

NUDECK — An Efficient and Economical Precast Prestressed Bridge Deck System

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Stay-in-place (SIP) precast, prestressed concrete deck panels serve as permanent formwork and also house positive moment reinforcement. In addition, prestressing results in deck slabs designed to have uncracked bottom fibers under service loading and, thus, improved durability. Despite these positive attributes, the full potential of SIP panel systems has not been realized, partly due to the drawbacks discussed in this paper. This paper presents an improved SIP precast, prestressed concrete panel system called the NUDECK. This system eliminates the major drawbacks of conventional SIP concrete precast panels while maintaining their structural and economic efficiencies. The precast panel covers the entire width of a bridge, resulting in elimination of the need for forming of the overhangs. The system also reduces the time and labor required for installing a larger number of individual panels between girder lines. The proposed panel is unique in that it is continuous transversely and longitudinally, and it utilizes reinforcing bars without concrete to preserve the strand prestress across girder lines. No proprietary materials are used in producing the panels.

The large majority of bridge decks in the United States are built using cast-in-place slabs. Recently, however, a number of bridge deck systems have been developed for the construction of new bridges and for the rehabilitation of deteriorated bridge decks. These systems include full-depth prefabricated deck panels and stay-in-

place (SIP) precast, prestressed concrete panels with a cast-in-place composite topping.

Full-depth cast-in-place bridge decks allow for field adjustment of the profile of the bridge riding surface. Despite the widespread use of this system, however, it has several drawbacks. They include slow speed of construction, high cost of forming, extensive fieldwork and limited use during cold or inclement weather. Also cast-in-place concrete decks are susceptible to cracking.

The full-depth prefabricated deck system has been used successfully in New York, Pennsylvania, Indiana, and other states. There are a number of full-depth prefabricated deck systems available, such as the Exodermic Deck, the Steel Grid Deck, the Inver-set Deck, and the Full-Depth Precast Prestressed Deck developed at the University of Nebraska.¹ Among the advantages of these systems are high speed of construction and elimination of field forming. However, they do not easily allow for a smooth riding surface and often require top surface grinding or a thin overlay.

Stay-in-place precast concrete panels with a cast-in-place composite topping form a system that has been used successfully in Florida, Texas, Pennsylvania, West Virginia, Indiana, Kentucky, Missouri and several other states.² This system provides a thin solid precast, prestressed panel of 3 to 4 in. (76 to 102 mm) to serve as a form for the cast-in-place topping and to house the positive moment reinforcement.

The precast panels are produced in 4 to 8 ft (1.22 to 2.44 m) widths. They are butted against each other without any continuity between them. They are usually set on variable thickness bearing strips to allow for elevation adjustments. This system is faster to build than conventional cast-in-place decks; however, field forming for the overhangs is still required.

Reflective cracks over the transverse joints between the SIP panels are considered a major problem in this system. Their formation is believed by the authors to be due primarily to the discontinuity between the precast panels, which leads to a significant drop in the deck stiffness.

Table 1. Drawbacks of conventional SIP panel system and proposed solutions.

Drawbacks of conventional SIP panel system	Proposed solutions
1. Need for forming for the overhangs.	The first interior precast panel should extend to cover the overhang to provide forming for the cast-in-place concrete topping.
2. Handling and installing of a large number of pieces.	A full width precast panel should be used to cover the full width of the bridge.
3. Prestressing is not fully developed.	Strands should not be cut over girders.
4. Reflective cracking over the transverse joint between precast panels.	Precast panels should be longitudinally connected in a simple way that does not complicate the production and installation of the panels.
5. Handling of the finishing machine loads without the need of additional brackets.	The continuous precast panel should be designed to support the finishing machine loads, the self weight of the cast-in-place topping concrete and the panel self weight without the need of forming.
6. Longitudinal cracking over girder lines due to creep of individual SIP prestressed panels.	The SIP panel should be continuous over girders.

This paper presents the details of a proposed precast, prestressed stay-in-place concrete panel system that was developed to overcome the major drawbacks of the existing SIP precast panel system. It includes the results of full-scale fatigue and ultimate load testing. Also included is the basis for design.

It is shown that due to conservative design assumptions, the actual capacity of the proposed system exceeds that required by dead and live loads. The system is shown to be much faster to build than cast-in-place full depth decks and somewhat faster than a conventional SIP deck system. As a result of the extensive analysis and testing, panel details have been refined to produce further economies

EVOLUTION OF PROPOSED SYSTEM

The drawbacks of a conventional SIP concrete panel system were identified based on a literature review^{2,5} and on testing at the University of Nebraska.⁶ Each of these drawbacks was studied and a solution was proposed as detailed in Table 1.

According to the proposed solutions for the drawbacks of the conventional SIP panel system, it was determined that the SIP precast panel should cover the entire width of the bridge and should be continuous both longitudinally and transversely.

DESCRIPTION OF PROPOSED SYSTEM

A detailed description of the system is best given through an example of a 44 ft (13.41 m) wide bridge. The bridge deck width consists of three 12 ft (3.66 m) spans and two 4 ft (1.22 m) overhangs. Fig. 1a shows the cross section of the deck. The deck thickness is composed of a 4 1/2 in. (114 mm) thick SIP precast continuous panel and a cast-in-place concrete topping. The thickness of the cast-in-place topping may vary from 3 1/2 to 4 1/2 in. (89 to 114 mm), based on the recommended minimum concrete cover, girder spacing, and type of live load.

Fig. 1b shows a plan view of the precast panel with the panel covering the entire width of the bridge. The panel length can vary from 4 to 12 ft (1.22 to 3.66 m), depending on the transportation and lifting equipment available in the field. A full-length gap, a continuous concrete blockout with reinforcement continuing uninterrupted, is provided at girder locations to accommodate the shear connectors. High strength concrete is used in the panel. A specified concrete release strength of 4000 psi (28 MPa) and a specified 28-day compressive strength of 8000 psi (55 MPa) were found to be required for the girder spacing under consideration.

The panel is pretensioned from end to end with twelve 1/2 in. (12.7 mm) diameter low-relaxation, 270 ksi (1.86 GPa) strands. The strands are provided

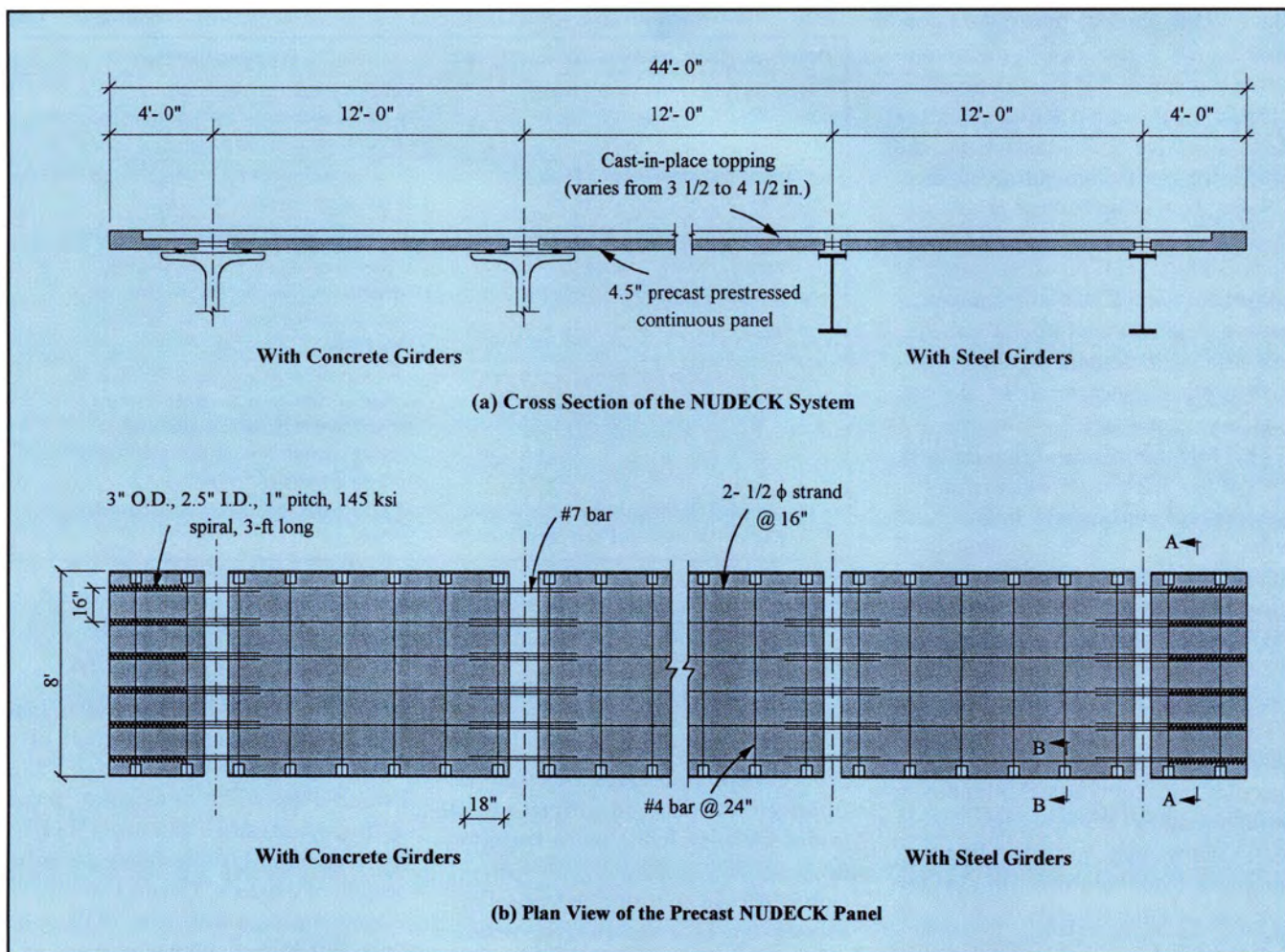


Fig. 1. Precast NUDECK panel system. **Note:** 1 in. = 25.4 mm, 1 ft = 0.3048 m.

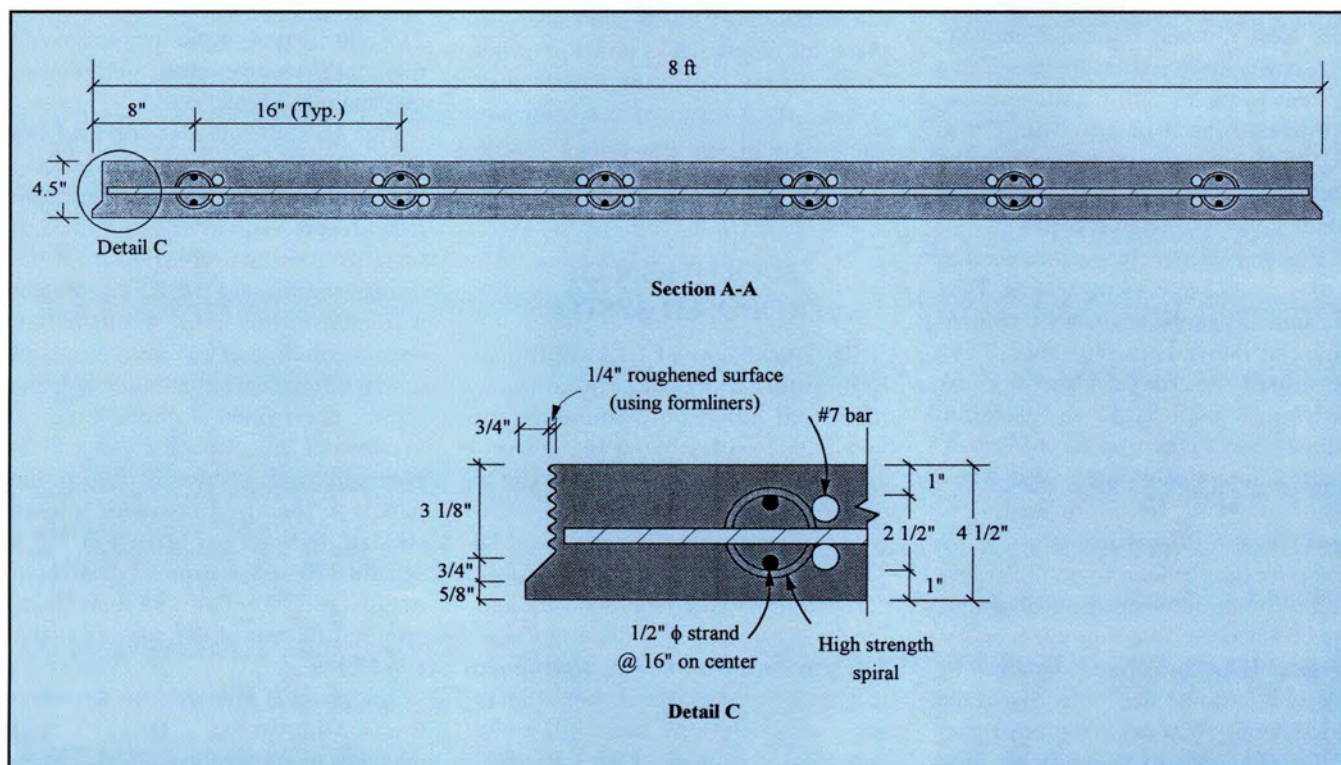


Fig. 2. Section A-A of proposed system. **Note:** 1 in. = 25.4 mm; 1 ft = 0.3048 m.

in two layers and are uniformly spaced at 16 in. (406 mm), as shown in Section A-A in Fig. 2. A minimum clear concrete cover of 1 in. (25 mm) is used for both the top and bottom layers of strands. The 1 in. (25 mm) concrete bottom cover satisfies the AASHTO Standard Specifications requirement, Article 9.25.1.2.⁷

Twenty #7 (#22) reinforcing bars are used in two layers, as shown in Fig. 1b and Section A-A in Fig. 2, to maintain the gap and to transmit the

prestressing force from one bay, between girder lines, to another. These bars have an 18 in. (457 mm) embedment length to transmit the compression force. Reinforcement spacing satisfies the requirements of the AASHTO Standard Specifications,⁷ Articles 8.21.2 and 9.25.2.

The top surface of the panel is roughened to an amplitude of $\frac{1}{4}$ in. (6 mm) to provide for the composite action between the SIP precast panel and the concrete topping. A recent study con-

ducted by Kumar and Ramirez (1996)⁸ showed that a broom-roughened surface can provide full composite action without the need for shear connectors.

Development of Strands Over Short Distances

According to the AASHTO Standard Specifications,⁷ the $\frac{1}{2}$ in. (12.7 mm) diameter strand needs a distance of about 8 ft (2.4 m) to be fully developed. Therefore, in the case of short or no overhangs, the flexural capacity of

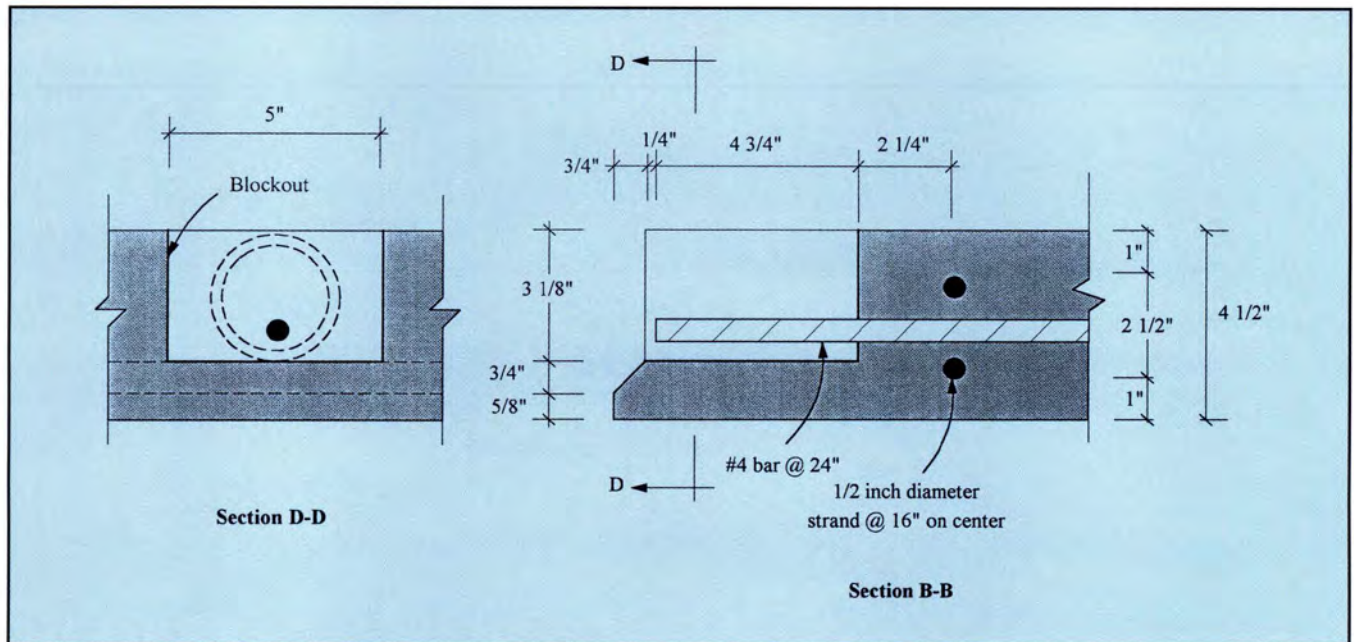


Fig. 3. Details of shear key and reinforced pocket. **Note:** 1 in. = 25.4 mm; 1 ft = 0.3048 m.

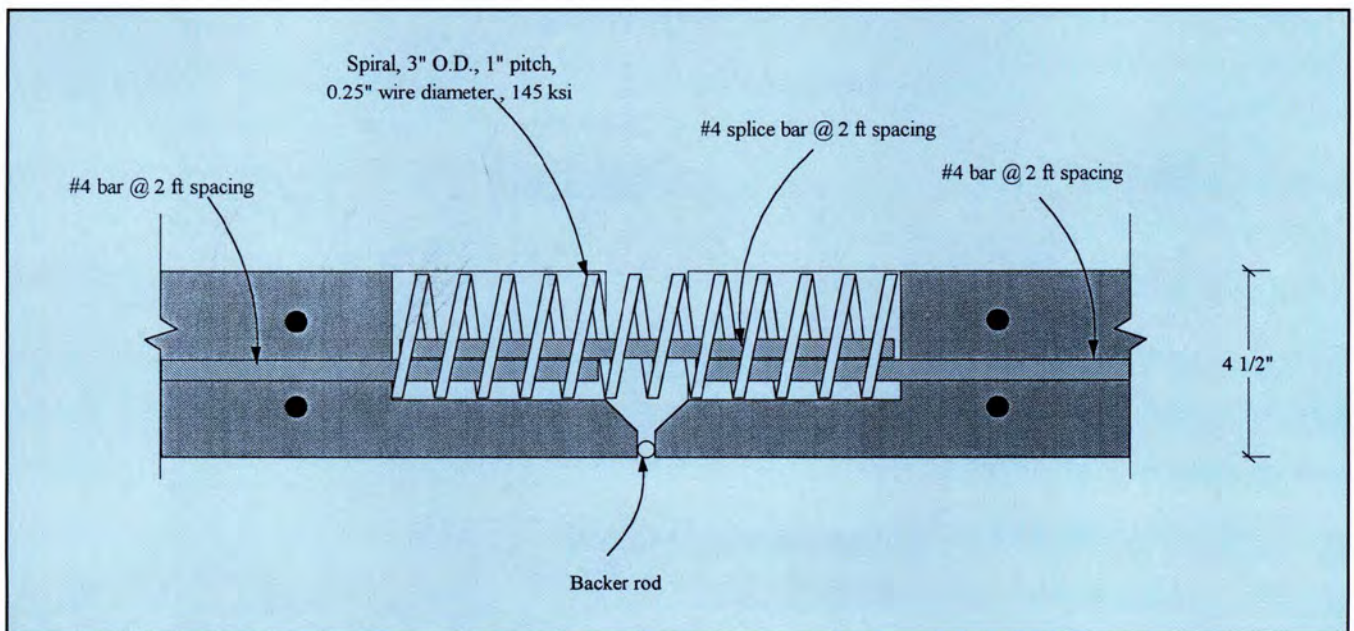


Fig. 4. Panel-to-panel connection. **Note:** 1 in. = 25.4 mm; 1 ft = 0.3048 m.

the overhang, if any, and the exterior span of the deck will be affected. Tests of strand pull-out specimens⁹ showed that confining concrete around the strands, with a high strength spiral for a distance of 3 ft (0.9 m), fully developed them. Thus, each group of two strands in the precast panel is confined with high strength spiral for a distance of 3 ft (0.9 m), as shown in Detail C in Fig. 2.

Panel-to-Panel Connection

To maintain continuity in the longitudinal direction between the adjacent precast panels, shear keys and rein-

forced pockets are provided. Detail C in Fig. 2 shows the dimensions of the proposed shear key. The vertical side of the shear key is roughened in order to produce a shear interlock mechanism. Surface roughening can be provided by attaching form liners to the side forms of the panel.

Reinforced pockets are spaced at 2 ft (0.61 m) on center. Fig. 3 gives the dimensions of the pocket. The pocket can be formed with a stay-in-place light gauge metal sheet or a thin plastic tube.

The panel is reinforced longitudinally with #4 bars (#13 bars) spaced at

2 ft (0.61 m) at the pocket location. To provide for tension development for the #4 bars (#13 bars), they are spliced using a special confinement technique, as shown in Fig. 4. The splice consists of a loose 11 in. (279 mm) long, #4 bar (#13 bar) and a high strength spiral whose properties were independently evaluated with tension specimens and found to produce the full bar yield strength of 60 ksi (414 MPa).¹⁰

Adjusting Elevation of Panel

A simple leveling device is developed as shown in Fig. 5 to level the panels over the supporting girders.

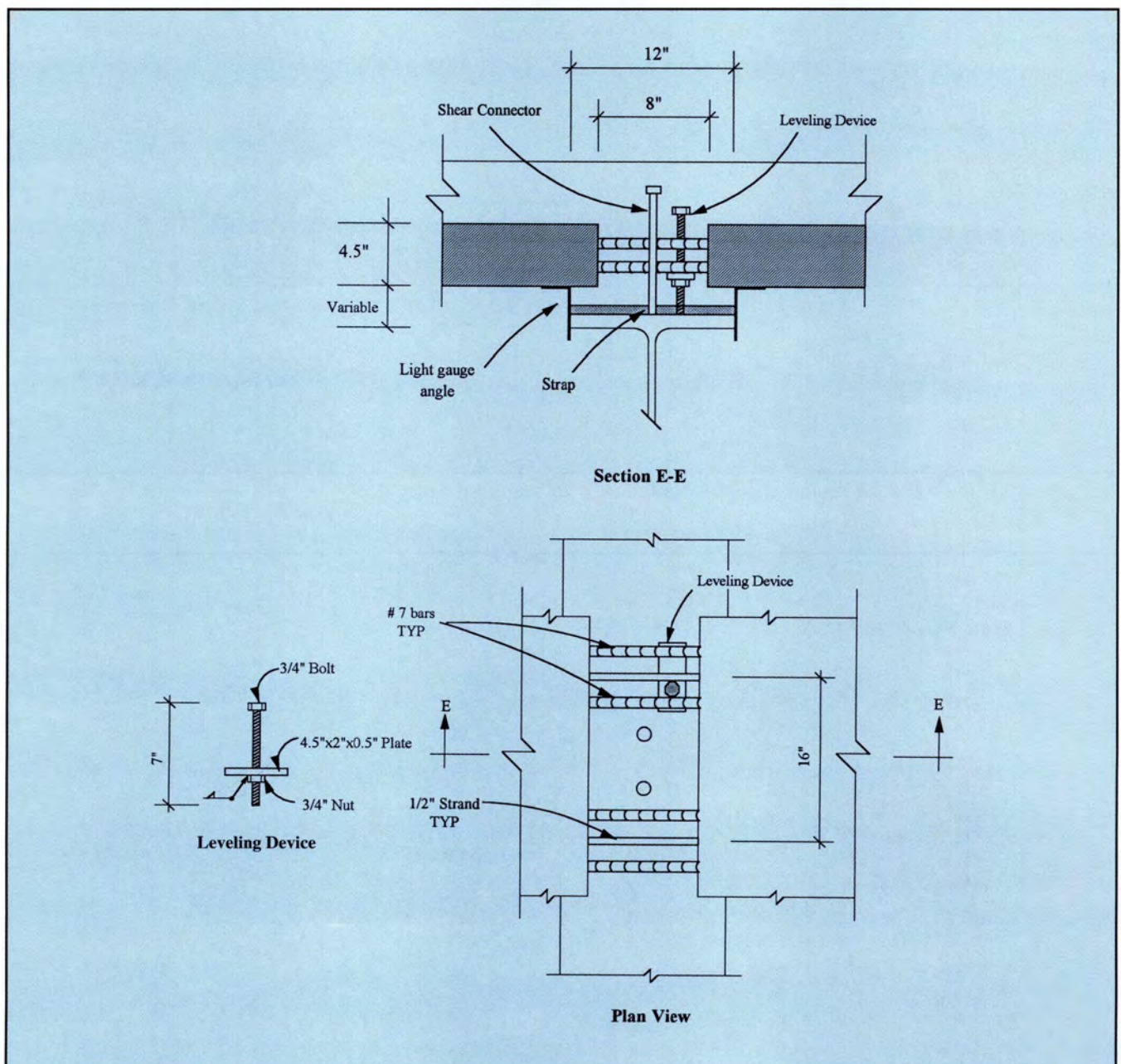


Fig. 5. Leveling assembly. **Note:** 1 in. = 25.4 mm; 1 ft = 0.3048 m.

The leveling device consists of a 1/2 in. (12.7 mm) thick plate, a 7 in. (178 mm) long, 3/4 in. (19 mm) diameter bolt, and a 3/4 in. (19 mm) nut welded to the bottom surface of the plate, as shown in Fig. 5. The bolt can be recovered after grouting the gap.

Girder-to-Panel Connection

Before the precast panels are set on the supporting girders, a grout barrier is installed along the girder. Once the panels are placed over the girders and adjusted with the leveling devices, gaps over the girders are grouted with a flowable mortar mix. The mortar mix should have a compressive strength of 4000 psi (28 MPa) or higher at the time of casting the top slab.

The mortar provides a compression block needed to resist the negative moment over the girders due to loads imposed by the concrete finishing machine and the weight of the concrete topping. The mortar also provides a bearing for the precast panels over the girders. Table 2 gives the components of a mortar mix that was developed and tested at the University of Nebraska. Fig. 6 shows the compressive strength gain vs. time at which the proposed mix reaches the required compressive strength after 5 days.

Section E-E in Fig. 5 shows the details of the grout barrier that can be used with steel girders. It consists of two light gauge metal angles that are connected with a strap. The grout barrier can be adjusted in place by tying the strap with the panel reinforcement. With concrete girders, backer rods can be used as grout barriers by attaching them to the top flange of the girder using construction adhesive, as shown in Fig. 7.

Some of the prestressed concrete girders have a wide top flange. As an example, the NU girder has a top flange of 48.2 in. (1225 mm) and the PCI bulb-tee girder has a top flange of 42 in. (1067 mm). Shear reinforcement in the web of the concrete girders is usually extended outside the top flange to provide for the horizontal shear reinforcement required for composite action.

In some cases, such as in the NU girder, the web reinforcement is pro-

Table 2. Properties and components of mortar mix for gaps.

Specified strength at 7 days	4000 psi (27.58 MPa)
Water-cement ratio	0.35
Slump	High > 10 in. (254 mm)
Unit weight	148 lbs per cu ft (2371 kg/m ³)
Fine aggregate to total aggregate ratio	100 percent
Components	
Screen sand, C33	2725 lbs per cu yd (1617 kg/m ³)
Cement	650 lbs per cu yd (386 kg/m ³)
Fly ash	280 lbs per cu yd (166 kg/m ³)
Total cementitious materials	930 lbs per cu yd (552 kg/m ³)
Water	329 lbs per cu yd (195 kg/m ³)
Low-range water reducer (322N)	37.8 oz. per cu yd (1240 ml/m ³)
High-range water reducer (Rheobuild 1000, ASTM C494 Type A&E)	153 oz. per cu yd (5018 ml/m ³)

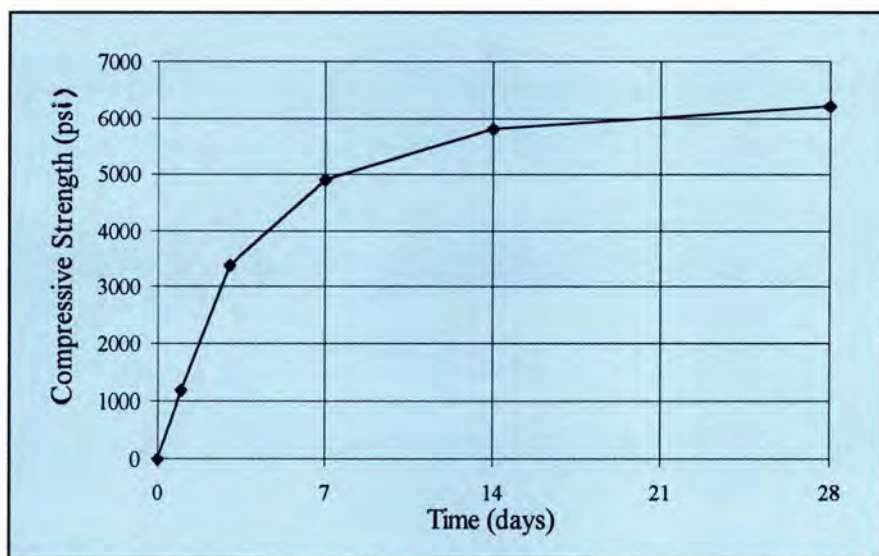


Fig. 6. Concrete compressive strength vs. time of mortar mix. **Note:** 1 in. = 25.4 mm; 1 ft = 0.3048 m.

vided with a horizontal bent for the required development length. Also, to provide for replaceability of the deck and to protect the girder flange tips from damage during the deck replacement, some state agencies, such as the Nebraska Department of Roads, recommend debonding of the outside 8 in. (203 mm) of the top flange of the NU girders.

Fig. 7 shows the suggested details for using the proposed panel with wide flange girders. The width of the block-outs over the girder is increased to 14 in. (355 mm) to provide enough space for the web reinforcement. Backer rods are used as a grout stopper. They can be placed at any distance from the tip of the flange to provide for the required debonded distance.

Cast-in-Place Concrete Topping

After the mortar mix reaches the specified compressive strength, a cast-in-place topping reinforced with epoxy coated reinforcing bars or welded wire fabric is used. The topping thickness should allow for 2 in. (51 mm) of clear cover for corrosion protection and a 1/2 in. (12.7 mm) additional wearing surface.

DESIGN PROCEDURE

AASHTO Specifications⁷ are used to design the NUDECK system where the deck is considered as a continuous beam supported by the longitudinal girders. A number of studies^{11,12} have demonstrated that using this approach to estimate the bending moment in the

transverse direction is conservative. Overdesign is mainly caused by two factors: (1) ignoring concrete slab arching between girders, and (2) assuming that the girders are rigid, rather than flexible supports.

The design procedure consists of an analysis for four conditions: (1) release of prestress; (2) handling and shipping; (3) placement of topping; and (4) service. The first, second and third conditions involve a non-composite precast concrete section, while the fourth condition involves a precast/cast-in-place concrete composite section.

Release of Prestress

Service stresses in the precast concrete panel and service stresses in the

reinforcement in the gap are checked. Compatibility and equilibrium equations are applied at the section at the gap to calculate the compressive stress gained in the #7 (#22) bars and the tensile stress lost in the prestressing strands. It can be shown that the elastic strain in the reinforcement in the gap is:

$$\epsilon = \frac{A_p f_p}{A_s E_s + A_p E_p} \quad (1)$$

where

f_p = tensile stress in strands

A_s = area of reinforcing bars

A_p = area of prestressing strands

E_s = modulus of elasticity of reinforcing bars

E_p = modulus of elasticity of prestressing strands

With f_p taken as the value just before release f_{pi} , the strain, ϵ , can be calculated. The compression stress in the reinforcing bars is:

$$f_s = \epsilon(E_s) \quad (2)$$

The tensile stress in the prestressing strands is:

$$f_p = f_{pi} - \epsilon(E_p) \quad (3)$$

A similar analysis at the midspan between the girder is carried out to determine the elastic shortening loss and the tensile stress in the prestressing strands at that location. This is needed for the positive moment design.

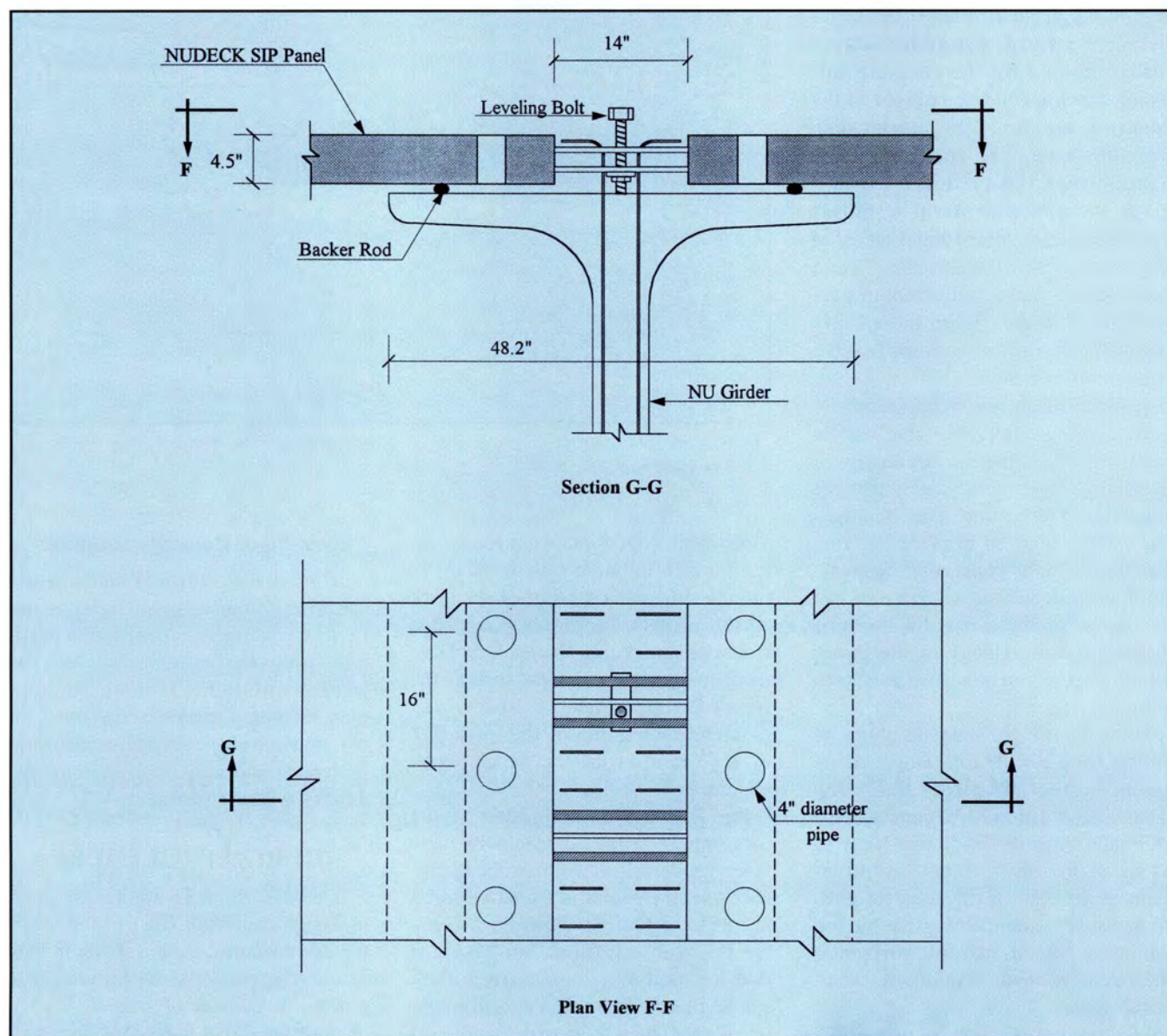


Fig. 7. NUDECK system with wide flange concrete girder. **Note:** 1 in. = 25.4 mm; 1 ft = 0.3048 m.

The reinforcing bars in the gap must be adequate to satisfy two design criteria: (1) preserve as much prestress in the strands as possible, and (2) transfer prestress to the adjacent concrete without excessive stress concentrations. The first criterion is met by calculating the prestress in the strands, as discussed in the preceding paragraph, and by checking the #7 bars (#22 bars) at the gap against buckling as follows:¹³

$$F_a = \frac{\left[1 - \frac{(Kl/r)^2}{2C_c^2}\right]}{\left[\frac{5}{3} + \frac{3(Kl/r)}{8C_c} - \frac{(Kl/r)^3}{8C_c^2}\right]} \quad (4)$$

$$C_c = \sqrt{\frac{2\pi^2 E_s}{F_y}} \quad (5)$$

where

F_a = allowable stress of reinforcing bars

K = effective length factor
 l = width of gap

F_y = yield strength of reinforcing bars
 r = radius of gyration of reinforcing bars, which is 0.25 times the bar diameter for circular sections

The second criterion is satisfied by using the tension development length as the minimum required embedment into the concrete. Although this may be too conservative because the bars are expected to be predominantly in compression, the tension development length used provides for a more gradual transfer of prestress and does not significantly affect the overall cost of the system.

Handling and Shipping

It is recommended that the panel be lifted at the locations of the girder lines. Refer to the later section on handling and shipping for more information. During lifting, the panel self-weight will create negative moments at the lifting points resulting in additional compressive stresses in the #7

bars (#22) in the bottom layer of reinforcement. Eqs. (1) and (2) should be used to verify the buckling resistance of these bars.

The strand stress, f_p , used in Eq. (1) to determine the strain under this condition should include relaxation loss between the time of release and handling. The intrinsic relaxation loss for low-relaxation strand, Δf_p , can be calculated as follows:¹⁴

$$\Delta f_p = \frac{\log t}{45} \left(\frac{f_{p,int}}{f_{py}} - 0.55 \right) f_{p,int} \quad (6)$$

where

$f_{p,int}$ = initial prestress

t = time in hours under stress, $f_{p,int}$

f_{py} = yield stress of prestressing steel

Once the bar stress is calculated from Eq. (2), the incremental stress due to panel weight should be added before the stress is checked against buckling using Eqs. (4) and (5). The incremental stress can be calculated using the standard (M_y/I) formula.

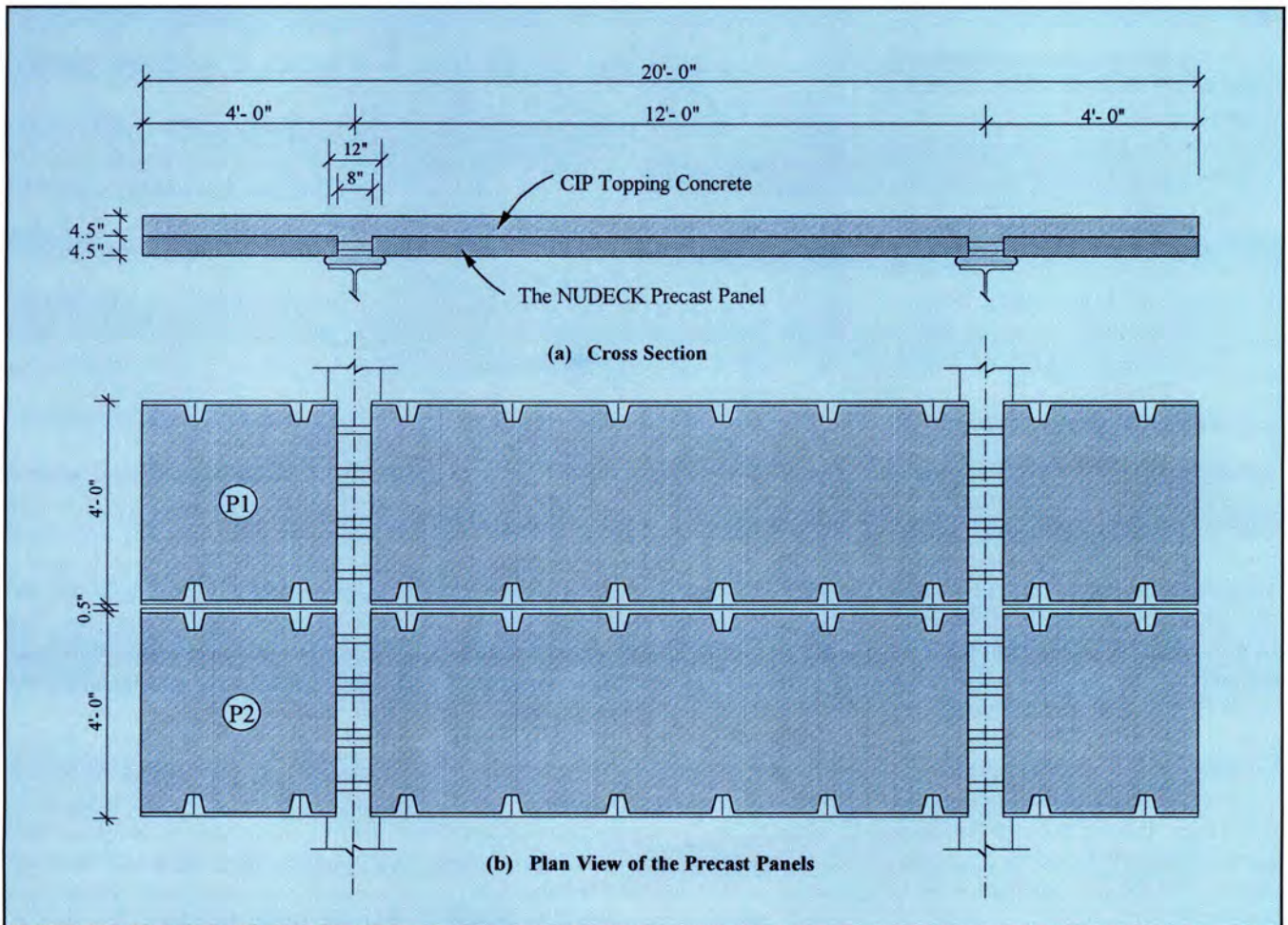


Fig. 8. Test specimen. **Note:** 1 in. = 25.4 mm; 1 ft = 0.3048 m.

Placement of Topping

The precast panel is designed to support the precast panel self-weight, the topping weight, a construction load of 50 lbs per sq ft (2.394 kPa), and the loads provided by the concrete finishing machine. The finishing machine load is applied at the edge of the precast panel. At this condition, it is assumed that the gaps over the girder lines have been filled with mortar and that the mortar has attained adequate strength.

Three locations should be checked in flexure. These are the maximum positive moment section between girder lines, the maximum negative moment section at interior supports, and the maximum negative moment section at exterior supports. Concrete stresses due to unfactored loads and ultimate flexural capacity are checked at the maximum positive moment section while the ultimate flexural capacity is checked at the maximum negative moment sections.

At Service

Concrete stresses due to unfactored loads and the ultimate flexural capacity of the composite section are checked at the maximum positive moment. The negative moment sections at the interior and exterior girders should be designed as reinforced concrete sections. Appendix B gives the design of a 44 ft (13.41 m) wide bridge using the proposed deck system.

TESTING OF PROPOSED SYSTEM

A 20 ft (6.10 m) wide bridge was constructed in the laboratory. The bridge consisted of two steel girders spaced at 12 ft (3.66 m) and two 4 ft (1.22 m) overhangs. The supporting steel girders had a 12 in. (305 mm) flange width. Fig. 8a shows the cross section of the test specimen. Two 20 x 4 ft (6.10 x 1.22 m) precast panels, P1 and P2, were produced in the construction laboratory, as shown in Fig. 8b.

Wood forming was used to form the transverse shear keys and the 8 in. (203 mm) gap. Polystyrene foam was used to form the reinforced pockets. In commercial production, steel forms would be used for the shear keys and



Fig. 9. Forming of gap at girder line.



Fig. 10. Details of completed forming.

for the gap over the girder lines. Figs. 9 and 10 show the forming details.

A series of gauges were installed on the reinforcing bars and strands before casting the concrete of the precast panel. These gauges were used to check the stresses in the reinforcing bars and strands over the 8 in. (203 mm) gap at the time of release. Also, they were used to check the stresses in the strands at the time of failure of the composite section at the point of maximum positive moment (midspan section).

Due to the crude method of pre-stress release by relatively rapid flame cutting at the university laboratory, minor cracks at one end of the panels were observed. This type of crack has

occasionally been reported in the production of conventional stay-in-place panels.^{2,3} Although these cracks had no measurable effect on the structural performance of the complete tested specimen, avoiding a sudden unsymmetrical release of forces from the strands to the concrete can eliminate these cracks. This can be achieved by using gradual hydraulic detensioning, or by casting two "dummy" concrete blocks, one on each side of the precast panel.

Fig. 11 shows the precast panel during handling. Figs. 12 and 13 show the two panels in place over the girders and the reinforced pocket, respectively. No proprietary materials or



Fig. 11. Precast panel during handling displaying good stiffness.

equipment were used in constructing the test specimen.

Testing Plan

Two tests were conducted, a cyclic load test and an ultimate load test. The loading position was at the transverse joint between the precast panels. The test setup simulated the rear axle of

two HS25 trucks spaced at 4 ft (1219 mm), as shown in Fig. 14. The cyclic load test was designed for 2 million cycles to determine the performance of the transverse joint under cyclic service loads including cracks and leakage control. The ultimate load test was designed to find the mode of failure and to give an indication of the ca-

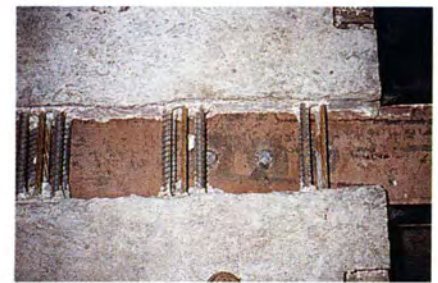


Fig. 12. Precast panels in place over girder lines showing large space available for placement of connector with reinforcement (see Fig. 13 below).

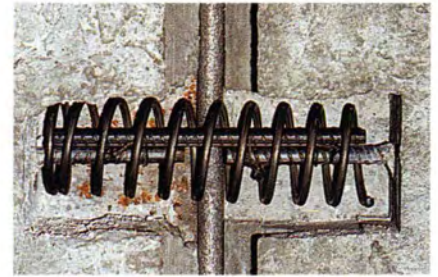


Fig. 13. Panel pocket showing connection detail for full development of longitudinal reinforcement (see Fig. 12 above).

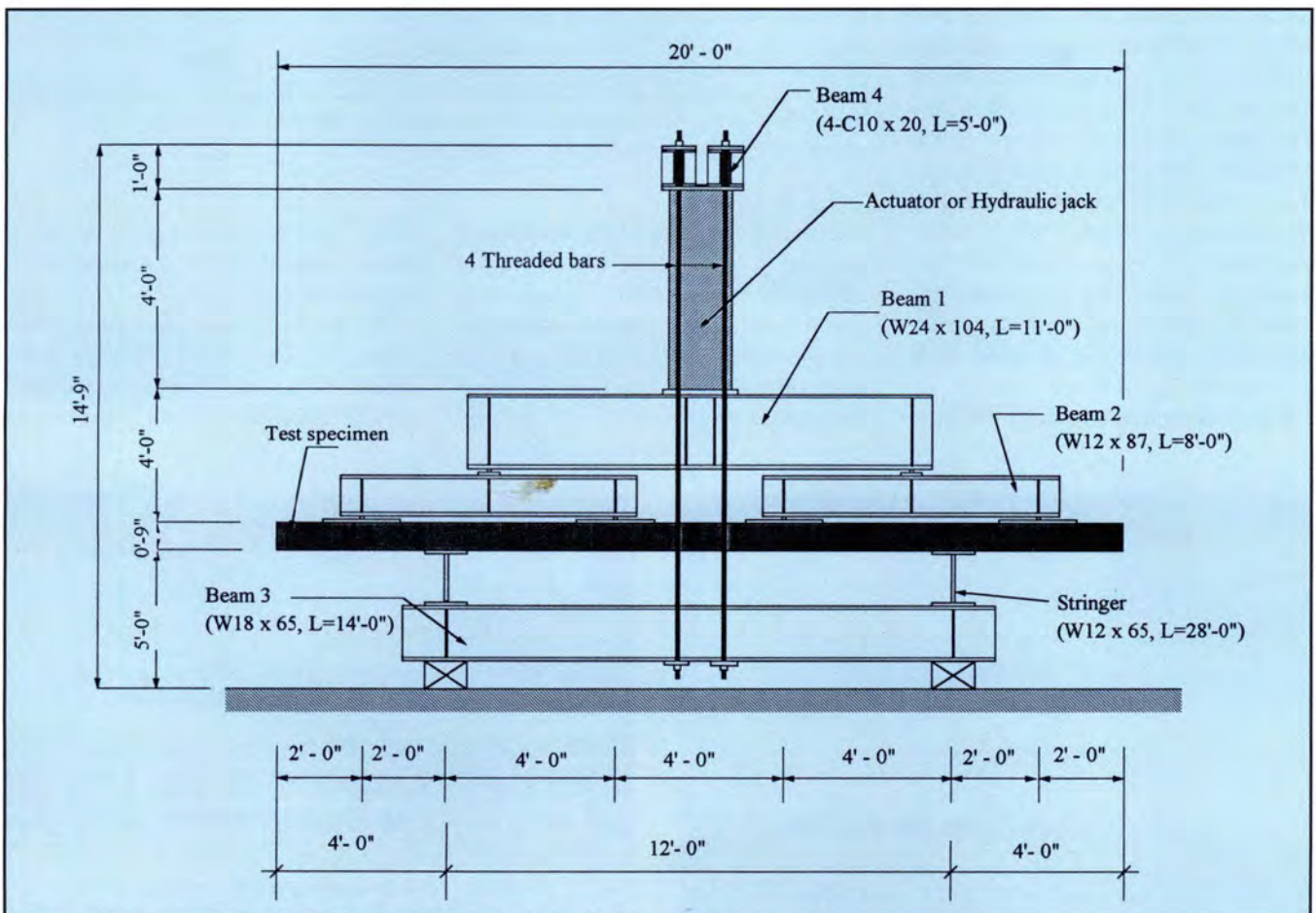


Fig. 14. Cross section of test setup. **Note:** 1 in. = 25.4 mm; 1 ft = 0.3048 m.

capacity of the system compared to the specified AASHTO factored loads.

Structural Behavior Under Cyclic Load

The top surface of the panel exhibited hairline cracks over the girders at 700,000 cycles. Only two hairline cracks were noticed, one over the north girder and another over the south girder. At 2 million cycles, the cracks did not extend the full 8 ft (2.44 m) length of the specimen, but were limited to about two-thirds of that length.

The number, size, and length of cracks reported in this system were much less than those of the conventional full-depth cast-in-place system and the conventional SIP panel deck system, which had been tested earlier.⁶ No reflective cracks over the transverse joint between the precast panels were observed. The research team felt this was because the connection between the precast SIP panels stiffened the system and led to a better load distribution.

Cracks due to cyclic loading over the girders were fully closed after load removal. Two factors contributed to this favorable behavior, namely, the continuity of the reinforcement of the panel over the girder lines and the redistribution of creep stresses causing the topping slab to be “prestressed” over time. This phenomenon was confirmed by plotting the relationship between the applied load and the resulting concrete stresses in the panel deck, as shown in Figs. 15a and 15b.

Strain measurements were taken at

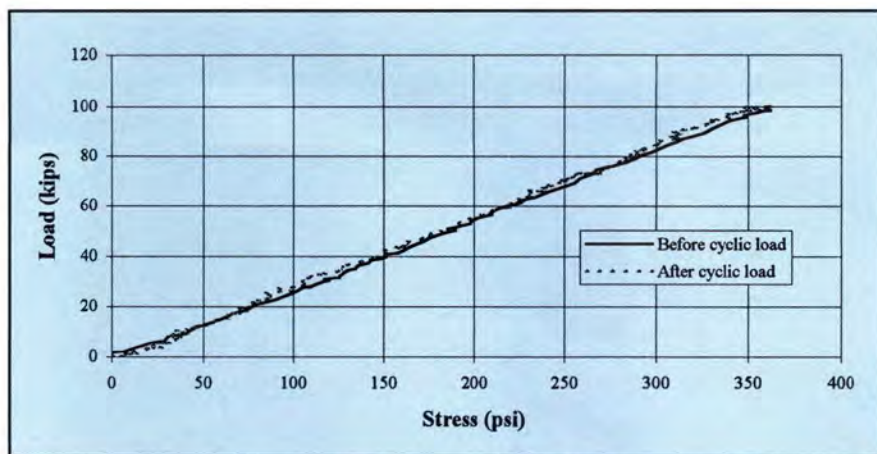


Fig. 15a. Load-stress relationship at bottom surface at maximum positive moment section. **Note:** Positive stress means compressive stress.

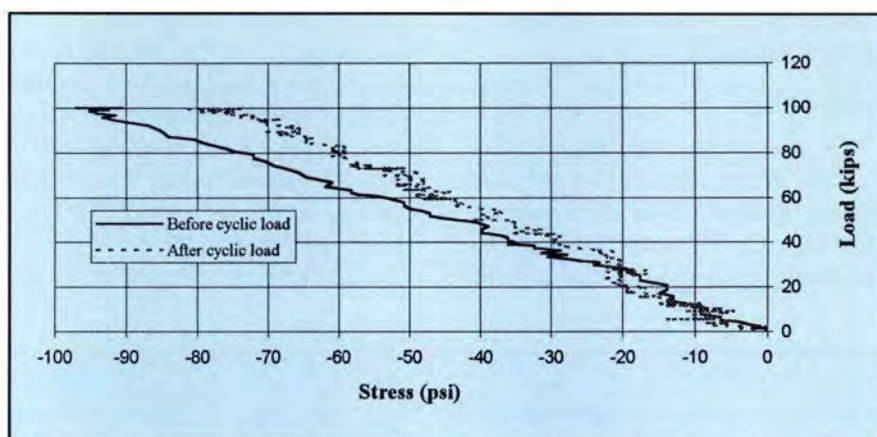


Fig. 15b. Load-stress relationship at top surface at maximum negative moment section. **Note:** Negative stress means tensile stress.

the bottom surface at the maximum positive moment zone and at the top surface at the maximum negative moment zone. Both figures show the load-to-stress relationship before and after the cyclic loading. For the maximum positive moment zone (see Fig.

15a), load-to-stress curves were almost identical before and after the fatigue load.

At the maximum negative moment zone (see Fig. 15b), the tensile stress decreased between the initial and final cycle of the cyclic load. This means



Fig. 16. Flexural cracking at ultimate load.



Fig. 17. Shear failure of conventional SIP panel system showing lack of tension tie development due to strand discontinuity.

that due to creep in the prestressed concrete panel, the cast-in-place topping gained some compression stresses, which reduced the total tension stresses.

Structural Behavior at Ultimate Load

The panel was loaded to failure. As the load increased, cracks at the top surface over the girders and at the bottom surface between the girders started to form. The load continued to increase until the maximum capacity of the hydraulic jack used in the test, 400 kips (1779 kN), was reached. At this stage, the maximum applied moments were 1.05 times the factored load moment required by the AASHTO Specifications⁷ using HS25 design truck. Although hairline cracks were observed both at the top and bottom surfaces of the deck, no signs of failure were noticed.

The test setup was rearranged to allow application of the load at the

Table 3. Comparison between NUDECK and cast-in-place deck systems.

Deck system	Construction time (minutes per sq ft)	Relative flexural capacity*	Mode of failure
Full-depth cast-in-place	1.80	157 percent	Flexure
Conventional SIP panel	0.92	84 percent	One-way shear
NUDECK	0.75	147 percent	Flexure

* As a percentage of required AASHTO factored capacity with HS25 Truck.

two central points between the girders, which resulted in higher positive bending moments and no negative moments. When the jacking load reached 140 kips (623 kN), signs of concrete crushing were observed at midspan between the girders at the edges of the deck (see Fig. 16).

At this stage, the maximum applied moment was 1.47 times the factored load moment required by the AASHTO Specifications⁷ using an HS25 design truck. Due to indications of possible concrete crushing and the large associated deflection, in excess

of 2 in. (52 mm), a decision was made to stop the test. This was an indication that the system had not yet reached its ultimate capacity.

The observed high capacity of the system was the result of an arching effect in the deck between girder lines. The continuous prestressing in the transverse direction provided the "tension tie" required for the arching effect. Such a tension tie exists in cast-in-place conventionally reinforced decks if the reinforcing bars are kept continuous over the entire deck width. However, the tension tie in the con-

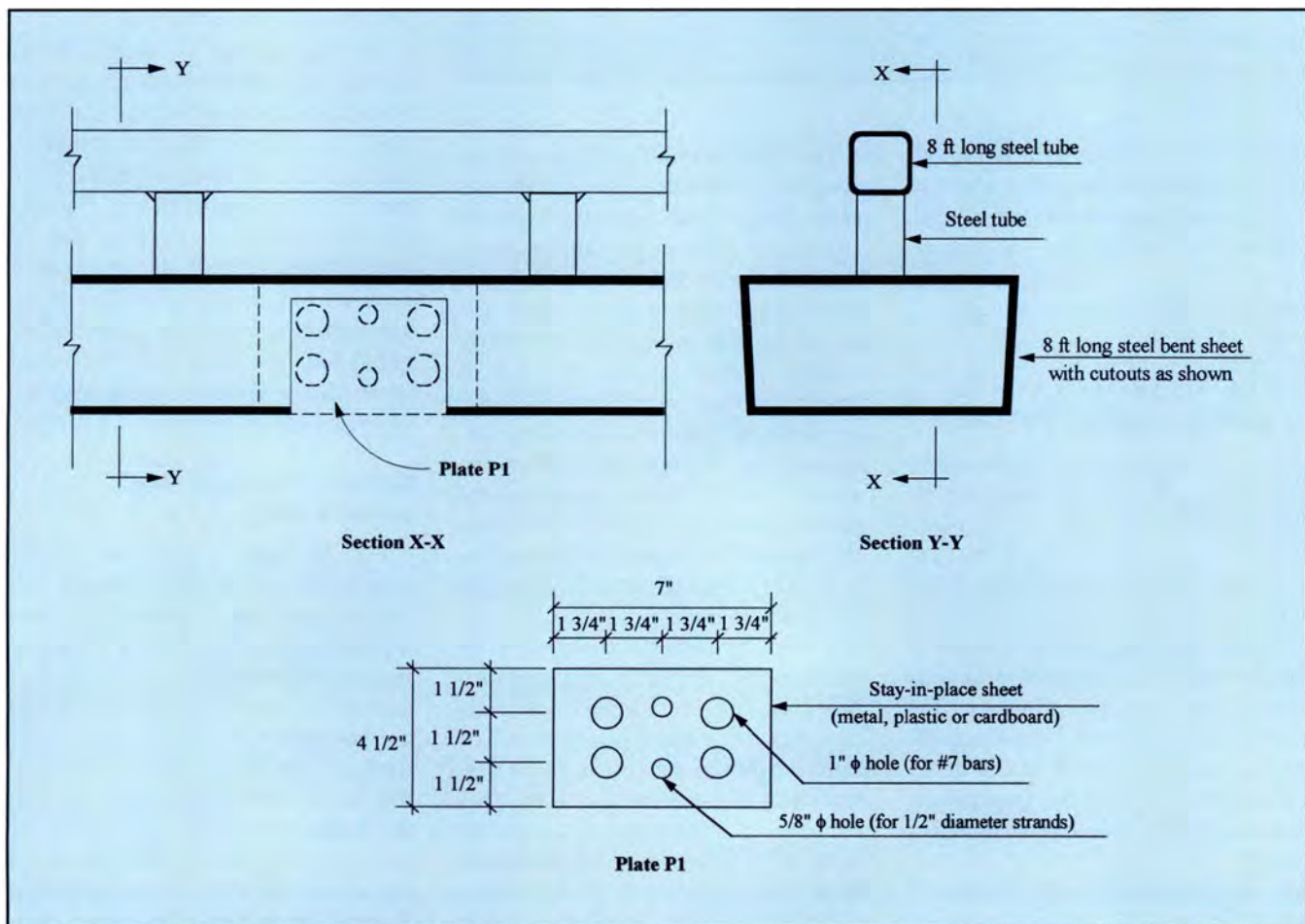


Fig. 18. Conceptual detail of gap forms. Note: 1 in. = 25.4 mm.

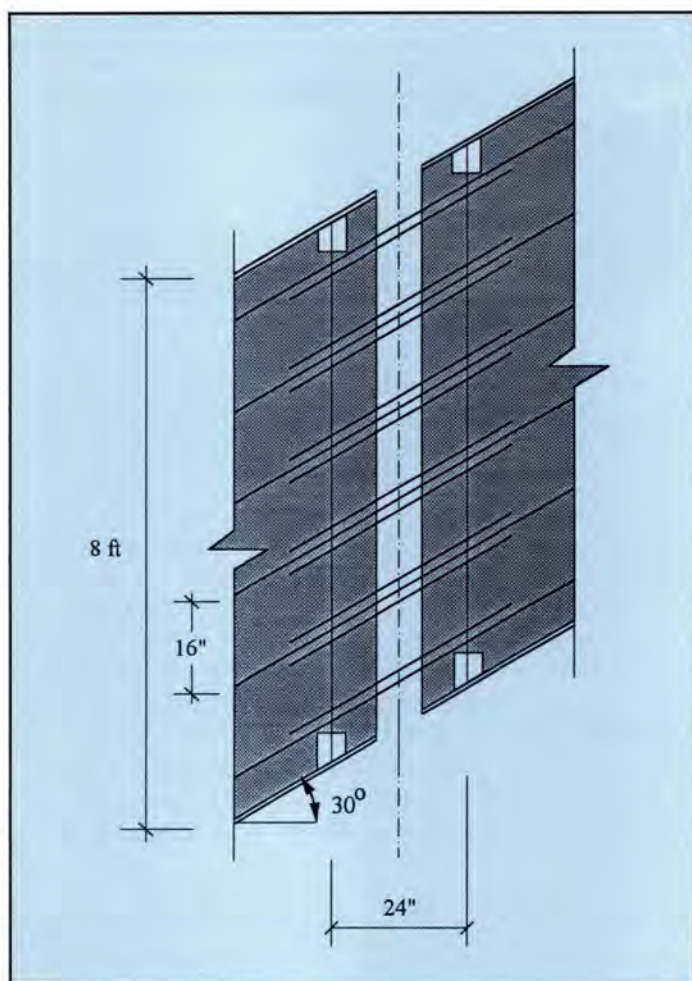


Fig. 19.
Plan view of a
skew NUDECK
precast panel.

Note:

1 in. = 25.4 mm;
1 ft = 0.3048 m.

ventional SIP panel system is lacking because the prestressing reinforcement is not continuous over the girder lines.

After removing the load, the deck returned to its original undeflected shape. No residual deflection was noticed.

COMPARISON WITH OTHER DECK SYSTEMS

A complementary testing program^{6,15} was conducted under the same laboratory conditions for two cast-in-place deck systems, the full-depth cast-in-place deck and the conventional SIP panel. The bottom reinforcement layer (positive moment reinforcement) of the full-depth cast-in-place deck was intentionally made continuous over the girders in order to study the arching mechanism formed in the deck. This resulted in a higher flexural capacity for the system than that required by the specifications. At failure, the maximum positive moment was 1.57 times the factored moment required by AASHTO Specifications.⁷

The conventional SIP panel test specimen, which had the same dimensions, level of prestress, and concrete strength as those of the proposed system test specimen, had a sudden one-way shear failure because of the lack of anchorage of the prestressing strands, as shown in Fig. 17. Lack of strand anchorage resulted in losing the tie of the arching mechanism and decreased the flexural capacity of the system. At failure, the maximum applied positive moment was 0.84 times the factored moment required by AASHTO Specifications⁷ using an HS25 design truck.

Table 3 provides a comparison of the relative speed of construction based on laboratory conditions, relative flexural capacity, and mode of failure between the cast-in-place systems and the proposed system. This comparison shows that the proposed system has a faster construction speed than other systems due to elimination of field forming and handling of fewer precast panels. Also, it has almost

double the flexural capacity of the conventional SIP panel system. This is because continuity of the prestressing reinforcement provides full anchorage of the tension tie compared to the conventional SIP panel system.

PRODUCTION ISSUES

The following production issues resulted from several meetings with the designers of the Bridge Division of the Nebraska Department of Roads (NDOR) and members of the Precast Concrete Association of Nebraska.

Details of Forms for Gap Over Girder Lines

Fig. 18 shows the details of an assembly that can be used for forming the gap over the girder lines. The assembly consists of 8 ft (2.44 m) long, full width panel, steel bent sheet formed in a trapezoidal shape to fit the geometry of the gap. Cutouts in the steel bent sheet are provided for the reinforcement passing over the gap, the #7 (#22) bars and the 1/2 in. (12.7 mm) diameter strands.

To avoid leakage of concrete during casting, thin metal sheets are used to blockout the cutouts of the steel bent sheet, as shown in Fig. 18 (Plate P1). The strands are threaded through the metal sheets and pretensioned. The steel forming assembly is installed and the metal sheets are then adjusted in their position and glued to the sides of the steel forming assembly. Then the #7 (#22) bars are installed. The metal sheets can be left in the panel after the concrete cures or be removed if desired.

Details of Proposed Panel for Skew Bridges

Fig. 19 shows a plan view of the proposed panel for skew bridges. The precast panel can be skewed by skewing the end side forms in the prestressing bed. The #4 (#13) reinforcing bars and the reinforced pocket blockouts are positioned in the direction of the traffic of the bridge. Because the reinforced pockets do not interfere with the continuous shear key, the shear key side form used for straight panels can also be used with skew panels. For forming the gap over the girder lines, longer metal sheets should be used.

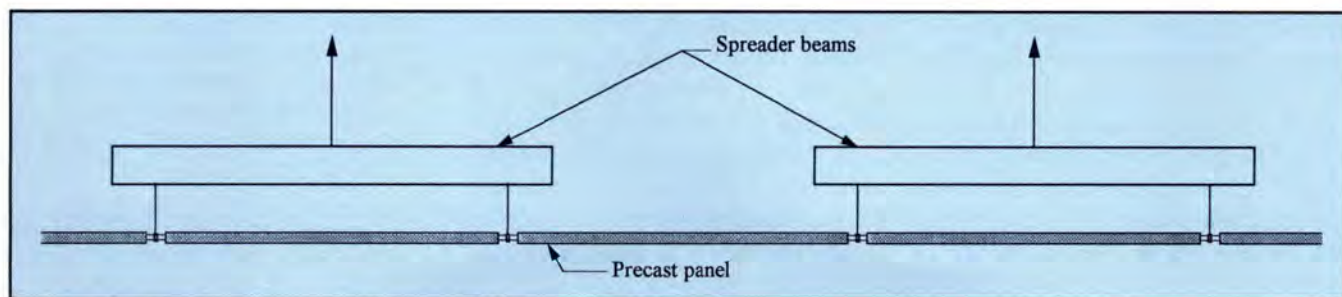


Fig. 20a. Handling using multiple lifting points.

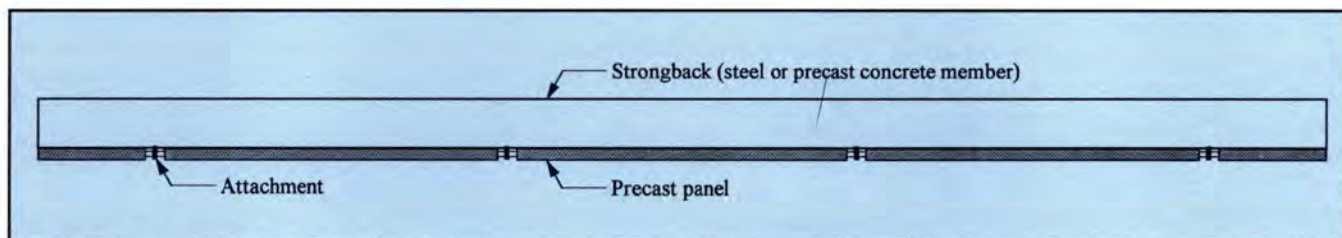


Fig. 20b. Handling using strongback.

HANDLING AND SHIPPING

It is recommended that the panel be lifted at the location of the girder lines. This can be achieved by two techniques. The first technique involves using multiple lift points with spreader beams,¹⁶ as shown in Fig. 20a. Each lifting position would have two lifting points near the sides of the panel.

For very wide and/or very thin panels, it may be advisable to use a second technique. This involves using a "strongback" attached to the panel at the location of the girder lines, as shown in Fig. 20b. The strongback can be a structural steel or precast concrete member. It would be attached to the precast panel before the panel is removed from the forms and removed after the panels are placed on the girders and shimmed to the required elevation.

SYSTEM ECONOMICS

To obtain a cost estimate for the proposed precast panel, a two-span bridge with a total length of 250 ft (76.20 m) and a width of 44 ft (13.41 m) is considered. These dimensions represent a fairly typical overpass structure. The precast panels are produced in 8 ft (2.44 m) wide segments. Table 4 gives the cost estimate of the materials for the precast panel.

Table 4 shows that the material cost of the proposed precast panel is approximately \$2.24 per sq ft. The total cost after adding overhead, profit, labor and shipping, will be in the range of \$5.50 to \$7.00 per sq ft. Although the cost of the proposed panel is expected to be higher than that for a conventional SIP panel, which usually ranges from \$4.50 to \$6.00 per sq ft, using the proposed system will save time and money because no field forming is required for the overhangs and fewer components need to be handled. In addition, the proposed system exhibits superior structural performance over the conventional SIP panel system (i.e., no reflective cracks), which results in lower maintenance costs and a longer expected life of the bridge deck.

CONCLUSIONS

A new precast concrete bridge deck system is proposed. This system consists of continuous stay-in-place (SIP) precast prestressed concrete panels and a cast-in-place concrete topping. Based on the results of this investigation, the following conclusions can be reached:

1. Use of the proposed system results in control of the transverse deck cracking encountered in cast-in-place decks due to concrete shrinkage. Because the proposed panels are precast, most of the shrinkage will have taken place before the panels are made integral with the rest of the superstructure.
2. Reflective cracking over panel joints, which has been a concern with conventional SIP panels, is expected to be eliminated in the proposed sys-

Table 4. Estimated cost of NUDECK panel system.

Materials	Quantity per panel*	Cost per panel (in U.S. dollars)
1/2 in. (12.7 mm) diameter strands at \$0.25 per ft (\$0.82 per m)	528 ft (160.93 m)	132.00
Reinforcing bars at \$0.30 per lb (\$0.66 per kg)	250 lbs (113.4 kg)	75.00
Concrete ($f'_c = 8.0$ ksi, $f'_{ci} = 4.0$ ksi) at \$85.00 per cu yd (\$111.00 per m ³)	4.9 cu yd (3.75 m ³)	416.50
Spirals at \$1.00 per ft (\$3.28 per m)	85 ft (25.91 m)	85.00
Leveling devices	—	80.00
Total		\$788.50 per panel \$2.24 per sq ft

* Panel dimensions: 8 x 44 ft (2.44 x 13.41 m).

tem. This is because the proposed panels are made continuous both longitudinally and transversely.

3. The proposed system exhibits improved fatigue resistance and crack control in the longitudinal direction over the girder lines.

4. Because field forming of the deck overhangs is eliminated and the SIP panels can be as wide as the full bridge width, construction speed is significantly improved.

5. The materials used in the production of the proposed panel are nonproprietary construction materials. Thus, it should be cost competitive with other available systems.

6. The proposed system exhibits significantly higher flexural capacity than the conventional SIP panel system because the strands are fully developed at the critical locations.

7. The proposed panels are designed to support the weight of the finishing machine. There is no need to support the overhangs with brackets.

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REFERENCES

1. Yamane, T., Tadros, M. K., Badie, S. S., and Baishya, M. C., "Full-Depth Precast Prestressed Concrete Bridge Deck System," *PCI JOURNAL*, V. 43, No. 3, May-June 1998, pp. 50-66.
2. PCI Bridge Committee, "Precast Prestressed Concrete Bridge Deck Panels," *PCI JOURNAL*, V. 32, No. 2, March-April 1987, pp. 26-45.
3. Barnoff, R. M., Orndorff, J. A., Harbaugh, R. B., and Rainey, D. E., "Full Scale Test of a Prestressed Bridge with Precast Deck Planks," *PCI JOURNAL*, V. 22, No. 5, September-October 1977, pp. 66-83.
4. Fagundo, F. E., Tabatabai, H., Soongswang, K., Richardson, J. M., and Callis, E. G., "Precast Panel Composite Bridge Decks," *Concrete International*, V. 7, No. 5, May 1985, pp. 59-65.
5. Bieschke, L. A., and Klingner, R. E., "Effect of Transverse Panel Strand Extensions on the Behavior of Precast Prestressed Panel Bridge," *PCI JOURNAL*, V. 33, No. 1, January-February 1988, pp. 68-88.
6. Tadros, M. K., and Baishya, M. C., "Rapid Replacement of Bridge Decks," National Cooperative Highway Research Program, NCHRP Report 407, Washington, D.C., 1998.
7. AASHTO, *Standard Specifications for Highway Bridges*, Sixteenth Edition, American Association of State Highway and Transportation Officials, Washington, D.C., 1996.
8. Kumar, N. V., and Ramirez, A. J., "Interface Horizontal Shear Strength in Composite Decks with Precast Concrete Panels," *PCI JOURNAL*, V. 41, No. 1, March-April 1996, pp. 42-55.
9. Tadros, M. K., Baishya, M. C., Saleh, M. A., Yehia, S., and Einea, A., "Strand Bond in Prestressed Concrete Members," *PCI JOURNAL*, V. 43, No. 1, January-February 1998, pp. 86-89.
10. Einea, A., Yehia, S., and Tadros, M. K., "Lap Splice with Confined Concrete," *ACI Structural Journal*, accepted for publication, 1998.
11. Kinnunen, S., "Punching of Concrete Slabs with Two-Way Reinforcement," Transactions of the Royal Institute of Technology, Stockholm, Sweden, Civil Engineering 6, No. 198, 1963, pp. 1-108.
12. Mufti, A. A., Jaeger, L. G., Bakht, B., and Wegner, L. D., "Experimental Investigation of Fiber-Reinforced Concrete Deck Slabs without Internal Steel Reinforcement," *Canadian Journal of Civil Engineering*, V. 20, No. 3, 1993, pp. 398-406.
13. *Manual of Steel Construction, Allowable Stress*, American Institute of Steel Construction, Chicago, IL, 1992.
14. Magura, D. D., Sozen, M. A., and Siess, C. P., "A Study of Stress Relaxation in Prestressing Reinforcement," *PCI JOURNAL*, V. 9, No. 2, April 1964, pp. 13-57.
15. Badie, S. S., "Structural Behavior of Bridge Deck Systems," Ph.D. Dissertation, Submitted to the Graduate College at University of Nebraska-Lincoln, Omaha, NE, December 1997.
16. *Precast Prestressed Concrete Bridge Design Manual*, Precast/Prestressed Concrete Institute, Chicago, IL, 1997.
17. Skogman, B. C., Tadros, M. K., and Grasmick, R., "Flexural Strength of Prestressed Concrete Members," *PCI JOURNAL*, V. 33, No. 5, September-October 1988, pp. 96-123.

APPENDIX A — NOTATION

A = cross-sectional area of non-composite section	I = impact fraction
A_c = cross-sectional area of composite section	I_c = moment of inertia of composite section
A_p = area of prestressing strands	K = effective length factor
A_s = area of reinforcing bars	l = width of gap
a = depth of equivalent rectangular stress block	M_n = nominal flexural capacity
CR_c = prestress loss due to creep of concrete	M_u = factored moment
CR_s = prestress loss due to steel relaxation	n = modular ratio between precast concrete and cast-in-place concrete
C_c = a factor defined by Eq. (5)	r = radius of gyration of reinforcing bars
d = distance from extreme compressive fiber to centroid of reinforcement	P_{25} = load on rear wheel of HS25 truck
ES = prestress loss due to elastic shortening	S = effective span
E_c = modulus of elasticity of concrete	SH = prestress loss due to shrinkage
E_{ci} = modulus of elasticity of concrete at release	S_b = non-composite section modulus for extreme bottom fiber
E_p = modulus of elasticity of prestressing strands	S_{bc} = composite section modulus for extreme bottom fiber
E_s = modulus of elasticity of reinforcing bars	S_t = non-composite section modulus for extreme top fiber
F_a = allowable stress of reinforcing bars	S_{tc} = composite section modulus for extreme top fiber
F_y = yield strength of reinforcing bars	t = time in hours under stress, $f_{p, int}$
f'_c = specified compressive strength of concrete at 28 days	y_{bc} = distance from center of gravity of composite section to extreme bottom fibers
f'_{ci} = specified compressive strength of concrete at release	w_c = unit weight of concrete
f_p = tensile stress in strands	Δf_p = prestress loss due to steel relaxation
$f_{p, int}$ = initial prestress	ϕ = resistance factor
f_{pi} = tensile stress in prestressing strands just before release	ϵ = elastic strain in reinforcement in gap
f_{py} = yield stress of prestressing steel	
f_s = compression stress in reinforcing bars	

APPENDIX B — DESIGN EXAMPLE

This appendix gives a design example of the NUDECK precast SIP panel deck system. A 44 ft (13.39 m) wide bridge is considered. The deck slab is supported over four AASHTO-PCI Bulb-Tee girders with a 42 in. (1067 mm) wide top flange, spaced at 12 ft (3.65 m) on centers. The deck slab has two 4 ft (1.22 m) overhangs. The design is carried out in accordance with the AASHTO Standard Specifications⁷ (STD) for Highway Bridges, 16th Edition, 1996.

B1. Input Data

Precast panel:

(length x width x thickness) = 8 ft x 44 ft x 4.5 in.; $l = 8$ in.

$f'_c = 8000$ psi; $f'_{ci} = 4000$ psi; $w_c = 150$ lbs per cu ft

The SIP panel is reinforced with:

(a) 12 1/2 in. diameter, 270 ksi strands; $E_p = 29000$ ksi; prestress just before release, $f_{pi} = 0.75 \times 270 = 202.5$ ksi; $f_{py} = 243$ ksi; $A_p = 12 \times 0.153/8 = 0.23$ sq in. per ft

(b) 20 #7 bars; $F_y = 60$ ksi; $A_s = 20 \times 0.6/8 = 1.5$ sq in. per ft, as shown in Figs. 1 and 2.

Cast-in-place concrete topping:

Overall thickness = 4.5 in.

Structural thickness = 4.0 in.; note that 1/2 in. is considered an integral wearing surface.

$f'_c = 4000$ psi; $w_c = 150$ lbs per cu ft

Loads:

Live loads = HS25 design truck

Future wearing surface = 2 in. of concrete

$w_c = 150$ lbs per cu ft

Construction load = 50 lbs per sq ft

Concrete finishing machine weights as shown in Fig. B1

Effective span:

$$S = [12 - 0.5 \times (42/12)] = 10.25 \text{ ft (STD Art. 3.24.1.2)}$$

B2. Section Properties per Linear Foot

Non-composite section:

$$A = 54 \text{ sq in.}; S_t = S_b = 40.5 \text{ in.}^3$$

$$E_{ci} = 3834 \text{ ksi}; E_c = 6063 \text{ ksi}$$

Composite section:

$$n = \sqrt{\frac{4000}{10,000}} = 0.63; A_c = 84.24 \text{ sq in.}; y_{bc} = 3.78 \text{ in.}$$

$$A_c = 84.24 \text{ sq in.}; y_{bc} = 3.78 \text{ in.}$$

$$I_c = 482 \text{ in.}^4; S_{bc} = 127.5 \text{ in.}^3; S_{tc} = 102.1 \text{ in.}^3$$

B3. Design of SIP Precast Prestressed Panel at Release

Check concrete stresses of precast concrete panel:

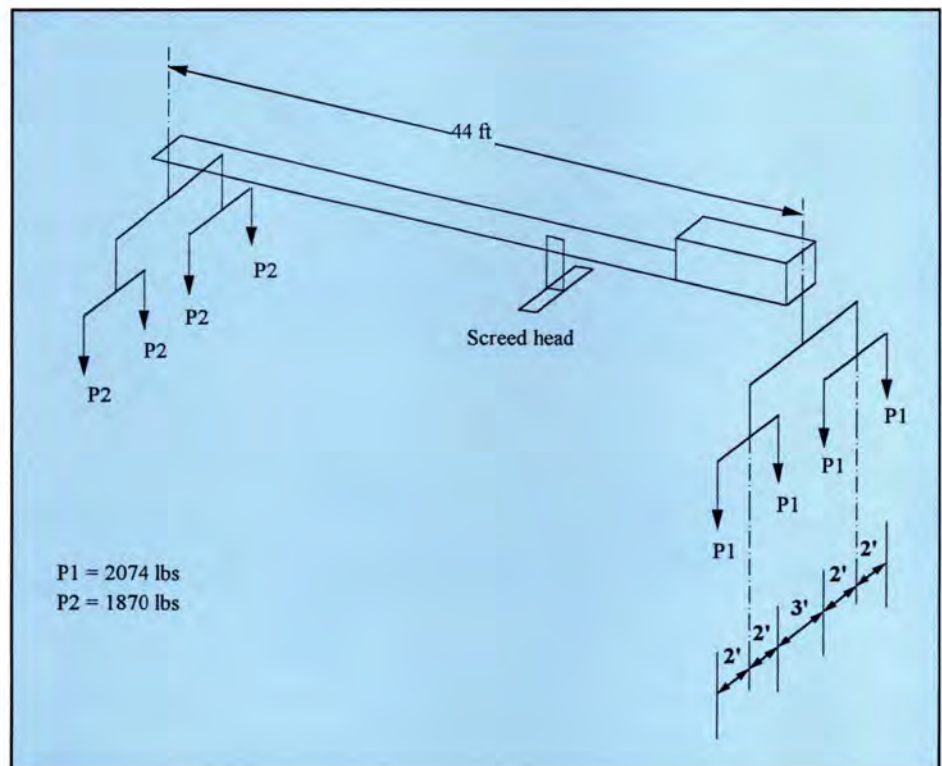
Loss due to elastic shortening, $ES = 6.098$ ksi

(STD Art. 9.16.2.1.2)

Effective prestress just after release = $202.5 - 6.098 = 196.402$ ksi

Effective prestress force just after release = $196.402 \times 0.153 \times 12/8 = 45.1$ kips per ft

Fig. B1.
Finishing machine loads for 44 ft wide bridge. **Note:** 1 ft = 0.3048 m.



Concrete stress of the SIP panel = $45.1/54 = +0.835$ ksi
 Allowable compression = $0.6f'_{ci} = 0.6 \times 4000/1000 = 2.4$ ksi
 > 0.835 ksi (ok)

Check buckling resistance of reinforcing bars at gap:

The strain in the reinforcement is:

$$\epsilon = \frac{0.23 \times 202.5}{1.50 \times 29,000 + 0.23 \times 28,000} = 9.326 \times 10^{-4} \quad \text{Eq. (1)}$$

The effective prestress in the strands just after release is:

$$202.5 - (9.326 \times 10^{-4} \times 28,000) = 202.5 - 26.113 = 176.387 \text{ ksi} \quad \text{Eq. (3)}$$

The compression stress in the bars is:

$$9.326 \times 10^{-4} \times 29,000 = 27.045 \text{ ksi} \quad \text{Eq. (2)}$$

$$C_c = \sqrt{\frac{2\pi^2 E}{F_y}} = \sqrt{\frac{2\pi^2 (29,000)}{60.0}} = 97.7 \quad \text{Eq. (5)}$$

For #7 bar, $r = d/4 = 0.22$ in.; thus, $Kl/r = (0.65 \times 8)/(0.1875) = 23.6 < C_c$

The allowable compression stress in the reinforcing bars is:

$$F_a = \frac{\left[1 - \frac{(Kl/r)^2}{2C_c^2}\right] F_y}{\frac{5}{3} + \frac{3(Kl/r)}{8C_c} - \frac{(Kl/r)^3}{8C_c^3}} \quad \text{Eq. (4)}$$

$$= 33.181 \text{ ksi} > 27.045 \text{ ksi (ok)}$$

B4. Design of Precast Panel at Time of Handling and Shipping

Check buckling resistance of reinforcing bars at gap:

Assume that the prestress is released after one day and that the precast panels are shipped at 28 days. Therefore, the relaxation loss in the strands between time of release and 28 days is:

$$\Delta f_p = \frac{\log(28-1)(24)}{45} \left(\frac{176.387}{243} - 0.55 \right) 176.387 = 1.938 \text{ ksi} \quad \text{Eq. (6)}$$

Effective prestress = $176.387 - 1.938 = 174.449$ ksi

Using Eq. (1) with $f_p = 174.449$ ksi; thus, $\epsilon = 8.034 \times 10^{-4}$

The compressive stress in the bars is:

$$(8.034 \times 10^{-4})(29,000) = 23.299 \text{ ksi} \quad \text{Eq. (3)}$$

Negative moment at the exterior gap due to the self-weight of 4 ft long overhang of the panel is:

$$\left(\frac{4.5}{12} \times 0.150 \right) \left(\frac{4^2}{2} \right) = 0.450 \text{ ft-kips per ft}$$

Moment of inertia of the reinforcement at the gap is:

$$\left(\frac{20 \times 0.6}{8} \times 0.813^2 + \frac{12 \times 0.153}{8} \times 1^2 \right) = 1.22 \text{ sq in. per ft}$$

Incremental stress in the bars due to the panel self-weight is:

$$(0.450 \times 12) \left(\frac{0.813}{1.22} \right) = 3.599 \text{ ksi}$$

Total compressive stress in the #7 bars at the top layer = $23.299 - 3.599 = 19.700$ ksi

Total compressive stress in the #7 bars at the bottom layer = $23.299 + 3.599 = 26.898$ ksi

The allowable compression stress in the reinforcing bars, $F_a = 33.181$ ksi > 26.898 ksi (ok)

Tensile stress in the strands at the top layer = $174.449 + 3.599 = 178.048$ ksi

Tensile stress in the strands at the bottom layer = $174.449 - 3.599 = 170.850$ ksi

B5. Design of Precast Panel at Time of Topping Placement

Check service stresses at maximum positive moment section between girder lines:

$SH = 6.500$ ksi; $ES = 6.098$ ksi; $CR_c = 9.957$ ksi; $CR_s = 3.567$ ksi (STD Art. 9.16)

Effective prestress at service = $202.5 - 6.500 - 6.098 - 9.957 - 3.567 = 176.378$ ksi

Effective prestress force at service = $176.378 \times 0.153 \times 12/8 = 40.5$ kips per ft

Allowable compression = $0.6f'_c = 0.6 \times 10,000/1000 = 6.0$ ksi

Allowable tension = $6.0 \sqrt{f'_c} = 6.0 \sqrt{10,000}/1000 = 0.6$ ksi

Maximum positive moment due to panel self-weight, cast-in-place topping self-weight, and construction load is:

$$\left(\frac{4.5}{12} \times 0.150 + \frac{4.5}{12} \times 0.150 + 0.050 \right) \left(\frac{10.25^2}{11} \right) = 1.552 \text{ ft-kips per ft}$$

Concrete stress at top surface is:

$$\frac{40.5}{54} + \frac{1.55 \times 12}{40.5} = +1.210 \text{ ksi} < +6.000 \text{ ksi}$$

Concrete stress at bottom surface is:

$$\frac{40.5}{54} - \frac{1.552 \times 12}{40.5} = +0.290 \text{ ksi} < +6.000 \text{ ksi}$$

Check flexural capacity at exterior gap:

Negative moment due to panel self-weight, cast-in-place topping self-weight, and construction load is:

$$\left(\frac{4.5}{12} \times 0.150 + \frac{4.5}{12} \times 0.150 + 0.050 \right) \left(\frac{4^2}{2} \right) = 1.300 \text{ ft-kips per ft}$$

As shown in Fig. B1, only three reactions of the finishing machine can be accommodated over the panel width (8 ft); each reaction is 2.074 kips. Therefore, the bending moment = $(3 \times 2.074 \times 4.0)/8 = 3.111$ ft-kips per ft

Total factored moment = $1.3(1.300 + 1.67 \times 3.111) = 8.444$ ft-kips per ft

Knowing the stress in the #7 bars and in the strands (from Section B4) and using the strain compatibility method,¹⁷ the flexural capacity of the grouted section at the gap is:

$\phi M_n = 0.9 \times 12.914 = 11.623$ ft-kips per ft > 8.444 ft-kips per ft (ok)

B6. Design of Deck at Service Loads

Check allowable stresses at maximum positive moment section between girders:

Bending moment due to non-composite loads (self-weight of the SIP panel and the cast-in-place topping) is:

$$\left(\frac{4.5}{12} \times 0.150 + \frac{4.5}{12} \times 0.150\right) \left(\frac{10.25^2}{11}\right) = 1.075 \text{ ft-kips}$$

Bending moment due to wearing surface is:

$$\left(\frac{2.0}{12} \times 0.150\right) \left(\frac{10.25^2}{11}\right) = 0.239 \text{ ft-kips per ft}$$

Bending moment due to live load (HS25 with impact) is:

$$0.8 \left(\frac{S+2}{32}\right) P_{25}(1+I) = 0.8 \left(\frac{10.25+2}{32}\right) (20)(1+0.3) \\ = 7.963 \text{ ft-kips per ft}$$

Concrete stress at top fiber of cast-in-place slab is:

$$0.63 \left(\frac{0.239 + 7.963}{102.1}\right) (12) = 0.607 \text{ ksi (note: } n = 0.63)$$

Allowable compressive stress = $0.4f'_c = 0.4 \times 4.0 = 1.600 \text{ ksi}$
 $> 0.607 \text{ ksi}$

Concrete stress at bottom surface of the SIP panel is:

$$\frac{40.5}{54} - \frac{(1.075 \times 12)}{40.5} - \frac{(0.239 + 7.963)(12)}{127.5} = -0.341 \text{ ksi}$$

Allowable tensile stress = $6 \sqrt{f'_c} = 6 \sqrt{8000} / 1000 = 0.537 \text{ ksi}$
 $> 0.341 \text{ ksi (ok)}$

Check flexural capacity of composite section at maximum positive moment section:

$$M_u = 1.3(1.075 + 0.239 + 1.67 \times 7.963) = 18.999 \text{ ft-kips per ft}$$

Knowing the effective stress in the strands (from Section B5) and using the strain compatibility method,¹⁷ the flexural capacity of the grouted section at the gap is:

$$\phi M_n = 1.0 \times 27.237 = 27.237 \text{ ft-kips per ft} > 18.999 \text{ ft-kips per ft (ok)}$$

Design of negative moment section over interior girder line:

As calculated in Section B6, the bending moment due to wearing surface = 0.239 ft-kips per ft and due to live load (HS25 with impact) = 7.963 ft-kips per ft.

Total factored bending moment, $M_u = 1.3(0.239 + 1.67 \times 7.963) = 17.598 \text{ ft-kips per ft}$

Neglecting the reinforcement in the precast panel and providing mild steel reinforcement in the cast-in-place topping, this section can be conservatively designed as a conventionally reinforced concrete section.

Using #6 bars at 6 in. and 2.5 in. clear concrete cover, $A_s = 0.88 \text{ sq in.}$, $d = 5.625 \text{ in.}$, $a = 1.29 \text{ in.}$, $\phi M_n = 19.721 \text{ ft-kips}$
 $> 17.598 \text{ ft-kips per ft (ok)}$

Design of negative moment section over exterior girder line:

This section is checked against two cases: (1) under a combination of dead and live loads and (2) under a combination of collision force and dead loads. Neglecting the reinforcement in the precast panel and providing mild steel reinforcement in the cast-in-place topping, this section can be conservatively designed as a conventionally reinforced concrete section. Because the design of this section depends on the geometry of the barrier, no calculations are provided.

Metric Conversion Factors

1 in. = 25.4 mm; 1 sq in. = 645.2 mm²; 1 cu in. = 16387 mm³;
1 in.⁴ = 416231 mm⁴; 1 sq in. per ft = 2117 mm²/m; 1 cu in. per ft = 53763 mm³/m; 1 psi = 0.006895 MPa; 1 lb per sq ft = 0.04788 kPa; 1 lb per cu ft = 16.02 kg/m³; 1 ft-kip = 1.356 kN-m; 1 kip per ft = 1486 kg/m; 1 ft-kip per ft = 4.448 kN-m/m;