

# Laboratory and Field Testing of Precast Bridge Elements Used for Accelerated Construction

Vernon W. Wineland  
Iowa State University  
176 Town Engineering Building  
Ames, IA 50011  
vernonw@iastate.edu

F. Wayne Klaiber  
Iowa State University  
176 Town Engineering Building  
Ames, IA 50011  
vernonw@iastate.edu

Thomas P. Schoellen  
Black Hawk County Courthouse  
316 East Fifth Street, Suite 211  
Waterloo, IA 50703  
tschoellen@co.black-hawk.ia.us

## ABSTRACT

Black Hawk County (BHC) has developed a precast modified beam-in-slab bridge (PMBISB) system for use with accelerated construction. Individual components of the system have been tested in the Iowa State University Structural Laboratory, and the overall system was tested in the field. Using the BHC system, the bridge superstructure can be assembled in two days and the bridge opened to traffic as soon as the cast-in-place concrete connection between the precast panels has reached the required strength, which is usually one week.

Precast components tested in the laboratory include two precast abutment caps, three different types of deck panel connections, and a precast abutment backwall. Load testing on the first abutment cap revealed that it had considerably more strength than was required. Therefore, the cap was modified to be more efficient. The original PMBISB panel connection was inefficient in the amount of time and concrete needed for construction. Thus, three other panel-to-panel connection designs were tested in the laboratory, one of which has already been used in the field. The final laboratory work was the service load and ultimate load testing of a precast abutment backwall panel.

Field testing of the first precast bridge (L = 40 ft., W = 32 ft.) in Black Hawk County was performed in June of 2007. The testing investigated service load stresses, lateral load distribution characteristics, and overall global behavior of the system. This paper presents the results of the laboratory and field testing previously described.

**Key words: accelerated construction—bridge construction—precast concrete**

## **INTRODUCTION AND PROBLEM STATEMENT**

Precast concrete elements can be used to construct a bridge much more quickly and efficiently than a traditional cast-in-place concrete bridge. In addition, the precast elements are usually more uniform and durable than cast-in-place concrete elements because of more controllable casting conditions and stricter quality control. The modified beam-in-slab bridge (MBISB) investigated in Iowa Department of Transportation Project TR-467 was improved upon by using precast concrete elements. Testing of the precast modified beam-in-slab bridge (PMBISB) was presented in a report to the Black Hawk County (BHC) engineer (Wipf et al. 2004). The PMBISB is a series of panels consisting of steel girders with concrete placed around the girders. Eighteen in. PVC pipe is cut in half and placed between the girders to remove the concrete from the tension zone of the slab. In addition to the superstructure, BHC developed a precast abutment cap that utilizes a steel W-section that fits on top of the abutment H-piles. A precast abutment backwall that is positioned between the abutment H-piles was also developed by BHC. Construction is accelerated by having these elements cast during the winter, thus reducing the amount of man hours and overall time required to assemble the bridge in the field. To determine the strength and behavior of the precast elements, they were individually tested in the laboratory. To determine their interaction, a bridge constructed using these individual elements has been service load tested.

## **OBJECTIVES AND METHODOLOGY**

The purpose of the laboratory testing was to determine the strength, serviceability, and constructability benefits of the system. Specifically, it was desired to determine the strength and behavior of the precast abutment caps, the different cast-in-place connections between the deck panels, and the precast abutment wall. Field testing of the system focused on service load stresses, lateral load distribution characteristics, and overall global behavior of the system.

### **Precast Abutment Caps**

The precast abutment cap developed by BHC is made by casting concrete around the upper half of a steel W-section lying on its side, as seen in Figure 1. Holes are torched in the embedded flange to create a shear connection and composite action between the steel and concrete. When positioned on the abutment piles, the web of the W-section rests on top of the H-piles, with the flanges providing lateral restraint. Reinforcing steel is cast in the top of the specimen to provide increased negative moment capacity over the piles and to act as compression steel in the positive moment regions. Stirrups are also cast into the cap to provide increased shear capacity.

The first cap to be tested was constructed by Black Hawk County with a W12x65 section, shown in Figure 1a. The W12x65 was chosen to give the cap adequate flexural strength. During the service testing, supports were spaced at 4.5 ft., and the maximum stress in the steel was found to be 6.14 ksi when a service load of 40 kips was applied. Supports were spaced at 17.5 ft. for the positive moment strength test and spaced at 15 ft. for the negative moment strength test. The strength testing determined that the positive moment strength is greater than 765 kip-ft., and the negative moment strength is 363 kip-ft. In comparison, the factored design moment for the cap is 157 kip-ft.

A modification to the abutment cap was proposed as the strength of the cap was significantly larger than what was required. To increase efficiency of the abutment cap, Black Hawk County constructed a second cap using a W12x26 section, shown in Figure 1b, to reduce the weight, the cost, and strength of the cap. To accommodate new backwall sections, support spacing for the service level testing was changed to 5.5 ft., with the maximum stress in the steel only 6.46 ksi at the same service load of 40 kips, a 5.0% increase

in stress. Only a positive moment strength test was performed on the smaller section, with support spacing at 15.5 ft. The cap failed at a moment of 465 kip-ft., significantly greater than the factored design moment of 156 kip-ft.

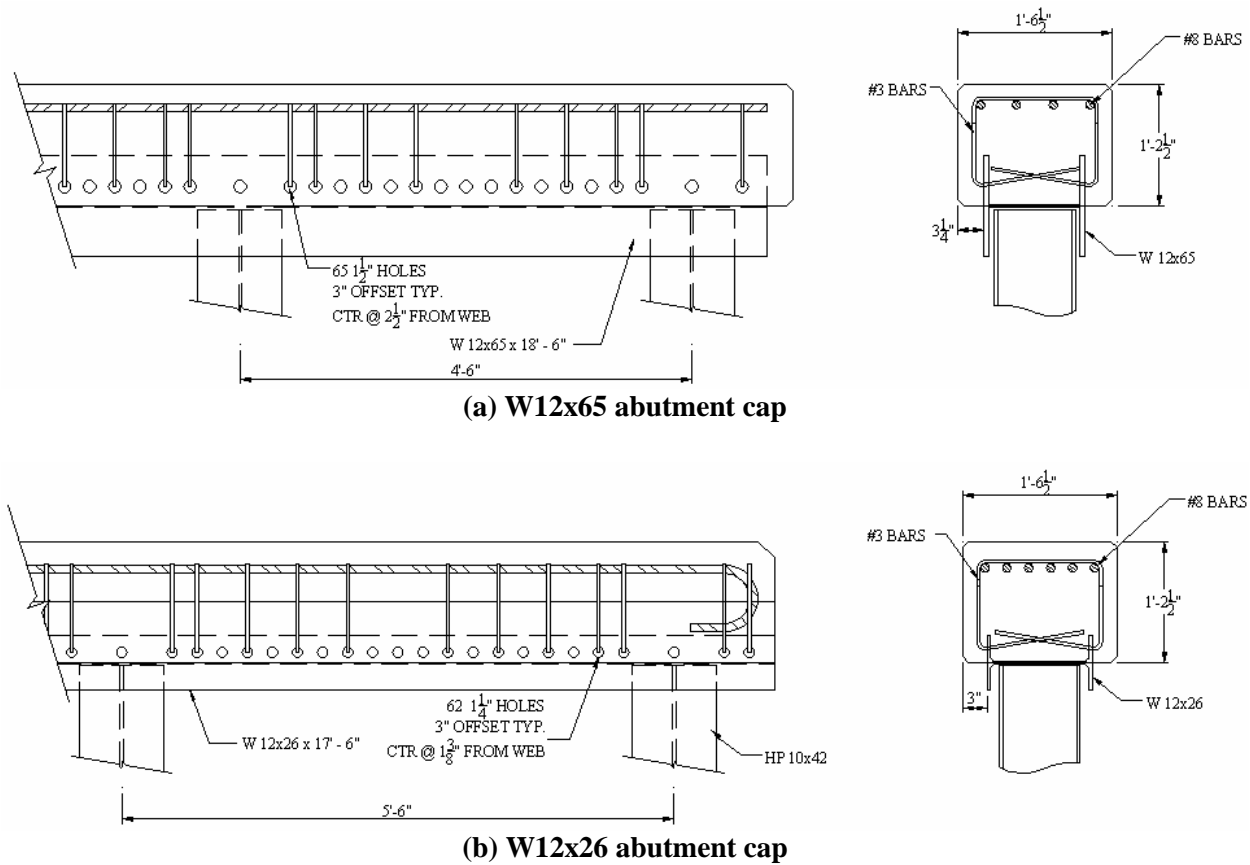


Figure 1. Precast abutment caps

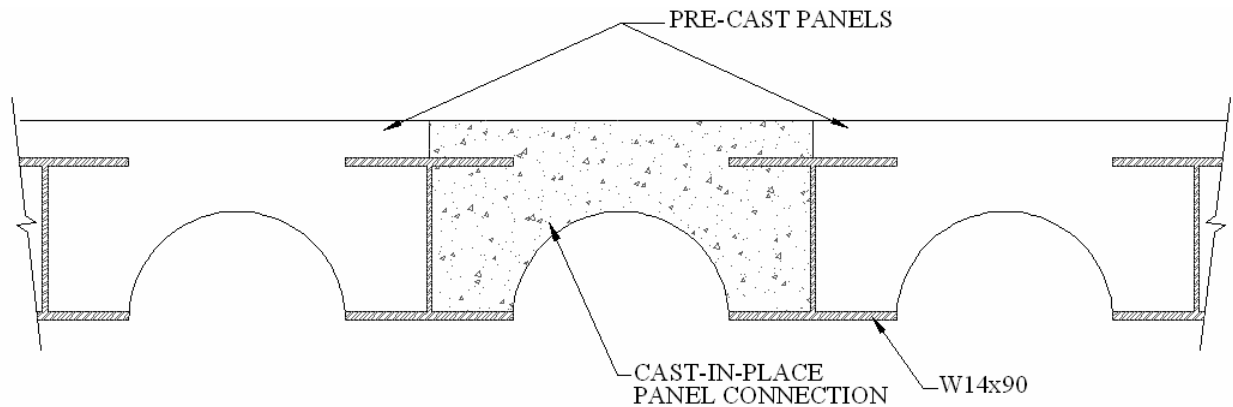
### Improving PMBISB Panel Connections

The original field connection, shown in Figure 2a, required a substantial amount of formwork to be constructed in the field and a significant amount of cast-in-place. Reducing the amount of time (for construction of the formwork) and concrete required in the construction was the main goal of redesigning the connection. The basic feature of the new designs is the use of a half-arch at the side of each panel. Above the half-arch is a notch that provides the formwork for the closure pour when two panels are placed next to each other. Three new designs were developed using the half-arch feature, and three specimens of each detail were cast and subjected to laboratory testing to determine their strength and behavior characteristics. For determining the strength of the connections symmetric point loads were applied on either side of the closure pour, creating a pure moment region.

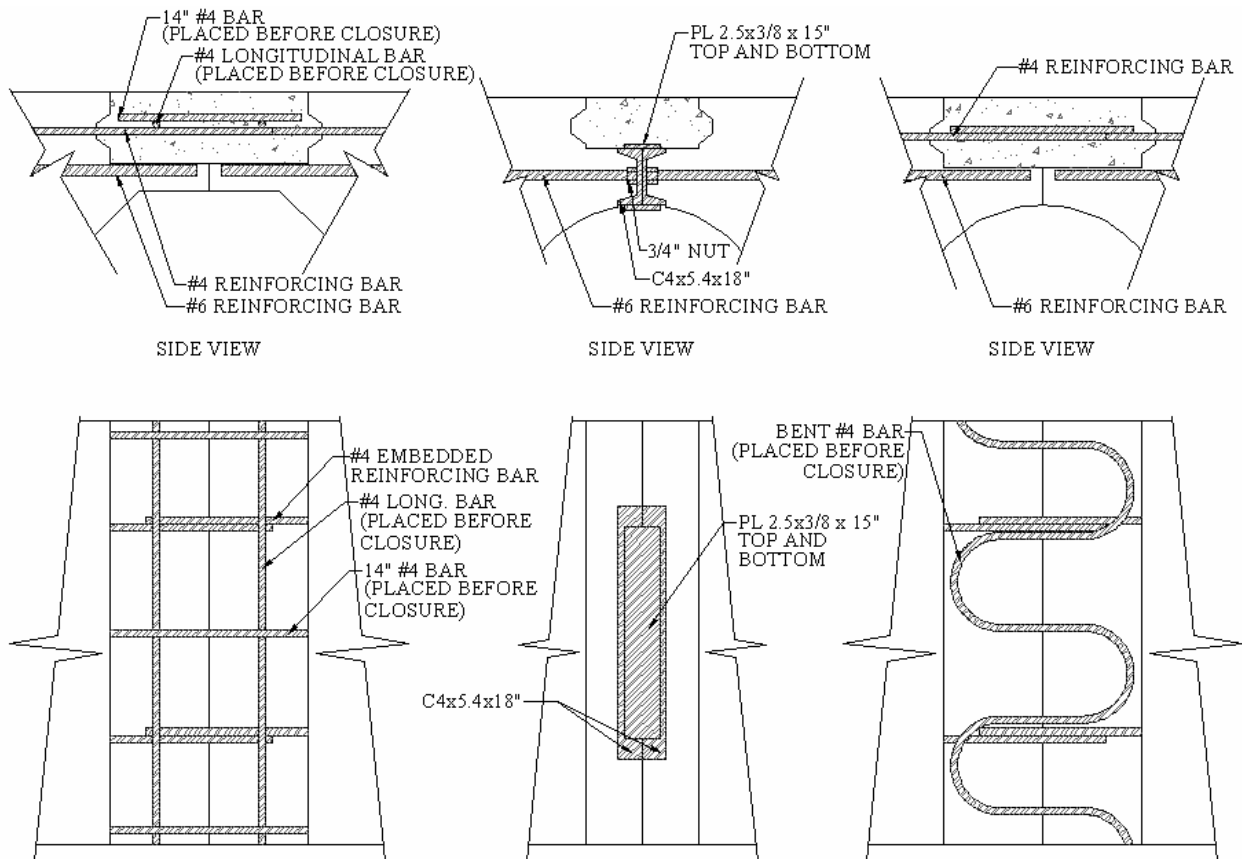
#### Type 1 Connection

The first connection tested has #4 reinforcing bars on 15 in. centers that protrude out of each precast panel into the notched area on the adjacent panel (see Figure 2b). Before concrete is placed into the closure area, two longitudinal #4 reinforcing bars are placed along the entire length of the joint along with

additional transverse #4 bars placed between the protruding reinforcing bars. This particular connection has already been used in the replacement bridge on Mt. Vernon Road in BHC.



(a) Original PMBISB field connection



(b) Type 1 Connection

(c) Type 2 Connection

(d) Type 3 Connections

Figure 2. Precast panel connection details

### *Type 2 Connection*

The Type 2 Connection removes the reinforcing bars from the closure area, as shown in Figure 2c, tying the panels together with two PL2.5x3/8 plates welded to the top and bottom of two embedded C4x5.4 channels. The channels are attached to the panels by welding the channels to the #6 reinforcing bars at the top of the arch. Welding area between the channel and the reinforcing bar is increased by welding a 7/8 in. nut onto the end of the bar. One obvious difficulty with this detail is the need to perform overhead field welding.

### *Type 3 Connection*

The Type 3 Connection is similar to the first in that reinforcing steel is placed in the field onto the protruding reinforcing bars before the closure pour. Instead of straight individual pieces of longitudinal and transverse reinforcing bar, the added reinforcing bar is bent into a series of S's, as shown in Figure 2d. The bent bar reduces the amount of steel tying required in the field.

Table 1 presents the ultimate moment capacities for each connection detail. The capacities were all normalized to the concrete strength of the Type 1 Connection.

**Table 1. Connection detail ultimate moment capacities**

Specimen	Normalized moment capacity (k-in./ft.)		
	Type 1	Type 2	Type 3
A	45.2	81.5	24.6
B	38.7	69.5	28.6
C	38.2	80.9	28.1
Average	40.7	77.3	27.1

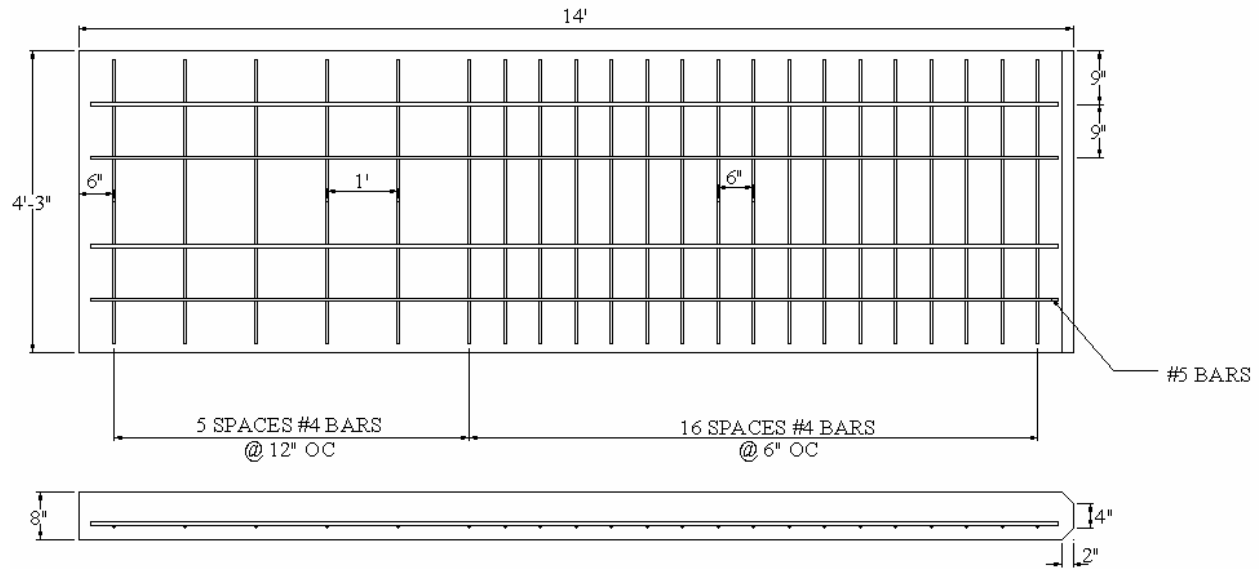
While it is the weakest, the Type 3 Connection is the easiest to construct in the field, requiring less tying than the Type 1 detail. However, the Type 2 Connection has the greatest moment capacity, nearly double the capacity of the Type 1 Connection which is currently in use. However, constructability is an issue with the Type 2 Connection since an experienced welder would be required for the overhead welding in the field. The Type 1 Connection has been used successfully in the field and is relatively easy to fabricate.

### **Abutment Backwall**

The precast backwall is a 14 ft. by 4.25 ft. reinforced slab of concrete designed to support the soil behind the abutment when supported by the flanges of the H-piles of the abutment. The variation in the transverse reinforcement accounts for the increased load with depth due to the lateral earth pressure of the soil: six #4 bars spaced on 12 in. centers for the first 5.5 ft. and 18 #4 bars on six in. centers for the remaining 8.5 ft. Longitudinal reinforcement is provided by four #5 bars that run the entire length of the slab, as seen in Figure 3.

Initially the backwall was service load tested without and with the supporting H-piles. The experimental cracking moment, determined from testing without the H-piles, was found to be 29.1 kip-ft., which is very close to the calculated cracking moment of 29.4 kip-ft. Three service level point loads were applied separately and simultaneously at linearly varying magnitudes to simulate lateral earth pressure. When the H-piles were added, the midspan deflection under the center load only was reduced from 0.091 in. to 0.049 in., a reduction of 46%. An ultimate load test was also performed by loading the slab at a single

point approximately 112 in. from the top of the wall. The slab was damaged during the first attempt when the spliced H-pile failed prematurely. At the time of the H-pile weld failure, the system was carrying 80 kips. After repairing the H-pile, the test was performed again, and the slab was still able to resist a point load of 100 kips. The expected resultant load from both a truck over the abutment and the lateral earth pressure from clay under end-of-construction conditions is approximately 63 kips. The end-of-construction soil condition provides the largest lateral earth loads. Even in its weakened state, the backwall system provided a factor of safety of 1.6 against failure.



**Figure 3. Precast abutment backwall**

### **Mt. Vernon Road Bridge**

The Mt. Vernon Road Bridge (L = 40 ft., W = 32 ft.) utilized two of the components investigated in the laboratory testing: the improved PMBISB connection detail, and the W12x65 precast abutment cap. The panels and abutment caps were precast at Black Hawk County facilities during the winter (Figure 4a). After the abutment walls were constructed, the abutment cap was placed on top of the H-piles (Figure 4b). At this point, the deck panels were transported to the site and placed on the abutment caps (Figure 4c), at which point the additional steel required for the joint detail was tied into the joint. Concrete for the closure joint was placed after all six panels were in their final positions and the joint reinforcement added. A standard C4 mix was used for the closure; workers were able to easily work the concrete using shovels and vibrators to fill the closure (Figure 4d). Total time to assemble the superstructure, including the closure pour (Figure 4e), was less than 40 hours. The bridge was opened to traffic after the concrete cured and a three beam guardrail system was installed (Figure 4f).

Field testing of the Mt. Vernon Road Bridge was conducted midway through June. A standard rolling test was executed over five different lanes on the bridge deck. Preliminary results show that the maximum tensile stress in the steel was 2.6 ksi, and the maximum compressive stress in the concrete deck was -0.25 ksi. The allowable stress for the steel used in the design of the panels was  $.55f_y$  (27.5 ksi), while the allowable stress in the concrete was  $.4f_c'$  (2 ksi). Maximum deflection attained was only 0.18 in., well below the American Association of State Highway and Transportation Officials (AASHTO) live load deflection limit for vehicular loads of Span/1000 which is 0.48 in. (AASHTO 2004).



(a) Casting the deck panels



(b) Abutment cap on abutment piles



(c) Placing the deck panels



(d) Casting the panel connections



(e) Completed bridge without guardrails



(f) Completed bridge with guardrails

**Figure 4. Construction of Mt. Vernon Road Bridge**

## SUMMARY AND CONCLUSIONS

Precasting the panels for the MBISB was an effective way to reduce the amount of time needed to construct the bridge. More reduction in construction time can be achieved through a redesign of the panel-to-panel connection. Precasting more of the bridge elements, such as the abutment cap and abutment backwall, further reduces construction time.

In the first design of the precast abutment cap, a W12x65 steel section was used. Due to its extra strength, it was decided to make the cap more efficient by using a W12x26 steel section. While expectedly having less strength than the original design, the new design still has more than enough strength to handle the loads expected to be placed on it from legal loads.

All three panel connection designs provide a more efficient and effective means of joining adjacent panels together in the field. The major drawbacks to the Type 2 Connection design are that it is very dependent on the skill of the welder and on favorable conditions under the bridge to facilitate overhead welding. The Type 3 Connection, while slightly easier to construct in the field, is markedly weaker than the Type 1 Connection. For these reasons, along with the simplicity of construction in the field, the Type 1 Connection is the preferred detail for joining the deck panels together.

A precast abutment backwall was proposed for the system to facilitate accelerated construction. Testing of the precast abutment wall, as expected, verified the system was much stronger when placed between the H-piles than it was when it was without the H-piles. Testing also determined that the strength of the abutment backwall system was more than adequate to resist the combined loads from the soil and a truck positioned above the abutment.

The construction of the Mt. Vernon Road Bridge was cost- and time-efficient. The PMBISB system allowed the superstructure to be built in less than 40 hours, while total time for the entire bridge construction was less than 22 days. Field testing determined very small stresses from the truck load, well below the recommended working level stresses. Measured deflections were also well within allowable values. Because of this, accelerated construction with the PMBISB system is a viable option for bridges on low-volume roads.

## **ACKNOWLEDGMENTS**

The research presented in this paper was funded by the Iowa Highway Research Board and the Iowa Department of Transportation, Ames, Iowa. The authors wish to thank Black Hawk County and the graduate and undergraduate students who helped on this investigation.

## **REFERENCES**

- AASHTO. 2004. LRFD Bridge Design Specification. Third Edition. Washington DC: American Association of State Highway and Transportation Officials.
- Wipf, T.J., F.W. Klaiber, A.V. Wessling, and T.F. Konda. 2004. *Investigation of a Pre-Cast Modified Beam-in-Slab Bridge System*. Project TR-467. Ames, IA: Iowa Department of Transportation.