Evaluation of Performance of Innovative Bridges in Wisconsin

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16. Abstract

Since the early 2000s, several federal programs have existed to provide bridge owners with funding to cover "delta" costs associated with implementing new, emerging, and innovative bridge technologies. While these programs have generally included an evaluation component, there generally has not been a concerted effort to track the performance of these innovative bridges following the completion of the initial project.

The goal of this work was to conduct field reviews of the condition and performance of several innovative bridge concepts constructed in Wisconsin. The completion of this work was to provide a much needed review of the performance of these bridge as they had been in service for several years.

This report documents the condition of 11 innovative bridges or innovative bridge features in Wisconsin. The bridges have innovative technologies consisting of the following: inverted T-beams, exodermic deck, geosynthetic-reinforced soil (GRS) abutments, fiber-reinforced polymer (FRP) components, steel free deck, bi-directional post-tensioning, stainless steel reinforcement, and precast substructure components. Collectively, these innovations represent departures from conventional bridge design and construction—but aren't so radical that further adoption would be impossible.

The results of the 11 bridge evaluations, each of which followed a protocol specific to the bridge, are contained in a mini-report as part of this final report. Each mini-report documents general bridge information, briefly describes the innovation used, and provides the result of the evaluation.

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Final Report
July 2016

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Executive Summary and Introduction

Since the early 2000s, several federal programs have existed to provide bridge owners with funding to cover “delta” costs associated with implementing new, emerging, and innovative bridge technologies. While these programs have generally included an evaluation component, there generally has not been a concerted effort to track the performance of these innovative bridges following the completion of the initial project.

The goal of this work was to conduct field reviews of the condition and performance of several innovative bridge concepts constructed in Wisconsin. The completion of this work was to provide a much needed review of the performance of these bridge as they had been in service for several years.

New and emerging bridge technologies many times offer opportunities for innovation in an engineering community not normally associated with innovation and risk taking. Unfortunately, the initial costs of innovative are usually higher than the use of conventional construction, design, or maintenance approaches. The bridges constructed in Wisconsin through programs such as the Innovative Bridge Research and Construction (IBRC) and the Innovative Bridge Research and Deployment (IBRD) programs created opportunities for bridge engineers to innovate without concern for large initial costs.

The work described in this report documents the condition of 11 innovative bridges or innovative bridge features in Wisconsin. The bridges evaluated in this work have innovative technologies consisting of the following: inverted T-beams, exodermic deck, geosynthetic-reinforced soil (GRS) abutments, fiber-reinforced polymer (FRP) components, steel free deck, bi-directional post-tensioning, stainless steel reinforcement, and precast substructure components. Collectively, these innovations represent departures from conventional bridge design and construction—but aren’t so radical that further adoption would be impossible.

Each of the 11 innovative bridges evaluated in this work, were evaluated following a custom protocol specific to each bridge. In some cases, the evaluation was limited to common visual inspection approaches. In other cases, the evaluation consisted of the installation and monitoring of electronic instrumentation. However, regardless of the specific evaluation protocol followed, the goal of each evaluation was to provide information on the current condition and behavior of the innovation such that decisions regarding future use might be made.

The results of each of the evaluations are contained in bridge specific mini-reports. Each mini-report documents general bridge information, briefly describes the innovation used, and provides the result of the evaluation.

B-13-0609: STH 19 over Token Creek Tributary, Sun Prairie, WI – Use of Inverted T-beams

General Information

Bridge B-13-0609 is located on STH 19 over a Token Creek tributary near Sun Prairie, WI. The bridge was constructed in 2013 using an innovative structural design, a superstructure consisting of inverted precast concrete Ts with cast-in-place concrete infill. This innovative feature has been more commonly used in various forms at several locations throughout the upper Midwest, with many in the State of Minnesota.

Description of Innovative Feature

The bridge measures 37' 1 ½" x 46'-6" in total length and width, respectively. The cross section, shown in Figure 1, indicates a total of eight precast inverted Ts set side by side.

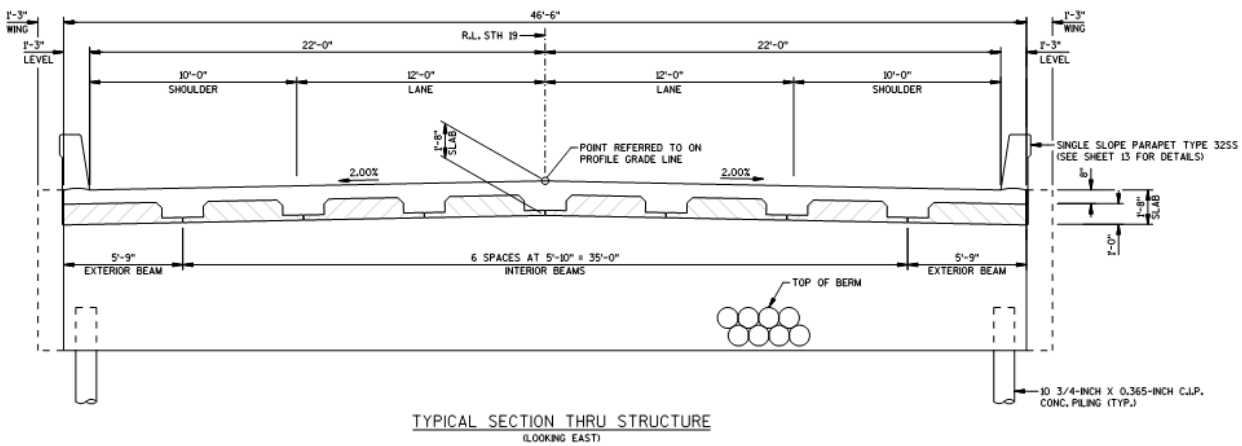


Figure 1. STH 19 over Token Creek Tributary bridge superstructure

Each interior panel measures 5'-10" wide and the exterior panels measure 5'-9" wide. Once the panels have been set in place, the superstructure is completed with a reinforced cast-in-place infill measuring 1'-0" thick over the Ts and 8" thick over the panel high side. The total superstructure thickness is 1'-8". Each panel is constructed with #4 stirrups along its length, which, when completed, forms a composite section with the cast-in-place infill.

Use of Innovative Feature

There are several advantages to using an inverted precast T bridge. The inverted Ts can expedite superstructure construction by serving as both the superstructure structure and the formwork for the cast-in-place infill. Time is saved by eliminating the need for the construction and tear down of superstructure formwork. To this point, the most common application of this innovative feature has been on shorter span bridges where the precast members are considered to be more manageable from a shipping and lifting point of view. Additionally, the use of precast concrete offers the opportunity for a higher quality concrete section than that which is usually obtained from cast-in-place concrete. This higher quality may lead to longer lived bridges.

Field Results

On September 16, 2015, the bridge was inspected for any signs of distress or underperformance. A visual inspection of the superstructure was completed with the specific goal of identifying any cracks that might propagate through the cast-in-place infill at the panel joints.

Visual Observation

The overall surface of the deck was in very good condition as seen in Figure 2.

Figure 2. STH 19 over Token Creek Tributary surface condition of superstructure

Only a few minor (<1/32”) cracks were observed at the interface between the approach slab and the bridge entrance. An example of these cracks is shown in Figure 3.

Figure 3. STH 19 over Token Creek Tributary minor cracking at bridge entrance

No top of deck cracks were observed at locations where reflective cracking might be expected. This indicates that the bridge is performing well with little to no differential displacement between adjacent inverted T beams.

The underside of the superstructure was also in very good condition. There were no observable cracks. The underside is shown in Figure 4 with a close-up picture shown in Figure 5.

Figure 4. STH 19 over Token Creek Tributary overall underside condition

Figure 5. STH 19 over Token Creek Tributary underside of superstructure close-up

Conclusions and Recommendations

Using precast inverted Ts as a stay-in-place form and structural component of the superstructure is a viable design that performs well under load and likely possesses long-term durability. Even more, if the design if further refined, construction could become more economical and quicker thereby becoming a practical solution for accelerated bridge construction. It is recommended that the use of the inverted-T bridges continue. However, future applications should consider the benefit of more optimized sections versus the observed high level of performance.

B-13-0608: STH 19 over Token Creek,
Sun Prairie, WI – Use of Inverted T-beams

General Information

Bridge B-13-0608 is located on STH 19 over Token Creek near Sun Prairie, WI. The bridge was constructed in 2013 using an innovative structural design, a superstructure consisting of inverted precast concrete Ts with cast-in-place concrete infill. This innovative feature has been more commonly used in various forms at several locations throughout the upper Midwest, with many in the State of Minnesota.

Description of Innovative Feature

The bridge measures 53'-0" x 46'-6" in total length and width, respectively. The cross section, shown in Figure 6, indicates a total of eight precast inverted Ts set side by side.

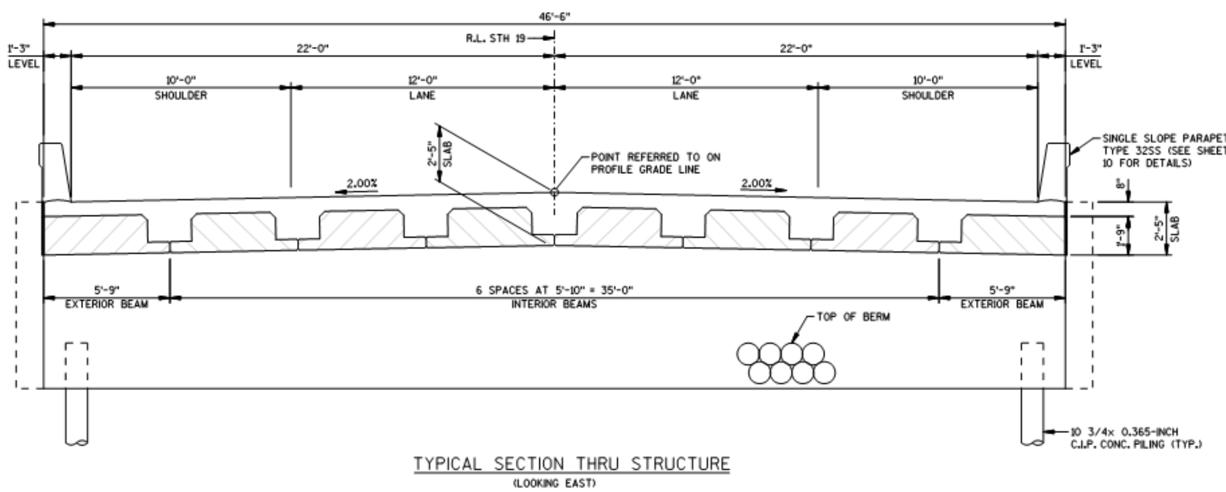


Figure 6. STH 19 over Token Creek bridge superstructure

Each interior panel measures 5'-10" wide and the exterior panels measure 5'-9" wide. Once the panels have been set in place, the superstructure is completed with a reinforced cast-in-place infill measuring 1'-9" thick over the Ts and 8" thick over the top of the panel. The total superstructure thickness is 2'-5". Each panel is constructed with #4 stirrups along its length, which, when completed, forms a composite section with the cast-in-place infill.

Use of Innovative Feature

There are several advantages to using an inverted precast T bridge system. The inverted Ts can expedite superstructure construction by serving as both a part of the complete superstructure system as well as serve as the formwork for the cast-in-place infill. Time is saved by eliminating the need for the construction and tear down of superstructure formwork. To this point, the most common application of this innovative feature has been on shorter span bridges where the precast members are considered to be more manageable from a shipping and lifting point of view. Another advantage of the inverted T-beam bridge system is that the precast concrete portion of the system is generally of higher quality than typical cast-in-place components. This higher quality concrete can lead to a structural system with a long service life.

Field Results

On September 16, 2015, the bridge was inspected for any signs of distress or underperformance. A visual inspection of the superstructure was completed with the specific goal of identifying any cracks that might propagate through the cast-in-place infill at the panel joints. In addition to the visual inspection, a load test was completed to measure the performance of the bridge when subjected to a loaded truck. The results of the visual inspection and load test are presented below.

Visual Observation

The surface of the deck was in very good condition and showed no signs of cracking (see Figure 7).

Figure 7. STH 19 over Token Creek surface condition of superstructure

The underside of the superstructure also appeared in very good condition as there were no observable cracks. The underside is shown in Figure 8.

Figure 8. STH 19 over Token Creek underside of superstructure

Overall, from a visual perspective, the bridge appears in excellent condition and does not appear as though there are areas of undue stress or deterioration as a result of the loads or any other conditions to which it is subjected.

Load Test

A load test was completed to identify the load distribution characteristics of the inverted-T superstructure. Three strain gages were placed on the bottom side of each precast T along the midspan of the bridge; one near each beam edge and one along the centerline. This placement allows for a side by side comparison of the strain at either side of the panel joints as well as the study of the general transverse load distribution. Additionally, differential deflection was measured at four locations to determine if the deflection magnitude was equal across panel joints. An overall picture of the bridge instrumentation is shown in Figure 9, while a close-up of two strain gages and a deflection gage at a panel joint is shown in Figure 10.

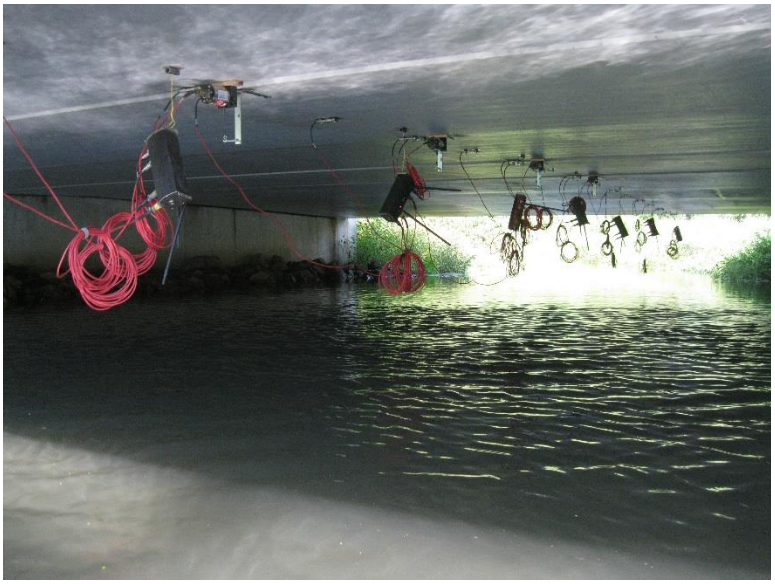


Figure 9. STH 19 over Token Creek load test instrumentation

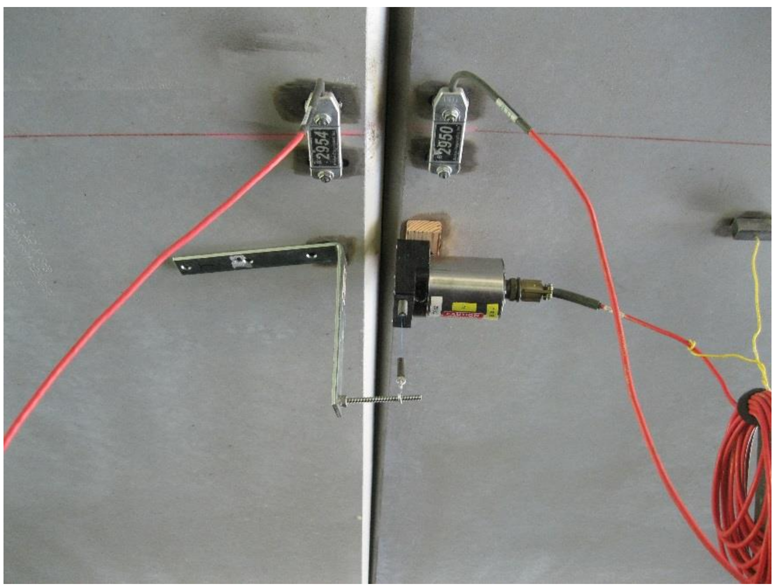


Figure 10. STH 19 over Token Creek typical gage configuration

The load test was completed using the loaded dump truck shown in Figure 11; the gross weight of the vehicle was just under 52 kips.

Figure 11. STH 19 over Token Creek load vehicle

The spacing between the front as last rear axle measured just under 18 ft and the total width measured just under 7 ft; the axle and tire configurations are shown in Figure 12.

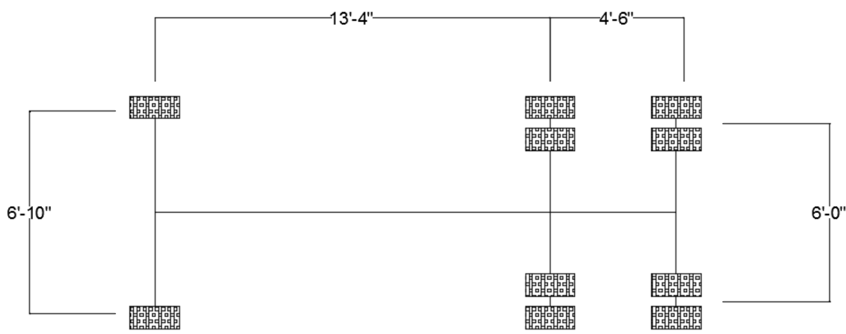


Figure 12. STH 19 over Token Creek axle configuration

During the testing process, the load vehicle traveled from west to east as shown in Figure 13 at a walking pace to reduce or eliminate any dynamic effects.

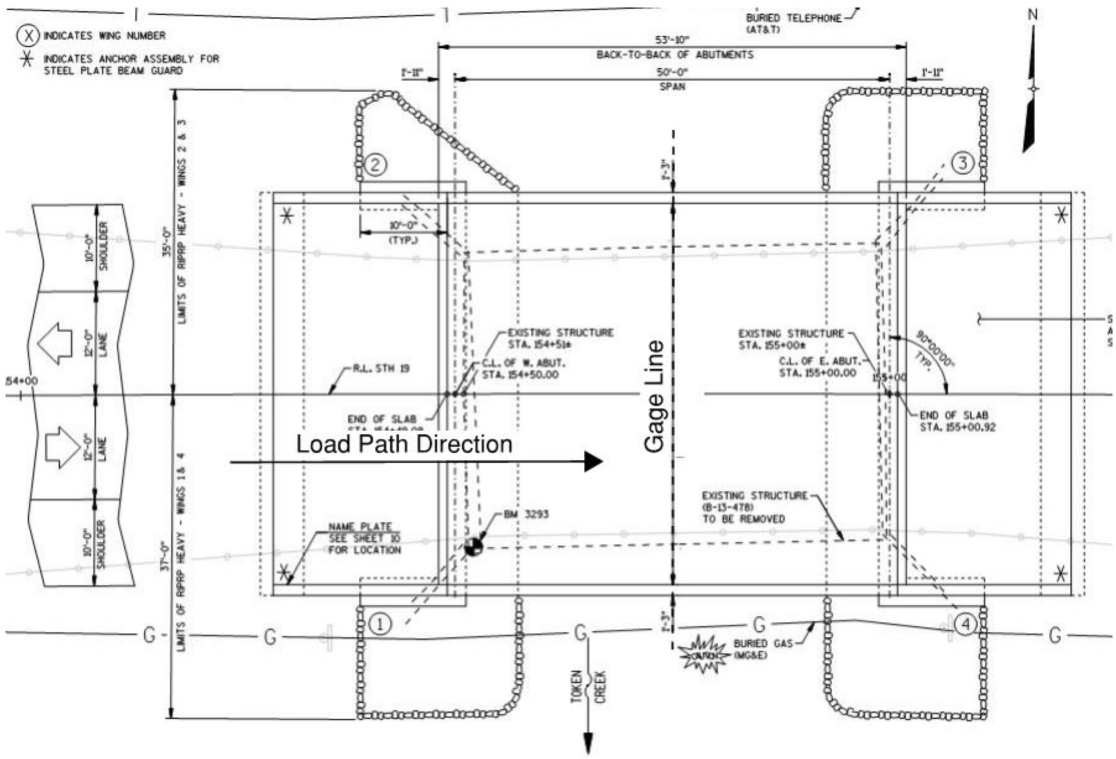


Figure 13. STH 19 over Token Creek load path direction and gage locations

Other traffic was prohibited from crossing the bridge while the researchers were collecting data. In total, five individual load paths were completed as shown in Figure 14.

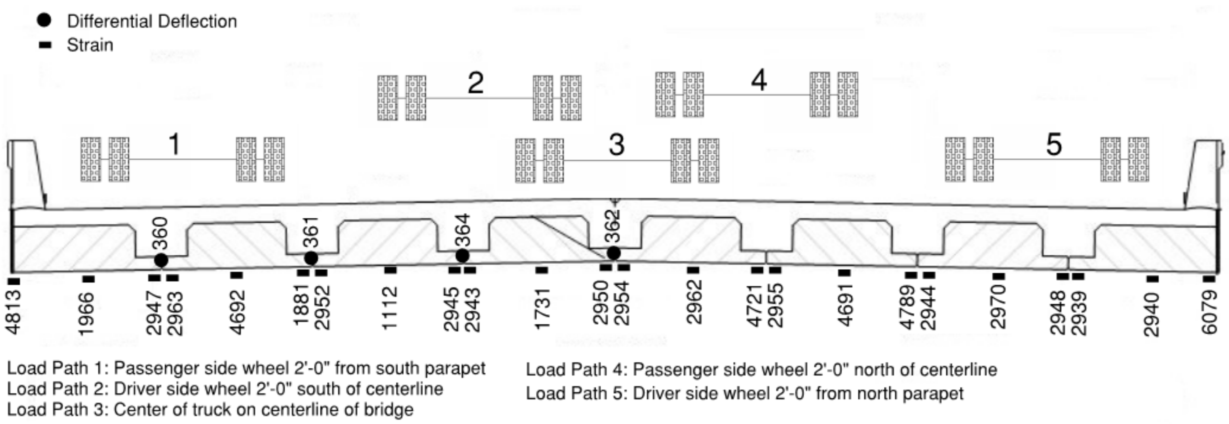


Figure 14. STH 19 over Token Creek load path and gage positions – looking west

Results

From the collected strain data for each load case shown in Figure 15 through Figure 24, one can see the overall magnitude of strain and relative distribution of load across the bridge. The maximum strain achieved in any one load case is approximately 14 ms (load path 5), equating to a relatively low stress. The strain pattern across the bridge indicates that the load is being distributed, at least in part, across the entire bridge, a good indicator of uniform stiffness and connectivity between panels.

The differential deflection measured between panels was less than the magnitude of the inherent instrumentation noise (10,000th of an inch), thus not illustrated in graphical form in the results below. The differential deflection is effectively zero across panels and thus reflective cracking in the deck directly above the panel joints is not anticipated (and was not observed).

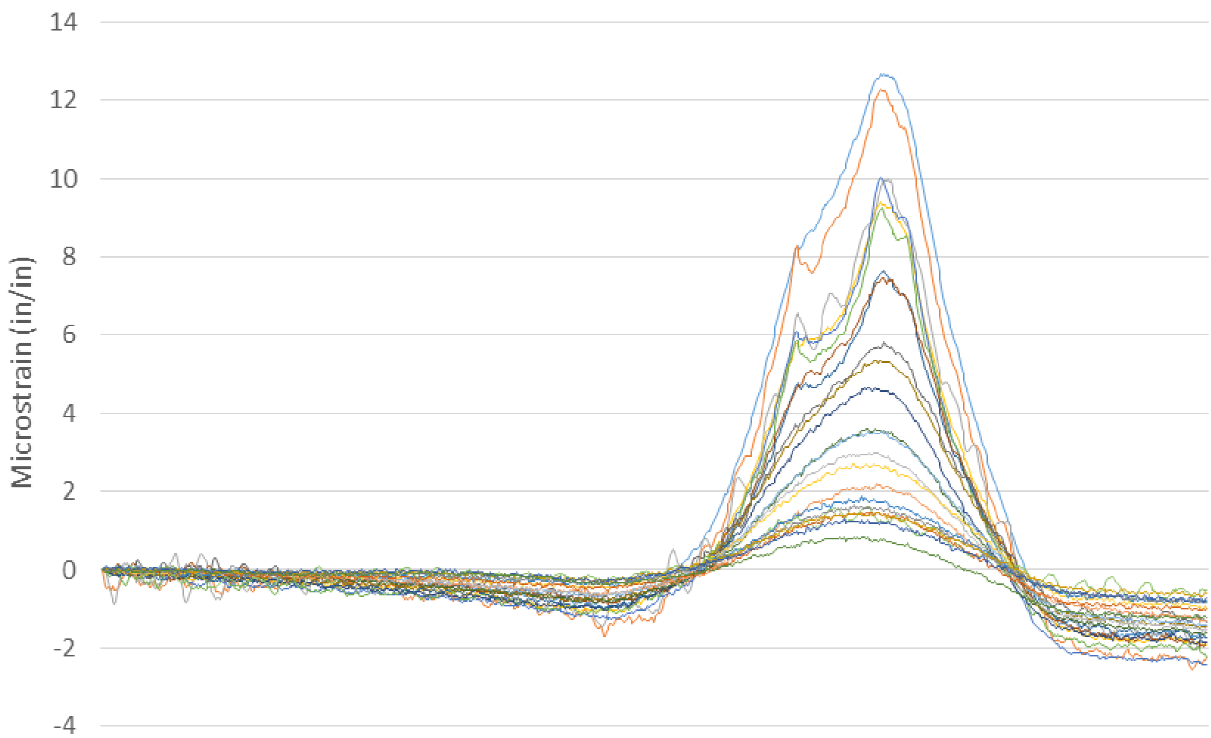


Figure 15. STH 19 over Token Creek Load Path 1

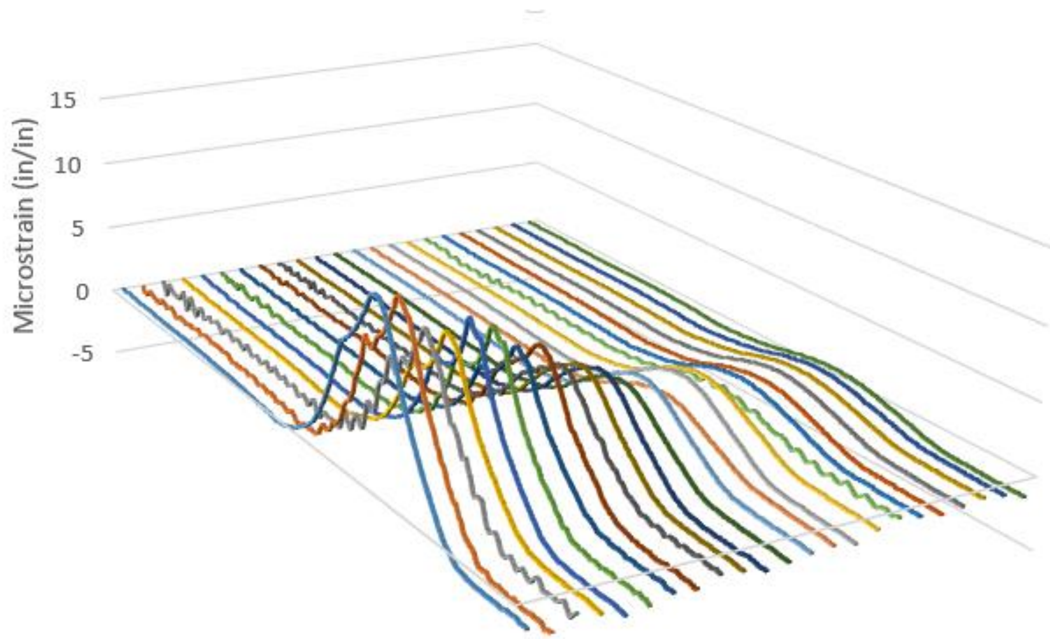


Figure 16. STH 19 over Token Creek Load Path 1 – strain distribution

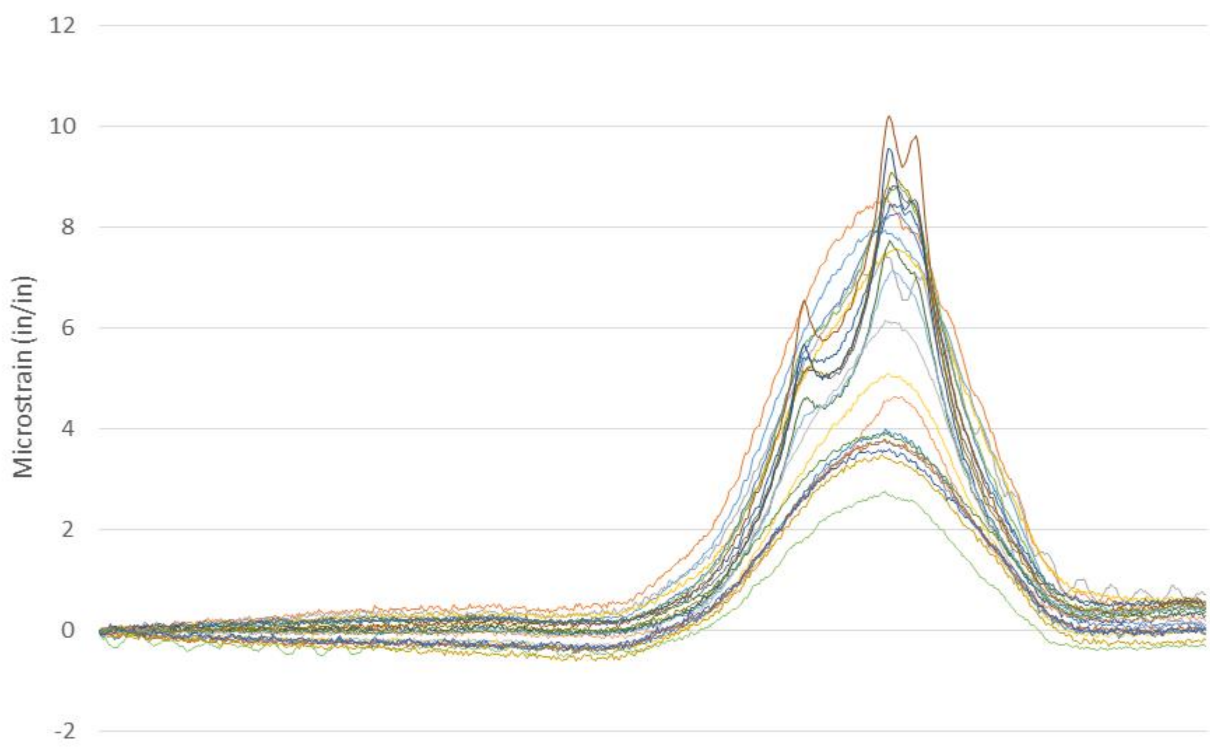


Figure 17. STH 19 over Token Creek Load Path 2

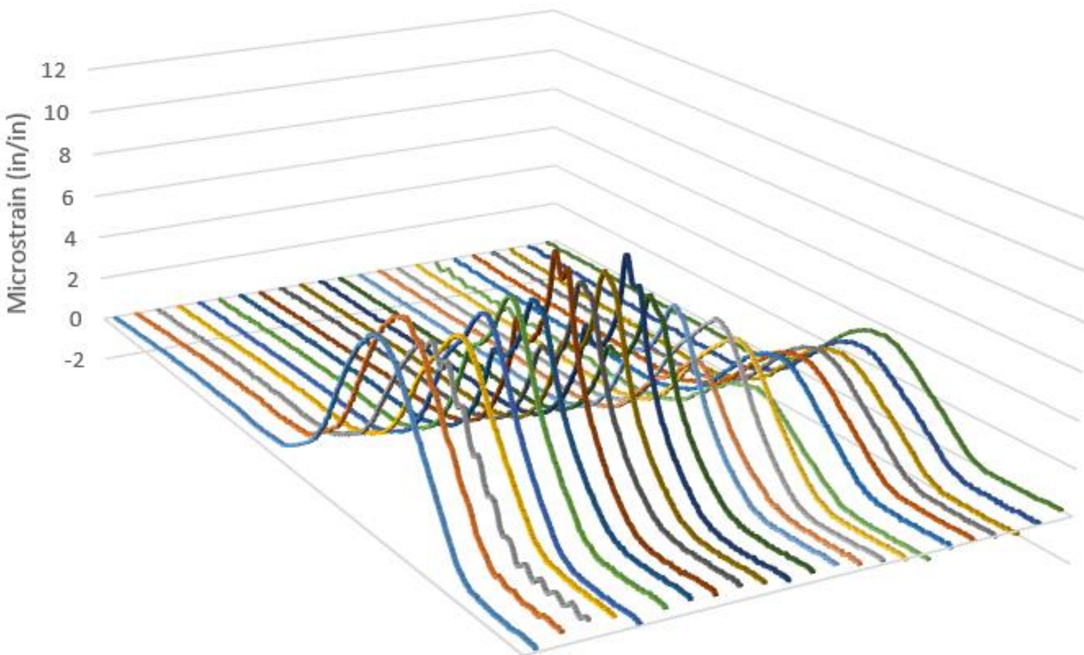


Figure 18. STH 19 over Token Creek Load Path 2 – strain distribution

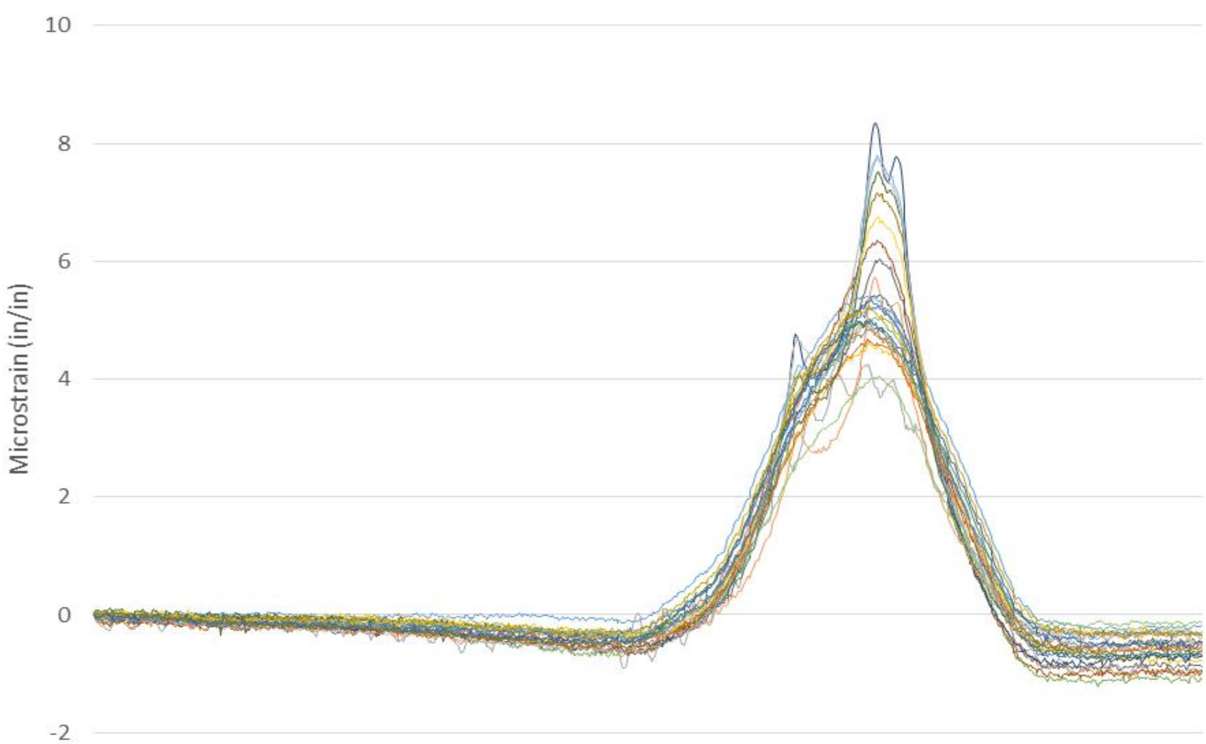


Figure 19. STH 19 over Token Creek Load Path 3

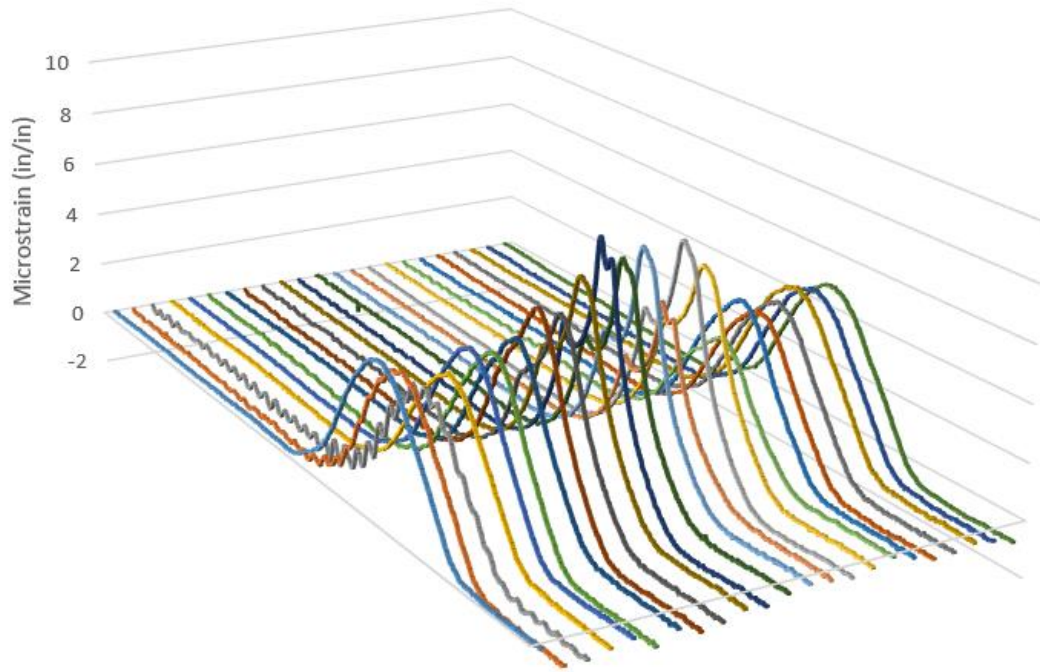


Figure 20. STH 19 over Token Creek Load Path 3 – strain distribution

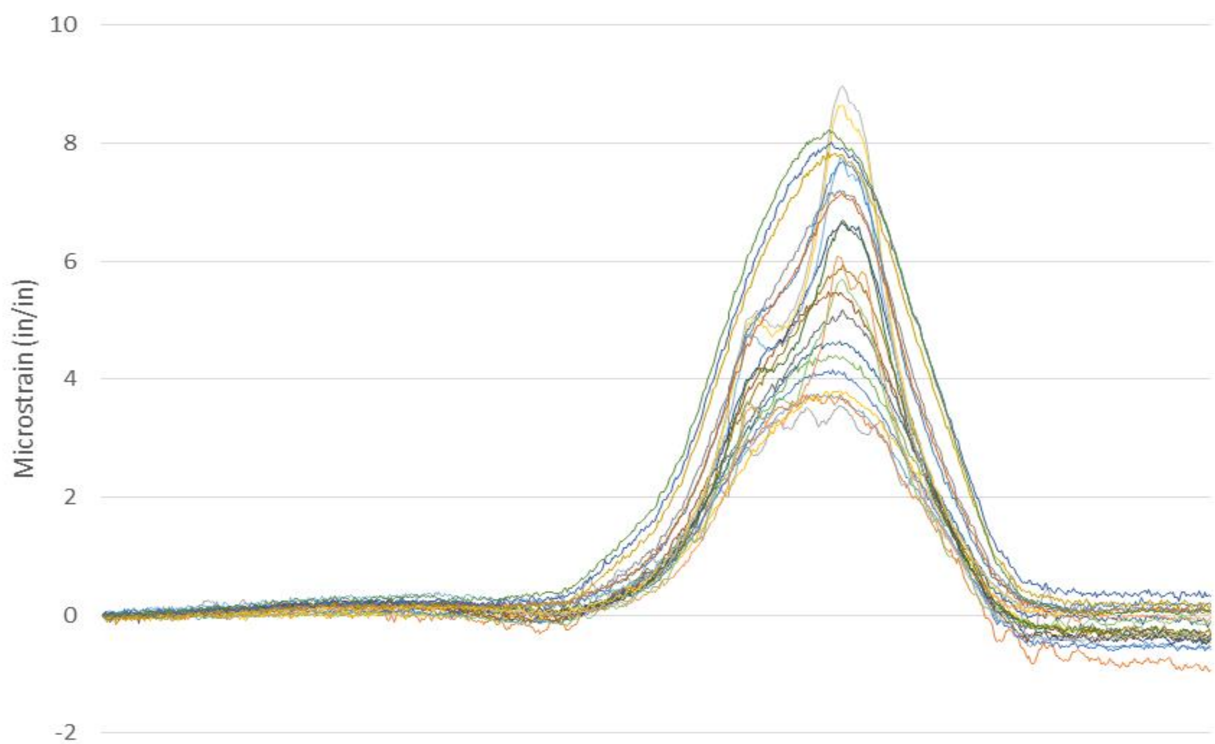


Figure 21. STH 19 over Token Creek Load Path 4

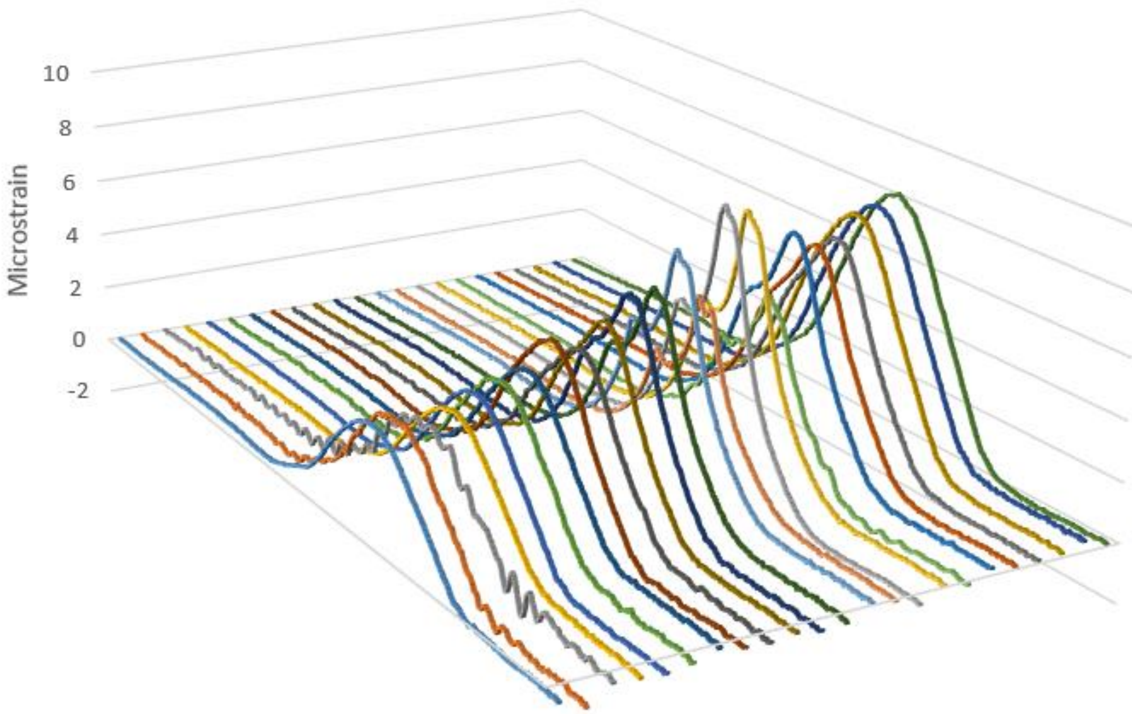


Figure 22. STH 19 over Token Creek Load Path 4 – strain distribution

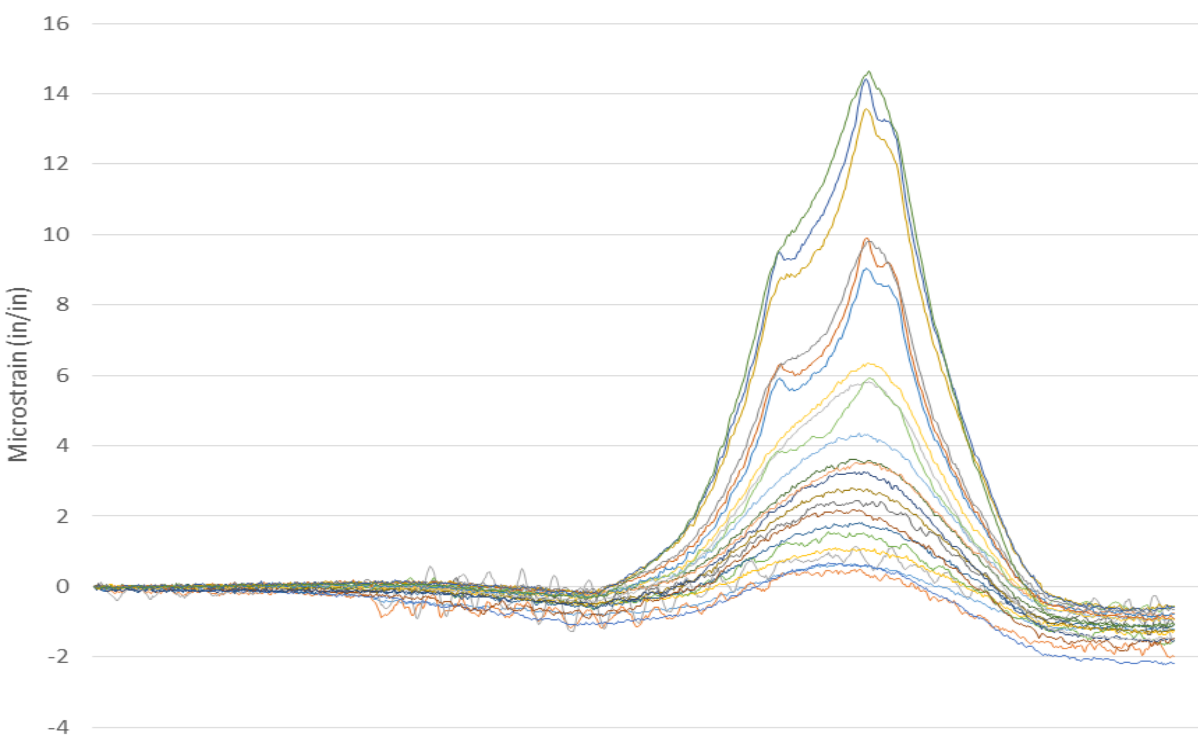


Figure 23. STH 19 over Token Creek Load Path 5

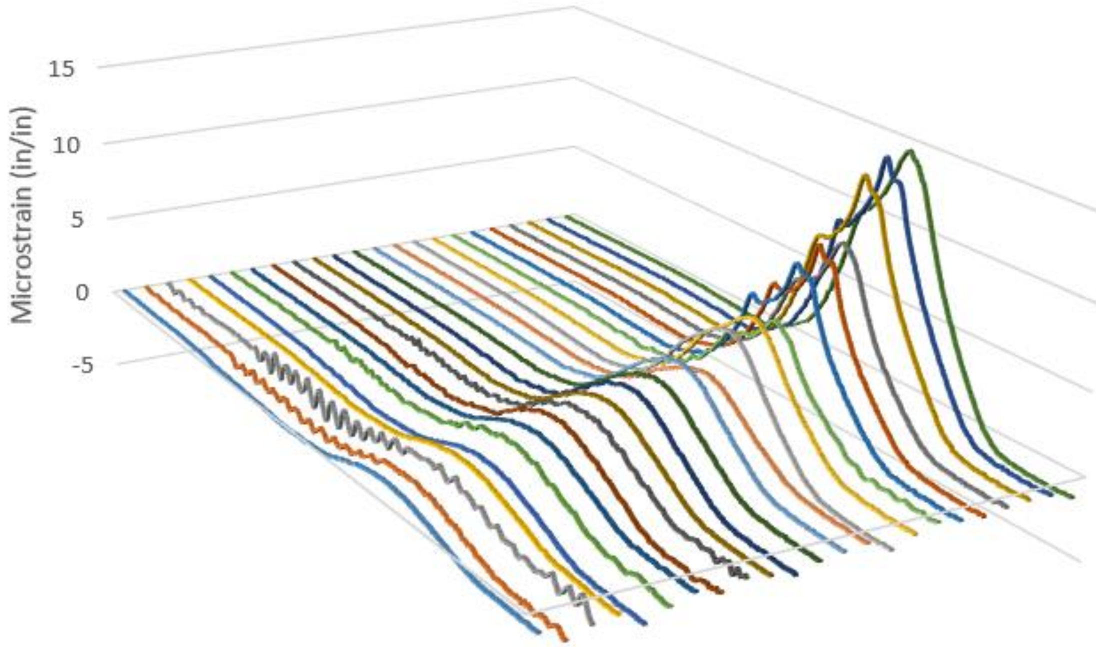


Figure 24. STH 19 over Token Creek Load Path 5 – strain distribution

Conclusions and Recommendations

The capacity and serviceability of the bridge appear to be well above that required for legal load limits given the peak strain values measured when subjected to a loaded truck of nearly 52 kips. It is possible that a more refined design, one that reduces the section size, could be implemented with the same success as the current design. Reducing the section size would yield smaller, lighter sections more easily managed during construction. A reduction in cost is also likely. However, a reduction in cross-section geometry could also result in a decrease in overall performance. The risk of any loss of performance should be weighed against any potential cost savings.

The stiffness of the bridge and the performance of the cast-in-place concrete allows for excellent lateral load distribution as is evident from the load test results. From that perspective, the observed level of in service performance, and relative simplicity of the concept, the design should be considered for more widespread implementation. If further bridges are to be constructed with inverted Ts, it may be wise to consider using these load test results to improve the load distribution currently specified in codified documents. Also, it may be advisable to use a fiber-reinforced concrete for the cast-in-place portion. Such fiber reinforced concretes have excellent crack control characteristics.

Using precast inverted Ts as a stay-in-place form and structural component of the superstructure is a viable design that performs well under load and likely possesses long-term durability. Even more, if the design is further refined, construction could become more economical and quicker thereby becoming a practical solution for accelerated bridge construction. The researchers recommend that the use of inverted-T bridges continue.

B-05-0613: US 41 over Ashwaubenon Creek, Ashwaubenon, WI – Use of Inverted T-beams

General Information

Bridge B-05-0613 carries US Highway 41 over Ashwaubenon Creek and consists of an innovative inverted T-beam superstructure that was constructed in 2012. The bridge is approximately 55’ long, 294’ 4” wide, and has 12 degrees of skew. Each of the precast beams are a nominal 6’ wide with the interior beams all having the same cross section and the two beams having a slightly different cross-section that accommodates a curb.

Description of Innovative Feature

The precast inverted T feature of the Ashwaubenon Bridge, seen in Figure 25, was first used within the United States in Minnesota.

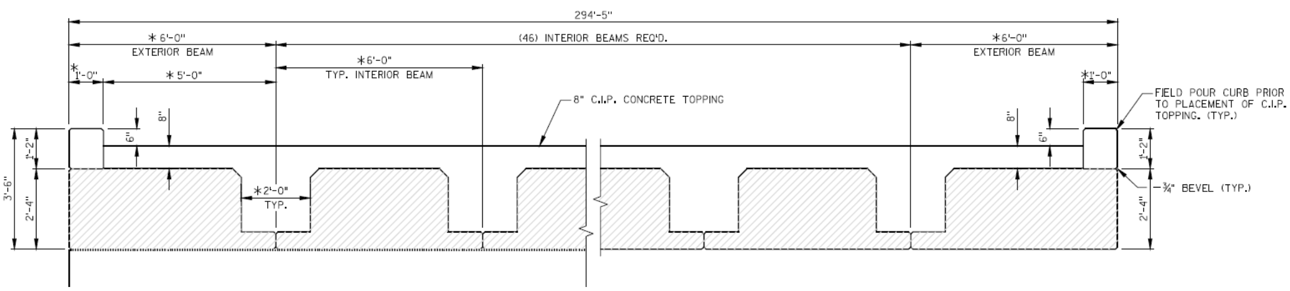


Figure 25. US 41 over Ashwaubenon Creek inverted T superstructure

The feature allows for accelerated construction, smooth joints between panels, and improved load distribution across panels. Precast inverted Ts are set side by side then topped with cast-in-place concrete to fill the voids between panels and create the wearing surface. The two elements are positively attached using reinforcement extending from the precast concrete to the cast-in-place concrete to create a shear transfer mechanism and therefore, a composite section. Reflective cracking at the joints can be minimized and likely eliminated altogether due to the significant depth of concrete placed at the joint in comparison to non-inverted T precast deck elements.

Use of Innovative Feature

Inverted Ts have become more commonly used in short-span bridge replacements where accelerated bridge construction is also a consideration. The precast inverted Ts are easily assembled on site using traditional construction machinery. The inverted Ts act in a structural capacity to support the cast-in-place concrete topping, and they act compositely with the topping upon curing to form the final bridge superstructure. The use of the precast elements in this way eliminates the need for traditional bridge deck formwork, thus saving the associated time and expense. Additionally, the fact that the primary tension carrying portions of the superstructure (i.e., the inverted Ts themselves) are precast typically means that a higher quality of concrete/curing was used. Such enhanced concrete properties can be a contributing factor to the relatively long-life of precast elements as compared to their cast-in-place counterparts.

Field Results

Visual Inspection Results

Visual inspection was completed of the top and underside of the bridge deck. The inspection revealed no signs of distress or cracking in either location. However, the top side inspection was limited to the vantage point from the roadway shoulder given that traffic was not controlled and the roadway was in use. See Figure 26.

Figure 26. US 41 over Ashwaubenon Creek roadway surface condition

During visual inspection, no evidence of extensive cracking was observed—neither in the precast concrete nor in the cast-in-place concrete. When viewed from above, no signs of reflective cracking were observed. By all accounts the inverted T panels remain in excellent condition.

Field Test Results

Several of the precast inverted Ts were instrumented with strain gages and deflection gages to determine how live loads are being transferred laterally, the stress at the bottom of the panel when subjected to live loads, and if any differential deflection is occurring between panels.

The instrumented panels included five panels directly beneath the two rightmost 12’-0” northbound lanes; each panel measures 6’-0” wide. The instrumented panels are shaded in Figure 27.

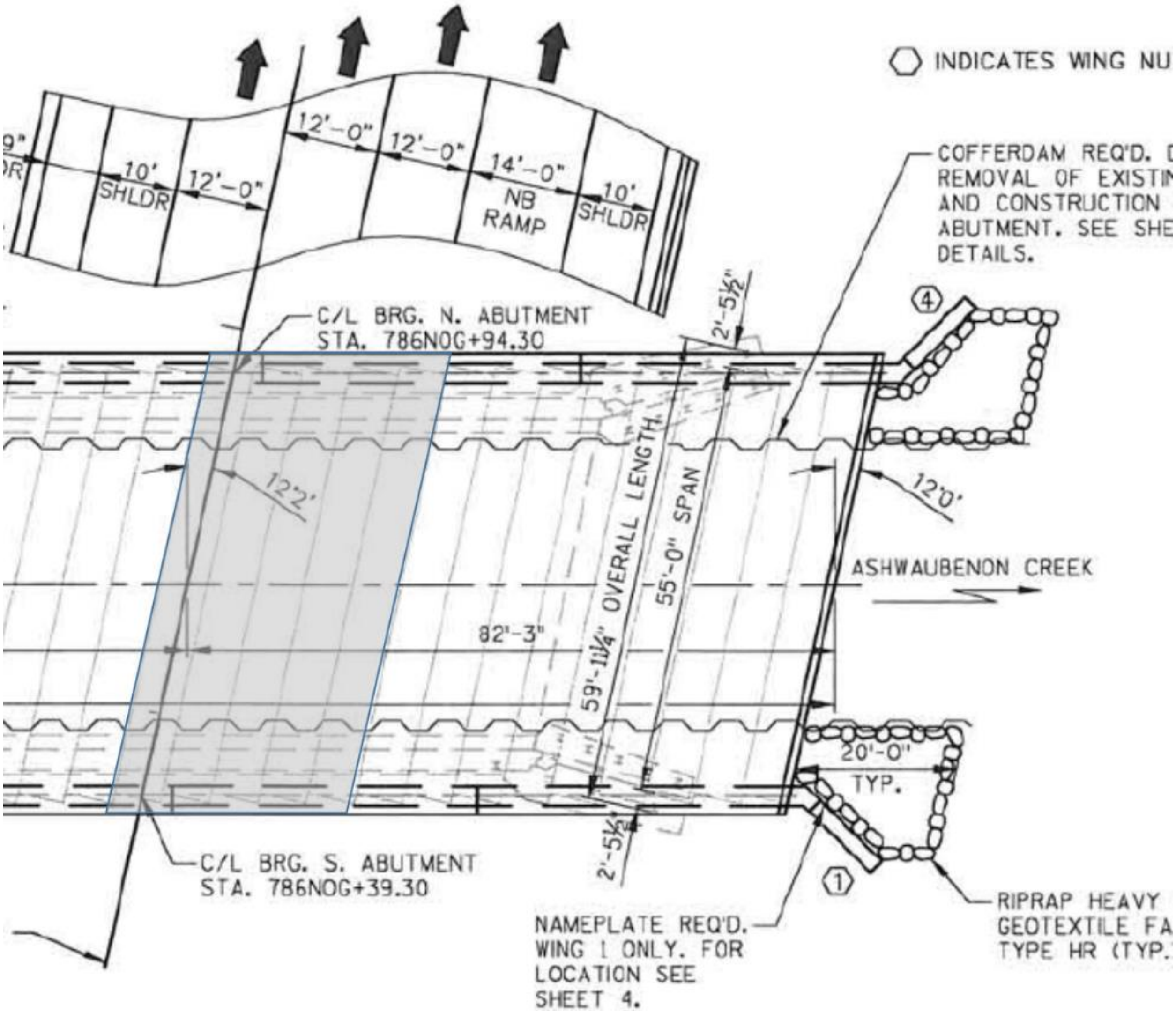


Figure 27. US 41 over Ashwaubenon Creek instrumented panels

Three strain gages were placed at the midspan of each panel with one of each near the panel's west edge, centerline, and east edge. To determine differential deflection between panels, deflection gages were placed at the panel joint very near the strain gages. Figure 28 and Figure 29 illustrate each of the gage locations and the photograph in Figure 30 shows the gages in their final position.

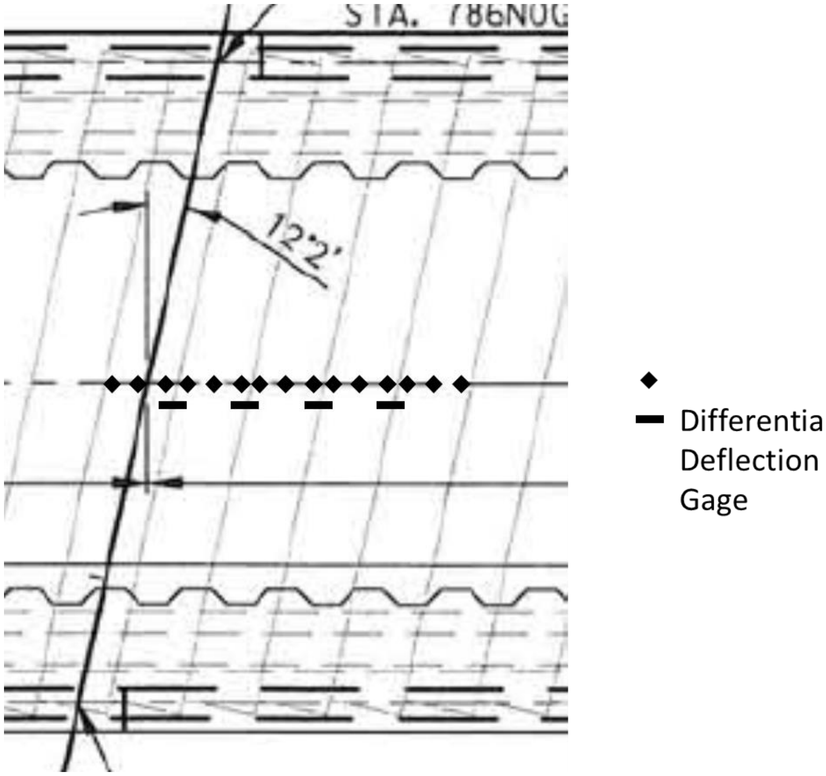


Figure 28. US 41 over Ashwaubenon Creek instrumentation locations - plan view

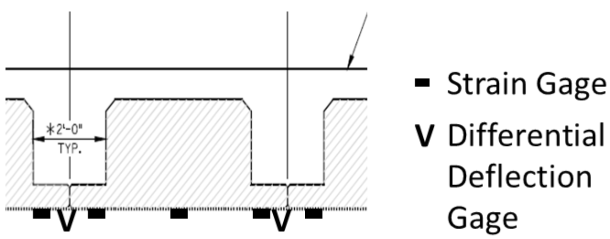


Figure 29. US 41 over Ashwaubenon Creek instrumentation locations - typical cross-section

Figure 30. US 41 over Ashwaubenon Creek panel instrumentation

The researchers collected data for approximately 15 minutes during rush hour while ambient traffic passed overhead. Numerous vehicles, many of which were large trucks (semis, dump trucks, etc.), passed during this period. See Figure 31 for a photograph of a typical truck.

Figure 31. US 41 over Ashwaubenon Creek ambient traffic load events

Although the characteristics of the vehicles were not known, the cross-section of vehicles appeared to be representative of traffic on a State Highway.

The results depict how the bridge is behaving under ambient traffic. As shown in Figure 32, the strain history is characterized by individual peaks, which indicate vehicle events; larger peaks indicate heavier vehicles, while smaller peaks indicate lighter vehicles.

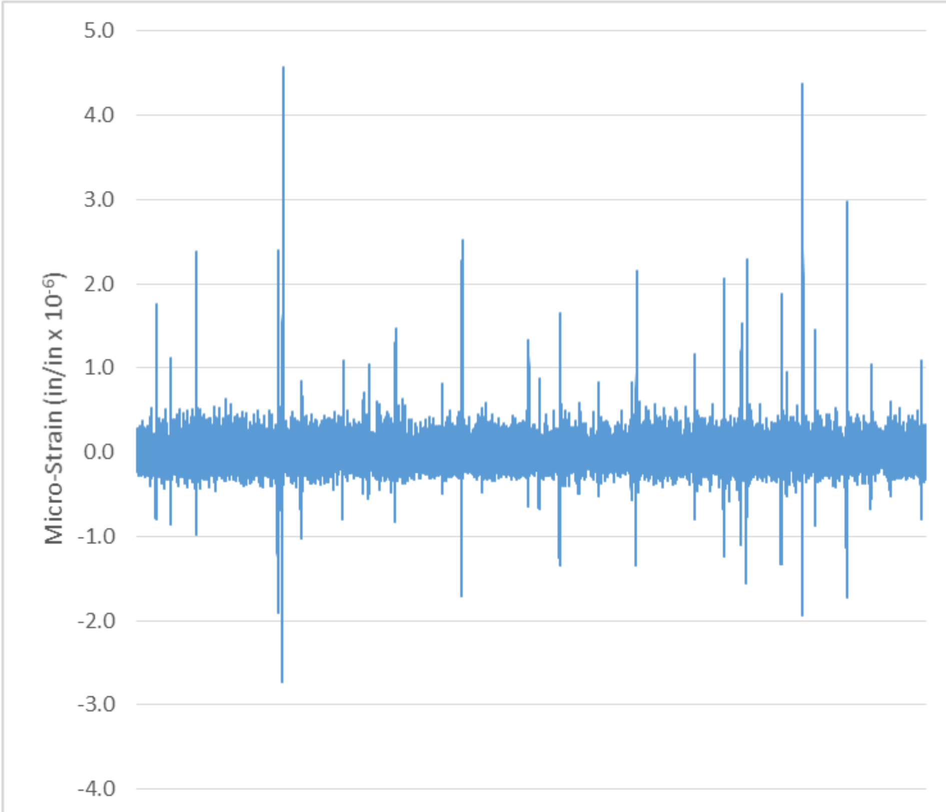


Figure 32. US 41 over Ashwaubenon Creek strain time history

Although it is clear that individual vehicles were detected, note that the greatest recorded strains were relatively low in comparison to what one might expect at the midspan of a concrete girder on a typical highway bridge (i.e., concrete girder/concrete deck). The strain history of this particular gage did not exceed five microstrain. In terms of stress, this is approximately a 15 to 20 psi tensile stress. In other words, the strain and corresponding stress level is quite low considering the loads that the bridge is carrying.

The vehicle events that resulted in the four highest recorded strain values were extracted from the data. A plot, shown in Figure 33, was then created using the strain value of each gage at the point in time when the maximum strain was recorded.

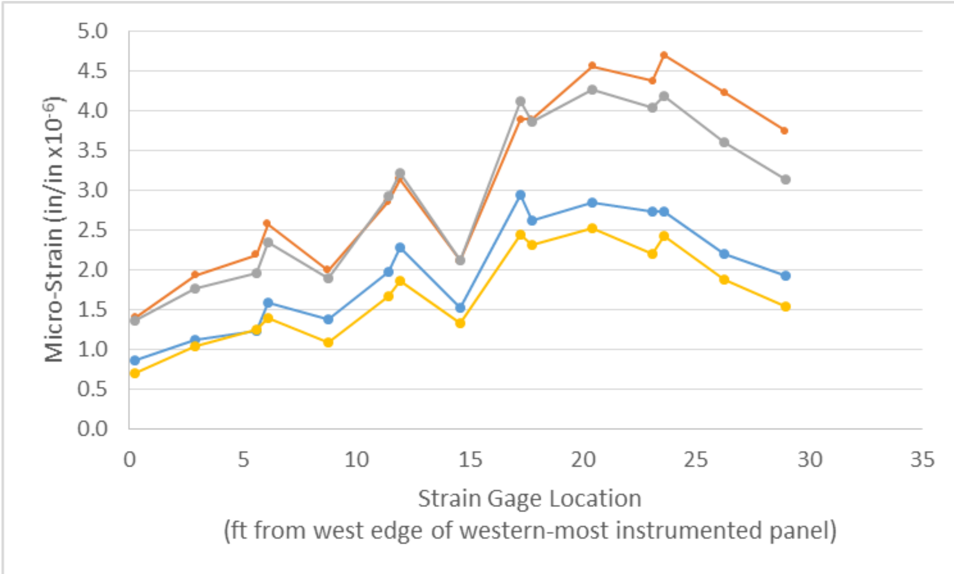


Figure 33. US 41 over Ashwaubenon Creek strain distribution under load

The lines in the plot show the transverse distribution of the four loads. The plot shows the strain gage locations relative to the west edge of the western-most instrumented panel. Each of the cases shown have similar distribution patterns and were likely the result of a heavily loaded vehicle traveling in the eastern-most travel lane. The load distribution does not strictly follow a gradually increasing nor decreasing trend. Nonetheless, the overall trend indicates that the load is being shared amongst multiple panels load sharing among the adjacent panels. In addition to the global distribution of live loads, Figure 33 shows that strains on both sides of the joint were basically identical. This, coupled with the differential deflection data discussed next, indicates that the entire bridge is basically acting as one monolithic unit.

The differential deflection measurements indicated essentially no differential movement between adjacent panels (maximum values approximately 1/1000th of an inch). The measurements for one typical gage are shown in Figure 34.

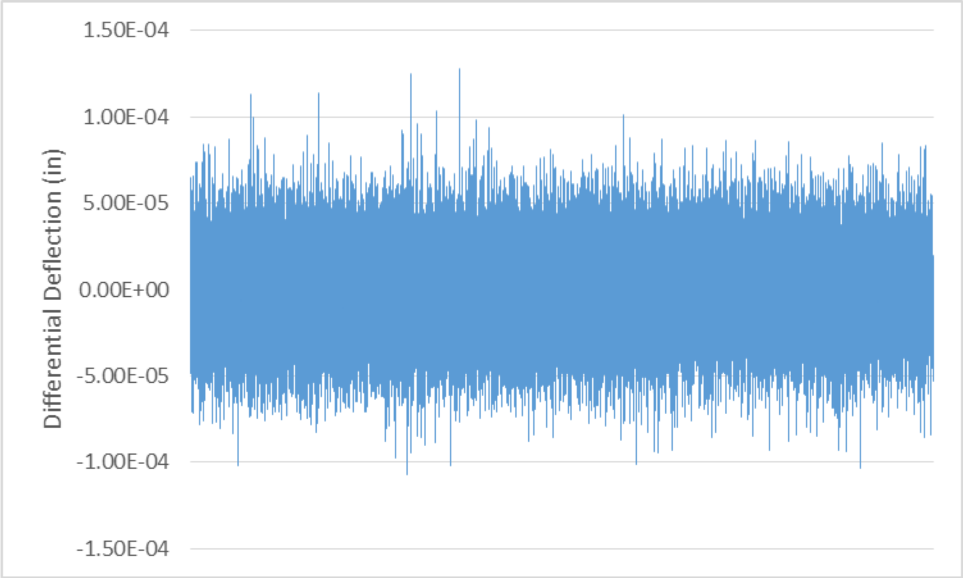


Figure 34. US 41 over Ashwaubenon Creek differential deflection under load

It seems that the combination of cast-in-place concrete and shear transfer reinforcement is acting like a fully composite system. Even more, the depth of section relative to the span length has created a very stiff superstructure with stress/strain levels on the bottom surface being very close to zero.

Conclusions and Recommendations

A review of the bridge condition indicated that the bridge is functioning quite well under the loading conditions to which it is subject. No signs of degradation or distress were apparent and the load test indicates that the capacity of the bridge is much greater than the demand currently being placed on it. Loads are distributed transversely quite well, indicating the concrete topping suitably provides the stiffness required to tie each of the panels together.

The innovative method of using inverted Ts lends itself to being considered for use in accelerated bridge construction projects. The relative simplicity of construction and the ability to eliminate the use of formwork can expedite the overall construction time. Even more, the machinery required to construct a bridge of this type is not atypical of machinery otherwise used for traditional bridge construction.

It is recommended that the design be further refined to reduce potential overdesign. This could lead to additional cost savings simply from reducing the section size of the inverted Ts and creating more manageable sizes, which can be more easily constructed.

The use of precast inverted-T bridges is recommended for continued use in Wisconsin. To ensure the long-term durability of such inverted T beam bridges, it may be wise to consider the use of fiber reinforced concrete in all such applications.

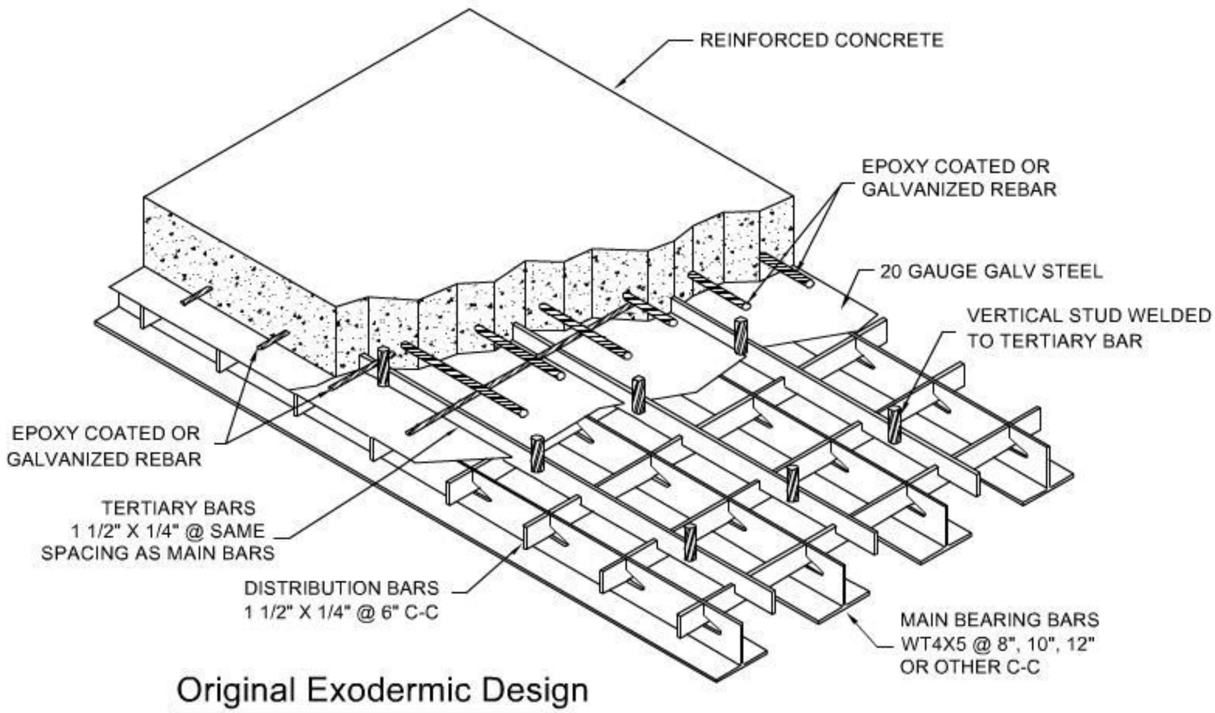
B-05-0311: US 141-Main Street over the Fox River,
Green Bay, WI – Use of Exodermic Deck

General Information

As a result of shifting piers, the original Main Street bascule bridge in Green Bay, WI, was not operating properly and thus closed in 1995. The original bridge, constructed in 1929 and consisting of an open grid steel deck, was reconstructed in 1998. As an alternative to the rough-riding, noisy, open grid steel deck, an exodermic deck system was proposed and ultimately constructed. The use of this type of deck was the first in the State of Wisconsin.

Description of Innovative Feature

The original exodermic deck design, shown in Figure 35, which was used for the Main Street Bridge, consists of a prefabricated, unfilled steel grid on which galvanized sheet steel is welded.

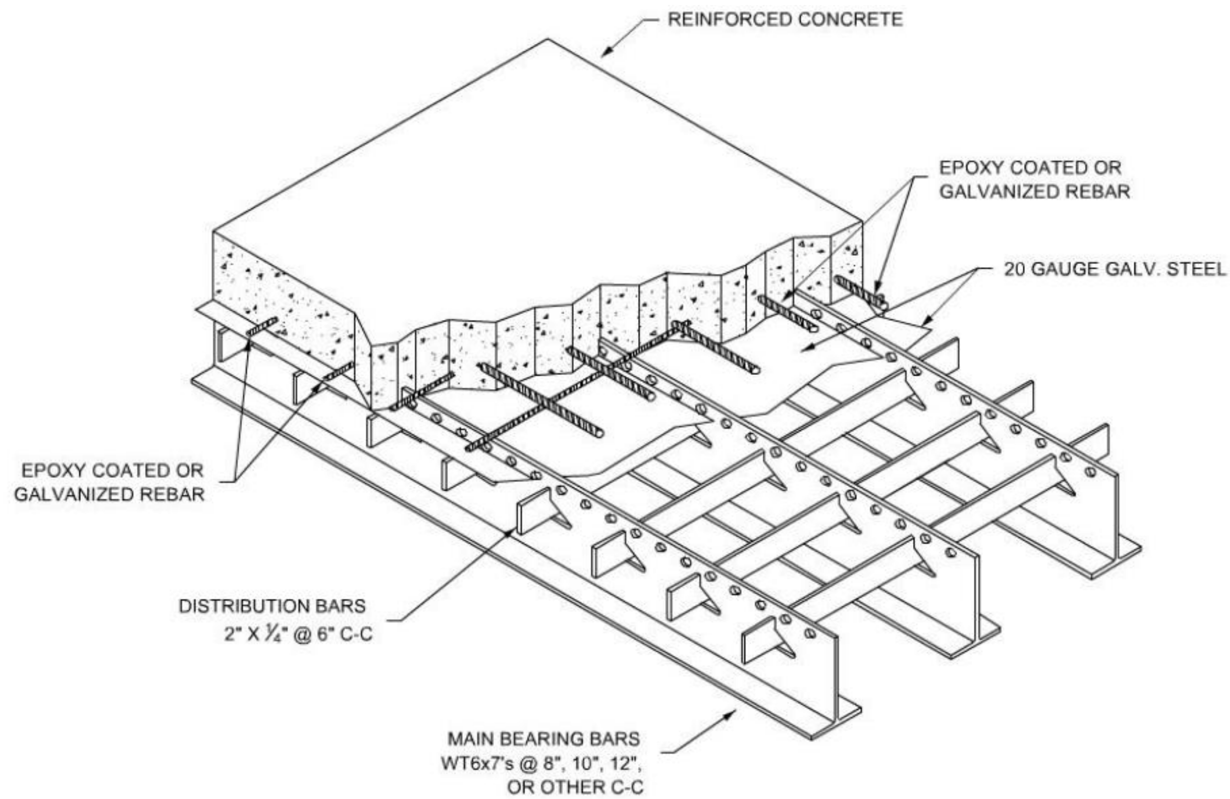


Battaglia and Bischoff 2010

Figure 35. US 141-Main Street over the Fox River original exodermic design

A reinforced concrete deck 3 to 5 inches thick is cast in place on top of the galvanized sheet. Vertical studs are welded to the steel grid and extend vertically into the concrete deck. The system takes advantage of the inherent material properties of steel and concrete: the steel is primarily in the tension zone while the concrete is in the compression zone.

An updated design, shown in Figure 36, which modifies the connection between the steel grid and the reinforced concrete deck, has since been created.



Battaglia and Bischoff 2010

Figure 36. US 141-Main Street over the Fox River updated exodermic deck design

In lieu of the vertical welded studs, a portion of the steel grid, which is fabricated with 3/4" diameter punched holes along its uppermost edge, is extended vertically into the concrete deck. This connection provides for shear transfer and, thus, the composite action required between the two elements.

Use of Innovative Feature

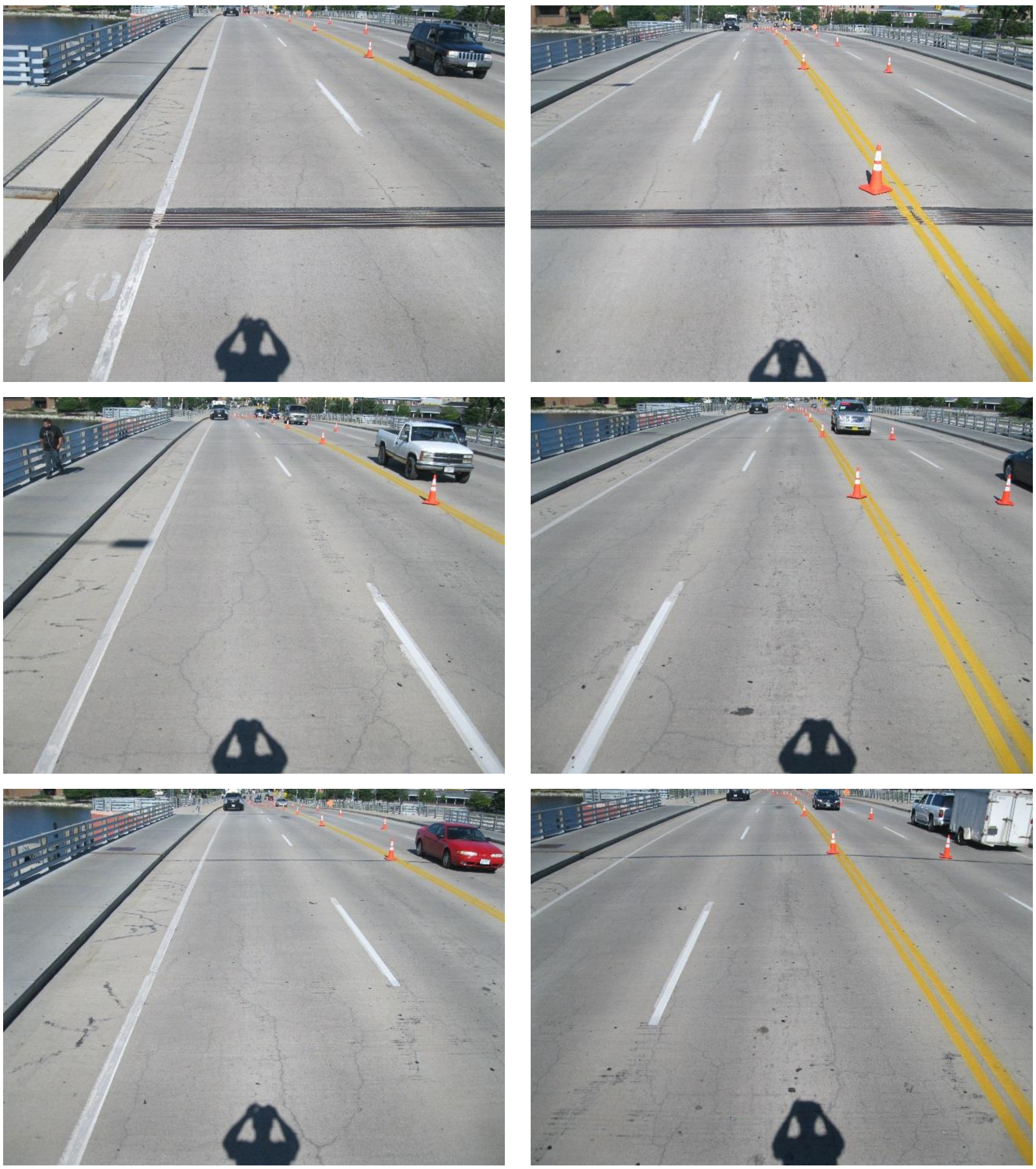
The use of an exodermic deck system eliminated the use of an open grid steel deck or reinforced concrete deck more commonly used on bascule structures; each traditional system has their advantages and disadvantages. Although considerably lighter weight than the full-depth concrete deck system, the open grid steel deck system typically resulted in a loud and rough crossing for motorists. Conversely, the mechanisms required to lift a full-depth concrete deck system become substantially larger due to the increased dead weight of the deck and associated support structure. To eliminate the rough and loud crossing of open grid steel deck but provide the lighter weight system that would not require the larger lifting mechanisms, the exodermic deck system was implemented.

Field Results

On August 11, 2015, the bascule bridge deck was inspected for surface cracking, delaminations, and/or other observable signs of deterioration. The entirety of the deck was sounded and crack survey was performed. A cursory review of the underside of the bridge was also completed although accessibility did not allow for an inspection of greater depth. The results are discussed below.

Visual Observation – Deck Surface

Overall, the surface of the deck appeared in good condition with only very small surface cracking visible. Cracks of this size and extent are similar to that that might be expected on a non-exodermic deck of similar age. Previous reports have indicated this cracking was present from a very early age likely due to initial shrinkage. A series of photos are shown in Figure 37 and Figure 38, for the eastbound and westbound lanes, respectively. These photos provide an overall view of the deck condition where the surface cracking can be seen.







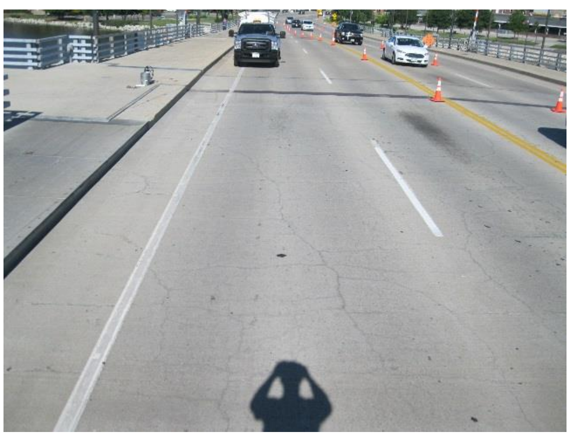
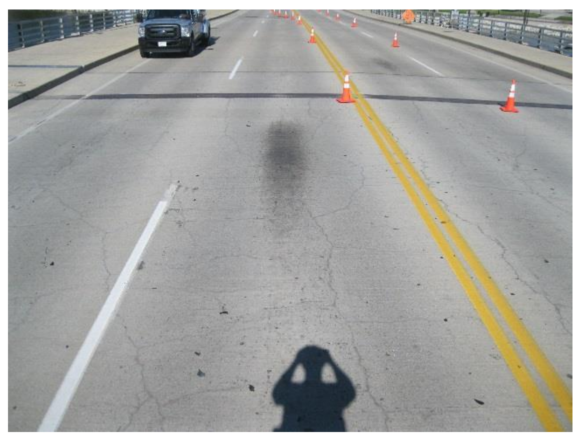
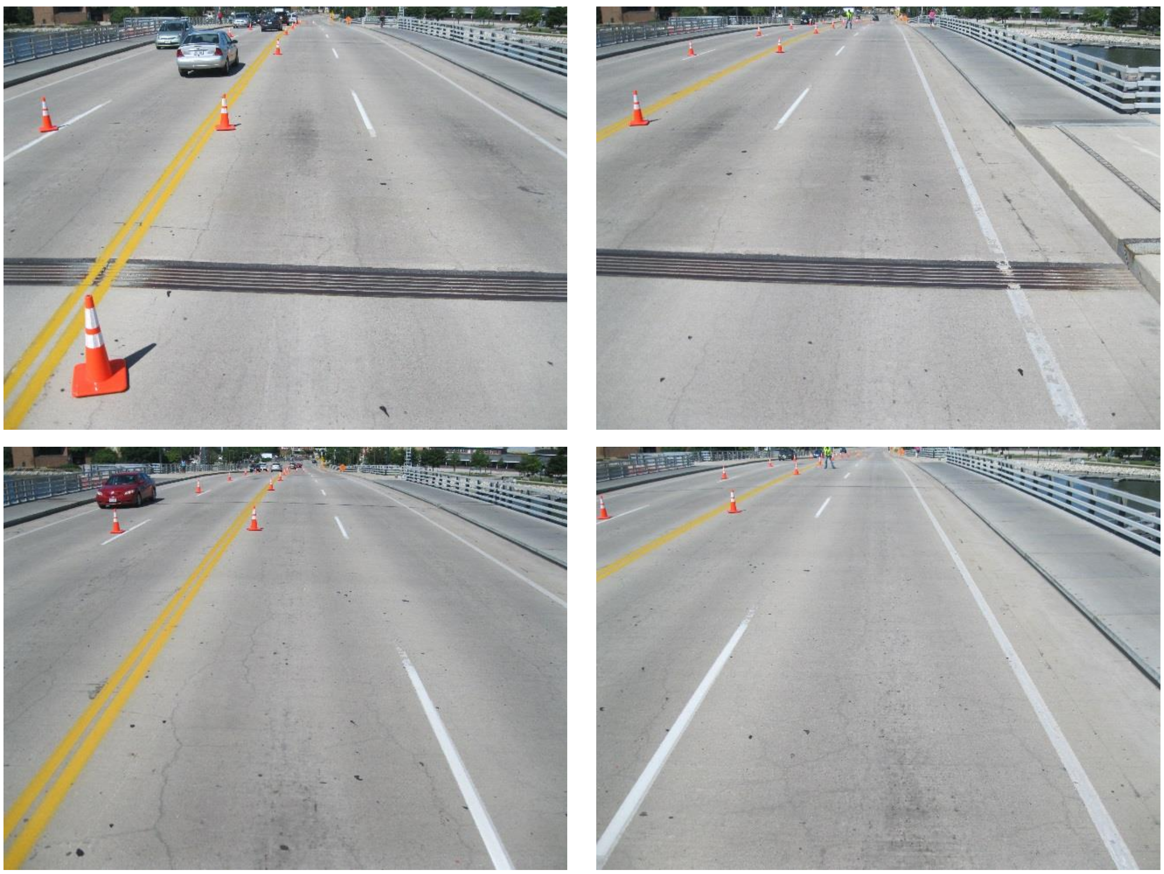
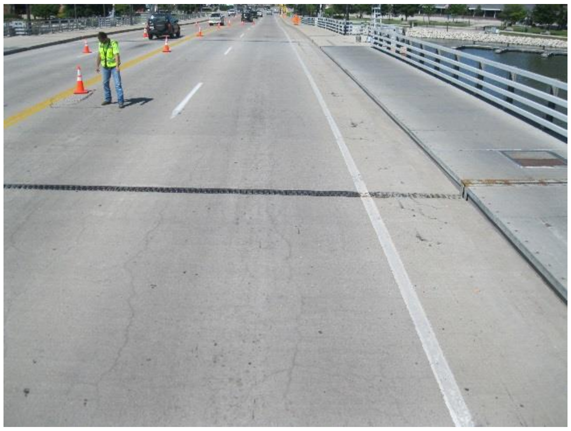



Figure 37. US 141-Main Street over the Fox River eastbound lanes looking west





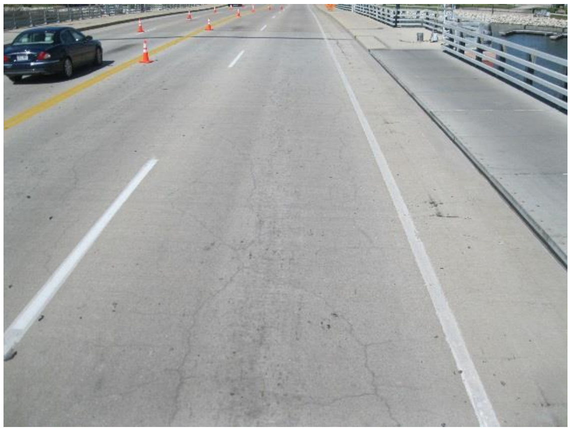


Figure 38. US 141-Main Street over the Fox River westbound lanes looking west

A close-up of a typical crack is shown in Figure 39 along with a mechanical pencil so one can more closely gauge the size.

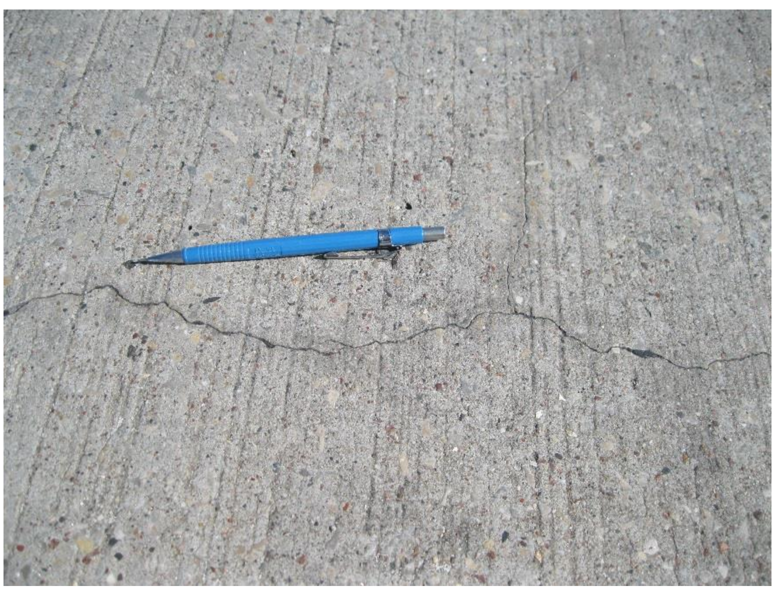


Figure 39. US 141-Main Street over the Fox River typical crack size observed

Maintenance on the bridge has included the use of a crack sealant. A majority of the observed cracks have been sealed and, to this point, the sealant appears to be performing well (i.e., fully intact within the cracks). Other cracks that have not been sealed do exist, but as indicated they were relatively small in size.

Visual Observation – Underside of Deck

Due to the limited access to the underside of the deck, many areas of the exodermic deck construction were not visible (see Figure 40).

Figure 40. US 141-Main Street over the Fox River exodermic bridge deck

As such, only a cursory review of what was visible could be completed. The visible areas, generally those areas directly above the walkways, as shown in Figure 41 did not indicate any distress or cracking and appeared to be in quite good condition.

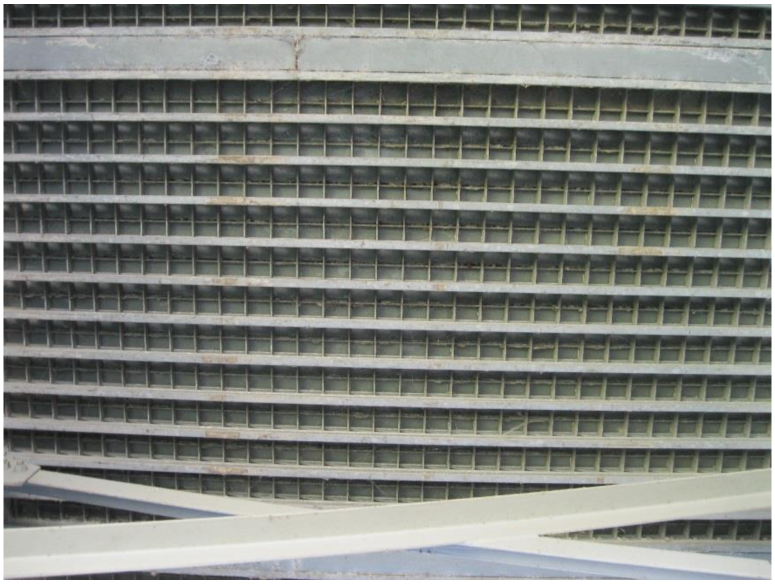


Figure 41. US 141-Main Street over the Fox River underside of bridge deck

Crack Survey

In order to better quantify the amount of deck surface cracking, a detailed crack survey was completed in four areas, each approximately 100 ft^2. The areas were randomly selected along the length of the bridge, although an area was purposely selected in each of the travel lanes. The approximate locations of the areas selected are indicated and labeled in Figure 42; photographs and crack maps of each area are given in Figure 43 through Figure 46.

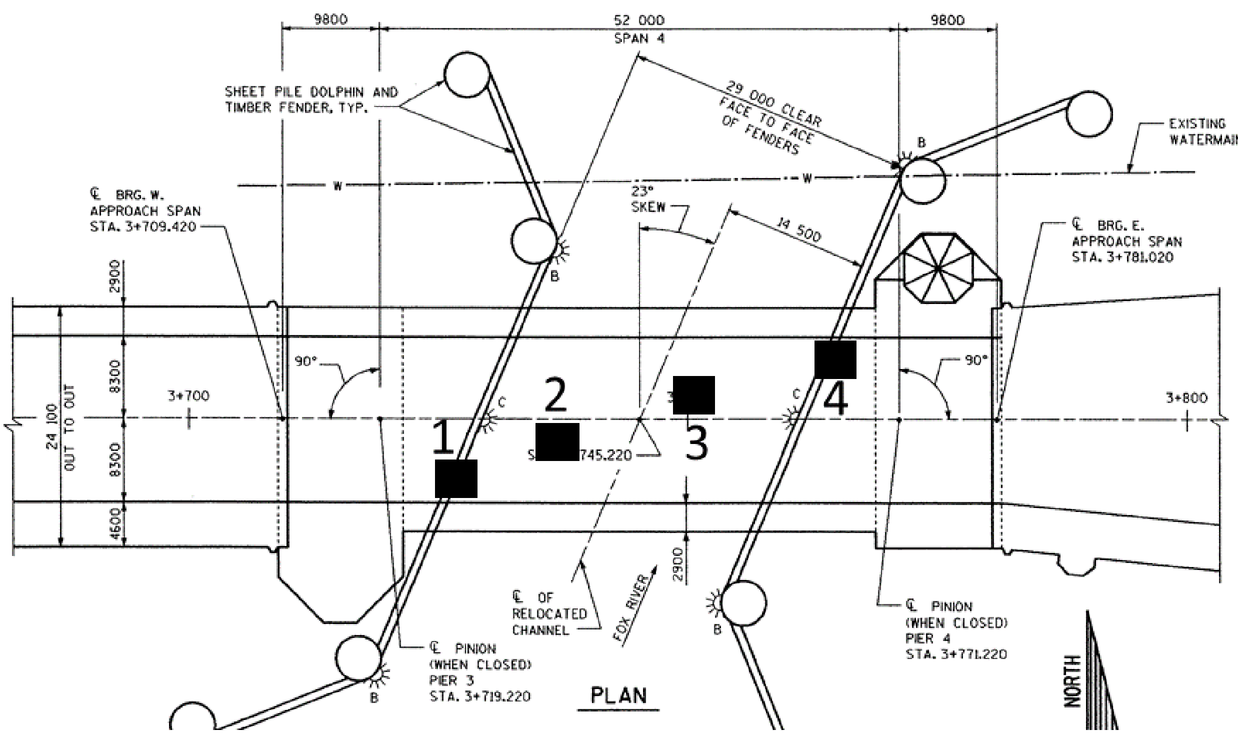


Figure 42. US 141-Main Street over the Fox River crack survey locations

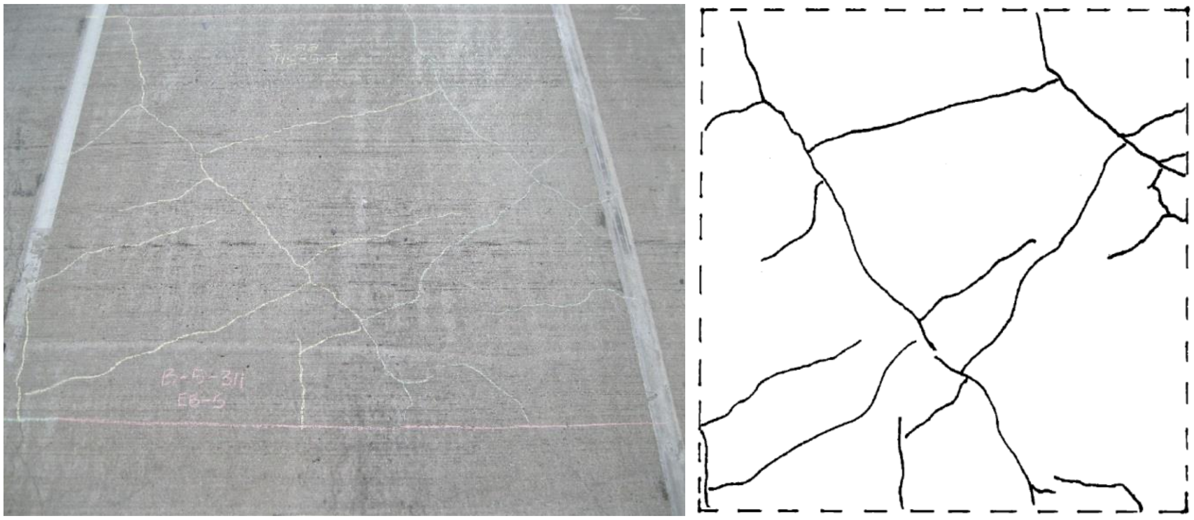


Figure 43. US 141-Main Street over the Fox River Area 1: south lane of eastbound traffic

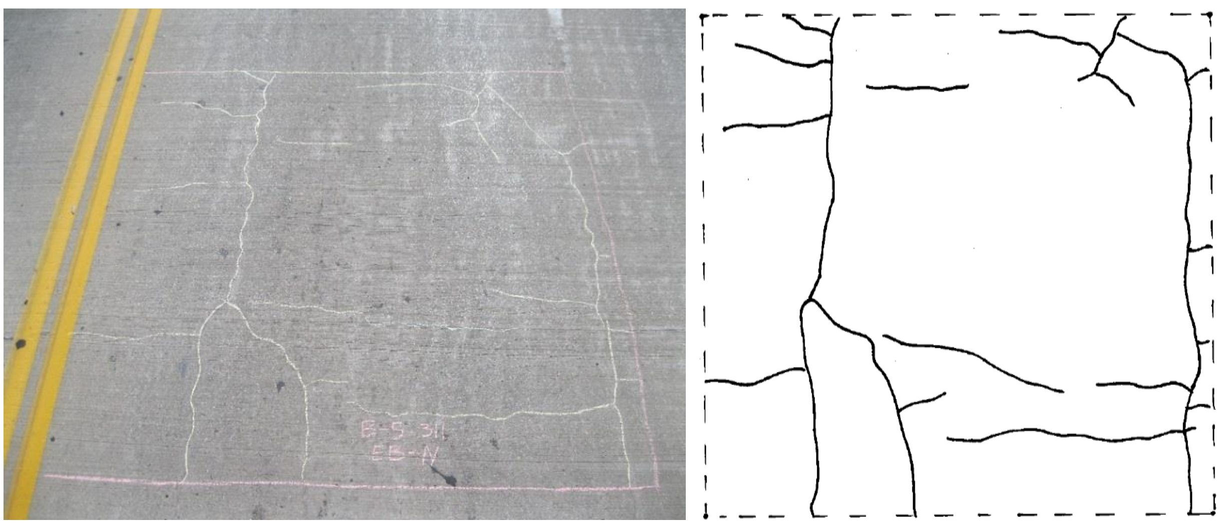


Figure 44. US 141-Main Street over the Fox River Area 2: north lane of eastbound traffic

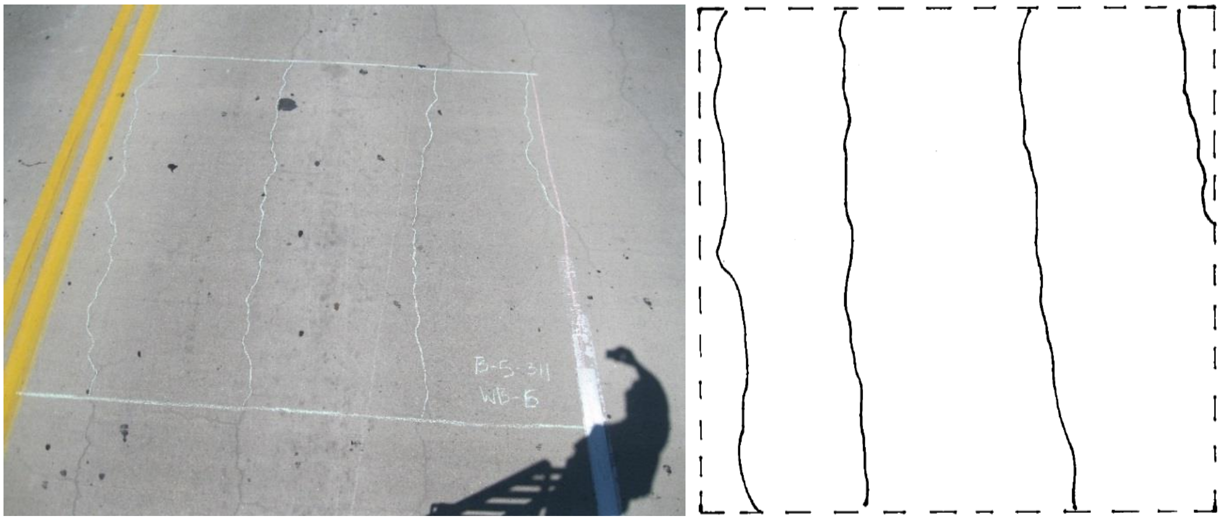


Figure 45. US 141-Main Street over the Fox River Area 3: south lane of westbound traffic

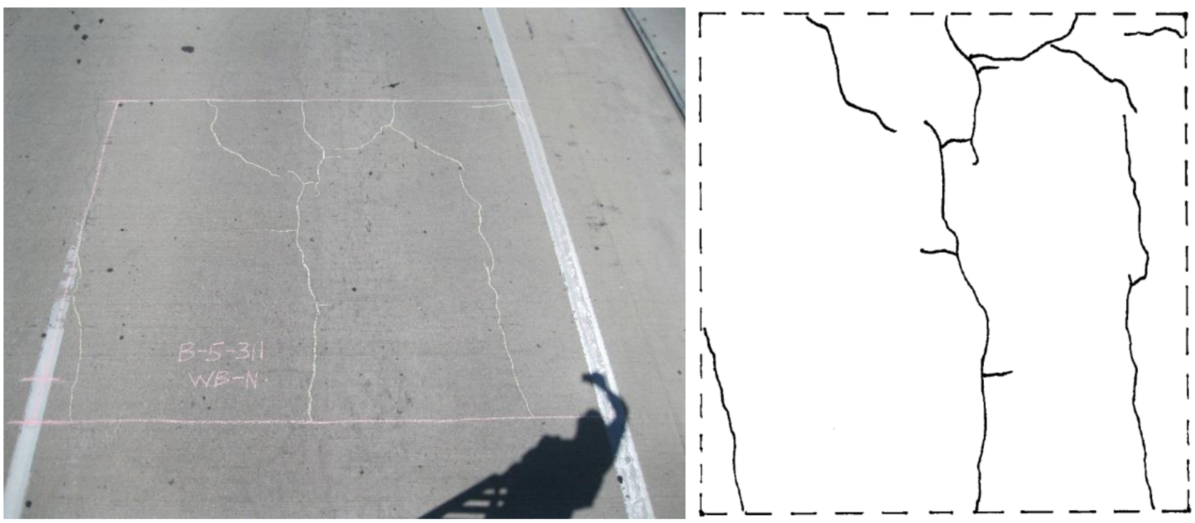


Figure 46. US 141-Main Street over the Fox River Area 4: north lane of westbound traffic

The crack patterns varied among each of the selected areas. There was not a well-established pattern in any one lane or side of the bascule opening, although a cursory review would indicate that cracks in the eastbound lanes on the western half of the bascule (1 and 2) varied with both transverse and longitudinal cracking; whereas, those in westbound lanes on the eastern half of the bridge (3 and 4) tend to be more consistently longitudinal.

As means for quantitatively assessing the observed cracks, measurements of the crack lengths were summed for each of the selected areas. The quantitative results of the crack survey are provided in Table 1.

Table 1. US 141-Main Street over the Fox River crack survey results

Survey Deck Location	Area	Total Length of Cracks	Crack Length/ Area
20’-0” from E.O.B. in South Lane of Eastbound Traffic: West half of bascule	112 sq. ft	62’-6”	0.55 ft/sq. ft
60’-0” from E.O.B. in North Lane of Eastbound Traffic: West half of bascule	108 sq. ft	58’-7”	0.54 ft/sq. ft
110’-0” from E.O.B. in South Lane of Westbound Traffic: East half of bascule	111 sq. ft	35’-1”	0.32 ft/sq. ft
160’-0” from E.O.B. in North Lane of Westbound Traffic: East half of bascule	115 sq. ft	37’-1”	0.32 ft/sq. ft

*E.O.B = West end of bridge where bascule hinges and bascule bridge deck terminates

One can see from the quantitative results that the frequency and overall length of cracking was found to be greater in the eastbound lanes on the western half of the bascule—nearly 75 percent greater. Even though there appears to be some consistency within the results from eastbound and westbound lanes and/or the east and west halves of the bascule, one is cautioned not to broadly apply this trend over the entirety of the bridge surface area given the total area surveyed is only a small percentage of the total deck surface.

Chain Sounding

The entirely of the bascule deck was sounded using a 3/8” chain. Using this method, shown in Figure 47, no areas of delamination were detected indicating no areas of significant deterioration were present.

Figure 47. US 141-Main Street over the Fox River chain sounding

Conclusions and Recommendations

As a result of shifting piers, the original Main Street bascule bridge in Green Bay was not operating properly and thus closed in 1995. The original bridge, constructed in 1929 and consisting of an open grid steel deck, was then reconstructed in 1998. As an alternative to the rough-riding, noisy, open grid steel deck, an exodermic deck system was proposed and ultimately constructed. The use of this type of deck was the first in the State of Wisconsin.

A review of the bascule structure was completed by the researchers during the summer of 2015. The bridge deck was visually inspected and checked for soundness using chain drag methods; no areas of delamination were detected. A detailed crack survey was completed in four randomly selected areas; hairline cracking was prevalent throughout the bridge deck surface. The bridge is performing well given its age and function and the deck cracking does not appear to be having a significant adverse effect on the structural integrity. The cracks are quite small and they have not led to advanced deterioration. Previous reports have indicated this cracking was present from a very early age likely due to initial shrinkage.

The exodermic design eliminates the dead weight of a full depth concrete deck structure and the roughness and noise common to open grid steel deck structures. Given its advantages over these systems and the performance of this bridge to date, the design of this bridge is recommended for future use when the added cost and complex construction warrants its use. One should note, however, that special consideration should be given to the concrete deck mix and curing methods used such that the initial concrete shrinkage cracking can be minimized.

References

Battaglia, I. K. and D. Bischoff. 2010. *Exodermic Bridge Deck Performance Evaluation*. Wisconsin Department of Transportation, Madison, WI.

B-09-0380: SH 40 over Hay Creek, Bloomer, WI – Use of GRS Abutment

General Information

Bridge B-09-0380 is located on State Hwy 40 over Hay Creek near Bloomer, WI. The bridge was constructed in 2012 to replace a single-span concrete slab bridge on timber abutments. The new structure uses geosynthetic-reinforced soil (GRS) retaining wall abutments with a cast-in-place concrete slab superstructure. The bridge measures 40' in length and 38'-6" in width.

Description of Innovative Feature

The GRS abutments consist of modular block and geosynthetic fabric reinforcement as shown in Figure 48.

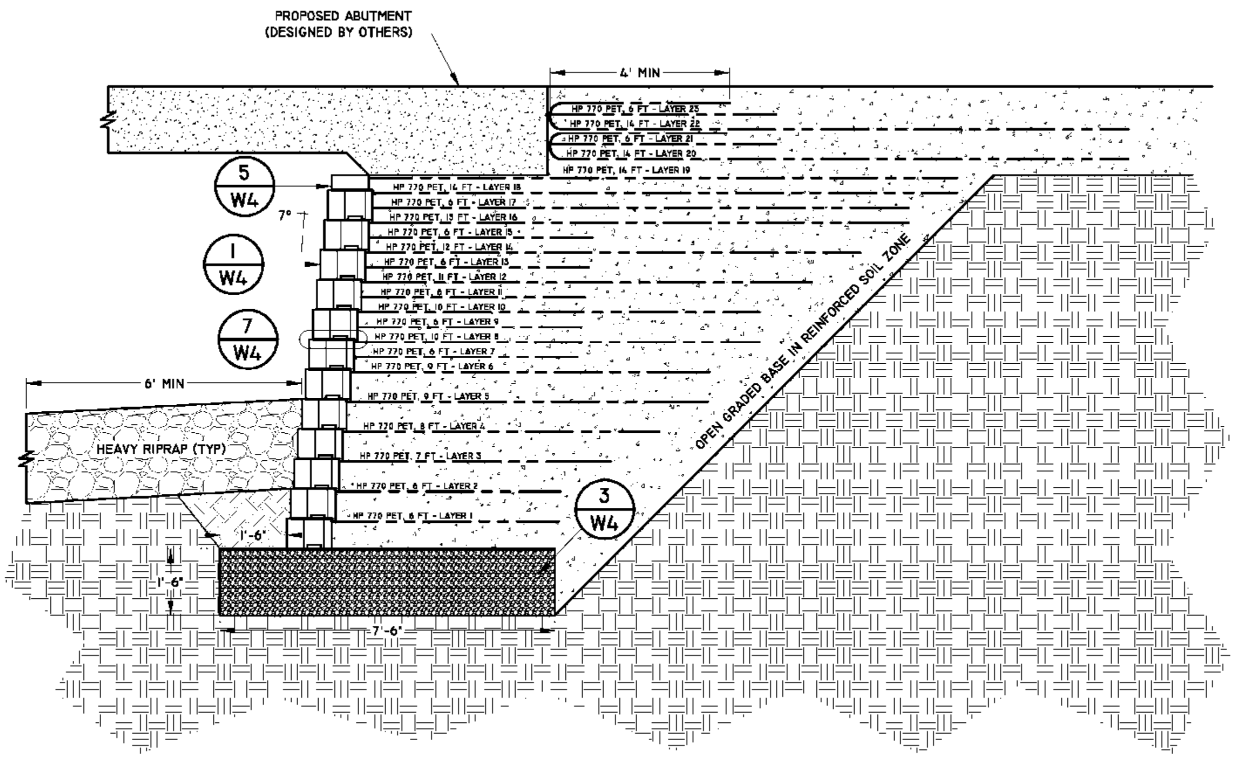


Figure 48. SH 40 over Hay Creek GRS abutment section

Collectively, the assembly provides the support for a superstructure that is consistent with the support provided by a traditional cast-in-place system. GRS abutments have been used successfully in many applications around the United States, although none have been constructed in combination with a cast-in-place slab superstructure. Benefits of using GRS abutments include relative ease to construct, generally forgiving construction tolerances, and relatively short construction times. Although not the primary reason for using GRS abutments, they are receiving much national attention for use in accelerated bridge construction (ABC) projects.

Use of Innovative Feature

By using a GRS abutment bridge, the substructure can be constructed without the use of specialized equipment and mostly by the use of simple manpower. This enables substructure construction in areas where construction abilities or contractors are limited. The bridge was constructed in a rural area that sees limited traffic. However, many of the vehicles that cross the bridge are approaching legal weight and some are even overweight due to a gravel quarry in the area. The use of the GRS abutments in this location was not absolutely necessary but allowed for the assessment of their use in Wisconsin without substantial risk given the relatively low traffic volumes. The use of GRS abutments is very attractive to local system owners who are in the habit of constructing their own bridges. The lack of the need for specialized equipment and the straightforward construction approaches make them ideal for competent, but not specialized bridge construction crews.

Field Results

Visual Inspection Results

On September 15, 2015, the bridge was visited and a visual inspection was completed. The inspection consisted of a cursory review of the GRS abutments, shown in Figure 49, and a more in depth inspection of the cast-in-place slab superstructure.

Figure 49. SH 40 over Hay Creek general bridge condition

As previously mentioned, the superstructure is the only of its kind on a GRS abutment and, as such, the interest is more in the performance of the superstructure, not the substructure.

The GRS abutment showed no signs of distress and appears to be performing quite well. In contrast, there is considerable and extensive transverse hairline cracking on the underside of the superstructure (see Figure 50 and Figure 51).

Figure 50. SH 40 over Hay Creek transverse cracking on superstructure underside

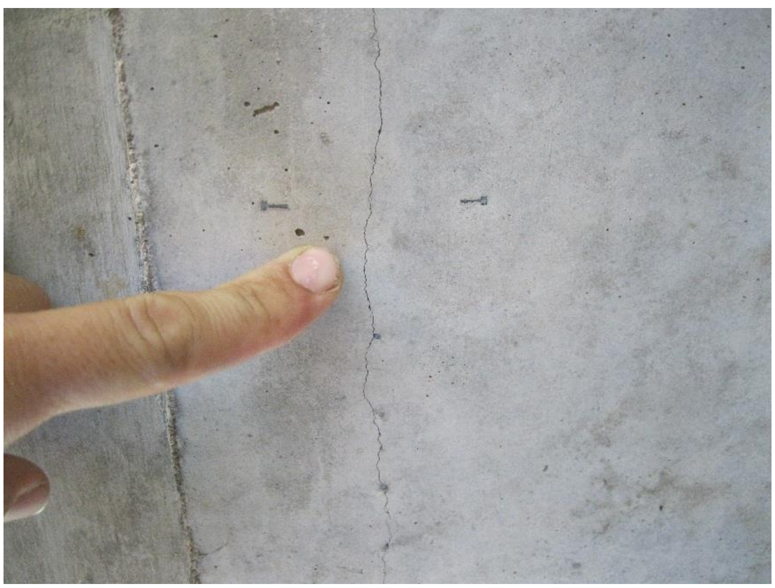


Figure 51. SH 40 over Hay Creek typcial size of transverse cracking

Crack mapping was completed on the underside of the southwest quarter of the bridge (18’-6”x 19’-1 1/2”), which was representative of the remaining bridge condition. A total of 156’ of transverse cracking was measured. The crack map is shown in Figure 52. Similar cracking was observed throughout the entire bridge length.

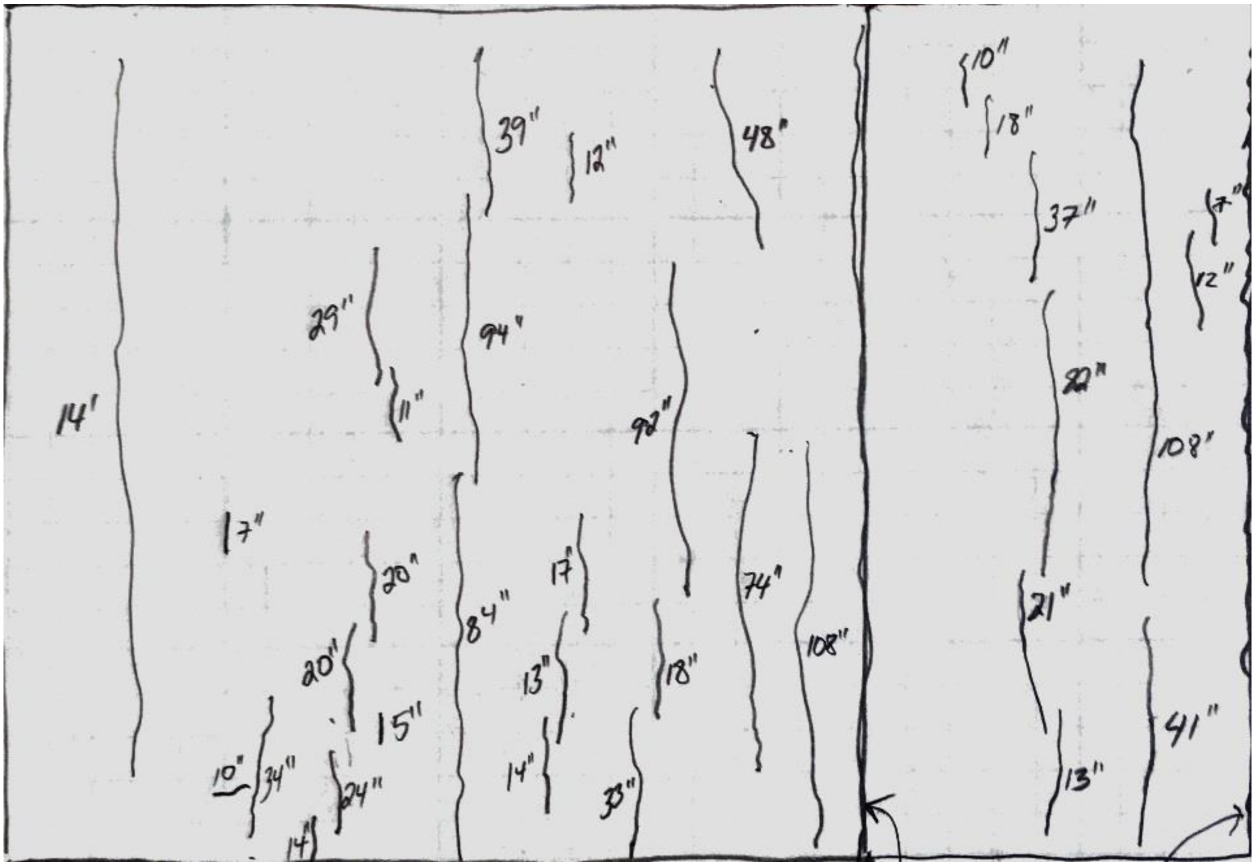


Figure 52. SH 40 over Hay Creek crack map

In contrast to the bottom surface of the superstructure, longitudinal cracking was observed on the top surface, although not to the extent of that seen on the underside (see Figure 53).

Figure 53. SH 40 over Hay Creek longitudinal cracking on deck surface

Performance Evaluation and Summary

As the innovative feature, the GRS abutments appear to be performing well. A visual inspection produced no signs of distress or under performance. The interface between the substructure and superstructure also appears to be performing well. By most accounts, the abutment performance is similar to most cases where GRS abutments have been utilized.

The superstructure, however, appears to have an unusual amount of transverse cracking, especially for a bridge of its age. It is possible the cracking is a result or combination of shrinkage and overloading. However, knowing that the bridge is quite close to a rock quarry, it is very likely that the observed bottom of superstructure cracking is the result of overstress or underdesign. It is concerning that in addition to the bottom of deck cracking that there is also a notable amount of top of deck cracking. Although specific crack widths were not measured, this top of deck cracking appears wide enough that it may be possible to provide a pathway for the introduction of water and chlorides.

Recommendations

Given the results of this study, the use of GRS abutments is recommended in similar applications (short span, rural roads). Extending the use of GRS abutments to other applications could be as successful, although a recommendation accordingly cannot be given since there was not a direct observation of a GRS abutment in another application. Even so, it is recommended that another trial project be completed where the application differs. A suggested application would be on a higher volume roadway to ascertain if GRS settlement occurs under the vibrations associated with higher traffic volumes.

The superstructure faults do not appear to be a direct reflection of the abutments upon which it is founded. Rather, it appears to be a function of the superstructure design and the loading to which it has been subjected. A review of the design and appropriate modifications should be made to reduce or eliminate the transverse cracking. In future applications of the superstructure, consideration should be given to at least one of the following design changes: deeper cross section, more reinforcement, or the addition of post-tensioning. If implemented correctly, these design modifications may help to lessen the extent of load induced cracking. For the current bridge, it may be advisable for a thin polymer overlay to be applied to the bridge. Such an overlay does not add much additional dead load and, when successfully installed, provide good protection against moisture intrusion.

B-20-0133: US 151 Northbound over STH 26 Service Road, Waupun, WI – Use of FRP Components

General Information

Through the Innovative Bridge Research and Construction Program, a bridge construction method using fiber-reinforced polymer (FRP) reinforcing materials to reinforce cast-in-place concrete bridge decks was developed. In 2003, two sister bridges in Fond du Lac County were to undergo reconstruction and provided a good opportunity to directly compare the performance of a traditionally constructed bridge with that using the FRP reinforcing materials. The superstructure cross-section shown in Figure 54 includes the FRP innovative features. The specific sizes and location of the FRP features are discussed in greater detail in the following section.

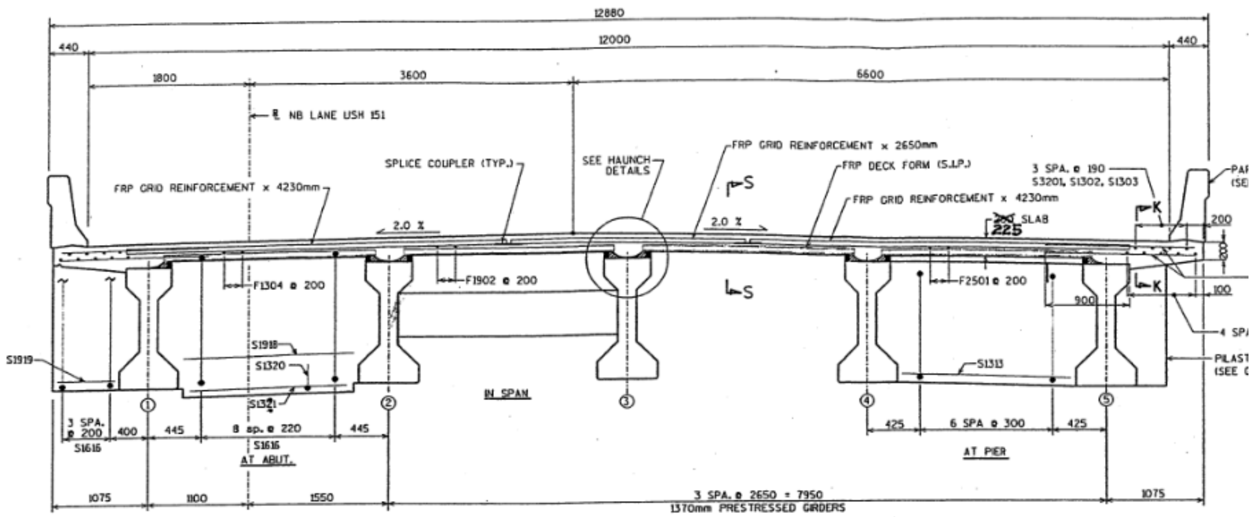
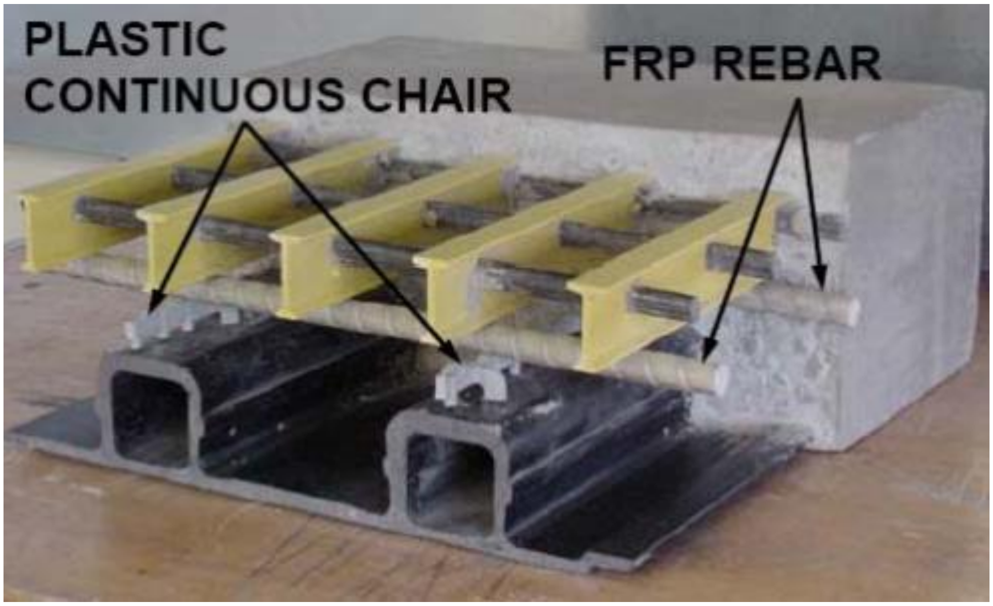


Figure 54. US 151 northbound over STH 26 Service Road bridge superstructure plan

Description of Innovative Feature

The innovative feature consisted of three main components: 1) FRP stay-in-place deck panels, 2) FRP bi-directional grid, and 3) FRP reinforcing bars seen in Figure 55.



Bank et al. 2005

Figure 55. US 151 northbound over STH 26 Service Road FRP deck system

The panels are laid transversely across the precast concrete girders and the reinforcing is place atop. Once the deck concrete has been placed, each of the innovative components are permanent to the bridge.

Use of Innovative Feature

The use of the FRP panels and reinforcing is intended to demonstrate an alternative to conventional deck reinforcing, which is often the subject of deterioration that occurs more frequently and extensively than that of other bridge components. The initial cost associated with FRP reinforcing is greater than that of traditional reinforcing but it is believed that the service life of the deck will be extended thereby justifying the initial cost. For comparison purposes, the following costs were the published let costs for Bridge B-20-0133: $592,906.71 ($677.29/sq ft) with the superstructure being $475,206.50 ($552.15/sq ft). For Bridge B-20-134 (conventionally constructed sister bridge), the let costs were: $347,856.91 ($404.18/sq ft) with the superstructure being $2525,311.50 ($293.17/sq ft).

Field Results

On August 12, 2015, the researchers visited the bridge. A cursory visual inspection aimed to identify any signs of distress that may be present and a live load test were completed. The results are discussed below.

Visual Observation

Observation of the deck surface revealed numerous cracks less than 1/16" in width primarily in the transverse direction, although several in the longitudinal direction existed (see Figure 56).

Figure 56. US 151 northbound over STH 26 Service Road bridge deck surface cracking

Maintenance on the bridge has included the use of a crack sealant. The benefit of sealing a deck without concrete is probably minimal and, at best, probably helps void freeze-thaw related deterioration. A majority of the observed cracks have been sealed and, to this point, the sealant appears to be performing well (i.e., fully intact within cracks). It is unknown if the cracking is a direct result of the FRP reinforcing. However, a direct contrast to the adjacent bridge where traditional reinforcing was used would indicate that it is not. Figure 57 shows the deck surface of the adjacent bridge and similar crack patterns exist. As expected, no signs of deck delamination were found.

Figure 57. US 151 northbound over STH 26 Service Road surface of adjacent bridge

Aside from the surface cracking that was observed, the remaining components of the bridge appeared in excellent condition with little wear or signs of distress apparent. See Figure 58.

Figure 58. US 151 northbound over STH 26 Service Road condition of underside of bridge

Performance Evaluation and Summary

A live load test of the bridge was completed to determine if its performance under load is dissimilar to that of a traditionally constructed bridge. Strain gages were used at multiple locations including the bottom side of the each girder at midspan of the west span and between each girder on the underside of the stay-in-place deck panels at approximately quarter span. The gages placed on the girders, highlighted by the triangles in Figure 59, were oriented in the longitudinal direction so as to measure the strain and thus the transverse load distribution across the bridge during any given load pass.

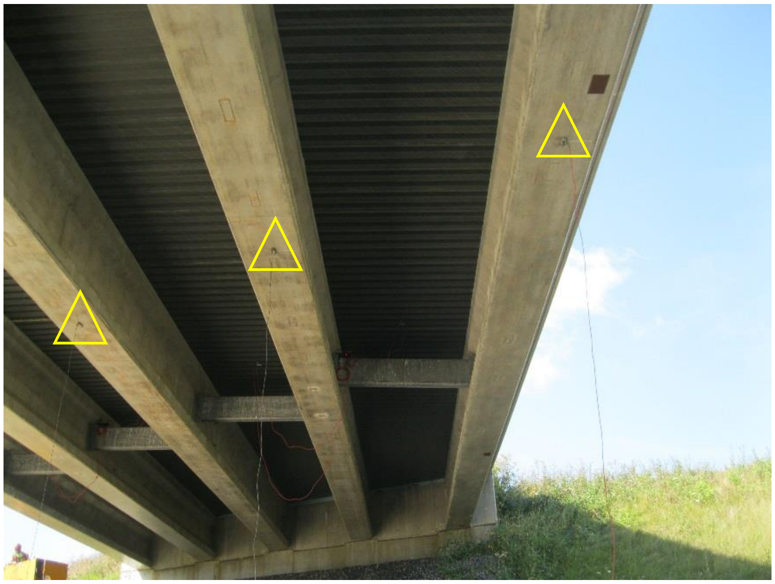


Figure 59. US 151 northbound over STH 26 Service Road girder strain gages

The gages placed between the girders were placed in pairs, one in the longitudinal direction and one in the transverse direction, to determine if there is any appreciable difference in performance between the two directions. A typical gage pairing is shown in Figure 60.

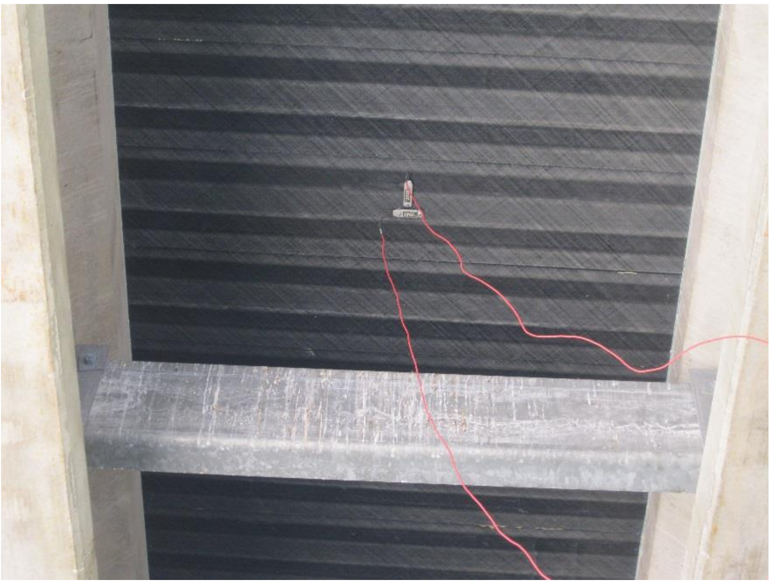


Figure 60. US 151 northbound over STH 26 Service Road deck strain gage pairing

To complete the bridge test the south lane was closed, while maintaining traffic in the north lane. A heavy truck weighing approximately 60,000 lbs was provided through the Wisconsin DOT. The truck tires and axles were in the configuration shown in Figure 61.

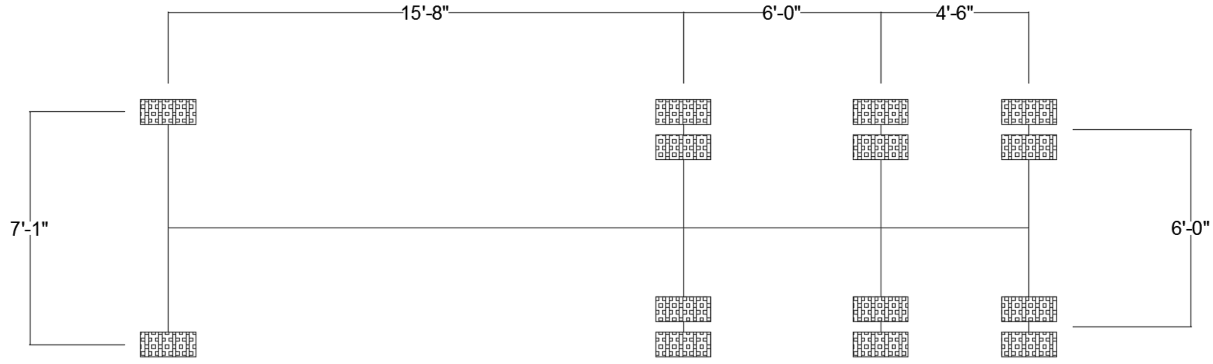


Figure 61. US 151 northbound over STH 26 Service Road truck axle configuration

The transverse position of the truck is shown in relation to the strain gages mentioned previously in Figure 62.

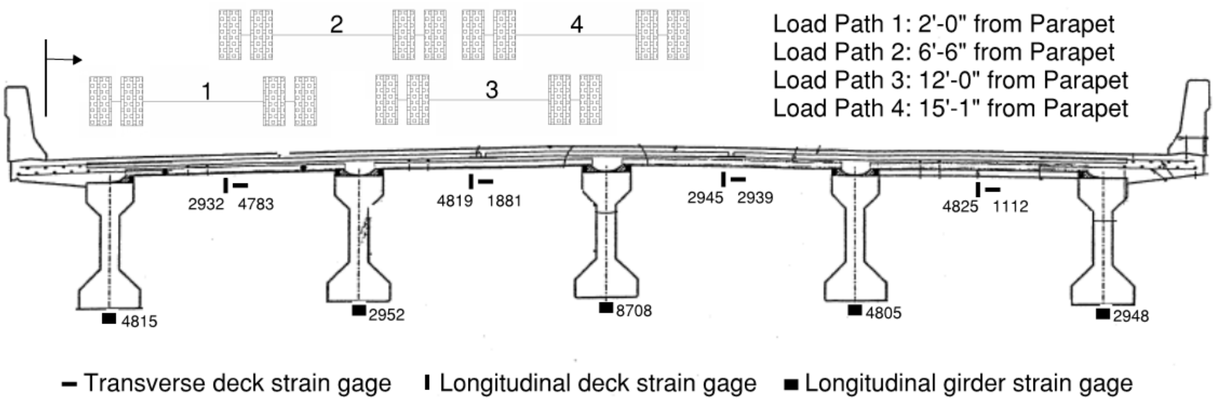


Figure 62. US 151 northbound over STH 26 Service Road load path and gage locations - looking west

Again, note that the north lane remained open to traffic limiting the load paths to the south side of the bridge.

Results

Strain data collected throughout the duration of each load path are presented below: Load Path 1 in Figure 63 through Figure 67; Load Path 2 in Figure 68 through Figure 72; Load Path 3 in Figure 73 through Figure 77; and Load Path 4 in Figure 78 through Figure 82.

Load Path 1 – Strain Results

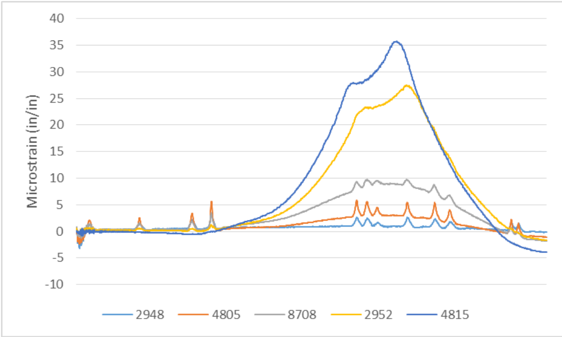


Figure 63. Girder strain – Load Path 1

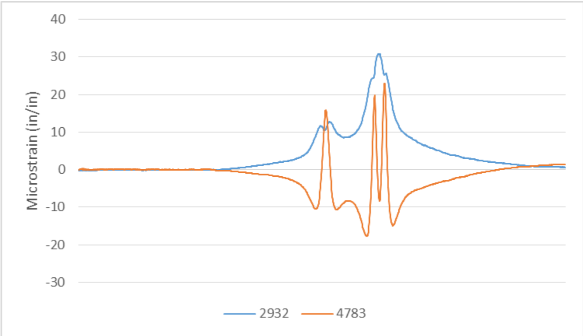


Figure 64. Bay 1 deck strain – Load Path 1

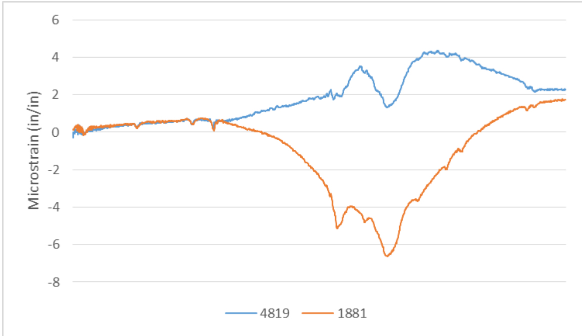


Figure 65. Bay 2 deck strain – Load Path 1

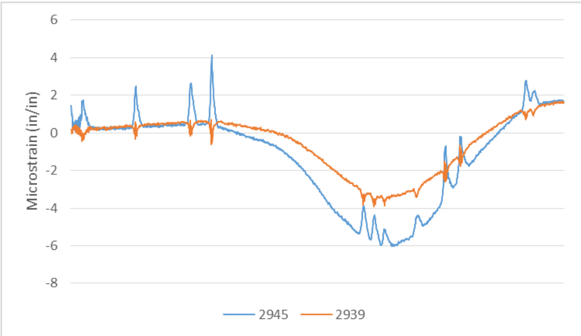


Figure 66. Bay 3 deck strain – Load Path 1

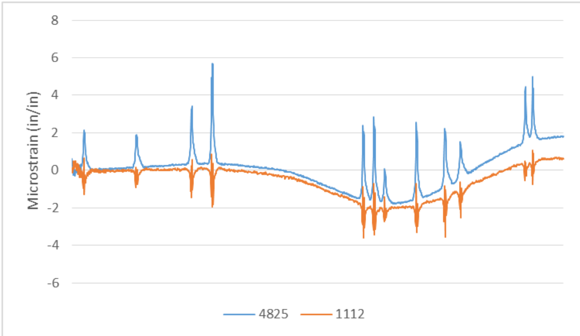


Figure 67. Bay 4 deck strain – Load Path 1

Load Path 2 – Strain Results

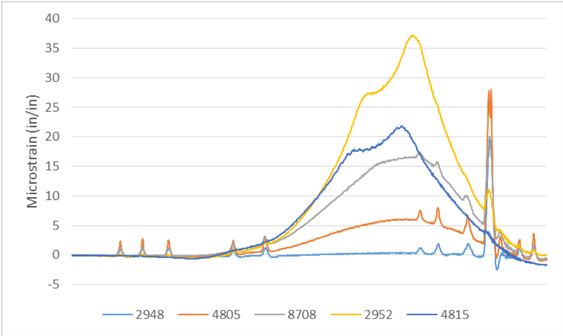


Figure 68. Girder strain – Load Path 2

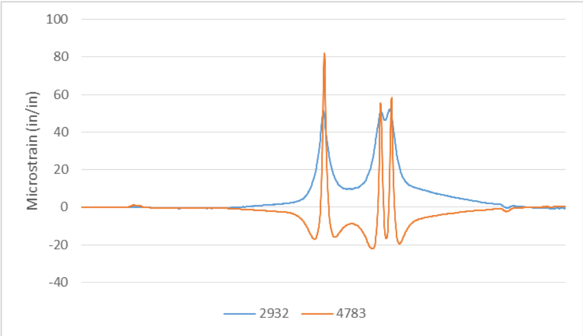


Figure 69. Bay 1 deck strain – Load Path 2

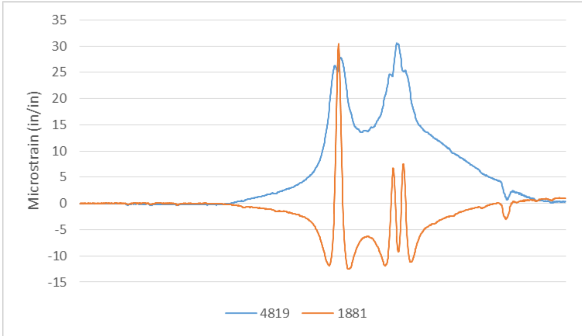


Figure 70. Bay 2 deck strain – Load Path 2

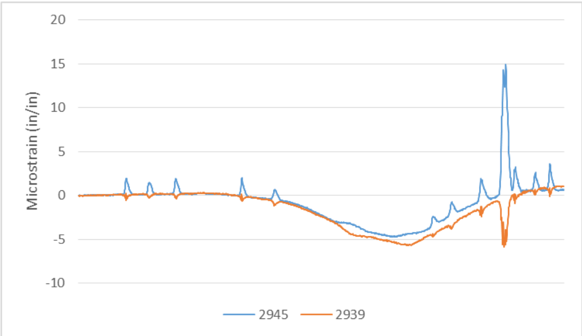


Figure 71. Bay 3 deck strain – Load Path 2

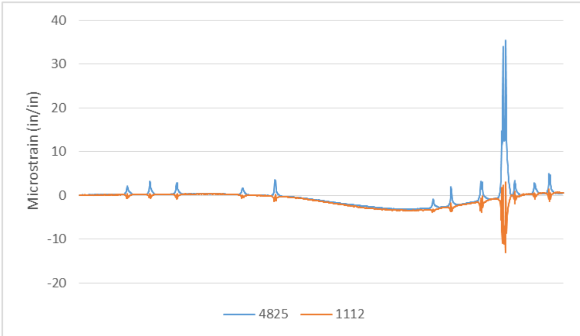


Figure 72. Bay 4 deck strain – Load Path 2

Load Path 3 – Strain Results

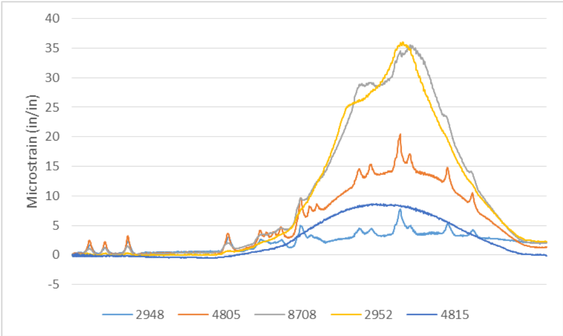


Figure 73. Girder strain – Load Path 3

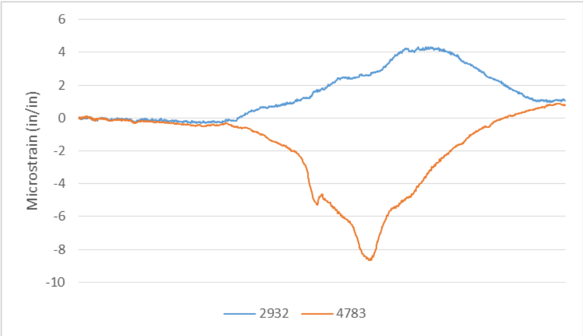


Figure 74. Bay 1 deck strain – Load Path 3

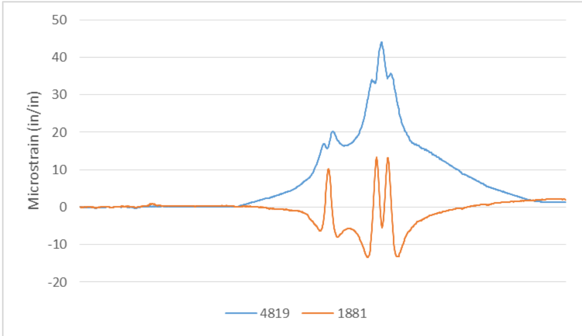


Figure 75. Bay 2 deck strain – Load Path 3

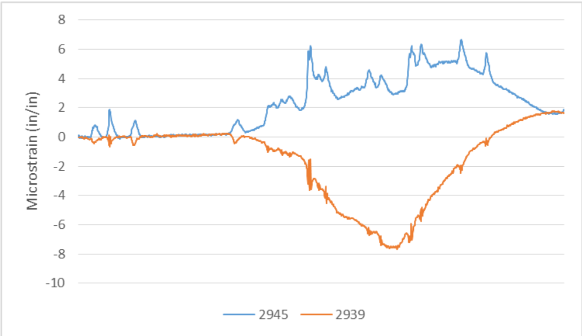


Figure 76. Bay 3 deck strain – Load Path 3

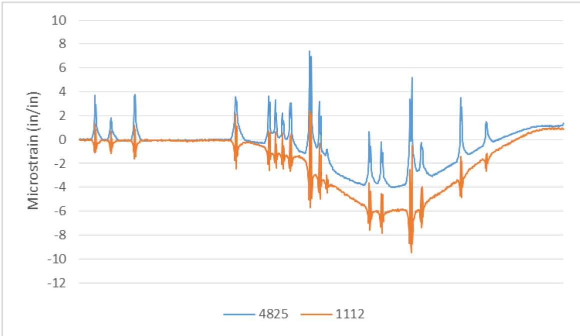


Figure 77. Bay 4 deck strain – Load Path 3

Load Path 4 – Strain Results

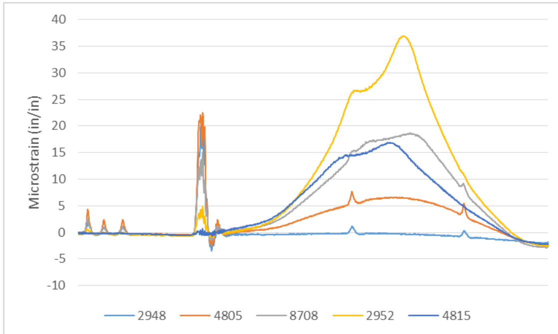


Figure 78. Girder strain – Load Path 4

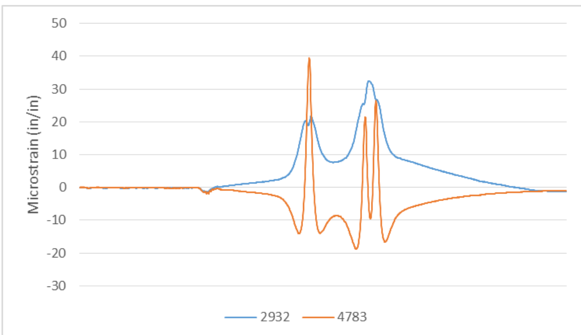


Figure 79. Bay 1 deck strain – Load Path 4

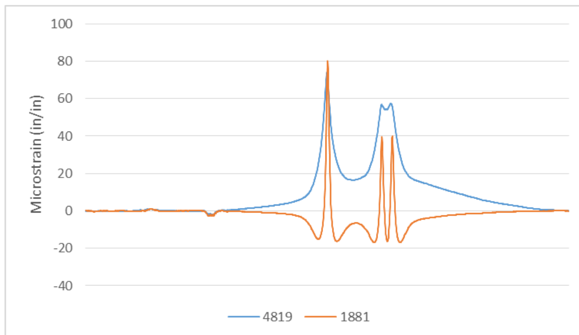


Figure 80. Bay 2 deck strain – Load Path 4

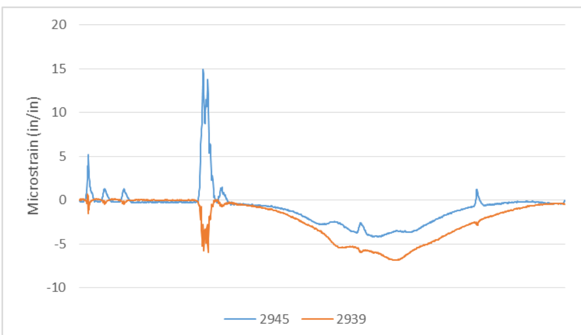


Figure 81. Bay 3 deck strain – Load Path 4

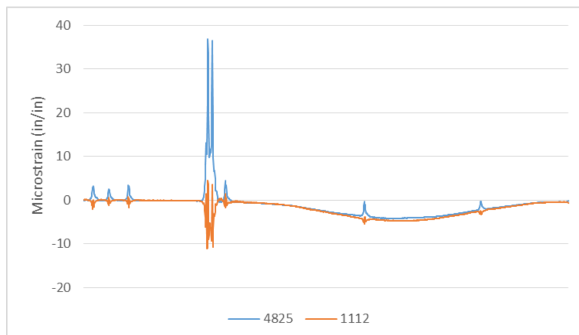


Figure 82. Bay 4 deck strain – Load Path 4

Summary of Results

The results illustrate the load distribution through the deck and girders. For each case, those data spikes that occur over a short duration are attributable to traffic passing in the north lane at a much greater rate of speed than the load truck. For all intents and purposes, these spikes are to be ignored while recognizing the overall data trend.

The load distribution across the bridge appears to be in line with that of one with a traditionally constructed deck. As one might expect, the girders directly beneath or close to the truck were more heavily loaded than those away from the load path. The deck stiffness, however, was sufficient to shed at least a portion of the load to all girders across the bridge – albeit the girder farthest away from the load generally resisted very little load. Assuming equal or nearly equal stiffness in all girders, the greatest percentage of load any one girder resisted was approximately 47 percent.

Similarly, the data collected from the deck gages illustrated deck behavior consistent with that of a traditionally constructed deck. Those placed in the longitudinal direction closest to the load path primarily measured tensile strains that peaked as individual axles would cross directly overhead. The longitudinal deck gages placed away from the load path registered compressive strain, although the magnitude was quite small. The transverse deck gages generally registered modest compressive strains (<10 ms) as the truck traversed the bridge. However, for those gages where a wheel line resided in the corresponding bay, the gage would undergo a stress reversal at the points in time when the wheel would travel immediately above the gage; peak tensile strains ranged from approximately 30 ms to 80 ms depending on where the wheel line was transversely in relationship to the gage.

In the end, the bridge appears to be performing similarly under load to a traditionally constructed bridge. No evidence suggests that the FRP stay-in-place forms and reinforcement alter the behavior of the bridge.

Discussion and Recommendations

After more than 10 years of being in service, the innovative feature of FRP stay-in-place forms and FRP deck reinforcement shows no noteworthy degradation that wouldn’t be expected at a bridge of its age. Overall, the bridge is performing well and no evidence suggests that the performance of the bridge is any different than that of the traditionally constructed bridge immediately adjacent. The deck of each bridge has experienced a modest amount of cracking although it cannot be concluded that it is the direct result of using traditional reinforcement or FRP reinforcement. In fact, the crack patterns appear to be very similar between the two decks and is likely a result of something other than the reinforcement altogether. It could be argued that since each bridge deck has experienced cracking that could allow chlorides to penetrate the deck to the level of reinforcing, the FRP reinforced deck may allow maintenance procedures to be altered or delayed since the FRP will not corrode. For this reason, the use of FRP reinforcing might be worth the initial investment. It is widely known that bridge decks are most often the component that requires maintenance or replacement sooner than other bridge components.

B-20-0148: US 151 over De Neveu Creek,
Fond du Lac, WI – Use of FRP Deck Reinforcement

General Information

Bridge B-20-0148 on US 151 over De Neveu Creek near Fond du Lac, WI was reconstructed using an innovative deck reinforcing feature. After construction, positive and negative results were identified. As part of this study, the bridge was visited and a review of its current condition was completed.

Description of Innovative Feature

The innovative feature consisted of a three-dimensional, prefabricated, two-layer FRP reinforced concrete deck. The deck was placed on a single span, 130 ft long bridge having 2 lanes sitting on precast/prestressed concrete bulb-Ts. The total bridge deck width and thickness is 44'-9 ¼" and 8", respectively. The superstructure, complete with details of the FRP reinforcing, is shown in Figure 83 through Figure 85.

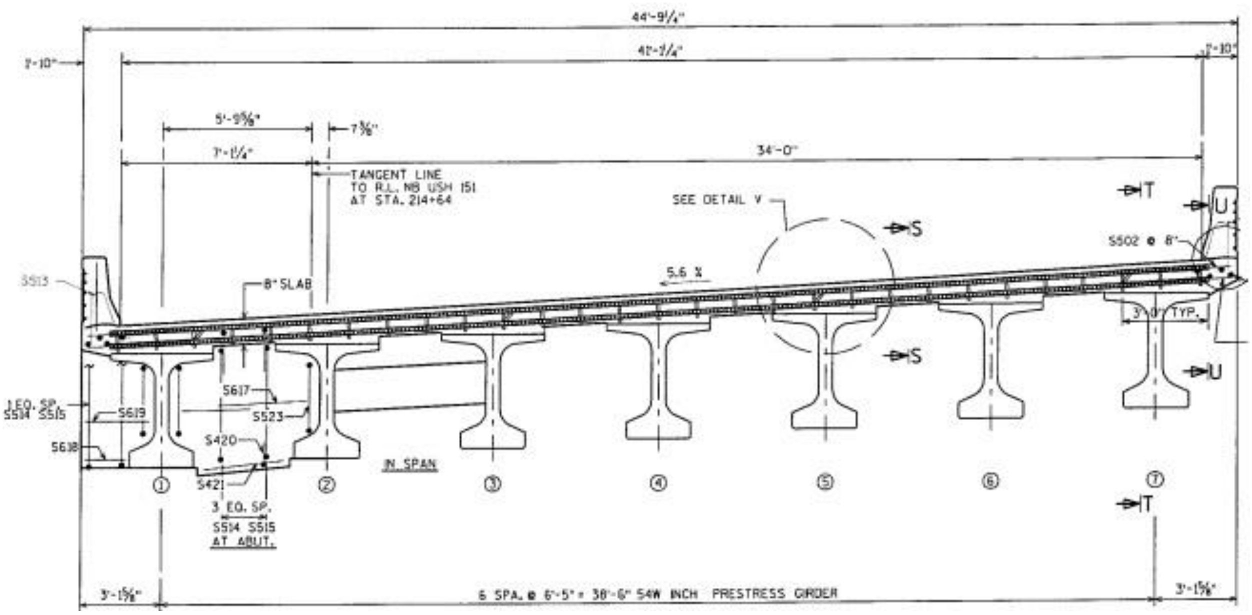


Figure 83. US 151 over De Neveu Creek bridge superstructure

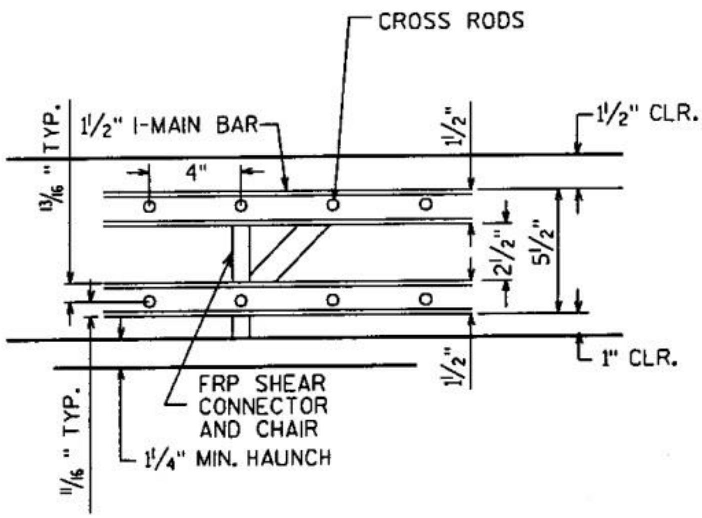


Figure 84. US 151 over De Neveu Creek FRP deck reinforcement – 1

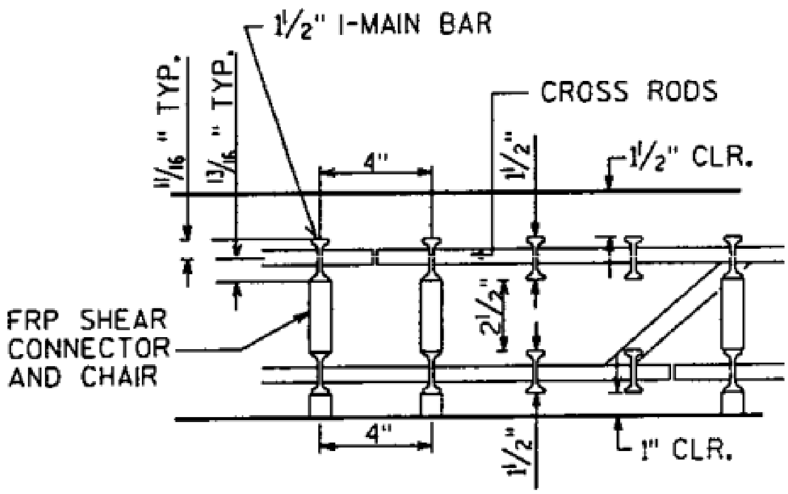


Figure 85. US 151 over De Neveu Creek FRP deck reinforcement – 2

Use of Innovative Feature

The innovative feature intends to prolong the service life of the bridge deck. Without steel reinforcement the deck is not subjected to the same rate of degradation related corrosion as would likely be seen in a traditionally constructed deck. Usually the deck is the first element of a bridge to require maintenance and subsequent maintenance is required more frequently. If the service life of the deck is prolonged, it seems reasonable to assume the dollars required to maintain the bridge would be lessened.

Field Results

On August 12, 2015, the bridge deck was inspected for surface cracking, delaminations, and/or other observable signs of deterioration. Six randomly selected areas measuring 10 ft by 10 ft were surveyed for cracks and a cursory review of the remaining structure was conducted. The results are discussed below.

Several positive and negative results derived from the initial construction were reported within the FHWA IBRC/IBRD Project Summary Report. These are paraphrased in the following.

Positive Results

- Rapid construction of the deck was achieved using the FRP reinforcement; 93 man hours versus 275 man hours for placing epoxy coated rebar on an identical adjacent structure
- The FRP deck strength was tested to be 7.5 times the service wheel load
- The FRP grids could be manufactured and shipped in lengths equal to the bridge width

Negative Results

- The cost was higher than the traditionally constructed adjacent bridge

In addition, some lessons learned were listed as follows:

- FRP reinforcing is feasible for rapid and durable construction
- Construction speed for the deck can be doubled
- FRP material costs are likely to decrease with continued automation advances
- A lightweight FRP plate could be added to the system to serve as formwork thus eliminating the need for traditional formwork

When coupled with the survey results of this study, the lessons learned would indicate that the current upcharge for using FRP deck reinforcement would likely be justifiable. The reduced maintenance and prolonged deck service life results in fewer life cycle dollars required.

Visual Observation

The deck surface was in sound condition, although numerous longitudinal cracks 1/32” to 1/16” in width, were observed throughout the entirety of the deck. Crack sealant has been placed in many of these cracks and appears to be performing well (i.e., fully intact within cracks). However, the benefit of sealing a deck that has no internal steel reinforcement is probably minimal. It is unknown the exact date of placement or how long the cracks existed prior to the sealant being placed. A series of photos showing the general condition of the south lane from west to east is provided in Figure 86 through Figure 92.

Figure 86. US 151 over De Neveu Creek deck surface condition – 1

Figure 87. US 151 over De Neveu Creek deck surface condition – 2

Figure 88. US 151 over De Neveu Creek deck surface condition – 3

Figure 89. US 151 over De Neveu Creek deck surface condition – 4

Figure 90. US 151 over De Neveu Creek deck surface condition – 5

Figure 91. US 151 over De Neveu Creek deck surface condition – 6

Figure 92. US 151 over De Neveu Creek deck surface condition – 7

The underside of the deck showed very little degradation or cracking and appeared in very good overall condition. See Figure 93.

Figure 93. US 151 over De Neveu Creek underside condition

None of the longitudinal cracking observed on the top surface was seen on the underside, indicating that the cracks were not full depth. As shown in Figure 94, the only noteworthy observation on the deck underside was at the end of the deck near the northeast acute corner where hairline cracking with efflorescence was found.

Figure 94. US 151 over De Neveu Creek hairline cracking at bridge end

Crack Survey

In order to better quantify the amount of deck surface cracking, a crack survey was completed in six areas, each 100 ft^2. The areas were randomly selected along the length of the bridge in the north lane, which was closed to traffic. The approximate locations of the areas selected are indicated and labeled in Figure 95 followed by photographs and crack maps of each area in Figure 96 through Figure 101.

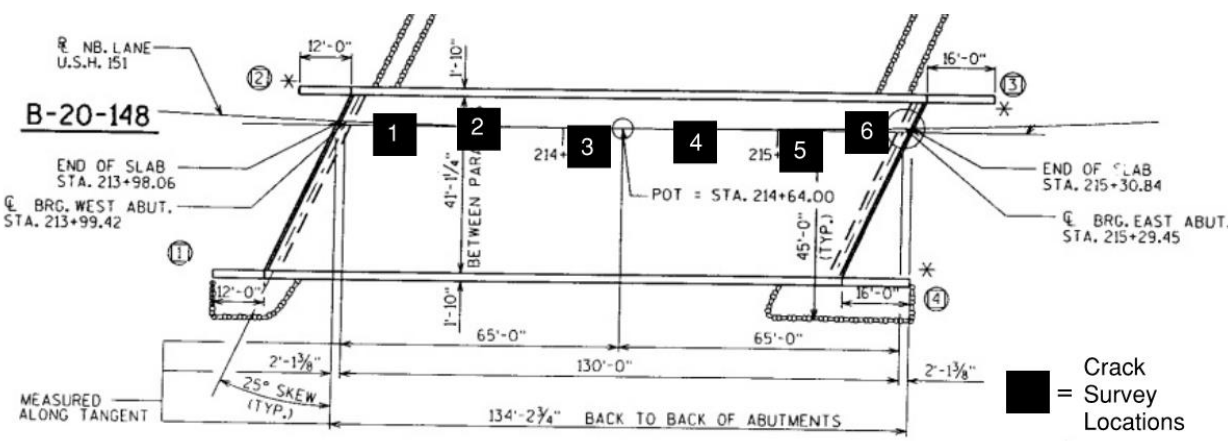


Figure 95. US 151 over De Neveu Creek crack survey locations

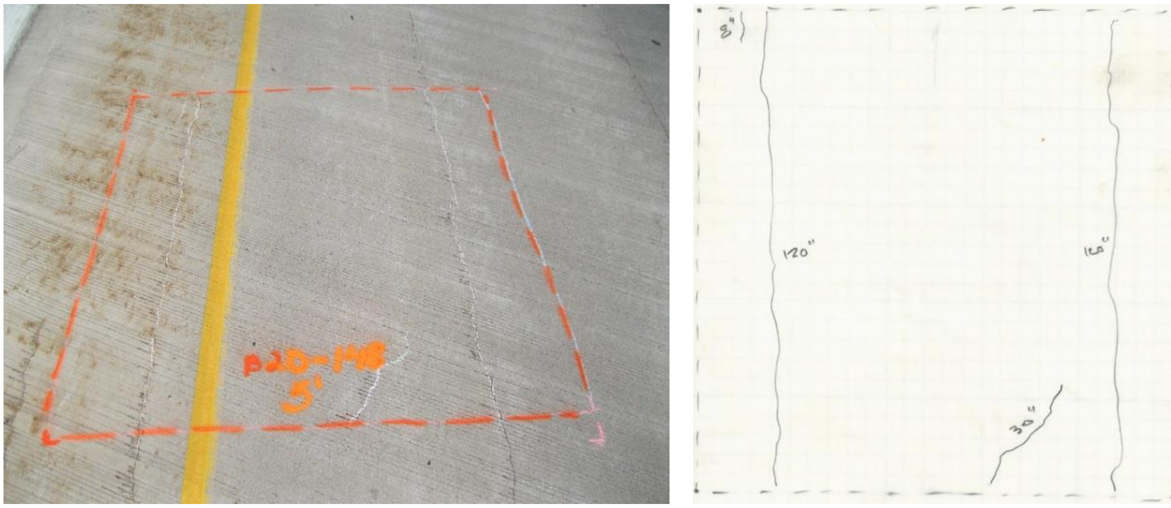


Figure 96. US 151 over De Neveu Creek crack survey – Area 1

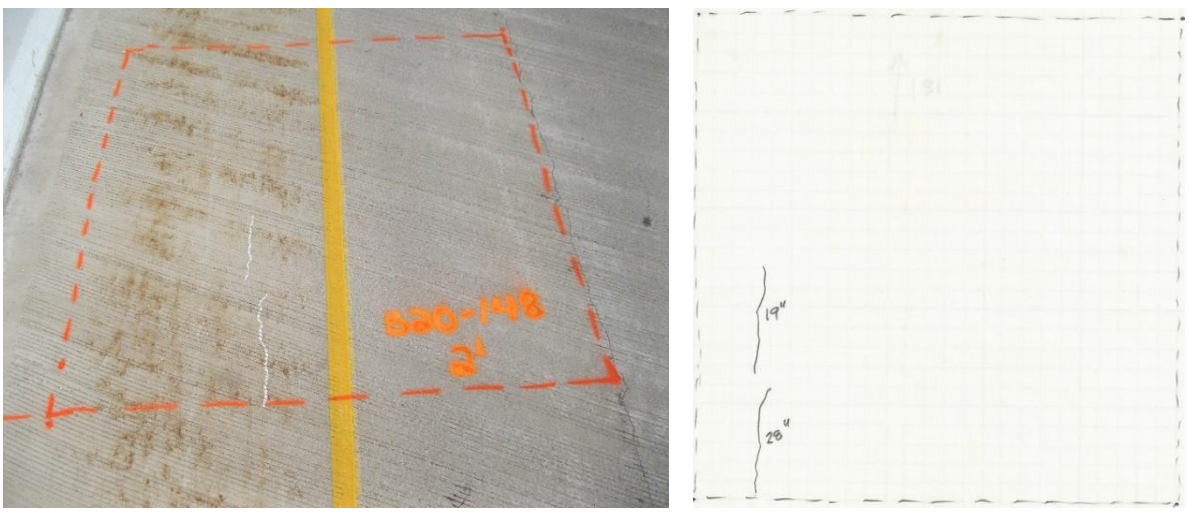


Figure 97. US 151 over De Neveu Creek crack survey – Area 2

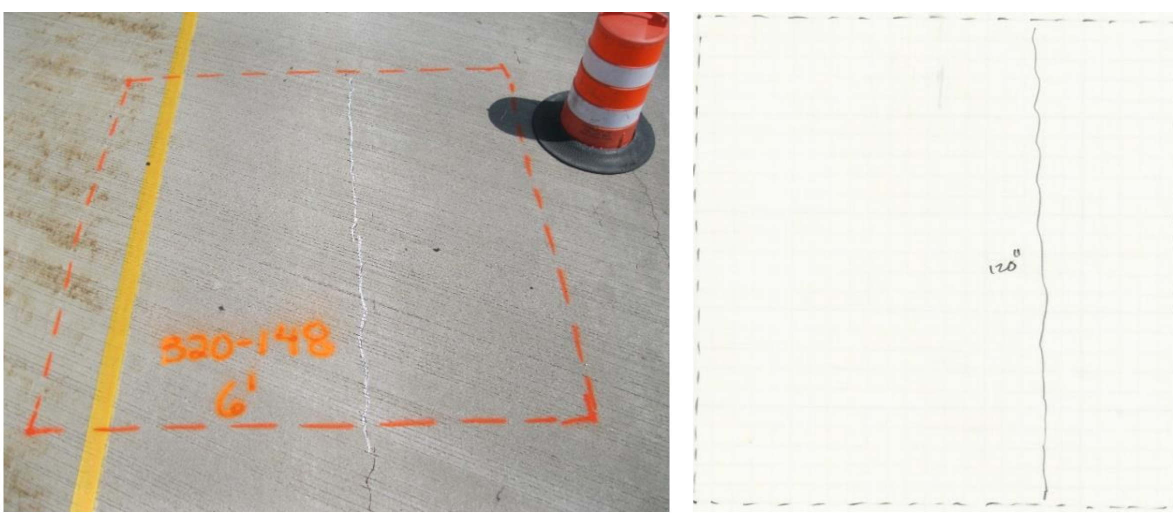


Figure 98. US 151 over De Neveu Creek crack survey – Area 3

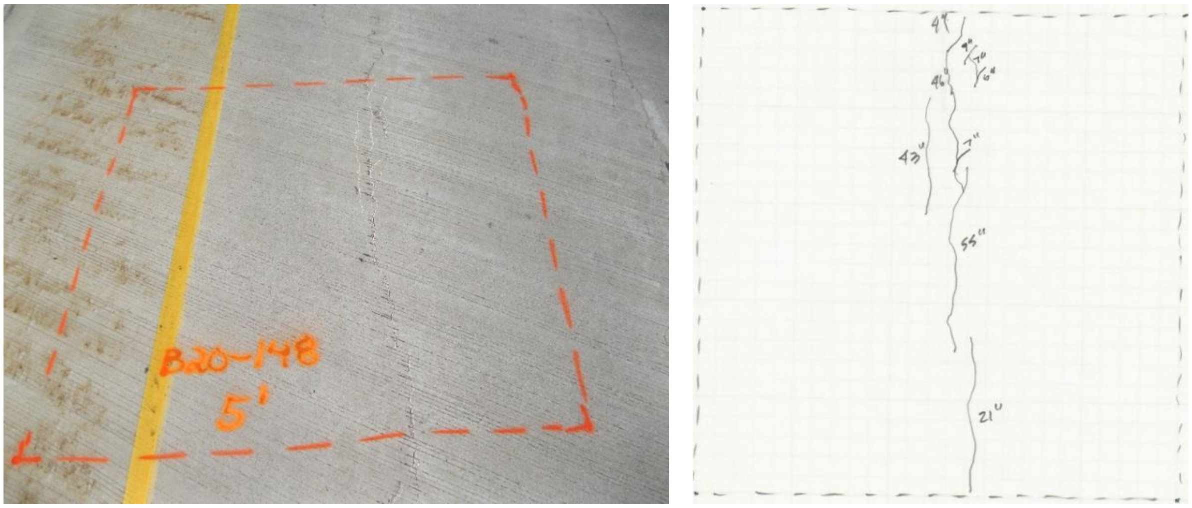


Figure 99. US 151 over De Neveu Creek crack survey – Area 4

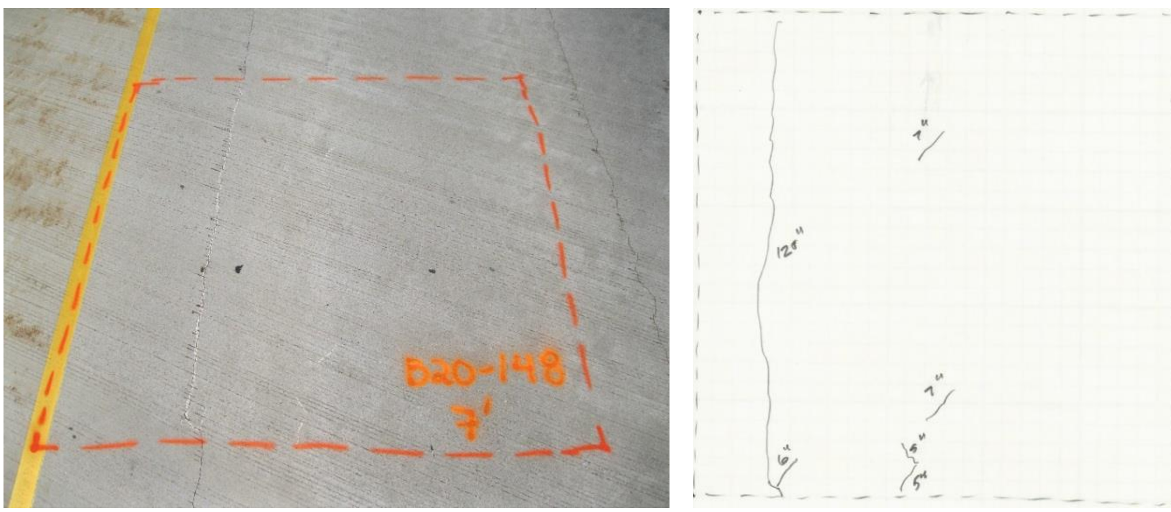


Figure 100. US 151 over De Neveu Creek crack survey – Area 5

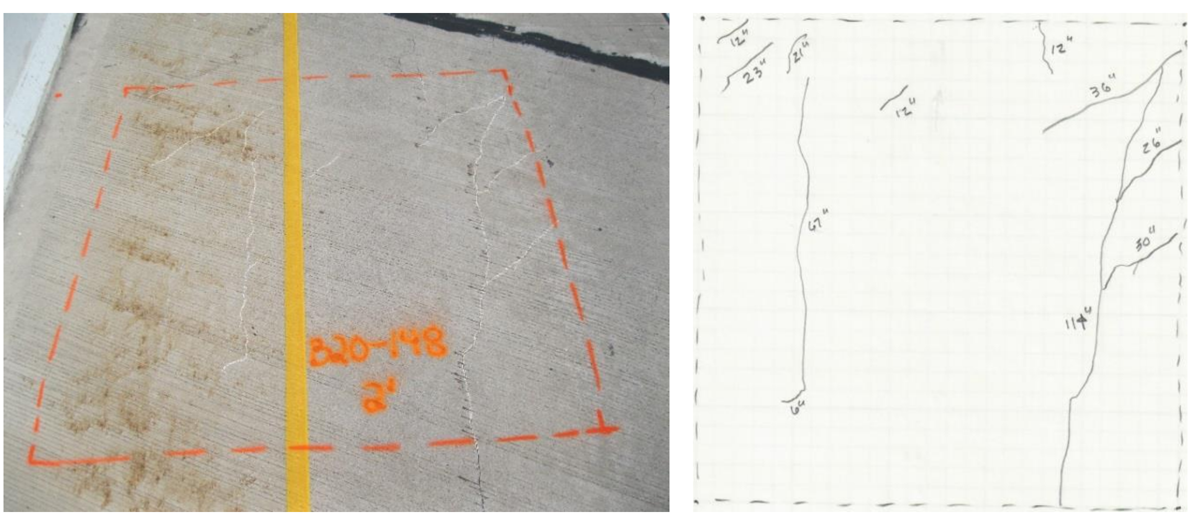


Figure 101. US 151 over De Neveu Creek crack survey – Area 6

The crack patterns varied among each of the selected areas. There was not a well-established pattern in any area aside from the primary crack direction being longitudinal. The quantity of cracking also varied. As means for quantitatively assessing the observed cracks, measurements of the crack lengths were summed for each of the selected areas. The quantitative results of the crack survey are provided in Table 2.

Table 2. US 151 over De Neveu Creek crack survey results

Survey Deck Location	Area	Total Length of Cracks	Crack Length/ Area
1) 5’-0” from E.O.B. in North Lane/ 4’-0” from North parapet	100 sq. ft	23’-2”	0.23 ft/sq. ft
2) 25’-0” from E.O.B. in North Lane/ 2’-0” from North parapet	100 sq. ft	3’-11”	0.04 ft/sq. ft
3) 50’-0” from E.O.B. in North Lane/ 6-0” from North parapet	100 sq. ft	10’-0”	0.10 ft/sq. ft
4) 75’-0” from E.O.B. in North Lane/ 5’-0” from North parapet	100 sq. ft	16’-6”	0.17 ft/sq. ft
5) 100’-0” from E.O.B. in North Lane/ 7’-0” from North parapet	100 sq. ft	12’-6”	0.13 ft/sq. ft
6) 115’-0” from E.O.B. in North Lane/ 2’-0” from North parapet	100 sq. ft	29’-11”	0.30 ft/sq. ft

*E.O.B = Northwest corner of bridge where bridge deck meets approach slab

One can see from the quantitative results that the sum length of cracking varied considerably between areas surveyed, with as little as 0.10 ft/sq. ft to 0.30 ft/sq. ft. That said, only one of the areas came near to the 0.30 ft/sq. ft, (the area at the east end of the bridge), with most of the other areas in the range of 0.10 to 0.17 ft/sq. ft. The greater amount of cracking in area 6 can seemingly be attributed to the several cracks propagating perpendicular to the deck edge. This crack pattern has been a common observation at deck ends especially on skewed bridges.

Performance Evaluation and Summary

Using FRP deck reinforcing has provided a means to potentially increase the service life of the bridge deck. FRP will not corrode when subjected to chlorides like black rebar or damaged epoxy coated rebar. Since the reinforcing has proven to be effective from a structural point of view, it is likely the rate of degradation will decrease and reduce the required deck maintenance or deck replacement over the lifespan of the bridge.

Cracking observed in the bridge surface cannot be conclusively attributed to the use of or, conversely, the lack of FRP reinforcement. In fact, similar crack patterns are seen on traditionally constructed bridges of the same configuration. The crack pattern does not appear to be directly correlated to the type of deck reinforcement used when comparing FRP grids and epoxy coated rebar.

**B-27-0150: US 12 over Coffee Creek,
Black River Falls, WI – Use of Steel Free Deck and FRP Stay in Place Forms**

General Information

Bridge B-27-0150 is located on US 19 over Coffee Creek in Black River Falls, WI. The bridge was constructed in 2008 using a steel free deck and FRP stay-in-place forms as part of the FHWA IBRC/IBRD program.

Description of Innovative Feature

Eliminating the reinforcement from the deck runs contrary to traditional deck design. Traditionally, the deck design is based on resisting transverse flexure of the deck. However, much research has been conducted on bridge deck behavior and has repeatedly shown that deck strength is controlled by punching shear (as long as sufficient outward restraint is provided). For this bridge, the basis for design was based on punching shear rather than flexure, using a strut-and-tie method for analysis. The elimination of the deck steel resulted in accelerated construction, increased job site safety due to the shorter construction time, and reduced material costs. A further long term benefit is the fact that elimination of the deck steel means there will not be a mechanism for traditional deck deterioration.

The bridge measures 55’-0” wide x 100’-0” long. The cross section, shown in Figure 102, shows eight girders spaced at 7’-0” with a stay in place FRP panel spanning between girders serving as the formwork.

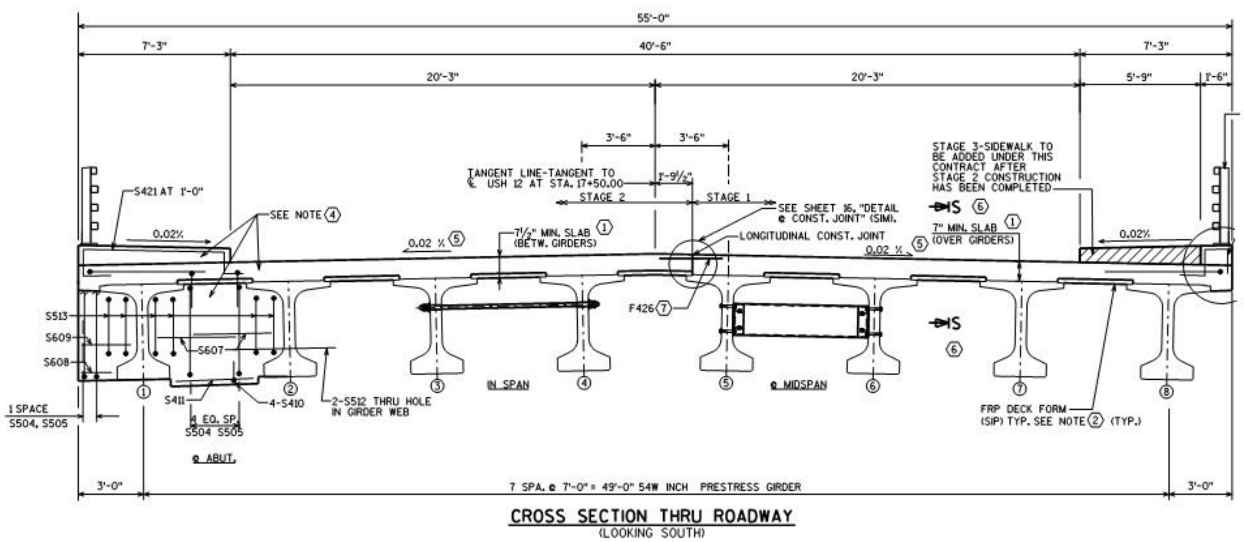


Figure 102. US 12 over Coffee Creek superstructure cross-section

Tension ties are anchored transversely between the girders spaced at 6’-0” along the length of the bridge. It is through these tension ties that arching action is formed in the deck (in a basic way, the tension ties carry the stresses typically thought to be carried by the deck steel). In addition, a steel channel diaphragm ties each of the girders to one another at midspan.

Use of Innovative Feature

There are several advantages to using a steel free bridge deck. As previously mentioned, there is increased safety associated with shorter construction and maintenance, reduced costs, and accelerated construction. For these reasons, the innovative feature is desirable and, with continued success, could be implemented on a more regular basis. It should be pointed out that steel free decks have been used for many years in Canada with much less application in the United States.

Field Results

On September 15, 2015, a cursory inspection of the bridge was conducted in an attempt to identify any signs of distress or degradation. Specifically, the top and bottom side of the deck and the tension ties were of primary focus. In addition to the visual inspection, a load test was completed to measure the performance of the bridge when subjected to a loaded truck. The results of the visual inspection and load test are presented below.

Visual Observation

The surface of the deck was in good condition although several cracks (1/32") were observed. No particular pattern was identified; rather, a pattern of random direction and spacing. Some of the cracks are shown in Figure 103 through Figure 105.

Figure 103. US 12 over Coffee Creek deck crack – 1

Figure 104. US 12 over Coffee Creek deck crack – 2

Figure 105. US 12 over Coffee Creek deck crack – 3

The underside of the superstructure appeared in very good condition, although it should be noted that the underside of the deck is concealed by the FRP stay in place form. The underside is shown in Figure 7. Overall, from a visual perspective, the bridge appears in fine condition and does not appear as though there are areas of undue stress or deterioration as a result of the loads or any other conditions to which it is subjected.

Figure 106. US 12 over Coffee Creek underside condition of superstructure

Load Test

A load test was completed to identify the load distribution characteristics of the deck and the behavior of the tension ties between girders. Two strain gages were placed on the FRP stay-in-place form at mid-space between girders in the longitudinal and transverse direction 39'-8" from the west abutment. A single strain gage was placed on the girder bottom at the same distance and a single strain gage was placed at each tension tie nearest the strain gage placement. Figure 107 through Figure 109 illustrate the types and locations of the strain gages.

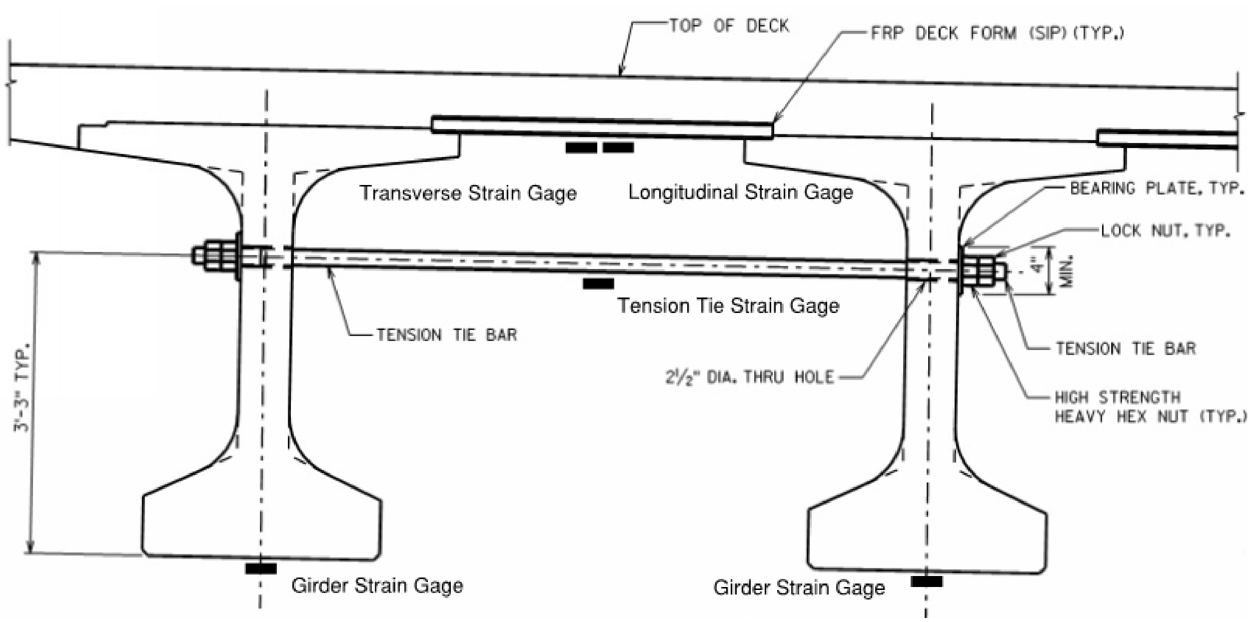


Figure 107. US 12 over Coffee Creek typical gage configuration

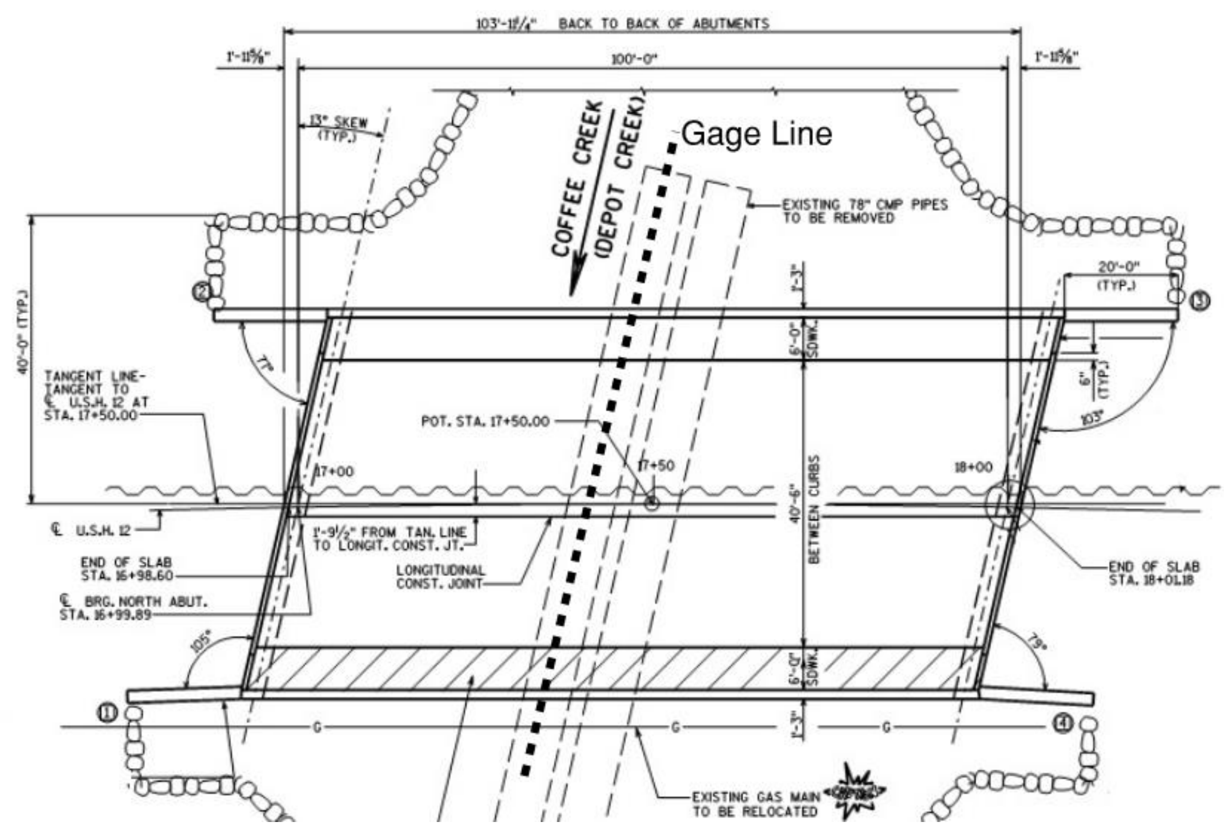


Figure 108. US 12 over Coffee Creek gage line

Figure 109. US 12 over Coffee Creek typical gage configuration installed

The load test was completed using a fully-loaded dump truck weighing approximately 64,000 lbs provided by Jackson County. The tire and axle configuration are shown in Figure 110. The spacing between the front and last rear axle measured 22 ft and the total width measured just over 7 ft. The researchers collected data while the truck crossed the bridge from south to north at a walking pace for five specific load cases. The load cases are illustrated in Figure 111 and Figure 112.

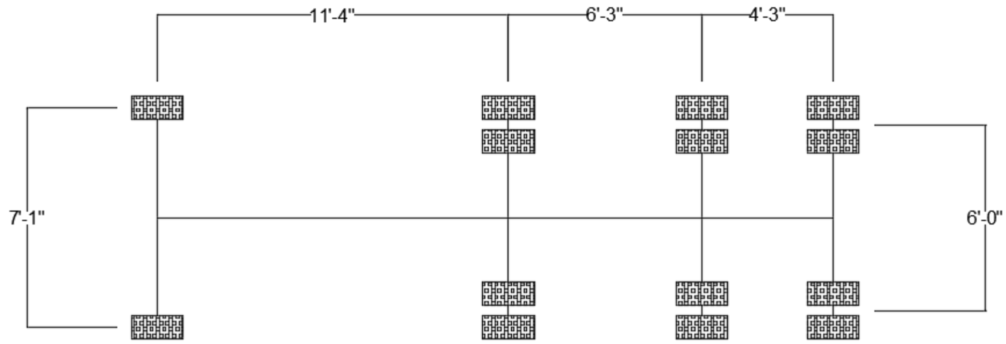


Figure 110. US 12 over Coffee Creek tire and axle configuration

Load Path 1: Passenger side wheel 2'-0" from East curb
Load Path 2: Driver side wheel 2'-0" East of Centerline
Load Path 3: Truck centered on Centerline
Load Path 4: Passenger side wheel 2'-0" West of Centerline
Load Path 5: Driver side wheel 2'-0" from West curb

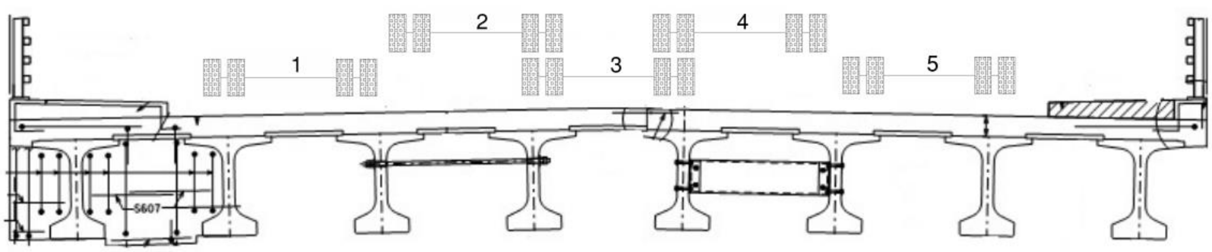


Figure 111. US 12 over Coffee Creek load paths - looking south

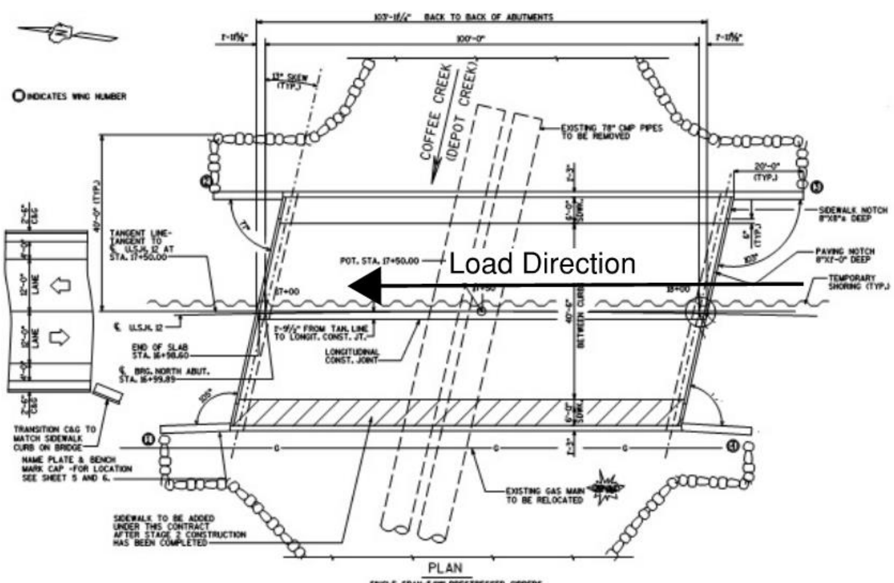
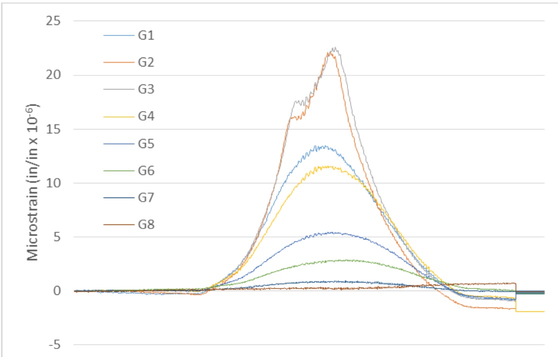


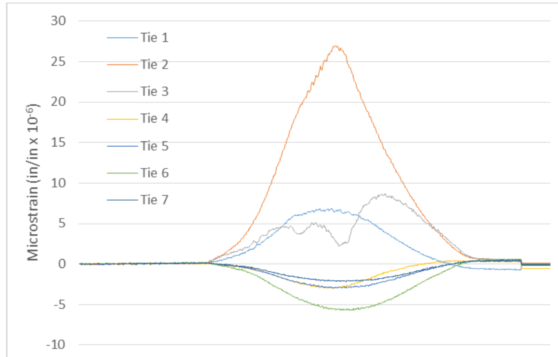
Figure 112. US 12 over Coffee Creek load direction

Results

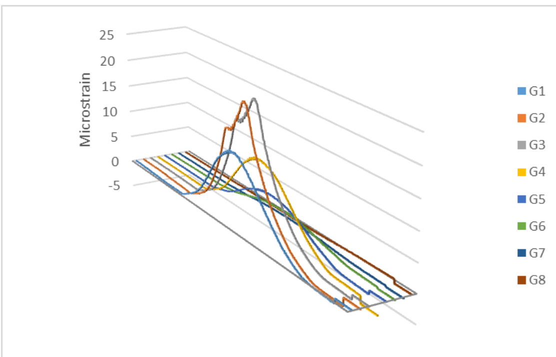
The results of the load test are presented in Figure 113 through Figure 117 for load paths 1 through 5, respectively. For each load path, the girder strain and girder strain distribution, the tension tie strain and tension tie strain distribution, and deck strain time history plots are presented.



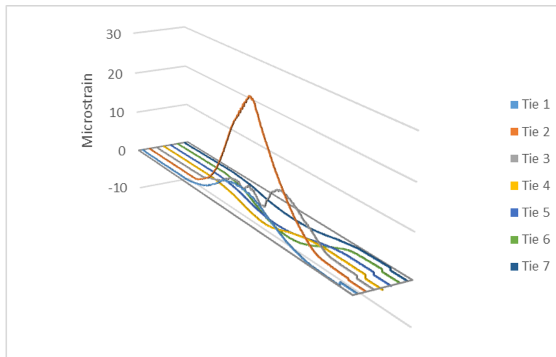
Girder Strain



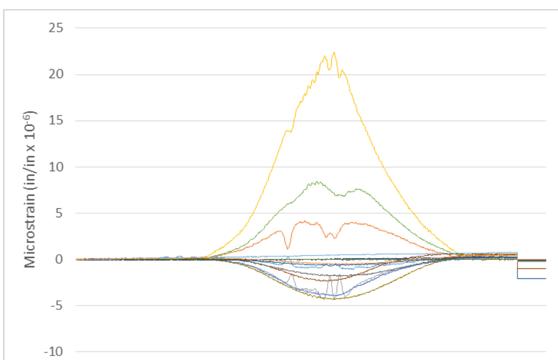
Tension Tie Strain



Girder Strain Distribution

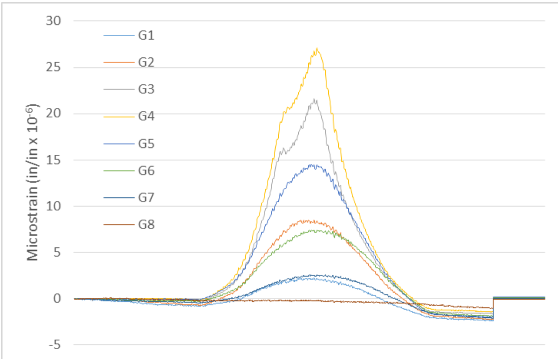


Tension Tie Strain Distribution

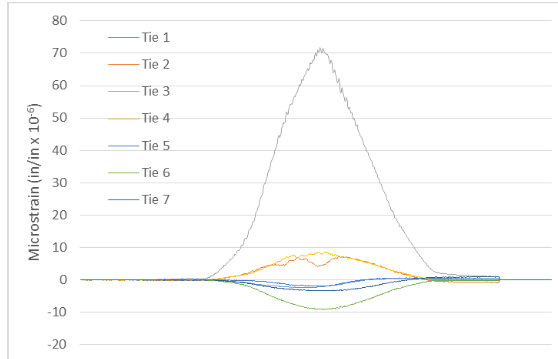


Deck Strain

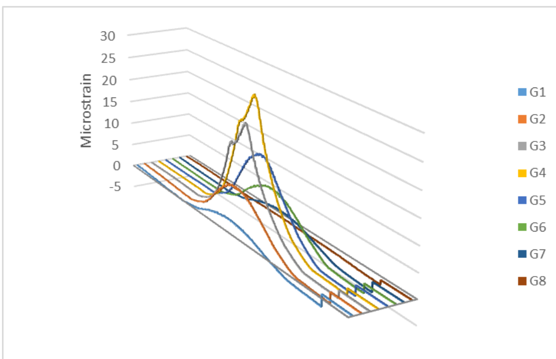
Figure 113. US 12 over Coffee Creek Load Path 1 results



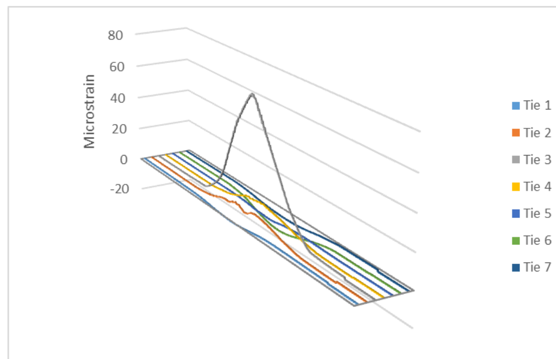
Girder Strain



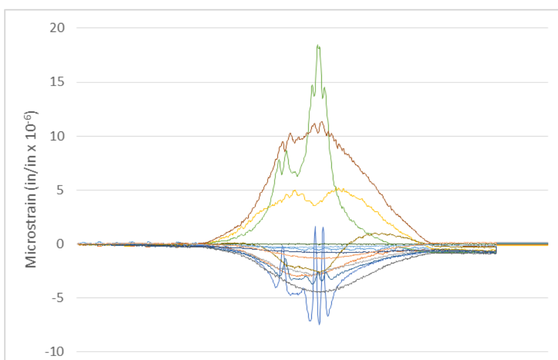
Tension Tie Strain



Girder Strain Distribution

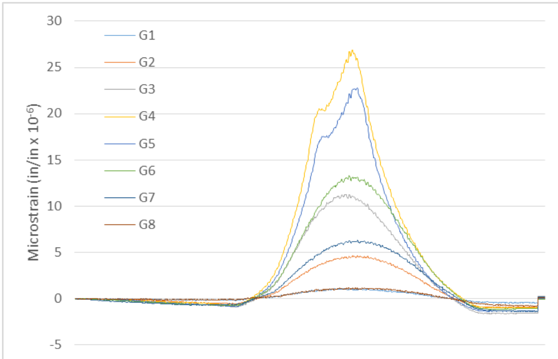


Tension Tie Strain Distribution

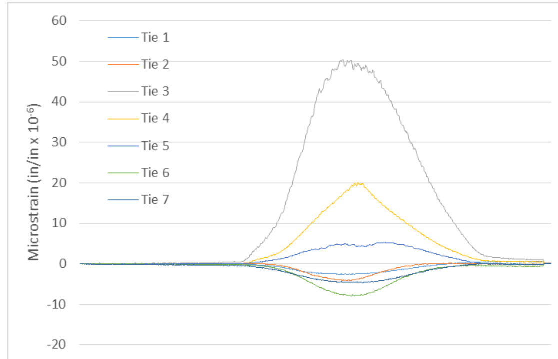


Deck Strain

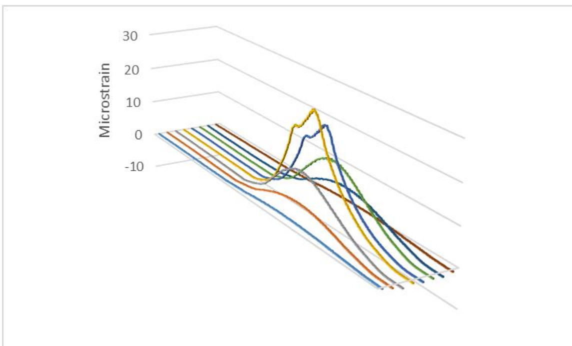
Figure 114. US 12 over Coffee Creek Load Path 2 results



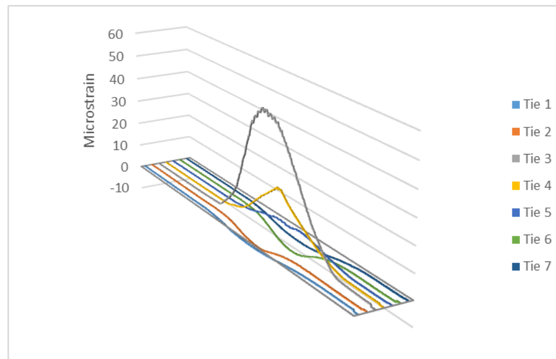
Girder Strain



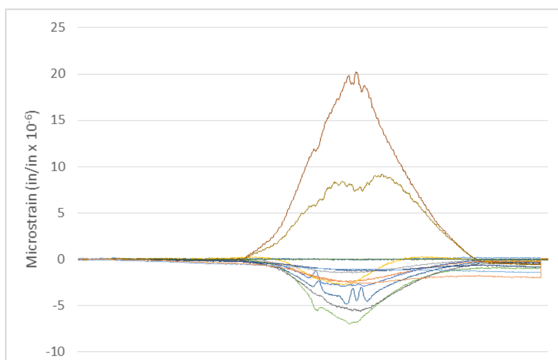
Tension Tie Strain



Girder Strain Distribution

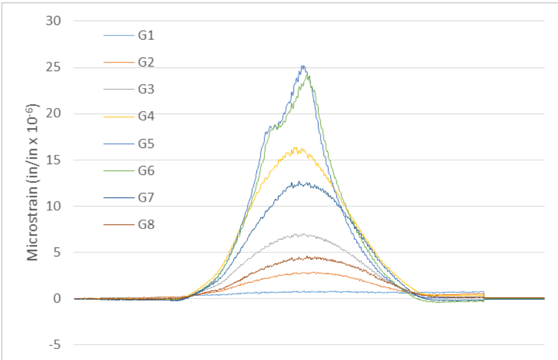


Tension Tie Strain Distribution

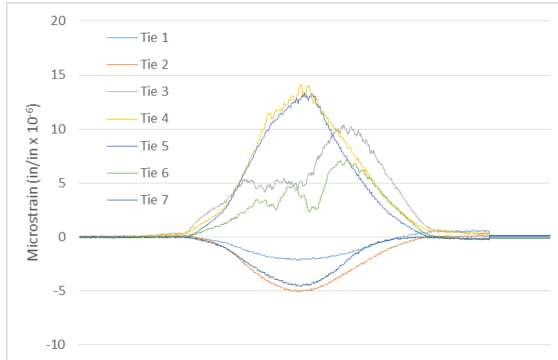


Deck Strain

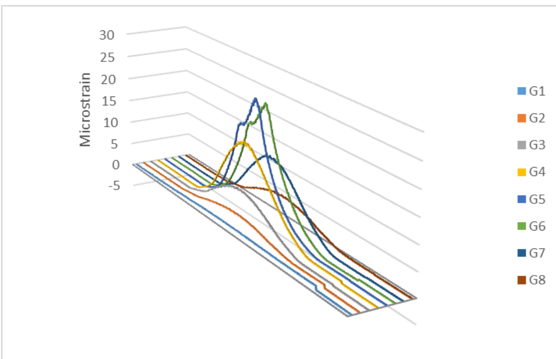
Figure 115. US 12 over Coffee Creek Load Path 3 results



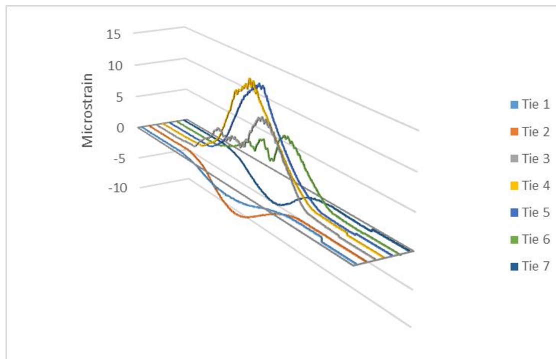
Girder Strain



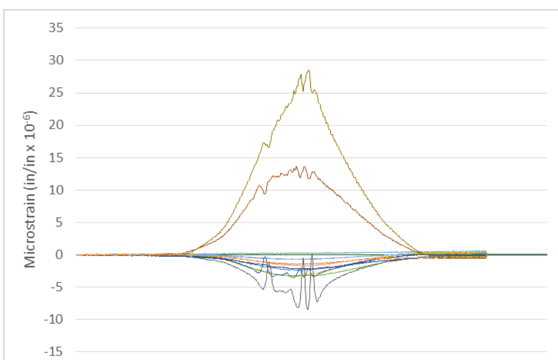
Tension Tie Strain



Girder Strain Distribution

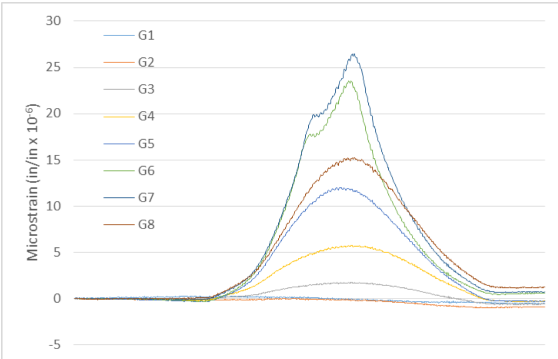


Tension Tie Strain Distribution

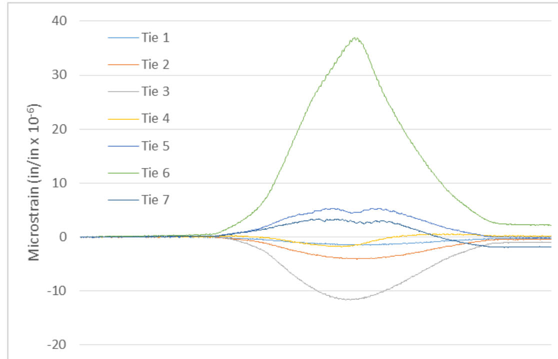


Deck Strain

Figure 116. US 12 over Coffee Creek Load Path 4 results



Girder Strain



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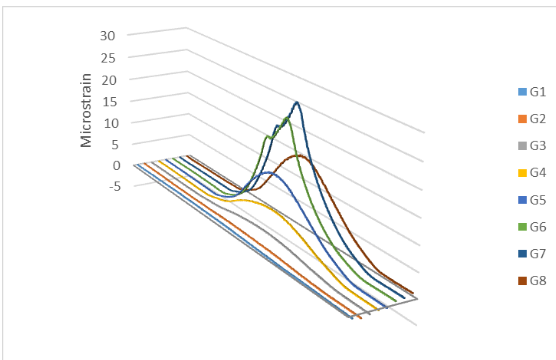
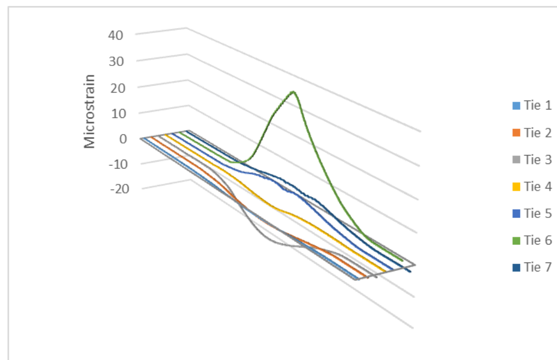
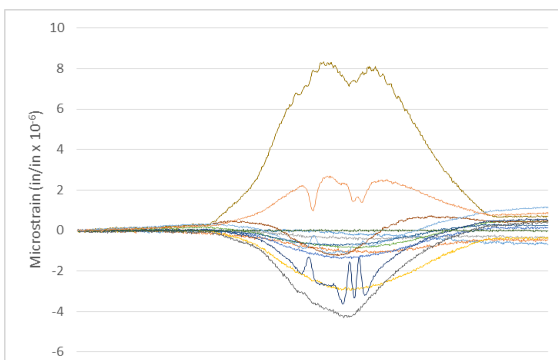


Girder Strain Distribution



Tension Tie Strain Distribution



Deck Strain

Figure 117. US 12 over Coffee Creek Load Path 5 results

The girder strain never exceeded 30 microstrain and the maximum never fell below 25 microstrain for any load path. To put this in terms of stress, the maximum never exceeded 120 psi (assuming a 5,000 psi compressive strength and associated modulus of elasticity). Note that the girder strain gages were not placed directly at midpsan where a greater strain is likely to occur, so the maximum strain values on the structure are probably slightly higher. The strain distribution appeared a bit unusual in comparison to traditionally constructed bridges. For example, typically the maximum strains will be located on girders directly under the load path. For each of the load paths evaluated in this work, the maximum strains tended to be at the middle girders of the bridge. Some of this phenomenon is likely due to the additional stiffness of the bridge near the east and west sides due to the sidewalk and railing assembly. Since the strain is a direct measurement, a girder of greater stiffness will register less strain when subjected to the same load as that of a less stiff girder. Even so, in cases like load paths 1 and 2, it is anticipated that a greater strain in relation to the center girders would have been achieved at the girders under the load. Seemingly, a larger percentage of the total load is being transferred to the center girders. It is not clear if this atypical load distribution pattern might create a situation where the bridge capacity may be less than needed; however, it is possible. For example, if the live load distribution factor assumed during design is less than that actually occurring, a potential for overloading does exist.

In a similar way, the behavior of the tension ties appeared unusual with respect to the range of strain magnitudes. For all of the load cases except for load path 4, all tension ties aside from one had strains in the range of approximately +/- 10 microstrain. For the tension tie located in the third bay from the east the strain measurement consistently exceeded the average strain magnitude by a significant amount (e.g., 70 microstrain for load path 1). Only in load path 4 did the strain magnitude come into the range of the other tension ties. Keep in mind that measurements were only taken on one line of the many tension tie sets across the bridge. It is possible that discrepancies in the measurements could be seen across many sets and in different patterns. Nonetheless, the strain data speak to the likelihood that achieving a consistent tautness in the ties is very slim causing variation in the load transfer behavior. Even more, the tension tie is an important part of the entire system contributing to the strength of the deck. It is the opinion of the research team that the tension ties may have been placed too low in the girder cross section to allow for the full development of the strut-and-tie behavior. Visually, the tension ties appeared to be in good condition.

The deck strain behavior was fairly consistent with what has been measured on traditionally constructed bridge decks. Gages located in bays, which are in line with a wheel line, tend to be more sensitive to individual wheel crossing overhead. This is evident by the more abrupt reversals shown in the strain plots. Strain measurements away from the wheel line tend to gradually rise or fall with relatively small and similar magnitudes. Whether tension or compression is measured is often determined by the bay position with respect to the wheel line bays, as in the case of a multi-span beam with a point load on a single span.

Conclusions and Recommendations

Bridge B-27-0150, located on US 19 over Coffee Creek in Black River Falls, WI, was constructed in 2008 using a steel free deck and FRP stay-in-place forms as part of the FHWA IBRC/IBRD program.

The condition of the bridge is not unlike bridges of similar age. The underside of the superstructure appeared in very good condition, while the topside was in good condition with only a few apparent small cracks on the bridge deck. To this point, it is unknown if the cracks are a function of the steel free deck or of another cause.

The load test results indicate that the steel free deck and tension tie mechanism transfer load differently across the bridge than what would otherwise typically be seen on a traditionally constructed bridge deck with steel reinforcement (as is to be expected). For most bridge tests, the maximum girder strain magnitudes are most typically observed directly under the load path. In this case, the maximum girder strain magnitudes were generally nearer the centerline of the bridge regardless of the respective load path position. In future designs, it is recommended that either greater than normal distribution factors be used or that an analytical model be created that considers the reduced deck stiffness.

The behavior measured in the tension ties appeared inconsistent with the respective load paths. In most load cases, one of the tension ties was receiving a greater percentage of the load than the others. Note that only a single line of tension ties was instrumented across the bridge and that the behavior measured on the instrumented line may not be what is measured on another line. In fact, it is likely that the behavior would be different between individual lines since each tension tie is individually tightened and the continuous tightening of ties along the length of the bridge and in all bays effects the previously tightened ties by inducing more tension or relaxing the tie.

The strain measurements taken from between girder lines on the bottom side of the deck indicate that the localized behavior of the deck is similar to that of a traditionally constructed bridge deck. Tension or compression strains were measured across all the bays respective of the gage position in relation to the wheel lines. Those gages located on the wheel line were more sensitive to individual wheels crossing overhead as evidenced by abrupt spikes in the data, contrasted with the gradual increase and/or decrease in strain measured by the other gages.

Though the data would indicate that some of the behaviors of this innovative design differ from the behaviors anticipated with a traditionally constructed bridge, the bridge still functions well. With respect to the deck construction, there is likely cost savings by using the stay-in-place FRP forms and no steel within the deck. That said, specific attention should be paid to deck crack growth going forward to identify any crack patterns that will be revealed in the future. To date, although a few cracks exist, there is no repetition in the crack pattern that would indicate a failure mechanism inherent to the deck construction.

The continued use of bridges of this type are recommended in Wisconsin with the caveat that observation of the structure be continued over the coming years to identify any potential degradation patterns specific to the construction method. To this point resistance to the use of steel free decks has been based more upon the idea that they may not perform as well as traditionally constructed decks. Research, testing, and trials of these types of decks have consistently shown that the behavior and condition is no worse than traditional decks. Perhaps the largest impediment to widespread implementation is the idea that “we’ve always had steel in our decks”.

B-45-0095: STH 33 over the Milwaukee River,
Saukville, WI – Use of Bi-Directional Post-Tensioning

General Information

Bridge B-45-0095 is located on STH 19 over the Milwaukee River in Saukville, WI. The bridge was constructed in 2005 using an innovative structural design, a slab superstructure with longitudinal and transverse post-tensioned tendons. To allow continuous traffic over the river throughout the duration of the project, the old bridge was removed in multiple stages while at the same time the new bridge was constructed in multiple stages.

Description of Innovative Feature

The bridge shown in Figure 118 is a four span structure and measures 250’-0” x 67’-0” in total length and width, respectively.

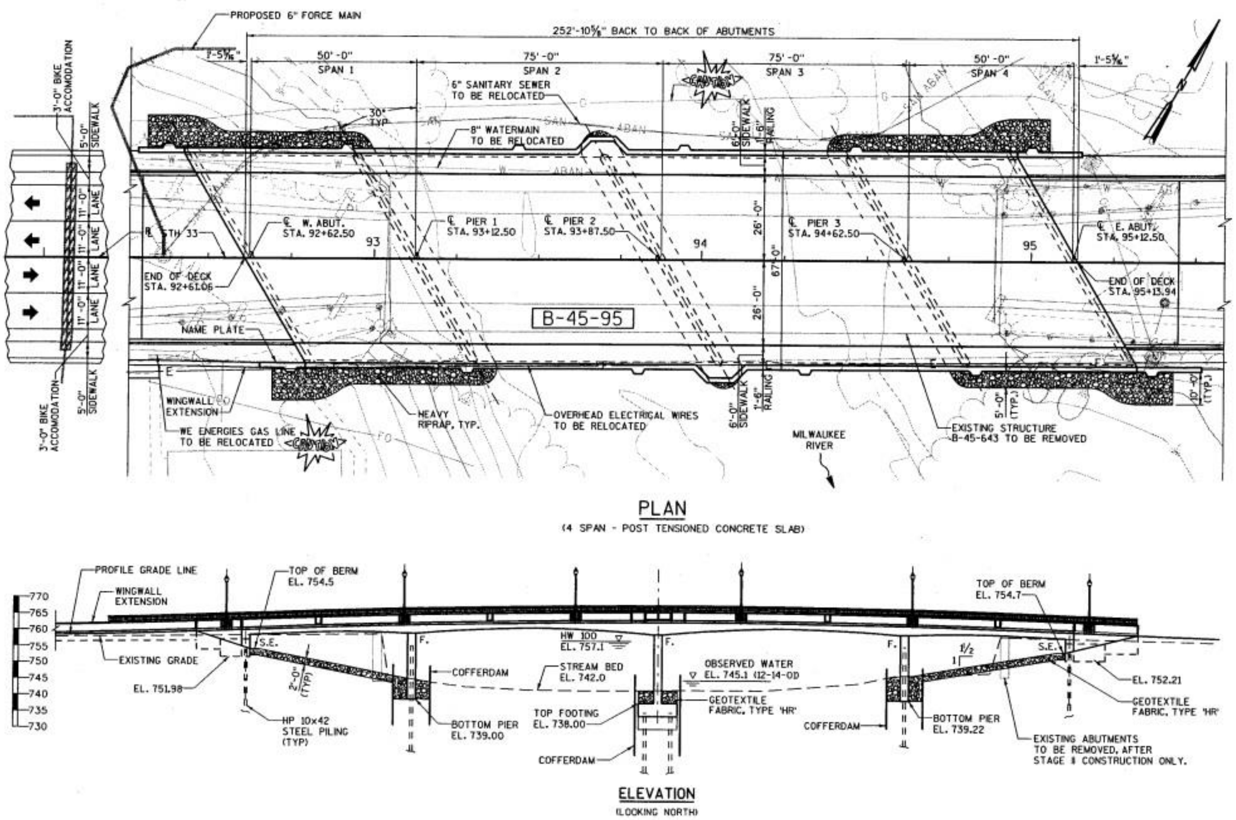


Figure 118.STH 33 over the Milwaukee River plan and section views

Both entry spans measure 50’-0” and the intermediate spans measure 75’-0”. The depth of the cast-in-place concrete superstructure varies along each span, becoming thickest near the abutments (32”) and piers and thinnest at midspan (18”).

The superstructure is mildly reinforced with #6 bars at 12” o.c. transverse top and bottom and #5 bars at 12” o.c. longitudinal top and bottom. The post tensioned tendons are placed within the top and bottom reinforcing mats numbering 20 in the longitudinal direction and 38 in the transverse direction. Figure 119 and Figure 120 show the tendon configuration for the longitudinal and transverse tendons, respectively.

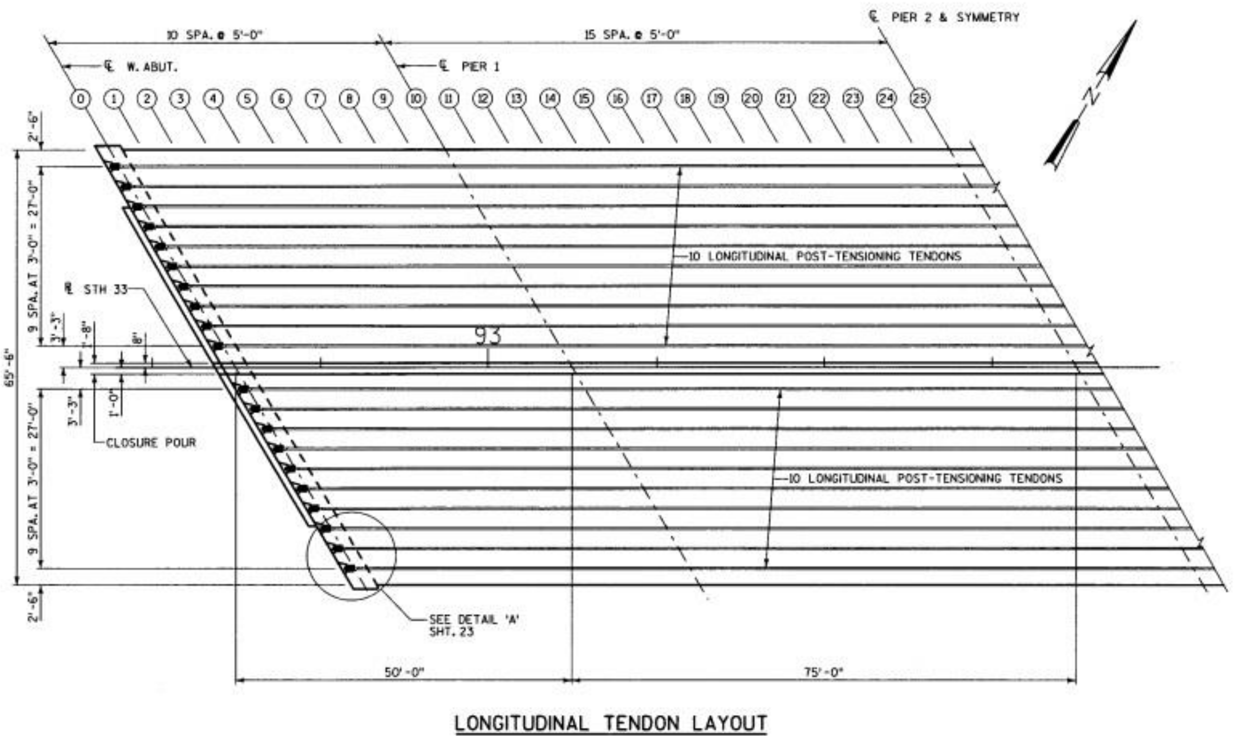


Figure 119.STH 33 over the Milwaukee River longitudinal tendon configuration

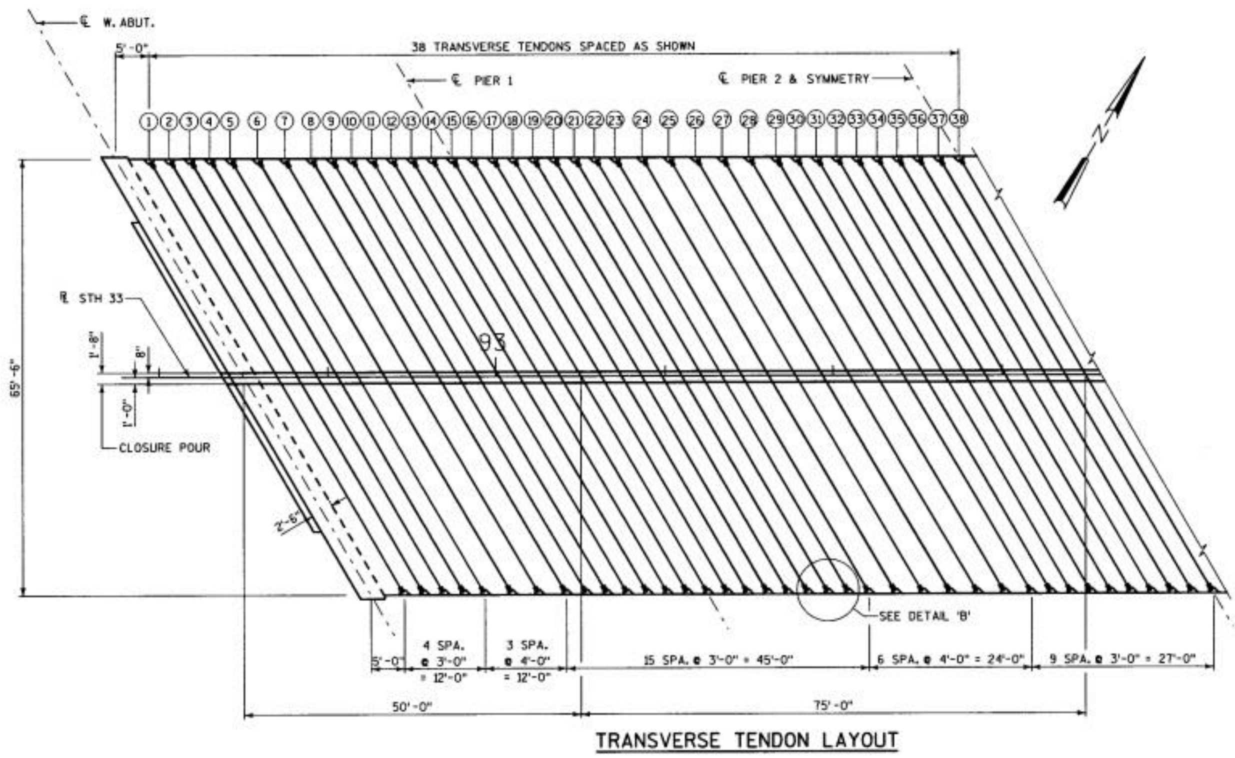


Figure 120.STH 33 over the Milwaukee River transverse tendon configuration

Use of Innovative Feature

The decision to use bi-directional post tensioned tendons was partially driven by the necessity for staged construction and a desire for an arched slab bridge. Using the tendons allowed for an overall shallower structure than what would otherwise be required given the span configuration.

Visual Observation

On August 14, 2015, the bridge was inspected for any signs of distress or underperformance. A visual inspection of the superstructure was completed with the specific goal of identifying any cracks that were present in the post-tensioned superstructure.

The underside surface of the superstructure was in excellent condition. No cracking or efflorescence was observed at any of the four spans. Examples of the underside condition are shown in Figure 121 and Figure 122.

Figure 121.STH 33 over the Milwaukee River surface condition of superstructure – south side Span 2

Figure 122.STH 33 over the Milwaukee River minor cracking at bridge entrance – north side Span 2

Anchorage points, although solidly grouted and, thus more difficult to fully assess, also appeared to be in very good condition.

The two main stages of bridge construction are divided by a closure pour through which the transverse post tensioned tendons pass. The condition of the closure pour was of particular interest since there is a cold joint present on either side between the north and south halves of the bridge. The closure pour, seen in Figure 123 and Figure 124, did not appear to be in a condition any poorer than what was observed on the remaining superstructure.

Figure 123.STH 33 over the Milwaukee River closure pour condition

Figure 124.STH 33 over the Milwaukee River closure pour close-up

No signs of water infiltration were present, which could indicate a relaxation in the transverse tendons.

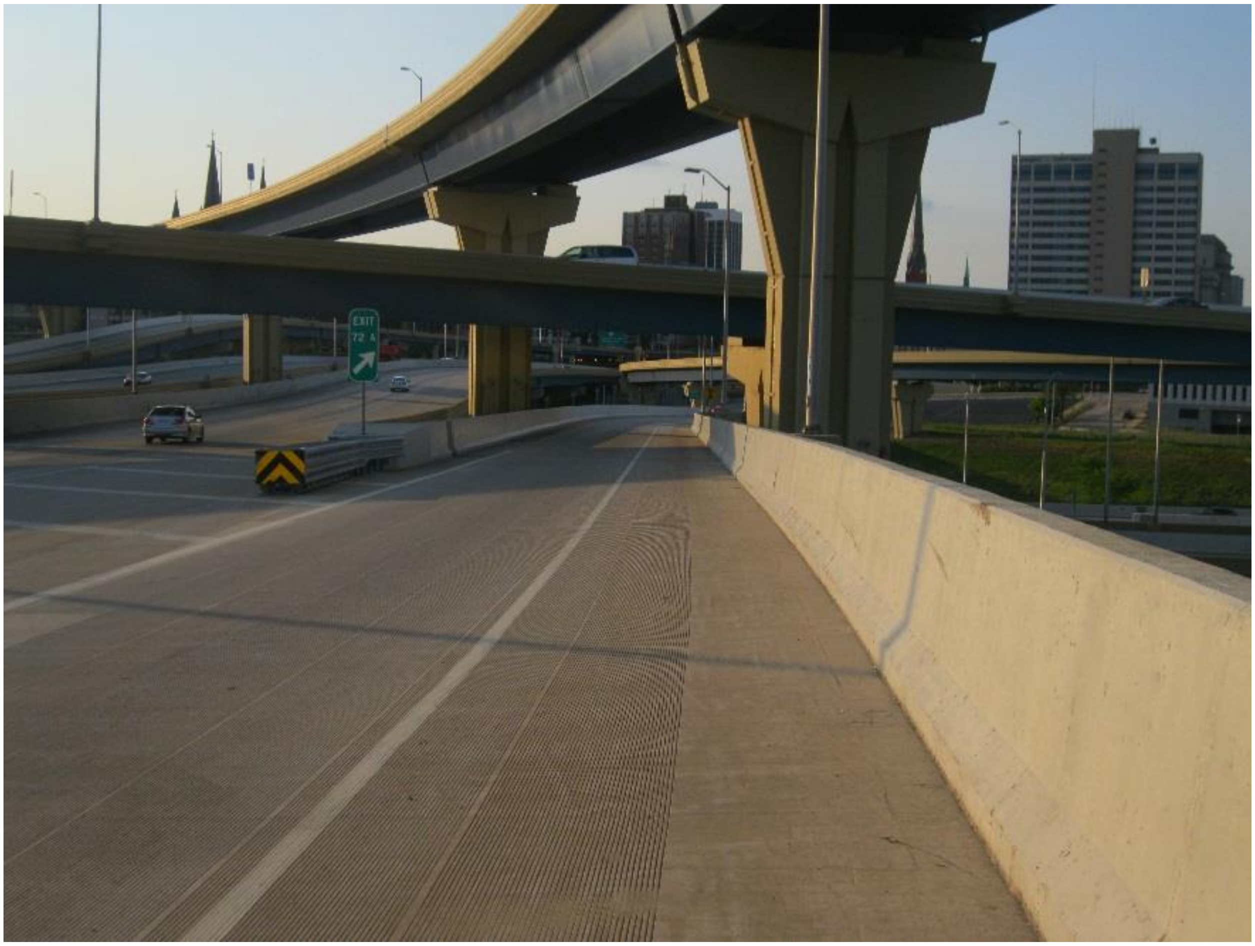
The top surface of the bridge, shown in Figure 125, has been overlaid with asphalt and thus the condition of the concrete surface could not be verified. The overlay was placed in 2006 primarily to seal the deck and secondarily to improve the ride.

Figure 125.STH 33 over the Milwaukee River asphalt overlay

Even so, the asphalt appeared in very good condition and given the condition of the underside of the deck, it is assumed the top surface is also in very good condition.

Conclusions and Recommendations

The use of bi-directional post-tensioned tendons in a cast-in-place deck has proven to this point to be an effective innovative bridge construction method. Though the method is unlikely to supplant traditional bridge construction methods with respect to cost and constructability, it does has a place for where the site restrictions and user needs dictate an unconventional method. The bridge is in very good condition and is not showing any signs of distress that would indicate a failing component. It is concluded that the design be an option for projects of similar requirements.



B-40-1132: IH 43 Northbound to Michigan Avenue,
Milwaukee, WI – Use of Stainless Steel Reinforcement

General Information

Description and Use of Innovative Feature

Bridge B-40-1132 was constructed in 2004 in a similar fashion to conventional bridges except that the bridge deck steel reinforcement was stainless steel in lieu of epoxy-coated reinforcing steel bar.

This innovative feature aims to extend the service life of the bridge deck to an age greater than that of a conventionally constructed bridge by taking advantage of the corrosion resistance of stainless steel. The bridge deck consists of an 8 1/2” concrete slab with a 2” wearing surface overlay, similar in thickness to that of a conventional deck. The reinforcement consists of longitudinal bars spaced at 7 1/2” o.c. top and bottom where the clear cover is 2 1/2” and 1 1/2”, respectively, and transverse bars spaced at 7” or 7 1/2” (non-pier/pier locations).

As part of the Marquette Interchange in Milwaukee, this bridge is subjected to heavy dosages of chlorides, and dosages that often expedite the rate of degradation in uncoated rebar or even polymer coated rebar bridge decks. Even more, the bridge serves a large number of vehicles per day in an area where repair or reconstruction is highly undesirable. Accordingly, observation of the deck performance, even at this early stage in life, will help those responsible for its maintenance and eventual replacement best plan for those respective procedures, if necessary.

On August 14, 2015, the researchers conducted a cursory review of the deck condition to determine if there were any areas where uncharacteristic degradation has occurred. Being as the observation was non-invasive, the ability to observe degradation at the reinforcing steel bar level was not possible. Despite this fact, the top and bottom surface of the deck could be observed and a general assessment could be made.

Visual Inspection Results

The bridge serves as an exit ramp off of I-43 northbound to Michigan Avenue. Its alignment is generally straight with a slight curve approximately at the mid-length of its four spans as shown in Figure 126.

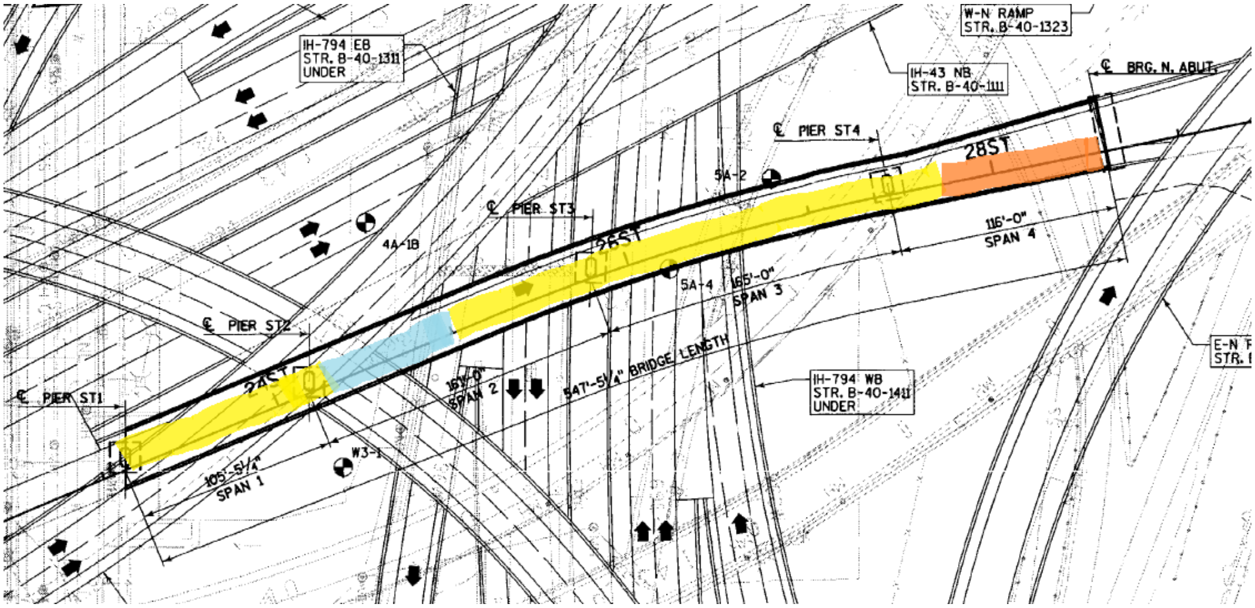


Figure 126. IH 43 northbound to Michigan Avenue transverse crack pattern

The bridge deck appeared in very good condition with no apparent cracking aside from a commonly occurring transverse cracking pattern over the full length of the bridge where transverse cracks were seen from parapet to parapet. The transverse crack spacing varied depending on the longitudinal position. The cracks were spaced at approximately 4’ o.c. over the length of span 1 (indicated in yellow), 10’ o.c. over the first half of the length of span 2 (indicated in blue), and 4’ o.c. over the second half of span 2 and span 3 (indicated in yellow). The cracks continued in this way through approximately the first quarter of span 4 and then became very sporadic over the remaining length of span 4 (indicated in orange).

As shown in Figure 127, the cracks were hairline in size, although clearly visible.

Figure 127. IH 43 northbound to Michigan Avenue typical transverse crack size

Several photos of the shoulder facing south (left in Figure 126), which are typical of their respective location descriptions, are provided in Figure 128 through Figure 131. The cracks were highlighted with chalk to aid the visibility in the photographs.

Figure 128. IH 43 northbound to Michigan Avenue Span 1 transverse cracks

Figure 129. IH 43 northbound to Michigan Avenue south end of Span 2 transverse cracks

Figure 130. IH 43 northbound to Michigan Avenue Span 3 transverse cracks

Figure 131. IH 43 northbound to Michigan Avenue Span 4 transverse cracks

It was common to observe cracking in the parapet walls at or near the location of the intersection between the transverse deck crack and parapet. This condition is shown in Figure 132.

Figure 132. IH 43 northbound to Michigan Avenue parapet wall crack

Areas where water and chlorides have penetrated and passed through the deck were visible from below. This was apparent by the white deposits coinciding with the transverse cracking observed on the top of the deck. A few examples of the observed leaching and efflorescence are shown in Figure 133 through Figure 135.

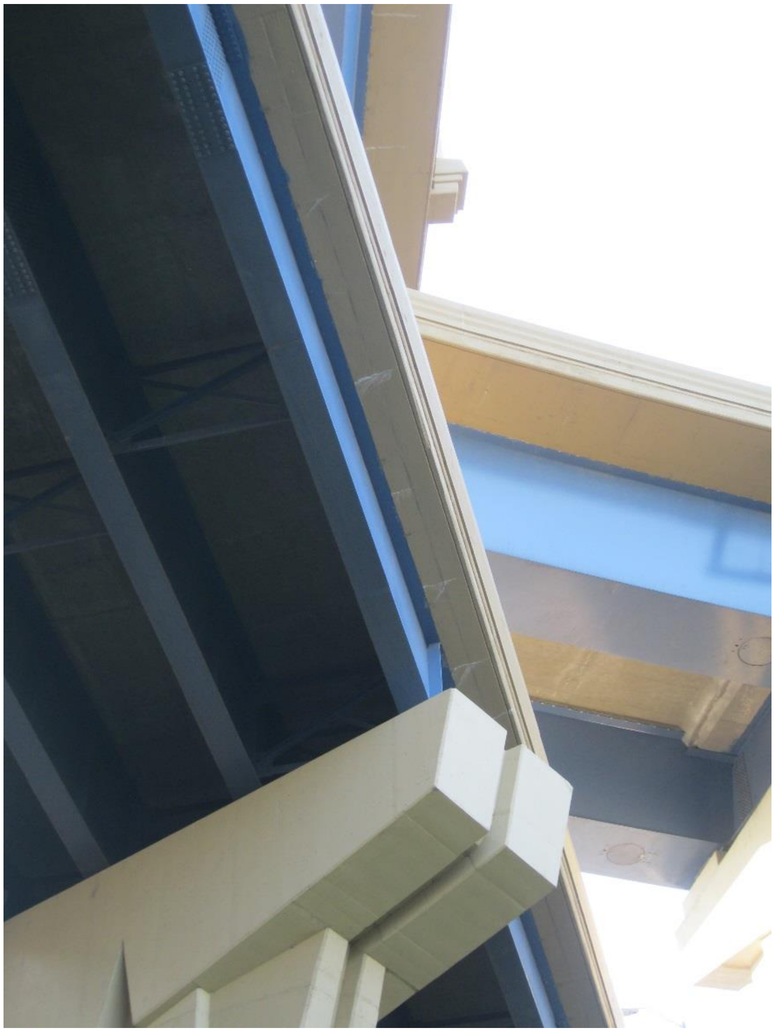


Figure 133. IH 43 northbound to Michigan Avenue chloride leaching

Figure 134. IH 43 northbound to Michigan Avenue chloride staining on deck underside – 1

Figure 135. IH 43 northbound to Michigan Avenue chloride staining on deck underside – 2

Conclusions and Recommendations

The bridge deck generally appears to be in good condition. Even so, numerous transverse cracks exist and it is apparent that water and chlorides have infiltrated the deck through these cracks and have propagated through the deck. Given this condition, it becomes clear that rebar with some type of corrosion resistance is desired to maintain a serviceable deck. Epoxy coating of rebar has shown to provide a much greater resistance to corrosion than uncoated black bar. That said, the epoxy on epoxy-coated rebar is subject to damage (nicks and cuts in the rebar, commonly known as holidays) during the construction process and corrosion could become a problem at these damaged locations. Some research indicates that, when holidays are present, epoxy-coated reinforcing steel actually corrodes faster than uncoated reinforcing steel. Conversely, due to the uniformity of the material, stainless steel is not subject to corrosion in the same way. Given that Bridge B-40-1132 is a critical element in the city's infrastructure, its corrosion should be limited to the greatest extent possible to eliminate perpetual maintenance practices or reduced service life. For that reason, the use of stainless steel reinforcement in this bridge can be considered a successful use of the innovative feature.

For future projects of similar magnitude, it is recommended that stainless steel or other non-corrodible emerging technologies be considered. The increased cost should be weighed against the potential for future longevity of the bridge deck. A study completed by the research team as part of another project evaluated the economic viability of the use of stainless steel reinforcement. During this evaluation, traditionally reinforced decks were cost-compared with decks reinforced with stainless steel. From this study it was found that from an economic perspective, the use of stainless steel was only economically viable when the so-called empirical design procedure was followed. One concern associated with empirical design is that some feel that it leads to a greater level of cracking – and, thus, corrosion of traditional reinforcing steel. In situations where stainless steel is used, this is obviously less of a concern. Future applications in which stainless steel might be used should consider the coupled use of empirical design procedures.

B-55-0217: US 63 over the Rush River,
Baldwin, WI – Use of Precast Substructure Components

General Information

Bridge B-05-0217 is located on US Hwy 63 over the Rush River near Baldwin, WI. The bridge is approximately 50 feet in length and 44 feet wide. The replacement of this bridge provided an opportunity to implement a method of accelerated bridge construction using a precast substructure system. Construction of the substructure began during the summer of 2008 and was quickly completed. Upon completion, the superstructure was constructed in a traditional manner.

Description of Innovative Feature

The substructure system shown in Figure 136 is composed of precast concrete panels measuring approximately 11 feet in width and 10 feet in height.

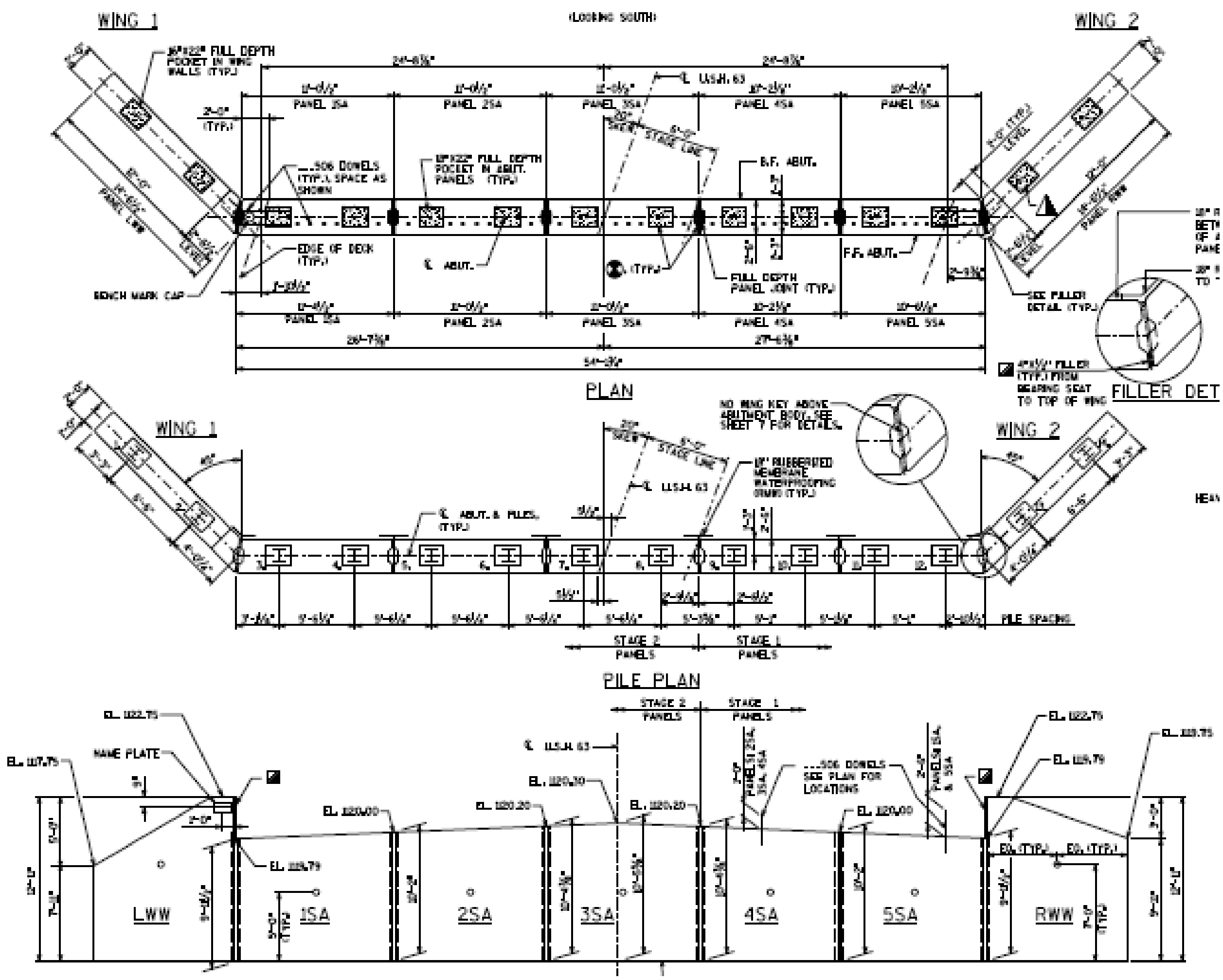


Figure 136. US 63 over the Rush River precast abutment panel plans

The panels collectively make up the abutment at each end of the bridge. Two full-depth pockets are formed into each panel from top to bottom. To begin the construction, steel H-piles are driven in a traditional manner corresponding to the pocket locations. After, the panels are lowered onto the piles through the formed pockets and the pockets are grouted solid.

Additionally, shear keys are formed into the panels along adjacent edges and fully grouted. The end product resembles and functions nearly the same as a traditionally construction abutment.

Use of Innovative Feature

The advantage of using this system is having the ability to quickly construct a bridge abutment without having a need for highly skilled labor or specialized equipment. In this case, it was reported that with only 3 to 4 laborers all panels within one abutment could be set in place in one day and fully grouted the next. This allowed for formwork construction to begin on the third day. In comparison to the anticipated length of time for traditional abutment construction, using precast abutment panels was believed to save 1 to 2 weeks. Additionally, it was believed that with a more refined panel design and greater contractor experience the process could be further expedited.

Field Results

Visual Inspection Results

On September 14, 2015, the bridge was visited and a visual inspection was completed. The inspection focused on the vertical joints between panels and any grout pockets that could still be seen after the completion of construction (i.e., wingwalls). Additionally, a cursory review of the water-side panel faces was also completed to identify any signs of distress.

Overall, the joints and panels were in excellent condition with few noteworthy considerations (see Figure 137).

Figure 137. US 63 over the Rush River overall condition of precast panels

All panels that are under and protected by the superstructure showed no wear or unusual degradation, and likely appeared to be no different than the day they were installed. No cracking was observed at the joints, which would indicate differential movement between panels or within the faces of each panel (see Figure 138).

Figure 138. US 63 over the Rush River close-up condition of precast panels

As shown in Figure 139, only on the underside of the superstructure were a few cracks observed and these are not believed to be a reflection of the substructure construction but likely to have occurred regardless.

Figure 139. US 63 over the Rush River cracking in cast-in-place deck at abutment

The wingwall panels were the only panels where degradation was observed. As shown in Figure 140, the tops of the grout pockets are exposed to the elements without any sort of protection.

Figure 140. US 63 over the Rush River wingwall-exposed grout pockets

At several of the grout pockets, cracks radiated from the corners outward to the panel face (see Figure 141 and Figure 142).

Figure 141. US 63 over the Rush River wingwall cracking at grout pocket top surface

Figure 142. US 63 over the Rush River wingwall cracking at grout pocket vertical surface

Inherently this is the weakest location of the panels because of the transition from full-thickness concrete (2’-6”) to that at the grout pocket (6”). The possibility exists for water entry into the grout pocket along the interface of the precast panel and grout and any subsequent freezing/thawing could induce the cracks observed.

Performance Evaluation and Summary

The use of a precast substructure system on this bridge demonstrated several advantages over traditional abutment construction, especially in cases where the closure of a bridge for extended periods of time could be costly to road users. This project showed that, even on a short bridge of average width with better than average accessibility, time could be saved over that of traditional abutment construction.

The simplicity of construction contributed to the overall success. Only a few laborers were able to fully construct the abutments in a couple of days without the use of specialized equipment.

There were no apparent failures of the system, although some improvements could be made. The design of the panels could be refined so that the weight could be reduced; currently, the panels are overdesigned. Improvement in this area coupled with a contractor who is more comfortable with the process would likely further expedite the construction process.

Recommendations

The use of this system is recommended where the expedited construction warrants its use. There are no apparent limitations of this system for smaller or larger bridges. The protection of the wingwall grout pockets should be addressed to avoid any unnecessary degradation to the precast panels. The implementation of a protection system would be achieved by placing a barrier between the end of the grout pocket and the elements.