 m) long by 20 ft (6.1 m) wide was constructed using 4-W21x62 A36 girders set on 72 in. (1,830 mm) centers to investigate the lateral load distribution, the flexural capacity and the punching strength of the arched deck. Like in the previous specimens, composite action was developed by incorporating the ASC. Reinforcement was limited to that required in the ASC, stiffening the free ends of the specimen and controlling possible cracking. An additional

transverse layer of #10,  $f_y = 410$  MPa reinforcement was placed over  $1/2$  the model resting on  $3/4$  in. (19 mm) chairs positioned on the top flanges of the girders to determine if it is required to prevent spalling or excessive cracking directly over the embedded flanges.

The transverse arch was formed using the custom rolled formwork; the arch profile, shown in Figure 4, was further modified to eliminate approximately 52% of the concrete from the cross section. After the deck concrete had cured, the formwork was removed and cleaned for future use.

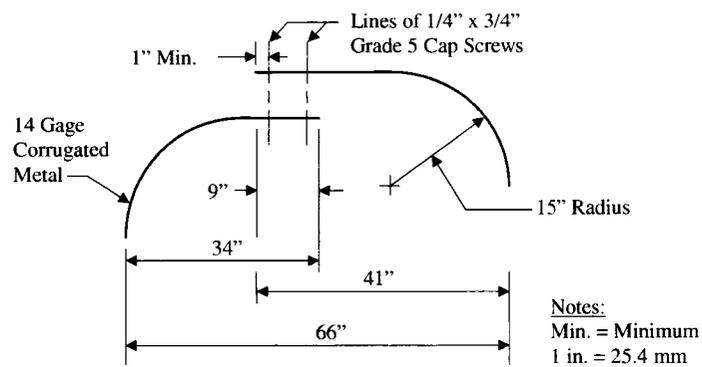


Figure 4 Profile of the custom rolled formwork used in the model bridge.

Diaphragms (W6x9 A36 sections) were installed  $1 \frac{5}{8}$  in. (41 mm) below the top flange of the longitudinal girders at the midspan to brace the compression flange during the placement of the deck. At the west end of the model (see Figure 5) a 12 in. (305 mm) thick concrete backwall was cast with the deck to simulate in-field conditions. End bracing consisting of W6x9 A36 sections bolted 3 in. above the bottom flange of the longitudinal girders were installed on the opposite end. Transverse straps (3 in. x  $3/8$  in. (76 mm x 10 mm) A36 steel) were welded to the bottom flanges of the longitudinal girders at the third points. The straps were installed with double shear bolted connections isolating the middle portion of the strap, allowing for their removal.

To evaluate the flexural response of the model bridge, both strain gages and deflection transducers were installed at the quarter, mid and three quarter span points. The three diaphragms

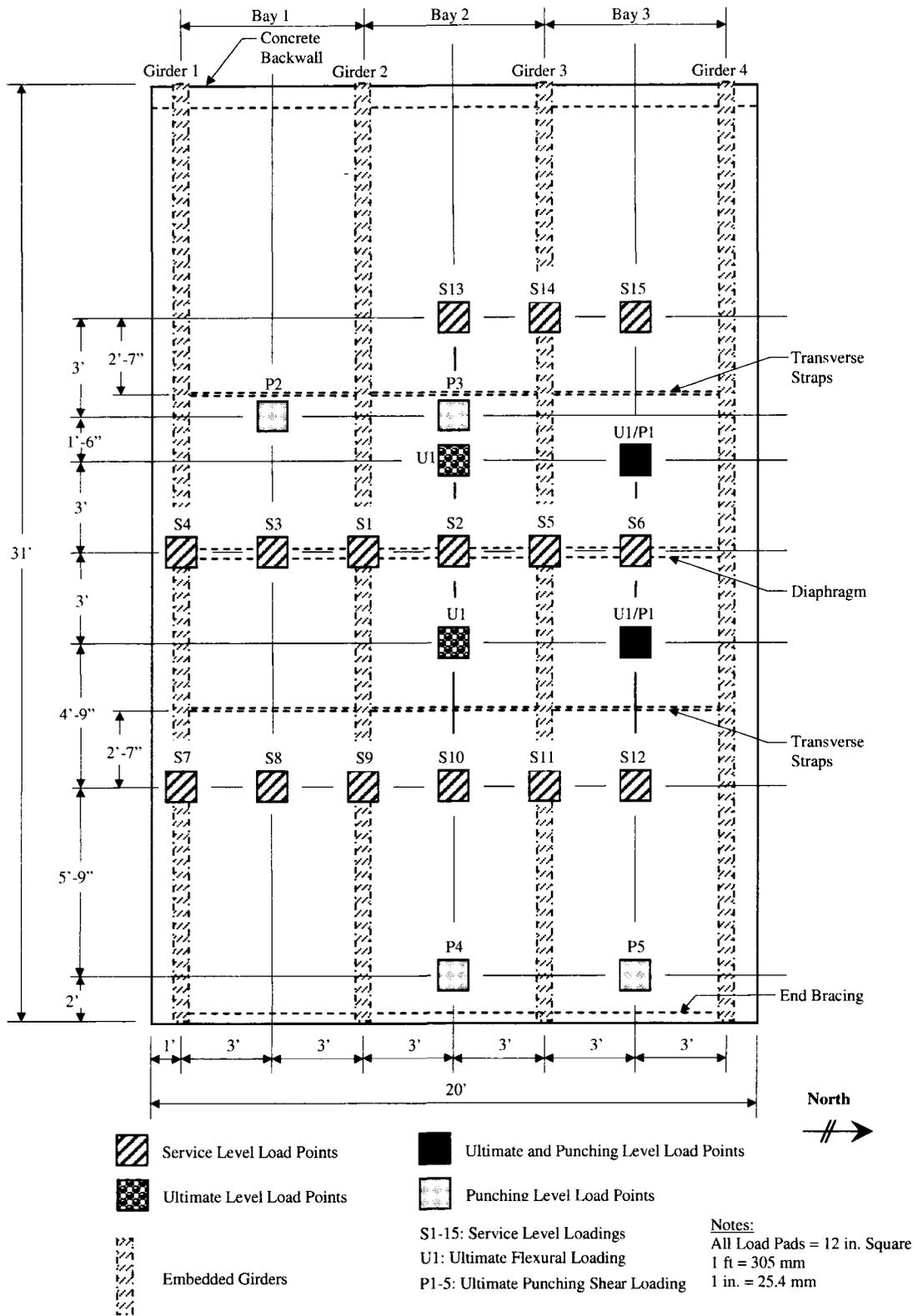


Figure 5 Layout of service and ultimate loading points on the model bridge.

and two end brace members had strain gages on the top and bottom flanges at the midspan of each bay. Strain gages were installed on the transverse straps to determine the amount of confinement provided and necessity to the integrity of the transverse arch. The elimination of the transverse straps was desired since their installation involved welding on the fracture critical tension flange of the longitudinal girders.

Due to the specimen size and equipment limitations, all loads were applied to the model bridge through 12 in. x 12 in. (305 mm x 305 mm) load pads, a more severe condition than was used on the previous specimens. To determine the lateral load distribution, 45 kip (200 kN) service loads were applied at the fifteen locations indicated in Figure 5. With the completion of the service loadings, a global ultimate flexural test was performed, followed by localized punching shear tests of the deck. A photograph of the model bridge subjected to the ultimate flexural loading is shown in Figure 6.

## **TESTING RESULTS**

For all specimens, elastic behavior was observed for the service loads and each specimen had more than adequate structural strength to resist a factored HS- 20 truck wheel load. In the following paragraphs, test results from the three single bay and the three bay model bridge are presented.

### **Specimen 1**

In the service load tests, when load was applied at LP1, the maximum recorded strain was approximately 1/3 the yield strain of the girders, validating the elastic response of the system (see Figure 7a). The experimental neutral axis was determined to be 7.75 in. (197 mm) below the top surface of the deck; corresponding to the theoretical value of 7.66 in. (195 mm) indicating the ASC had developed composite action.

For the ultimate loading at LP1 (see Figure 3b) , the steel girders yielded at 126 kips (560 kN) and deflected 5.6 in. (142 mm) at midspan with minimal distress in the arched deck. The load was

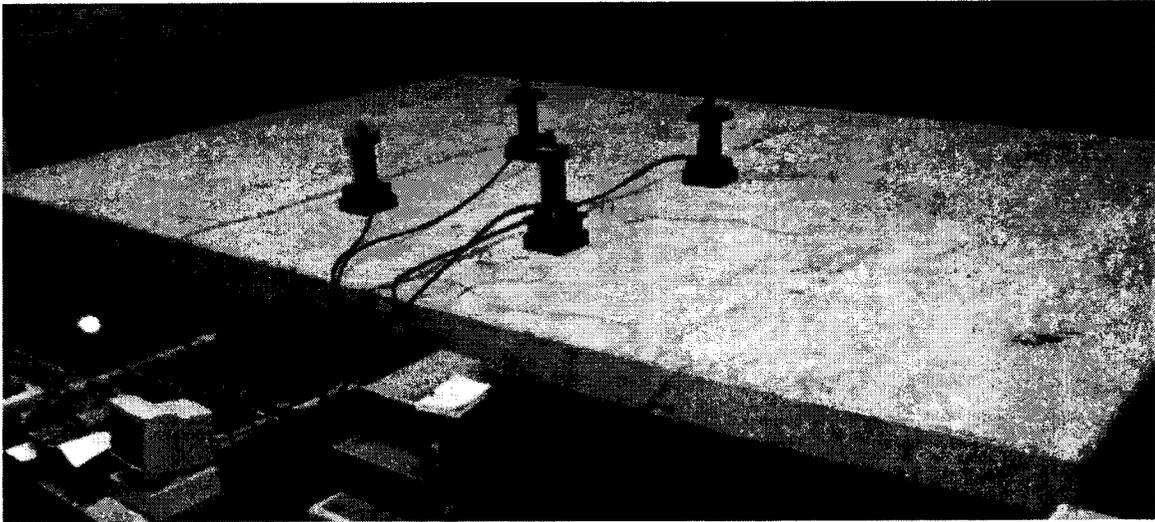


Figure 6 Ultimate flexural load test of the model bridge.

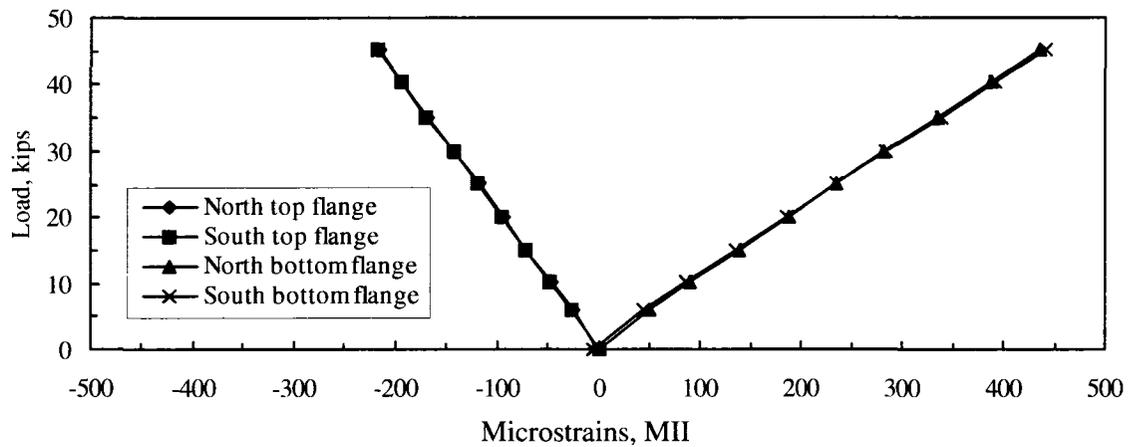
removed and supports were added at the midspan so that punching strength data could be obtained.

When load was reapplied; a weld on one of the transverse straps fractured at 177 kips (787 kN) causing specimen to split; loading was continued with the deck failing in a punching/splitting mode at 242 kips (1,077 kN). The ultimate load test of this specimen indicated the transverse straps provide a significant portion of the transverse restraint since there was instant splitting on the specimen when the weld fractured.

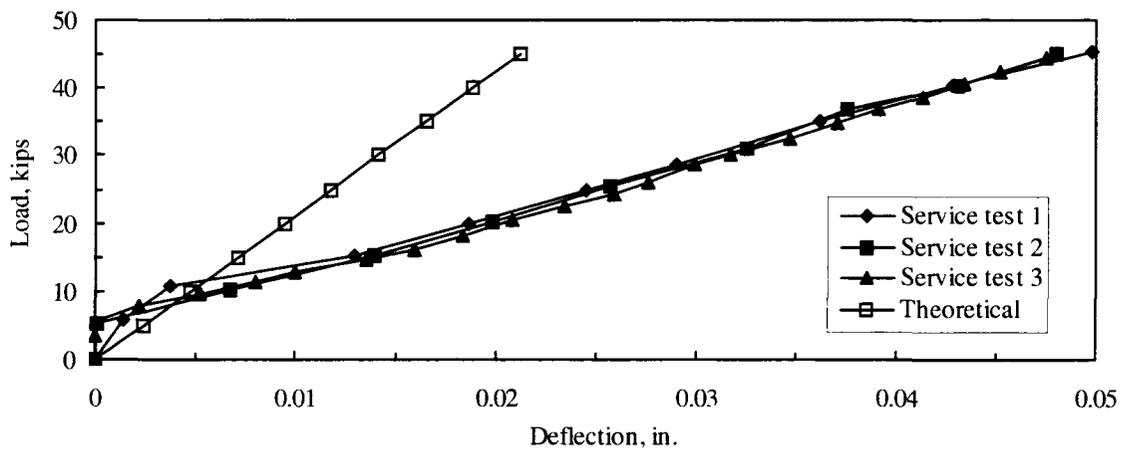
For the LP2 loading, the center of the load pad was only 2 ft (610 mm) from the support; therefore, flexural yielding in the girders prior to failing the deck was not a concern. However, damage caused by the loading at LP1 was expected to negatively affect the capacity of the deck. This was confirmed by the service level displacements under the loading point which were approximately 2.5 times larger than predicted values as shown in Figure 7b. After completing the service level loading, the specimen failed in a punching shear mode at 171 kips (761 kN) (Klaiber et al. 2000).

### **Specimen 2 and 3**

Due to the similarities between the two specimens, minus the formwork system and



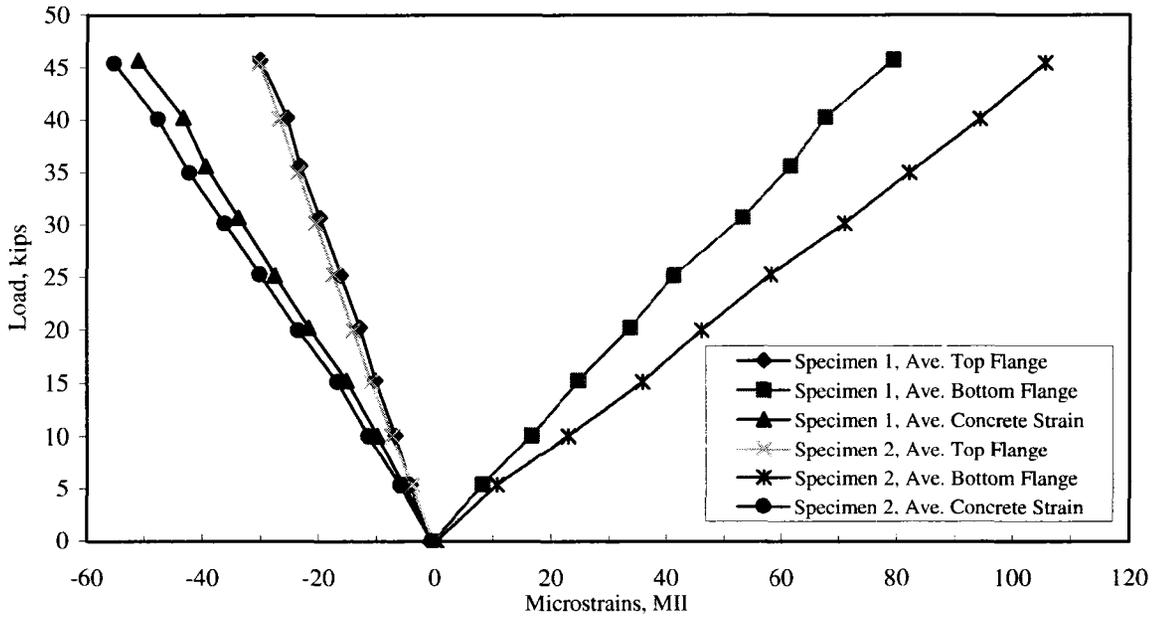
a. Service level midspan strains resulting from loading at LP1.



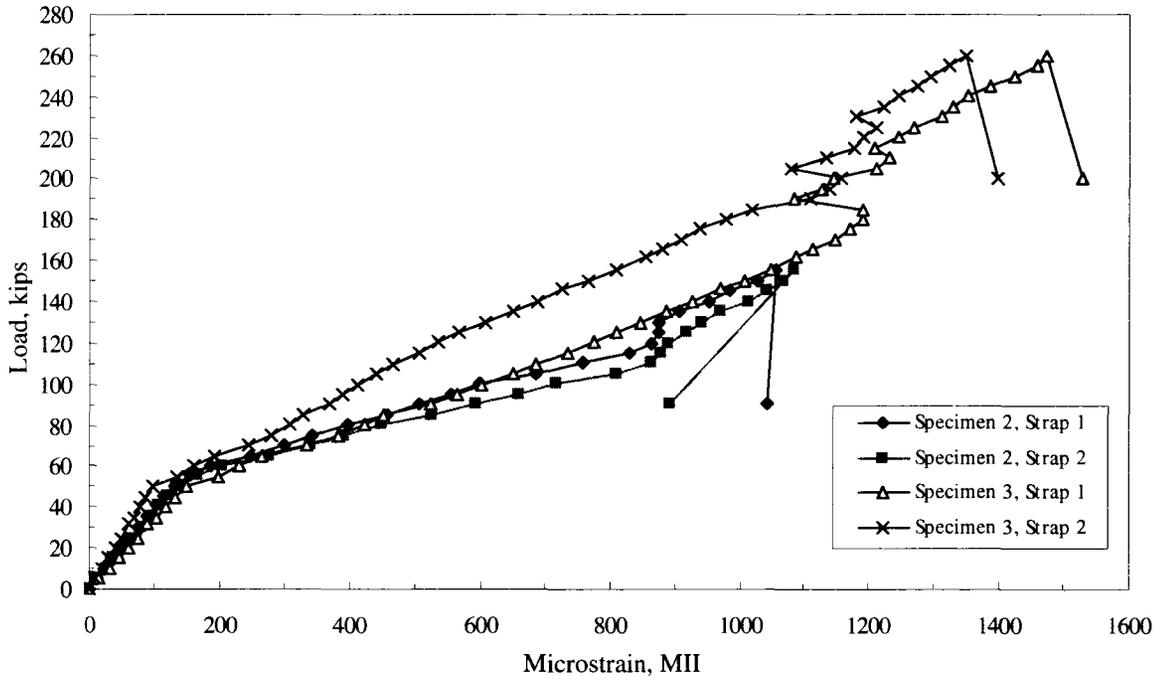
b. Service level deflections directly under loading point LP2.

Figure 7 Service level responses of Specimen 1.

transverse straps, the experimental results from each specimen will be discussed in this section. The average midspan strains for Specimen 2 and 3 due to the service load as presented in Figure 8a indicate elastic behavior. The theoretical gross section neutral axis of 8.7 in. (221 mm) for Specimen 2 and 8.51 in. (216 mm) for Specimen 3 were reflected by experimental values of 8.9 in. (226 mm) and 8.0 in. (203 mm), respectively, indicating the effectiveness of the ASC in developing composite action. Observations of the deck provided no indication of distress at the service load level.



a. Average midspan service level strains for Specimen 2 and 3



b. Transverse strap strains for Specimens 2 and 3 due to the ultimate loading

Figure 8 Service and ultimate load results for Specimens 2 and 3.

The development of internal arching action was evaluated by failing the specimens and observing the mode of failure. Specimen 2 experienced an immediate splitting failure at a load of 155 kips (690 kN) when the bolted connections connecting the transverse straps experienced net section fracture due to an axial tensile force of 15.7 kip (70 kN). The tensile force was calculated by multiplying the measured strains due to the ultimate loading for Specimen 2 (as presented in Figure 8b) by Young's Modulus = 29,000 ksi (200,000 MPa) and the cross sectional area of the strap. The strap failure indicated the need for larger straps in Specimen 3, which resulted in a punching shear failure at 260 kips (1,157 kN). However, the straps in Specimen 3 still underwent strain hardening at the bolted connections indicating failure was imminent. Based on the recorded strains, the Specimen 3 straps were carrying approximately 3 times more axial force than those in Specimen 2, indicating the necessity of lateral confinement.

When comparing the strap strains presented in Figure 8b, the behavior of the two specimens is almost identical to approximately 55 kips (223 kN) at which time a considerable increase in the strap response occurs. Testing log records indicate that concrete cracking was audible in both specimens at this load level followed by the development of a visually detectible longitudinal splitting crack at the centerline of each specimen. Based on these observations, the mode of resistance of the arched specimen changed from one of flexural combined with arching to a tied arch configuration relying on the transverse straps to maintain structural integrity. This increased reliance on the transverse straps is reflected by the increased strain response as shown in Figure 8b. The discontinuity in the strap strains in Specimen 3 at approximately 200 kips (890 kN) is due to slippage that occurred in the bolted connections. In both cases, the longitudinal flexural strains in the girders remained well below yield as expected.

Specimen 1, 2 and 3 resisted, at a minimum, 3.4 times the factored HS-20 truck wheel load prior to the onset of deck failure, indicating that the arched deck/ASC combination possesses sufficient strength for LVR applications. The ultimate capacity and mode of failure for the specimens

was governed by the amount of lateral confinement provided by the transverse straps since the ASC reinforcement was similar in all cases.

### **Model Bridge**

#### *Transverse Straps In Place*

Service loads of 45 kips (200 kN) were applied at the 15 locations indicated in Figure 5 to determine the lateral load distribution characteristics and the confinement provided by the transverse straps. The strain present in the straps was evaluated for each load position and converted to an axial tensile force with a maximum value of 6.5 kips (29 kN) occurring in the east strap of Bay 2 when load was applied to S10. However, for a majority of the load positions, an axial tension force of less than 2.5 kips (11 kN) occurred. Due to the low confining forces, the straps were released and the service load series was repeated.

#### *Transverse Straps Removed*

With the straps removed, the measured girder strains and deflections increased for all 15 load positions. However, the maximum increase that occurred in the midspan deflections was less than 15%; the overall behavior of the specimen remained essentially unchanged. The largest difference in the midspan deflection between the two restraint conditions occurred when the load was applied at S1; the midspan deflection in Girder 1 increased from 0.133 in. (3.4 mm) to 0.149 in. (3.8 mm) a 0.016 in. (0.4 mm) increase. To illustrate the similar behavior, the midspan deflection profiles for the S1 service load position with the transverse straps in place and then removed are presented in Figure 9. While the deflection increases for all points, the profile of the bridge maintains its original shape indicating that the transverse straps are not critical to the performance of the model bridge under service loads.

With the transverse straps removed, an increased strain was expected in the three midspan diaphragms and three end bracing members due to the altered load path. The maximum recorded tension strain (294 microstrain, a 15% increase from the original condition) for a diaphragm member

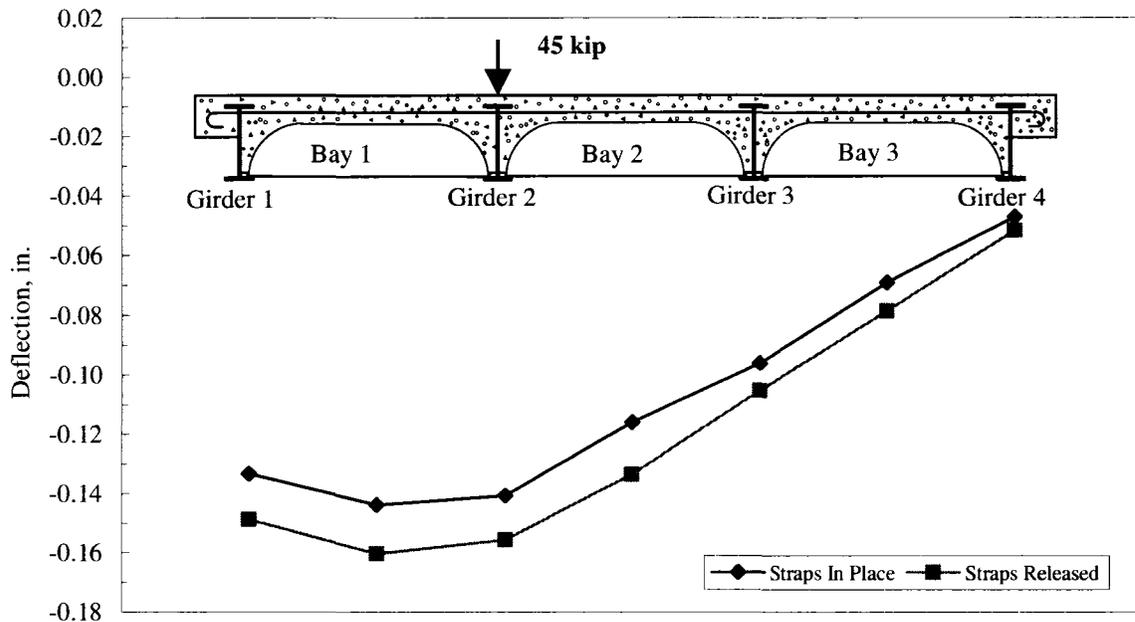
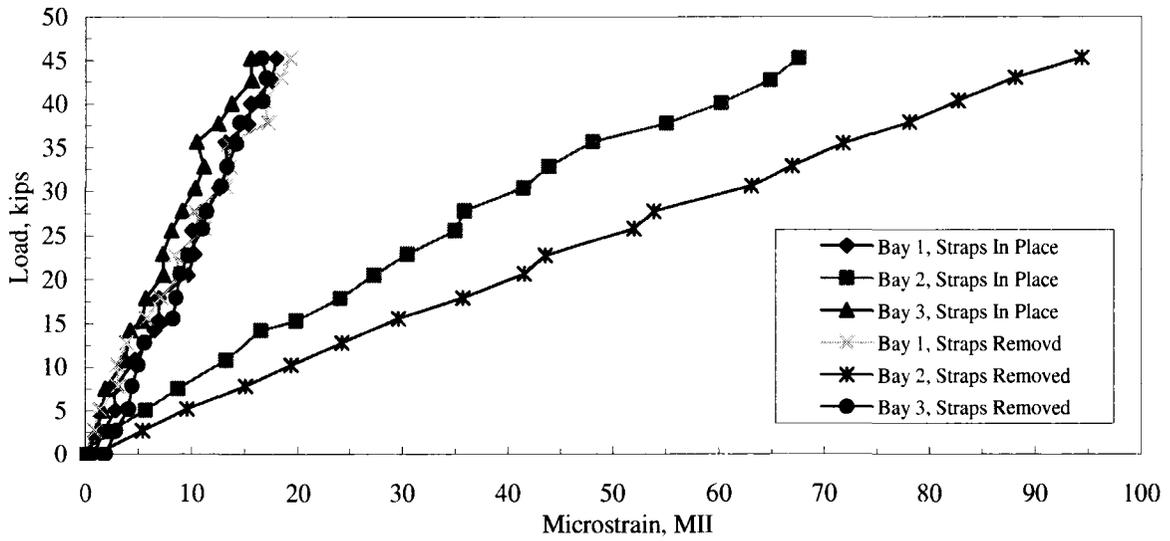


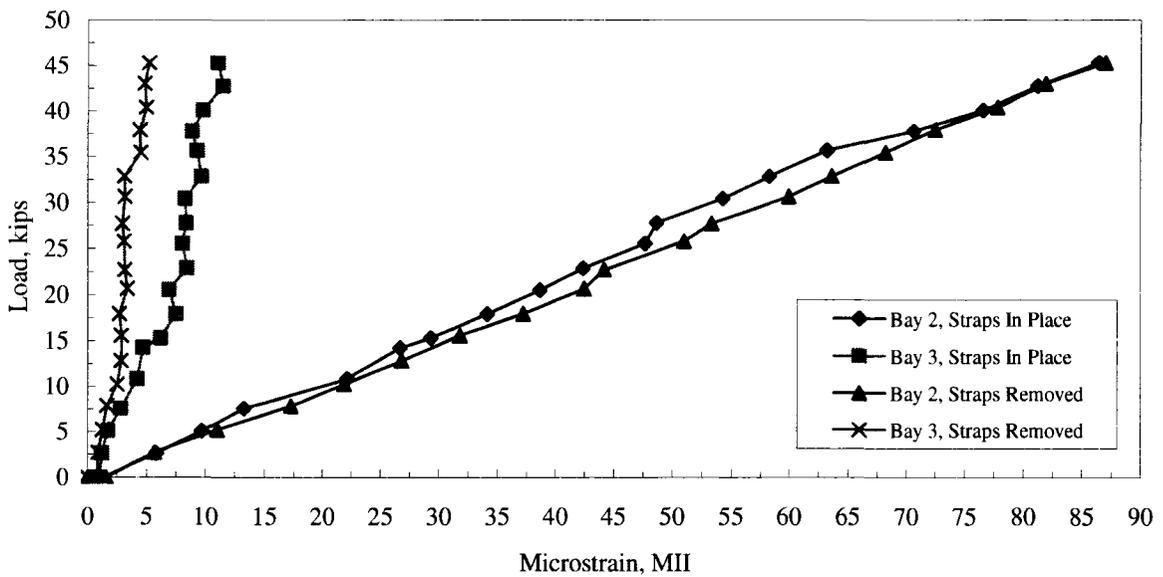
Figure 9 Comparison of midspan deflections for load at S1, straps in place and removed.

with the transverse straps removed occurred in Bay 2 during the S2 loading. The largest percentage increase in the tensile strain was developed in the Bay 2 diaphragm member for the S10 load position as shown in Figure 10a. While a 40% increase in strain occurs between the two conditions, the maximum values of 95 MII are small (less than 10% for the S10 case) indicating that the diaphragm members provide a minimal amount of lateral confinement. The diaphragm response indicated is representative of the 15 service load positions in that the strain increase is limited to the bay in which the load is applied and the effects of removing the straps diminishes in adjacent bays.

The behavior of the end bracing members was essentially unchanged when the measured strains are compared for the two lateral confinement conditions. As a representative sample, Bay 2 and 3, (Bay 1 was not instrumented) tension flange strains for the end bracing members due to loading at S10 are presented in Figure 10b. The resulting plot indicates that the lateral confinement provided by the end bracing members is not substantially influenced by the transverse straps. Likewise, the effects of the loading are greatly reduced in the bays adjacent to that which is being



a. Diaphragm strains



b. End Bracing strains

Figure 10 Response of diaphragms and end bracing members with a load at S10 with and without transverse straps.

loaded. The limited difference in response when the transverse straps were removed indicated the model bridge did not rely on a tied arch configuration for lateral confinement. By comparing the

response of the diaphragms and end bracing with the straps in place and then removed, one observes that the arched deck relies on the adjacent composite girders and the arched configuration to provide a majority of the lateral confinement. This conclusion is based on the limited increase in the midspan deflection and strains present in the transverse members.

Since the behavior of the model bridge did not change significantly, the service level performance of the structure was evaluated based on the data gathered with the transverse straps removed. The maximum flexural effects measured in the girders occurred at the midspan of Girder 1 with the load applied at S4. For the 45 kip (200 kN) service load, a deflection of 0.29 in. (7.4 mm), corresponding to an  $L/1,380$  deflection ratio and a longitudinal live load tensile stress of 11 ksi (76 MPa) was calculated based on the measured longitudinal flexural strains. The largest deflections and longitudinal strains were expected to occur in Girder 1 since it is an edge girder.

#### *Lateral Load Distribution*

Lateral load distribution factors were determined by calculating the percentage of the total moment carried by each effective section using the longitudinal tensile strains recorded at midspan for the 45 kip (200 kN) service load. The cross section of the model bridge was similar to a concrete slab deck/steel girder bridge, thus the experimental distribution factors were compared to AASHTO LRFD Bridge Specification design values for a single lane loading (1994). Since the MBISB system is intended for use on LVRs, the applicable multi-presence and Average Daily Truck Traffic (ADTT) factors were applied to both the experimental and design values for a consistent comparison. The experimental distribution factors are reflective of a single point load as opposed to a full truck(s) occupying the bridge; therefore, representing an extreme localized loading effect.

The maximum experimental lateral load distribution factor for live load moment applied to an interior girder was 45%, which is the same as the AASHTO LRFD design value. The experimental distribution factor for the exterior girder was slightly larger (56% vs. 54%) than the specified design values. However, due to the severity of the single point load placed directly over the exterior girder

(Girder 1), a conservative experimental result was not expected. Based on the results obtained, AASHTO LRFD Bridge Specifications for slab/girder bridges would be applicable to determine the lateral load distribution factors for the design of similar MBISBs.

Flexural cracks were visible in the concrete below the composite neutral axis under the 45 kip (200 kN) service loads. Such cracking of the concrete was expected due to the span length and the applied service load; the cracking however did not compromise the structural integrity of the deck. The experimentally determined neutral axis, approximately 7 in. (178 mm) below the deck surface, reflected the cracked condition and indicated composite action was developed by the ASC.

#### *Ultimate Flexural Test*

Similar to Specimen 1, the model bridge was subjected to an ultimate loading to determine its strength and behavior under these conditions. With the transverse straps removed, load was applied at the four points indicated in Figure 5 and Figure 6, creating a constant moment region about the midspan. The tension flange of Girder 3 began to yield at a total load of 164 kips (730 kN) followed by yielding in Girders 4 and 2 as the load was increased to 302 kips (1,344 kN) before the test was terminated due to equipment limitations. The midspan profile at the maximum load is presented in Figure 11 with Girder 3 undergoing a deflection of 4.0 in. (102 mm). Due to the eccentric positioning of the load about the centerline of the model, the deck experienced a torsional effect, causing extensive cracking in all three bays, in both overhangs and in the west backwall. In spite of the distressed deck and displaced profile, there was no indication of deck failure at the maximum value of 75.5 kips (336 kN) per load point, 4.7 times greater than an AASHTO HS-20 design truck wheel load (1994).

#### *Ultimate Punching Tests*

Five additional tests on the model bridge were completed to evaluate the reserve capacity (i.e. punching shear strength) of the transverse arch. The transverse straps were not replaced in these tests;

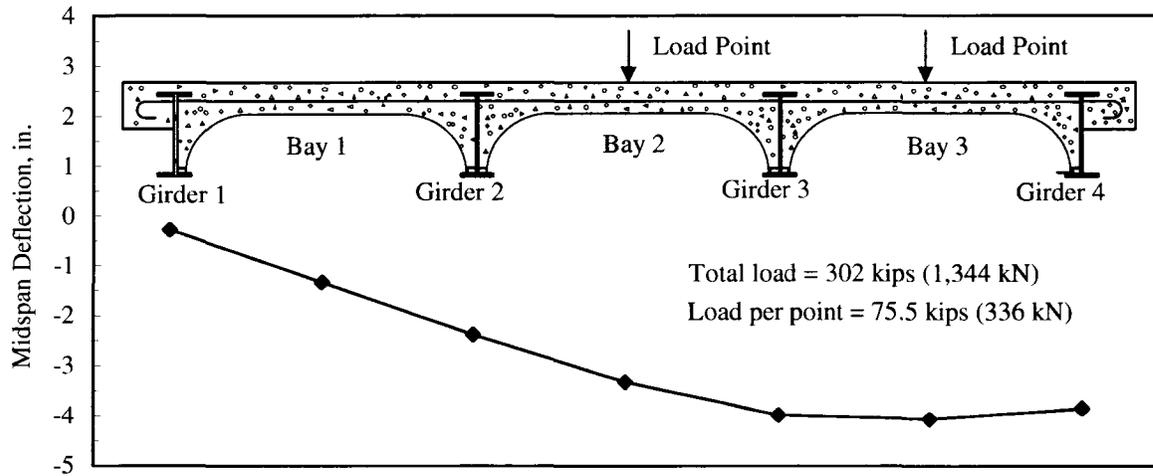


Figure 11 Midspan profile of Specimen 3 when ultimate flexural load was applied.

load was applied through the previously described 12 in. x 12 in. (305 mm x 305 mm) pads. In the first test, two loads were placed in Bay 3 at the U1/P1 position (see Figure 5); a punching failure occurred at 117 kips (521 kN). The lower individual value was attributed to the effect of the second load of equal value 6 ft (1830 mm) from the point of failure. A second and third test with a single point load were performed in Bays 1 and 2 (load at P2 and P3, respectively) causing punching shear failures at 158 kips (703 kN) and 150 kips (668 kN), respectively. The mode of failure and failure loads values indicate the development of in plane arching action even with the transverse straps removed.

To obtain information on the behavior of the arch near a free edge, Tests 4 and 5 were undertaken with the load pad centered two feet from the east end of the deck as shown in Figure 5 (i.e. positions P4 and P5). The deck failed in punching shear at 142 kips (632 kN) and 125 kips (558 kN), respectively in these two tests. The reduction in capacity in the fifth test was attributed to a loss of stiffness and confinement resulting from the damage occurred in the fourth test.

The results from the punching tests indicate arching action is the mode of structural resistance due to the deck failing in punching shear at a minimum of 2.6 times the factored HS- 20 truck wheel load. Thus, the lateral confinement of the arched deck by the combination of the ASC reinforcement, the diaphragm and end bracing members and the out of plane bending stiffness of the adjacent composite girders is confirmed. This behavior was observed with and without the transverse straps in place which had been crucial in maintaining the structural integrity of the single bay specimens. The results from the model bridge indicate that the transverse straps are not necessary to resist the applied wheel loads, eliminating the need to weld on the fracture critical tension flanges of the longitudinal girders.

The punching shear tests also verified the validity of reducing the deck reinforcement to that which is required to complete the ASC and stiffen the ends of the bridge. The additional reinforcement for the purpose of crack control provided no discernable improvement in the deck behavior. The poured in place backwall proved to be simple to construct and structurally adequate, thus was adopted for use in future designs.

## **CONCLUSIONS**

Based on the results from the three single bay specimens and the model bridge, the combined modifications met the goals of reducing the self weight of the section and developing composite action. The compiled results verify the fact that the modifications are more than adequate to resist a factored HS-20 wheel load and can be used in LVR bridges (i.e. the MBISB). This global conclusion was supported by the following conclusions determined from the laboratory testing.

### **Single Bay Specimens**

- Wheel loads are resisted through a tied arch action.
- The development of arching action is dependent upon the resistance provided by the transverse straps.

- With adequate transverse straps, a punching shear failure results at a load several times that of the factored HS-20 wheel load.
- The transverse arch is best formed by implementing the custom rolled formwork.
- The ASC develops composite action and provides longitudinal confinement of the deck.
- Single bay specimens behave significantly different than the modifications in a multiple bay bridge configuration.

### **Model Bridge**

- Applied wheel loads are resisted by arching action.
- The transverse straps provided a minimal amount of resistance and thus were removed.
- Lateral confinement of the arched deck was provided by:
  - Adjacent composite girders.
  - Diaphragms and abutment members.
  - ASC reinforcement.
- Lateral load distribution factors are comparable to AASHTO LRFD Bridge Specification design values.
- The punching shear capacity of the deck was several times that of the factored HS-20 wheel load even with the transverse straps removed.
- The only deck reinforcement required is that in the ASC and temperature and shrinkage reinforcement.
- The ASC developed composite action.

By including the modifications, the applicability of the original design, (i.e. BISB), can be increased, allowing for longer spans and the use of more efficient girder sections and spacing. Thus, the MBISB system provides LVR managers with a lower cost, in-house constructed alternative design which can span up to 80 ft (24.38 m).

## ACKNOWLEDGEMENTS

The investigation presented in this paper was conducted by the Bridge Engineering Center at Iowa State University through funding provided by the Iowa Department of Transportation, Highway Division and the Iowa Highway Research Board. The authors wish to thank the County Engineers in Tama County, Iowa and Doug Wood, ISU Research Laboratory Manager, for their help with the construction, instrumentation, and testing of the specimens.

The opinions, findings, and conclusions expressed in this publication are those of the authors and not necessarily those of the Iowa Department of Transportation.

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#### 4. IN-FIELD PERFORMANCE OF THE MODIFIED BEAM-IN-SLAB BRIDGE A LOW VOLUME BRIDGE REPLACEMENT OPTION

A paper submitted and accepted for presentation at the Sixth International Bridge Engineering  
Conference

Sponsored by:  
Transportation Research Board

Travis F. Konda<sup>1,2</sup>, Terry J. Wipf<sup>1,3</sup>, F. Wayne Klaiber<sup>1,3</sup>

#### ABSTRACT

Managers of most low volume roads (LVRs) face a deteriorating bridge population where frequently replacement is the most cost effective solution. With more structures in need of replacement than available funds to do so, low cost alternatives that are constructible by in-house forces are a desirable option. The Modified Beam-in-Slab Bridge (MBISB) is one such alternative that has been developed specifically for LVRs.

The MBISB system consists of longitudinal steel stringers that support a transverse concrete arched deck. Composite action is obtained by using an Alternative Shear Connector (ASC), rather than shear studs. Other than nominal transverse reinforcement, which is part of the ASC, the MBISB requires minimal additional reinforcement - approximately a 70% reduction of that required in conventional decks. Two demonstration bridges, MBISB 1 (L = 50 ft (15.24 m), W = 31 ft (9.45 m)) and MBISB 2 (L = 70 ft (21.34 m) W = 32 ft (9.75 m)), saved the bridge owner slightly more than 20% of the cost of conventional bridge systems.

Field testing of both demonstration bridges determined the behavior of the bridges under service loading. Strains and deflections were measured at critical locations; the resulting data were used to confirm composite action and to determine the load distribution characteristics for use in the

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<sup>1</sup>Graduate Student, Distinguished Professor, and Professor respectively, Department of Civil, Construction and Environmental Engineering, Iowa State University

<sup>2</sup>Author for correspondence, primary researcher and author

<sup>3</sup>Researcher and author

design methodology that was developed. Based on analyses and the field data, the demonstration MBISBs exceed AASHTO design requirements; supporting documentation will be presented describing the construction and structural behavior of the MBISBs.

## **INTRODUCTION**

The condition of the bridge population in the United States is in need of improvement since approximately 27% of all bridges are either structurally deficient, functionally obsolete or both. The State of Iowa follows the national trend in that approximately 28% of all the bridges located in the state are structurally/functionally deficient; the deficiency rating is higher (31%) when only bridges on Iowa's Low Volume Roads (LVRs) are considered (1, 2). Maintenance of LVRs and the structures on them are the responsibility of the local government (i.e. Iowa County Engineers).

These engineers are responsible for the LVR bridge population; however, in most counties, they are constrained by limited resources. One method to extend available funds is to use alternative replacement bridges that can be constructed with county forces. These alternatives must cost less than traditional systems and be relatively easy to construct while still meeting applicable design criteria. Such alternatives are specifically for use on LVRs; those with an Average Daily Traffic (ADT) count of less than 400 vehicles (3). Since many of the 99 Iowa counties have in-house forces dedicated to maintaining off system structures, these same forces can be used to construct the alternative bridges.

### **Successful Alternatives**

The Beam-in-Slab Bridge (BISB) design is an alternative bridge replacement that has been used extensively in Iowa with 80 + structures in service (Lyle Brehm, Former Benton County, IA Assistant County Engineer, unpublished data). Developed in Benton County, Iowa over 25 years ago, the BISB is remarkably simple, consisting of longitudinal W12 sections spaced on 2 ft (610 mm) centers with

an unreinforced concrete 'deck' placed between the girders. The plastic concrete, which is supported by plywood formwork that remains in place, is struck off even with the top flange of the beams. A typical BISB cross section is presented in Figure 1 (a). The structural behavior of the BISB design was evaluated by the Iowa State University (ISU) Bridge Engineering Center (BEC); through laboratory field tests the BISB was determined to be a conservative design (4). While being simple to design and construct, the BISB is limited to spans of 50 ft (15.24 m) or less due to its self weight stresses and deflections. Since there is no composite action in the BISB system, the efficiency of the system is reduced.

### **MODIFIED BEAM-IN-SLAB BRIDGE DESIGN**

The Modified Beam-in-Slab Bridge (MBISB) design was proposed to increase the applicable span lengths and structural efficiency while maintaining the simplistic nature of the original BISB.

Modifications included developing composite action between the concrete deck and steel girders plus reducing the self weight of the system.

#### **Developing Composite Action**

Rather than using traditional shear studs to develop composite action, an alternative shear connector (ASC), was developed by ISU researchers. This system can be fabricated by county forces with minimal equipment. The ASC design consists of 1 1/4 in. (32 mm) diameter holes on 3 in. (76 mm) centers that are either torched or cored through the web of the longitudinal girder one hole diameter below the bottom of the top flange. Composite action results when plastic concrete flows through the holes and cures, forming a concrete shear dowel which mechanically connects the steel girders and the concrete deck. Reinforcement (#4 or #5 Grade 60 reinforcing steel) is placed through every fifth (15 in. (380 mm) spacing) hole to confine the concrete shear dowels (4, 5). A typical layout of the ASCs is presented in Figure 1 (b).

An additional increase in flexural rigidity is obtained by placing concrete above the top flanges of the girders. Encasing the top flange of the girders within the deck reduces the corrosion potential. However, the inclusion of the 3 in. minimum cover requires a significant change in the construction techniques since the concrete can no longer be stuck off using the top flanges of the girders as a guide.

### **Reducing Self Weight**

Recognizing that the unreinforced concrete below the neutral axis (tension side) does not contribute to the strength of the structure, removing such concrete has a minimal impact on the structure's flexural resistance. By forming a transverse arch between the longitudinal girders, a majority of the ineffective concrete can be removed. The implementation of the transverse arch reduced the self weight of the system which made it possible to use the system in longer spans. The transverse arch also allowed for the use of deeper girders set at a larger spacing ( $> 2$  ft (610 mm)), further improving the efficiency of the system.

The implementation of the transverse arch changed the mode of structural resistance within the deck from flexure to arching. Such behavior was predicted based on the findings of Canadian researchers who investigated bridge deck slabs that were both longitudinally and transversely restrained, resulting in an in-plane arching action which resisted the wheel loadings (6). Punching shear became the failure mode for the restrained decks, eliminating the need for traditional deck reinforcement for flexural resistance (7). Similar behavior was expected in the MBISB system due to the transverse arches, which lead to a considerable reduction in the amount of deck reinforcement required.

### **Formwork Systems**

To maintain simple construction methods, numerous systems to create the transverse arch were investigated (8). Two possible designs resulted, a stay-in-place system and a removable/reusable system.

### *Stay-in-Place Formwork*

The stay-in-place formwork system consists of circular sections cut from Corrugated Metal Pipe (CMP) that rest on the bottom flanges of the longitudinal girders similar to the plywood 'floor' of the BISB system. The system is extremely simple, but has a limited range of use due to the limited sizes of CMP available and compatible girder heights; it also obviously does not permit visual inspection of the underside of the deck.

### *Removable/Reusable Formwork*

A removable/reusable custom rolled arch formwork system, constructed from corrugated metal, was developed. This system removes a larger amount of ineffective concrete and is adaptable to a wider range of girder depths and spacing in comparison to the stay-in-place CMP formwork. The custom rolled formwork rests on the bottom flanges of the longitudinal girders and readily accommodates the ASC. Sketches of the stay-in-place and the reusable custom rolled arched formwork are shown in Figures 1 (c), (d), respectively.

### **Laboratory Verification**

Three single bay specimens using both modifications were constructed to investigate the strength and failure mode of the proposed MBISB design, a typical single bay specimen is presented in Figure 2 (a). Only the reinforcement in the ASC was used; all specimens were found to have more than adequate resistance to a simulated wheel loading (5, 8, 9).

While providing information on the strength and mode of failure, the single bay specimens obviously do not adequately represent the behavior of a multiple transverse arch cross section that would be used in a bridge. Therefore, a three bay model bridge ( $L = 30.5$  ft (9.3 m),  $W = 20$  ft (6.1 m)) was designed, constructed and tested in the laboratory; a cross section of the model is presented in Figure 2 (b). A series of 45 kip (200 kN) point loads were applied at predetermined locations to quantify the lateral load distribution properties in the model bridge. The results indicated that the load distribution in the model was similar to values for slab/girder bridges given in the

American Association of State Highway Transportation Officials (AASHTO) Load Resistance Factored Design (LRFD) Bridge Specification (9, 10).

In the ultimate flexural load test of the model bridge shown in Figure 2 (c), a maximum load of 302 kips (1,344 kN) was applied which caused yielding of the steel girders and a deflection of 4 in. (102 mm). The bridge deck, although heavily cracked, remained intact facilitating five additional punching shear tests; failure loads in these tests ranged from 117 kips (521 kN) to 158 kips (703 kN) indicating significant capacity even after being heavily damaged in the previous test.

In both the single bay specimens and the model bridge, the ASC provided full composite action with no evidence of slip or failure. A more detailed description of the laboratory testing phase and the data collected is presented in Iowa Department of Transportation (DOT) Final Report TR-467, Volume 1 (9).

### **Demonstration Bridges**

Laboratory results indicated the two modifications improved the efficiency and thus the applicability of the BISB design. Two demonstration bridges implementing the two modifications previously described were constructed to further verify that the MBISB was an acceptable replacement bridge design. As previously noted, to be a desirable alternative, the resulting demonstration structures had to cost less than traditional systems, be readily constructible by in-house forces and had to satisfy strength and serviceability requirements. Field testing of the resulting structures provided an opportunity to obtain experimental data to document their behavior, load distribution characteristics, etc.

#### ***MBISB 1***

The first demonstration bridge, MBISB 1, was constructed in Tama County, Iowa by in-house forces on a rural LVR. The design of the 50 ft (15.24 m) long, 31 ft (9.45 m) wide structure closely followed the original BISB design and consisted of 16-W12x79 Grade 50 steel sections on 2 ft

(610 mm) centers. The modifications reduced the self weight of the structure by approximately 20% and increased the flexural rigidity of the structure by approximately 22% over that of the original BISB design.

**Construction of MBISB 1** Construction of MBISB 1 was similar to the procedures used in constructing the original BISB except the two modifications were included. The holes for the ASCs were fabricated in the county yard by torching holes through the webs of the girders. Cross slope in the deck was introduced by placing steel plates of sequential thickness on the abutment caps (i.e. introducing the necessary elevation difference) at the girder support points. The prepared W sections were restrained with steel straps at the fifth points to maintain the 2 ft (610 mm) girder spacing during the concrete placement. The transverse arch was created with stay-in-place formwork cut from 16 gage, 24 in. (610 mm) diameter CMP; the dimensions of the formwork sections for MBISB 1 are presented in Figure 1 (c).

After the #4 Grade 60 ASC reinforcement was installed, the deck concrete was placed and struck off even with the top flanges of the girders. Care was taken during the concrete placement to ensure thorough consolidation thus forming the desired shear dowels. After adequate curing, guardrails were added and the bridge was opened to traffic. Note the only reinforcement in MBISB 1 is the transverse #4 bars in the ASC.

**Cost Savings** MBISB 1 cost approximately \$50 psf ( $\$775/\text{m}^2$ ) to construct which included the cost of both the sub and super structure as well as labor. This is a considerable cost savings when compared to traditional designs of similar length which range as high as \$70 psf ( $\$1,085/\text{m}^2$ ) (costs compiled by Tama County Engineer's Office, Tama County, Iowa, unpublished data).

#### ***MBISB 2***

MBISB 2 was designed following AASHTO LRFD Bridge Specifications whose applicability was verified in the testing of the laboratory model bridge (10). Also constructed in Tama County, Iowa, MBISB 2 deviated considerably from the original BISB design in that the girders were 27 in.

(686 mm) deep (nominal) and the spacing was 6 ft (1,830 mm). The desired 70 ft (21.34 m) long, 32 ft (9.75 m) wide structure has the outward appearance of a typical slab/girder bridge supported by 6-W27x129 Grade 50 steel sections. However, through the implementation of the modifications, the longer span was easily constructed by in-house forces. As one example of the increased efficiency, although MBISB 2 was 20 ft (6.1m) longer, it required 9.02 kips (40.1 kN) less structural steel than MBISB 1, however, more concrete was required since there was more deck surface.

**Construction of MBISB 2** Prior to shipping to the bridge site, the girders were cambered and cored (i.e. holes for ASC) in a fabrication shop. The girders were placed and laterally aligned using tension rods attached to the bottom flanges; clips were used to attach the tension rods to the bottom flanges of the girders to avoid welding on a fracture critical member. Two lines of diaphragms were installed to brace the compression flanges. Concrete backwalls were placed at each abutment to confine the girders, support the approach soil and provide a vertical casting surface for the arched formwork.

Formwork for MBISB 2 consisted of interior and exterior systems. The previously discussed custom rolled arched formwork was used as the interior system creating the transverse arch between the longitudinal girders. As shown in Figure 1 (d), the custom rolled interior formwork consisted of two individual components that were bolted together forming 24 in. (610 mm) wide (nominal) sections which were bolted into batteries consisting of 4 or 5 individual sections and placed in the bridge. To fully develop the ASC reinforcement, it was necessary to include a 12 in. (305 mm) x 12 in. (305 mm) deck overhang on the exterior girders. Details on the exterior formwork system used to create this overhang are presented in Iowa DOT Final Report TR-467 (11).

MBISB 2 had 3 in. (76 mm) of concrete cover over the top flanges of the girders which increased the flexural rigidity of the interior girders by 28%. Due to the cover, the deck surface had to be finished with a power screed which rested on the exterior formwork.

**Reinforcement** Deck reinforcement in MBISB 2 was limited to the transverse #5 Grade 60 ASC reinforcing bars on 15 in. (380 mm) centers and temperature and shrinkage reinforcement

(#4 longitudinal and #3 transverse). When compared to a typical concrete bridge deck, the amount of reinforcement required was reduced by approximately 70%.

After the deck concrete was placed and cured, a guardrail was installed and the structure was opened to traffic. During the construction off season, the custom rolled arched formwork was removed and stored for future use.

**Cost Savings** MBISB 2 cost approximately \$52 psf (\$806/m<sup>2</sup>) to construct including the sub and superstructure and all labor; although not as cost effective as MBISB 1, the alternative replacement design goals were met. The presented cost includes the procurement and assembly of the custom rolled formwork sections depreciated over five uses (costs calculated by Tama County Engineer's Office, Tama County, Iowa and ISU BEC, unpublished data).

### **Field Testing**

The demonstration bridges were field tested to quantify the lateral load distribution, service level stresses and the overall bridge stiffness. The field tests will be described in the following sections. The location of the instrumentation and the loading lanes for MBISB 1 and 2 are shown in Figure 3 and Figure 4, respectively. As can be seen, 20 strain gages, 24 strain (BDI) transducers, and 14 deflection gages (Unimeasures) were used on MBISB 1 and 26 strain gages, 18 BDIs and 19 deflection gages were used on MBISB 2 to measure the primary flexural effects.

### *Instrumentation Placement*

The Unimeasures were placed at the quarter, mid and three quarter spans to develop both a transverse and longitudinal deflected profile. BDI transducers were placed near the abutments on both bridges to determine the presence of end restraint. Both BDI transducers and the appropriate type of strain gage (steel and concrete) were placed at the quarter and mid spans to measure longitudinal flexural strains. The strain instrumentation was placed not only on the longitudinal girders but on the deck surface to establish an experimental neutral axis. The guardrails were also instrumented with strain transducers to quantify any additional flexural resistance.

Additional strain instrumentation was used on both structures to quantify transverse effects - the strains in the transverse straps on MBISB1 and in the transverse tension rods and diaphragms in MBISB 2.

#### *Loading Lanes/Test Vehicles*

The bridges were divided into five loading lanes as shown in Figures 3 and 4 to produce maximum effects in both the interior and exterior sections as well as investigating transverse symmetry. Service level static and quasi-static (rolling) loads were applied to the bridge with loaded tandem axle trucks; the axle spacing and weights are presented in Figure 5 (a). For the static tests, both test vehicles were positioned on the structure to create a maximum midspan moment. The rolling tests consisted of the heavier of the test vehicles tracking in the various test lanes at approximately 2 mph (3.2 km/h) while data were continually recorded. The test vehicles for MBISB 1 in the typical static test configuration are shown in Figure 5 (b) while the Lane 3 rolling test for MBISB 2 is shown in Figure 5 (c).

The data collected from both bridges were analyzed to quantify their structural behavior. In the following sections, the results are presented for each bridge: the resulting deflections, strains, and lateral load distribution characteristics due to the service level loadings.

#### **MBISB 1 Field Test Results**

The results from the MBISB 1 field tests are presented and compared to AASHTO design specifications in the following sections.

#### *Deflections*

The maximum midspan live load deflection of 0.73 in. (18.5 mm), attributed to the static loading with both trucks on the bridge, is less than the suggested AASHTO service level live load deflection limitation of  $L/800$ , which for MBISB 1 is equal to 0.74 in. (19 mm) (10). The largest deflection that resulted during a rolling test, 0.56 in. (14 mm), occurred in the exterior girder during the Lane 5 test. The maximum midspan deflections for the Lane 1 and 5 and Lane 2 and 4 loading are shown in Figure 6 (a) and compared to investigate the transverse symmetry about the bridge centerline. As can

be seen in the presented profiles, transverse symmetry is present and the service level loads are laterally distributed to the adjacent girders.

### *Strains*

The maximum midspan flexural strains occurred during the static load case; 154 (tensile) microstrain was measured in the bottom flange of Girder 6 and a 128 (compressive) microstrain was measured in the top flange of Girder 4. When converted to stresses (assuming  $E_s = 29,000$  ksi (200,000 MPa)), Girder 6 experiences a maximum stress (tensile) of 4.5 ksi (31.0 MPa). When combined with the calculated self weight stress in the section, the steel girder experiences a combined tensile stress of 11.5 ksi (79.2 MPa), which is less than a fourth of the 50 ksi (344.5 MPa) yield stress.

The strain measured at the abutments indicated that little, if any end restraint was present which verified the assumption that the structure is simply supported. The strains measured in the tension straps were small and were only influenced when a wheel load passed directly over the instrumented strap.

### *Lateral Load Distribution*

The lateral load distribution in MBISB 1 was calculated using the measured tension strains and the flexural rigidity of the individual composite section; however, the possible contribution of the guardrail was not included. The maximum percentage of the total applied moment resisted by an interior and exterior girder for a single lane loading, 12% and 11%, respectively, resulted from the Lane 1 loading case; all five single lane cases were investigated. Since the values are nearly equal, for design, the lateral load distribution factor can be taken as 12%. The values were not directly compared since the AASHTO LRFD Bridge Specifications does not have information on cross sections that are similar to that used in MBISB 1 (10).

### *Grillage Analysis*

Using ANSYS finite element software, a simply supported grillage model was developed and calibrated by comparing to the experimental data (12). Elemental properties used were based on a

gross elastic section following the grillage modeling methods described in Hambly (13). Point loads representing the test vehicle were applied at the appropriate nodes of the structure.

The resulting midspan deflection values for the initial model were significantly larger than the experimental values, a representative plot for the Lane 1 loading can be seen in Figure 6 (b), indicating that some portion of the overall flexural resistance of MBISB 1 was underestimated in the model. When the strains in the guardrails were evaluated, it was evident that the guardrails resisted some of the applied load; depending upon the positioning of the applied loading, strains in excess of 90 microstrains were observed. The contribution of the guardrail to the exterior flexural rigidity was developed by independently modeling the guardrail system as a truss attached to the exterior beam. When the stiffness of the guardrail is included in the model, the correlation between the analytical deflections and the experimental deflections is improved as shown in Figure 6 (b). Thus it was determined that the guardrails (depending on the loading condition) increase the total longitudinal flexural rigidity of the structure by approximately 15 to 20 percent.

### **MBISB 2 Field Test Results**

Since similar strength and behavior information was desired on MBISB 2, the recorded field data were analyzed following the format established for MBISB 1.

#### *Deflections*

All measured deflections were less than 50% of the suggested service level live load deflection limitation of  $L/800$  which for MBISB 2 is equal to 1.04 in. (26 mm). This includes the two static tests where the test vehicles were first in Lanes 2 and 4 and then Lanes 1 and 5. Maximum deflection in both the interior (Girder 3) and exterior girder (Girder 1) was 0.50 in. (13 mm). The largest deflection that resulted from the rolling tests, 0.48 in. (12 mm), occurred in Girder 1 during the Lane 1 rolling test. The resulting deflections for MBISB 2 are smaller than those measured for MBISB 1 even though the span is 20 ft (6.1 m) longer; the smaller deflections were expected since MBISB 2 had larger girders which greatly increased the flexural rigidity of the structure.

### *Strains*

As expected, the largest midspan flexural strains occurred during the static tests since approximately twice the load was occupying the bridge. Maximum tensile bottom flange strains of 191 microstrain (Girder 1, exterior) and 189 microstrain (Girder 3, interior) were recorded during the static load tests. When converted to stress (assuming  $E_s = 29,000$  ksi, (200,000 MPa)) this is 5.54 ksi (38.2 MPa) and 5.48 ksi (37.8 MPa) respectively. When combined with the calculated stress due to the larger dead load, a maximum flexural stress of 26.2 ksi (180.5 MPa) is present in Girder 3, slightly more than 50% of the girder yield strength.

A typical midspan strain profile for MBISB 2 is presented in Figure 7 (a). When the corresponding tension and compressive strains are plotted versus the depth of the section, the experimental neutral axis can be determined. The location of the neutral axis indicated an effective composite section resulting from the ASC.

Minimal end restraint was present at the abutments in MBISB 2, verifying the assumption that the bridge was simply supported. The forces in the transverse tension rods attached to the bottom flanges of the girders were dependant upon the truck position with the maximum force (calculated from the measured strain) equal to 1 kip (4.45 kN). Similar to the laboratory results, this low force was very small and thus verified that the tension rods can be removed once the deck concrete has cured without changing the behavior of the system (9).

### *Lateral Load Distribution*

Lateral load distribution factors were calculated following the same methods used for MBISB 1; however, both interior and exterior load distribution factors for single and multiple lane loadings were developed for MBISB 2. The experimental distribution fractures are compared to AASHTO LRFD lateral load distribution factors for a steel girder/concrete slab bridge cross section in Table 1; all values are multiplied by applicable multi-presence and Average Daily Truck Traffic (ADTT)

reductions. When compared to the experimental values, the AASHTO distribution factors are conservative and thus were used in the design methodology developed (10).

#### *Grillage Analysis*

A simply supported grillage model was also developed for MBISB 2 following the same format as MBISB 1. The resulting deflections for the initial model also overestimated the midspan deflections when compared to the experimental values, as can be seen in a representative plot shown in Figure 7 (b). Strains measured in the guardrails followed a pattern similar to those in MBISB 1 but the effect of the guardrail on the model only added 7% percent to the flexural rigidity of the exterior members. The midspan deflections of the model, when compared to the experimental deflection values, has a maximum error of 0.1 in. (2.5 mm) and thus was considered to have acceptable design tolerances.

## **CONCLUSIONS**

The MBISB system, developed through laboratory and field testing, is a viable alternative bridge design for use on LVRs. Building from the original BISB system, two modifications, the ASC and the transverse arch, were introduced to develop composite action and reduce the self weight, and thus increase the efficiency of the design and the applicable span lengths. The two modifications proved to be successful when evaluated in the laboratory, first through single bay specimens and then with a model bridge.

Two demonstration bridges, MBISB 1 and MBISB 2 were designed and constructed implementing the two modifications. MBISB 1 more closely followed the original BISB design while MBISB 2 increased previously spanned distances by 20 ft (6.1 m). Constructed by in-house forces, the MBISB system saved the bridge owner approximately 20% when compared with conventional designs.

MBISB 1 and MBISB 2 were instrumented and load tested to determine their structural behavior under service level conditions. Maximum deflections, lateral load distribution and service level stresses were determined to be below critical values. The lateral load distribution factors in MBISB 1 were determined to be 12% for design purposes in both the exterior and interior members. Lateral load distribution factors in MBISB 2 were determined to be close to AASHTO LRFD design values and thus it is recommended that AASHTO LRFD values be used in future designs (10).

Using the data from the field tests and analytical analyses, a design methodology has been developed following AASHTO LRFD Bridge Specifications (10). The design methodology is presented in Iowa DOT Final Report, Volume 2, and is intended for LVR bridge managers to use in assisting with the design of future MBISB structures (11).

#### **ACKNOWLEDGEMENTS**

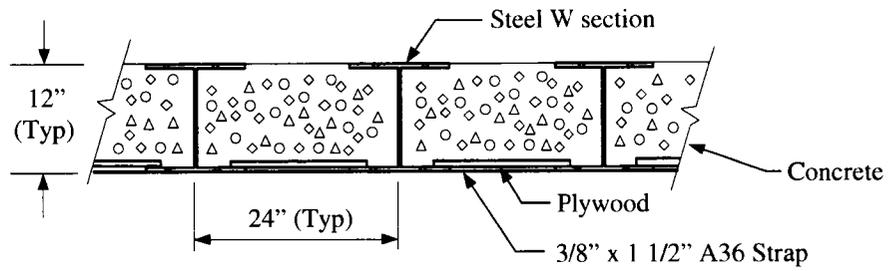
The study presented in this paper was conducted by the Bridge Engineering Center at Iowa State University through funding provided by the Iowa Department of Transportation, Highway Division and the Iowa Highway Research Board. The authors wish to thank the County Engineers and Doug Wood, ISU Research Laboratory Manager, for their help with the construction, instrumentation, and field testing of the demonstration bridges.

The opinions, findings, and conclusions expressed in this publication are those of the authors and not necessarily those of the Iowa Department of Transportation.

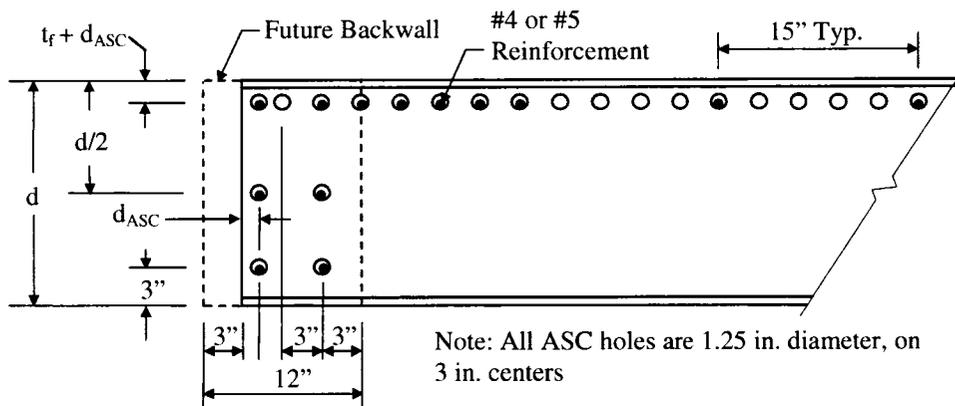
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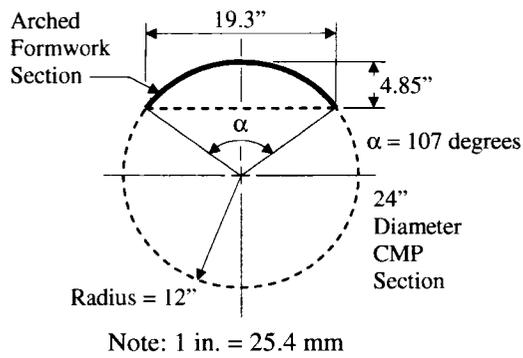
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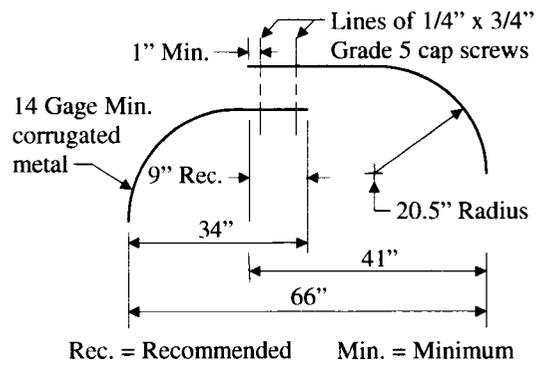
(a) Typical cross section for the original BISB



(b) Typical layout of the ASC and reinforcement

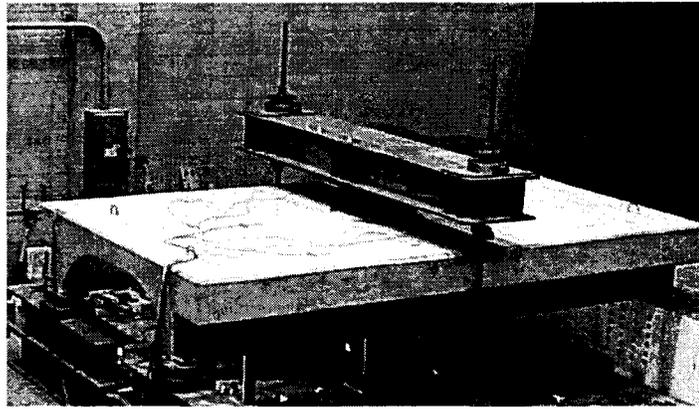


(c) Layout of typical stay-in-place formwork

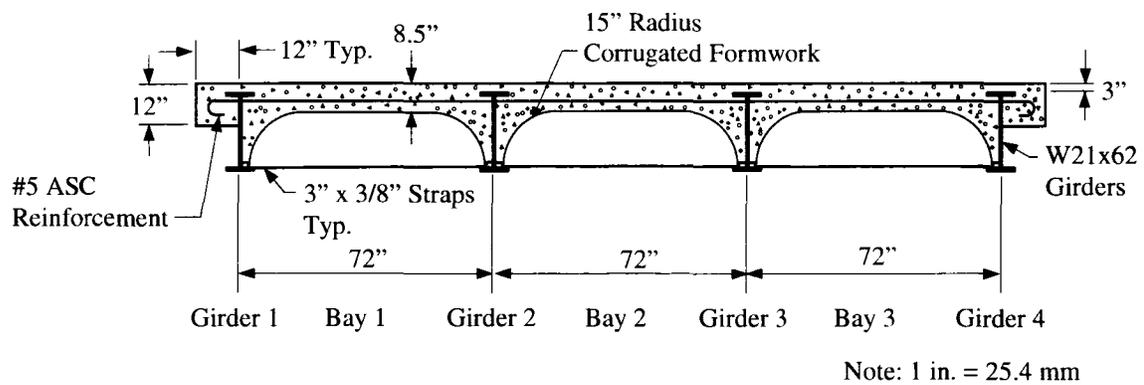


(d) Typical custom rolled arched formwork for W27x129 girders at 6 ft spacing

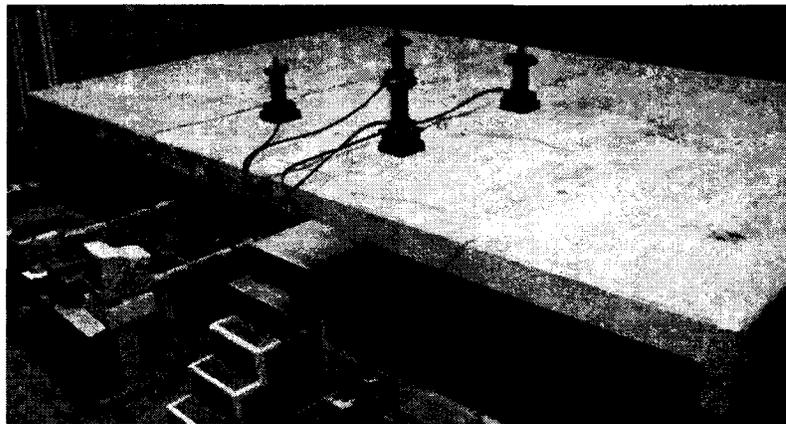
Figure 1 Original BISB and the modifications applied to the MBISB.



(a) Typical single bay specimen



(b) Cross section of the model bridge



(c) Ultimate flexural loading setup on the laboratory model bridge

Figure 2 Laboratory testing and verification of the two modifications.

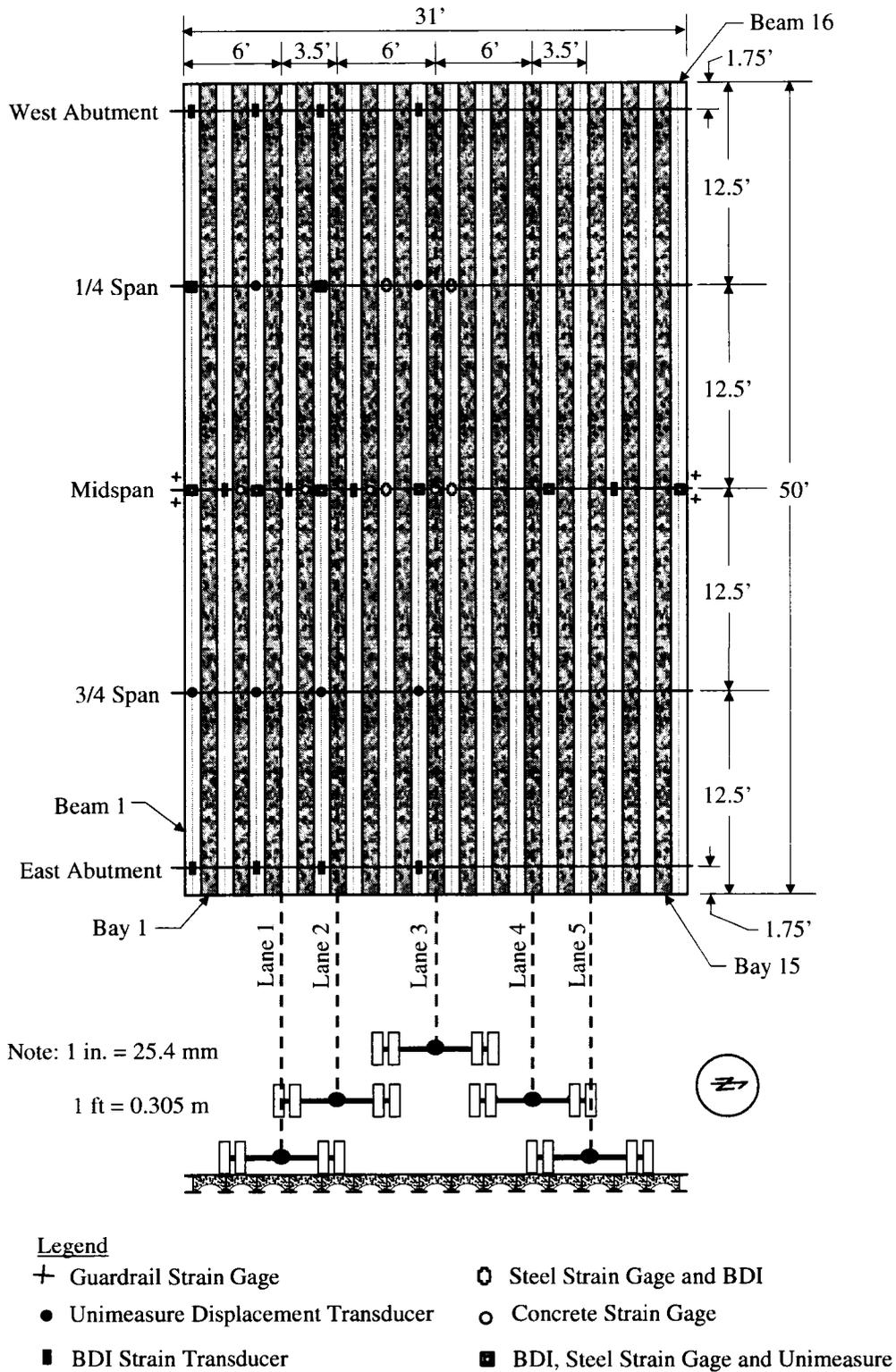


Figure 3 Layout of main flexural instrumentation and test vehicle placement for MBISB 1.

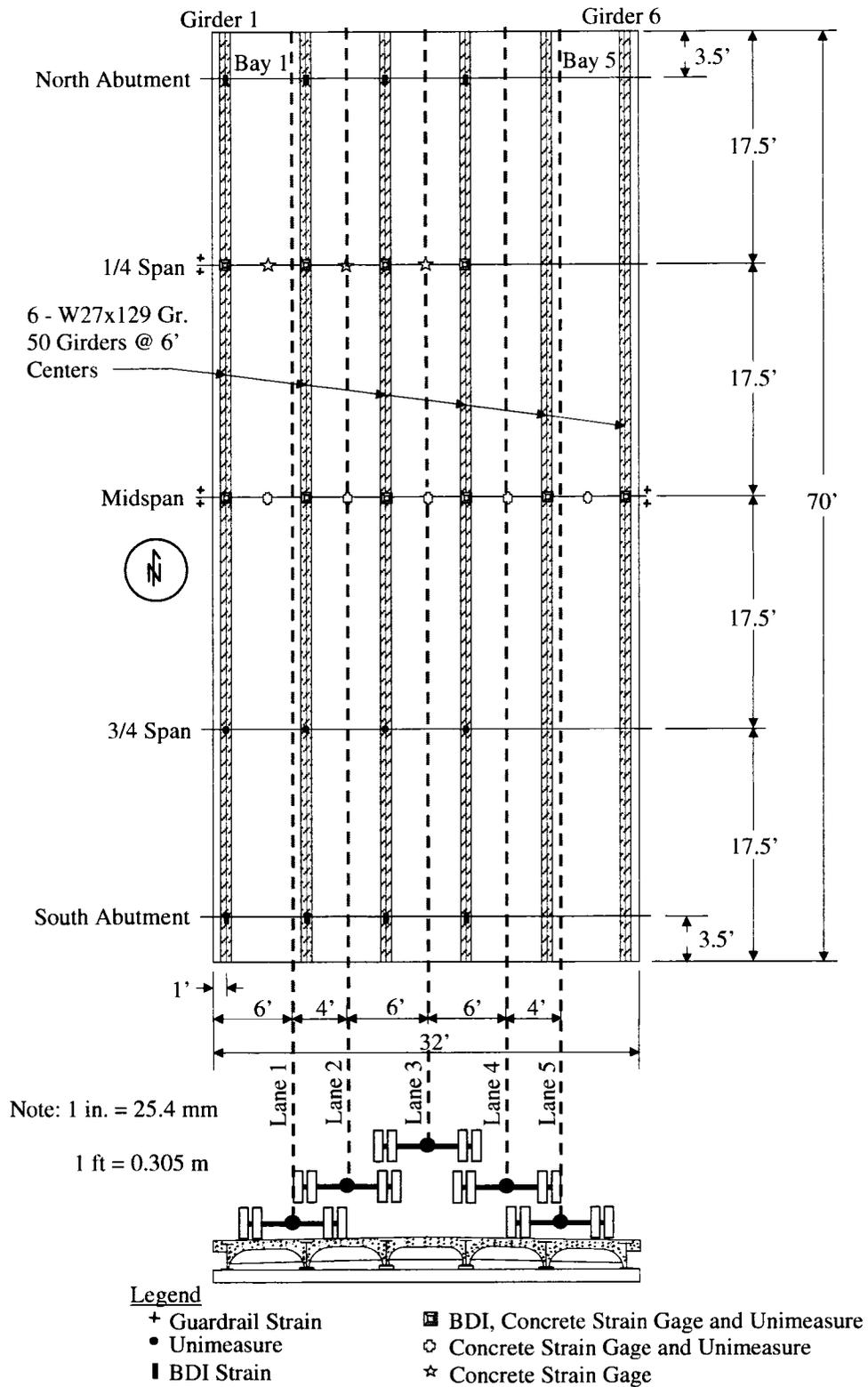
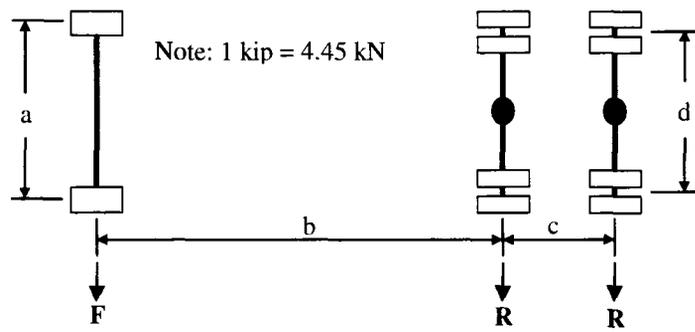


Figure 4 Layout of main flexural instrumentation and test vehicle placement for MBISB 2.



Test Vehicles for MBISB 1							
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
Test Vehicles for MBISB 2							
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

(a) Wheel and load configurations of the field test vehicles

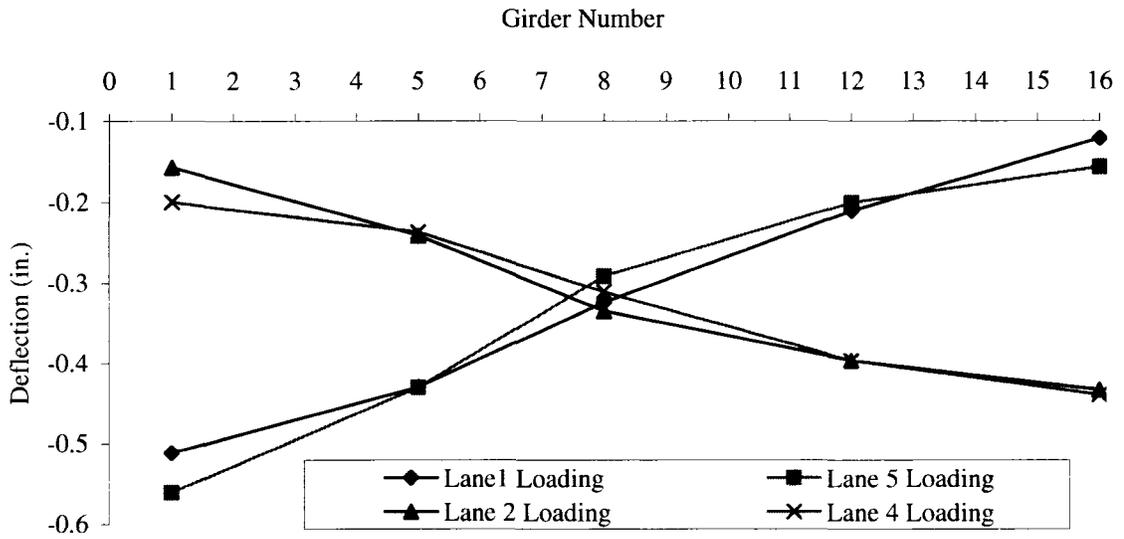


(b) Test vehicles for the static load test occupying MBISB 1

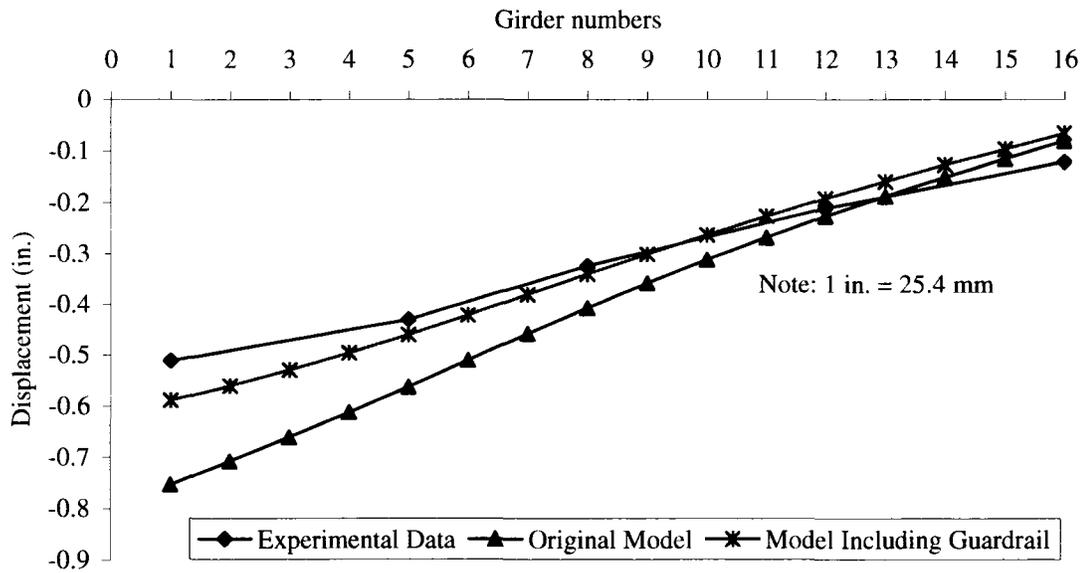


(c) Test vehicle for the Lane 3 rolling test occupying MBISB 2

Figure 5 Vehicles used to test the demonstration bridges.

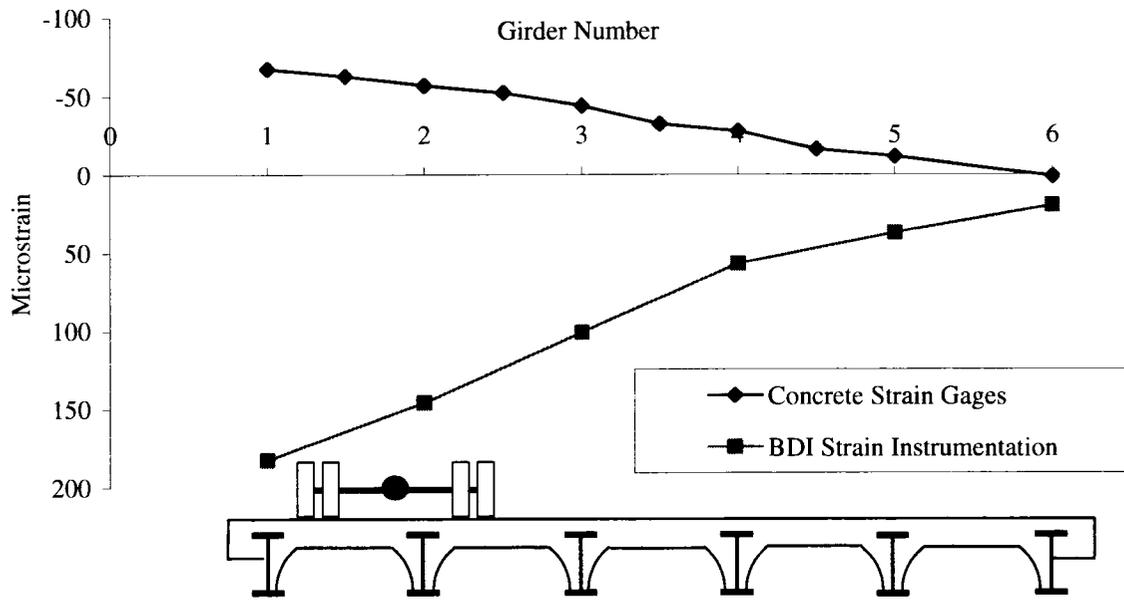


(a) Transverse symmetry based on midspan deflections

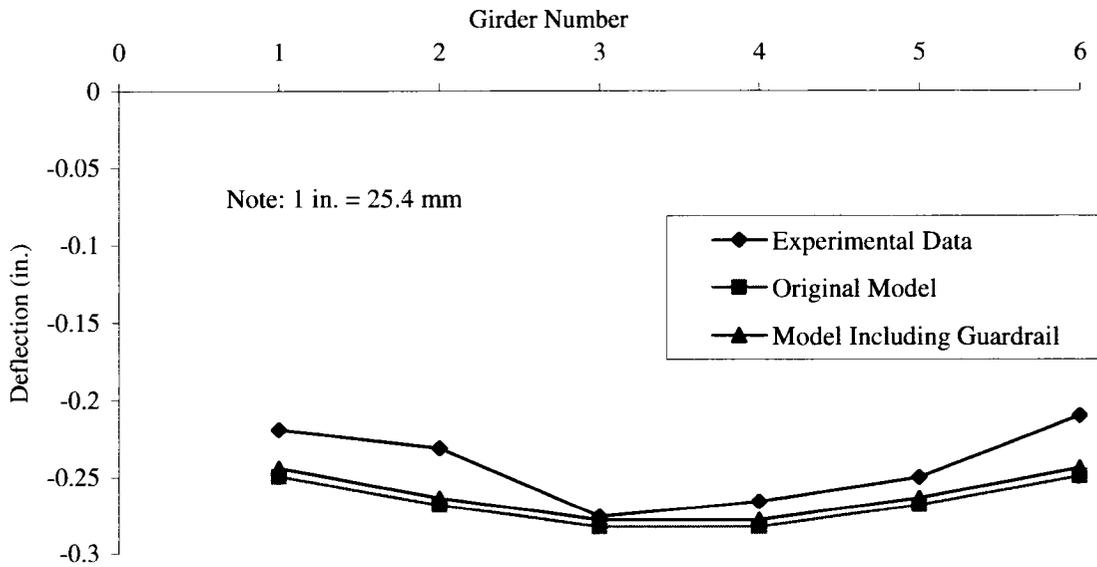


(b) Grillage analysis for Lane 1 loading, deflections at midspan

Figure 6 Field test and analytical results for MBISB 1.



(a) Midspan strain profile for Lane 2 rolling test



(b) Grillage analysis for Lane 3 loading, deflections at midspan

Figure 7 Field test and analytical results for MBISB 2.

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

## 5. GENERAL SUMMARY AND CONCLUSIONS

The current research work encompasses the development of the MBISB system, an alternative replacement for use on LVRs. The details of the experimentation performed and the results obtained have been presented in the three previous papers whose topics include:

- Description of the original BISB system
- Laboratory evaluation of the proposed modifications to determine their strength and applicability
- The implementation of the modifications in two demonstration bridges
- Field evaluation of the demonstration bridges

Based on the results obtained in the laboratory and field evaluations, the MBISB system was determined to be a valid alternative replacement for use on LVRs. An overview of the laboratory and field testing performed and the results and conclusions that support the stated global conclusion are presented in the following sections.

### ORIGINAL BISB SYSTEM

The original BISB system, as previously described, has the following positive attributes:

- Costs approximately 75% of conventional designs for similar applications
- Simple design and reduced construction time
- Requires minimal maintenance

The span length for the BISB design, however, is limited to 50 ft and the unreinforced concrete between the longitudinal steel sections does not contribute to the flexural strength. Two modifications, the ASC and the transverse arch, were proposed as a means to increase the applicable span length by improving efficiency through reducing the self weight and developing composite action.

## **MBISB SYSTEM**

The MBISB system, based on the original BISB design with the inclusion of the modifications, was proposed as a means to span greater distances while maintaining the benefits of the original design. However, the feasibility and performance of the modifications were unknown.

### **Laboratory Testing**

A total of three single bay specimens and a three bay model bridge were evaluated to determine if the modifications were applicable for use in a LVR bridge. Each of the specimens was subjected to service level loading and ultimate loading to determine its strength and mode of failure. An overview of the results and conclusions for the laboratory testing is presented in the following sections.

#### *Single Bay Specimens*

The single bay specimens were evaluated to:

- Determine if arching action was developed in the deck
- Determine if the ASC developed composite action
- Develop a feasible means of constructing the transverse arch

When adequate confinement was provided in both the longitudinal and transverse direction, internal arching action was developed and the deck experienced a punching shear failure. When the transverse arch was not adequately confined, the specimens failed by splitting along the center line of the specimen. Such a failure occurred in both Specimen 1 (177 kips) (13) and Specimen 2 (155 kips) when the transverse straps, which were welded to the bottom flanges of the longitudinal girders, failed in tension. For Specimen 3, larger straps were used, confining the transverse arch and forcing a punching shear failure at 260 kips.

The following conclusions were made:

- The development of arching action is dependent upon the confinement provided by the transverse straps
- When adequately confined, a punching shear failure occurs
- All specimens resisted loads several times greater than a factored HS-20 wheel load

Results from previous research determined that the ASC performed well in push out and single beam flexural tests, (i.e. developing composite action between the steel section and the surrounding concrete (11, 13)). As expected, the ASC developed composite action in the single bay specimens; this was determined by comparing the experimentally determined neutral axis with the theoretical neutral axis. Thus, it was concluded that the ASC will develop composite action within the MBISB section.

The transverse arch was formed between the longitudinal girders, reducing the self weight of the system by removing a majority of the ineffective concrete. Several systems to form the arch were evaluated. Custom rolled steel formwork, constructed from the same material as CMP, was determined to be the best system since it is removable and reusable and removed the largest portion of the ineffective concrete.

#### *Model Bridge*

The single bay specimens demonstrated that the arched deck possessed sufficient strength to resist a factored HS-20 wheel load through arching action and that the ASC developed composite action. However, the single bay specimens did not accurately model the behavior of the modifications when incorporated in a multiple bay configuration similar to a full scale bridge. Therefore, the previously discussed three bay model bridge was constructed to determine:

- The necessity of the transverse straps to develop arching action
- The lateral load distribution in the system
- Ultimate flexural capacity of the model bridge

- Mode of failure in the model bridge
- If the deck reinforcement can be reduced to only that needed in the ASC

Results from the first series of service level loads indicated that the transverse straps did not contribute significantly to the development of arching action and thus were removed. All subsequent tests were performed without the straps in place. The necessary lateral confinement was provided by the adjacent composite girders, the diaphragms, the abutment members, and the ASC reinforcement. Therefore, the transverse straps are not required to resist the applied wheel loads, eliminating the need to weld on a fracture critical section.

The lateral load distribution factors for a single lane loading were determined and compared to AASHTO LRFD Bridge Specification design values for a slab/girder bridge (19). The experimental values, 45% for an interior girder and 56% for an exterior girder, were comparable to the design values of 45% and 54% for the interior and exterior girders, respectively. This result provided evidence that the AASHTO LRFD design values were applicable to the design of the MBISB system.

The model bridge was loaded to failure in flexure (ultimate load = 302 kips); yielding occurred in three of the girders and there was extensive cracking throughout the deck. The remaining strength of the distressed deck was then determined by completing five individual punching shear tests. In all five cases, the load required to cause a punching shear failure was several times greater than a factored HS-20 wheel load. This result confirmed:

- Arching action was developed in the “failed” model bridge
- The flexural cracks did not compromise the structural integrity of the deck
- The deck reinforcement can be limited to that which is necessary in the ASC

The results from the laboratory testing supported the conclusion that the two modifications had more than adequate strength and are applicable for use in an actual bridge. However, the feasibility and behavior of the system when implemented on a LVR need to be verified.

### **Demonstration Bridges**

Two demonstration bridges were designed and constructed to:

- Verify the applicability of the modifications for use in LVR structures
- Determine the feasibility of constructing a MBISB in the field
- Verify the design specifications that were developed

#### *MBISB 1*

The first demonstration bridge (L = 50 ft, W = 31 ft), as previously discussed, was similar to the original BISB design with the exception of the applied modifications. After completing construction and performing the field tests, the following conclusions were made:

- The structure met strength and serviceability criteria
- The lateral load distribution factors = 12% and are not directly comparable to AASHTO design specifications
- The guardrail system increased the total flexural resistance of the bridge by approximately 15%

#### *MBISB 2*

The second demonstration bridge (L = 70 ft, W = 32 ft), as previously described, incorporated larger girders (W27x129 Grade 50) on 6 ft centers. AASHTO LRFD design specifications for a slab/girder bridge were used to design the second demonstration bridge. After completing the construction and the field tests, the following conclusions were determined:

- The design is constructible by in-house forces

- A total cost savings, including labor and materials, of approximately 20% was obtained when compared to conventional designs for the site
- The structure met strength and serviceability criteria by a minimum factor of two, indicating excess capacity in all required areas
- The lateral load distribution factors were determined to be conservative, however were still comparable with design values
- Deck structural reinforcement can be reduced to that necessary to complete the ASC; however, additional temperature and shrinkage reinforcement is required
- The guardrail system increases the total flexural resistance by approximately 2% which is significantly less than in the first bridge

## **DESIGN METHODOLOGY**

Results from testing the model bridge and the MBISB 2 support the applicability of AASHTO LRFD Bridge Specification for use in the design of future MBISBs (19). Therefore, the design methodology developed was based on the AASHTO LRFD Specifications for slab/girder bridges. The design process follows that of a standard composite slab/girder bridge, meeting applicable strength and serviceability criteria with the exception of developing composite action and determining deck reinforcement. The two modifications developed take precedence over the standard design procedure for these details.

A series of MBISB designs for spans ranging from 40 ft to 80 ft have been developed by applying the previously described design methodology and are included in Volume 2 (Design Manual) of the final report for Iowa DOT project TR-467 (1). A PowerPoint presentation documenting the construction of MBISB 2 is also included in the Design Manual. The information presented in the Design Manual is meant to be used by engineers to design a specific MBISB. A detailed description of the MBISB design methodology is presented in Volume 3 (Design Guide) of

the final report for Iowa DOT project TR-467 (2). The Design Guide describes the individual AASHTO strength and serviceability requirements and how the MBISB design fulfills these requirements.

### **PRE-CAST MBISB**

In support of the conclusion that the MBISB design is applicable to LVR applications, county engineers in Blackhawk County, Iowa have developed an alternative bridge design that combines the efficiency of pre-cast technology with the MBISB design. The resulting design is a bridge that consists of four pre-cast panels which are connected by an in-field concrete pour. The system is constructible by in-house forces and reduces total costs by approximately 15% when compared to other designs for a particular site.

A pre-cast MBISB structure was field tested by ISU BEC researchers and was determined to exceed strength and serviceability requirements. A more detailed description of the pre-cast MBISB system and the results of the field test are presented in Appendix B.

The results from the field test corroborate those from the laboratory testing to support the strength and applicability of the modifications. The resulting pre-cast MBISB design is of lower cost, relatively simple to construct and can span up to 47 ft. A full description of the design, field test and the analysis performed on the pre-cast system can be found the following report “Investigation of a Pre-Cast Modified Beam-in-Slab Bridge System”, by Wipf et al. (20).

## 6. RECOMMENDATIONS FOR FUTURE RESEARCH

On the basis of the work completed and the resulting conclusions, the following areas of investigation are proposed to improve the MBISB system and its implementation on Iowa's LVRs.

1. Develop short courses to inform engineers about the several LVR bridge alternatives that have been developed.

Several alternative superstructure systems have been developed for use on LVRs; however, only a limited number of these have been constructed beyond the initial demonstration bridges. A series of short course should be developed to inform county engineers of the several replacement structures that have been developed; the courses would promote the benefits of using alternative designs by:

- Providing an overview of the various replacement superstructures developed
- Presenting in detail a design example including:
  - Selecting a specific design
  - Providing instructions on the use of the software that has been developed
  - Preparing a set of construction drawings for the various designs
- Developing a construction plan by presenting the methods used in constructing the demonstration bridges

2. Further development and evaluation of both pre-cast sub and super structure systems.

The use of pre-fabricated bridge elements and systems to simplify and expedite construction while lowering costs is currently being promoted by the FHWA. The PMBISB system, described in Appendix B, has demonstrated the benefits of a pre-cast superstructure by reducing construction time and cost. By combining the PMBISB or similar pre-fabricated

superstructure elements with pre-cast abutments, a lower cost LVR standardized bridge system would be available for county engineers. An investigation of such a pre-cast bridge system would include:

- Cataloging applicable existing systems
- Developing a standard pre-cast substructure that is compatible with multiple pre-fabricated and constructed in place superstructures
- Design and construct four full scale pre-cast abutments (two bridges) for demonstration in conjunction with a “partner” county. A minimum of two different superstructures would be utilized to increase the range of applicability
- Field test the resulting structures (instrumenting both the substructure and superstructure) to determine the adequacy of the design
- Develop a set of standardize pre-fabricated bridge (substructure and superstructure) plans for use on LVR bridges

3. Evaluate the use of Carbon Fiber, Glass Fiber bars or MMFX steel as ASC reinforcement.

Sufficient laboratory and in-field data has been completed on the use of steel reinforcement in the ASC. However, the use of black steel reinforcement increases the risk of corrosion in the bridge deck. By implementing non-ferrous ASC reinforcement, or MMFX steel reinforcement, the potential for deck corrosion would be essentially be eliminated. A limited series of laboratory flexural beam tests would be required to evaluate the applicability of using the Carbon or Glass Fiber bars as reinforcement for the ASC. It would not be necessary to investigate the MMFX steel in the laboratory. Upon the successful completion of the laboratory testing phase, the different reinforcements could be implemented by:

- Developing composite action with MMFX steel in a demonstration MBISB or

PMBISB and developing composite action with carbon or glass fiber in a second  
MBISB or PMBISB

## 7. ACKNOWLEDGEMENTS

I wish to express my deepest gratitude to my Co-major Professors, Dr. Terry J. Wipf and Dr. F. Wayne Klaiber for their guidance and motivation throughout this project. Their expertise in low volume road bridges and vision of a simple to construct alternative design provided the foundation of this project.

On an individual level, I wish to thank Dr. Klaiber for extending this opportunity to me, I will be forever grateful. He has been an outstanding mentor and has provided me with an example by which to conduct myself. I also wish to thank Dr. Wipf for providing moral support through the duration of this project. His positive reinforcement helped keep me on track, especially during difficult times. I would also like to acknowledge the many opportunities outside of this project that Dr. Klaiber and Dr. Wipf provided me, including teaching, traveling and presenting at conferences. Thank you, both; again for taking a chance on me, I hope I met your expectations.

I would like to thank my committee members: Dr. Vernon Schaefer, Dr. Fouad Fanous, and Dr. Loren Zachary for evaluating my research efforts and challenging me. I especially appreciate their co-operation in scheduling exam times. I also want to thank all of them for being positive role models; their demonstrated professionalism has provided me a benchmark by which to measure one's progress.

I extend my gratitude to the Project Development Division of the Iowa Department of Transportation and the Iowa Research Board for funding this project. I also wish to thank the various Iowa DOT and county engineers who helped with this project and provided their input and support. In particular, I would like to thank Lyle Brehm, County Engineer, Tama County, for his assistance with the project.

Appreciation is also extended to the Tama County Bridge Crew, Ron Green, Stacy Skala, Tim (Beast) Martens and Steve Van De Walle for their efforts during the construction of the demonstration bridges. Working with these gentlemen was the highlight of this project for me.

Special thanks are extended to Doug Wood, Manager of the ISU Structural Engineering Laboratory for his advice, guidance and assistance during the various tests carried out to complete this project.

To the following people, I extend my deepest gratitude for without their hard work and dedication, this project would have never come to fruition. Their willingness to work long hours, sweat, sacrifice and finally persevere was unmatched: Curtis Holub, Jonathon Greenlee, Riley Smith, Ben Woline, Ben Drier, Toshia Akers, Toni Tabbert, Emily Allison, Michelle Heikens, Eric Cannon, Alfred Wessling, Milan Jolley, Holly Boomsma and Kristine Palmer.

I wish to thank Dr. Richard A. Reid and Dr. Arden B. Sigl for encouraging me from afar over the past four years. I hope that I made South Dakota State University proud while I was here at ISU.

I wish to thank my family for their support over the decade plus that I have been involved in higher education. Yes, Mom and Dad, I will be getting a real job sometime in the future. Thank you for raising me with the backbone to stand up, continue on and persevere through sheer determination; I will love you both forever.

Finally to the person that I owe the most, Miranda Bruna, this has been a long four years Sweetie and I know that without your constant support, I would have ceased to continue long ago. Your strength and faith have bolstered me through the difficult times and I want you to know that I love you for it.

**APPENDIX A**

**Modified beam-in-slab bridge grillage analysis**

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## A1. MODIFIED BEAM-IN-SLAB BRIDGE GRILLAGE ANALYSIS

The grillage method of analysis was selected for modeling both demonstration bridges. A grillage analysis refers to the mathematical modeling of the deck slab/longitudinal girder continuum by a series of connected longitudinal and transverse beam elements. The longitudinal and transverse beam lines are established at a spacing that allows for the distribution of the loads and physically represents the structure being modeled. If modeled appropriately, the internal member forces and deflections of any beam within the grillage should equal the resultant effects that would be present in the portion of the slab represented by the beam element ( $I$ ). This is accomplished by assigning flexural inertias and torsional constants to the longitudinal and transverse elements; the values assigned are based on the geometry of the actual structure. The resulting steel members are then transformed to equivalent concrete members by applying appropriate modular ratios.

A plane grillage analysis was conducted for both demonstration bridges. A plane grillage analysis is one in which all members, both longitudinal and transverse, are assumed to be in the plane of the structural model. For the MBISB models, the structural plane was taken as the neutral axis of the concrete deck with both the longitudinal and transverse flexural properties transformed to the structural plane. This methodology follows Hambly's development of a plane grillage model for a slab/girder bridge ( $I$ ). However, this method of analysis does not account for changes in the structural plane between the interior and exterior girders due to changes in deck thickness and the effective flange width. Likewise, geometric consistency between the longitudinal and transverse members is lost since the centroids of each are normally not within the same plane, as was the case with both MBISB models. By not accounting for this separation between the longitudinal and transverse elements, the model does not distribute load as efficiently or account for changes in the neutral axis due to membrane action ( $I$ ). To eliminate the problems attributed to the plane grillage methodology a more refined and complex method of analysis is required, such as a three dimensional solid element finite element analysis. However, for the purpose of this project, the results from a

plane grillage analysis were determined to be sufficient for verifying measured bridge behavior and for providing a basis for future analytical analysis and design.

## **ELEMENT TYPES**

Due to the unique geometry of the MBISB deck, the beam elements used to model the transverse members required that they be tapered. Therefore, the finite element program ANSYS 8.1 was chosen to perform the analysis since it contains elements that describe tapered beam behavior as well as prismatic beams of constant cross section. The element types used in both models, along with a brief description of their capabilities and restrictions as described in the ANSYS User Manual are presented in the following sections (2). For each element, individual models using a single element were created independent of the grillage model and tested against known results to ensure proper orientation and selection of boundary conditions.

### **BEAM4 Element**

For both models, the longitudinal beams segments were described using BEAM4 elements since the sections were of constant cross section.

From the ANSYS 8.1 Users Manual:

“BEAM4 is a uniaxial element with tension, compression, torsion, and bending capabilities. The element has six degrees of freedom at each node; translation in the nodal x, y, and z directions and rotations about the nodal x, y, and z axes.”

“The geometry, node locations, and coordinate system are shown (see Figure A1). The element is defined by two or three nodes, the cross-sectional area, two area moments of inertia (IZZ and IYY), two thicknesses (TKY and TKZ), an angle of rotation about the element x-axis, the torsional moment of inertia, and the material properties.”

“The beam must not have zero length or area. The moments of inertia, however, may be zero if large deflections are not used. The beam can have any cross sectional shape for which the moments of inertia can be computed. The stresses, however, will be determined as if the distance between the neutral axis and the extreme fiber is one-half of the corresponding thickness.”

### **BEAM44 3-D Tapered Unsymmetrical Beam Element**

For the transverse elements in both models, the arched geometry of the deck was approximated by applying tapered BEAM44 elements.

“BEAM44 is a uni-axial element with tension, compression, torsion, and bending capabilities. The element has six degrees of freedom at each node: translations in the nodal x, y, and z directions and rotations about the nodal x, y, and z axes (see Figure A2). The element allows different unsymmetrical geometry at each end and permits the end nodes to be offset from the centroidal axis of the beam.”

### **MODEL DEVELOPMENT**

Both demonstration bridges were considered to be slab/girder bridges, thus the models were developed using a similar methodology. For both bridges, each longitudinal girder was represented by a beam line; the actual beam spacing was assumed to represent the effective flange width which was supported by the strain profiles determined from the laboratory and field testing. The longitudinal flexural stiffness (moment of inertia) and torsional stiffness (torsional constant) of the composite steel girder and concrete deck were based on the approximated geometric cross section of the full effective flange width.

The transverse beams of the grillage simulated the transverse flexural stiffness and torsional stiffness of the concrete deck. The transverse elements were spaced at a distance less than the recommended  $1/8$  the span length suggested by Hambly ( $l$ ); a smaller transverse spacing was expected to refine the behavior of the model. For the second demonstration bridge, the transverse spacing was varied between 2, 3 and 4 ft to provide nodes for applying point loads which simulated the truck loading. A more detailed description of the individual models is presented in the following sections.

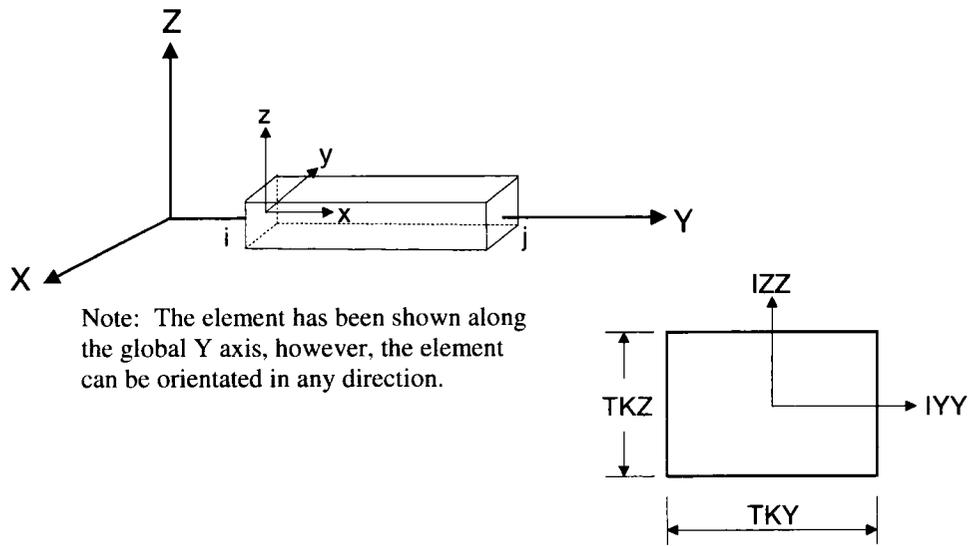


Figure A1. Geometry of a typical BEAM4 element.

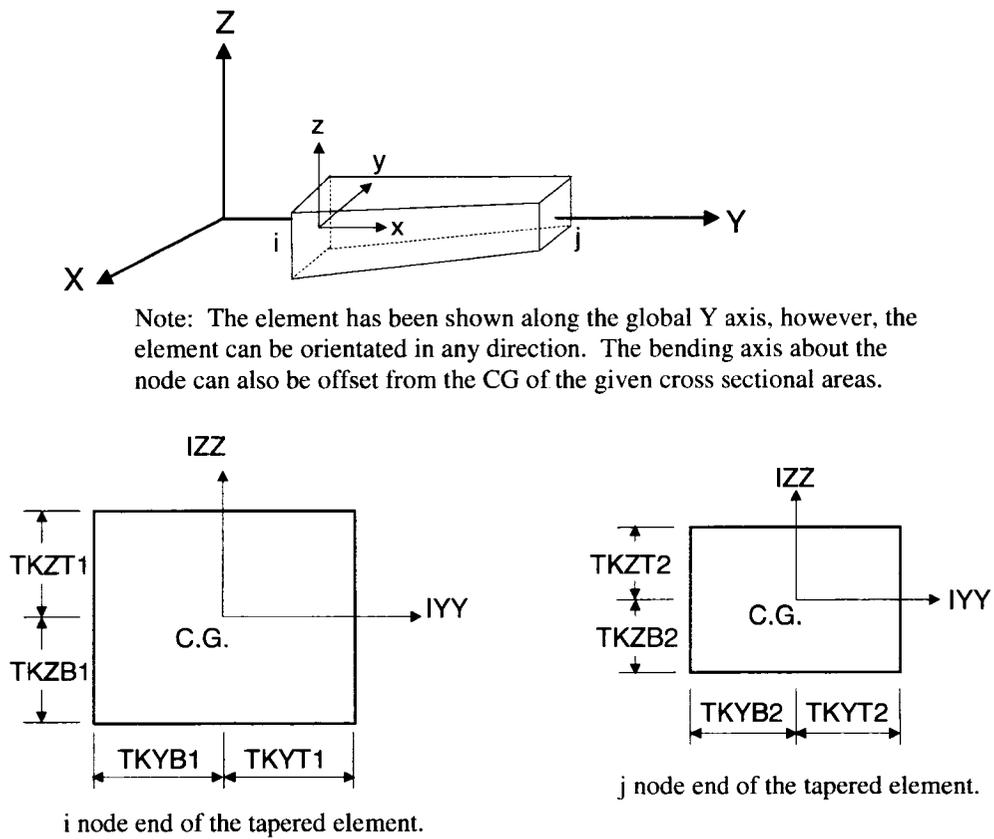


Figure A2. Geometry of a typical tapered BEAM44 element.

**MBISB 1**

The first demonstration bridge consisted of 16-W12x79 girders set on 2 ft centers with the transverse arch placed between the girders. The Alternative Shear Connector (ASC) was used to develop composite action with the concrete slab that is placed between the girders; a typical cross section of the first demonstration bridge is presented in Figure A3a. For the grillage model, each longitudinal beam line was represented by a line of BEAM4 elements with the properties listed in Table A1. The bending properties for the longitudinal beam elements were calculated base on the gross section properties for the approximated cross section presented in Figure A3c. The flexural properties are transformed about the neutral axis of the concrete deck and the steel girders were transformed to an equivalent concrete material by applying a modular ratio of 7 corresponding to a concrete compressive strength of 5.5 ksi.

Calculating the torsional constant for the longitudinal members was difficult due to the unique cross section of the deck. The torsional stiffness of the cross section was shown by Prandtl to be proportional to the volume under an “inflated bubble” stretched across a hole of the same shape as the member being torqued ( $I$ ). An approximation was made in that the area of the concrete in the cross section was replaced an equivalent rectangle with the depth of the slab varied to determine the equivalent area. This depth was then substituted into Equation A1 which was used to calculate the torsional resistance of a slab ( $I$ ). The torsional resistance of the W section was neglected since it is small in comparison to the concrete deck.

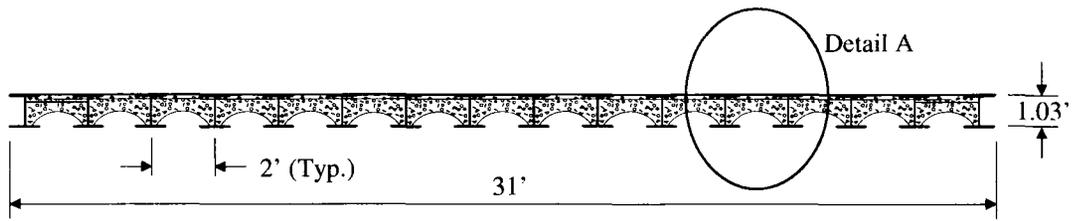
$$C = bd^3 / 6$$

Eqn A1

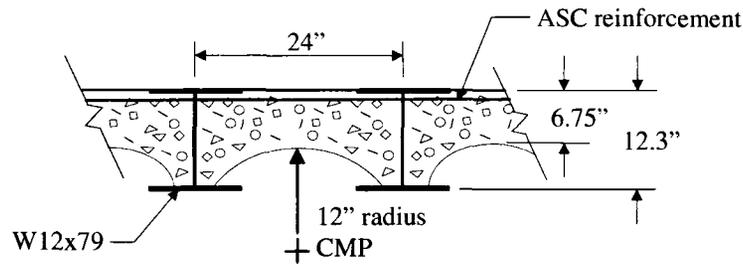
$C$  = Slab torsional constant (in<sup>4</sup>)

$b$  = Width of slab (in.)

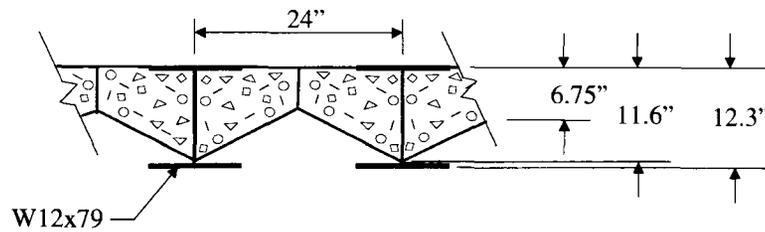
$d$  = Depth of slab (in.)



a. MBISB 1 cross section



b. Detail A



c. Approximated MBISB 1 cross section for the grillage model

Figure A3. Geometric configuration of the cross section for MBISB 1.

Table A1 Longitudinal member properties for the MBISB 1 grillage analysis.

Longitudinal BEAM4 Element Properties		
Section Prop.	Exterior Girder	Interior Girder
Area (in <sup>2</sup> )	274	382.6
I <sub>zz</sub> (in <sup>4</sup> ) (flexural constant)	5,868.4	6,748.3
I <sub>xx</sub> (in <sup>4</sup> ) (torsional constant)	1,545	3,089.5
TKZ (in.)	0.5	0.5
TKY (in.)	12	12

The transverse element section properties were based upon the thickness of the deck at the respective ends (11.6 in. at the “thick end” and 6.75 in. at the “thin end”) and a 1 ft wide spacing. All flexural properties were taken with respect to the structural plane and the torsional stiffness was calculated by applying Equation A1. The girder spacing was utilized to provide truck loading points in addition to keeping the aspect ratio of the transverse beams proportional to the longitudinal girder spacing. The sectional properties of a typical tapered beam element used to model the arched cross section are listed in Table A2.

Table A2 Transverse member properties for the MBISB 1 grillage analysis.

Transverse BEAM44 Element Properties			
Section Prop.	“Thin end”	Section Prop.	“Thick end”
Area (in <sup>2</sup> )	81	Area (in <sup>2</sup> )	139.2
I <sub>zz</sub> (in <sup>4</sup> ) (flexural constant)	972	I <sub>zz</sub> (in <sup>4</sup> ) (flexural constant)	1,670
I <sub>xx</sub> (in <sup>4</sup> ) (torsional constant)	615	I <sub>xx</sub> (in <sup>4</sup> ) (torsional constant)	3,121
TKZB1 (in.)	0.95	TKZB2 (in.)	5.8
TKYB1 (in.)	6	TKYB2 (in.)	6
TKZT1 (in.)	5.8	TKZT2 (in.)	5.8
TKYT1 (in.)	6	TKYT2 (in.)	6

The boundary conditions for the grillage model were considered to be pinned and “rollered” allowing for rotation at each support and translation in the longitudinal direction at one end of the model as indicated in Figure A4. The use of the applied boundary conditions was supported by the strain data measured in the field tests. A line of nodes between each longitudinal beam line was necessary to join the two tapered beams. Localized deflections between the transverse elements were of such small magnitude that the nodes were not connected in the longitudinal direction.

The position of the test vehicle for the maximum midspan moment for the Lane 1 loading condition is also shown in Figure A4; the resulting midspan deflection of the model is compared in

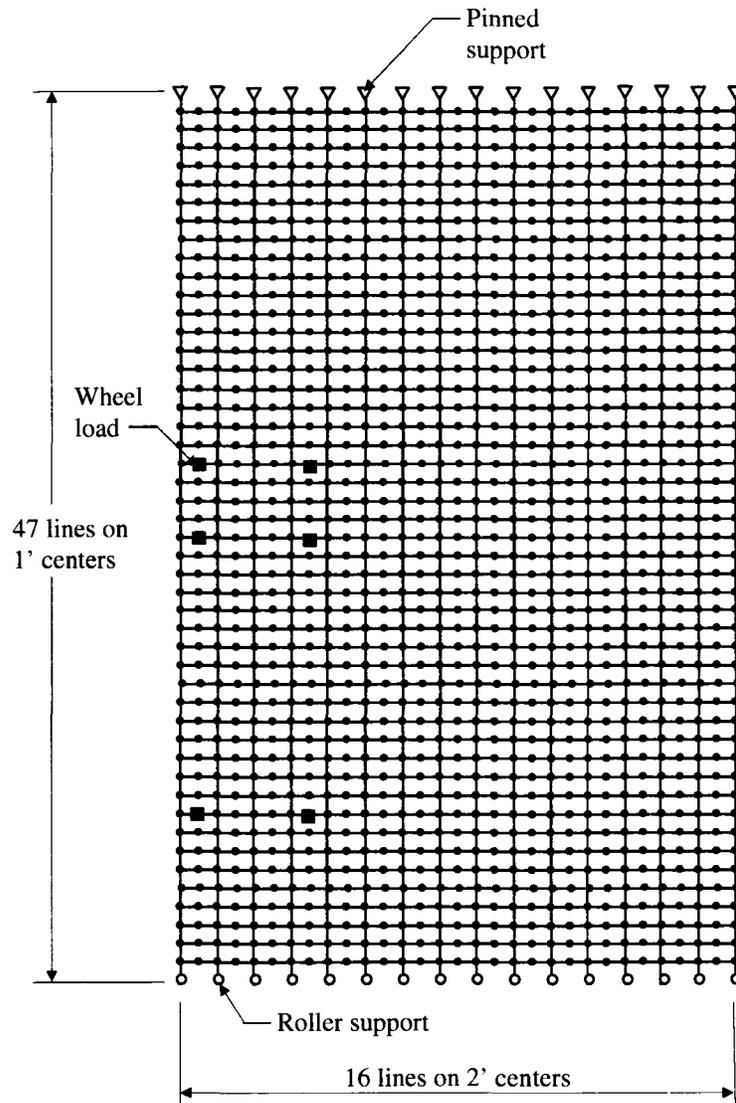


Figure A4. Grillage mesh for MBISB 1, Lane 1 truck loading included.

Figure A5. The resulting output from the grillage model was found to overestimate the midspan deflections when compared to values measured in the field tests. Several parameters were then investigated to determine the effect on the resulting behavior of the model including changing the boundary conditions to a fixed-fixed configuration, increasing the modulus of elasticity for the concrete deck and decreasing the transverse element spacing.

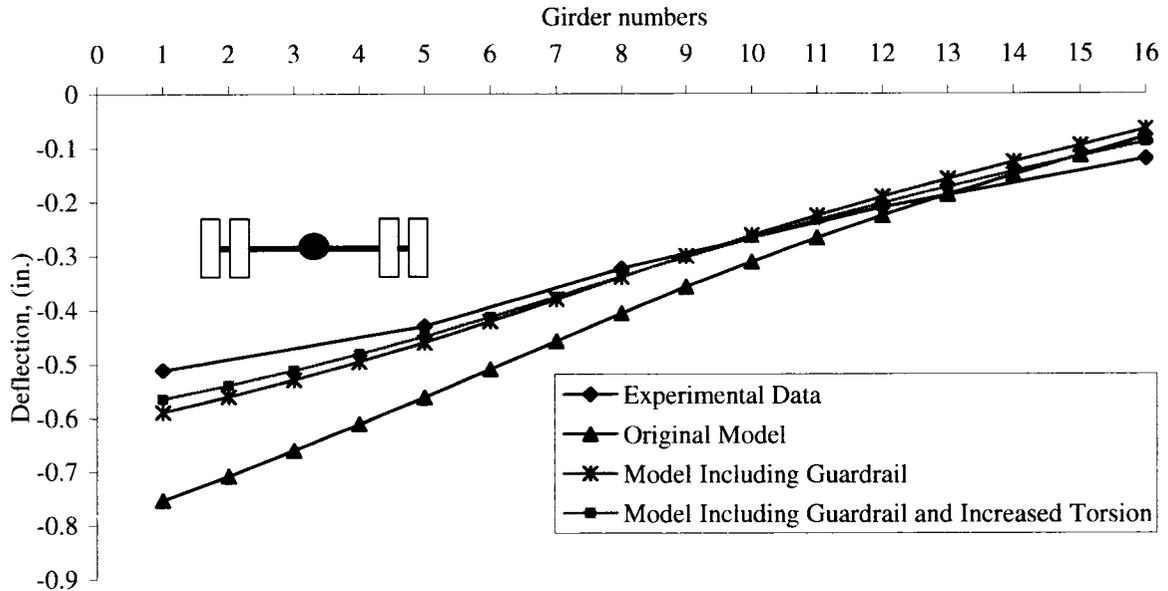


Figure A5. MBISB 1 midspan displacements for the Lane 1 loading.

The resulting deflections from the fixed-fixed boundary conditions were approximately 1/4 that of the pinned and “rollered” conditions, underestimating the experimental midspan deflection. This boundary condition was eliminated since neither the experimental strains or deflection data supported the modeled results. By increasing the concrete stiffness, the resulting model deflections were reduced but the increase in material stiffness could not be supported by physical realities. Reducing the transverse element spacing from 12 in. to 6 in. was found to have no effect on the midspan deflections. This result was consistent given the methods used to calculate the transverse element properties are linear for a section of constant depth.

After completing the first parametric study, the effect of the guardrail on the total structural response was investigated. To determine the increase in flexural resistance due to the guardrail, an analysis was completed using Staad Pro (3). The midspan deflection of the exterior composite girder due to half of the truck loading was found to be 3.87 in.

A guardrail section ( $I_{zz} = 13.34 \text{ in}^4$ ,  $A_{xx} = 2.01 \text{ in}^2$ ), as shown in Figure A6, was then rigidly attached to the exterior girder and the C10x20 guardrail posts; a midspan deflection of 1.35 in. resulted. The connection between the longitudinal girder, the channel posts and the guardrail member was “reduced” by releasing the moments. The resulting midspan deflection for the reduced connectivity configuration was 1.56 in. When comparing the midspan deflections for the fully fixed and released conditions, the increase in flexural stiffness of the exterior girder due to the guard rail was taken as three. This value was used since an increase in flexural stiffness of three is between the fully fixed and the released condition which was expected to more closely model the actual in-field conditions of the guardrail connectivity.

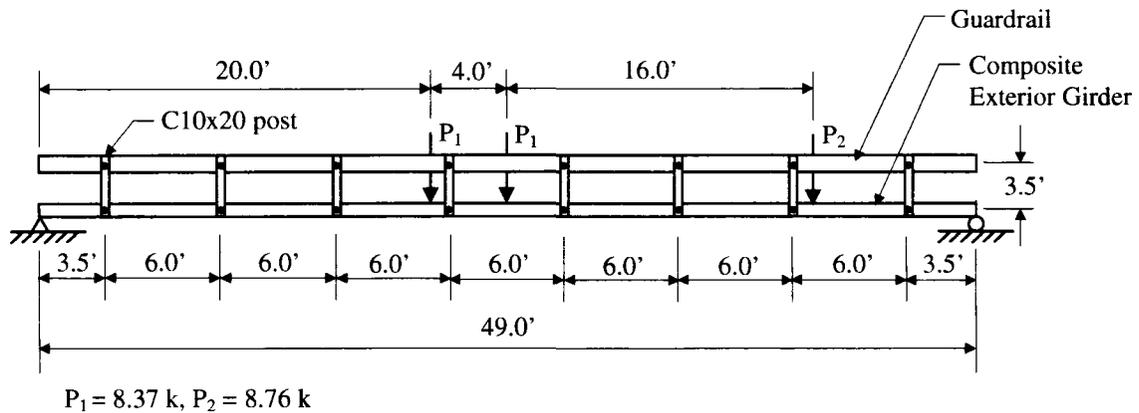


Figure A6. MBISB 1 guardrail configuration for determining the increased flexural stiffness.

The flexural rigidity of the longitudinal exterior girders was then increased by a factor of three to account for the increased longitudinal flexural stiffness due to the guardrail. The resulting midspan deflection was then reduced as indicated in Figures A5 and A7 which provided a result more closely defined by the experimental midspan data indicating that the guardrail provides a significant amount of flexural stiffness to the system.

A parametric model was developed to determine the effect of varying the torsional constant and the validity of the assumptions. For the parametric model, the longitudinal torsional stiffness for

both the interior and exterior girders was increased by 1.5 times with all other variable held constant; the midspan deflections resulting from this case were compared to previous models. The results from the increased torsional moment of inertia indicated an improved cross sectional deflection but the change was small in comparison to global displacements as shown in Figure A5 and A7. Thus, the initial method of approximating the torsional stiffness was deemed acceptable. Thus it was decided that the model provided a satisfactory description of the behavior of the first demonstration bridge.

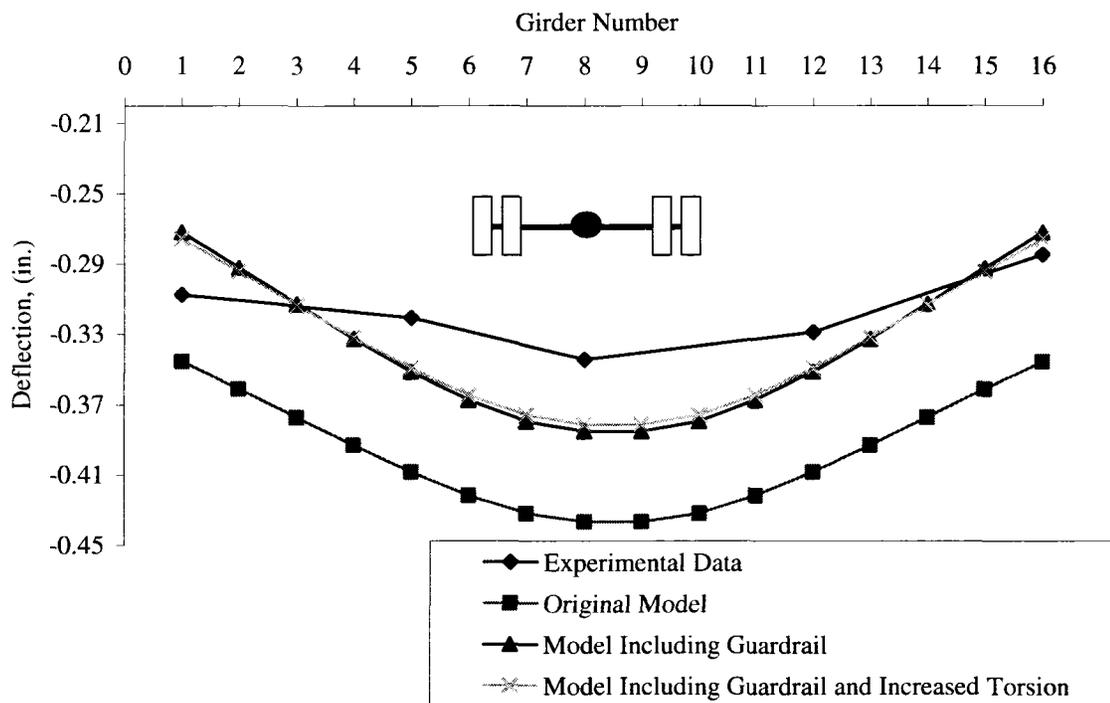


Figure A7. MBISB 1 midspan displacements for the Lane 3 loading.

## MBISB 2

The second demonstration bridge consisted of 6-W27x129 sections set on 6 ft centers with a transverse arch placed between the girders; composite action was developed through the ASC. A



concrete deck which was taken as the structural plane for grillage model. The steel girders were transformed to equivalent concrete sections by applying a modular ratio of 6 corresponding to a concrete compressive strength of 6.27 ksi.

Table A3 Longitudinal member properties for the MBISB 2 grillage analysis.

Longitudinal BEAM44 Element Properties		
Section Prop.	Exterior Girder	Interior Girder
Area (in <sup>2</sup> )	848.4	1,196
$I_{zz}$ (in <sup>4</sup> ) (flexural constant)	72,350	92,224
$I_{xx}$ (in <sup>4</sup> ) (torsional constant)	10,342	16,905
TKZB1 (in.)	0.5	0.5
TKYB1 (in.)	21.8	21.8
TKZT1 (in.)	0.5	0.5
TKYT1 (in.)	8.85	8.85

The torsional constants for the longitudinal members were calculated by applying the methodology suggested in Hambly (1) and Barker and Puckett (4). When confronted with a cross section of various shapes, the torsional stiffness of the combination can be estimated by summing the torsional constants for the individual shapes. Thus, the approximated sections shown in Figure A8b were further approximated to include a slab of uniform thickness and a triangular stem portion; the approximated torsional cross section is presented in Figure A8c. The torsional constant for the triangular was calculated using the relationship from Roark which is presented in Figure A8c while the uniform slab was calculated using Equation A1 (5).

The transverse members were modeled using three individual BEAM44 elements to approximate the arched profile; a cross section of the elements is shown in Figure A9. The element section properties were based on the depth at each end and the respective spacing which was varied

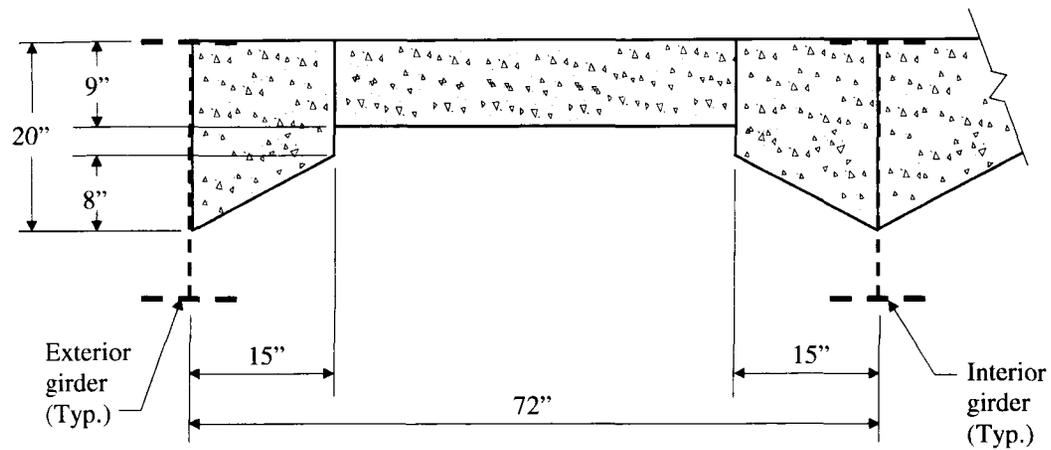


Figure A9. Geometric configuration of the transverse grillage beam elements for MBISB 2.

between 2, 3 and 4 ft to account for the truck placement. The flexural properties for each element were transformed about the structural plane and the torsional constant was calculated by applying Equation A1. Thus, the resulting properties for the different spacing were a function of the width since the section depths remained constant. The sectional properties for a typical tapered element are presented in Table A4 and the properties of the constant depth section for the constant depth section are presented in Table A5; both beam elements are taken to be 2 ft wide.

Table A4 Member properties for the tapered transverse beam elements for MBISB 2.

Transverse BEAM44 Element Properties			
Section Prop.	“Thin end”	Section Prop.	“Thick end”
Area (in <sup>2</sup> )	288	Area (in <sup>2</sup> )	480
I <sub>zz</sub> (in <sup>4</sup> ) (flexural constant)	5,795	I <sub>zz</sub> (in <sup>4</sup> ) (flexural constant)	16,635
I <sub>xx</sub> (in <sup>4</sup> ) (torsional constant)	6,912	I <sub>xx</sub> (in <sup>4</sup> ) (torsional constant)	32,000
TKZB1 (in.)	12	TKZB2 (in.)	12
TKYB1 (in.)	3.15	TKYB2 (in.)	11.15
TKZT1 (in.)	12	TKZT2 (in.)	12
TKYT1 (in.)	8.85	TKYT2 (in.)	8.85

Note: Tributary width = 24 in.

Table A5 Member properties for the constant depth transverse beam element for MBISB 2.

Transverse BEAM44 Element Properties	
Section Prop.	
Area (in <sup>2</sup> )	216
I <sub>zz</sub> (in <sup>4</sup> ) (flexural constant)	5,545
I <sub>xx</sub> (in <sup>4</sup> ) (torsional constant)	2,945
TKZB1 (in.)	12
TKYB1 (in.)	0.15
TKZT1 (in.)	12
TKYT1 (in.)	8.85

Note: Tributary width = 24 in.

The boundary conditions for the grillage model were considered to be pinned and “rolled” similar to the MBISB 1 grillage model; the beam lines and boundary conditions for the grillage model are presented in Figure A10. The selection of the applied boundary conditions was supported by the strain data measured in the field tests. Two lines of nodes between each longitudinal beam line were necessary to join the three BEAM44 elements. The local deflections between the lines of transverse elements were of such small magnitudes that the secondary nodes were not connected in the longitudinal direction.

The position of the test vehicle for the maximum midspan moment for the Lane 1 loading condition is also shown in Figure A10; the resulting midspan deflection of the model is compared to the values determined from the field tests in Figure A11. The resulting output from the grillage model overestimated the midspan deflection when compared to the values determined in the field test. Of the variables investigated in the parametric studies conducted for MBISB 1, only inclusion of the guardrail in the grillage model produced improved results which were supported by the physical attributes of the demonstration bridge. Therefore, only adding the guardrail was considered in the evaluation of the MBISB 2 grillage model.

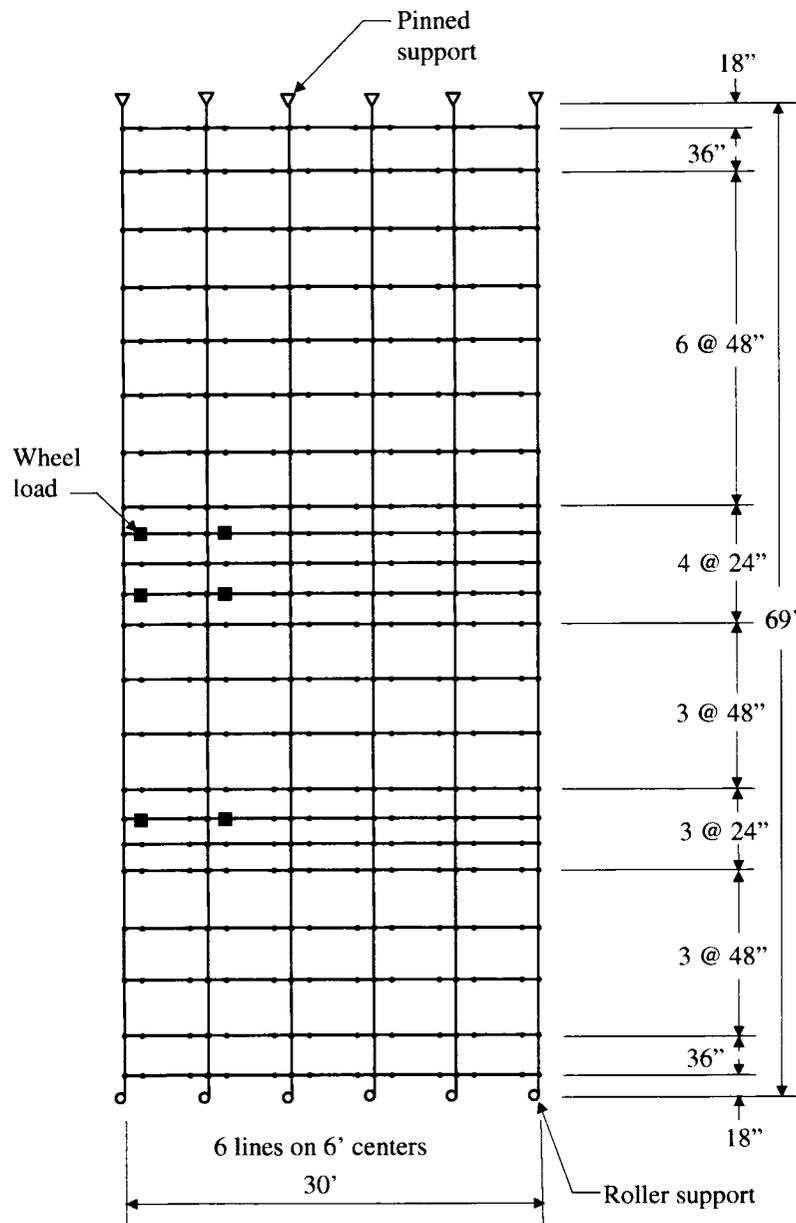


Figure A10. Grillage mesh for MBISB 2, Lane 1 truck loading included.

The effect of the guardrail on the flexural resistance of the grillage model was determined following the same procedure applied to the first demonstration bridge. The midspan deflection of the composite exterior beam due to a half truck loading was found to be 0.859 in. A guardrail section ( $I_{zz} = 137.4 \text{ in}^4$ ,  $A_{xx} = 3.21 \text{ in}^2$ ), as shown in Figure A12, was then rigidly attached to the exterior

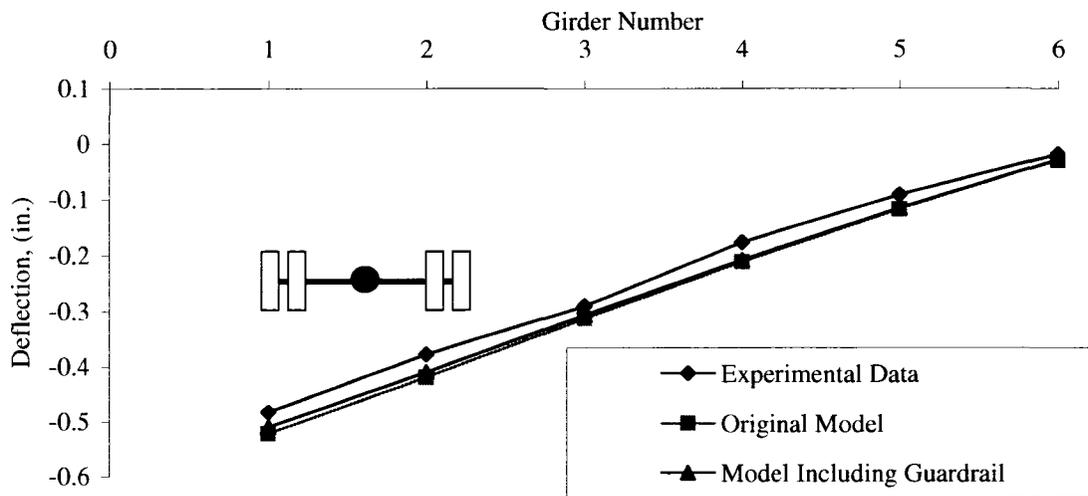


Figure A11. MBISB 2 midspan displacements for the Lane 1 loading.

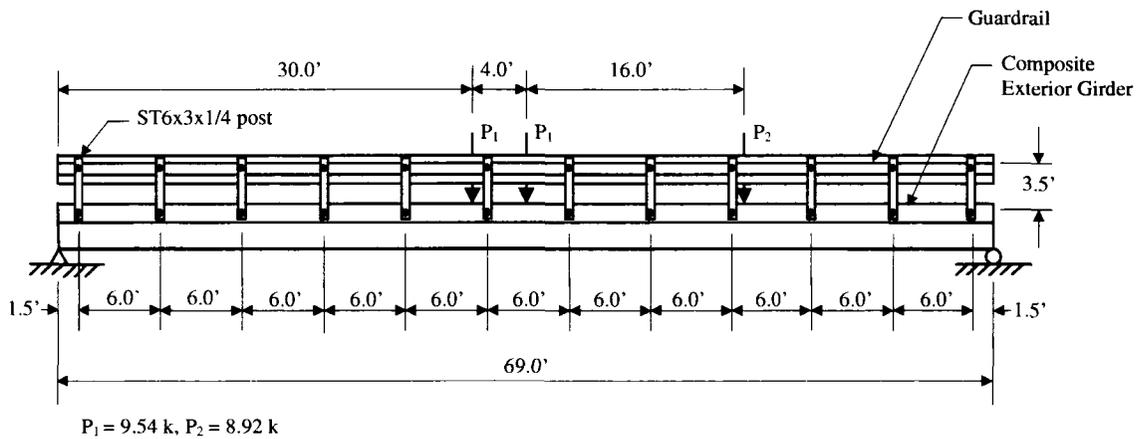


Figure A12. MBISB 2 guardrail configuration for determining the increased flexural stiffness.

girder and the ST6x3x1/4 posts; a midspan deflection of 0.792 in. resulted. The connection between the longitudinal girder, the structural tube posts and the guardrail member was “reduced” by releasing the moments at the joints between the guardrail member and the structural tubes. The resulting deflection for the partially released configuration was 0.813 in.

When compared to the midspan deflections of the exterior composite beam and the partially released conditions, the increase in the flexural stiffness of the exterior girder was only 7%. This result indicated the guardrail had a minimal influence on the flexural resistance of the demonstration bridge which was expected since the flexural stiffness of the guard rail is approximately 1/100 of the flexural stiffness of the exterior longitudinal member. The flexural stiffness of the exterior girders was increased by 7%; the resulting deflections are compared to the experimental results for the Lane 1 and 3 loading in Figures A11 and A13, respectively. The resulting midspan deflections were reduced due to including the guardrail but the overall effect when compared to the global deflections is small.

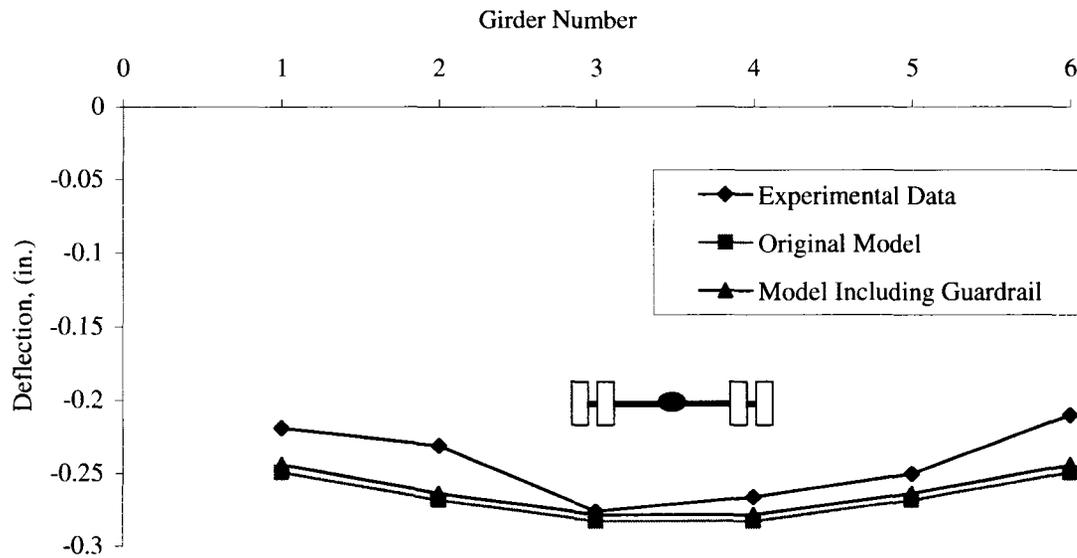


Figure A13. MBISB 2 midspan displacements for the Lane 3 loading.

## SUMMARY AND CONCLUSIONS

The results obtained for the demonstration bridges indicate the structures can be modeled using a plane grillage analysis to obtain reasonable deflection results that can be used for future analysis and design. The arched cross section of the MBISB system required the approximation of

the member section properties; through parametric studies, the methods employed were validated. The guardrails were determined to influence the global behavior of both structures; however, the effects were more critical in MBISB 1. This result was expected since the guardrail accounted for approximately 15% of the total flexural stiffness of the structure. The effect of the guardrail was minimal in the second demonstration bridge where the guardrail only accounted for approximately 2% of the total flexural stiffness. The grillage models were determined to be sufficiently accurate for validating the in-field behavior of both demonstration bridges.

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**APPENDIX B**

**Pre-cast MBISB alternative bridge design**

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## ABSTRACT

Replacing deficient bridges on low volume roads poses a financial challenge to Iowa county engineers. To extend available funds, county engineers in Blackhawk County, Iowa, have developed a low cost alternative design by combining the Modified Beam-in-Slab Bridge (MBISB) concept with the efficiency of pre-cast concrete. The resulting design is a 40 ft long, 32 ft wide bridge assembled on site with four pre-cast panels, allowing for rapid in-field construction.

The panels are constructed in the county maintenance yard and stored for installation at a later date. An individual panel consists of 3-W14x90 Grade 50 steel girders with the alternative shear connector (ASC) installed to develop composite action. The transverse arch is formed by using half sections of 18 in. diameter PVC pipe removing a majority of the ineffective concrete. The panels are installed upon a completed abutment and joined by an in-field concrete placement. The structure was constructed in its entirety by in-house forces and saved approximately 15% when compared to conventional systems.

The Iowa State University Bridge Engineering Center performed a field test on the first Pre-cast MBISB (PMBISB) to determine the structural behavior of the design. Particular attention was given to determine the lateral load distribution, support conditions and the level of continuity between the pre-cast panels.

Results indicated the PMBISB to be a stiff structure with excellent lateral load distribution; maximum measured deflections and stresses are well below limiting values. By comparing deflections between adjacent panels, the cast-in-place concrete joints were found to effectively create a continuous structure by providing adequate load transfer. Thus, the PMBISB was determined to possess excess capacity and is a sufficient alternative replacement option.

## **B1. PRE-CAST MODIFIED BEAM-IN-SLAB BRIDGE ALTERNATIVE**

During the completion of Iowa Department of Transportation (Iowa DOT) Project TR-467, a limited number of Iowa county engineers, with guidance from the Iowa State University (ISU) Bridge Engineering Center (BEC), have developed and constructed derivatives of the Modified Beam-in-Slab Bridge (MBISB). One such system, the Pre-cast Modified Beam-in-Slab Bridge (PMBISB) was developed by the Blackhawk County Engineering Office. The resulting system combines the basic MBISB design with the efficiency of pre-cast concrete construction.

### **DESCRIPTION OF THE PMBISB**

The PMBISB system consists of four pre-cast panels, two interior and two exterior, which implement the transverse arch and the alternative shear connector (ASC), resulting in a cross section similar to that of the MBISB system. Each panel was constructed by Blackhawk County forces at their casting facility, cured, stored, transported to the construction site and set in place leaving an 18 1/2 in. gap between adjacent panels. The three joints between the four panels are closed by an in-field concrete pour, resulting in a 32 ft wide bridge. The PMBISB design has been used exclusively on 40 ft spans, though the system is applicable for spans of up to 47 ft. As reported by the Blackhawk County Engineering Office, the PMBISB systems cost between \$56 to \$60 psf which is a 15% to 20% cost savings over conventional bridge systems.

### **CONSTRUCTION OF THE PMBISB**

The construction of the PMBISB system, which can be divided into two stages, the pre-casting of the panels and the in-field assembly, has been performed entirely by the Blackhawk County Bridge Crew.

#### **Pre-casting of the Panels**

Each pre-cast panel, which has 3-W14x90 Grade 50 girders, is fabricated in the following sequence. The ASC holes are torched into the webs of the girders following the spacing presented in Volume 2 of the Iowa DOT final report TR-467 (1). The girders are spaced 33 1/4 in. on center to

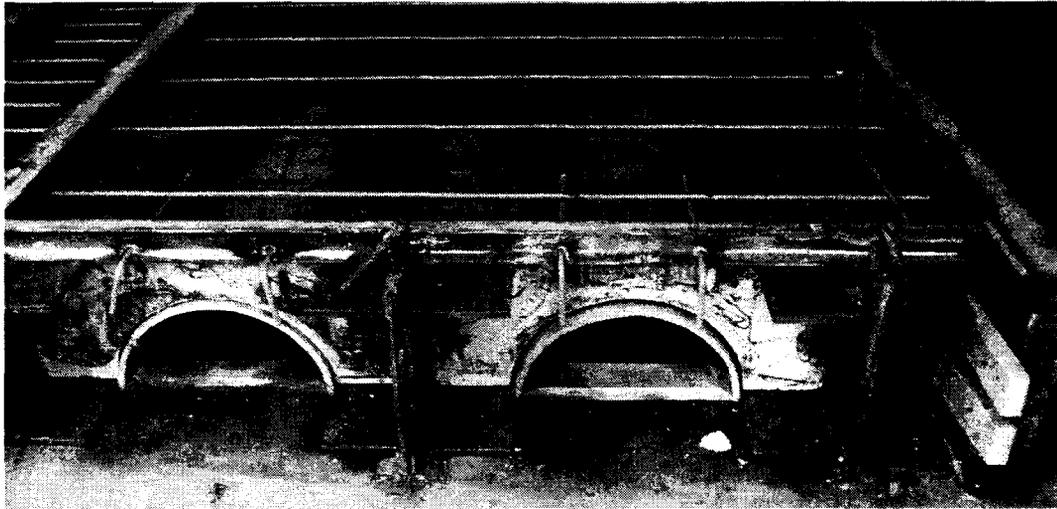
accommodate a half section of 18 in. diameter (ID) PVC pipe that forms the transverse arch between the girders. The pipe sections which are part of the casting bed are shown in Figure B1.1a. A small camber is introduced at the midspan of the panels by placing a 3/8 in. thick steel plate at the midspan and then anchoring the ends of the girders to the casting floor. By supporting the girders in this fashion, deflections due to the self weight are significantly reduced since the full composite section is available to resist the self weight and other loads.

Reinforcement, similar to that used in the MBISB design is placed in each panel. Transverse #4 Grade 60 reinforcing steel is passed through every fifth ASC hole and extends approximately 20 in. past the centerline of the exterior girders in each panel to provide development and a “tie” to adjacent panels. Since the PMBISB design incorporates a minimum 3 1/4 in. cover over the top flanges of the girders to reduce the potential for corrosion and increase the flexural rigidity, a layer of transverse #4 Grade 60 reinforcement, also on 15 in. centers, is placed over the top flanges to arrest temperature and shrinkage cracks. The concrete cover is linearly increased from 3 1/4 in. at the ends of the panels to 4 1/4 in. at the midspan to counter the self weight deflections. When combined with the previously described camber, a level driving surface results once the bridge is completed.

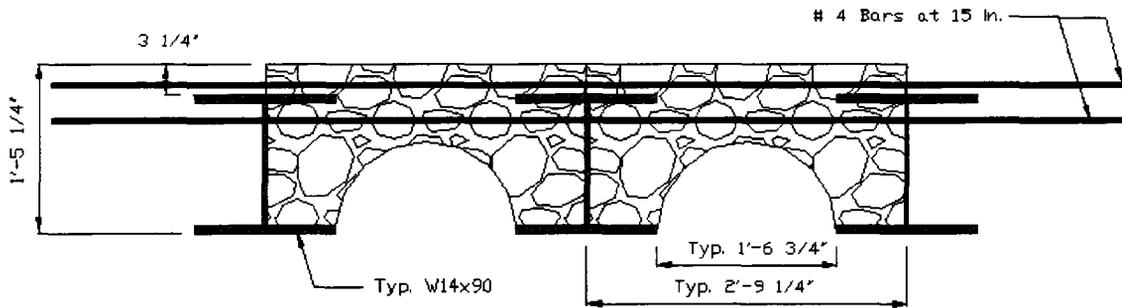
After the concrete is placed and cured, the completed 25 ton panels are placed in storage. The cross sections for typical interior and exterior panels are shown in Figures B1.1b and c.

### **On-Site Construction**

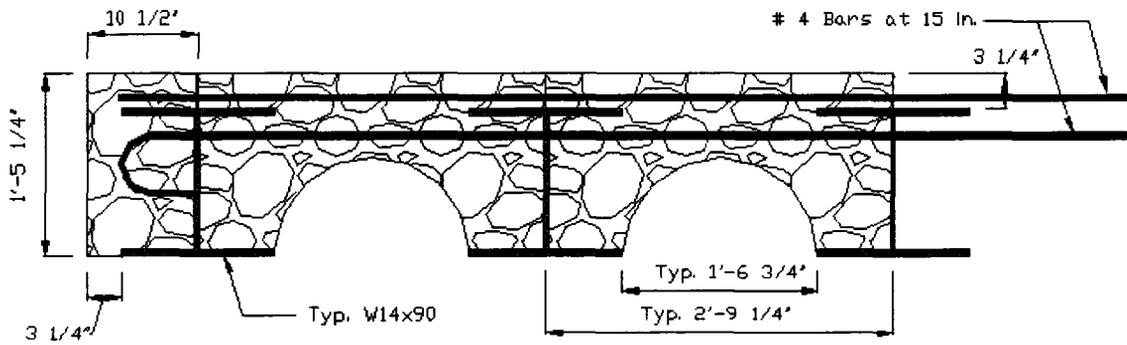
Since the PMBISB panels make up the superstructure of the bridge, the substructure must be completed prior to the placement of the panels. The first PMBISB structure is supported by steel piles combined with a sheet pile backwall and steel abutment cap that was sloped from the centerline of the abutment to introduce cross slope (see Figure B1.2a). The Blackhawk County Engineering Office has also developed a pre-cast backwall and abutment cap specifically for use with the PMBISB system; currently two such structures are in service with more being planned.



a. Transverse Arches between the Steel Beams

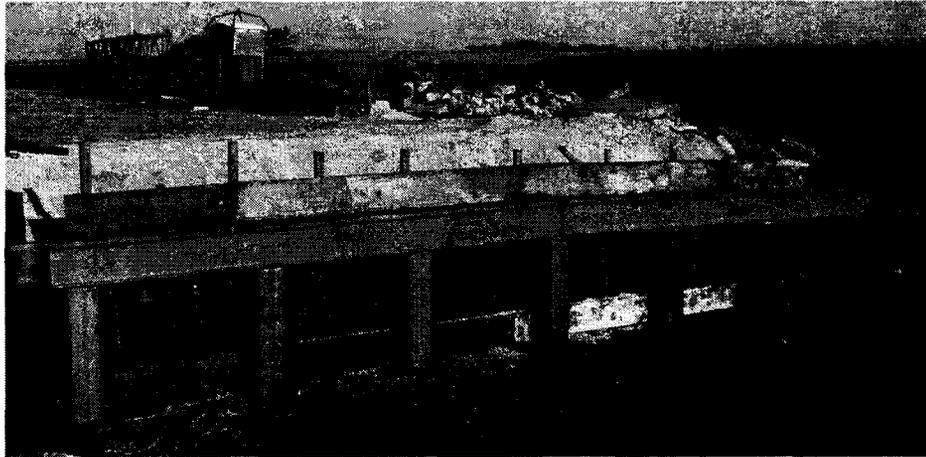


b. Typical interior panel

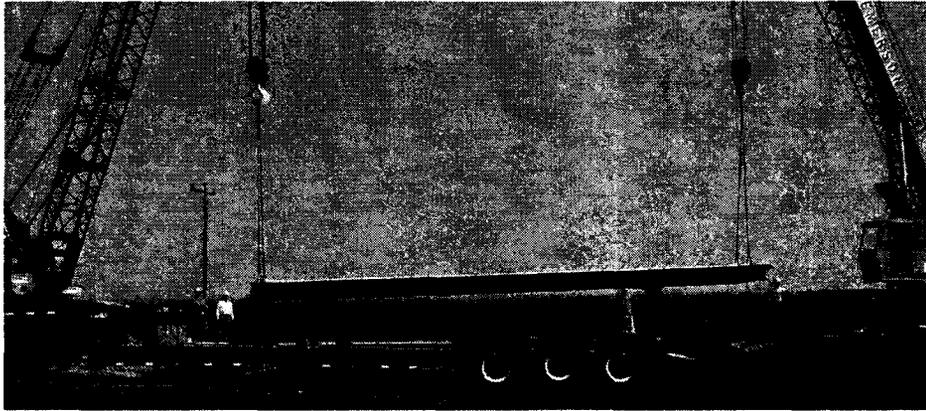


c. Typical exterior panel

Figure B1.1 Pre-cast panel construction and cross section.



a. Abutment and backwall for the first PMBISB

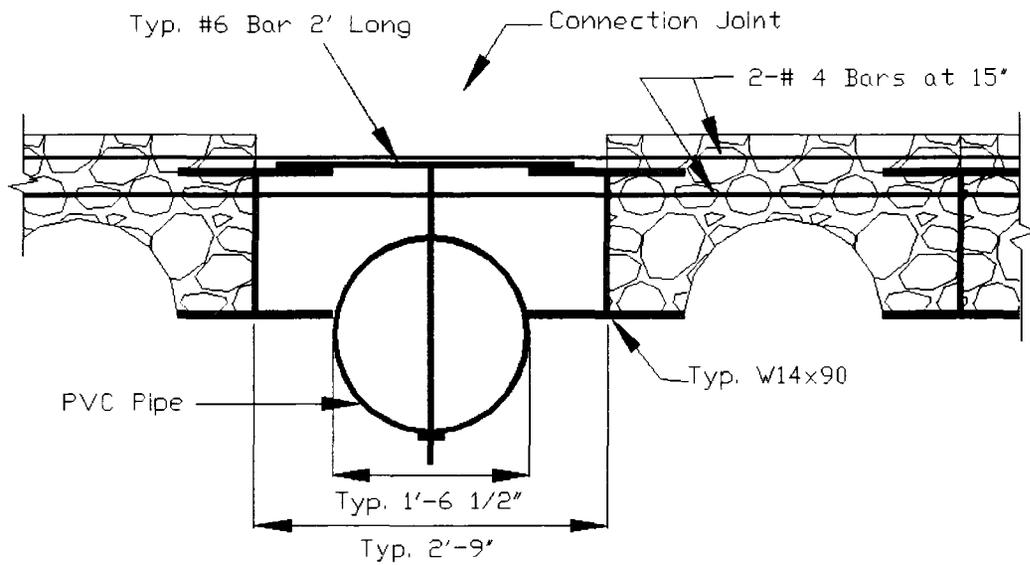


b. Placement of the pre-cast panels

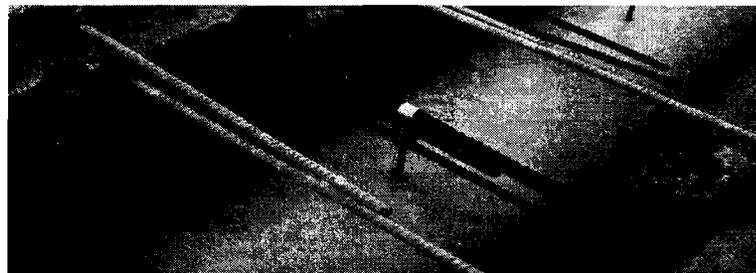
Figure B1.2 In-field placement of the pre-cast panels.

Once the substructure is completed, the PMBISB panels are loaded onto a flatbed truck and transported to the bridge site. Two cranes, as shown in Figure B1.2b, then set the panels in their final position leaving an 18 1/2 in. gap between each of the panels.

The gap between the panels is closed by a full section of 18 in. (ID) PVC pipe as shown in Figure B1.3a. The pipe is supported by a threaded rod connected to a hanger which in turn rests on a 2 ft long section of #6 reinforcing steel (see Figure B1.3b) that is supported by adjacent girders. A



a. Cross section of the in-field joint



b. Installed in-field formwork

Figure B1.3 In-field joint formwork.

backwall that includes a pavement notch is also formed at this stage of the construction. Concrete is placed in the gap between the adjacent panels and is struck off even with the top surface of the deck similar to the technique used in the construction of the original BISB system. After adequate curing, the formwork (PVC pipe) is removed, a guardrail system installed and the bridge is opened to traffic.

The implementation of the pre-cast system results in a considerable reduction of in-field construction time for the superstructure. The pre-cast system also allows for the construction of

additional panels in the traditional construction off-season (winter). When combined with the pre-cast substructure, the PMBISB system can be built in less than four weeks, reducing the inconvenience to the traveling public.

### B2. IN-FIELD EVALUATION OF THE PMBISB

Since the PMBISB system is similar to the MBISB design, ISU BEC researchers, at the request of the Blackhawk County Engineers Office, field tested the first PMBISB to evaluate its structural behavior. The bridge was tested to obtain additional data on the MBISB system, with particular attention given to the service level deflections and stresses. The behavior of the cast-in-place joint between the pre-cast panels was also of interest. The load tests were performed similarly to those completed for MBISB 1 and 2 with instrumentation placed at the abutments, quarter, mid and three quarter spans as shown in Figure B2.1 (2).

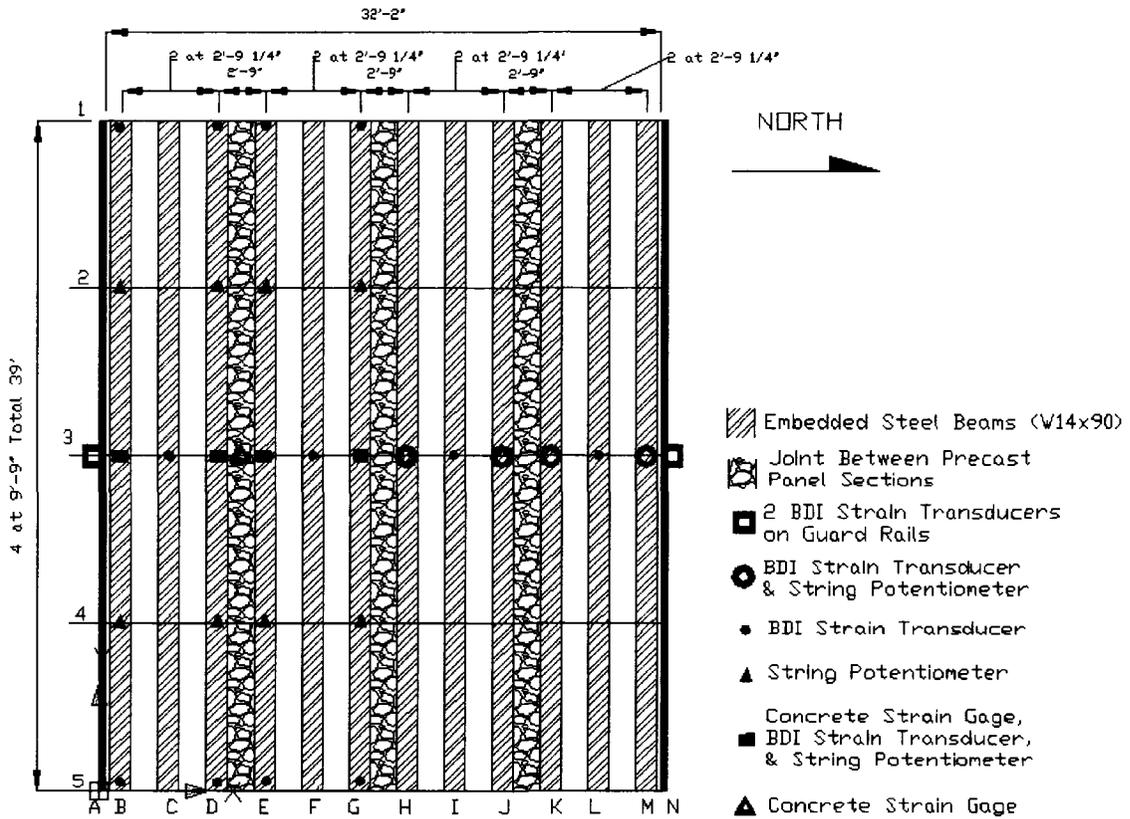


Figure B2.1 Plan view of the instrumentation layout.

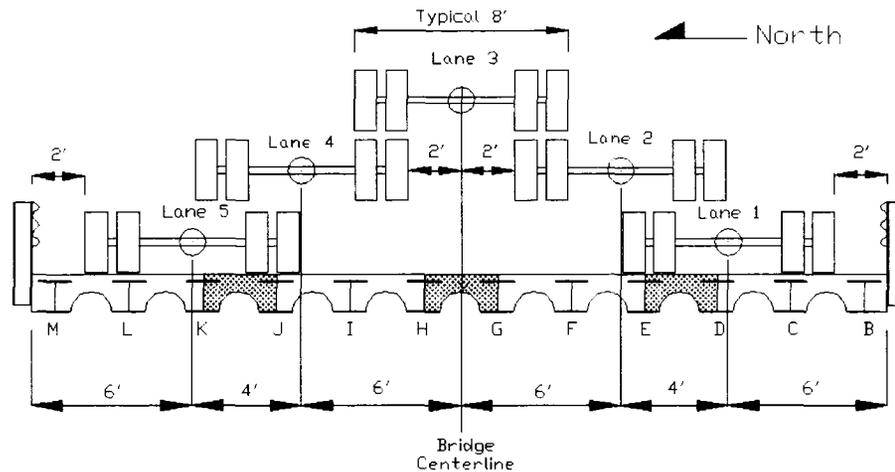


Figure B2.2 Layout of testing lanes.

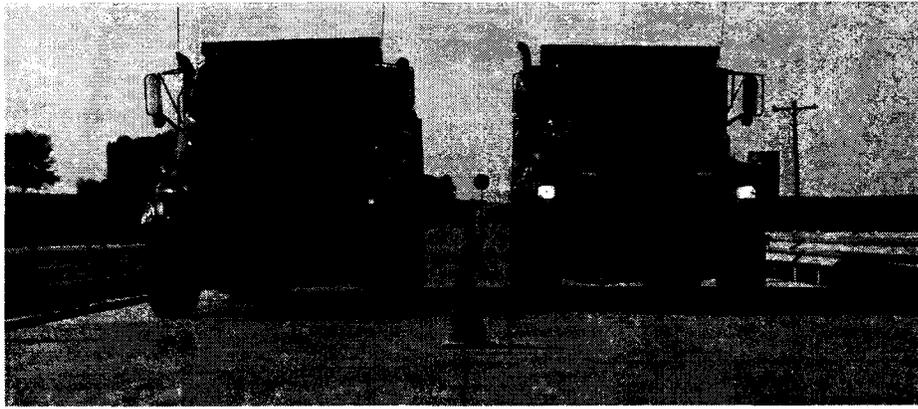
The bridge was divided into five lanes, as shown in Figure B2.2, to apply load at critical locations. Loaded tandem axle gravel trucks, supplied by Blackhawk County, were used to apply load to the PMBISB as described in the following sections.

### STATIC LOAD TESTS

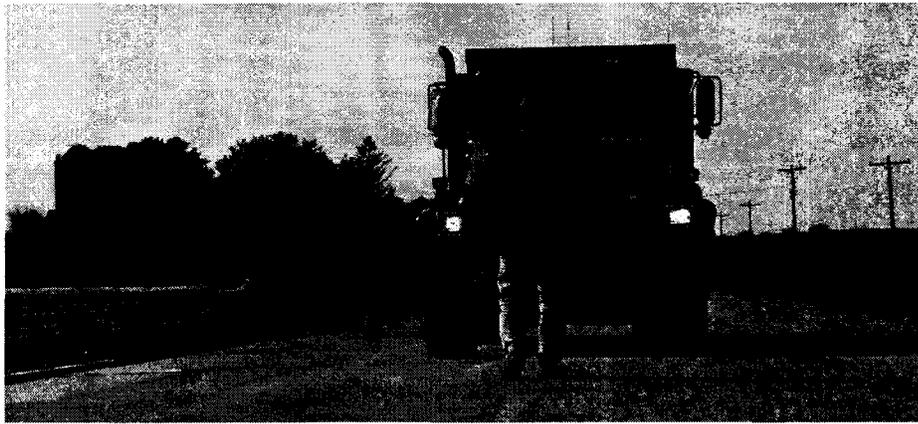
Following the procedures used in the testing of MBISB 1 and 2, two test vehicles were positioned on the bridge for the static load tests (2). In the first test, (Static Test 1) Lanes 2 and 4 were occupied, creating a maximum moment on the interior girders; Lanes 1 and 5 were occupied in the second static test (Static Test 2) which created a maximum load on the exterior members. In both tests, to create a maximum midspan moment, the trucks moved onto the bridge and stopped with the front axle of the tandem positioned at midspan. The test vehicles on the bridge for Static Test 1 are shown in Figure B2.3a.

### QUASI-STATIC (ROLLING) TESTS

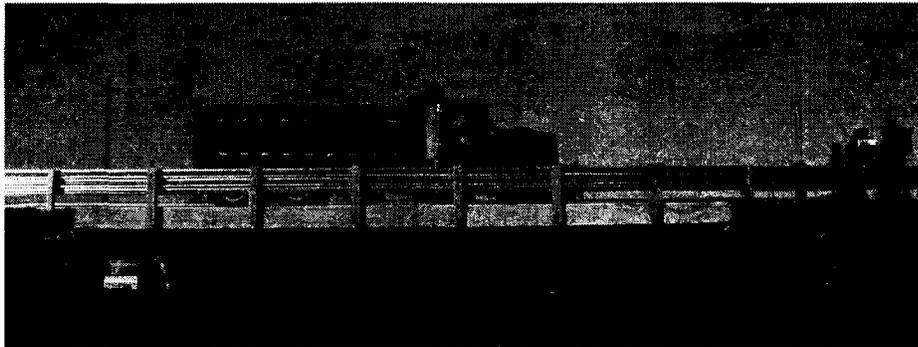
A total of 10 rolling tests, two per lane to check the reproducibility of the bridge's response and the data recorded, were performed; a single test vehicle in the selected lane crossed the bridge at approximately 2 mph while data were continuously recorded. The maximum load effect was taken to



a. Test vehicles in Lanes 2 and 4 for Static Test 1



b. Test vehicle in Lane 3



c. Profile view of a rolling test

Figure B2.3 Conducting the field test.

correspond with the time at which the front axle of the tandem was at the midspan of the bridge. A typical rolling test is shown in Figure B2.3b and c.

### **B3. FIELD TEST RESULTS**

The data recorded during the bridge test were analyzed to quantify the following behaviors: maximum midspan deflection, maximum flexural strains, rotational restraint at the abutments and lateral load distribution. Each of the investigated behaviors is explained in greater detail in the following sections.

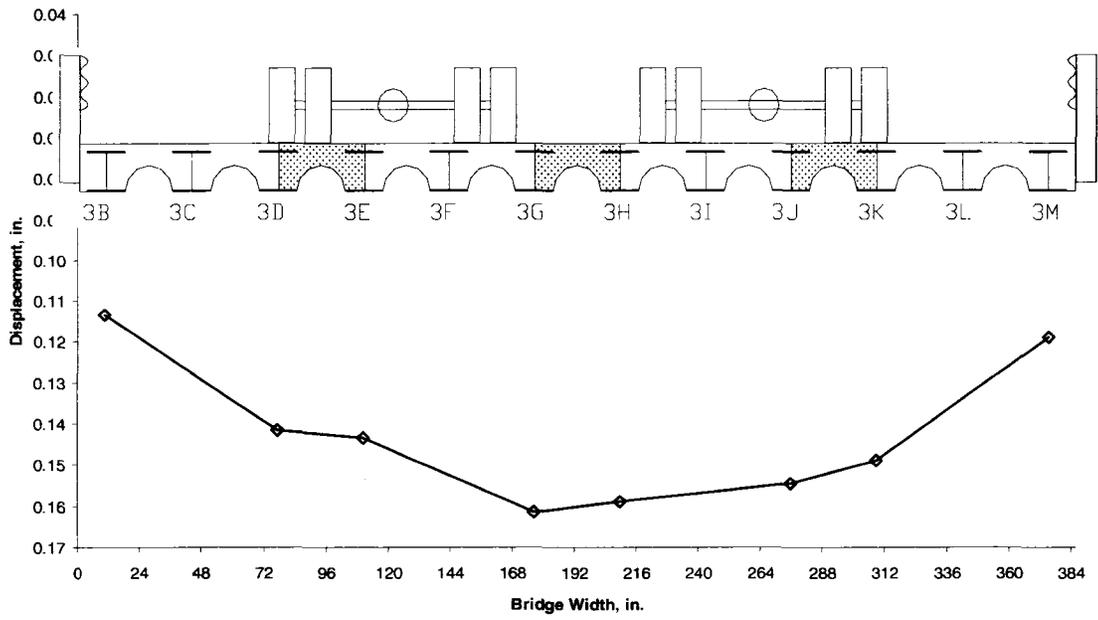
#### **MIDSPAN DEFLECTIONS**

The maximum midspan deflection profiles were plotted for each test; the absolute maximum recorded value of 0.17 in. occurred during the Static Test 1 when both vehicles were on the bridge. When compared to the suggested AASHTO LRFD Bridge Design Specification deflection limitation of  $L/800$  ( $= 0.59$  in. for the PMBISB), the maximum recorded deflection is 28% of the recommended limit, indicating the structure has adequate flexural stiffness (3).

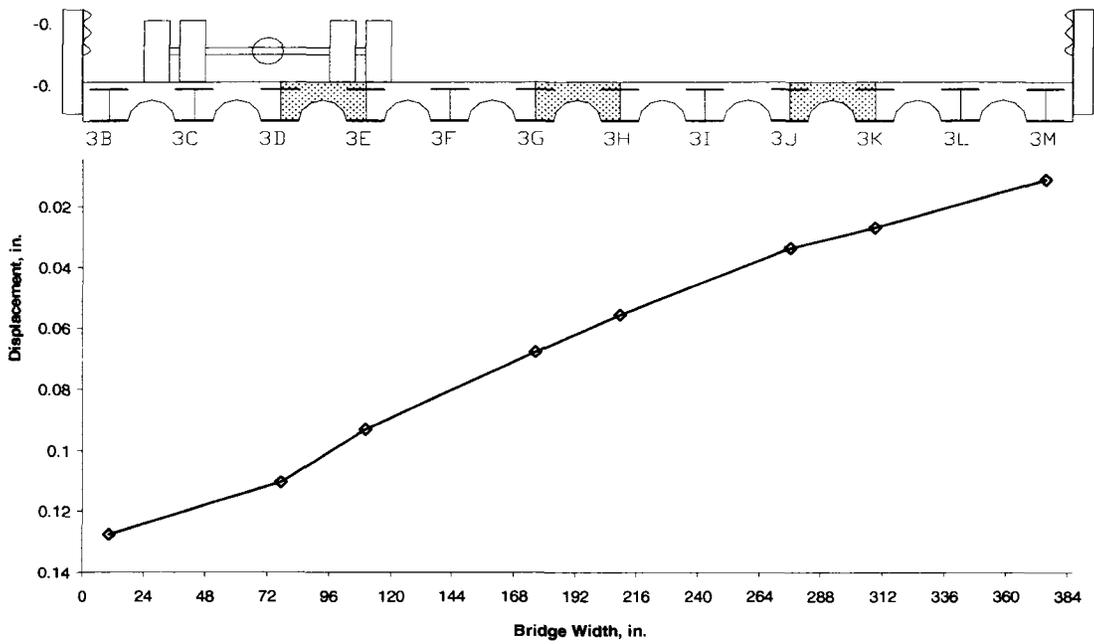
Continuity of the pre-cast panels at the joints was investigated by comparing the deflection on either side of the joint. As seen in Figure B3.1, a change in slope is present at the joints, indicating some loss in continuity; however, the change is small, and the joints are considered to provide an effective lateral transfer of the applied loads. Based on the deflected profiles, the PMBISB system distributes the loads across the whole bridge width; lateral load distribution in the system is discussed in greater detail.

#### **FLEXURAL STRAINS**

Strains were measured at the midspan to determine the maximum flexural strains and develop the lateral load distribution factors. The maximum tensile strain recorded in the steel beams was 106 microstrain (3.07 ksi tensile stress assuming  $E = 29,000$  ksi) which occurring in the exterior girders during Static Test 2. When combined with the calculated stress due to the self weight acting on the composite section, there is a maximum tensile stress of 7.3 ksi, which is approximately 15% of the girder yield stress (50 ksi).

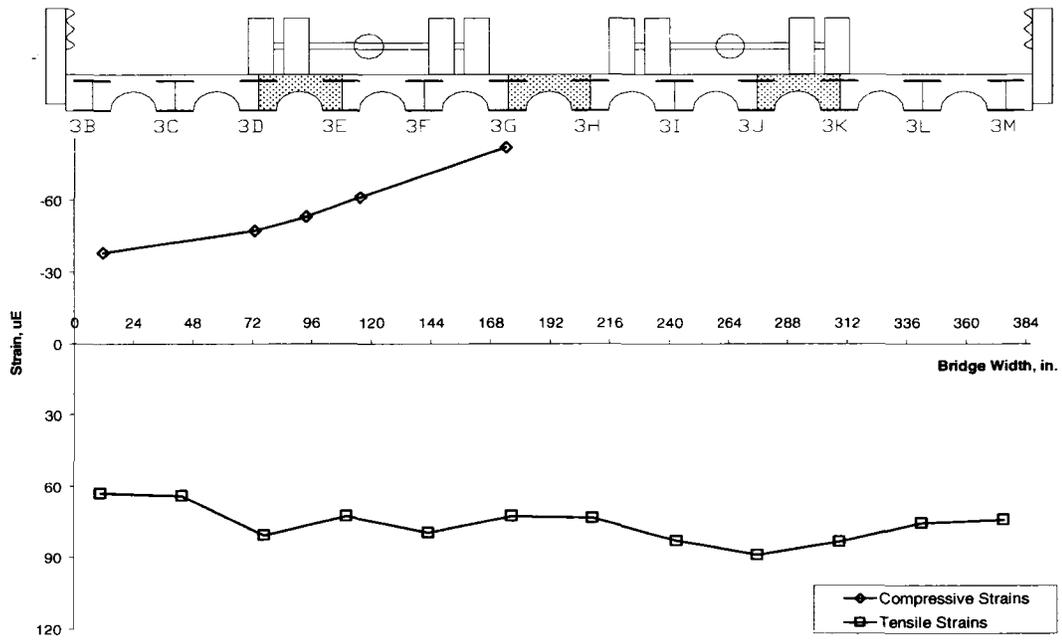


a. Static Test 1

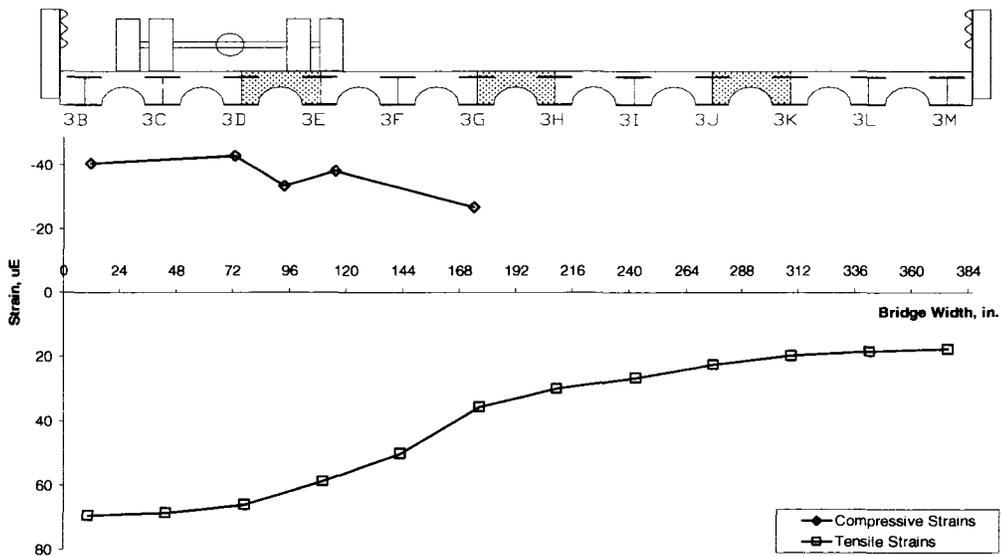


b. Lane 1 Test

Figure B3.1 Midspan deflection profiles.



a. Static Test 1



b. Lane 1 Test

Figure B3.2 Maximum midspan strain profiles.

The midspan strain profile for Static Test 1 and the Lane 1 rolling test, presented in Figure B3.2, indicates that the load is distributed across the entire width of the bridge. The

compressive strains measured on the top surface of the concrete deck were used in conjunction with the tensile strains to establish an experimental neutral axis and to confirm composite action was developed by the ASC.

Strain instrumentation, placed on the longitudinal girders near the abutments measured compressive strains with a magnitude of up to 20 microstrains indicating the presence of rotational resistance at the abutments. This restraint is attributed to the backwall connection shown in Figure B3.3 (Figure courtesy of the Blackhawk County Engineering Office). The presences of rotational restraint at the abutments increased the longitudinal flexural stiffness of the PMBISB system. Strains in the guardrails were also recorded and based on the measured values; the guardrails contribute to the total flexural resistance of the structure. The increase in the flexural rigidity due to the guardrail and the rotational restraint from the backwall, although contributing to the strength of the bridge, is a portion of the inherent stiffness and therefore not included in the design of such structures.

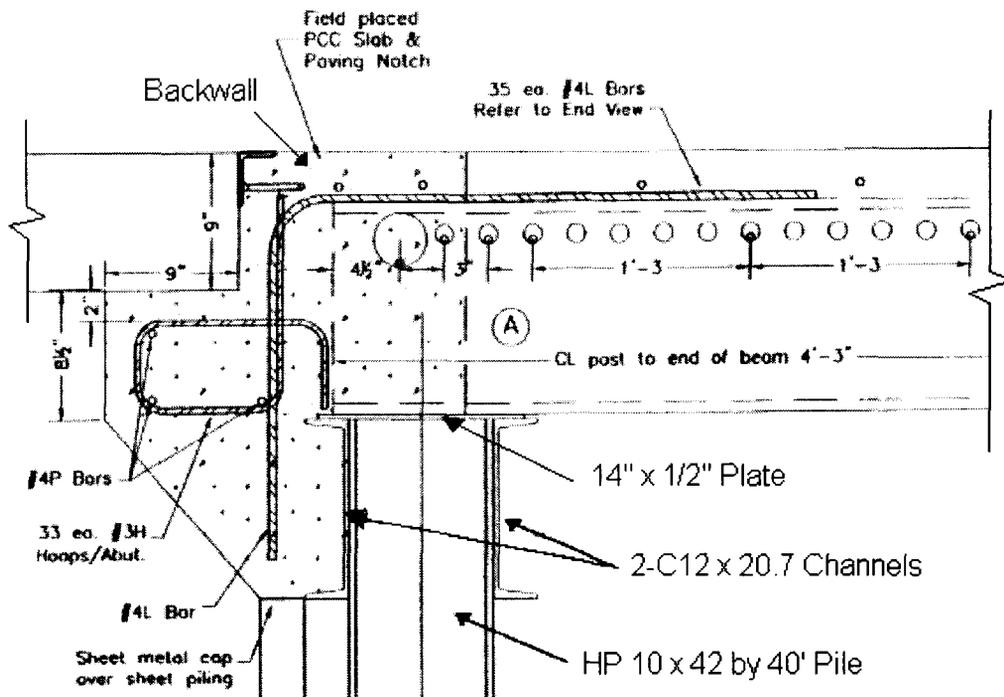


Figure B3.3 Profile view of the abutment and backwall detail.

## LATERAL LOAD DISTRIBUTION

The moments applied to the individual beam sections (the effective flange width was taken as the girder spacing) were calculated using the measured strain values and the flexural section properties. The fraction of the total applied moment carried by an individual beam section was determined by dividing the individual moment by the total moment. The resulting values are the lateral load distribution factors for the PMBISB. When the experimentally determined single lane loading values were adjusted for the Average Daily Truck Traffic (ADTT) and multi-presence factors as per AASHTO LRFD Bridge Specifications, the interior and exterior distribution factors are 15% and 16%, respectively (3). When compared to MBISB 1, which is similar to the evaluated PMBISB, the load distribution factors are approximately 4% larger. However, larger values are expected for the PMBISB since there are four fewer girders over which to distribute the loading.

The multiple lane experimental distribution factors, after adjusting for ADTT and multi-presence factors, are 20% for the exterior girder and 18% for the interior girders. These values are not directly comparable since multiple lane loadings were not performed on the exterior girders for MBISB 1. Based on the experimental load distribution factors, the exterior girders can be designed for 20% of the design vehicle moment which is considerably smaller than the 27% and 32% calculated using AASHTO LRFD Bridge Specifications. The conservative distribution factors were expected since the 33 in. girder spacing in the PMBISB is less than the lower bound AASHTO design spacing of 42 in. thus resulting in a quasi-beam/slab bridge behavior (3).

#### **B4. SUMMARY AND CONCLUSIONS**

The PMBISB system, developed by the Blackhawk County Engineers Office with input from the ISU BEC, is a derivative of the MBISB system. This system combines the ASC and the transverse arch of the MBISB with the efficiency of pre-cast concrete construction; three PMBISB are currently in service and more are planned. The system can be constructed by in-house forces and the panels can be fabricated in the off-season, fully utilizing available resources. Cost savings of approximately 20% have been realized when compared to conventional bridge replacement options. When combined with a pre-cast backwall and abutment system, also developed by the Blackhawk County Engineering Office, the PMBISB can be constructed in 4 weeks.

At the request of the Blackhawk County Engineer, the first PMBISB was field tested and its structural performance evaluated by the ISU BEC. The PMBISB exhibited more than adequate flexural stiffness by deflecting less than 30% of the recommended limit. Maximum tensile stresses in the longitudinal members, calculated from the measured strains, were 15% of yield, indicating more than sufficient strength in the PMBISB. Lateral load distribution factors were calculated based on the tensile strains at midspan for single (rolling tests) and multiple (static) lane loadings. The resulting distribution factors ranged from 16% to 20% respectively which are conservative when compared to AASHTO LRFD design values.

Overall, the PMBISB system provides for a reduction in cost and in-field construction time while exceeding strength and serviceability requirements. A more in depth report on the evaluation of the PMBISB system titled "Investigation of a Pre-cast Modified Beam-in-Slab Bridge System", was prepared by the ISU BEC and submitted to the Blackhawk County Engineering Office (4).

## **B5. ACKNOWLEDGEMENTS**

The investigation presented in this paper was conducted by the Bridge Engineering Center at Iowa State University through funding provided by the Iowa Department of Transportation, Highway Division and the Iowa Highway Research Board. The authors wish to thank the Blackhawk County, Iowa, County Engineers and Doug Wood, ISU Research Laboratory Manager, for their help with the construction, instrumentation, and testing of the specimens.

The opinions, findings, and conclusions expressed in this publication are those of the authors and not necessarily those of the Iowa Department of Transportation.

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