Special Report

Precast Prestressed Concrete Bridge Deck Panels

Prepared by

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The purpose of this report is to present guidelines for the design, manufacture and erection of precast prestressed concrete bridge deck panels. These guidelines are based on the latest research and practical experience gained from their use in various parts of the United States.

After a brief review of the history of bridge deck panels, suggestions are given for the dimensioning and detailing of panels, techniques for casting panels and helpful ideas in the handling, shipping and erection of panels.

An Appendix section provides a fully worked design example of a bridge deck panel meeting AASHTO Specifications. In addition, a large variety of bridge deck panel applications in pictorial form are included.

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1. INTRODUCTION

Precast prestressed concrete deck panels are widely used in the construction of bridges in the United States. A 1982 survey of the State Highway Departments by the PCI Bridge Committee showed that 21 states use the panels regularly and another seven states were trying the method through bidding options or were developing details prior to trial projects.

The panels are used as a composite part of the completed deck. They replace the main bottom (positive moment) transverse deck reinforcement and also serve as a form surface for the cast-in-place concrete upper layer that contains the top of deck (negative moment) reinforcement. A typical deck panel detail taken from the Illinois DOT design standards is shown in Fig. 1.

The use of precast panels has proven to be both economical and convenient.[1] Generally, when a deck is cast in place for its full depth, timber forms must be installed and later removed. This is expensive, time consuming and in many locations causes safety concerns to roadway traffic or pedestrians under the construction. On high level crossings, placing and removing deck slab forms is a safety concern to the workers.

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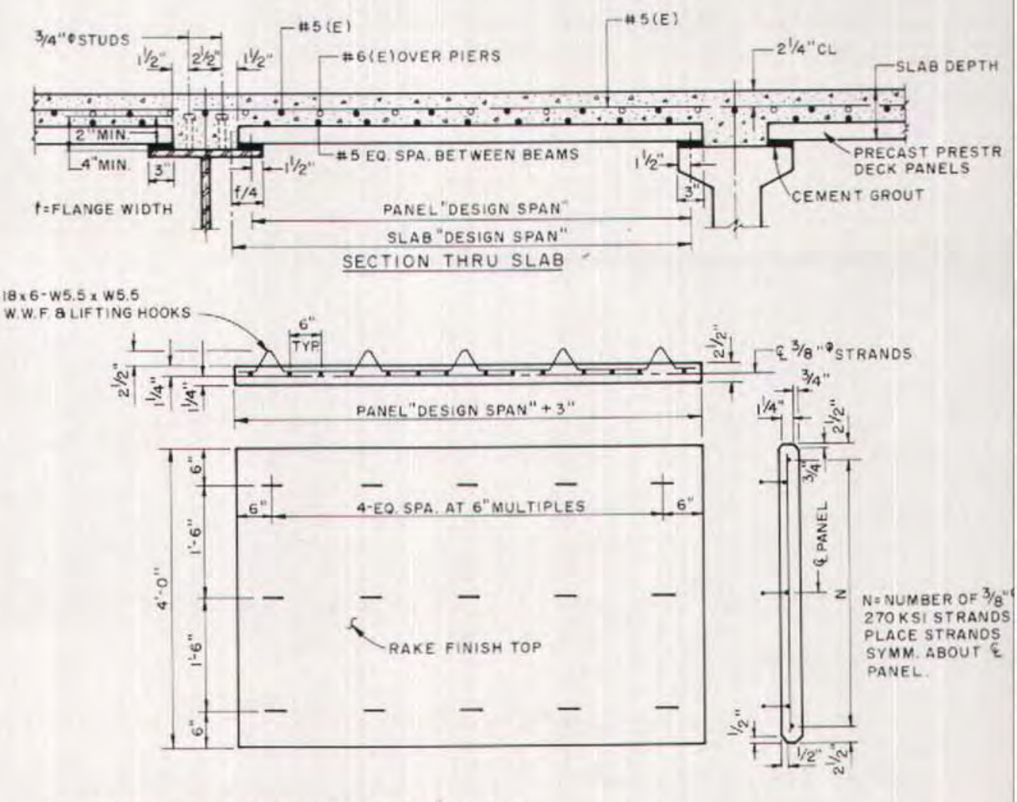
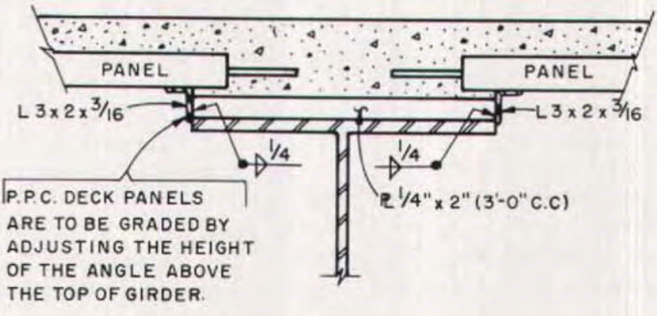


Fig. 1. Precast prestressed deck panel details.

Fig. 2. Welded steel plate girder stream crossing in eastern Kentucky used the detail shown below. The steel frames are fabricated on site after determining the height from top of flange (bottom of 2 in. bar) to bottom of deck (top of angle).



the Appendix. Included also are a large variety of applications of deck panels in pictorial form. First, however, a brief historical review of bridge deck panels is given.

2. HISTORY

Panels were first used in the nineteen fifties and a 1956 project on the Illinois Tollway is often cited as an early example. A number of states began to use the panels in the late sixties and early seventies. Several research projects were undertaken at that time. The research results were positive and current design is based on this work and the earlier designs that have functioned well.

The earliest installations of the deck panels are nearing 30 years of age and the decks continue to perform well.[2] Some bridge decks using panels have experienced reflective cracking in the bridge deck surface, directly above the butt joints between the prestressed panels.[3,4] These cracks normally terminate approximately halfway through the cast-in-place concrete slab, and have not been found to have any significant effect on the performance of the structure.

In fact, the cracking is often less severe than that found in carefully surveyed fully cast-in-place decks. Cracking problems related to nonrigid bearing of the panels on the longitudinal beams when positive bearing is not provided are a serious concern and the structural integrity of the slabs has been questioned.[4] This type of cracking has not been a problem in panels properly detailed with a grout or concrete bearing on the flange of the supporting beam.

3. DETAILS

A crucial aspect of the performance of panels has proven to be their detailing, especially member dimensioning, type of reinforcement and joint details. The transverse positive moment reinforcement in the deck is provided by pretensioning strands at mid-depth of the panels. Questions of strand development in the shorter panel lengths, appropriate design procedures and evaluation of wheel load distribution across the open transverse panel joints were answered positively in the early research.[5-8]

This research also indicated that chloride penetration (through the topping to the panel) did not occur. Corrosion protection of the strand is provided by the high quality concrete used in the panels.

Panel dimensions have varied, but practical ranges have been found to be:

Thickness: 2½ to 4½ in.

Width: 4 ft or 8 ft

Length: To provide a minimum of 3 in. of bearing on the beam flanges.

Some designers believe that the minimum thickness of panels should be 3½ in. in order to be in conformance with AASHTO Section 9.25.1.1 which requires 1½ in. clear cover. (The 2½ in. panel is based on 1 in. cover as stated in Section 9.25.1.2.2.) Also, the 3½ in. thickness would reduce the possibility of cracking in the panels due to handling and the longitudinal splitting at strands near panel edges.

The early panel projects used a detail with the end of strand extended outside the panel for 4 to 6 in. Research has shown that this extension is not required.[9] In the transverse joints, butt ends have proven to be a straightforward and durable solution to adequately transfer the wheel loads.

Research has also demonstrated that mechanical shear ties on the top of the panels are not required. Care in roughening during fabrication and providing a clean surface prior to placing the top course of concrete is necessary.

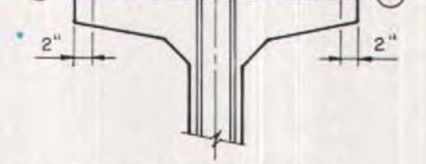
Recent reports on deck cracking[4] and good experience with other decks have demonstrated that the panels must be firmly bedded on grout or concrete on the longitudinal beams. A number of satisfactory details have been used that provide elevation adjustment between the top of the longitudinal beam and the panel and also provide grout space. Three of these are shown in Figs. 2, 3, and 4.

The strands in the panels are generally ⅜ in. diameter, but ⁷⁄₁₆ and ½ in. strands have been used in several states. Caution is advised when using strands larger than ⁷⁄₁₆ in. The strands should be centered in the depth of the panel and be spaced uniformly across the width. This will help prevent vertical bowing of the panel prior to placing in the structure. Most highway design groups have developed their own details and specifications. Many bid the cast-in-place deck and allow the contractor to use the panel system, based on standard drawings, as an alternate. The Illinois DOT standard (Fig. 1) is fairly typical although a number of details are handled differently from state to state. IDOT uses a variable depth grout bed.

Fig. 3. This view of the Colfax Bridge over the Saint Joseph River in South Bend, Indiana shows the diaphragms cast in place. The panels are stored on the beams to speed the final placement. These panels were placed on a fibrous material. An effective detail, shown below, is being used in a number of states, with good results.

(1) HIGH-DENSITY EXPANDED POLYSTYRENE FOAM BEDDING MATERIAL AND SHALL CONFORM TO THE FOLLOWING SPEC'S. : COMPRESSION STRENGTH 60 P.S.I. MIN., WATER ABSORPTION 0.125# FT.^{2} MAX. AND OXYGEN INDEX 24 MIN. AND SHALL BE IN ACCORDANCE WITH ASTM D-1621 (MIL-P-19644C).

(2) GROUT TO BE PLACED PRIOR TO PLACEMENT OF PANELS.



PANEL SUPPORT DETAIL

Welded wire fabric is used for longitudinal distribution steel and can also serve as lifting loops.

The reinforcement in the panels, placed perpendicular to the prestressing strands, is required for load transfer and strand development. Typical details are shown in Fig. 1 and are specified in AASHTO Section 9.23.2. The cast-in-place top course contains transverse negative steel over the longitudinal beams and longitudinal steel should be provided in accordance with AASHTO Section 9.18.2.2.

4. CASTING PANELS

Panels are cast on standard long line beds. The panel's top surface is roughened to provide mechanical bonding with the cast-in-place topping.

The purpose of this report is to present guidelines for the design, manufacture and erection of precast prestressed concrete bridge deck panels. These guidelines are based on the latest research and practical experience gained from their use in various parts of the United States.

After a brief review of the history of bridge deck panels, suggestions are given for the dimensioning and detailing of panels, techniques for casting panels and helpful ideas in the handling, shipping and erection of panels.

An Appendix section provides a fully worked design example of a bridge deck panel meeting AASHTO Specifications. In addition, a large variety of bridge deck panel applications in pictorial form are included.

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6. CONCLUSION

Use of precast prestressed concrete deck panels results in an economical deck system and fast deck construction.[10] Proper design and detailing are important, and, if properly executed, will result in high quality decks with good durability.

In a continuing effort to improve the industry's products, the Bridge Producers Committee of the Prestressed Concrete Institute has commissioned a consulting engineer to compile research data, study current design and detailing practices and investigate production procedures (including tolerances). The end result will be a "Recommended Practice for Bridge Deck Panels."

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9. Bieschke, L. A., and Klingner, R. E., "The Effect of Transverse Strand Extensions on the Behavior of Precast Prestressed Panel Bridges," Center for Transportation Research-Report 303-1F for Texas Highway Department.
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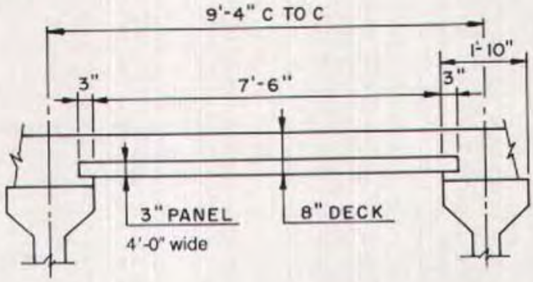
METRIC (SI) CONVERSION FACTORS

1 ft = 0.305 m	1 in.^{4} = 416,231 mm^4
1 in. = 25.4 mm	1 lb-in. = 0.1130 N-m
1 psf = 4.882 kg/m^2	1 kip = 4448 N
1 pcf = 16.02 kg/m^3	1 lb = 4,448 N
1 in.^{2} = 645.2 mm^2	1 psi = 0.006895 MPa
1 in.^{3} = 16,387 mm^3	1 ksi = 6.895 MPa

APPENDIX A — BRIDGE DECK PANEL DESIGN

Design Conditions



Design span:
$L = 7.50 + 2\,(0.25/2)$
$= 7.75$ ft for panel alone
$L' = 7.50$ ft for composite section
(AASHTO 3.24.1.2a)
Reference: AASHTO Specifications for Highway Bridges, 1983.

AASHTO notation will be used throughout the design example.
Loading: 35 psf future wearing surface; HS 20-44 truck; 50 psf working load during deck casting. These loads are in addition to the cast-in-place slab weight.

Material Properties

Precast concrete:
$f'_c = 5000$ psi; $f'_{ci} = 4000$ psi
Deck concrete: $f'_c = 4000$ psi

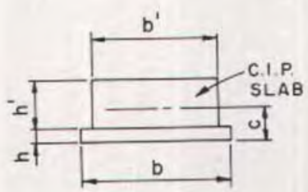
All concrete: 150 pcf
Stress-relieved strand: $f'_s = 270{,}000$ psi
Reinforcing bar: $f'_{sy} = 60{,}000$ psi

Allowable Stresses (AASHTO)

Type of stress	Temporary stresses	Final stresses
Compression	$0.6f'_{ci} = +2400$ psi	$0.4f'_c = +2000$ psi
Tension	$-3\sqrt{f'_{ci}} = -190$ psi	$-3\sqrt{f'_c} = -212$ psi (corrosive conditions)
Tension	—	$-6\sqrt{f'_c} = -424$ psi (normal ambient conditions)

Strand at release $= 0.7f'_s = 189{,}000$ psi

Section Properties



$b = 12$ in. (unit width)
$b' = (E_s/E_p) \times 12$
$= (\sqrt{4000}/\sqrt{5000}) \times 12$
$= 10.73$ in.
$h = 3$ in.
$h' = 5$ in.
Panel:
$I = bh^3/12$

$I = 12(3)^3/12$
$= 27$ in.^{4}
$S_{tp} = I/c_t$
$= 27/1.5$
$= 18$ in.^{3}
$S_b = I/c_b$
$= 27/1.5$
$= 18$ in.^{3}

Composite deck:

	ΣA	y	ΣAy	z	Az^2	I
Cast in place	53.7	5.5	295	1.61	139	112
Precast	36.0	1.5	54	2.39	206	27
Total	89.7	—	349	—	345	139

Composite:

$c = \Sigma Ay/\Sigma A$
$= 349/89.7$
$= 3.89$ in.
$I_c = Az^2 + I$
$= 345 + 139$
$= 484$ in.^{4}
$S_t = (I_c/c_t)(\sqrt{5}/\sqrt{4})$
$= (484/4.11) \times 1.12$
$= 132$ in.^{3}
$S_{tp} = I_c/c_{tp}$
$= 484/0.89$
$= 543$ in.^{3}
$S_b = I_c/c_b$
$= 484/3.89$
$= 124$ in.^{3}

Moments and Stresses

Dead load (panels)
$M = 12\,w\,L^2/8$
$= 12\,(37.5)\,(7.75)^2/8$
$= 3378$ lb-in.
$f_{tp} = M/S_{tp}$
$= 3378/18$
$= 188$ psi
$f_b = M/S_b$
$= -3378/18$
$= -188$ psi

Dead load (slab)
$M = 12\,w\,L^2/8$
$= 12\,(62.5)\,(7.75)^2/8$
$= 5630$ lb-in.
$f_{tp} = M/S_{tp}$
$= 5630/18$
$= 313$ psi
$f_b = M/S_b$
$= -5630/18$
$= -313$ psi

Live load (during deck casting)
$M = 12\,w\,L^2/8$
$= 12\,(50)\,(7.75)^2/8$
$= 4505$ lb-in.
$f_t = M/S_{tp}$
$= 4505/18$
$= 250$ psi
$f_b = M/S_b$
$= -4505/18$
$= -250$ psi

Dead load (FWS/overlay)
$M = 12 \times 0.1\,(w)\,(L')^2$
$= 12 \times 0.1\,(35)\,(7.5)^2$
$= 2363$ lb-in.
$f_t = M/S_t$
$= 2363/132$
$= 18$ psi
$f_{tp} = M/S_{tp}$
$= -2363/543$
$= -4$ psi
$f_b = M/S_{bc}$
$= -2363/124$
$= -19$ psi

Live load plus impact
(Distribution Factor)
D.F. $= (S + 2)/32$
$= (7.5 + 2)/32$
$= 0.297$ (see AASHTO Section 3.24.3.1)

Impact (I) = 30 percent IF = 1.3
Continuity Factor (CF) = 0.80
(AASHTO Section 3.24.3.1)

$M =$ (DF)(IF)(CF)(wheel load)
$= (0.297)(1.3)(0.8)(16{,}000)(12)$
$= 59{,}280$ lb-in.
$f_t = M/S_t$
$= 59{,}280/132$
$= 449$ psi
$f_{tp} = M/S_{tp}$
$= 59{,}280/543$
$= -109$ psi
$f_b = M/S_b$
$= 59{,}280/124$
$= -478$ psi

Summation: Governing bottom tension in panel: $-188 - 313 - 19 - 478 = -998$ psi.

Prestress

Estimate the number of strands required:
Use 3/8 in. diameter stress relieved strands.

$$P_f = 0.8 \times 0.70 \times 270 \times 0.85$$
$$= 12.8 \text{ kips per strand}$$

Stress reduction required:
Load stresses — Allowable stresses
$= -998 - (-212) = -786$ psi

Strands will be centered in the depth of the panel and will, therefore, only be subject to P/A stresses.
Estimate the number of strands for stress:

$$\frac{P_{total}}{A} = 786 \text{ psi} = \frac{\text{No. of strands} \times 12{,}800}{12 \times 3}$$

No. of strands = 2.21 per ft of width, for stress or 2.21 x 4 ft = 8.84 (say, 10 strands per panel):

Consider ultimate:
Required moment:

$$M_u = 1.3\,[M_d + 1.67\,(M_{LL+I})]$$
$$= 1.3\,[3378 + 5630 + 2363 + 1.67\,(59{,}280)]$$
$$= 143{,}480 \text{ lb-in.}$$

$$M_u = A_s^* f_{su}^* d\,(1 - 0.6\,p^* f_{su}/f_c') \quad \text{[AASHTO Eq. 9-13]}$$

$$f_{su}^* = f_s'(1 - 0.5\,p^* f_s'/f_c') \quad \text{[AASHTO Eq. 9-17]}$$

$$p^* = A_s^*/(bd)$$

Check A^* = 10 strands per 4 ft width or $10 \times 0.085/4 = 0.2125$ in.^{2} per ft.

$$p^* = 0.2125/(12 \times 6.5)$$
$$= 0.0027$$
$$f_{su}^* = 270\,[1 - 0.5\,(0.0027 \times 270)/4]$$
$$= 245 \text{ ksi}$$

Check reduction of f_{su}^* due to limited development length:

$$(f_{su}^*)_{max} = \frac{l_x}{D} + \tfrac{2}{3} f_{se} \quad \text{(AASHTO Eq. 9-19)}$$
$$= \frac{96}{2 \times 0.375} + \frac{2 \times 151}{3}$$
$$= 229 \text{ ksi} < 245 \text{ ksi}$$

$$M_u = 0.2125 \times 229 \times 6.5\,(1 - 0.6 \times 0.0027 \times 229/4)$$
$$= 287 \text{ kip-in. or } 287{,}000 \text{ lb-in.} > 143{,}480 \text{ lb-in.}$$

Consider maximum and minimum steel:
Maximum:
$p^* f_{su}^*/f_c' \le 0.30$ (AASHTO Eq. 9-20)
$0.0027 \times 229/4 = 0.155 < 0.3$
Minimum:
$M_u > 1.2\,M_{cr}$ (AASHTO Section 9.18.2.1)
$1.2\,M_{cr} = 1.2\,[(f_{cr} + f_p)\,S_b + M_{dl}]$
where f_p is the prestress minus dead load acting on panel.

$$1.2\,M_{cr} = 1.2\,[(7.5\,\sqrt{5000} + \frac{10 \times 12{,}800}{3 \times 48} - 188 - 313)\,124 + 5630 + 3378]$$
$$= 147{,}400 \text{ lb-in.} < 287{,}000 \text{ lb-in.}$$

Calculate prestress losses (see AASHTO Section 9.16.2):

$$f_s = SH + ES + CR_c + CR_s$$
$$SH = 17{,}000 - 150\,RH$$

Assume a midwest location in which RH = 70.

$$SH = 17{,}000 - 150 \times 70$$
$$= 6500 \text{ psi}$$
$$ES = (E_s/E_{ci})\,f_{cir}$$

Assuming 10 percent prestress losses at release:

$$f_{cir} = \frac{0.7 \times 0.9 \times 270 \times 10 \times 0.085}{48 \times 3}$$
$$= 1.000 \text{ ksi or } 1000 \text{ psi}$$
$$ES = \frac{28 \times 10^6 \times 1000}{33 \times 150^{1.5} \times \sqrt{4000}}$$
$$= 7330 \text{ psi}$$
$$CR_c = 12 f_{cir} - 7 f_{cds}$$
$$f_{cds} = 0 + \frac{-4 - 19}{2}$$
$$= -11 \text{ psi}$$
$$CR_c = 12 \times 1000 - 7 \times 11$$
$$= 11{,}920 \text{ psi}$$
$$CR_s = 20{,}000 - 0.4\,ES - 0.2\,(SH + CR_c)$$
$$= 20{,}000 - 0.4 \times 7330 - 0.2\,(6500 + 11{,}920)$$
$$= 13{,}380 \text{ psi}$$
$$f_s = 6500 + 7330 + 11{,}920 + 13{,}380$$
$$= 39{,}130 \text{ psi}$$
$$f_{se} = 0.7 \times 270 - 39$$
$$= 150 \text{ ksi}$$

Percentage of prestress losses:

$$\frac{39{,}130 \times 100}{0.7 \times 270{,}000} = 20.7 \text{ percent}$$

which is about 20 percent losses. Therefore, there is no need to recalculate losses.

$$P_f = (1 - 0.207) \times 0.7 \times 270 \times 0.085$$
$$= 12.7 \text{ kips per strand}$$
$$f_{pe} = \frac{12.7 \times 10}{3 \times 48}$$
$$= 0.882 \text{ ksi or } 882 \text{ psi}$$

Check stresses (all in psi):

	At release (10 percent losses)		Place deck		Final		
Load	f_{tp}	f_b	f_{tp}	f_b	f_{tp}	f_b	f_t
P_s/A	1000	1000	882	882	882	882	
M_{panel}	188	−188	188	−188	188	−188	
	1188	812					
Allowable:	2400	2400					
M_{slab}			313	−313	313	−313	
$M_{LL\ on\ slab}$			250	−250			
			1633	−131			
		Allowable:	2000	−212			
M_{fwe}					−4	−19	18
M_{LL+I}					−109	−478	449
					1270	−116	467
		Allowable:			2000	−212	1600

Note: tension in the bottom of the cast-in-place concrete deck will be only −109 − 4 = −113 psi which is negligible.

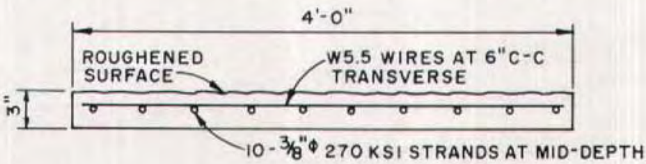
Distribution reinforcement in panel:

$p = 0.11$ sq in. per ft width (AASHTO Section 9.23.2)

Use W5.5 wires at 6 in. spacing equaling 0.11 sq in. per ft of welded wire fabric.

Summary

The resulting configuration and reinforcement of the panel is shown below.



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APPENDIX B — APPLICATIONS

Figs. B1 to B5 show how precast prestressed concrete panels were used in 1982 on the Jefferson Barracks Bridge (1-55) over the Mississippi River in East St. Louis. The project required 130,000 sq ft of 3 in. deep panels 4 ft wide with lengths of 8 ft 2 in. to 8 ft 7 in.

Fig. B1. Looking west, the high density expanded polystyrene foam (EPS) on 10 ft deep steel plate girders can be seen. Removing wood forms, over the water in this situation, would have been difficult.

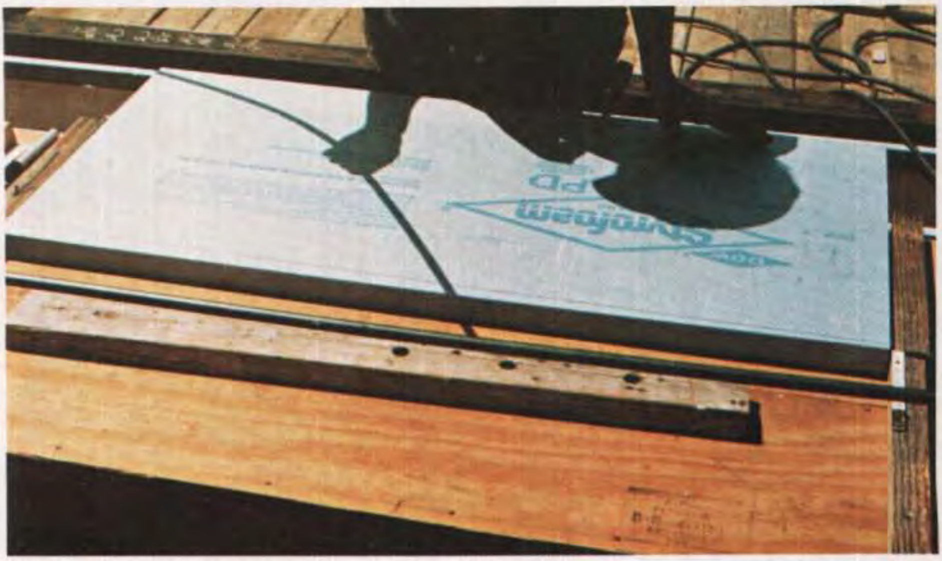

Fig. B2. The EPS being hot-wire cut to the correct fillet height before it is attached to the top flange.

Fig. B3. The panels supported on the EPS, before placing the deck concrete.

Fig. B4. The blockout for a scupper.

Fig. B5. The deck panels and steel shear connectors in place ready for reinforcing steel and deck concrete.

Fig. B6. On this eleven span steel plate girder bridge over a railroad in St. Clair County, Illinois, the narrow flange did not allow the use of EPS. PVC pipes were cut to fillet height and glued to the flange. 2 x 4 in. and 2 x 6 in. boards were wedged (not nailed) against the side of the flange to hold the grout bed in place after panels were set. Stripping was simple. Hooks with long handles were used from the ground to pull out the bowed transverse wood wedges.

Fig. B7. A three span wide flange steel beam bridge over a stream near Peoria, Illinois. The panels were made rectangular and cut in the field to fit the skew.

Fig. B8. An overall view of the same structure shown in Fig. B7 (above). The panels are in place ready for steel shear connectors, reinforcing steel and deck concrete.

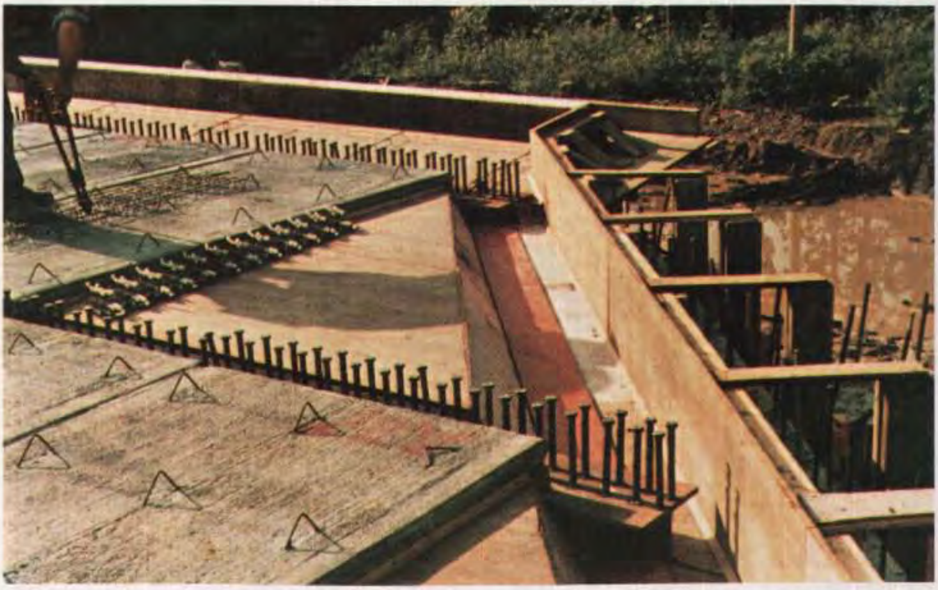

Fig. B9. Details at skewed end of a 33 WF steel beam three span bridge in Bureau County, Illinois. The 45 degree skewed end section was formed and poured full depth in the field.

Fig. B10. Placing panels at steel beam splice where fillets are small. Notice that small bolt head must be on top of flange plates. Common ironworker practice is to put bolt head on underside of top flange.

Fig. B11. An 18 span 66 ft wide steel plate girder high level structure at Lemont, Illinois crosses a highway, a ship canal and a railroad. This structure required 190,000 sq ft of 2½ in. thick x 48 in. wide panels. Because of curve at each end, the panels were bevelled on each side. The small crane set the panels after a large crane unloaded the trucks below.

Fig. B12. Placing 2½ in. thick panels on a 42 in. precast prestressed concrete I beam bridge in central Illinois. The EPS form was cut to the required fillet height. It is generally advisable to place grout while placing panels to avoid air pockets.

Fig. B13. The panels and the EPS are in place on the structure shown in Fig. B12.

Fig. B14. The workman is placing grout ahead of the panel setting.

Fig. B15. Placing panels on a precast prestressed concrete I beam structure. Note how a group of panels are unloaded from trucks and stored on the beams before lifting each one into place.

Fig. B16. 3½ in. thick panels on a steel plate girder bridge near New Orleans, Louisiana. These are set on steel angles prefabricated to proper fillet height shown in Fig. B2.