

Construction and Evaluation of the First Modified Beam-in-Slab Bridge

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ABSTRACT

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Funding for the repair and replacement of structurally deficient and functionally obsolete bridges is a nationwide problem and is magnified for managers of low volume road (LVR) systems with limited budgets; thus, innovative replacement alternatives are always being sought. The beam-in-slab-bridge (BISB) system, developed in Iowa, is one of the more successful designs. Two modifications, a reduction in self-weight and the incorporation of an alternative shear connector (ASC), were proposed to increase the efficiency of the system, making it applicable to longer spans. Laboratory testing provided evidence that the modifications increased the structural efficiency and were then incorporated in a field demonstration bridge referred to as Modified Beam-in-Slab Bridge #1 (MBISB#1).

MBISB#1 (L = 50 ft (15.24 m), W = 31ft (9.45 m)) was constructed using 16 W12x79 girders. The self-weight of the structure was reduced 22% by introducing a transverse arched section between the girders and the ASC was utilized to develop composite action. After being in service for 18 months the bridge was field-tested. Instrumentation including strain gages, strain transducers and potentiometers was applied at critical locations to develop the moment fractions and deflected profiles. Loads were applied to the structure with test vehicles to produce maximum effects at critical locations.

Analysis of the data gathered indicates a structure with excellent transverse load distribution. Results from the field test have been analyzed to corroborate the results predicted by theoretical and numerical analysis methods. Based on the data gathered, the MBISB system is an applicable alternative replacement design for use on low volume roads.

INTRODUCTION

One area of research undertaken by the Iowa State University Bridge Engineering Center, in conjunction with the Iowa Highway Research Board, has been the development of alternative replacement bridges for use on rural low volume roads, i.e. those with an average daily traffic (ADT) < 400. The developed designs are tailored to fulfill the needs of Iowa's county engineers who are responsible for a bridge population in excess of 19,900, with over 5,800 of those bridges being either structurally deficient or functionally obsolete (1). With more structures in need of repair or replacement than funds available to do so, some counties have turned to alternative designs as a solution. Alternative replacement designs focus on being of lower cost when compared to traditional systems and allow for in house construction by county forces, further stretching available resources.

One of the most successful alternative designs is the Beam-in-Slab Bridge (BISB) system developed in Benton County, Iowa over 25 years ago. Based on personal testimony from a former Benton County Assistant County Engineer, it is estimated that almost 100 such structures or derivatives of the original design are in service and more are being constructed (Lyle Brehm, former Assistant Benton County Engineer, unpublished data). The continued use of the system indicates that the BISB design meets the needs of county engineers for certain applications.

The original BISB design consisted of W12x79 girders spaced 24 in. (610 mm) on center spanning a pair of given abutments. Transverse confinement of the girders is provided by straps welded to the bottom flanges at the 1/4 points of the span. Placing plywood on the top of the bottom flanges creates a stay in place formwork "floor"; concrete is then placed between the girders and struck off even with the top flanges completing the BISB. It should be noted that no reinforcement is present in the original BISB design. This system has the advantage of being simple to design when compared to traditional systems and is readily constructible by county forces without the use of specialized equipment. A typical cross section of the original BISB system is presented in Figure 1 (a).

However, the span length of the original BISB system is limited to approximately 50 ft (15.24 m). For spans of greater distance both deflections and stresses become prohibitive due to the self-weight. In addition, the original design does not include provisions to develop composite action; the lack of full composite action was confirmed through field and laboratory testing (2). The girder depth and spacing is also limited, for an increase in either dimension results in an equivalent increase in self-weight.

MODIFICATIONS TO IMPROVE EFFICIENCY

Two modifications were proposed to increase both the structural efficiency and the applicability of the original design. Improvements in both categories are gained from a reduction in the self-weight by removing a majority of the ineffective concrete from the tension side of the neutral axis. This reduction was accomplished by placing an arched section between the girders, removing a significant portion of the ineffective concrete. The second modification involved developing composite action between the concrete filling the void space and the surrounding steel girders, increasing the flexural rigidity of the system upon the curing of the concrete.

The Alternative Shear Connector (ASC) was developed at Iowa State University as a means of gaining composite action without the use of traditional shear studs, which require the use of specialized equipment to install. In its final form, the ASC consists of 1 1/4 in. (32 mm) diameter holes that are either torched or drilled through the web of a girder. The holes are centered one diameter below the flange/web juncture and spaced on 3 in. (76.2 mm) centers along the length of the beam. When the plastic concrete flows through the holes and cures, a

shear dowel is formed, mechanically transferring shear forces due to flexure from the concrete to the girders. Reinforcing steel (#4 or #5 bar) is placed transversely through every fifth hole prior to the concrete placement providing transverse confinement for the dowels. (3). The ASC prior to concrete placement is presented in Figure 1 (b).

Laboratory Testing

A series of laboratory tests were conducted to evaluate the performance of the two modifications. Introducing a transverse arch between the girders reduced the self-weight allowing for the use of deeper sections and a wider girder spacing further increasing the efficiency of the structure. In addition, the transverse arch lent itself to a reduction in deck reinforcement. This reduction is contributed to the arching action present in the section similar to a steel free deck, a concept investigated and reported on by Canadian researchers (4). Their findings indicate that an adequately confined bridge deck resists wheel loads through an internal arching action as opposed to flexure. Adequately confined decks fail in punching shear at a load several times greater than the load that would produce a flexural failure.

Two 15 ft (4.57 m) long, single bay specimens using W21x62 girders in conjunction with the ASC were constructed to investigate the adequacy of the arched section under a single point load. Typical cross sections of the specimens are presented in Figure 2 (a) and Figure 2 (b). The specimens proved to be adequate, failing at levels of 3.4 and 5.8 times that of a factored wheel load. A third specimen, a 3 bay model bridge, (L = 31 ft (9.45 m), W = 20 ft (6.1 m)) was constructed using four W21x62 girders spaced at 6 ft (1.83 m) centers. The ASC was employed to develop composite action in the section and a transverse arch was formed in each of the bays. A photo of this specimen prior to the concrete placement is presented in Figure 2 (c).

The purpose of the third specimen was to evaluate the load distribution characteristics of the arched system and to determine the ultimate flexural capacity of such a bridge configuration. The specimen was first subjected to a series of 45 kip (200 kN) service level loads about the 1/4, mid and 3/4 span to evaluate the load distribution. The resulting moment fractions, based on the deflections of the individual girders were calculated to describe the amount each girder carried with respect to the total. Figure 3 (a) and Figure 3 (b) are plots of the moment fractions at the midspan based on the deflection caused by the point load positioned first at the center and then at the exterior girder. It can be observed that for the two extreme midspan load positions the maximum moment fraction for the interior girders is approximately 30 percent of the total effect. The moment fraction of the exterior girder ranges from approximately 30 percent for the interior load position, to approximately 60 percent for the exterior load position. The values presented are for the four-beam specimen and would be reduced in a field bridge due to a larger number of girders.

With the completion of the service level tests, the bridge model was subjected to an ultimate loading condition. The specimen resisted 302 kips (1,343 kN) of force and deflected 7.1 in. (180.3 mm) before the test was terminated due to equipment limitations. Though catastrophic failure did not occur, excessive yielding and cracking permanently deformed the model far beyond that of service level loadings and for all practical purposes the specimen had failed.

DESIGN AND CONSTRUCTION OF MBISB #1

The laboratory testing confirmed the applicability of the modifications and the first of two demonstration bridges was planned; this report focuses only on the first demonstration bridge. The new design, which included the modifications, is referred to as the modified beam-in-slab bridge (MBISB) system. The first structure, referred to as MBISB#1, incorporated both the ASC

to develop composite action and the transverse arched section to reduce the self-weight of the structure.

Constructed in Tama County, Iowa, MBISB#1 is 50 ft. (15.42 m) long and 31 ft. (9.45 m) wide and constructed from 16 W12x79 girders spaced on 24 in. (610 mm) centers. The self-weight of the section is reduced by 22% with the implementation of the transverse arch when compared to the plywood formwork of the original BISB system. The transverse arch was formed from a section of 24 in. (610 mm) diameter, 16 gage corrugated metal pipe (CMP). The dimensions of the arched section are given in Figure 4 (a) and a typical cross section of MBISB#1 is presented in Figure 4 (b). The dimensions of the arched section were developed to provide adequate cover and still allow the concrete to be in intimate contact with the bottom flange. Twenty-four in. (610 mm) diameter CMP was selected for the formwork because it best accommodated the previously described requirements.

Construction Sequence

Two abutments were constructed using 9 wood piles each, driven to refusal, and tied back to dead men anchors. A 10-gage sheet pile, stiffened with wooden whalers, forms the back wall behind the piles. This typical low volume road bridge abutment that supports MBISB#1 can be viewed in Figure 4 (c). A steel abutment cap constructed using C12x25 channels and a 1/4 x 12 in. (6.3 x 304 mm) plate is bolted into place on the top of the piles. Transverse crown is introduced in the deck by welding steel plates of successively increasing thickness to the abutment cap, increasing the girder elevation, peaking at the centerline of the bridge. Prior to the girders arriving onsite, county forces torched the holes in the girder webs to create the ASC. The girders were set in place and one end welded down; 1/4 x 2 in. (6.3 mm x 51 mm) straps were welded to the bottom flanges of the girders at the 1/5 span points to prevent lateral translation during the concrete placement.

The stay in place, arched formwork sections, cut from the CMP, were placed on the top of the bottom flanges and held in place by 1 in. (25.4 mm) angle clips tack welded to the top of the bottom flanges. Reinforcing steel (#4 bar) was run transversely through every fifth hole completing the ASC preparation. End forms were installed and the concrete was placed filling the voids between the girders. The concrete was placed in sequence to ensure the ASC was completed and then struck off even with the top flange. Upon curing, barrier rail posts, consisting of C8x11.5 channels, were welded to the exterior girders and a three beam barrier rail was attached. The resulting structure was very similar to the original BISB design except for the implementation of the two modifications.

Interior girder deflections due to the self-weight were measured with a surveying level and found to be within 8 percent of the 1.74 in. (44 mm) theoretical deflection. The deflection due to self-weight is equivalent to a ratio of $L/338$. The deflection due to self-weight would increase to 2.12 in. (54 mm) without the implementation of the transverse arch.

The final cost on the completed structure including both the substructure and superstructure was \$50.30/ft² (\$542/m²) of deck surface, saving an estimated \$20/ft² (\$215/m²) when compared to a conventional bridge structure.

EVALUATING MBISB #1

Quantifying the behavior of the bridge under service level conditions was desired to validate the applicability of the modifications in a field application. Thus, a field test was conducted to experimentally determine the structural behavior under service level loads. The following three main points were of interest: transverse moment distribution, deflected shape, and flexural

strains. A description of the points of interest and the method of obtaining them is explained below.

Transverse Moment Distribution (Moment Fraction)

The moment fraction, the portion of the total moment due to the applied loading carried by an individual girder, was desired to determine the amount of moment that each girder must resist. The moment fraction also describes the distribution of the loading transversely across the structure. The values obtained can be used to develop design guidelines for future structures. Moment fraction values can be calculated using either the flexural strains or deflections by taking the value for each individual girder and dividing by the total combined values at the cross section of interest.

Deflections

Deflections were measured at the 1/4 span, midspan, and 3/4 span. String potentiometers (Unimeasures) with a precision of 0.001 in. (0.025 mm) were used to measure the deflections. The measured deflections were used to develop both the transverse and longitudinal displaced shapes and in calculating the moment fractions. Longitudinal and transverse symmetry was established using both the 1/4 and 3/4 span deflections. Figure 5 provides a layout of both the strain and deflection instrumentation on the structure and the location of the test vehicles used on the bridge.

Flexural Strains

Longitudinal strains due to flexure were measured at the abutments, the 1/4 span and the midspan. Strains were measured with both traditional electrical resistance strain gages and strain transducers manufactured by Bridge Diagnostics Inc. (BDI). Strains were measured on both the top and bottom surface of the bridge at the midspan to establish the neutral axis as well as the moment fractions. Strains at the 1/4 span were measured to assist in establishing the neutral axis and symmetry in the structure both longitudinally and transversely. Strain gages were also installed on the barrier rails at the midspan to measure the flexural resistance provided. BDI gages were placed near the abutments to quantify the amount of restraint present due to a possible moment resisting connection between the girders and the abutments. The amount of restraint at the abutments will influence the strains and deflection occurring over the structure and must be known to accurately calibrate analytical models.

Instrumentation Installation

The instrumentation on the top surface of the bridge was limited to electrical resistance strain gages because the loading vehicles must be able to pass over the gages without damaging them. Steel plates, 3/8 in. (9.5 mm) thick, were placed over the strain gages on the deck surface for protection from the test vehicles. The BDI gages and the Unimeasure instruments were installed on the underside of the bridge to measure the flexural strains and deflection, respectively. All instrumentation was installed following the manufactures prescribed procedures. Once the instrumentation was completed, the bridge was subjected to loading from the testing vehicles. The vehicles used to apply load to MBISB#1 consisted of two tandem axle county trucks; the axle weights and truck dimensions are given in Figure 6 (a.).

Lane Assignments

Five loading lanes were identified on the bridge to maximize different load effects; the lane layout is presented in Figure 5. The purpose of Lanes 1 and 5 were to evaluate the effect of an edge loading on the bridge and check transverse symmetry across the structure. Lanes 2 and 4 were also used to check symmetry and the maximum effect of a loading in the normal driving lanes. The center lane, Lane 3, provides for a symmetric loading condition and a maximum load effect on the interior girders.

Testing Sequence

The bridge was subjected to one static load case and 5 rolling load cases of 2 sequences each. The static load case consisted of having both test vehicles on the bridge with Truck 927 occupying Lane 2 and Truck 929 occupying Lane 4. The trucks progressed onto the bridge so the front axle of the tandem rested at the midspan and the effects were recorded. This load location was chosen because it was simple to position the test vehicles and kept the center of gravity of the trucks closer to the midspan. Once the static load case was completed, only Truck 927 was used in the rolling load cases, starting in Lane 1 and proceeding through Lane 5. The rolling tests involved lining the vehicle up with the lane of interest and stopping it within 15 ft (4.58 m) of the abutment. Once the data acquisition equipment was engaged, the truck would slowly roll across the bridge and off the opposite end completing the test. This rolling test sequence was repeated twice for each of the five load cases to ensure the proper gathering of data. Figure 6 (b) and Figure 6 (c) are photographs of the test vehicles in the static test position and in a rolling test on Lane 4, respectively.

EXPERIMENTAL RESULTS

The experimental data was processed to investigate the previously stated points of interest, namely the moment fractions and deflections. To investigate the uniformity between the two sequenced lane loadings, each of the rolling tests was compared and found to be virtually the same. A typical comparison of the resulting maximum midspan deflection for the two loading sequences is presented in Figure 7 (a) for the Lane 3 load case indicating the consistency between the two tests. Thus, the second loading sequence was used for all 5 rolling load cases.

Moment Fractions

The moment fractions were developed from the midspan flexural strains, using the maximum load effect at the midspan. The maximum moment fraction for all load cases, approximately 11.5% occurring in Girder 2, results from the Lane 1 load case. The moment fraction for both the Lane 1 and Lane 5 load cases can be viewed in Figure 7 (b); the transverse symmetry of the structure can also be observed in Figure 7 (b) by comparing the moment fractions. Transverse symmetry is also evident based on the resulting moment fractions for the Lane 3 and the Static load cases as presented in Figure 7 (c). Also evident in Figure 7 (c), is the resulting moment fraction of the Lane 3 load case remains similar that of the Static load case even though approximately twice as much moment is applied in the latter load case. This result helps to establish that MBISB#1 has excellent transverse load distribution for a variety of loading conditions. The resulting moment fractions can also be used to calculate theoretical deflections and size girders for future design.

Deflections

The midspan deflections that occurred when the front tandem axle of the test vehicle was located at the midspan were plotted for each load case. Deflections were calculated at this load location to provide consistency with the static load case position. The resulting transverse deflected shape takes the same general shape as the resulting moment fraction, which is based on the flexural strain data for the investigated load cases. A maximum deflection of 0.73 in. (18.54 mm) occurred at Girder 8 during the static load case. Figure 8 (a) is a typical representation of the midspan deflections resulting from the Lane 1 load case.

Theoretical Deflections

The midspan deflection values obtained for each load case were compared to theoretical deflections in the following manner. The deflection of a single simply supported 49 ft (14.94 m) long beam was calculated for the given longitudinal load position. A length of 49 ft (14.94 m) was used to account for the width of the abutment caps. The transformed gross moment of inertia was calculated for both the interior and exterior tributary section for a single beam. Rectangles and triangles were used to approximate the arched section to simplify the calculation of the section properties. The transformed section properties were calculated using a modular ratio (n) of 7, resulting in a neutral axis depth of 5.22 in. (133 mm) from the top surface for the interior girders corresponding to a gross moment of inertia of 846 in.⁴ ($352 \times 10^6 \text{ mm}^4$) assuming a fully composite elastic section. The gross transformed moment of inertia for the exterior girders was reduced to 790 in.⁴ ($329 \times 10^6 \text{ mm}^4$) due to the reduced cross section.

Justification for using the gross moment of inertia was based upon the flexural strain values present on the tension side of the neutral axis. The maximum tensile strain for all load cases was approximately 155 $\mu\epsilon$, occurring during the static load test. This strain value is only 5% greater than the estimated strain necessary to crack the section based on a linear strain profile and the modulus of rupture of 5,500 psi (37,921 kN/m²) concrete. The resulting deflection was then reduced by the respective moment fraction developed from the strain readings resulting in a theoretical deflection of each individual equivalent beam.

In all cases, the theoretical deflection was larger than the deflection measured during the field test. Similar to Figure 8 (a), Figure 8 (b), a plot of the experimental and theoretical deflections for the Lane 2 loading, indicates a greater flexural stiffness present than that of a simply supported beam subjected to the test vehicle loading. It is hypothesized that end restraint and an increase in the flexural resistance due to the barrier rail system is reason for the smaller experimental deflection values.

Preliminary Analytical Analysis

A simply supported grillage model consisting of 16 longitudinal beam lines and 49 transverse lines of tapered beam elements was constructed using an ANSYS finite element software package. The longitudinal elements were assigned the same flexural values as previously stated and torsional moments of inertia equal to 293 in.⁴ ($122 \times 10^6 \text{ mm}^4$) for the interior girder and 120.5 in.⁴ ($50 \times 10^6 \text{ mm}^4$) for the exterior girder was also assigned. The torsional moments of inertia were based on rectangular sections 6.75 in. (26.57 mm) in depth and the tributary width wide. The flexural and torsional properties of the tapered transverse members were calculated based on the cross section at each end of the element. Tapered elements were used to better model the arched section. Equivalent point loads equal to the axle loadings of the respective test vehicles were assigned to the longitudinal beams. The method of grillage analysis was based on theory presented by Barker and Puckett (5).

Each load case was analyzed and the resulting deflections were compared to the values determined from the field tests and the theoretical deflection calculated by the moment fractions. It was found in all cases the grillage model predicted deflections similar to those found by the theoretical calculations, thus overestimating the deflection when compared to the field test values. Typical results are presented in Figure 8 (a) and Figure 8 (b).

Two possible explanations for the larger predicted deflections are the presence of end restraint and an increase in flexural rigidity of the exterior members due to the barrier rail system. Since the analysis is a work in progress, the amount of end restraint and the influence of the barrier rails have yet to be fully investigated. Further results will become available as the analysis is extended and refined.

Composite Action

The ASC was implemented in MBISB#1 to develop composite action within the cross section resulting in an increase of the flexural rigidity equal to 1.28 times when compared to the individual steel section. Due to the arched section, the neutral axis will be raised when compared to the non-composite section. The experimental neutral axis was determined by evaluating the strains on the top and bottom surfaces of the bridge. By plotting the strains at the correct section depth the experimental neutral axis can be determined. Figure 9 is a typical strain profile for an interior section at the midspan for the Lane 1 loading with the truck located at the previously described position. The theoretical depth of the neutral axis for the composite section is under estimated by approximately 5%. Though error is present, the position of the experimental neutral axis provides evidence that the ASC is developing composite action and increasing the flexural rigidity of the section.

CONCLUSIONS

Due to a large number of structurally deficient and functionally obsolete bridge structures in Iowa, the Iowa State University Bridge Engineering Center has been conducting research to develop alternative replacement designs specifically for low volume roads. The original BISB system is an example of a successful alternative design. Its applicability is increased by introducing composite action and reducing the self-weight of the structure thereby increasing the structural efficiency.

The resulting design, referred to as the MBISB system, was investigated in the laboratory and the strength and applicability of the system was confirmed. A field demonstration bridge, MBISB#1, was constructed and field-tested to evaluate the performance of the MBISB design under service level conditions. Data from the field test has been reduced to investigate the moment fractions, deflections and flexural strains present in the structure.

Using the flexural strain values, the moment fractions were calculated and the bridge was found to have excellent lateral moment distribution. The measured deflections were found to be smaller than the deflections arrived at by the theoretically calculated values based on the moment fraction and the preliminary grillage analysis results. It is hypothesized that end restraint and an increase in flexural rigidity due to the barrier rails is reducing the actual displacement; these two effects will be investigated as possible explanations as the analysis progresses. Based on flexural strain values and the position of the experimental neutral axis, it was determined that the ASC was developing composite action in the section.

The results indicate the modifications were successful in increasing the efficiency of the system thus widening the range of application for county engineers who choose to implement the MBISB system as an alternative replacement design.

ACKNOWLEDGEMENTS

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

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LIST OF FIGURES

FIGURE 1 Concepts previously investigated at Iowa State University.

FIGURE 2 Experimental Specimens

FIGURE 3 Midspan Moment Fractions for the Model Bridge

FIGURE 4 Construction Details of MBISB#1

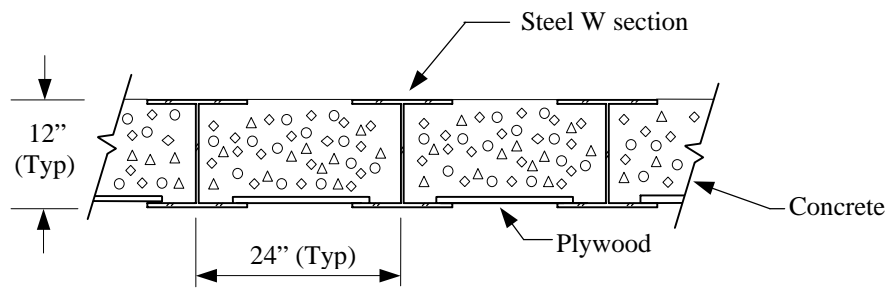
FIGURE 5 Layout of Instrumentation and Location of Lanes

FIGURE 6 Test Vehicles Used On MBISB#1

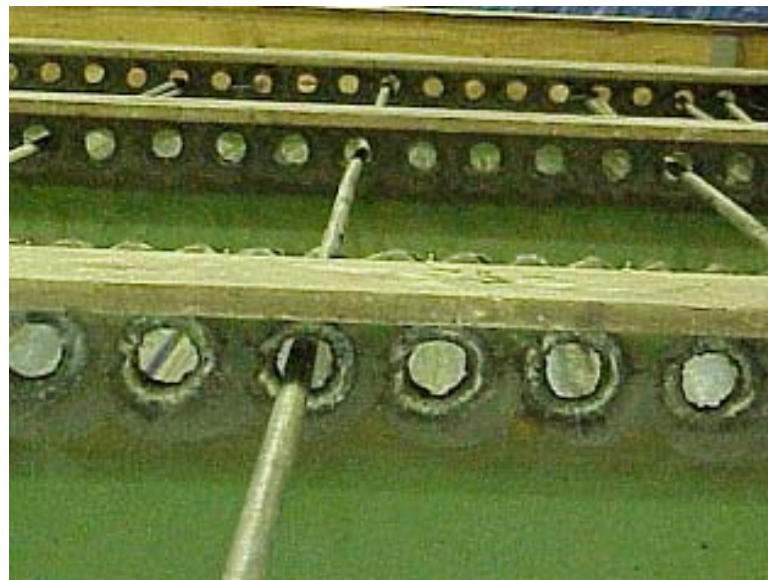
FIGURE 7 MBISB#1 Behavior

FIGURE 8 Typical MBISB#1 Midspan Deflection Profiles

FIGURE 9 Experimental Neutral Axis for Girder Nine

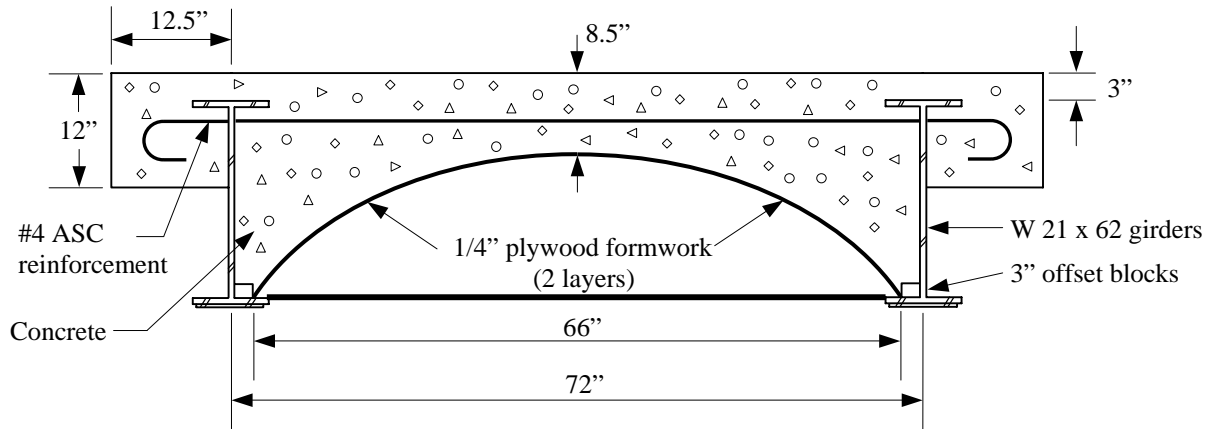


(a) Typical section of the original BISB

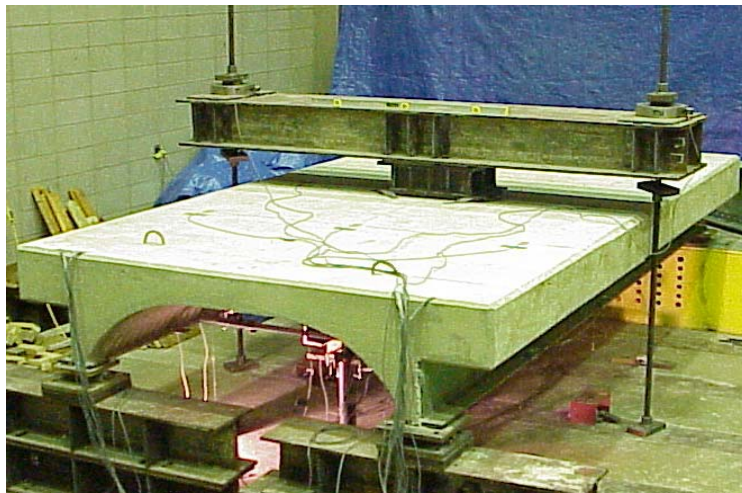


(b) Final design of the Alternative Shear Connector (ASC)

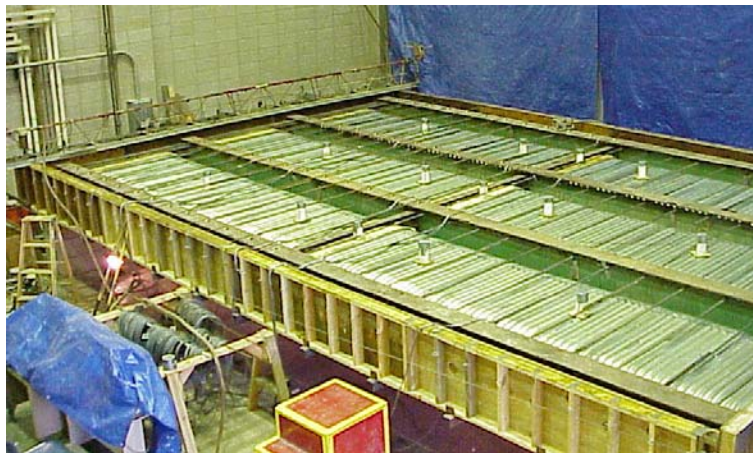
FIGURE 1 Concepts previously investigated at Iowa State University.



(a) Typical cross section for the first arched specimen

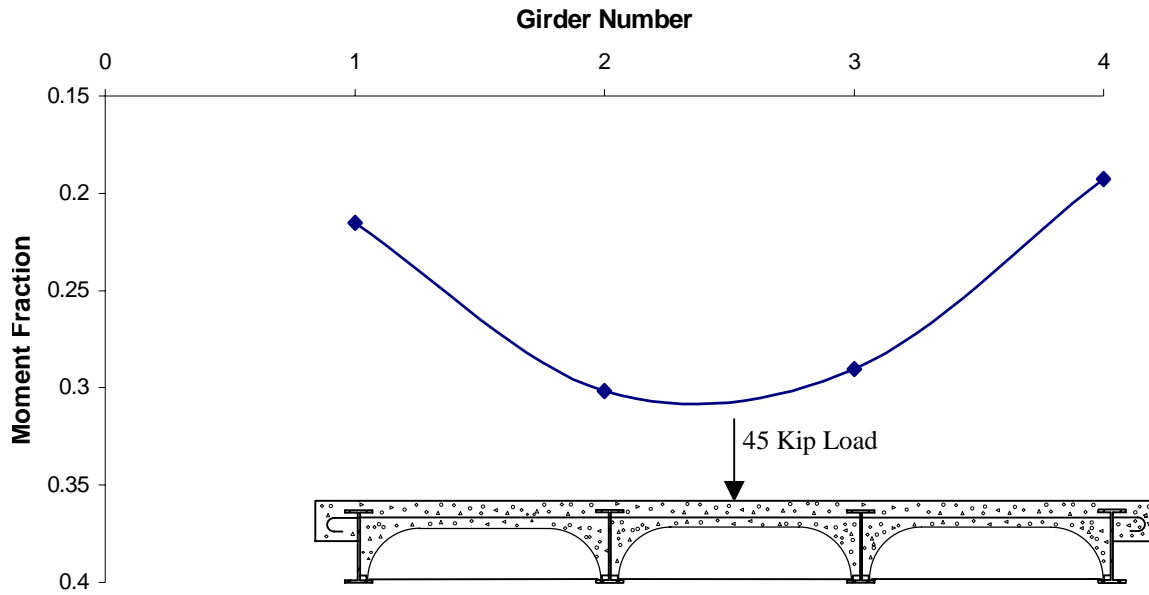


(b) Second arched section under ultimate load setup

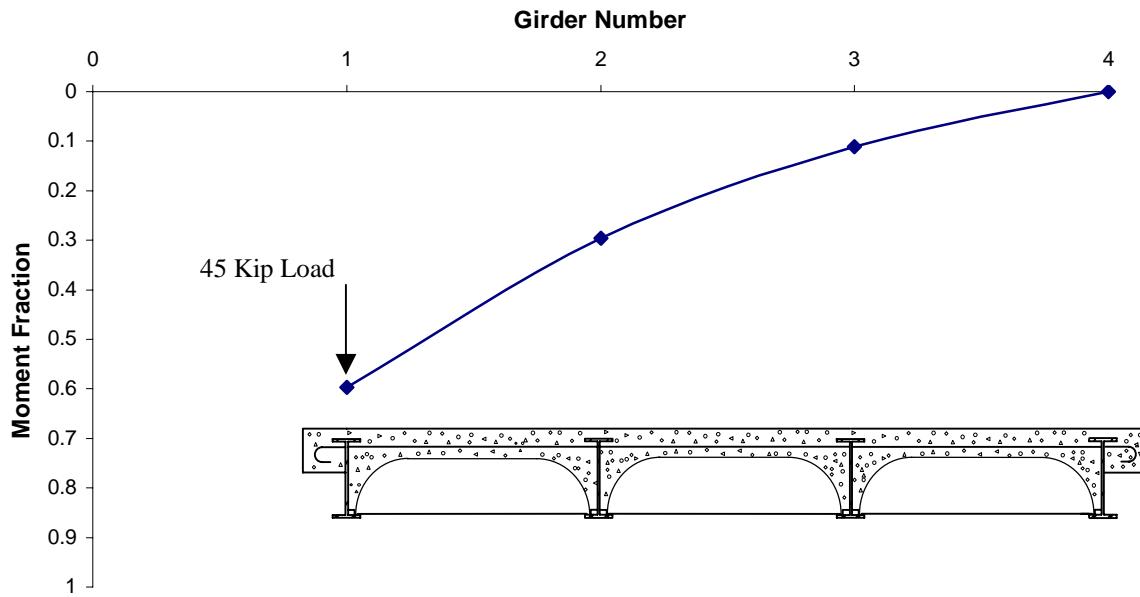


(c) Third experimental specimen prior to concrete placement

FIGURE 2 Experimental Specimens

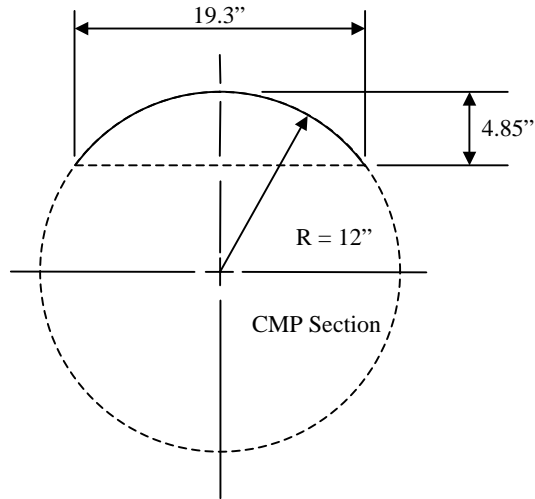


(a) Centerline loading

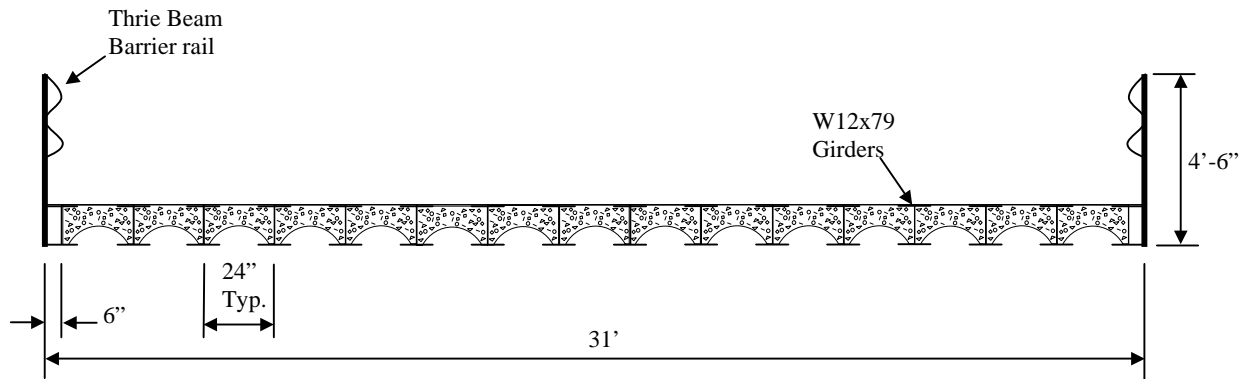


(b) Exterior girder loading

FIGURE 3 Midspan Moment Fractions for the Model Bridge



(a) Section of 16 gauge Corrugated Metal Pipe (CMP) used for formwork

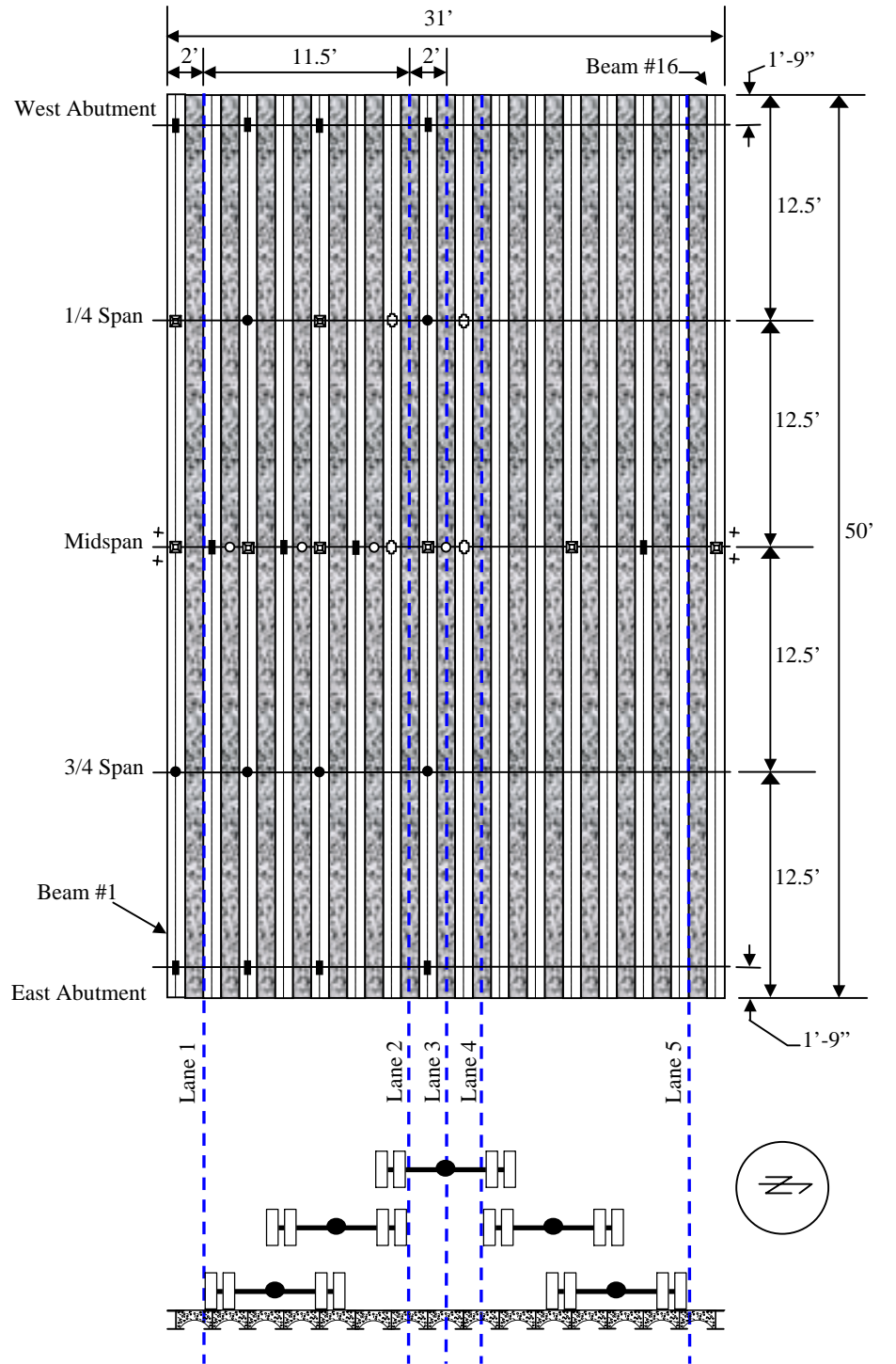


(b) Typical cross section



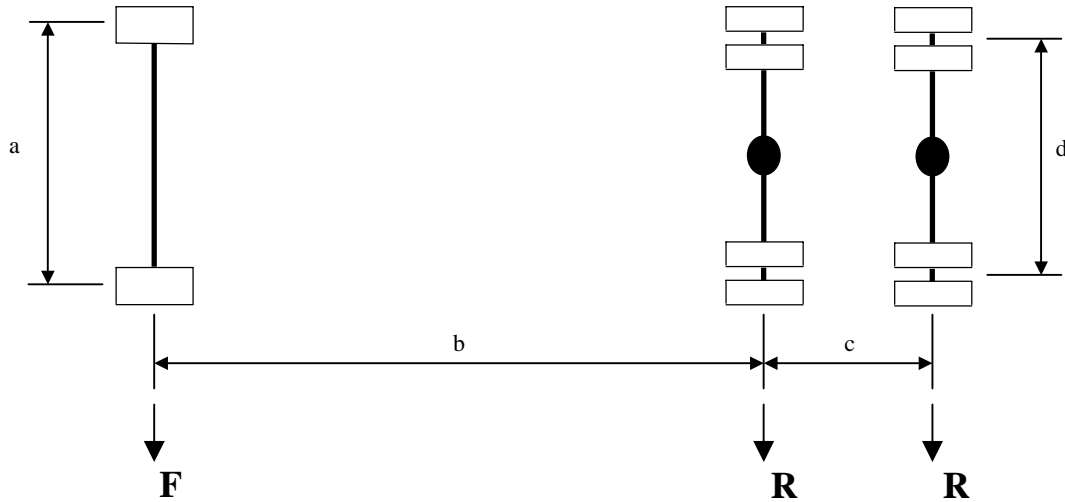
(c) Abutment under construction

FIGURE 4 Construction Details of MBISB#1



- Legend**
- ⊕ Guard Rail Strain Gage
 - Unimeasure Displacement Transducer
 - BDI Strain Transducer
 - Steel Strain Gage and BDI
 - Concrete Strain Gage
 - ▣ BDI, Steel Strain Gage and Unimeasure

FIGURE 5 Layout of Instrumentation and Location of Lanes



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

(a) Wheel and load configuration for the test vehicles

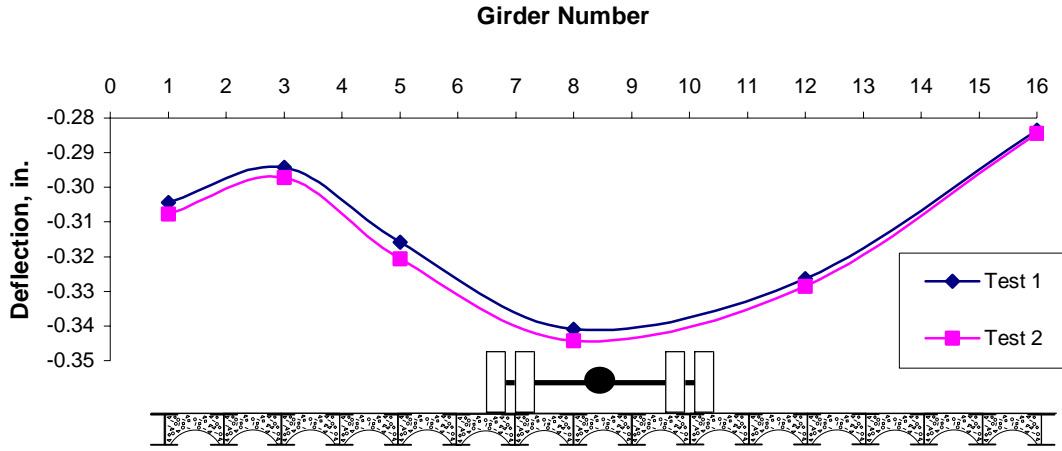


(b) Static loading

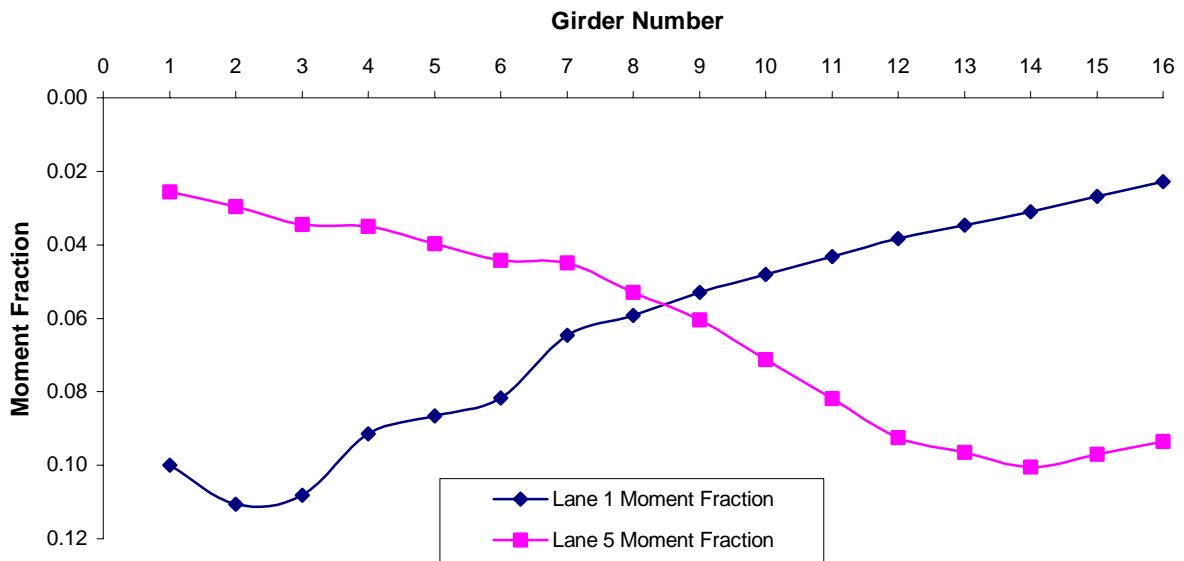


(c) Lane 4 load loading

FIGURE 6 Test Vehicles Used On MBISB#1

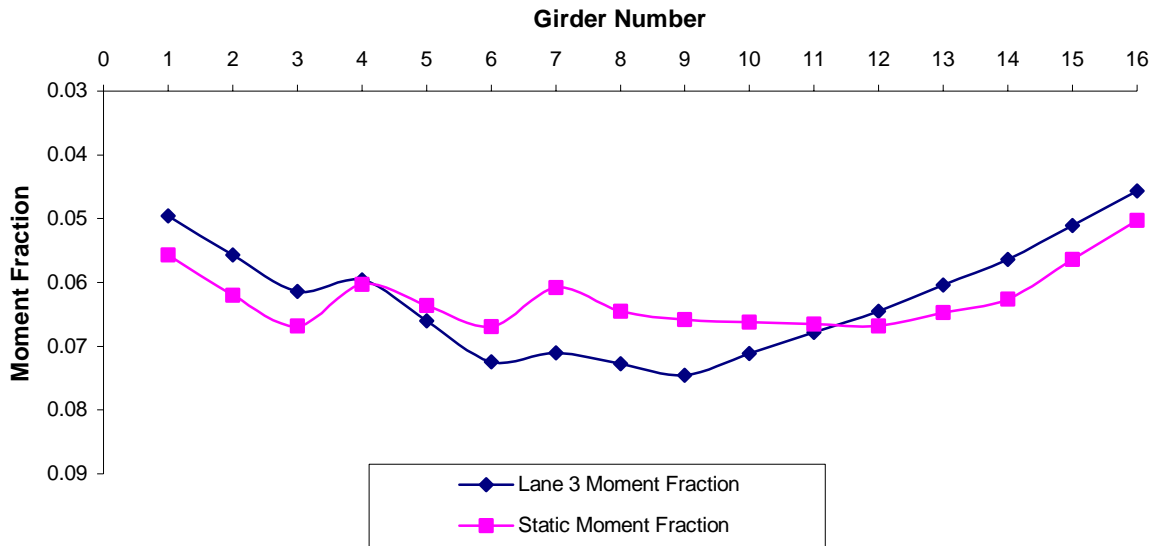


(a) Comparison of Test 1 and Test 2 centerline deflections (Lane 3 loading)



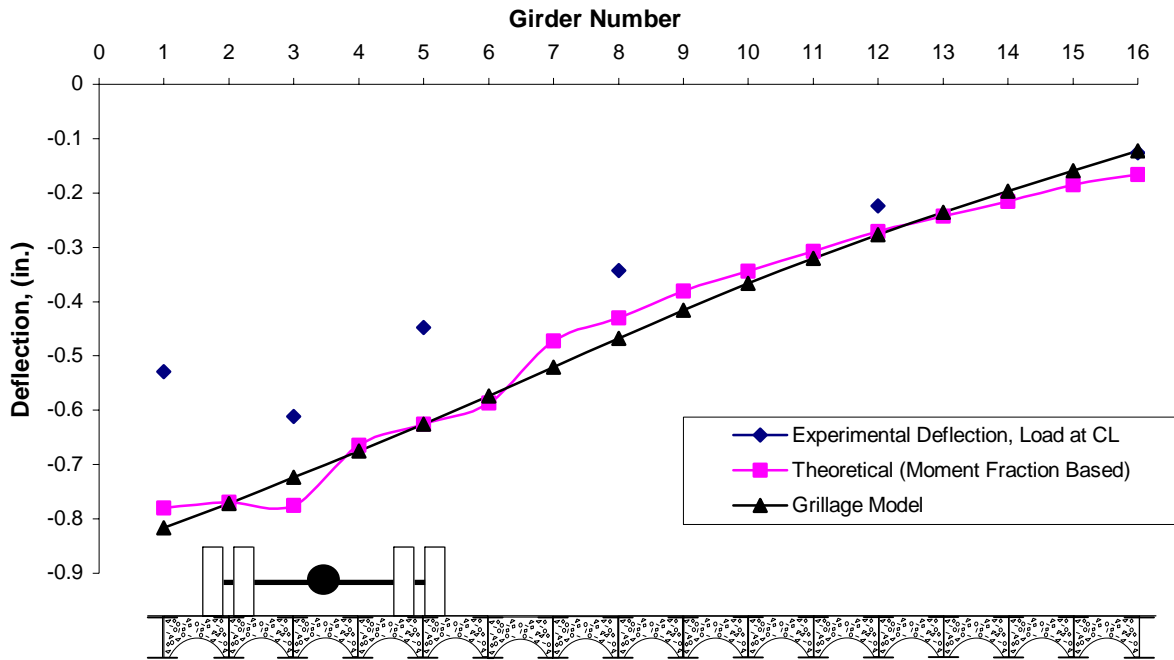
(b) Comparison of Load Case 1 and Load Case 5 midspan moment fractions

FIGURE 7 MBISB#1 Behavior

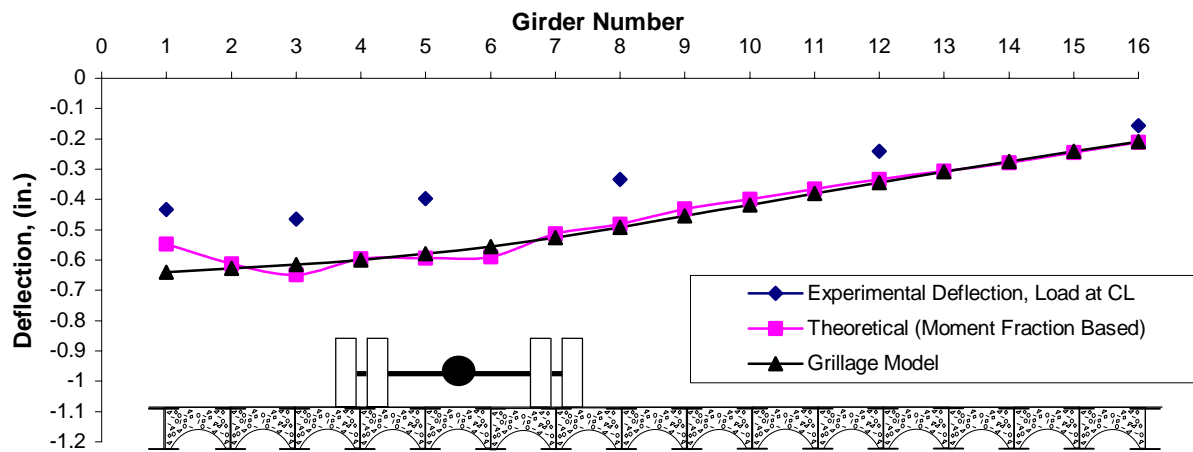


(c) Comparison of Load Case 3 and Static Load Case midspan moment fractions

FIGURE 7 Continued



(a) Midspan deflection for the Lane 1 load case



(b) Midspan deflection for the Lane 2 load case

FIGURE 8 Typical MBISB#1 Midspan Deflection Profiles

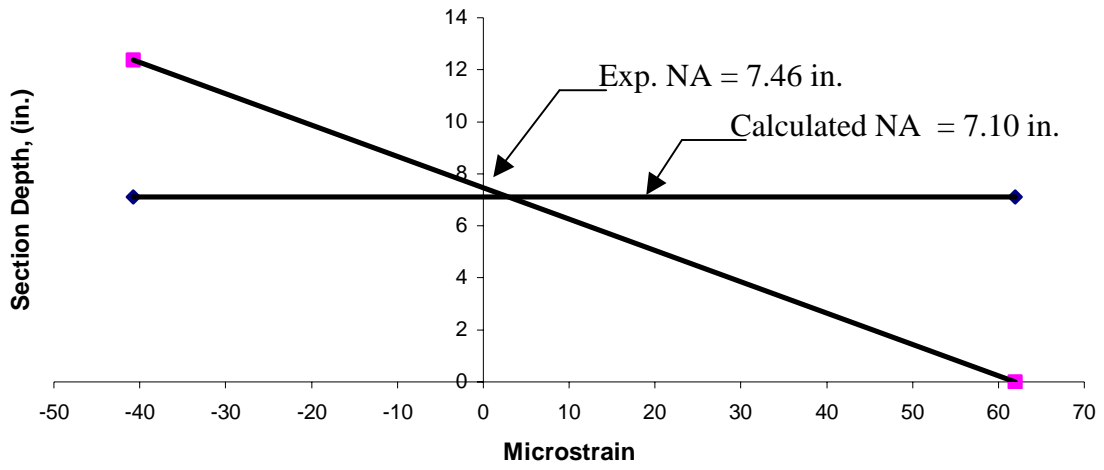


FIGURE 9 Experimental Neutral Axis for Girder Nine