Implementation of A New Precast Concrete Deck System to the Kearney East Bypass Project

George Morcous, University of Nebraska, (402) 554-2544, gmorcous2@unl.edu
Fouad Jaber, Nebraska Department of Roads, (402) 479-3967, fouad.jaber@nebraska.gov
Jason Volz, Nebraska Department of Roads, (402) 479-4452, Jason.volz@nebraska.gov

ABSTRACT
This paper presents the first implementation of a new precast concrete deck system to the Kearney East Bypass bridge project in Kearney, NE. The project has twin bridges: southbound bridge constructed using cast-in-place concrete deck; and northbound bridge constructed using the new full-depth full-width precast concrete deck system. Each bridge has two spans that are 166 ft long each. Each span has five precast/prestressed concrete girders (NU1800) at 8 ft 6 in. spacing for a total width of 41 ft 8 in. The successful experience and lessons learned from the fabrication and construction of the superstructure along with post-tensioning and grouting operations are presented.

BACKGROUND
Precast concrete deck systems are designed as either composite or non-composite with the supporting girders. Composite systems are more common due to their structural efficiency, which results in smaller girder sections, longer spans, and economical construction. Existing full-width full-depth composite precast concrete deck systems typically use either continuous open channels along the girder lines or discrete open pockets at approximately 2 ft spacing to accommodate the shear connectors of the supporting girders. These channels or pockets are usually grouted and the deck surface is, then, protected by an overlay similar to cast-in-place (CIP) concrete decks, which increases construction duration and cost. In addition, transverse joints between adjacent precast concrete deck panels are either conventionally reinforced or longitudinally post-tensioned using strands inside embedded ducts. These transverse joints and post-tensioning ducts have to be grouted to protect reinforcement, which complicate the construction and, consequently, reduce the attractiveness of precast concrete deck systems as an accelerated, economical, and more durable alternative to conventional CIP concrete decks.

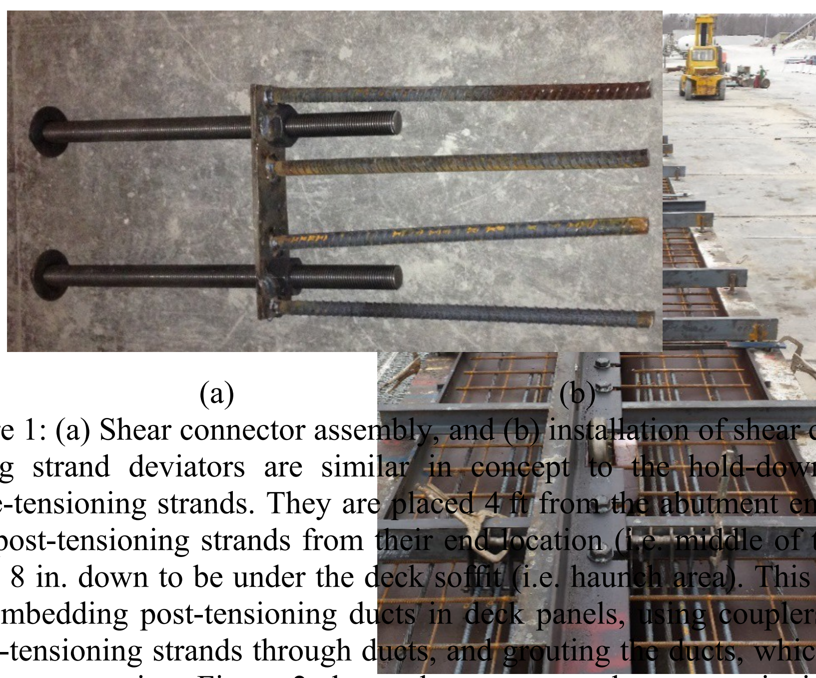
A new precast concrete deck system was recently developed jointly by Nebraska Department of Roads (NDOR) and University of Nebraska-Lincoln (UNL) to address the shortfalls of the traditional systems. The main features of the new system are: using full-width, full-depth, and 12 ft long precast deck panels to reduce the number of panels, transverse joints, and cast-in-place operations; increasing the spacing between shear connectors to 4 ft, which is the maximum spacing currently allowed by AASHTO LRFD, to simplify girder and panel fabrications; using covered pockets in the panels to limit penetrations of the deck surface, which eliminate the need for an overlay and enhance deck durability; eliminating reinforcement across transverse joints and placing longitudinal post-tensioning strands in the haunch area under deck panels and over girder lines to simplify post-tensioning operations and eliminate the need for threading strands and grouting ducts. Several analytical and experimental investigations were conducted at UNL to evaluate the structural performance and constructability of the new system prior to its implementation. For more information about these investigations, refer to Morcous, et al. (2013) and Morcous, et al. (2015).

PROJECT DESCRIPTION

The Kearney East Bypass project is located in Kearney, NE over the US-30 and Union Pacific railroad tracks. The project consists of twin two-span bridges: southbound bridge with CIP concrete deck; and northbound bridge with precast concrete deck. Construction of the superstructure (girders, deck, and rail) of the two bridges was done concurrently starting in May 2015 and was completed in September of 2015. Each span is 166 ft long and consists of five NU1800 precast/prestressed concrete girders with 8 ft 6 in. spacing. Bridge width is 41 ft 8 in. and has a 14º skew. Both CIP and precast concrete decks are 8 in. thick and have closed CIP concrete rail. The following sections summarize the fabrication of precast concrete girders and deck panels of the northbound bridge conducted by Coreslab Structures Inc., Plattmouth, NE, and the erection conducted by Hawkins Construction, Inc., Omaha, NE.

Girder Production

The two differences between the production of precast/prestressed concrete girders for CIP deck and for precast concrete deck are: 1) shear connectors; and 2) post-tensioning strand deviators. Shear connectors for the precast concrete deck consist of two 1.25 in. diameter A193 Grade B7 coil rods spaced at 4 ft along the girder to resist a horizontal shear flow of 3.2 kip/in. Each connector is made in an assembly of two rods, four nuts and washers, one plate, and 4-#6 bars as shown in Figure 1(a). Connectors are placed in the girder web and attached to pre-slotted steel angles, shown in Figure 1(b), to ensure that spacing is within the specified tolerance (± ¼ in.). The portion of each rod embedded in the girder is greased to allow cranking it up later to adjust the connector height as he girder camber varies. According to the precast producer, making, greasing, and placing shear connector assemblies with the specified tolerance required almost double the time and labor required for placing shear reinforcement in traditional girder production.



(a) (b)

Figure 1: (a) Shear connector assembly, and (b) installation of shear connectors

Post-tensioning strand deviators are similar in concept to the hold-down devices used in depressing pre-tensioning strands. They are placed 4 ft from the abutment end of each girder to depress deck post-tensioning strands from their end location (i.e. middle of the deck thickness) approximately 8 in. down to be under the deck soffit (i.e. haunch area). This concept eliminates the need for embedding post-tensioning ducts in deck panels, using couplers to connect ducts, threading post-tensioning strands through ducts, and grouting the ducts, which are cumbersome, costly, and time consuming. Figure 2 shows the custom-made post-tensioning strand deviators for 12-0.6 in. diameter post-tensioning strands. These deviators are anchored to the top flange of the girder using 10-#5 straight bars and to the web using 2-#5 U bars to resist an uplift force of approximately 90 kip (500 kip x tan 10º). In addition, the girder top flange is thickened by 1 in. at the location of the deviators to increase its anchoring capacity. Figure 3 shows the shear connectors, post-tensioning strand deviators, and lifting inserts in five NU1800 girders at the precast yard. All girders had 60-0.6 in. diameter strands, a specified 28-day compressive strength of 10,000 psi, and release strength of 7,200 psi.

Figure 2: Post-tensioning strand deviators

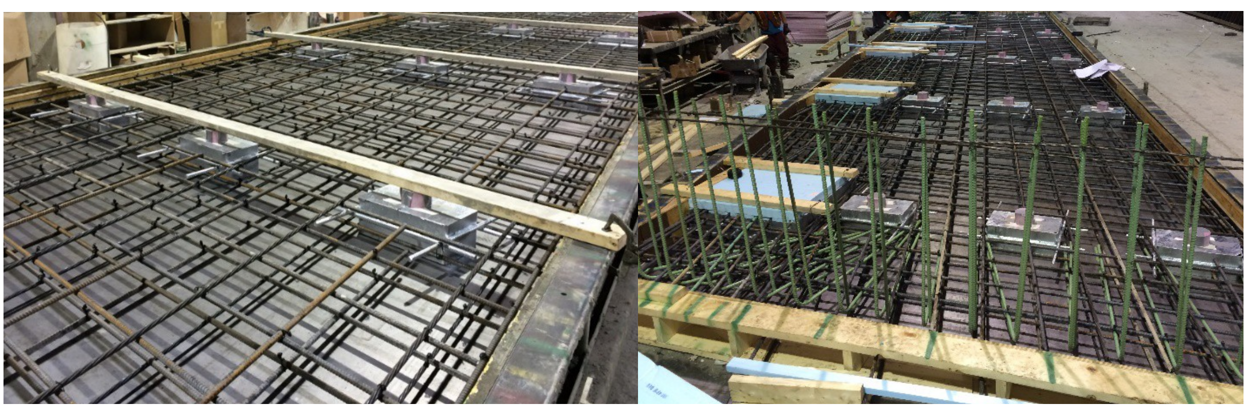
Figure 3: Precast/prestressed concrete NU1800 girders for the new precast concrete deck system

Deck Production

The 12 ft long, 41 ft 8 in. wide, and 8 in. thick precast concrete deck panels were produced using the same prestressing bed used for precast concrete sandwich wall panel production. A total of 26 typical panels were produced at a rate of four panels at a time, in additional to two special end panels that have anchorage blocks. Each panel has a total of 15 pockets (3 pockets for each girder line at 4 ft spacing) for shear connectors. Each pocket consists of 5.5 in. tall hollow structural section (HSS) 16x8x¼ with 4-#5 bars welded to each side and a 1/16 in. thick top plate that has 4 in. diameter hole for grouting. Custom-made lifting inserts are added to some of the pockets, as shown in Figure 4(a), to be used in panel handling and eliminate the need for additional penetrations to the panel surface. All pockets are galvanized and positioned precisely by gluing wooden sheets to the bed as shown in Figure 4(b). All panels have 14^{o} skew angle and are pre-tensioning transversely using 6-0.6 in. diameter strands at the level of the top reinforcement and 6-0.5 in. diameter strands at the level of bottom reinforcement. Styrofoam is used to form the grouting holes and anchorage blocks, while plywood is used to form the shear keys along the panel edges. Figures 5(a) and 5(b) show the forming and reinforcement of a typical panel and end panel respectively.

(a) (b)

Figure 4: (a) Shear pocket detail, and (b) wood forms used to position the pockets in the bed



(a) (b)

Figure 5: Forming and reinforcement of (a) typical panel, and (b) end panel

All panels are made using the standard NDOR bridge deck concrete mixture but with a specified 28-day compressive strength of 6,000 psi and release strength of 3,500 psi. The use of cement type III and/or accelerators was not allowed for exposed surfaces unless extensive durability testing is conducted, which resulted in two-day cycle of panel production. Figure 6(a) shows the completed panels with 4 in. diameter holes for grouting and panel lifting, and the projecting rail reinforcement, while Figure 6(b) shows an end panel with the galvanized anchorage blocks and the supporting rockers used for shipping the panels. The precast producer built six sets of these rockers to help distribute the load during transportation and avoid cracking. These rockers were reused as the panels were shipped at a rate of 6 panels per day. All the shear keys were sandblasted at the precast yard to enhance the bond with the transverse joint cast-in-place concrete. According to the precast producer, the custom-made lifting inserts and anchorage blocks were easy to make and install, and deck panel production was not more difficult than wall panel production.

(a)

(b)

Figure 6: Completed (a) typical, and (b) end panels at the precast yard before shipping

CONSTRUCTION

Girders of the precast concrete deck bridge were shipped and erected similar to those of the CIP concrete deck bridge. According to the contractor, they paid slightly more attention to positioning girders in their exact locations, however, it did not take more time than traditional construction. After placing end and intermediate diaphragms, galvanized steel bent plates were welded to the metal tabs embedded in the girder top flange according to girder profile data obtained from shim shots and specified deck profile and haunch thickness. This deck support system also works as side forms for grouting the haunch area later. Therefore, a compressible sealing strip was attached to the top of the bent plates and caulking was used at the bottom of the bent plates, as shown in Figure 7, to prevent grout leakage. Figure 7 also shows the 12-0.6 in. diameter post-tensioning strands laid over the top flange for each girder line and through the end deviators, which greatly simplified the post-tensioning process according to the contractor and kept the strands protected in the haunch area away from the deck surface. Figure 8 shows the 10-#8 Grade 60 bars installed over the girder top flange (i.e. in the haunch area) over the intermediate pier to provide live load continuity and resist negative moment. Figure 9 shows the greased rods of the shear connectors cranked up to the calculated elevation that achieves the specified embedment of each rod into the shear pocket (5±1/4 in.). Only 9 rods out of total of 900 rods could not be cranked up and had to be cut and spliced using couplers to achieve the specified embedment as shown in Figure 9.

Figure 7: Deck support system and post-tensioning strands.

Figure 8: Live load continuity reinforcement over intermediate support.

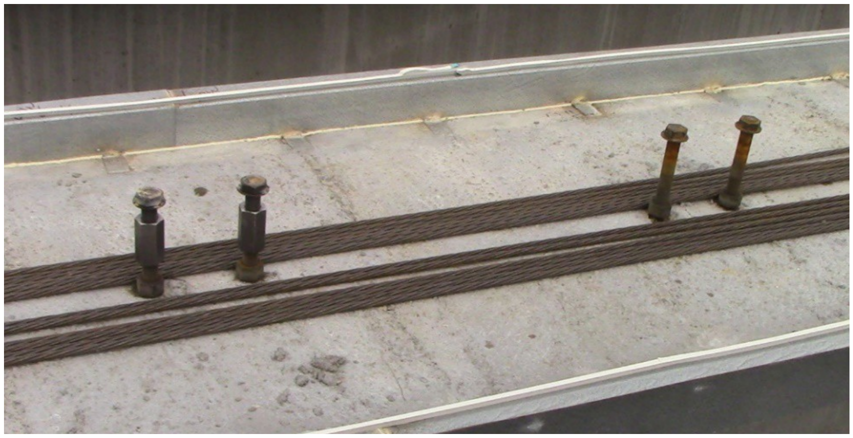


Figure 9: Adjusting the elevation of the shear connectors and using couplers if needed.

The contractor made a digital "template" of the shear stud location prior to shipping the panels to ensure there were no problems with the tolerances. Panel erection was done using a single crane and the 8 lifting inserts embedded in the pockets as shown in Figure 10(a). The installation of the first panel occurred at the pier location as shown in Figure 10(b) and remaining panels were then installed in both directions until end panels were installed at the abutment locations as shown in Figure 10(c). Installation duration was approximately 30 minutes per panel and was limited to the six panels delivered in one day (total duration was 5 days). It should be noted that the custom made lifting inserts performed satisfactorily and no single shear connector interfered with the pocket (i.e. production tolerance was adequate).

(a)

(b)

(c)

Figure 10: Erection of precast concrete deck panels

Figure 11(a) shows one of the transverse joints between adjacent deck panels (2 – 3 in. wide) with ¾ in. backer rod along the entire width of the bridge to close the bottom of the joint. Figure 11(b) shows the placing of 6 in. slump conventionally vibrated concrete at the joint using a bucket and shovel. The concrete mix was the same one used in panel production as specified by NDOR to ensure consistent performance. This operation took approximately 7 yd^3 and all joints were completed in approximately 3 hours. Transverse joints were covered immediately with curing compound then later with wet burlap. Once the joint concrete achieved at least 3,500 psi compressive strength, post-tensioning operations were conducted using a mono-strand jack from one end as shown in Figure 12 to apply 500 psi stress across the deck section. All the 60 strands (12 strands per girder line) were tensioned initially to 2 kips, then tensioned finally to 43.9 kips (0.75 f_{pu}) in a symmetrically alternating manner to minimize eccentricity effects on the deck. Measured elongations of all strands were with ±5% of the predicted elongation (25.5 in.) and the total elastic shortening of the deck was 0.5 in. (as predicted)

(a) (b)

Figure 11: Placing concrete for transverse joints.

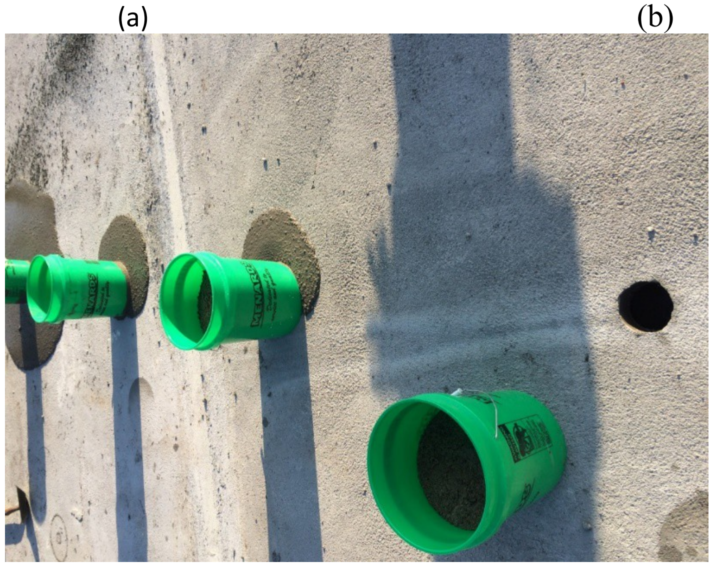
Figure 12: Post-tensioning strands and anchor block.

The grouting of the haunch, shear pockets, and anchor blocks were conducting using highly flowable and high strength self-consolidating concrete (SCC). The mix had a nominal maximum size of 3/8 in. limestone aggregate, specified strength of 8,000 psi, and slump flow of 28 in. Measured slump flow ranged from 27 – 30 in. as shown in Figure 13(a), and measured strength averaged 7,400 psi after 4 days and 8,900 psi after 28 days. SCC was pumped using 3 in. diameter hose from one location (at the bridge pier) to the 4 in. diameter grouting holes on the deck surface using the apparatus shown in Figure 13(b) (hopper, bucket, and inverted cone). SCC was placed from one grouting hole and flowed in both directions until it overflowed from the adjacent holes, which were covered with sand-filled buckets before moving to the next grouting hole as shown in Figure 13(c). This process ensured complete filling of haunch and pockets and took 1 hr. per girder.

The ends of each girder line were formed and filled with SCC so that the anchorage blocks and post-tensioning chucks are fully embedded in concrete.

(a) (b)



(c)

Figure 13: Placing SCC in the pockets and between the girders and deck

Figure 14(a) shows the deck top surface after grouting and curing, while Figure 14(b) shows the final deck surface after placing the rail and grinding up to ½ in. The grinding operation took 1 day similar to the texturing operation done on the CIP deck. The final deck surface will not have an overlay and will be monitored for any signs of deterioration in the next few years. It should be noted that a few transverse cracks were observed at the transverse joints before deck post-tensioning. These cracks were closed after post-tensioning was applied and the final deck surface does not have any cracking. Also, there is no clear discoloration in the final deck surface between precast concrete and CIP concrete used in grouting the transverse joints and shear pockets. Figure 15 shows the bottom side of the precast concrete deck system and the side of the haunch area after removal of the exterior deck support system to check the consolidation of SCC.

(a) (b)

Figure 14: Final deck surface: before grinding (a), and after grinding (b)

(a) (b)

Figure 15: Bottom of precast concrete deck: between girders (a), and at the overhang (b)

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