Experimental Evaluation of Precast Channel Bridges

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ABSTRACT

The precast channel bridge (PCB) is a short span bridge that was commonly used on Iowa's secondary roads approximately forty years ago. Each PCB consists of eight to ten simply supported precast panels ranging in length from 5.8m to 11.0m. The panels resemble a steel channel in cross-section; the web is orientated horizontally and forms the roadway deck and the legs act as shallow beams. Bundled reinforcing bars in each leg act as the primary flexural reinforcement.

Many of the approximately 600 PCBs in Iowa show signs of significant deterioration. Typical deterioration consists of spalled concrete cover and corrosion of the bundled primary reinforcement. The objective of this research was to access the structural sufficiency of the deteriorated PCBs through field and laboratory testing.

Four deteriorated PCBs were instrumented with strain gages to measure strains in both the concrete and reinforcing steel and transducers to measure vertical deflections. Response from loaded trucks was recorded and analyzed. Test results revealed that all measured strains and corresponding stresses were well within acceptable limits. Likewise, measured deflections were much less than the recommended AASHTO value.

Laboratory testing consisted of loading twelve deteriorated panels to failure in a four point bending arrangement. Although all panels exhibited significant deflection prior to failure, the experimental capacity of eleven panels exceeded their theoretical capacity. The experimental capacity of the twelfth panel, an extremely distressed panel, was only slightly below its theoretical capacity.

Key words: deterioration—flexural capacity—precast channel bridge—precast concrete bridge

INTRODUCTION

Recent data compiled by the National Bridge Inventory revealed 29% of Iowa's approximate 24,600 bridges were either structurally deficient or functionally obsolete. This large number of deficient bridges and the high cost of needed repairs create significant problems for Iowa and many other states. The research objective of this project [1] was to determine the load capacity of a particular type of deteriorating bridge – the precast channel bridge (PCB) – that is commonly found on Iowa's secondary roads. The number of these precast concrete structures requiring load postings and/or replacement can be significantly reduced if the deteriorated structures are found to have adequate load capacity or can be reliably evaluated.

Approximately 600 PCBs currently exist in Iowa. These bridges were constructed primarily in the 1950's and 1960's. A typical PCB span is 5.8m to 11.0m long and consists of eight to ten simply supported precast panels. Abutments and piers typically consist of cast-in-place reinforced concrete caps supported by timber piles. A curb is cast along the top edge of the two exterior panels and steel or concrete rail post bolt to the sides of these panels. A typical bridge cross-section is presented in Figure 1.

Two similar standard panel designs are found in Iowa. The primary difference between the two designs is the configuration of the joint between adjacent panels. Type I panels, shown in Figure 2, are joined by transverse bolts and a continuous grouted shear key. The joint between adjacent Type II panels consists of a concrete-filled galvanized pipe and transverse bolts. Details of the Type II panels are presented in Figure 3.

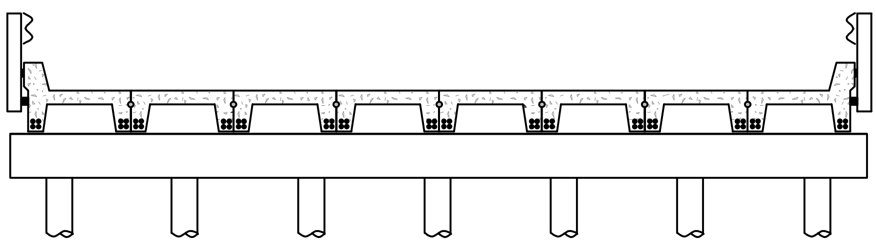

FIGURE 1. Typical PCB Cross-Section near Abutment (Roadway Crown not Shown)

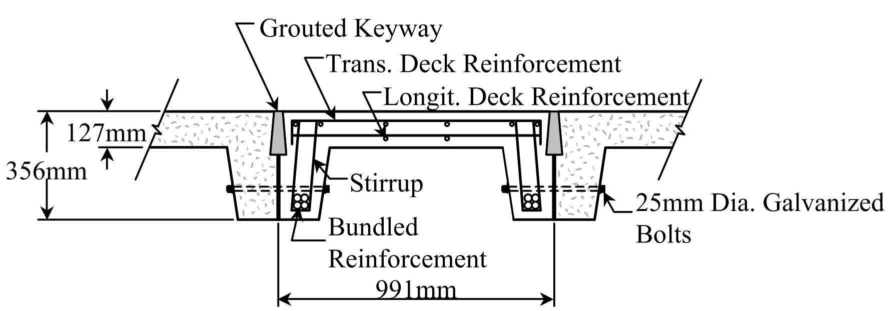

FIGURE 2. Type I PCB Panel Cross-Section

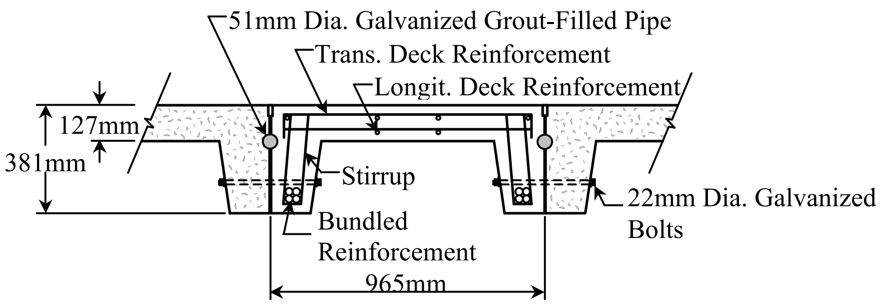

FIGURE 3. Type II PCB Panel Cross-Section

Many of Iowa's PCBs are heavily deteriorated. Typical deterioration consists of spalled concrete and significant corrosion of the primary reinforcing steel. In many cases, as shown in Figure 4, the bottom bars in a stem are exposed over almost the entire span. In other cases, longitudinal cracks and rust stains are often found on the bottom and side of the panel stems. The effects of this deterioration, such as loss of cross-sectional reinforcement area and bond, have lead to concerns over the ability of a deteriorated PCB to safely support legal loads.

FIGURE 4. Typical PCB Deterioration

FIELD TESTING

Four deteriorated PCBs were selected for field testing from the results of a questionnaire sent to all 99 Iowa counties. Selection criteria were the extent of the deterioration, whether the bridge was scheduled for replacement, and location. The geometry and panel type of each bridge is presented in Table 1.

TABLE 1. Geometry and Panel Type for Field Tested PCBs

Bridge	1	2	3	4
No. of Spans	1	1	2	1
Out-to-Out Panel Length (m)	9.4	11.0	9.4	7.6
No. of Panels/Span	10	8	9	9
Panel Type	II	II	I	I

TESTING METHOD

Each bridge was instrumented in a similar fashion. Since each span was simply supported, the instrumentation was located at midspan so that the maximum response could be measured. Electrical resistance strain gages were bonded to the primary reinforcement and concrete deck. Transducers were attached to the panel stems to measure vertical deflection and differential deflection across the panel joints.

Legally loaded tandem axle dump trucks or a truck tractor-simitrailer combination was used to load the bridges. The trucks crossed each bridge in three designated lanes (left, right, and center) at a slow speed. Data were continuously recorded by an electronic data acquisition system (DAS); tape switches connected to the DAS identified the longitudinal location of the truck.

TEST RESULTS

The field test results were very useful for accessing two specific areas of performance. First, deflection and strain data from a given panel were analyzed to determine if the loads placed on the panel induced stresses above allowable limits. Second, deflection data from all of the panels for certain transverse and longitudinal load positions were used to calculate transverse load distribution factors.

Maximum tensile steel strains for all four PCBs ranged from 110 microstrain for Bridge 3 to 208 microstrain for Bridge 2. Taking Young's modulus of the reinforcement as 200 MPa relates to maximum stress levels ranging between 0.022 MPa and 0.042 MPa. The Iowa Department of Transportation standard by which the PCBs were constructed stipulates that stress in the reinforcement not exceed 0.138 MPa. Including the stress caused by the self weight of the panel (approximately 0.034 MPa) results in a total stress in the reinforcement that is slightly more than one half of the allowable stress. This indicates that the legally loaded trucks induced a safe level stresses in the PCBs. Similarly, live load deflections were well below acceptable limits given by AASHTO [2]. In terms of live load deflection-to-span ratios, Bridge 2 had the largest ratio (L/1525) and Bridge 3 had the smallest ratio (L/2213). AASHTO recommends that the live load deflection be limited to L/800.

Due to weather and traffic conditions, accurate concrete strain measurements could only be taken on Bridge 2. The largest recorded concrete compressive strain was 110 microstrain. The corresponding calculated live load concrete stress was 3.05 kPa and the dead load stress was 4.62 kPa. As was the case for the reinforcement stress and live load deflection, the total concrete stress, 7.67 kPa was well below the specified maximum, 13.8 kPa.

Load fractions, a measure of transverse load distribution, were calculated for each bridge from the midspan deflection data. The load fraction for a panel represents the fraction of a wheel line carried by that particular panel. In most cases, the two panels on which the truck was tracking on had the greatest load fractions of all the panels. Hereafter, the greatest load fraction for each bridge will be simply referred to as the load fraction for that bridge. All load fractions were calculated when the tandem axles were longitudinally centered at midspan.

The load fractions varied greatly from bridge to bridge. This was due to variations in the connections between adjacent panels. Inspection of all four bridges revealed that the concrete-filled pipes were not in place on Bridge 2 and grout was not packed in the keyways on Bridge 4. The keyways on Bridge 4 contained only gravel from the gravel wearing surface.

Bridge 3, with its fully grouted keyways, had the lowest load fraction for all PCBs tested. Its load fraction, 0.42, was also well below the design one lane load fraction, 0.58, calculated in accordance to AASHTO. The load fraction for Bridge 1 was 0.49 and was also below the design one lane load fraction, 0.57. These test results verify the effectiveness of the shear connection when installed properly.

When the shear connectors were not properly installed, the load fractions were greater than the design values. For Bridge 4, this difference was only marginal. The gravel in the shear keyways apparently aided in the transfer of shear to some degree. A larger difference occurred in Bridge 2 where the load fraction was 0.68, which was considerably greater than the design load fraction of 0.56. This is an important deficiency since a panel is supporting 21% more load than what it was designed to resist.

The effectiveness of these shear connectors is graphically show in Figure 5. This plot shows midspan deflection for Bridge 3 and 4 when the center lane was loaded. As one can see, the grout in the keyways of Bridge 3 prevented differential displacement across the panel joints. When the keyways were not grouted, significant joint slip occurred. Also, rigid joints transfer more load to neighboring panels and thus the load fractions for Bridge 3 were less than the load fractions for Bridge 4.

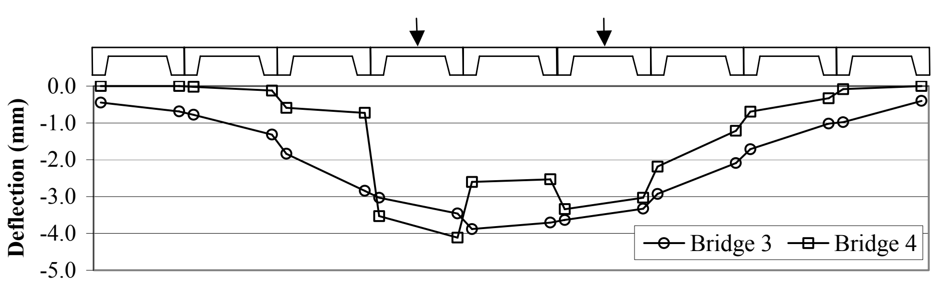

FIGURE 5. Transverse Centerline Deflections: Center Lane Loading

LABORATORY TESTING

A total of twelve deteriorated PCB panels from three different bridge replacement projects throughout Iowa were tested for ultimate flexural strength. The panels ranged in length from 7.6m to 11.0m and varied in the amount of deterioration. Some had relatively minor spalling and corrosion of the reinforcement while on others the majority of the primary reinforcement was exposed and heavily corroded. This variation was very useful since the effects of the deterioration could be seen. Additional information gained from the laboratory testing included panel strength, failure mode, stiffness, and strength of the concrete and reinforcement.

TESTING METHOD

Instrumentation used in the laboratory tests was similar to the instrumentation used in the field. Strain gages were bonded to the primary reinforcement and concrete deck at midspan. Transducers were used to measure vertical deflection at midspan and also at the quarterpoints. Each panel was loaded in a four point bending arrangement with 1.8m separating the middle load points. Load was applied by hydraulic actuators and was gradually increased until a failure occurred. During each load test, strain, deflection and load magnitude data were recorded by the DAS at predetermined levels of applied load. Following the failure, concrete cores and lengths of reinforcement were removed from undamaged portions of the panels and tested to determine the yield strength and the compressive strength of the reinforcement and concrete, respectively.

TEST RESULTS

The PCB panels performed well given their deteriorated state. Experimental ultimate strengths were found to generally exceed the theoretical ultimate strengths. Two factors contributed to this performance. First, large hooks on the ends of the bottom pair of bundled reinforcing bars effectively eliminated the need for development bond throughout the span. Secondly, the yield strength of the reinforcement was found to be considerably greater than the specified yield strength. Likewise, the concrete strength was found to also exceed its specified strength. The failure mode of all panels was a compression failure of the concrete deck preceded by excessive deflection.

Experimental and theoretical ultimate strengths for each panel are presented in Table 2; names indicate the county from where various panels were obtained. Cedar 1-3 were the shortest panels tested and therefore theoretically had the lowest ultimate strength. However, due to high concrete strength and reinforcement that experienced considerable strain hardening, the Cedar panels actually had the highest ultimate strength of the panels tested. Cedar 4 was the only panel tested with an attached curb. This curb increased the experimental strength of the panel by approximately 31% over the other Cedar panels.

As shown in Table 2, the experimental strength of the PCB panels exceeded their theoretical strength in all but one case: Butler 3. Butler 3 was by far the most deteriorated panel. In addition to heavy spalling and corrosion of the primary reinforcement, approximately 50% of deck surface had delaminated and spalled. The uncommon form of deterioration weakened the panel. With less deck available to resist compressive stresses, the remaining deck became overstressed at a lower load.

TABLE 2. Experimental and Theoretical Ultimate Strengths for Laboratory Tested Panels

Panel	Ultimate Strength (kN*m) - Experimental/Theoretical			
	1	2	3	4
Cedar	632/285	662/285	636/285	842/476
Butler	473/362	494/362	334/362	441/362
Black Hawk	521[1]/405	488[1]/405	583/405	549/405

[1]Failure not reached due to loading system limitations; value is midspan moment at maximum applied load.

Although the extent of the deterioration varied within a group of like panels, the experimental ultimate strengths for the various groups were relatively close. For Cedar 1-3, ultimate strength varied by only 30 kN*m and ultimate strength of the Butler panels varied by only 53 kN*m when the heavily damaged Butler 3 is excluded. Similar variation occurred for the two failed Black Hawk panels. This is an indication that the ultimate strength of these panels is only affected by extreme deterioration.

Midspan moment verses midspan deflection plots for the Black Hawk PCB panels are presented in Figure 6. As one can see, well defined regions of elastic and plastic behavior existed. This behavior was typical for all PCB panels tested. Elastic behavior occurred up to the point when the stress in the primary reinforcing exceeded its yield stress. The panels then deflected excessively while supporting only a small increase in load. This behavior continued until a compression failure occurred in the deck. Also shown in Figure 6 is the midspan moment induced by AASHTO HS20 loading. The magnitude of this loading is well within the elastic range and is considerably less than the ultimate strength of these panels.

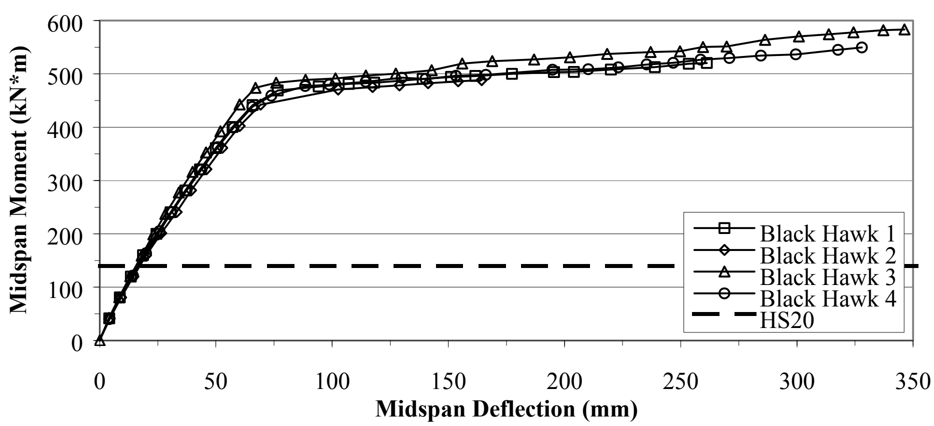

FIGURE 6. Moment Verses Deflection Plots For Black Hawk Beams

SUMMARY AND CONCLUSION

The most common form of deterioration found on PCBs was corrosion of the primary reinforcement and spalling concrete cover for this reinforcement. Through laboratory and field tests, it was determined that this deterioration had minimal effect on the performance of these bridges. Hooks on the ends of some of the primary reinforcement reduced the need for bond along the span. Another less common form of deterioration, deck delamination, did however cause a decrease in the ultimate strength of the panels.

The shear connection between panels was found to significantly affect the performance of the PCBs. When shear connectors were not properly installed, transverse load distribution was less than required by AASHTO.

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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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