DESIGN AND CONSTRUCTION OF A REINFORCEMENT FREE CONCRETE BRIDGE DECK ON PRECAST BULB TEE GIRDERS

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ABSTRACT

This presentation describes the concepts involved in design and construction of a reinforcement-free bridge deck intended to provide cost effective, durable and more rapid construction for highway bridges. The new wide flanged or bulb tee precast prestressed concrete girder sections currently used in bridges allow for a shorter effective span for the bridge deck because of the wide girder flanges. The failure mode of concrete decks on these girders is expected to be a punching shear mode as the span of the deck becomes shorter. The shear forces from the vehicle wheel loads in the deck of this innovative bridge are designed to be carried by restrained membrane action without using reinforcing in the concrete. The restrained membrane action is enhanced by the natural lateral stiffness of the wide girder flanges and by tying adjacent girders together with steel tie rods placed between the webs. A construction case study of the newly developed system as applied by the Wisconsin DOT in a prototype bridge on a U.S. highway and the relevant development and testing conducted by the University is included.

Keywords: Bridges, Bridge decks, FRP reinforcing, Stay in place forms, Bulb tee girders

INTRODUCTION

United States designers are using new types of precast concrete bridge girders to reduce costs and increase efficiency. These new girders are referred to as bulb-Tees or wide flanged girders. They typically have top flanges that are 4 ft to 6 ft wide and are often spaced 6 ft to 10 ft center to center. The wide flanges result in a short span for the bridge deck, less than 6 ft, and traditional methods of deck design for flexural moment may no longer be appropriate or efficient.

The research described in this paper is aimed at developing an innovative approach to designing and building bridge decks on these precast concrete girders. This new approach is intended to allow more rapid construction, improve the deck durability, reduce costs and improve efficiency by making the design process reflect the actual deck behavior.

Bridge deck load testing conducted as part of this project, as well as tests described by others[1-4], show that decks on short spans fail in punching shear rather than flexure. While moving wheel loads can create serious flexural cracking, ultimate strength failure occurs in a shear mode. This type of failure would appear to cause a dilemma for bridge designers because deck design is now and has historically been based on an assumed flexural failure. The AASHTO LRFD Bridge Design Specifications[5] ask a designer to determine flexural forces in a strip of slab that spans across girders and to size the slab and reinforcement to resist the bending. Shear design is not required.

The new design procedure is based on deck shear capacity, rather than flexure. The deck is designed to resist shear in combination with compressive membrane action. Deck compression is created by constraints from the remainder of the deck away from the area of loading, the adjacent girders, and special steel bars used to tie girders together. This approach eliminates the need for flexural reinforcement in the deck to provide strength.

A prototype bridge was constructed in 2007 during the development of this new design and construction process. Prior to constructing a bridge, the behavior of the deck system was investigated through review of research, laboratory testing, linear and non-linear analysis and simplified modeling. The analysis and testing was specially employed to develop construction details that would increase the speed of construction and deck durability while reducing construction costs.

DECK BEHAVIOR

Normal design procedures base bridge deck thickness and reinforcement on flexural bending moments obtained by examining a transverse strip removed from the deck and loaded with design wheel loads. The strip is assumed to be on rigid supports where it crosses the tops of girders, but is continuous over those supports.[5]

Laboratory tests of short span bridge decks prove, however, that failure occurs in a punching shear fashion rather than in flexure.[1-4] Under moving fatigue loads the crack patterns before failure differ considerably from those found under static loading and the capacity is reduced, but failure still appears with punching.[6, 7] If deck design were based on shear failure, there would be little necessity for requiring extensive flexural reinforcement.

Fig. 1 exhibits the type of behavior that would be expected in a bridge deck on bulb-Tee girders if only one-way force resisting action existed. Before the deck reached the state of Fig. 1 it would naturally try to resist the point (wheel) load through flexure. But as the wheel load increases the flexural cracking capacity in negative bending at the girder and positive bending at midspan will be exceeded and cracking occurs. The condition shown in Fig. 1 would develop with the crack locations like hinges if traditional flexural reinforcement is not provided. If the adjacent slab and girder provided no lateral constraint the slab would collapse as a mechanism.

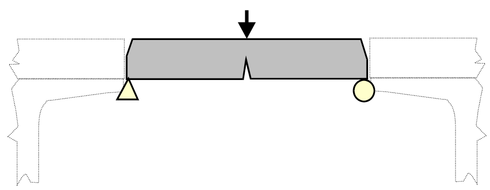

Fig. 1 Flexural cracking that would develop in deck strip.

If, however, the girders and adjacent slabs are laterally stiff and constrain the deck as shown in Fig. 2, then collapse cannot occur because the compression struts in the deck will continue to resist the load. The capacity of the deck in this cracked state depends on the lateral constraint provided by the rest of the structure in developing the horizontal compression component. Clearly the effectiveness of the compression struts in resisting the vertical load depends on their angle to the horizontal. With longer spans between girders, they would be less effective. The close spacing of flange edges in precast bulb-Tee girders, however, is ideal in generally making this span short and creating an effective strut.

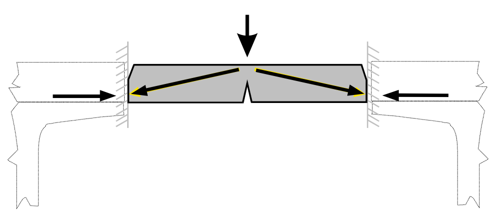

Fig. 2 Behavior after flexural cracking in a constrained deck strip.

The actual compression strut in the deck varies in width and in compression stress along its length and might be modeled using strut and tie behavior as suggested in Fig. 3 (where the tie shown is actually replaced by the constraint force of adjacent material). Detailed inelastic finite element analysis of the stresses developed in the deck plus bulb-Tee system indicates that this is the manner in which the deck-girder system actually resists wheel loads. The compression stresses, represented by the direction and length of lines of predicted stress in Fig. 4, appear to match the simple modeling of Fig. 3.

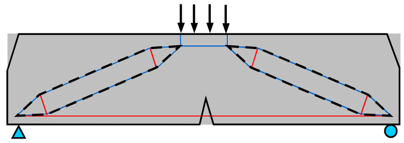

Fig. 3 Compression struts that develop to resist loads.

In fact the system illustrated in Fig. 3 may be the essential approach needed to actually predict the failure strength of a constrained deck slab. The model shown, combined with compression field theory for concrete, could predict the punching shear failure mechanism illustrated in Fig. 5. Thus, it seems that simple strut-tie design of a bridge deck system may offer a much more rational approach to deck design than the current flexural method. In addition, since shear-punching is the actual mode of failure, the cracking mechanism of Figs. 1 and 2 can be accepted and no flexural reinforcement is required. This approach can significantly affect the construction speed, cost, and durability of a bridge deck. Since placement of two perpendicular layers of reinforcement is not necessary – construction labor and cost are reduced. Removing steel reinforcement from the deck has the potential of improving durability in climates where salts are applied for winter de-icing.

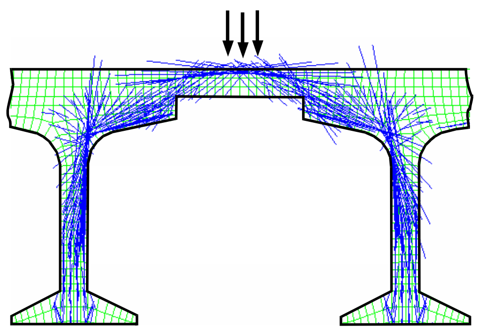

Fig. 4 FEM model with lines showing direction and size of compression stress.

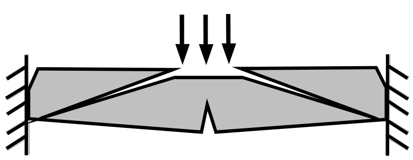

Fig. 5 Punching shear failure of constrained deck.

The punching shear behavior and the modeling technique has been illustrated above for a 2-dimensional deck strip between girders. In fact the actual failure is a 3-dimensional deck punching but the approach outlined for design may still be used in a modified version. Figs. 6 and 7 show the testing of a ½ scale bridge deck on bulb-Tee girders. Wheel loads were simulated by applying load over a scaled area of the deck surface in three different deck spans. Fig. 7 shows the underside surface of the deck after failure and the generally circular periphery of the punched surface.

Under moving loads and repeated fatigue loading the crack patterns developed in a deck will be slightly different than described above, and will include transverse cracks as well as longitudinal cracks. This pattern will reduce the capacity below the amount measured from the static tests but the deck failure should still be in punching shear.[6, 7]

Fig. 6 View of underside of multi-span test deck on bulb-Tee girders.

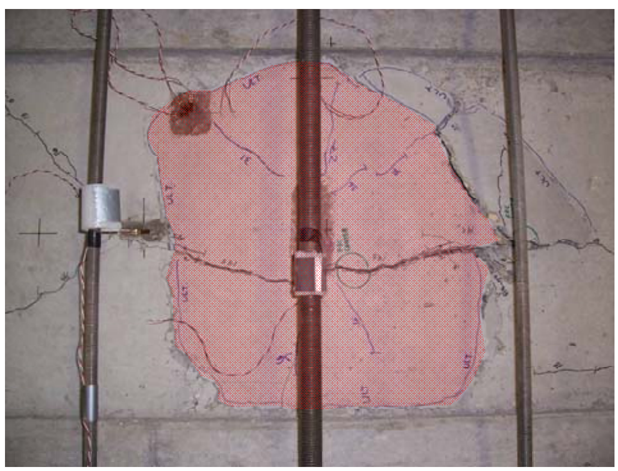

Fig. 7 Punch through failure surface (shaded) with girder at top and bottom.

IMPROVING DECK BEHAVIOR

The performance of the deck in resisting punching depends on the development of lateral constraint forces at the girder. As the lateral constraint increases, the punching shear capacity of the deck increases. A unique characteristic of precast concrete bulb-Tee girders is that they have high lateral stiffness, especially within the wide top flange. This girder stiffness, combined with the overall “diaphragm” stiffness of the surrounding deck provides a large amount of restraint to an area resisting wheel loads.

Canadian designers have been taking advantage of deck constraint in design of bridge decks built over steel and concrete “I” girders[8-10]. In the Canadian system special steel ties at the bottom of the deck are used to provide the constraining force, since the steel beams or concrete “I” girders do not possess the high lateral stiffness that may be available in bulb-Tee girders. In some instances the ties have been rectangular steel straps welded to the top flanges of adjacent beams with a concrete slab placed directly above. They have dubbed this system as “arch action” where a compression arch is assumed to develop within the deck concrete and the arch is constrained by the steel ties.

While bulb-Tee girders may not need the addition of steel ties to constrain the deck, the performance of the concrete deck may be further enhanced by using ties between the girders, but the relative contribution is not as great as found in the Canadian applications on laterally flexible steel and concrete “I” girders.

CONTROLLING CRACKING

Since punching shear is the expected mode of failure in the short span decks there is no incentive to provide multiple top and bottom layers of steel flexural reinforcement. Flexural reinforcement can be eliminated, but it is still desirable to have control over concrete cracking due to shrinkage. While fiber reinforcement usually does not provide much or any strength improvement to concrete, unless steel fibers are used in high volume, fiber reinforcement has been shown to be effective in controlling shrinkage cracks. Thus the steel reinforcement for flexural strength may be removed and shrinkage reinforcement can be replaced by using fiber reinforced concrete.

The behavior of a deck without flexural reinforcement may change significantly under service loading when compared to normally reinforced bridge decks. The deck naturally acts as a flexural member when load is first applied. As soon as the rupture stress of the concrete is exceeded the flexural crack at midspan shown in Figs. 1 and 2 may develop. Even though reinforcement is not required for strength, it would be effective in controlling the width of any individual crack and would induce multiple small cracks to form. Without reinforcement there is a tendency for a single large longitudinal crack to form at midspan of the deck and run parallel to the girders. The fibers in fiber reinforced concrete are not effective in controlling the size of this crack.

Studies of reinforcement free bridge decks in Canada have found that this longitudinal crack can be unsightly, though the capacity of the deck is not affected. As a result of the cracking found in those bridge decks it is recommended by the studies in Canada that the reinforcement free bridge decks be provided with a mesh of reinforcement near their bottom surface for crack control (no longer reinforcement free).[11]

The negative moment cracking over the girders is not of a concern since studies found that the crack occurs when the applied load was at about 80% of the failure load and the failure load level is much higher than the service load level.[12]

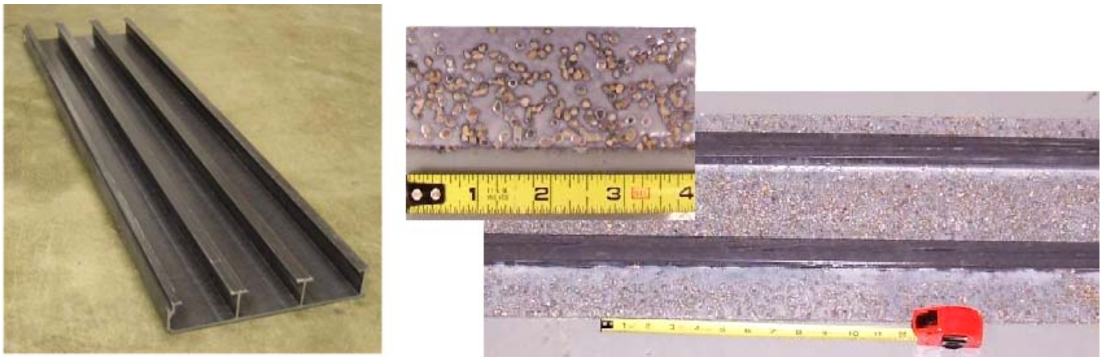

Fig. 8 FRP plank used as stay-in-place formwork with aggregate for bonding.

Bridge deck construction can be accomplished easier, safer and more quickly if normal formwork composed of wood or other removable forming is not used. In fact the short spans that often exist between the flanges of bulb-Tee girders make normal forming with plywood appear to be excessive. Lightweight fiber reinforced polymer (FRP) planks could be used as stay-in-place forming, replacing plywood and eliminating the need to remove forms, and might also serve as a durable crack control "secondary reinforcement" mechanism for the concrete deck.

FRP planks with glass reinforcement, as shown in Fig. 8, were examined to see if they could provide a secondary crack control function as well as formwork. These are common planks used in scaffolding, but turned upside down for this application. A thin layer of small aggregate is epoxy bonded to the surface of the plank to develop bond with the cast-in-place concrete deck.

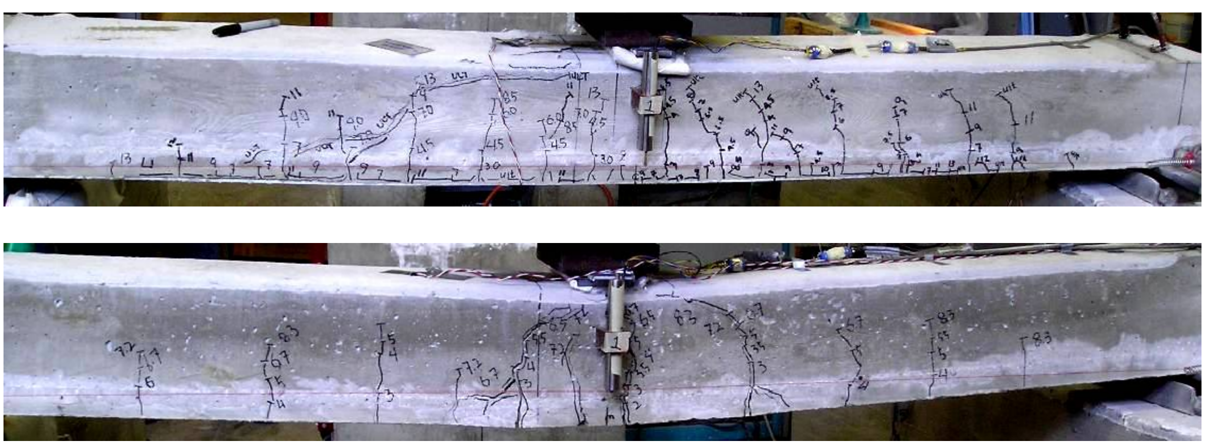

Fig. 9 Comparison of cracking in beam at top with FRP form and beam below with uncoated steel reinforcing bars.

The flexural behavior of a beam with just the FRP formwork is compared with a beam using the normal design amount of steel reinforcement for a bridge deck in Fig. 9. The FRP beam shows a better distribution of smaller cracks than the steel reinforced beam. If the steel reinforcement had been epoxy coated for durability, it would have been even less effective at controlling crack size with the few cracks. Clearly the FRP stay-in-place form serves as an excellent crack control mechanism by distributing strain to many small cracks.

In the design process no strength contribution from the FRP forming was assumed since sufficient bond at the ends is unlikely to develop within the region of the compression strut. The forms only provide the secondary function of crack control.

PROTOTYPE BRIDGE CONSTRUCTION

The Wisconsin Department of Transportation (WisDOT) in cooperation with the University of Wisconsin applied the use of a reinforcement free bridge deck on bulb-Tee girders to a bridge replacement project on U.S. Highway 12 in Wisconsin. The actual superstructure design plans and specifications were produced by Alfred Benesch, Inc. as a sub-contractor to the University.

The bridge is on a two lane highway with sidewalks and spans 100 ft. with a single span superstructure. In addition to the reinforcement free deck a number of other innovations were incorporated into this structure as listed below:

- steel tie rods placed between girder webs just below flanges at 6 ft spacing,
- FRP stay in place form system used as secondary crack reinforcement,
- girders placed at a slight tilt to allow the top flange to match the 2 % deck cross-slope,
- abutments were built with a 2 % top slope without separate steps for girder seats,
- girder flanges were detailed with recesses at the edges to hold FRP forms.

Fig. 10 shows the 54 in. deep bulb-Tee girders (WisDOT 54W sections) as delivered from the precaster (County Materials) just before placement on the abutments. In the design process tie rods were included to insure sufficient lateral constraint for the deck. The tie rods design is related to stiffness needed for constraint, not strength. Future work may prove that the tie rods are not necessary in all cases.

The contractor elected to place the 2 in. diameter galvanized tie rods through the girder webs while the girders were on the truck bed. This reduced the work required under more difficult conditions after the girders were placed. The ties were then slid into adjacent girders as in Fig. 11. Since the bridge was on a skew, the tie rods were not in a continuous line transversely, but rather offset between each set of girders to follow the skew (Fig. 11). Bearing plates and double locking nuts were attached at each end of the galvanized bars as shown in Fig. 12.

The girders were then placed on the abutments, but a level bearing pad was not used. Instead the bearing pad was set with a slope that matched the desired deck cross slope or crown.

With this type of bearing each girder had a slight tilt sideways to provide a 2 % slope. This slight tilt causes eccentric loading of the girders, with biaxial bending and torsion, but did not cause any significant additional undesirable girder stress. A solid concrete diaphragm cast between the girders at the abutments provided bracing for the girders.

The WisDOT decision to set girders with this tilt simplified the contractor's construction process. The abutments did not have to be built with stepped bearing pads, each set at a different elevation to provide cross slope. The girder top flanges had a cross slope that matched the deck so different depth haunches were not needed on each edge of every girder. Because the girder top matched the deck cross slope installation of the FRP stay-in-place forms was simplified.

Fig. 10 Placing galvanized steel tie rods through the 54 in. bulb-Tee girders.

Fig. 11 Placing bearing plates and nuts on the steel ties between girders.

The construction was further simplified by detailing the deck without any haunches over the girders. At midspan the deck was designed with a 7.5 in. thickness. Since the girders were cambered from the prestressing, elimination of haunches required the cast-in-place deck to be thicker at the abutments than at the midspan where upward camber is the highest. The roadway had a slight downward sag vertical profile where it crossed the bridge, further increasing the deck thickness near the supports. This increase in deck thickness near the supports had little impact on the girder flexural moments at midspan, but did increase the girder shear at the supports requiring addition of a few extra stirrups.

Fig. 12 Side view of bridge showing the tie rods, bearing plates and double nuts.

The superstructure is shown just before placement of the concrete deck in Fig. 13. The bridge construction was done in 2 separate stages for maintenance of traffic. Thus two separate deck pours occurred a month apart. The black FRP deck forms, visible between the girder flanges, came in 1 ft. wide pieces precut to length and were easily carried and placed by a single worker. Though it is not apparent in the photo, the two girders at the far right tilt in two opposite directions. The centerline of the roadway falls between the girders, requiring different cross slopes on either side to create the roadway crown. The concrete was reinforced with a combination of mono-fiber and fibrillated fiber polypropylene fibers used at the combined rate of 5 lbs per cubic yard of concrete.

Fig. 13 Bridge superstructure before deck pour with black FRP forms.

The total cost of the galvanized tie rods and attachment hardware was approximately $15,000 in this bridge, less than the expected cost of normal reinforcing bars and labor for placement. The cost for the stay-in-place FRP formwork was about $15/sq. ft. in place. The FRP planks, without the sand coating or placement labor, generally cost about $5/sq. ft. which is similar to the per square foot cost for Wisconsin contractors to remove deck formwork. The planks were cut to length and the sand coating was applied by the manufacturer.

As part of the overall development program for this new bridge system load testing and monitoring over time will be used to evaluate the behavior and performance of the system. Gages are embedded in the deck concrete to measure crack widths and concrete compression strains at various locations and gages are permanently attached to the girder tie bars to measure tie stresses and to determine the amount of deck constraint actually developed by use of the tie bars shown in Fig. 13.

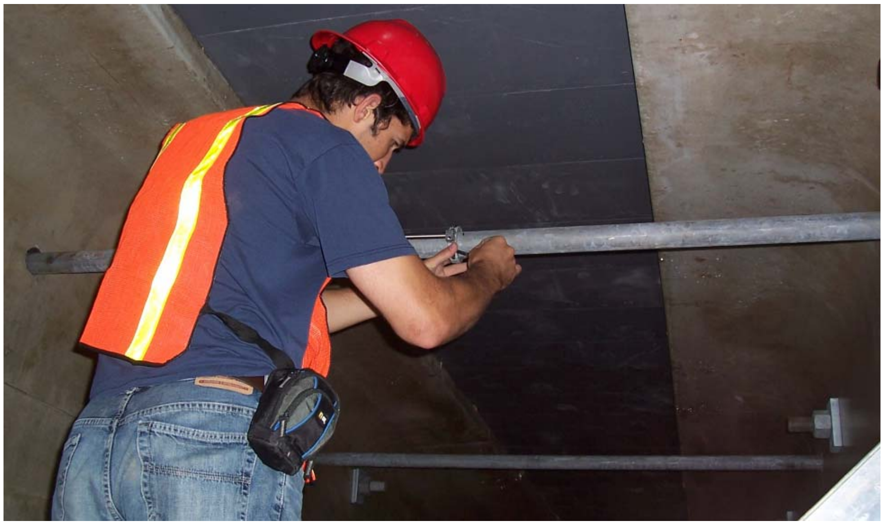

Fig. 13 Permanent instrumentation being attached to girder tie bars.

PROTOTYPE BRIDGE BEHAVIOR

After one month of usage the bridge was load tested and the deck examined to evaluate preliminary performance of the system. Though the duration of usage was obviously too short assess the durability of the system, the first evaluation provided a positive assessment of the system performance.

Virtually no cracking was observed to have developed in the half portion of the bridge cast as part of the stage 1 construction. Only two small cracks, of 4 foot length, had developed over the end diaphragm near the corner of the deck with the acute angle. These were likely the result of shrinkage restraint caused by the diaphragm which had been cast before the deck. The deck in the stage 2 construction had transverse cracks starting from the joint with the stage 1 deck and running from 6 ft. to 12 ft. away from that joint. The cracks were spaced at an average distance of 10 ft. apart longitudinally. These cracks appeared shortly after casting and were judged to be shrinkage cracks caused by short reinforcement dowels that were used to join the two stages at the construction joint. The dowels allowed the stage 1 deck to restrain shrinkage of the stage 2 deck.

Measurement of deck strains during the load testing showed that the deck stresses were very low, less than the rupture tension stress, and would be below the cracking rupture stress under design service plus impact loading. The steel tie rods were working, but the stress levels in the tie rods were also low.

SUMMARY

Traditional methods of design for highway bridge decks are based on providing sufficient flexural strength for a strip of deck spanning across the width of a bridge and resisting design wheel loads. Research work and laboratory testing of bridge decks shows that flexural cracking has an impact on the deck behavior, especially under fatigue loading with moving wheel loads, but deck failure actually occurs due to punching shear.

An alternate approach to deck design, using punching shear as the basis, appears to be a much more rational method. Since the strength is controlled by shear, the function of the flexural reinforcement used in bridge decks becomes questionable. Flexural steel reinforcement may be eliminated, but provision still must be provided for crack control under service loads to insure visual serviceability.

A prototype bridge was built as part of this overall design development project and will be monitored for behavior and performance. The new design method was accompanied by a series of details intended to increase construction efficiency, work zone safety, improve deck strength efficiency, and reduce bridge construction costs. These details included removal of steel reinforcement from the deck (except at guardrail attachment), use of polypropylene fiber reinforcement in the concrete for shrinkage crack control, elimination of deck haunches between the girder and deck, adapting the girder flanges to receive stay-in-place forms, use of FRP planks for stay-in-place forms and as secondary crack control reinforcement, placing the girders on a slight tilt so that the girder flanges matched the 2 % deck cross slope, placing girder bearings at the 2% cross slope, eliminating stepped girder bearings from the bridge abutments and using 2 in. diameter galvanized steel ties between the girder webs.

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REFERENCES

1. Bank, L., Oliva, M., Russell, J., Jacobson, D., Conachen, M., Nelson, B., and McMonigal, D., "Double Layer Prefabricated FRP Grids for Rapid Bridge Deck Construction: Case Study," *ASCE J. of Composites in Construction*, V. 10, No. 3, May-June 2006, pp. 204-212.
2. Dieter, D., Dietsche, J., Bank, L., Oliva, M., and Russell, J., "Concrete Bridge Decks Constructed with FRP Stay-in-Place Forms and FRP Grid Reinforcing," Transportation Research Record No. 1814, *Journal of Transportation Research Board*, 2002, Washington D.C., pp. 219-226.
3. Braimah, A., Green, M., and Soudki, K., "Polypropylene FRC Bridge Deck Slabs Transversely Prestressed with CFRP Tendons," *ASCE J. of Composites in Construction*, V. 2, No. 3, November 1998, pp. 149-157.
4. Thorburn, J., and Mufti, A., "Design Recommendations for Externally Restrained Highway Bridge Decks," *J. Bridge Engineering*, V. 6, No. 4, July-Aug 2001, pp. 243-249.
5. American Association of State Highway and Transportation Officials, "AASHTO LRFD Bridge Design Specifications," 4th Ed., AASHTO, Washington D.C., 2007.
6. Petrou, M., Perdikaris, P., and Wang, A., "Fatigue Behavior of Non-Composite Reinforced Concrete Bridge Deck Models," Transportation Research Record No. 1460, *Journal of Transportation Research Board*, 1994, Washington D.C., pp. 73-80.
7. Perdikaris, P., Beim, S., and Bousias, S., "Slab Continuity Effect on Ultimate Fatigue Strength of Reinforced Concrete Bridge Deck Models," *ACI Struc. J.*, V. 86, No. 4, July-Aug 1989, pp. 483-491.
8. Bakht, B., and Lam, C., "Behavior of Transverse Confining System for Steel-Free Deck Slabs," *J. of Bridge engineering*, V. 5, No. 2, May 2000, pp. 139-147.
9. Salem, A., El-Aghoury, M., Sayed-Ahmed, E., and Moustafa, T., "Composite Steel-Free Deck Bridges: Numerical Modeling and Pilot Parametric Study," *Cn. J. Civ. Eng.*, V. 29, 2002, pp. 662-678.
10. Newhook, J., and Mufti, A., "A Reinforcing Steel-Free Concrete Deck Slab for the Salmon River Bridge", *ACI Concrete International*, V. 18, No. 6, June 1996, pp. 30-34.
11. Bakht, B., and Lam, C., closure to "Behavior of Transverse Confining Systems for Steel-Free Deck Slabs", *J. of Bridge Engineering*, V. 7, No. 1, Jan-Feb 2002, pp. 68-70.
12. Mufti, A., Jaeger, L., Bakht, B., and Wegner, L., "Experimental Investigation of Fibre-reinforced Concrete Deck Slabs without Internal Steel Reinforcement," *Cn. J. Civ. Eng.*, V. 20, No. 3, June 1993, pp. 398-406.