SECOND GENERATION NUDECK PRECAST BRIDGE DECK SYSTEM

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ABSTRACT

The NUDECK full-depth precast bridge deck system is a rapid construction method that results in a high quality composite girder/deck structure. The first generation NUDECK panel research resulted in the construction of SKYLINE Bridge in Omaha, Nebraska. The full depth precast panels were pretensioned transversely and post-tensioned in the direction of traffic flow. There has been a joint effort by the Nebraska Department of Roads and the University of Nebraska to improve and optimize the system.

These efforts resulted in a more efficient design and detailing of the full-depth precast deck system. In this system, the overlay will be replaced by grinding the riding surface. New efficient detailing for the post-tensioning was developed, as well as NUDECK precast rail details. This system is scheduled to be implemented in the 176th Street over I-80 Bridge near Lincoln, Nebraska. This paper summarizes the design and the detailing of the second generation NUDECK precast bridge deck system on this bridge.

Keywords: Precast, Deck, Panel, Full-depth, Bridge, Concrete, Prestressed, Post-tension

PROJECT INTRODUCTION

Due to the projected continuation of traffic flow increase between Lincoln and Omaha, Nebraska, the Nebraska Department of Roads (NDOR) is widening Interstate 80 between the two cities. The interstate will be widened from two lanes to three lanes in each direction. The existing 176th Street bridge carrying traffic over Interstate 80 must be lengthened to accommodate the increase in roadway width beneath it.

The bridge is located east of Lincoln and serves as an overpass for a county road. The new bridge has twin spans of 126 feet- 7 inches to yield a total of 253 feet- 2 inches. The structure is 38 feet- 8 inches wide and has a skew of 9 degrees. The deck is supported by four NU 1350 concrete girders with 10 feet center to center spacing. The girder design requires 40 – 0.6 inch prestressing strands and utilizes innovative negative moment continuity for deck self weight and superimposed dead loads. Prior to the placement of the deck, 50 feet long 150 ksi high strength threaded rods are centered on top of the girders above the pier in the negative moment region. Concrete is cast around the threaded rods to create mechanical interlocking for the transfer of shear and negative moment forces between the girders. The ends of the girders in this section have unique c-shape reinforcement to allow the placement of the threaded rods and confine the concrete.

The bridge deck system utilizes the NUDECK Second Generation full depth precast panels. These panels are 12 feet in length, 8 inches in depth, and are the full width of the bridge deck– 38 feet- 8 inches. A major goal of the second generation system is to avoid an overlay. This is accomplished by casting the panels one quarter inch over the required depth of 7.75 inches and designing the post-tensioning to occur beneath the deck panels. The panels will be surface ground up to one quarter of an inch to provide a smooth and uniform driving surface. The precast panels have a skew of 9 degrees to avoid special end panel design.

The second generation deck panels no longer have an open channel at the girder lines for post-tensioning. The post-tensioning still occurs at the girder lines, but the strands are placed beneath the panels prior to panel installation. The precast bridge deck panels are not pretensioned transversely, but crack control is achieved through the use of welded wire reinforcement (WWR). An easily replaceable precast safety barrier is implemented in the NUDECK system. Composite action is achieved through high strength threaded rod and shear pockets spaced at 48 inches on center. The circular shear pockets are five inches high with three inches of concrete covering and are grouted after post-tensioning. A simplified view of the deck panel is shown in Figure 1.

DECK PANEL DESIGN AND DETAILS

The design procedure and the modified standard details used for the NUDECK Second Generation full-depth precast deck panel are presented in this section. The NUDECK panel is designed based on LRFD Bridge Design Specifications.[1]

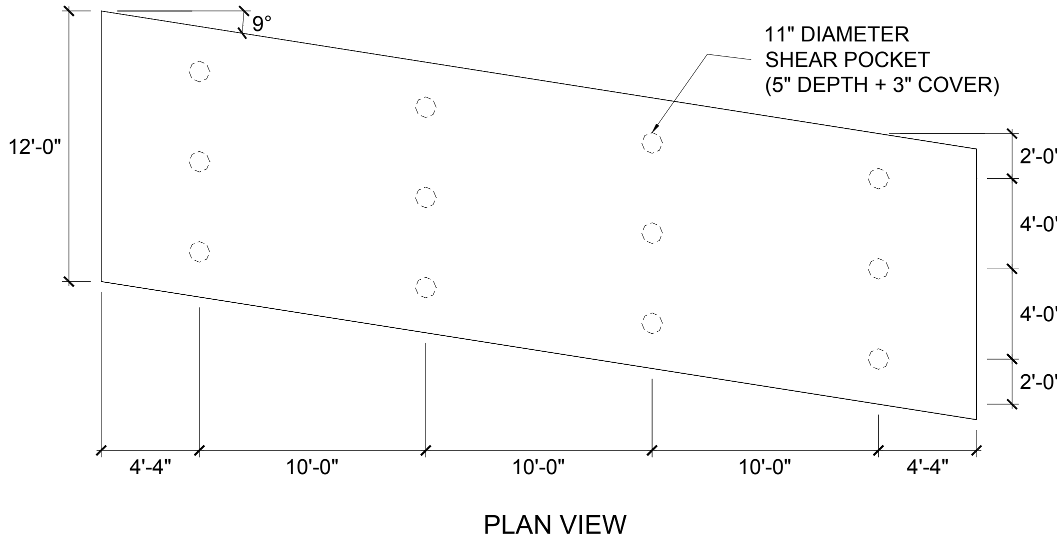

Figure 1 176th and I-80 Bridge typical deck panel.

Positive and Negative Moment Sections

Although the 176th Street over I-80 Bridge utilizes a precast deck system, the NUDECK is designed using the Empirical Design method. The NUDECK meets all of the requirements for using Empirical Design except for being a fully cast-in-place (CIP) deck. Even though the NUDECK panels are not CIP, the deck is made fully composite with the supporting girders (see the section titled "Composite Action – Horizontal Shear"). The deck is uniform in depth except for haunches at the girder flanges. The design depth of the slab is 7.25 inches and is greater than the required minimum 7.0 inches. The total depth of the slab is 8.0 inches (7.25 + ½ inch sacrificial wearing surface + ¼ inch grinding surface). The specified 28-day strength is 8.0 ksi which is double the 4.0 ksi minimum. The effective length of the deck using NU I-girders spaced at 10 feet is 7.75 feet which is less than the 13.5 feet maximum. Also, the ratio of effective length to design depth is 12.8 and satisfies the lower and upper boundaries of 6.0 and 18.0. Research will be conducted at the University of Nebraska to verify that the Empirical Design method can be applied to precast composite deck systems.

The reinforcement is located as close to the outside surfaces as permitted by cover requirements. For the NUDECK, the bottom clear cover requirement is 1.0 inch, and the top clear cover requirement is 2.0 inches below the design depth. The minimum amount of Grade 60 ksi steel transverse reinforcement for the bottom and top layers is 0.27 in^2/ft and 0.18 in^2/ft, respectively. All reinforcing steel in the NUDECK is Grade 60 ksi or higher, and only lap splices are used. For the main reinforcement, the NUDECK design uses a Grade 80 ksi welded wire reinforcement that has a double layer of corrosion protection.

The Empirical Design method recognizes that the primary structural action by which concrete deck slabs resist concentrated wheel loads is not pure flexure but an internal membrane stress state referred to as internal arching. The arching action is created when the concrete cracks in the positive moment region and the neutral axis shifts upward. In-plane membrane forces sustain the arching action and develop due to confinement provided by the surrounding concrete slab and the supporting components acting compositely with the slab. The minimum transverse reinforcement is more than adequate to resist the remaining component of flexural moment resulting from the live loads.

The purpose of longitudinal reinforcement in the bridge deck is to distribute the live load and control cracking. The longitudinal reinforcement provided by the WWR effectively controls transverse cracking because a greater number of smaller diameter reinforcement bars spaced closely together limits the maximum crack width[2]. In addition, cracks are further controlled by mechanical interlocking. The interlocking occurs because the WWR mesh has cross-wires welded with minimum shear strength of 35,000 psi at two and three inch spacing.

Conventionally tied bar reinforcement has lower yield strength, lower crack control properties and requires more time to fabricate compared to using WWR. WWR has been investigated in bridge decks as part of the “Rapid Replacement of Bridge Decks” Project for the National Cooperative Highway Research Program (NCHRP) Project 12-41[3]. It was found to satisfy all AASHTO design criteria. However, it was too expensive to use as a replacement of epoxy coated bars.

The WWR proposed for use in the NUDECK is unlike any used before. The WWR is an 80 ksi steel that is first welded, then galvanized, and finally coated with polyvinyl chloride (PVC) by fusion bonding. All this processing is done at one plant, without the expense of transportation and mark-up for corrosion protection and for sale to precasters or contractors only through previously certified distributors. Thus, this WWR has double corrosion protection and is superior to other WWR and to ordinary tied reinforcement, at a comparable cost. This WWR is expected to reduce labor costs for fabricating the panels while providing a corrosion free, low maintenance, and long life reinforcement. At this time, there is only one source of this product. However, due to the potential size of the market, other WWR producers are expected to rapidly compete for this business.

The largest WWR produced by the manufacturer is W2.9. This corresponds to a diameter of 0.192” +/- 0.004” not including the PVC corrosion coating thickness. The WWR will be produced 7 feet wide and have a length equal to the width of the deck panel minus clear cover spacing or rail reinforcement spacing. The wires perpendicular to traffic flow will be parallel to the deck transverse edge to match the 9 degrees skew. Two WWR sheets will be used for both the top and bottom layers creating two feet of overlap in the center. The minimum reinforcement requirement is met using Grade 80 ksi 3 x 3 – W2.9 x W2.9 in the top layer and Grade 80 ksi 2 x 3 – W2.9 x W2.9 in the bottom layer. The 2 inch spacing is within the 7 feet spacing which creates $(7*12)/2 = 42$ spaces. At each shear pocket, two number 4 bars are used to replace the cut bottom layer of WWR. Figures 2 and 3 are the plan and elevation views of a typical panel’s reinforcement.

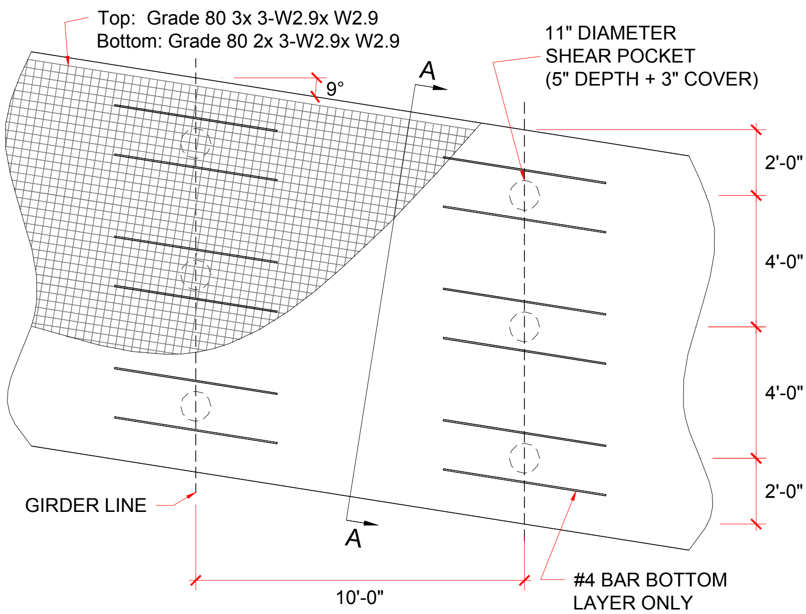

Figure 2 Plan view of the panel reinforcement.

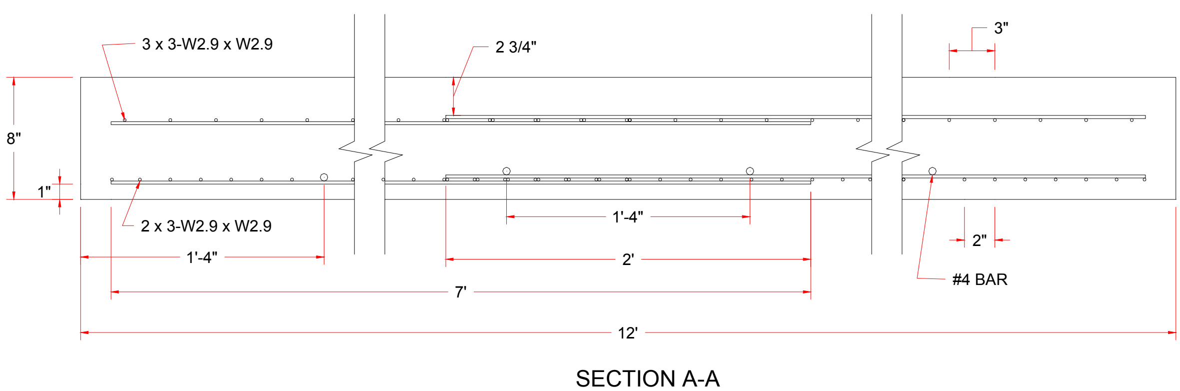

Figure 3 Elevation end view of the panel reinforcement.

Transverse Pretensioning and Panel Crown

Transversely pretensioned precast panels are beneficial for longitudinal crack control and strength design, but this pretensioning presents problems. If the panel was transversely pretentioned, creating a crown would be a more challenging task. The prestressing strands would have to be harped, or the deck panel would need to have an open longitudinal channel to rotate in after prestressing straight strands. Harping strands to match a crown is more difficult and time consuming for the precaster. The Skyline Bridge proved that an open longitudinal channel is an excellent way to create a crown, but the channel must be grouted. An overlay would be required if grouting the open channel interfered visually with driving lane lines for motorists. Another option would be to have a variable thickness precast panel with a flat bottom and with straight prestressing. Traditional methods of pretensioning would be appropriate, but this panel design would increase the deck weight and cost and would also require higher prestressing forces to account for the greater concrete area.

If the deck panels are not transversely pretensioned, a two percent crown can be formed and cast at the precast plant or at the job site. The conventional reinforcement will be able to take the same form as the crown. The production process is simplified by not using transverse pretensioning, but the potential for longitudinal cracking in the deck increases under service conditions. For this reason, the deck reinforcement should utilize welded wire reinforcement (WWR), and the reinforcement should be corrosion protected using galvanizing, epoxy coating, PVC coating or a combination of the methods.

Even without transverse pretensioning, the full-depth precast panels are less likely than a CIP deck to crack immediately following construction[4]. The majority of creep, shrinkage, and temperature change due to hydration in the precast deck panel occur prior to the panel being restricted from movement through composite action to the much stiffer girders. Since precast deck panels are subjected to much less differential creep and shrinkage and temperature gradients, the cracking potential is reduced.

In order to reduce the panel cost and simplify the crowning details, the owner has opted to not transversely pretension the precast deck panels. One process in the panel production is eliminated, and this method should induce competitive precasting practices. A typical bridge cross-section with the crown formed in the panel at a constant deck thickness is shown in Figure 4.

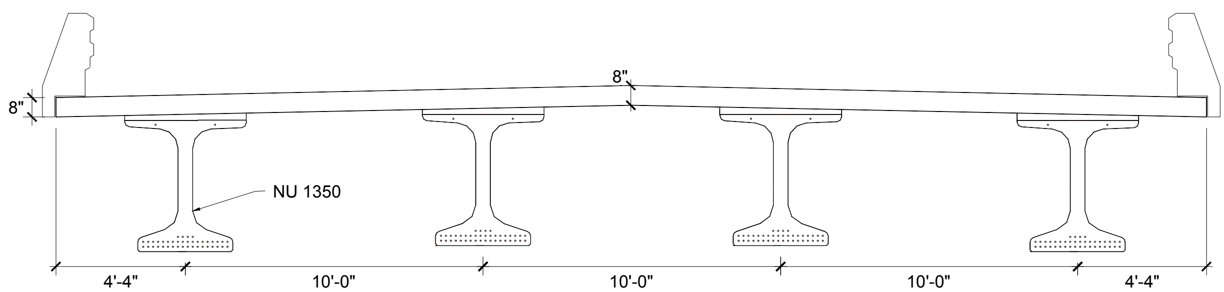

Figure 4 Typical positive moment cross- section of the 176th over I-80 Bridge.

Post-Tensioning

The NUDECK utilizes an innovative post-tensioning system. The post-tensioning strands lie beneath the deck panels on top of each girder line rather than inside a duct or threaded through the deck reinforcement in an open longitudinal channel. This detail allows the contractor to easily and rapidly install the post-tensioning strands. A major benefit of this detail is that an overlay is not required because the precast panels do not have longitudinal channels at the girder lines for post-tensioning. The only exposed grout visible to motorists is the small transverse joints every 12 feet which will not be confused as lane lines. Lastly, this detail is better for maintenance. The precast panel system is expected to last as long as the superstructure, but if a localized damaging event occurs to the deck, individual deck panels could be removed and replaced without cutting the post-tensioning.

The full-depth precast panels are post-tensioned in the longitudinal direction to reduce transverse joint separation, transverse cracking, and for negative moment reinforcement at the pier. The post-tensioning adds compressive force in the panel and across the transverse joint. Long term durability is enhanced by reducing transverse cracking potential and increasing the water tightness of the joints. The post-tensioning is applied after the transverse shear keys are grouted and attain adequate strength but before composite action is created through grouting the shear pockets. Thus, the post-tensioning force is completely applied to the deck rather than sharing the force with much stiffer girders. Through analysis under Service III conditions, 10 – 0.6 inch strands per girder line prestressed to 70 percent of the ultimate strength at transfer are required. The post-tensioning force yields a maximum tensile stress of 260 psi in the negative moment region directly above the center pier when the deck system is under all dead and live loads. The deck experiences compressive forces greater than 250 psi in all other regions besides the center pier. The maximum tensile stress is less than half the cracking limit of $0.19\sqrt{f'c}$ and can be reduced to zero by increasing the number of 0.6 inch post-tensioning strands per girder line to 16.

Anchorage Zone Detail

The anchorage zone for the post-tensioning is a critical detail to accommodate rapid construction and allow the system to effectively transfer the prestressing force to the deck without any harmful effects. In order to ensure the prestressing force is applied to the centroid of the deck, the strands are brought up to the mid-height of the deck in the last panel and inserted into a steel anchorage box at each girder line. The last four feet of both end panels have continuous open channels at each girder line 4 ½ inches deep, 22 ½ inches wide, and 48 inches long. These channels allow the prestress strands to freely angle from the bottom of the deck panel 4 inches to its mid-height. A ¾ inch thick steel box with shear studs welded to its sides is embedded into the deck to transfer the post-tensioning force to the deck. Figure 5 through Figure 8 are schematics of this detail.

The prestressing strands are placed on the girders prior to panel installment, and a minimum of 4 feet of extra strand is left at each end. Each end panel is installed without the 48 inch long, one inch thick removable end plate. The end plate has 10 - one inch diameter holes drilled 4 inches from the bottom and 2.25 inches on center (see Figure 8). The strands are easily threaded through a removable end plate beside the deck panel and are raised vertically through the notched ¾ inch thick end plate. The removable plate is then positioned to align the strands at 4 inches from the bottom of the panel. In order to provide a perpendicular jacking surface, the chucks are aligned parallel to the strands by fabricating the notched end plate at a 4.8 degree slope.

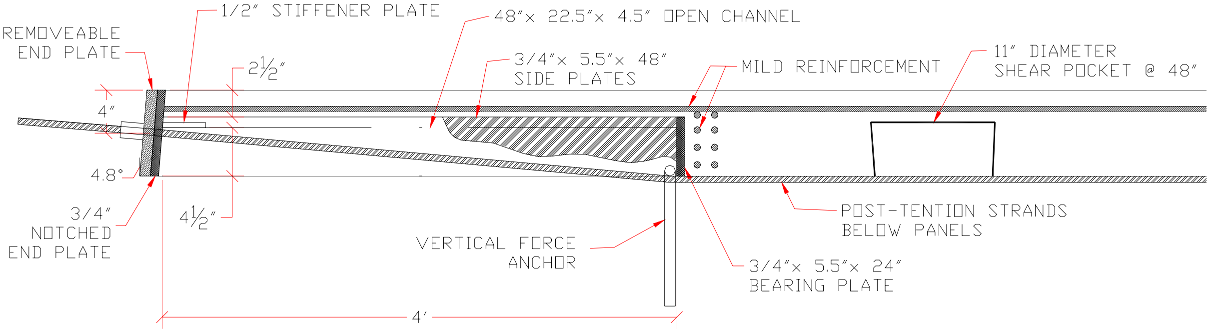

Figure 5 Elevation view of the longitudinal post-tensioning strands under the panel.

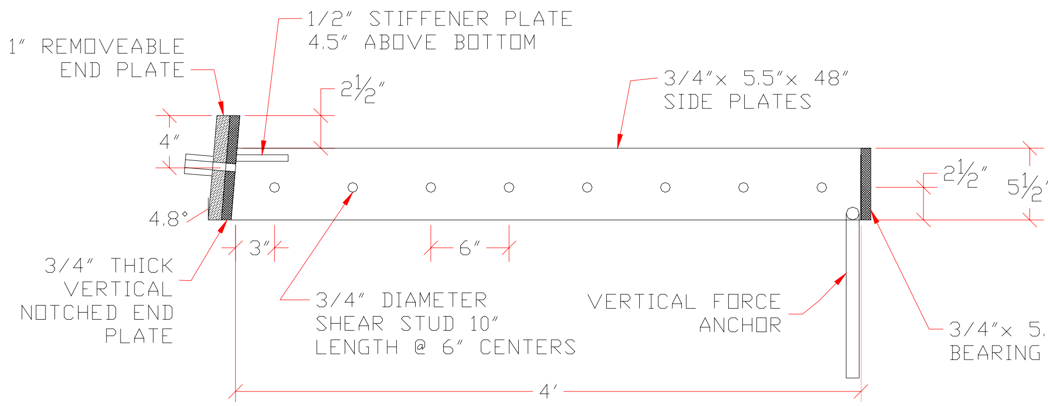

Figure 6 Side elevation view of the post-tensioning anchorage box.

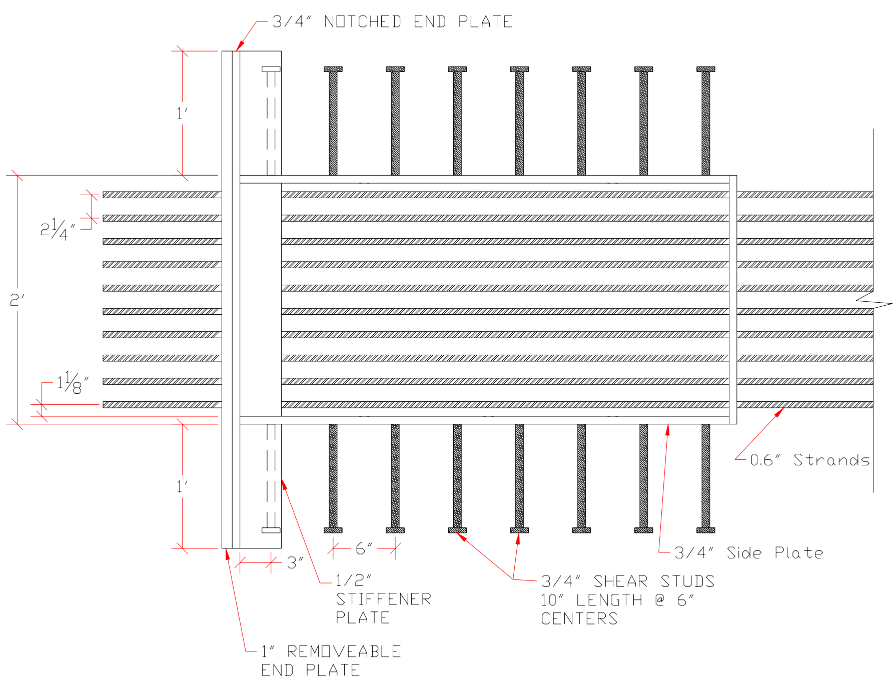

Figure 7 Plan view of the post-tensioning anchorage box.

The post-tensioning force is resisted by shear studs and bearing on the concrete through the steel box. Since high bearing stresses result in cracking, the post-tensioning force is largely transferred to the deck through the shear studs. A total of 16 - ¾ inch diameter shear studs (8 per side) are welded horizontally 2 ½ inches from the bottom of the panel. The shear studs are slightly above the bottom deck reinforcement and have 6 inches spacing. Either welded wire reinforcement or number 4 rebar is used to control the bursting cracks around the steel box. The bottom layer of reinforcement must be cut at the open channels to enable the prestressing strands to freely angle to the mid-height of the deck without threading the strands through reinforcement. The reinforcement tensile capacity is transferred through the steel box by overlapping the shear studs and the bottom deck reinforcement. This detail allows the contractor to easily and rapidly install the post-tensioning strands.

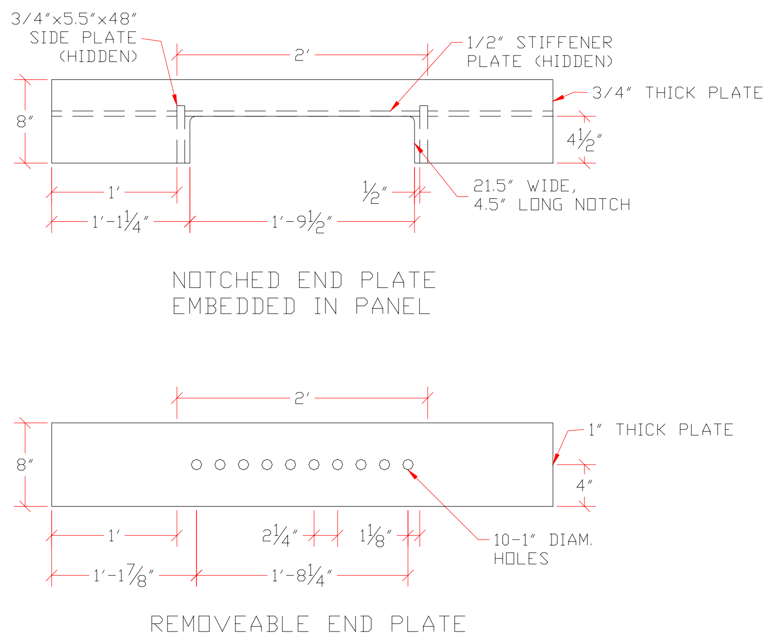

Figure 8 End view of the plates for the post-tensioning anchorage box.

Since the panels are not connected to the girders prior to post-tensioning, the moment created in the deck panels due to the eccentricity of the strands is a potential problem. The deck panel post-tensioning was modeled with an iterative process in RISA-3D using equivalent forces. The strands are assumed to be prestressed to 70 percent of the ultimate strength at transfer. This equates to approximately 410 kips per girder line when 10- 0.6 inch strands are used. Equivalent vertical forces are placed at the location of the assumed draping point. A deck panel width of 10 feet is assumed to apply to one girder line to evaluate a 1 kip per linear foot uniformly distributed self weight load. The deck is assumed to be a continuous member because the transverse shear keys are grouted and cured prior to the time of post-tensioning.

To begin with, vertical restraints were placed at 1 foot intervals along the entire span of one girder line. The vertical reactions due to post-tensioning alone and due to self weight alone were determined, and the difference between them was calculated. If any reaction was a tensile force, the vertical reaction was removed, and the model was reanalyzed. This iterative process was repeated until no tensile forces occurred. At this point, the deflection of the deck was determined. Using an 8 feet draping point in the model, the deck panel deflected upward a maximum of 0.13 inch directly above the draping point.

Since the post-tensioning moment in the deck panels creates a small camber in the model, an anchorage device is being devised to relieve the deck panel of the vertical force and direct it into the girder. A preliminary thought is to create a roller out of a pipe placed over a 1 1/8" rod, and bend the rod into a trapezoid (see Figure 9). The bent rod is embedded eight inches into the girder. The draping point was reduced to 4 feet from 8 feet since the vertical force is removed from the deck panel and detailing becomes easier. As shown in Figures 5 and 9, the vertical anchorage device is just less than 4 feet from the end of the deck and is contained within the open channel in the deck. The vertical anchor has a roller with a bottom elevation equivalent to the bottom of the deck. The prestressing strands pass beneath the roller and are draped to the same elevation as the bottom of the deck. After post-tensioning occurs, the open channel is grouted with the haunch, so the strands will be protected against corrosion.

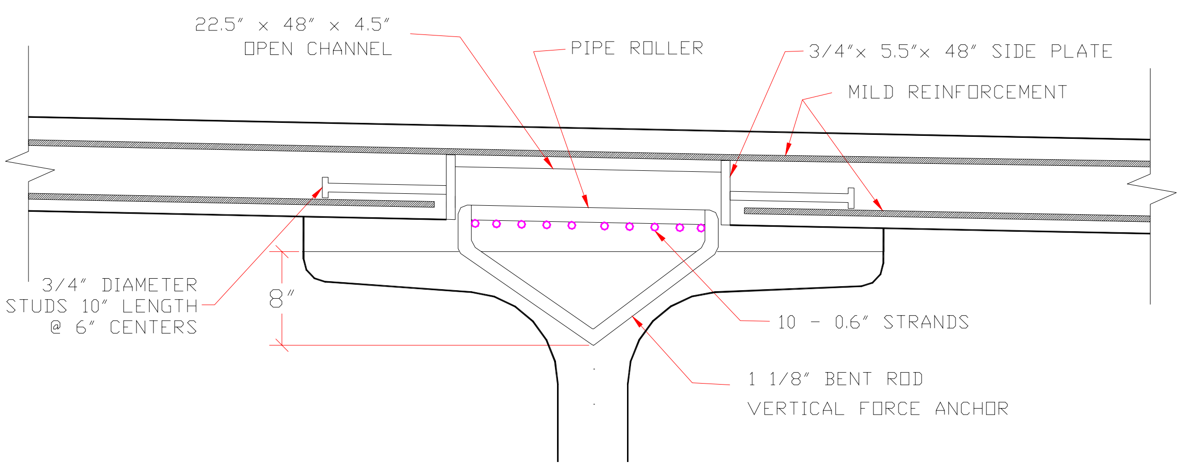

Figure 9 Cross section view of the vertical force anchor.

Composite Action – Horizontal Shear

After post-tensioning occurs, the deck panels are made composite with the girders through a simple and efficient detail. Horizontal shear reinforcement in precast girders designed for CIP decks typically extends the vertical shear reinforcement past the top flange of the girder and bends the reinforcement parallel to the flange. This method is impractical for precast deck panels because the horizontal shear reinforcement is not clustered to fit in the deck panel shear pockets.

As a general rule, the fewer number of shear pockets that a panel has results in cost reduction for forming the panel. The panels are also easier to install because less hardware has to be aligned. The Standard and LRFD Specifications do not provide requirements for the maximum spacing of stud clusters. Based on recommendations given by the AASHTO specifications for shear connectors used with CIP deck slabs, the common practice is to limit the spacing to 24 inches[5].

In order to minimize the number and size of shear pockets required in the deck panels, high strength threaded rod is used for the shear connectors. Two 1 3/8 inch diameter, 150 ksi threaded rods[6] are placed in 11 inch diameter shear pockets 5 inches deep. The threaded rods and shear pockets are spaced at 48 inches on center along the girder line. In the pocket, the threaded rods are spaced 6 inches apart parallel to the girder line to satisfy the AASHTO specifications that the minimum stud spacing is 4 times the stud diameter.

The shear pockets do not extend to the top of the panel, so they will not be visible to motorists. A loose nut is placed near the bottom end of a greased threaded rod, and the hardware is situated in the girder prior to casting. Once the girder is in its final position and the camber and deflection have been determined, the threaded rod can be unscrewed or cut off to adjust to the required elevation of the shear pocket. A 2 ¾ inch nut is then placed on top of the threaded rod at its final elevation to provide interlocking and prevent uplift. The shear pockets are grouted with the haunch after the panels are post-tensioned. This detail is illustrated in Figure 10.

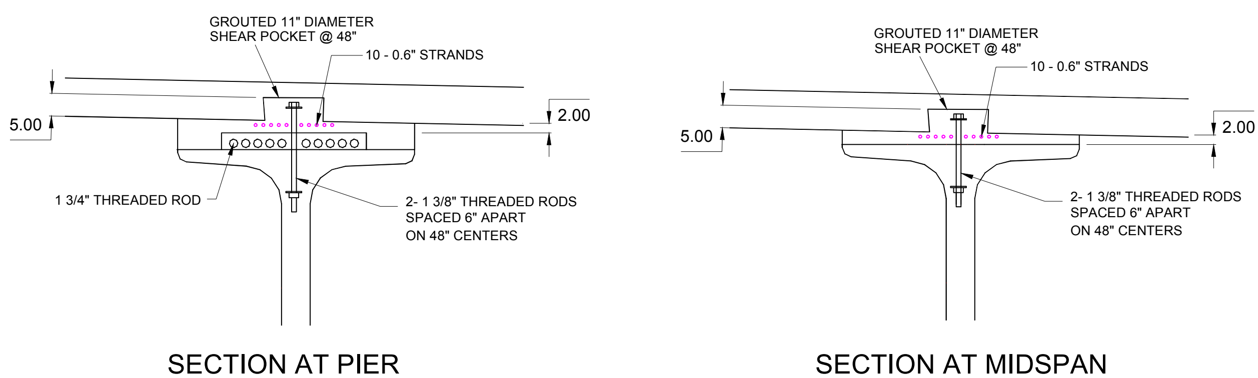

Figure 10 Horizontal shear provided by high strength threaded rod.

PRECAST BARRIER DESIGN

Another very important goal of the second generation NUDECK was to design and implement a precast barrier to speed up construction and to ease repair if damage occurs. Forming, pouring, and curing a CIP concrete barrier is time consuming, and a replaceable precast barrier can eliminate this obstacle. In case of a destructive event, the damaged section of the precast barrier can be removed and replaced with a new section with relative ease. This section presents two preliminary designs for the precast barrier system.

Original Precast Barrier System

The original precast barrier design is based on TL-4 loading criteria for a bridge with open rail drainage. The precast safety barrier consists of one individual 4 feet wide post centered on each 12 feet long panel with rails that span either the distance between two or three posts. The post and rails are connected together using four 1 ¼ inch, 150 ksi threaded rods. These four rods are inserted through vertical ducts in the rail, post, and deck and provide the flexural moment and shear force capacities. Figure 11 is a cross sectional view of the post and rail reinforcement. In this system, the ducts are grouted to provide shear resistance and to protect the threaded rods from corrosion.

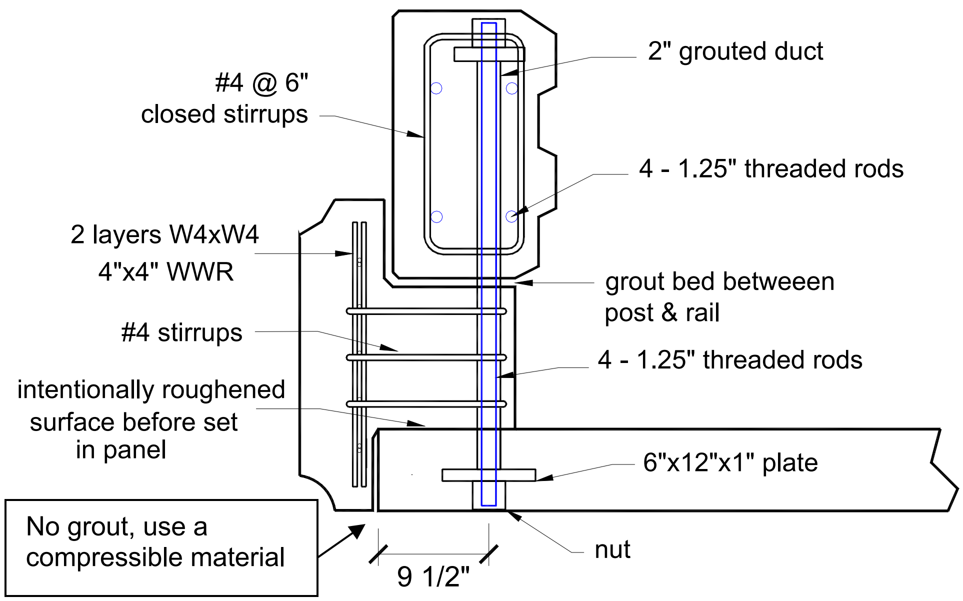

Figure 11 Original precast barrier post and rail cross section.

The rails are reinforced with four 1 ¼ inch, 150 ksi threaded rod and confined with closed number 4 stirrups, as shown in Figure 11. Since the maximum negative moment occurs at the center of the post, the interior face threaded rods must be connected together to resist the negative moment (see Figure 12). The exterior face threaded rods do not need to be connected because the rail only experiences positive moment between the posts but not at the location of the post. The connection between the threaded rods is created by using a 6 inch long, ¼ inch thick, Grade 50 ksi hollow structural section (HSS). The pockets for connecting the threaded rods and the gaps between the rails are grouted in the end step to create a solid section.

This system will be produced for a full-scale crash test study at the Midwest Roadside Safety Facility, at Lincoln, Nebraska. The results of the testing will be used to determine if the precast safety barrier meets the TL-4 criteria.

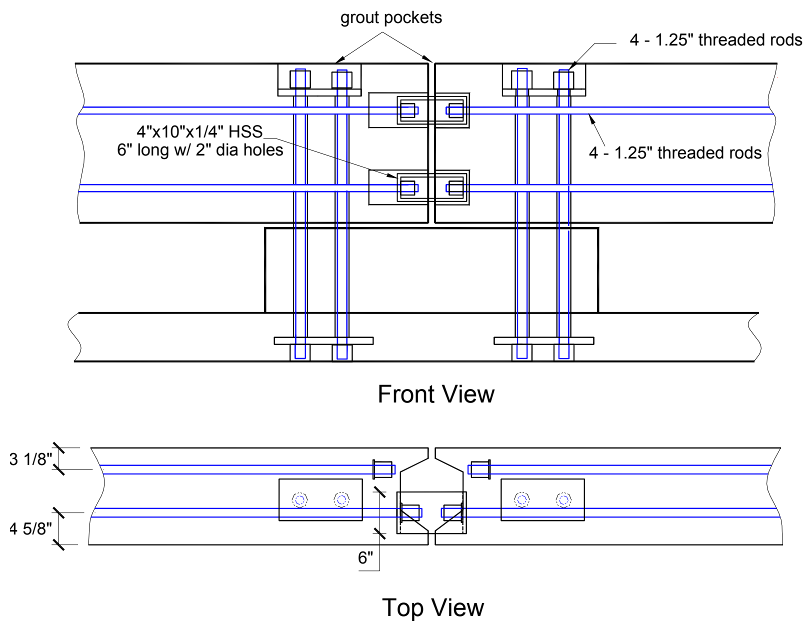

Figure 12 Post and rail reinforcement front and top views.

176th and I-80 Precast Barrier System

Based on the application of the 176th and I-80 Bridge, several adjustments were made to refine the precast barrier. Since this bridge is an overpass for Interstate 80, a closed barrier system must be utilized to prevent drainage onto the roadway beneath it. The less stringent TL-3 load criteria are used for design because this bridge is serving a county road. This system is non-grouted and is made so that the individual rails are more easily interchangeable. The precast barrier is designed as one unit with the same length as each panel, 12 feet, and is attached to the deck at the construction site. The barriers are not continuous between the panels, and the exterior face of each barrier can have numerous aesthetic options applied to it. The front elevation view of this system is shown in Figure 13.

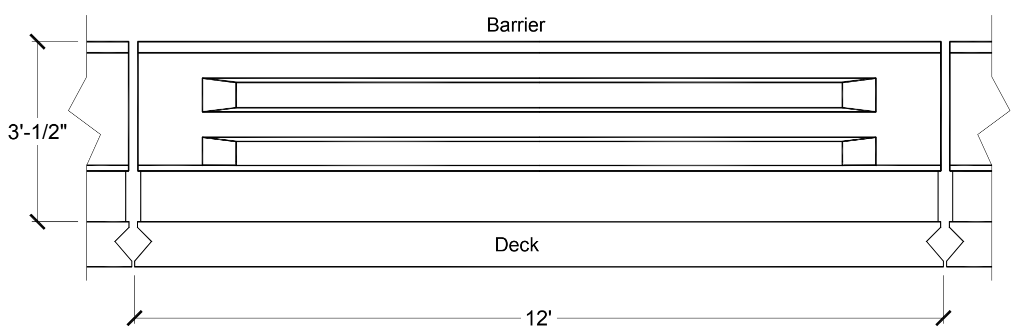

Figure 13 176th and I-80 barrier front elevation view.

In order to satisfy the TL-3 impact loading criteria, the static load range required is generally 60 to 75 kips. The barrier system is conservatively designed using the upper value of 75 kips static load applied 34 inches above the bridge deck. The force is assumed to be distributed over 4 feet of distance, and this barrier system acts similar to a continuous post without a rail. The flexural bending force in the barrier is resisted by 1 3/8 inch diameter 150 ksi threaded rods at 2 feet spacing along the length of the panel. The threaded rods are passed through ducts in the precast barrier and deck. The rods are threaded into an embedded nut welded to a plate at the bottom of the deck. A nut is then tightened on the upper end of the rod to connect the barrier to the deck.

The horizontal force between the barrier and deck panel is resisted by a shear key made out of structural angle iron welded to u-bars. One 2 x 4 x ½ inch angle iron is embedded in the barrier and welded to u-bars for reinforcement. A complimentary 2 x 4 x ½ inch angle iron is resting on top of the precast deck and is connected to the deck by weldable u-bars. An energy absorption pad is positioned between the two pieces of angle iron. The cross section and reinforcement details are shown in Figure 14.

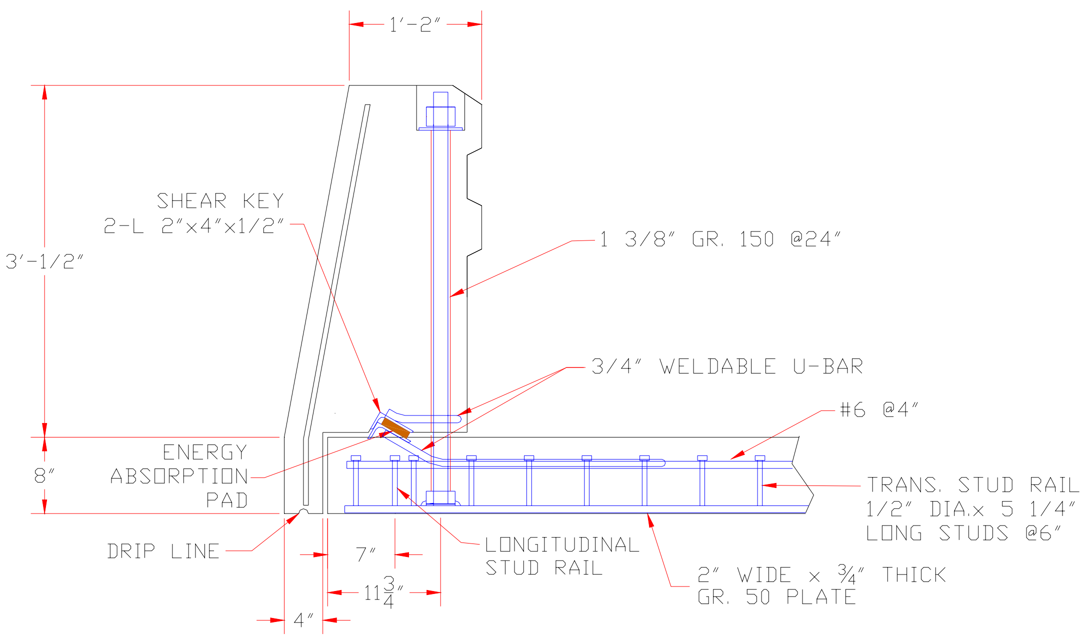

Figure 14 Flexure and shear reinforcement in the 176th & I-80 barrier system.

The deck is reinforced with number 6 bars at 4 inches spacing and stud rails. The stud rails are utilized to resist punching shear and consist of ½ inch diameter studs welded every 6 inches to a 2 inches wide by ¾ inch thick plate. One stud rail is transversely positioned 4 inches on center on both sides of each threaded rod. Also, one stud rail is positioned longitudinally 7 inches from the edge of the slab. The bottom layer of WWR will not be included in this zone due to placement problems. Figure 15 is a plan view of the deck reinforcement for the overhang. The schematic does not show the weldable u-bars or the top layer of WWR. The precast barrier system is still under design and investigation.

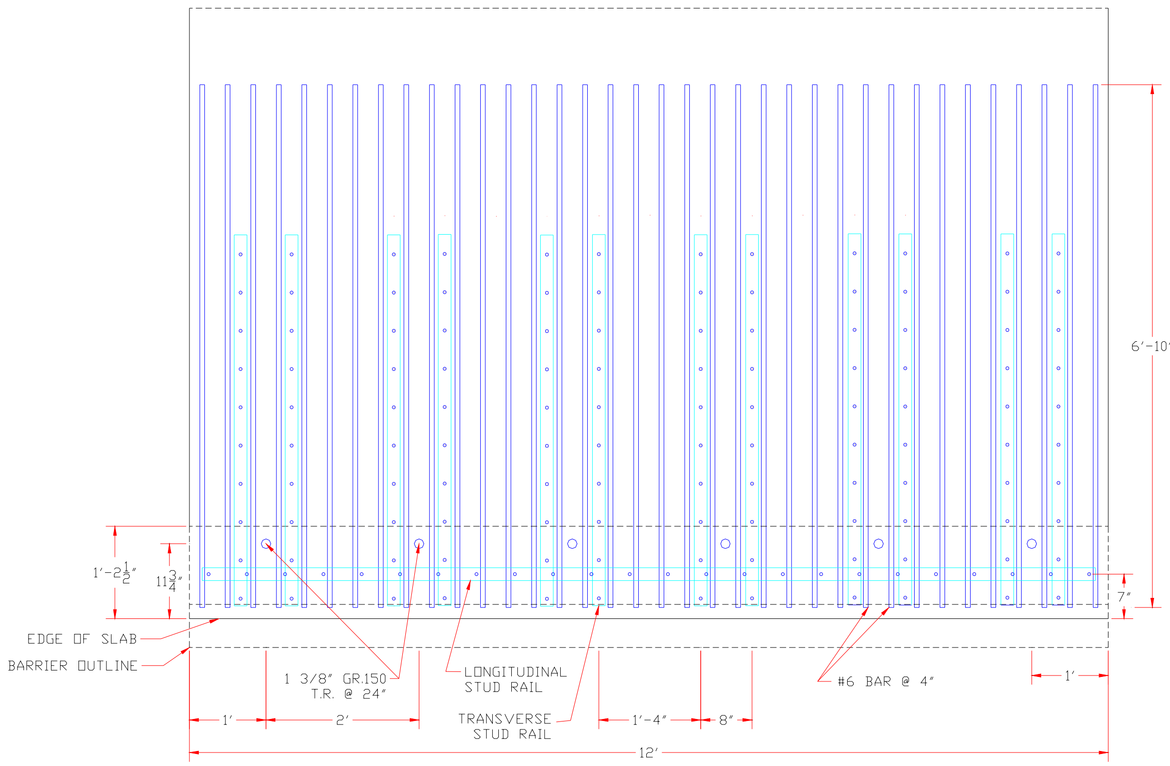

Figure 15 Deck reinforcement for bending moment and punching shear.

CONCLUSIONS

This paper summarizes the modifications to the NUDECK system from the 1st generation and provides a design summary for this deck in the 176th & I-80 Bridge. More detailed design information can be found in the Thesis: "Second Generation NUDECK system—Optimization of full-depth precast bridge deck panels[7]." The NUDECK 2nd generation system has optimized the original NUDECK system that was implemented on the Skyline Bridge. The Second Generation NUDECK has increased construction speed and reduced cost to produce an overall more efficient and effective system.

The speed of construction will be increased greatly by the new post-tensioning details. According to the contractor's supervisor for the Skyline Bridge, the two most time consuming construction steps of the 1st generation NUDECK were threading the post-tensioning strands through the open longitudinal channels and pouring and curing the concrete overlay. In the 2nd generation NUDECK, the strands are easily and rapidly placed on top of the concrete girders prior to deck panel installation. The anchorage box details facilitate easy and rapid post-tensioning.

The concrete overlay is a very time consuming step and detracts from the rapid construction concept. The time required to pour and cure a concrete overlay is approximately equivalent to the time required when pouring and curing for a CIP deck. With the modified NUDECK system, an overlay is eliminated. The longitudinal channels are removed, and the panel is cast with an extra sacrificial grinding depth. A smooth roadway profile is provided through surface grinding, all while maintaining adequate corrosion protection for the deck reinforcement.

Based on the rapid construction concept, ease of replacement criterion, and the application of the 176th & I-80 Bridge, a modified precast barrier system is being developed. The time required for forming, tying the reinforcement, placing the concrete, and curing the concrete for a standard CIP barrier design is eliminated. The prefabricated barrier system is attached to an individual deck panel as a single unit for the width of one panel. The barrier is non-continuous and non-grouted. The system is very rapid and convenient to repair or replace in the event of a vehicular crash.

The original NUDECK system's cost has been reduced through simplifying the panel details and construction steps. The panel length was increased from eight feet to twelve feet. The production, transportation, and construction costs are less for this size. Even though transverse pretensioning is beneficial, the precast panels are designed without transverse pretensioning to simplify detailing of the crown and to reduce cost. This eliminates one step of the panel production and induces competitive precasting practices. A double corrosion protected WWR is utilized as the main panel reinforcement to control cracking and decrease the amount panel fabrication time.

Another simplifying detail in the NUDECK panels is the horizontal shear connections. Composite action between the precast deck panels and the girders is achieved with a simple and efficient detail utilizing high strength threaded rod with anchorage nuts. The shear pockets and threaded rods are spaced at 48 inches to reduce panel fabrication cost and installation constraints. The high strength threaded rod allows for the least number and smallest size of shear pockets.

The NUDECK full depth precast concrete bridge deck panel system has benefits over using cast-in-place decks. One of the greatest benefits of the NUDECK system is quality. In general, the quality of precast deck systems is superior to cast-in-place construction because the fabrication and casting occurs in plant controlled conditions. The work occurs at ground level and adequate time is used to create high tolerance details. A major drawback of using CIP decks is the shrinkage cracks that result from curing the deck concrete with the relatively stiff girder composite reinforcement. Within a few hours after the concrete is placed, the concrete begins to harden and becomes part of the girder/deck composite system. However, the concrete has a very low tensile strength at this time. Due to shrinkage and temperature drop as the heat of cement hydration dissipates, the concrete deck volume would reduce, but the girder and composite reinforcement resists the reduction. This problem often results in shrinkage cracks in the transverse direction within 60 days of the concrete age. Precast panels are cured without girder restraints and are free to reduce in volume. Thus, the shrinkage cracking is greatly reduced by using precast deck panels.

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