Full Depth Precast and Precast, Prestressed Concrete Bridge Deck Panels

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Since 1970, there has been a dramatic increase in the use of full depth precast and precast, prestressed concrete deck panels for the rehabilitation and new construction of bridges. In order to document the various applications of bridge deck panels in North America, a questionnaire survey was sent to the departments of transportation (DOTs) of individual states and one province. Requested information in the survey included type of construction, deck dimensions, deck supporting system, panel dimensions and reinforcement, type of connecting system between panels and supporting system, type of joint between the adjacent panels, type of bonding material used to fill the joints, problems associated with the joints, reasons for adverse results, and type of protection system. The returned questionnaires were compiled and analyzed. This paper summarizes the significant results of this survey.

The dramatic increase in the rehabilitation of existing steel or concrete girder bridges on major highways with high traffic volumes has focused attention on the use of precast concrete deck panels to shorten the time of reconstruction and reduce the time of bridge closures. In selecting a bridge deck rehabilitation system, the following criteria must be adequately addressed:

1. Strength and serviceability
2. Performance and durability
3. Rapid construction
4. Minimum interference with traffic
5. Minimum maintenance requirements

Fig. 1. View of system under construction.

Fig. 2. Transverse joint between precast panels during construction.

Table 1. Departments of transportation (DOTs) response summary.

Number of surveys mailed out to DOTs	53
Number of DOTs responding by mail	38
Number of DOTs responding by telephone	13
Total number of DOTs responding	51
Number of DOTs reporting using this system	13
Number of DOTs reporting not using this system	38
Number of DOTs interested in using this system	43
Number of DOTs not interested in using this system	8

Table 2. Type of construction.

Department of Transportation	Number of bridges	Type of construction			
		Precast panels	Precast, prestressed panels	Rehabilitation	New
Alaska	19	—	✓	December 1991 October 1992	—
Maine	1-5	✓	—	May 1987 October 1987	—
Indiana	2	—	✓	—	1970
Washington	> 5	—	✓	—	1986
California	1-5	✓	—	✓	✓
Iowa	1-5	✓	—	—	Spring 1992 Fall 1992
Connecticut	1-5	—	✓	1990	—
Maryland	2	—	✓	1983	—
New York	12	✓	✓	1972, 1991	
Ohio	1-5	✓	—	1986	—
Illinois	2	—	✓	1986, 1987	—
Ontario, Canada	1-5	✓	—	1990	—
Texas	1	✓	—	1985	—
Virginia	2	✓	—	1985	—

Table 3. Condition of joints.

Department of Transportation		Alaska	Maine	Indiana	Washington	Iowa	Connecticut	Illinois	California	Maryland	Virginia
Age (years)		1½	4½	24	7	Under construction	4	7	15	10	8
Problems associated with joints	Cracking	None reported	✓	—	None reported		None reported	—	✓	✓	—
	Leaking		✓	✓				✓	✓	✓	—
	Settlement		—	—				—	—	—	—
	Deterioration		—	—				—	✓	✓	✓
Reasons for adverse results	Material quality		✓	—				—	✓	—	✓
	Maintenance		—	—				—	—	—	✓
	Construction procedures		✓	✓				✓	✓	✓	—

Table 4. Type of protection systems.

Department of Transportation	Protection systems
Alaska	None
Maine	¼ in. epoxy waterproofing overlay (did not work properly)
Indiana	None
Washington	Waterproofing membrane system with Class I (bituminous surfacing)
Iowa	Low-slump dense concrete (LSDC) 2 in.
Connecticut	Waterproofing membrane system with Class I
New York	Waterproofing membrane system with Class I
Ohio	Waterproofing membrane system with Class I
Ontario, Canada	Waterproofing membrane system with Class I
Illinois	2 in. minimum Class I with waterproofing membrane system
California	Epoxy concrete overlay
Maryland	2 in. latex modified concrete
Texas	Not reported
Virginia	Waterproofing membrane system with Class I and II

Note: 1 in. = 25.4 mm.

With advancements in precasting technology, there has been a justifiable trend towards the application of low cost, full depth precast concrete bridge deck replacement as an alternative to conventional bridge deck systems. In this type of construction, the entire bridge deck is constructed of precast concrete. There is no additional field-cast concrete acting structurally, except that used in the connections and slab closures.

A view of the system is shown in Fig. 1, where the precast concrete panels are being placed on the existing supporting elements. Fig. 2 shows the transverse joint between adjacent precast panels along with the openings for the shear connector pockets.

The advantages of this system can be summarized as follows:[1-5]

1. This type of construction is applicable to all requirements of repair or replacement, as well as for new bridge construction.

2. Prefabrication of components can significantly reduce the out-of-service time for bridge repair or replacement.

3. Cost savings are possible because of the reduction in the employed field labor and the reduction of the added expenses of the bridge users due to the decrease in traffic delays.

Prior to 1975, research was conducted on the use of precast, prestressed concrete bridge deck panels.[6] In addition, research also included testing for the feasibility of using these types of systems.[7]

In 1982, a survey conducted by the PCI Bridge Committee revealed that 21 states utilized precast, prestressed concrete bridge deck panels regularly while another seven states were starting to incorporate the method through bidding options or by developing details prior to trial projects.[8] In 1986, another survey was conducted by the PCI Bridge Producers Committee from which specifications governing the design of such systems were developed; as a result, a recommended practice for precast composite bridge deck panels was published.[9]

QUESTIONNAIRE SURVEY

At the start of this investigation there was no systematic method for identifying and evaluating the performance and durability of precast, prestressed concrete bridge decks, especially under severe environmental conditions. In order to identify the proposed bridge deck system in North America, a detailed questionnaire survey was sent to 53 departments of transportation (DOTs), transportation and thruway authorities and turnpikes, in the United States and Ontario, Canada.

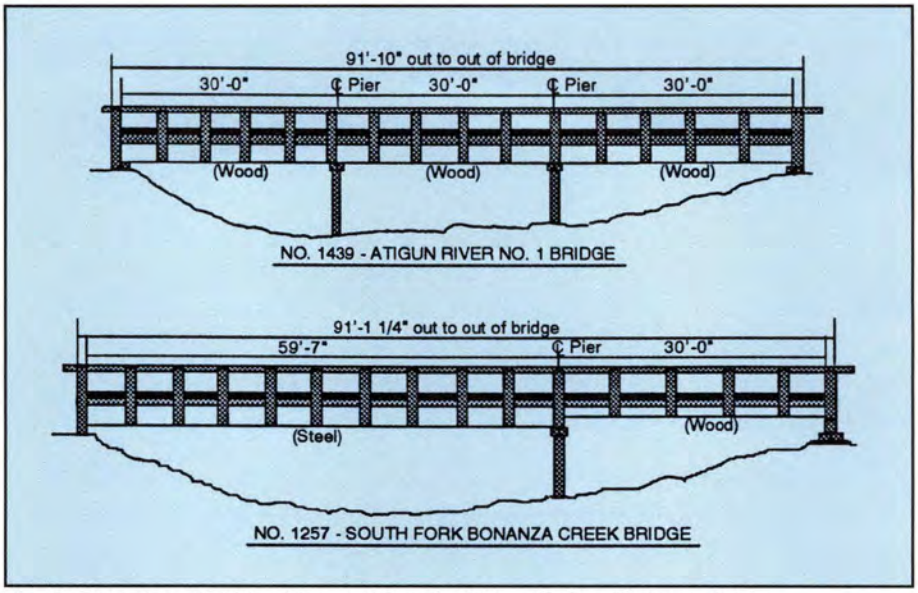

Fig. 3. Elevation of Bridge Nos. 1439 and 1257 — Dalton Highway Bridge rehabilitation.

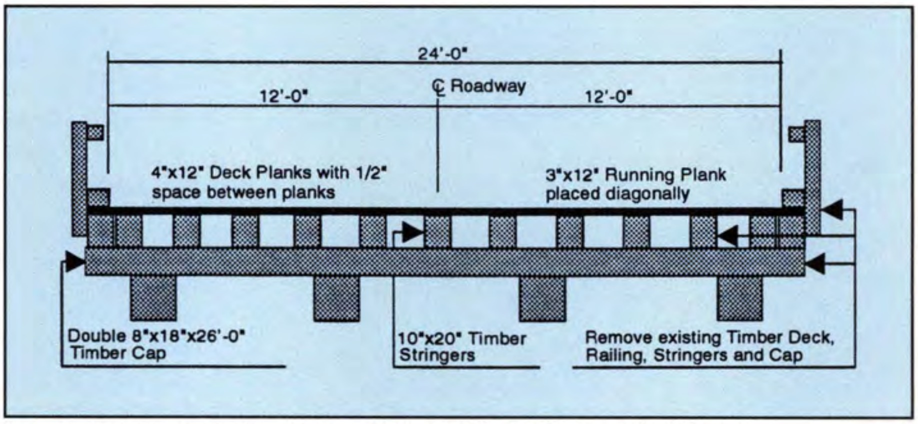

Fig. 4a. Existing typical section — Dalton Highway Bridge rehabilitation.

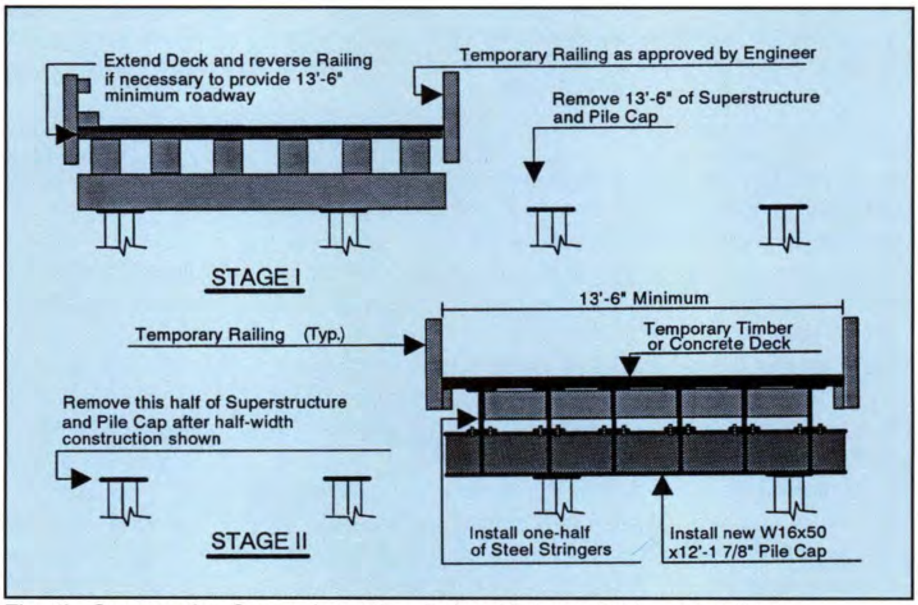

Fig. 4b. Construction Stages I and II — Dalton Highway Bridge rehabilitation.

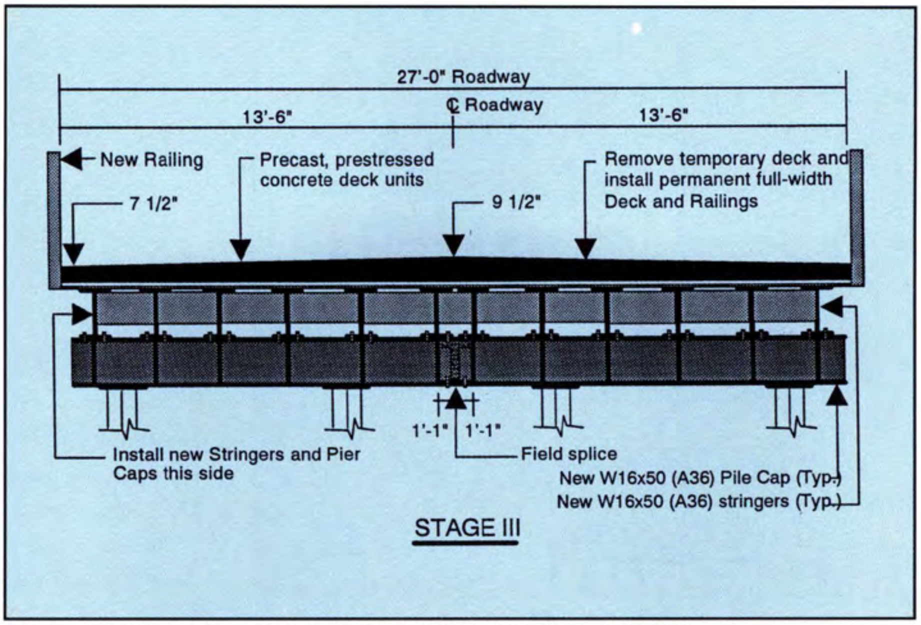

Fig. 4c. New typical section — Dalton Highway Bridge rehabilitation.

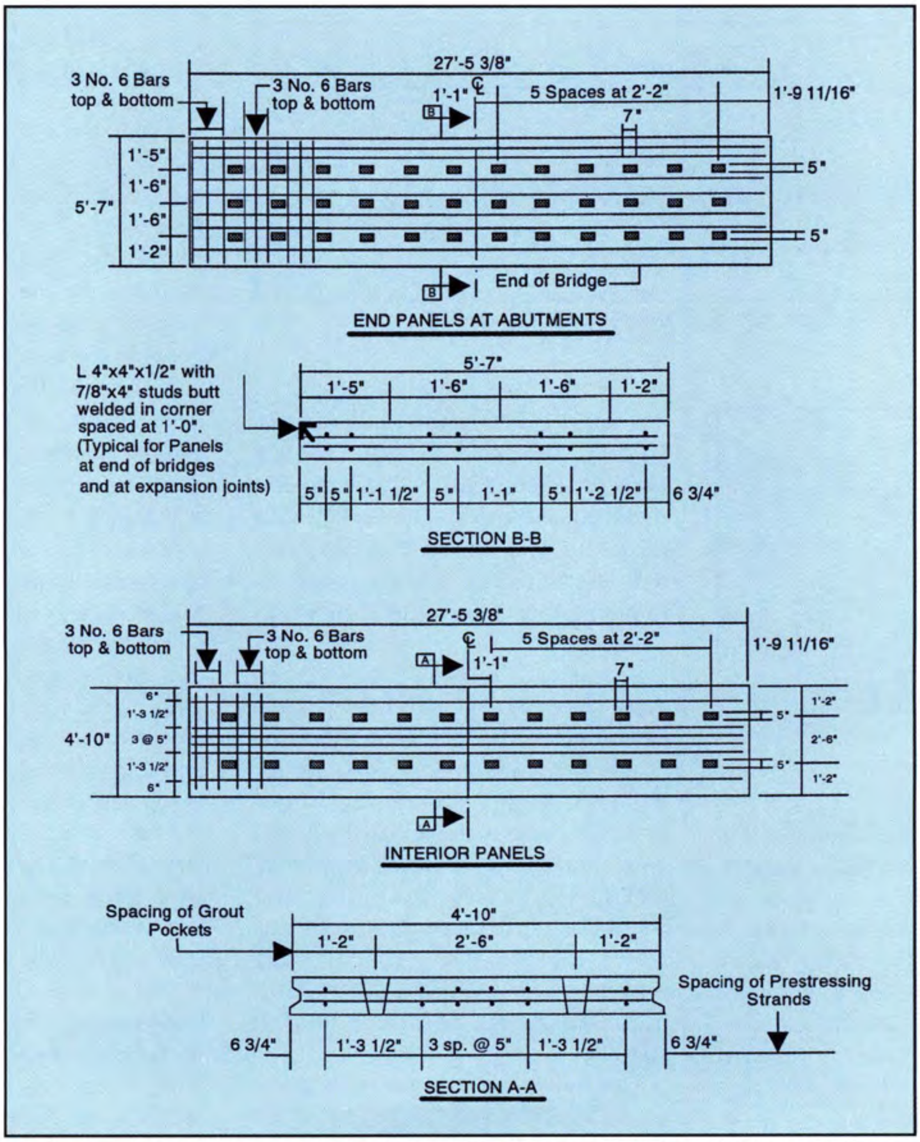

Fig. 5a. End and interior panels for 30 ft (9.14 m) spans — Dalton Highway Bridge rehabilitation.

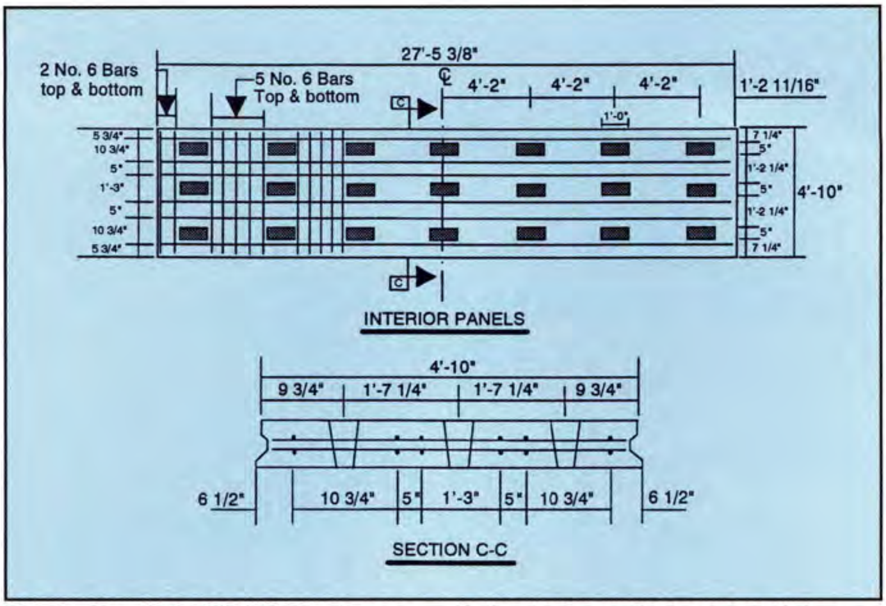

Fig. 5b. Panels for 60 ft (18.3 m) spans — Dalton Highway Bridge rehabilitation.

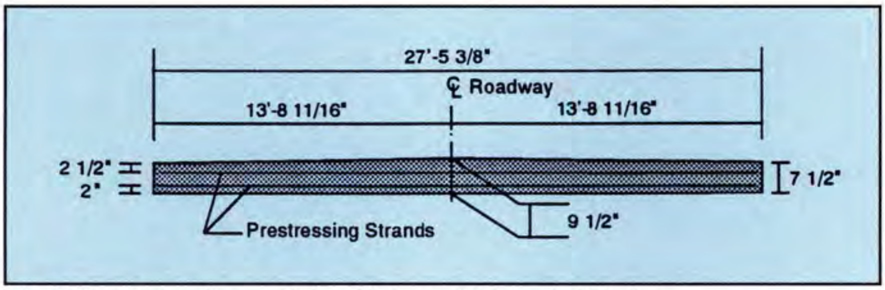

Fig. 6. Panel elevation — Dalton Highway Bridge rehabilitation.

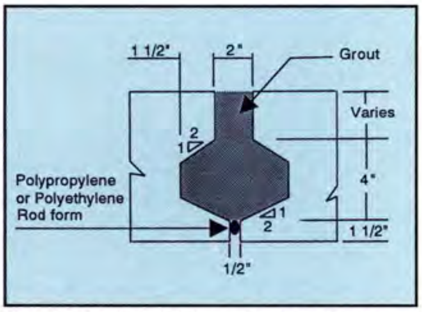

Fig. 7. Typical joint — Dalton Highway Bridge rehabilitation.

The objective of this investigation was to evaluate the stability, durability, and performance of the proposed bridge deck system exposed to harsh conditions. Over the years, many states have experimented with precast concrete slabs for deck replacements by offering a wide variety of design and construction methods. The first trials were started in the early 1970s by Indiana,[10] New York, and Alabama.[11] All these early applications had approximately the same characteristics. The spans did not have any skew or superelevation. More projects involved new construction rather than rehabilitation. The deck-stringer system was primarily noncomposite, although some composite action was observed.

Significant advances have been made since the mid-1970s through the beginning of the 1980s. Many of the spans were composite and some involved complex geometries. Major structures were built nationwide by the New York State Thruway Authority (NYSTA), the New York State Department of Transportation (NYSDOT), the Maryland State Highway Administration (MSHA), and the Illinois Department of Transportation (IDOT).

The following information was requested in the survey:

1. Type of construction and number of bridges built

2. Period of construction

3. Bridge dimensions and orientation

4. Deck supporting system

5. Deck dimensions and specifications

6. Panel dimensions and reinforcement

7. Concrete strength

8. Type of connecting system between panels and supporting system

9. Type of joints between adjacent panels

10. Type of bonding material used to fill the joints

11. Problems associated with the joints

12. Reasons for adverse results

13. Type of protection system

The returned questionnaires were compiled and analyzed. The summarized results are presented in the following text and tables.

SUMMARY OF SURVEY RESPONSES

Thirteen states, namely, Alaska, Maine, Indiana, Washington, California, Iowa, Connecticut, Maryland, New York, Ohio, Texas, Virginia, and Illinois, and Ontario, Canada, have reported using the precast, prestressed concrete bridge deck system. Table 1 presents the complete response of the departments, the number of departments that reported using this concept, and the number of departments interested in using this concept of construction in the future.

Some of the departments have used the precast, prestressed concrete bridge deck system for rehabilitation, including Alaska, Maine, Connecticut, Maryland, Ohio, Texas, Illinois, Virginia, and Ontario, Canada; some have used the system for new construction, including Indiana, Washington, and Iowa; and some have used the system for both rehabilitation and new construction, including California and New York. Table 2 presents the number of bridges constructed in each state and the type of construction.

The joints are important because bridge deck performance is manifested in the behavior of its joints. The survey focused on the problems associated with the joints and the reasons for the adverse results. Unfortunately, most of the departments have ignored

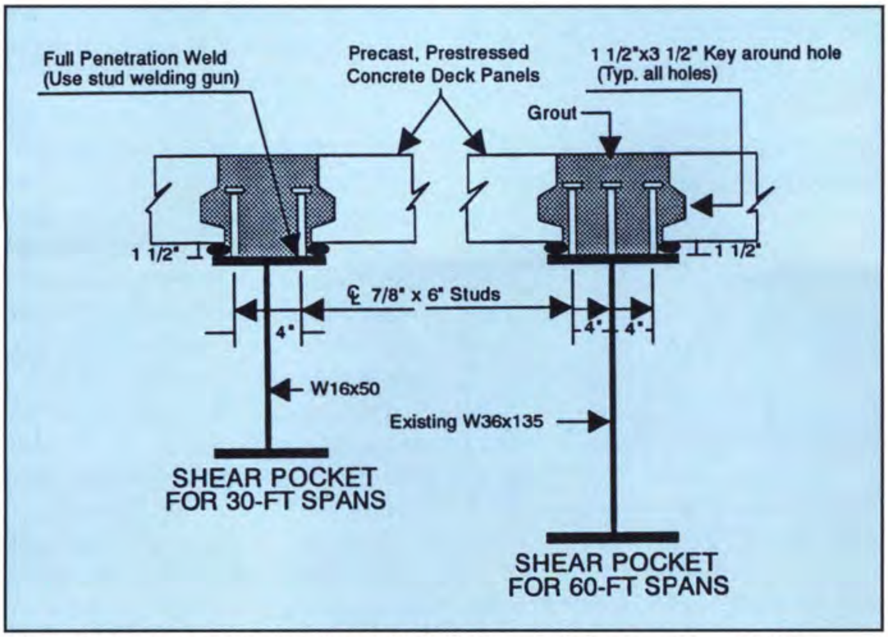

Fig. 8. Shear connectors for 30 and 60 ft (9.14 and 18.3 m) spans — Dalton Highway Bridge rehabilitation.

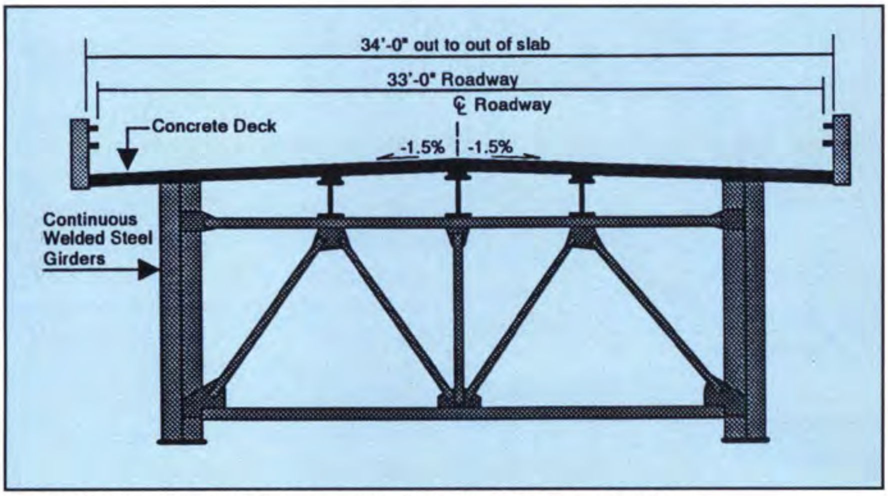

Fig. 9. Existing typical section — Chulitna River Bridge redecking.

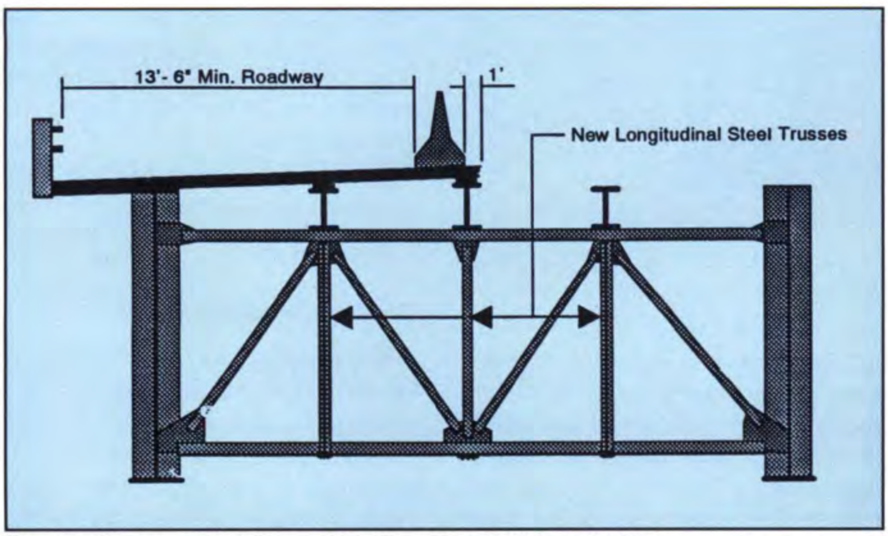

Fig. 10. Stage construction — Chulitna River Bridge redecking.

this portion of the survey due to lack of manpower for bridge investigation and maintenance. The condition of the joints is presented in Table 3. Material quality, construction procedures, and maintenance are the main reasons for adverse results, as demonstrated in Maine and Indiana.

Due to the harsh weather conditions in the northern regions, the survey also addressed the protection system. Few states have reported using a protection system and very few states reported that the protection system used is performing satisfactorily. Different types of protection systems were used. Maine has used a $^1/_4$ in. (6 mm) epoxy waterproofing overlay, yet, according to their report, this did not work properly. Washington and Connecticut reported using a waterproofing membrane system with Class I, and Iowa reported using low-slump dense concrete with a thickness of 2 in. (50 mm). The collected data are presented in Table 4.

Every department of transportation has used its own method of design and construction. The study introduced selected applications in a few states, such as Alaska, Maine, Indiana, Washington, California, Iowa, Connecticut, Maryland, New York, Ohio, Texas, Virginia, and Illinois, as well as Ontario, Canada. This paper will present the projects accomplished recently in these states.

ALASKA DOT AND PUBLIC FACILITIES

The Alaska rehabilitation program included two main projects that were completed in October 1992: the Dalton Highway Bridge rehabilitation and the Chulitna River Bridge redecking projects.

Dalton Highway Bridges

The first project contained 18 bridges in one contract. The existing bridges had timber decks supported on either steel stringers or timber floor beams, depending on the bridge span length. Steel stringers were used for spans of 60 ft (18 m) and timber stringers were used for spans of 30 ft (9 m), as shown in Fig. 3. The rehabilitation process re-

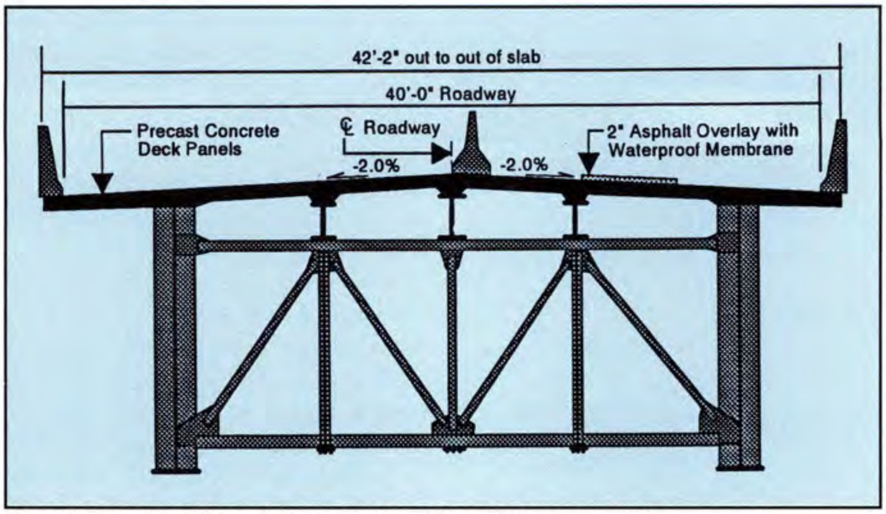

Fig. 11. New typical section — Chulitna River Bridge redecking.

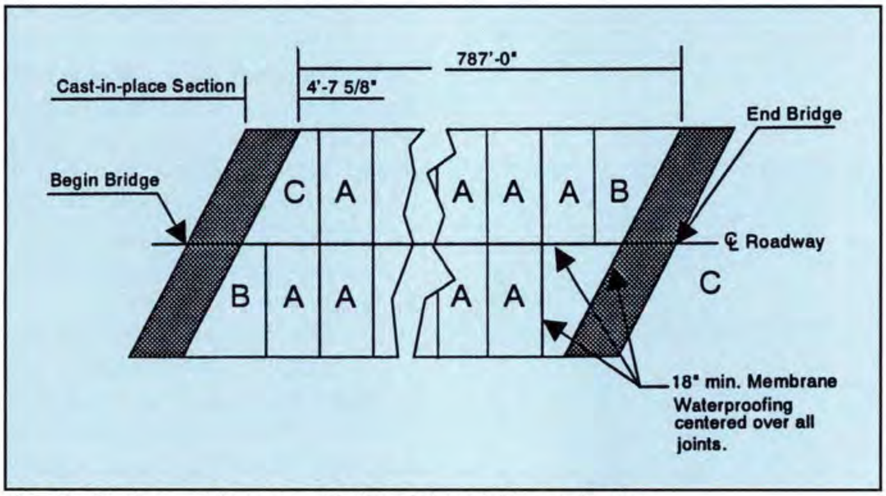

Fig. 12. Deck panel layout — Chulitna River Bridge redecking.

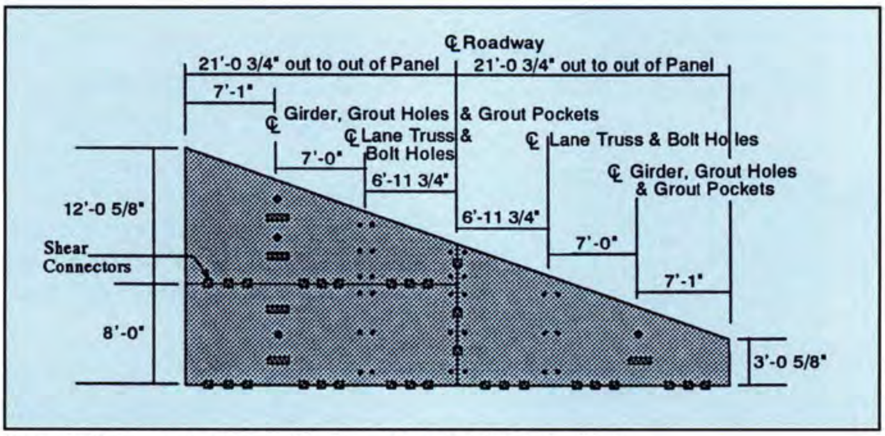

Fig. 13. Edge area plan — Chulitna River Bridge redecking.

moved the existing timber decks, railings, stringers, and pile caps, and installed permanent full width, full depth precast, prestressed concrete deck panels on new steel W shape (wide flange) stringers and pile caps.

Stage construction was adopted to maintain traffic flow during construction. The primary stage consisted of removing a half width of the superstructure and pile caps and installing a temporary railing at the free end. The new steel pile caps and stringers were installed and covered by a temporary timber or concrete deck. At the end of this stage, one-half of the bridge width was ready to support traffic flow.

The second stage was to work on the other half width, where new pile caps and stringers were installed. Finally, the temporary deck was removed, field splices between the separated pile caps were assembled, and permanent, full width precast, prestressed concrete panels were installed. The stage construction is shown in Figs. 4a, 4b, and 4c.

The difference in stringer sizes due to the difference in span lengths led to the use of various types of precast panels, as shown in Figs. 5a and 5b.

All precast panels were 9.5 in. (240 mm) thick at the centerline of the roadway and 7.5 in. (190 mm) thick at the edges, with one typical length of 27 ft 5$^3/_8$ in. (8.4 m) and two typical widths of 4 ft 10 in. and 5 ft 7 in. (1.5 and 1.7 m). Fig. 6 shows a typical panel elevation. Normal weight concrete was used with a strength of 5000 psi (34 MPa) at transfer and 6500 psi (45 MPa) at 28 days. The prestressing strands were $^1/_2$ in. (13 mm) diameter, seven wire, low relaxation strands with an ultimate strength of 270 ksi (1860 MPa). The jacking stress for the pretensioning strands was 189 ksi (1300 MPa) and the effective stress after all losses was 149 ksi (1030 MPa).

A typical female-to-female joint (see Fig. 7) was used between adjacent panels and an elastomeric compression joint seal was used in the expansion joint.

Two sizes of shear pockets were used. The first pocket size is 7 x 5 in. (180 x 130 mm) with two studs $^7/_8$ x 6 in. (20 x 150 mm) installed in each pocket for a 30 ft (9 m) span length. The second pocket size is 12 x 5 in. (300 x 130 mm) with three studs of the same size installed for a 60 ft (18 m) span length, as shown in Fig. 8.

Chulitna River Bridge

The second project was the redecking of the Chulitna River Bridge. The bridge has a total span length of 790 ft (240 m) and a total width of 34 ft (10 m). The existing structure has a concrete deck on steel trusses and stringers, as shown in Fig. 9.

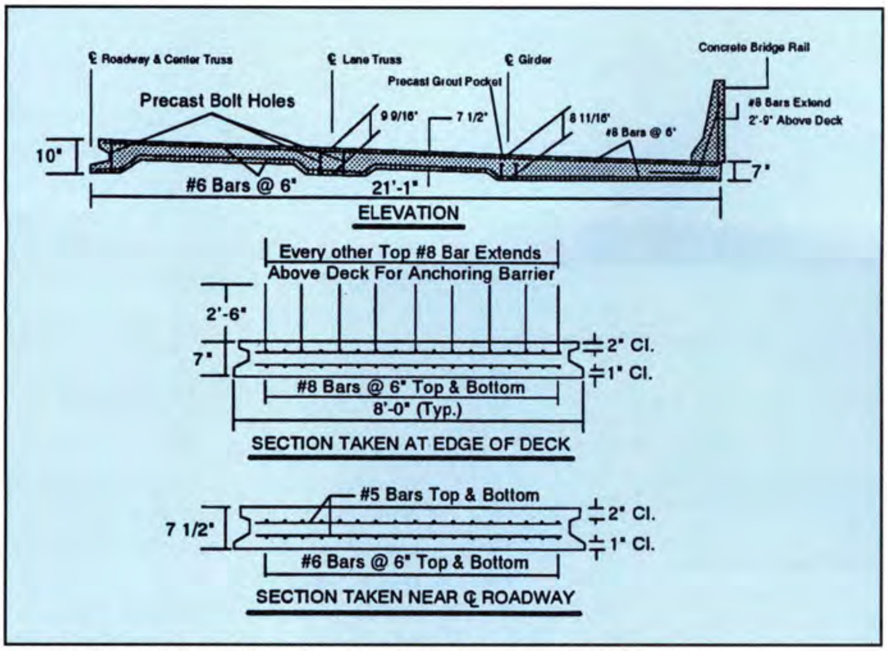

Fig. 14. Typical Panel A — Chulitna River Bridge redecking.

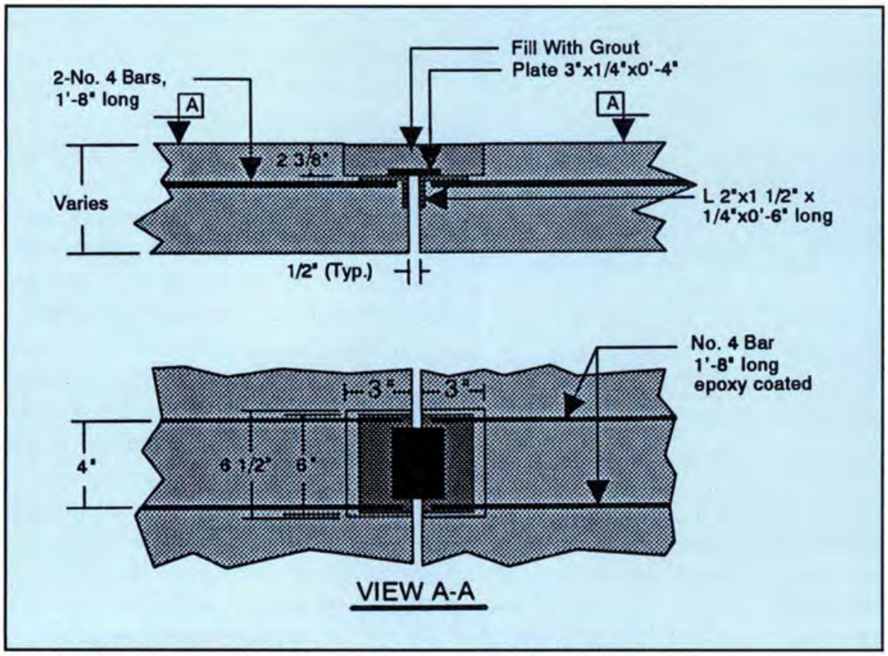

Fig. 15. Shear connector detail — Chulitna River Bridge redecking.

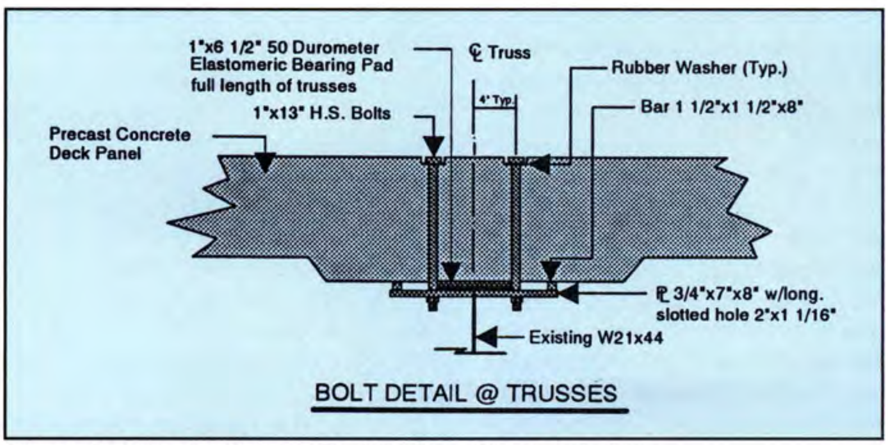

Fig. 16. Bolted connection — Chulitna River Bridge redecking.

The new construction required removal of the 34 ft (10 m) wide cast-in-place deck and replacement with 42 ft 2 in. (13 m) wide full-depth precast concrete deck panels. A stage construction was adopted in order to maintain the traffic flow during the rehabilitation process. The stage construction was very similar to that used on the first project, as shown in Fig. 10. The new typical section is shown in Fig. 11.

Three types of precast panels, A, B and C, were used to fit the geometry of the deck, as shown in the deck panel layout and edge area plan (see Figs. 12 and 13).

Normal weight concrete with a strength of 5000 psi (34 MPa) and epoxy coated mild steel reinforcement were used. The elevation of a typical Panel A and the cross sections are shown in Fig. 14. A typical female-to-female joint was used between the adjacent panels, which is similar to the joint used in the Dalton Highway Bridge rehabilitation (see Fig. 7). The joint has the same configuration; however, the dimensions are different.

A system of two 2 x 1$^1/_2$ x $^1/_4$ in. (50 x 38 x 6 mm) steel angles 6 in. (150 mm) long, 3 x $^1/_4$ in. (75 x 6 mm) steel plates 4 in. (100 mm) long, and four No. 4 bars 1 ft 8 in. (510 mm) long was used as a shear connector (see Fig. 15). Magnesium-phosphate grout was used to fill the pocket.

Two types of connections between the panels and the supporting system were used. Details of the bolted connections on truss elements including also the grouted pocket connections on the steel stringers are shown in Figs. 16 and 17. The design called for the use of two different types of connections for two reasons. First, the truss flanges were too narrow for a grouted pocket connection, and second, the bolted connection provided some support for the structure prior to grouting.

IOWA DOT

One bridge under construction utilizes full-depth precast, prestressed concrete deck panels that are designed to act compositely with the steel floor beams and girders. This cable-stayed bridge over the Mississippi River at

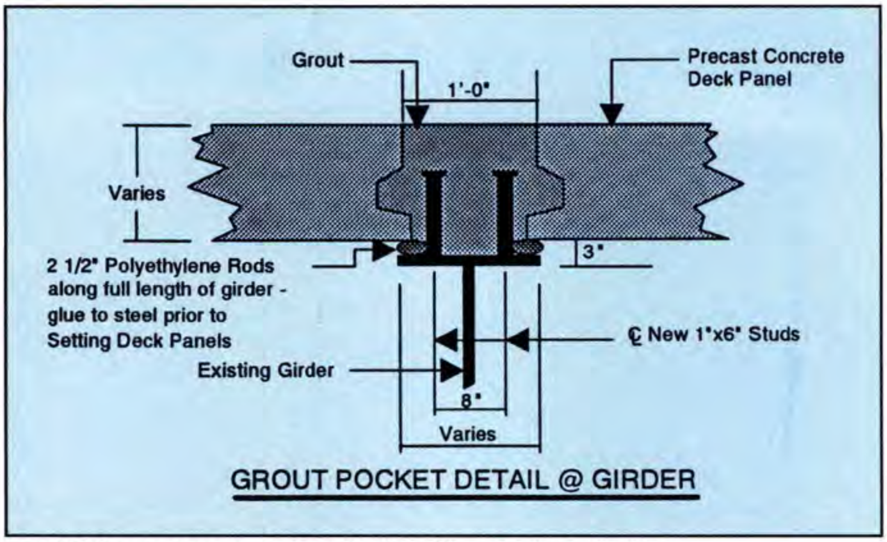

Fig. 17. Grouted connection — Chulitna River Bridge redecking.

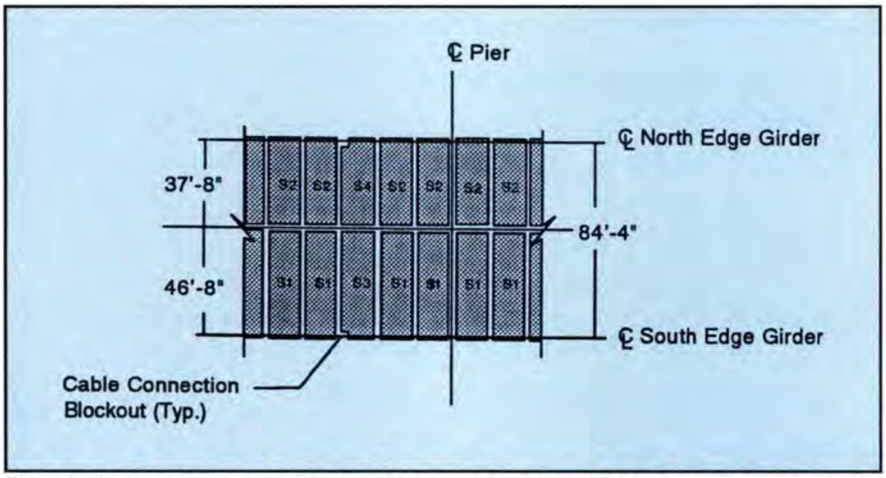

Fig. 18. Portion of precast slab panel plan — Burlington Bridge, Iowa Department of Transportation.

Burlington, Iowa, is 87.5 ft (27 m) wide and 1065 ft (325 m) long with two spans of 660 and 405 ft (201 and 124 m). The support system consists of transverse floor beams, with a spacing of 15 ft (4.6 m), carried by two girders at the north and south bounds. The precast panels were 10 in. (250 mm) thick, 13 ft 9 in. (4.2 m) long, and 47 ft 3 in. or 38 ft 3 in. (14.4 or 11.7 m) wide. They were installed transversely on the floor beams, as shown in Fig. 18.

A variety of precast panel types were needed to meet the post-tensioning requirements in the longitudinal direction. Two panel types are shown in Fig. 19.

Post-tensioning in the transverse direction was applied to the panels for handling and erection, as shown in Fig. 20. The entire post-tensioning system (thread bars, nuts, couplers, and anchor plates), except the ducts, was epoxy coated. The initial post-tensioning force was 89 kips (396 kN) for the 1 in. (25 mm) diameter thread bar and 166 kips (738 kN) for the $1^3/_8$ in. (35 mm) diameter thread bar. Cast-in-place concrete was used to fill the 24 in. (610 mm) wide longitudinal and 15 in. (380 mm) wide transverse joints.

The shear connector pockets were 9 in. (230 mm) long and 3 in. (75 mm) wide. All of these pockets were distributed on the edge girders with a spacing of 9 in. (230 mm). Fig. 21 shows the plan and cross section of the shear connector pockets. Non-shrink grout was used to fill the pockets and the space between the precast panels and girder flanges.

Leveling screws were used to adjust the level of the precast panels, as shown in Fig. 22. A 2 in. (50 mm) layer of low-slump dense concrete was used as protection for the precast deck.

The Iowa DOT has not utilized this type of deck construction in the past, so they could not comment on the performance and durability of the precast deck construction.

CONNECTICUT DOT

The Connecticut DOT is currently undertaking a $7 billion Infrastructure Renewal Program. Part of this program involves the rehabilitation of approximately 1640 bridges at an estimated cost of $1.6 billion. Many of these bridges involve complete deck replacements requiring complicated stage erection sequences and occasional bridge closures during construction. In an attempt to expedite the construction process, a design using precast concrete deck slabs was incorporated for one of the structures (Connecticut Bridge 03200).

Prior to the slab design, research was undertaken to evaluate proper design and construction methods in order to accomplish deck replacement in the shortest time possible. The procedure of using night closures and day openings to provide uninterrupted peak hour service was not applicable for this project for two main reasons:

1. Bridge 03200 is a composite plate girder bridge; as a result, removal of the existing composite deck would be a time consuming process due to the existing shear connectors.

2. The possibility of avoiding significant overstress in the girders at the midspan area, where a small non-composite joint would occur between the old slab and the new precast deck during construction, had to be avoided.

For these reasons, it was decided to shut down the bridge for the reconstruction period because a reasonable detour existed nearby.

This six-span bridge has a total length of 700 ft (213 m) consisting of straight composite plate girders running on tangents from pier to pier. Three of the spans are continuous with a hung span supported by pins and hangers. The structure is located on a horizontally compound curve requiring various degrees of deck superelevation.

To account for the curvature, each slab was designed as a trapezoid. One end of the slab would be 8 ft (2.4 m) wide and the other slightly less, depending on the curvature. Two different shapes were chosen because there are two different curves on the structure. Because the bridge is only 27 ft 6 in. (8.4 m) wide, it was decided to use full width precast panels with 8 ft (2.4 m) width, 26 ft 8 in. (8.1 m) length, and 8 in. (200 mm) depth. Fig. 23 shows a typical section of the precast concrete slabs.

Because the slabs had to be composite, blockouts were required to allow for the installation of shear connectors. This would mean that the transverse location of the blockouts would be different for each slab. A coordinated geometry CADD program was used to calculate the required locations of all the blockouts. It was determined that by oversizing the blockouts, their location in the slabs could be limited to three different patterns.

The shear connector blockouts were rectangular, 18 x 5 in. (460 x 130 mm) at the top and tapered from top to bottom. The spacing of these blockouts was 2 ft (610 mm) on centers for each slab. Three $^7/_8$ in. (22 mm) welded stud shear connectors were placed in each blockout, as shown in Fig. 24.

The length of the precast panel does not allow full development; hence, a complete prestressed concrete design for the slabs was not possible. Thus, the slabs were designed using epoxy coated reinforcing steel, with a minimal amount of prestressing needed to prevent cracking during handling and installation.

A leveling bolt system was used for grade adjustment in the field. The bolt would be cut below the surface of the slab and the void grouted (see Fig. 25).

A standard shear key configuration filled with high strength, non-shrink grout was chosen for the transverse joints (see Fig. 26). Longitudinal post-tensioning was designed to provide continuity. The design called for three 0.6 in. (15 mm) diameter strands per cut. The strands were pulled through plastic

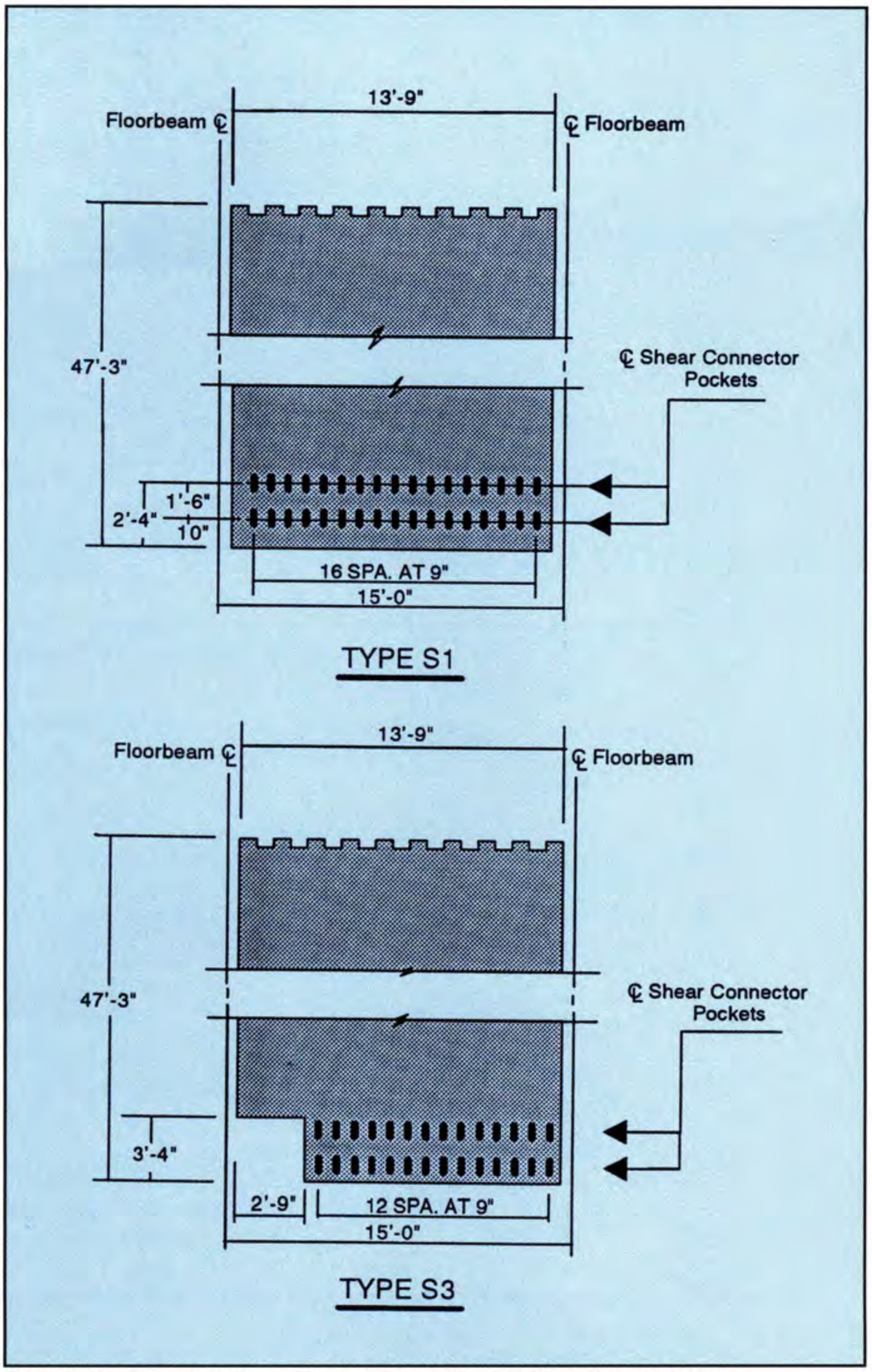

Fig. 19. Precast slab panel Types S1 and S3 — Burlington Bridge, Iowa Department of Transportation.

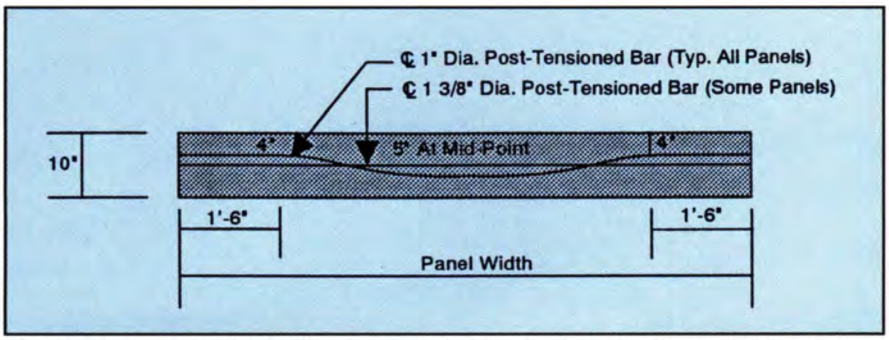

Fig. 20. Post-tensioning profiles in precast slab panels — Burlington Bridge, Iowa Department of Transportation.

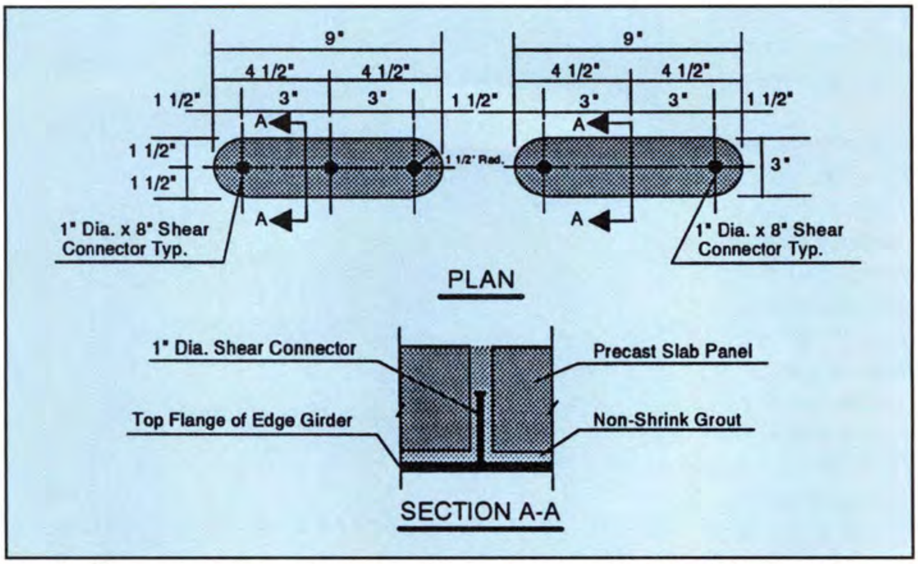

Fig. 21. Shear connectors — Burlington Bridge, Iowa Department of Transportation.

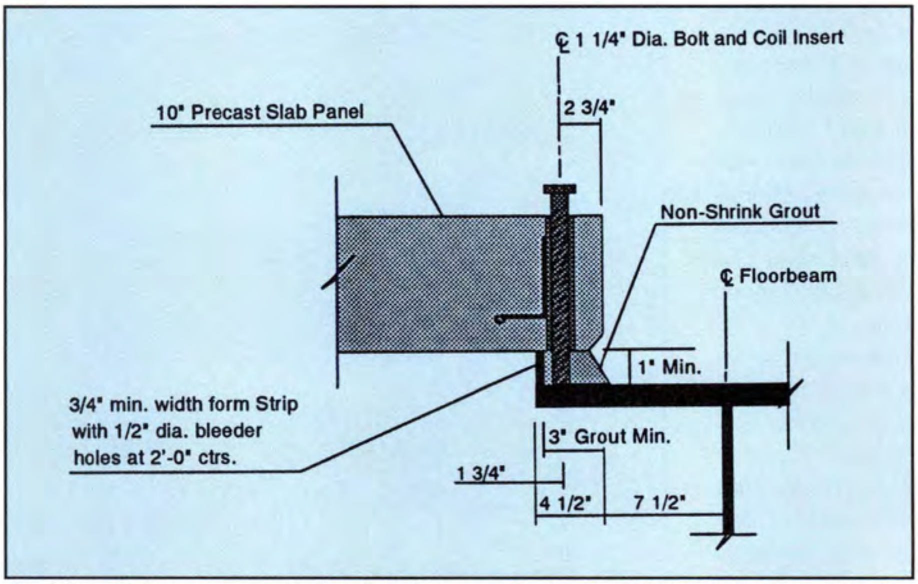

Fig. 22. Leveling screws detail at floor beam — Burlington Bridge, Iowa Department of Transportation.

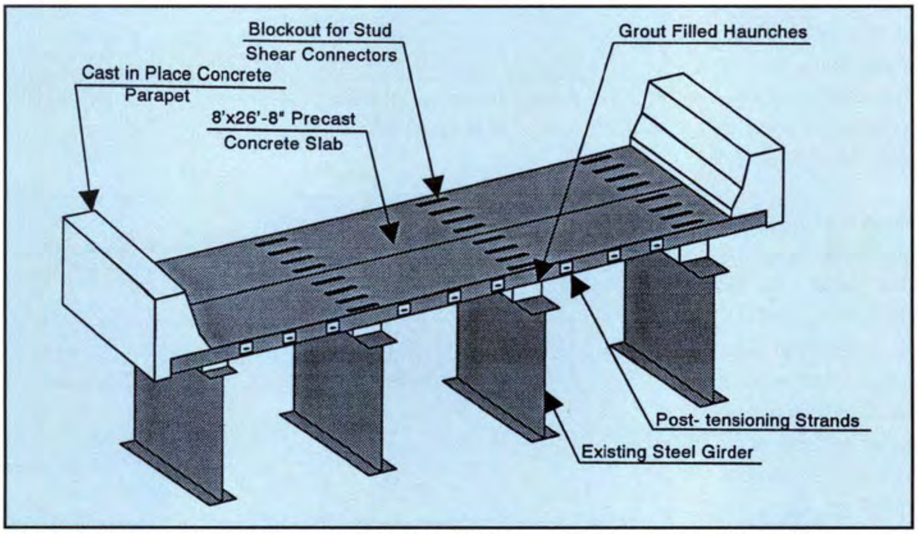

Fig. 23. Typical section of precast concrete slabs — Bridge 03200, Connecticut Department of Transportation.

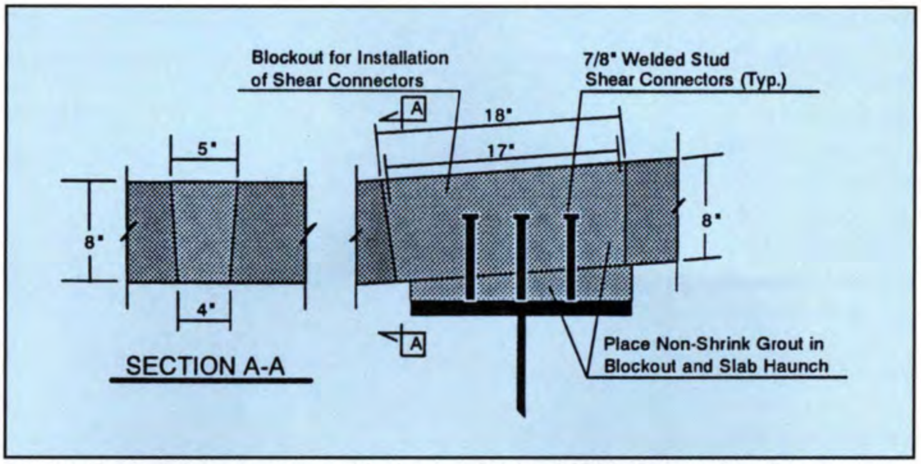

Fig. 24. Typical shear connector blockout — Bridge 03200, Connecticut Department of Transportation.

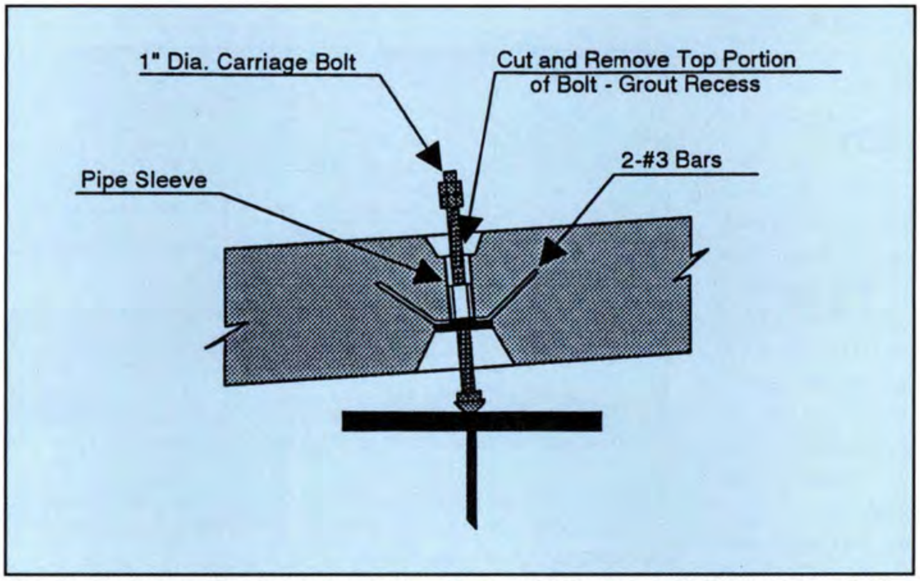

Fig. 25. Typical section at leveling bolt — Bridge 03200, Connecticut Department of Transportation.

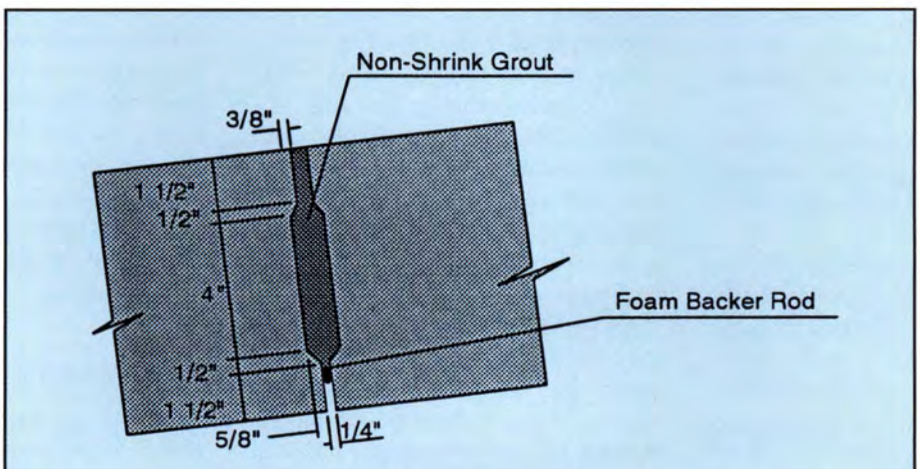

Fig. 26. Standard shear key configuration — Bridge 03200, Connecticut Department of Transportation.

ducts that were spliced at each transverse joint through small blockouts. An arbitrary stress of 150 psi (1.03 MPa) was chosen for the simple spans and was appreciably increased to 300 psi (2.07 MPa) in the three-span continuous portion of the bridge to account for the significant composite dead load and live load stresses. After the strands were installed and tensioned, the ducts were completely grouted.

At the end of each span, a small cast-in-place closure pour was used to account for time-dependent deformations in the precast slabs and to protect the post-tensioning system, as shown in Fig. 27. In order to properly seal the deck, the finished slab was topped with a membrane waterproofing system and a $2^1/_2$ in. (65 mm) bituminous wearing surface.

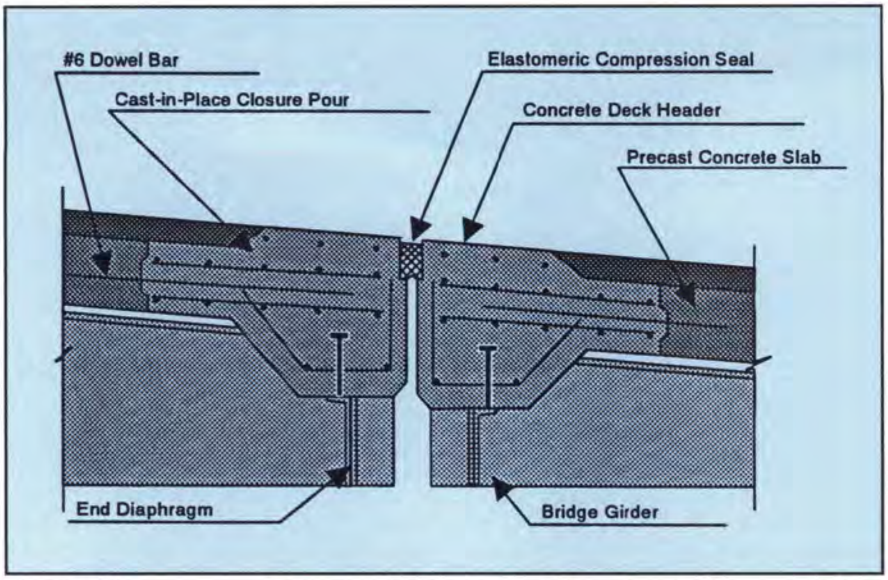

Fig. 27. Closure pour — Bridge 03200, Connecticut Department of Transportation.

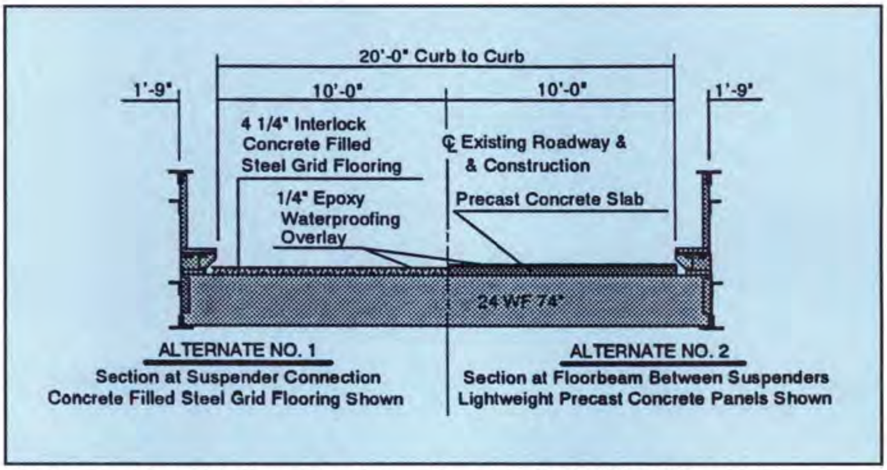

Fig. 28. Deck construction alternates — Deer Isle-Sedgwick Bridge, Maine Department of Transportation.

MAINE DOT

The selected project was the deck replacement of the Deer Isle-Sedgwick Bridge over Eggemoggin Reach between Little Deer Isle and Sedgwick (Project No. BH-0250). The bridge consists of nine spans: four at 65 ft (20 m), one at 484 ft (148 m), one at 1080 ft (329 m), one at 484 ft (148 m), and two at 65 ft (20 m), with a total width of 23.5 ft (7.2 m) center to center of the suspended girders.

Two alternative solutions were presented by the designer. Alternate I was a concrete filled steel grid deck (including the main I-beam spacing); Alternate II was a precast concrete slab deck. Selection of either Alternate I or Alternate II was the contractor's option. Fig. 28 shows the cross section for the two alternatives. The supporting deck system consisted of two types of suspended transverse girders — WF14 x 42 for approach spans and WF24 x 74 for suspended spans, with floor beams in between.

Alternate II was adopted for the redecking process. The work started in May 1987 and was completed in October 1987 without major traffic interruption. The lightweight precast concrete deck panels were designed to cover a half width of the bridge to maintain traffic flow during construction. The panels were $6^1/_2$ in. (165 mm) thick, 9 ft 11 in. (3.0 m) wide and of variable length depending on the spacing of the suspended girders. Fig. 29 shows the plan and cross section of the blockouts. The welded connection blockout detail is shown in Fig. 30.

A typical female-to-female transverse joint was chosen (see Fig. 31). Joints and blockouts were filled with epoxy mortar after the shear connectors and plate connections were welded. No prestressing was applied to the slabs. Epoxy coated reinforcing steel was used.

All the panels had a $^1/_4$ in. (6 mm) epoxy waterproofing overlay applied prior to erection. The overlay covered the entire top surface of the panels within 6 in. (150 mm) of any blockout or shear key. After the shear keys and blockouts were filled, the epoxy waterproofing overlay was placed over these areas. Elastomeric compression joints were installed at the juncture of approach and suspended spans to absorb the cyclic movement of the bridge.

Recent inspection has revealed that this protective coating did not work properly. The transverse joints have cracks and the system is leaking. The reasons for these adverse results, according to the Maine DOT, were material quality, construction procedures, and the substantial movement of the bridge.

MARYLAND DOT

The Woodrow Wilson Memorial Bridge is the major crossing of Interstate 95 over the Potomac River south of Washington, D.C. This bridge, constructed in 1962, is 5900 ft (1800 m)

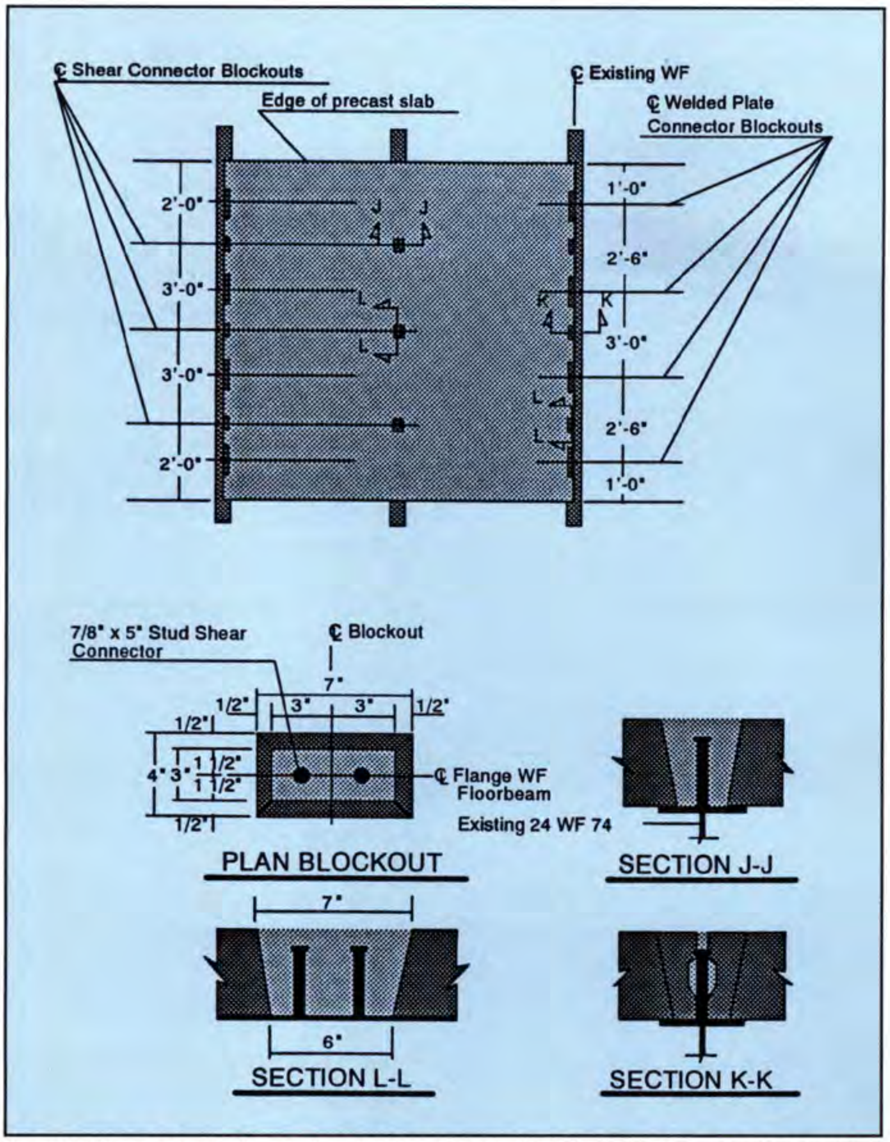

Fig. 29. Panel blockout layout and shear connector details — Deer Isle-Sedgwick Bridge, Maine Department of Transportation.

long consisting of 18 steel deck girder approach units, eight on the Virginia side and ten on the Maryland side. Most approach units are four-girder continuous multispan units. Floor beams between girders are spaced approximately 16 to 26 ft (5 to 8 m) on centers and carry five rolled beam stringers per roadway continuously over the floor beams.

The deck provided a six-lane roadway 76 ft (23 m) wide. The original 89 ft (27 m) width was subdivided by a longitudinal centerline roadway joint. Because of the heavy volume of traffic (110,000 vehicles per day), the study called for uninterrupted traffic flow: six lanes of traffic during peak hours; four or five lanes during off-peak daytime hours; and one lane in each direction during the night time periods.

Due to the above restrictions, it was decided to replace the deck part-by-part with precast, prestressed lightweight concrete panels. These panels were installed transversely for each roadway width. The new deck system provides two 44 ft (13 m) roadways to permit space for disabled vehicles that previously had caused commuter traffic delays.

The typical lightweight concrete panel is 46 ft 7$^1/_4$ in. (14 m) wide, 10 to 12 ft (3.0 to 3.7 m) long and 8 in. (200 mm) thick with a 5 in. (130 mm) haunch at the exterior girder.[11,12] A total of 1026 panels were utilized for the construction of the bridge. The panels were transversely post-tensioned at the fabrication plant. The $^1/_2$ in. (13 mm) diameter transverse strands were placed in pairs at approximately 12 in. (305 mm) on centers in the planes of the top and bottom reinforcing steel. At both edges of the panels, these strands were slanted to mid-depth of the slab for anchorage.

These panels were installed transversely to cover a half width of the bridge and post-tensioned in the longitudinal direction to provide sufficient compression to keep the transverse joints between panels closed. This was provided by 13 groups of four 0.6 in. (15 mm) diameter strands at the slab mid-depth. The post-tensioning connected segments in lengths of 140 to 285 ft (43 to 87 m). To ensure full bearing between deck panels under longitudinal post-tensioning, and to provide for construction tolerances, the plans called for a 1$^1/_4$ in. (32 mm) joint between panels to be filled with polymer concrete immediately prior to post-tensioning.

The bearings between panels and stringers consisted of a sliding steel bearing plate on the flange, keyed to the cast-in-place polymer concrete by welded studs. Pour holes through the panel are belled at the bottom to facilitate pouring and to provide the required resistance to shear forces. Pairs of hold-down bolts at the pads on three stringers tie the panels to the structural steel. Three bearings are provided on each of the five stringers and three or four bearings on the girders under each panel.

Because the construction work sequence required many steps and the need to open all lanes for traffic in the rush hours, it was necessary to use polymer concrete based on a methylmethacrylate monomer, to give the required strength in one hour and to hold the new panels in place under normal traffic. A two-coat, epoxy-and-sand membrane was applied to the top surface of the panels at the fabrication plant. All embedded reinforcing steel, prestressing hardware, and studs were epoxy coated.

Rehabilitation was completed 8 months ahead of schedule, $6 million under budget, and without disrupting the flow of traffic.

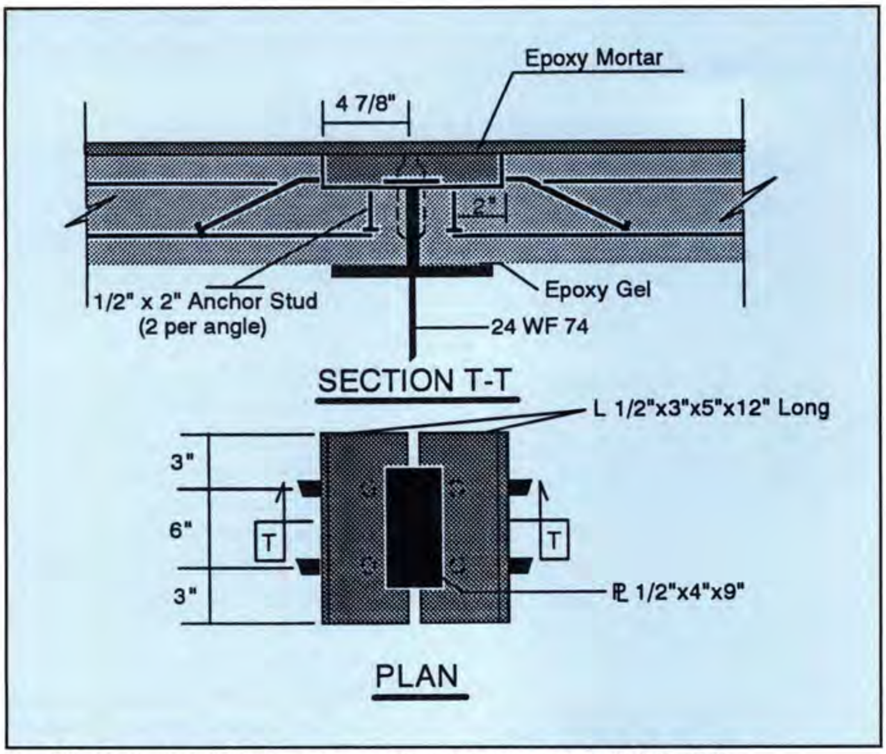

Fig. 30. Welded plate connection — Deer Isle-Sedgwick Bridge, Maine Department of Transportation.

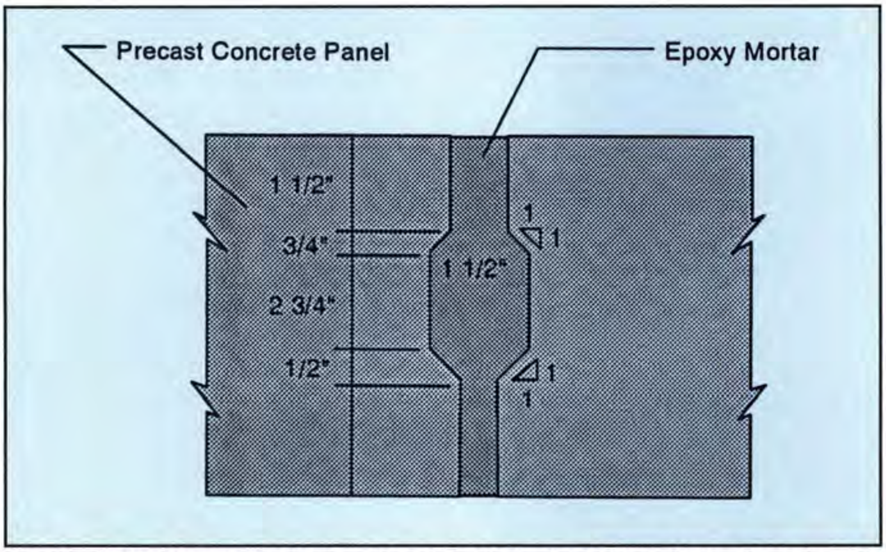

Fig. 31. Typical transverse joint — Deer Isle-Sedgwick Bridge, Maine Department of Transportation.

NEW YORK STATE DOT

The project is the rehabilitation of the Route 155 bridge over Normanskill State Highway 1928 in the town of Guilderland, New York. Two previous contracts were accomplished: the first was the original bridge construction in 1931; the second was the bridge deck resurfacing in 1972. The new contract aims to extend the life of the rehabilitated structure as a "stopgap" temporary repair until it is rebuilt. As stated in the contract, within 40 days the deck should be replaced and the bridge reopened to unrestricted two-way traffic.

The replaced area was 101 ft 10 in. (31 m) long and 25 ft 5$^{1}/_{2}$ in. (7.8 m) wide. The construction process aimed in Stage I to close 14 ft 3 in. (4.3 m) of the full width of the bridge leaving 9 ft 9 in. (3.0 m) as a traveling lane. In Stage II, the work will move to the other side of the roadway keeping 9 ft 11 in. (3.0 m) for traffic. Two types of precast panels were designed as intermediate and end panels with the same width of 6 ft 4 in. (1.9 m) and two different lengths of 12 ft 4 in. and 13 ft 4 in. (3.8 and 4.1 m), respectively. These panels were installed on a framing system of transverse girders, with a spacing of 12 ft 6 in. (3.8 m), spanning between two trusses at the north and south bounds.

The typical $^{1}/_{2}$ in. (13 mm) female-to-female longitudinal joint is filled with non-shrink cement grout. Every panel has four leveling bolt sleeves at the four corners to accomplish the required position of the panel. Shear studs were installed in the 2 in. (50 mm) transverse joints. Non-headed studs, $^{3}/_{4}$ in. (20 mm) in diameter and 4 in. (100 mm) long at 15 in. (380 mm) spacing, were used between intermediate panels and $^{3}/_{4}$ in. (20 mm) non-headed studs 1 in. (25 mm) long at 15 in. (380 mm) spacing were employed at the end panels.

OHIO DOT

The Ohio DOT reported the rehabilitation of approximately one to five bridges. Construction started in 1986 on a skew bridge consisting of six spans: 73, 95, 100, 100, 95, and 73 ft (22, 29, 30, 30, 29, and 22 m), with a bridge width of 56 ft (17 m) from the face of railings and clearance of 50+ ft (15+ m). The bridge had a concrete arch with cross beams as its deck supporting system.

Full depth precast panels of lengths 12 ft 1$^{1}/_{2}$ in., 9 ft 10$^{1}/_{2}$ in., 9 ft 6$^{1}/_{2}$ in., 9 ft 5$^{1}/_{2}$ in., and 10 ft 1 in. (3.7, 3.0, 2.9, 2.9, and 3.1 m) and panel width of 28 ft (8.5 m), along with a varying depth, were used. Non-prestressed steel was furnished as panel reinforcement to account for handling and erection stresses, and post-tensioned tendons for service load stresses.

The concrete stress level for the post-tensioning was about 1000 psi (6.89 MPa). Panels are supported on elastomeric bearings and are anchored down to floor beams using dowel bars. All the reinforcing steel was epoxy

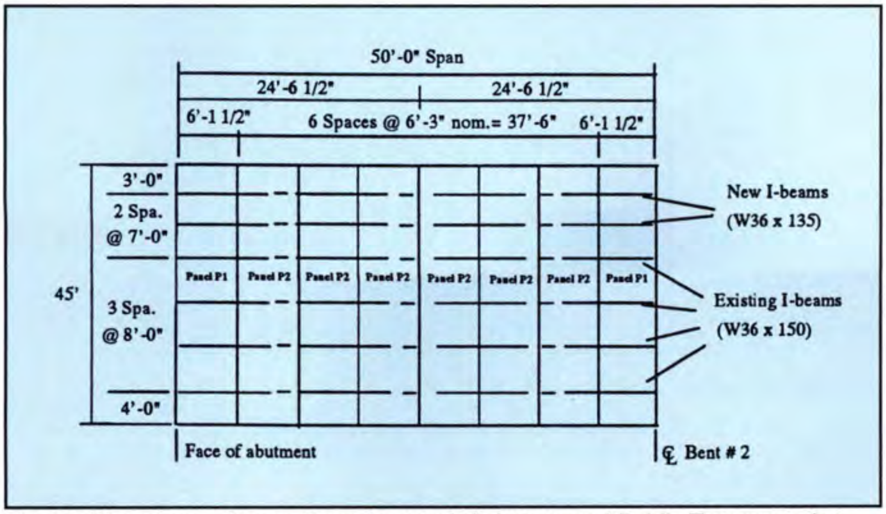

Fig. 32. Plan view — Texas Department of Highways and Public Transportation.

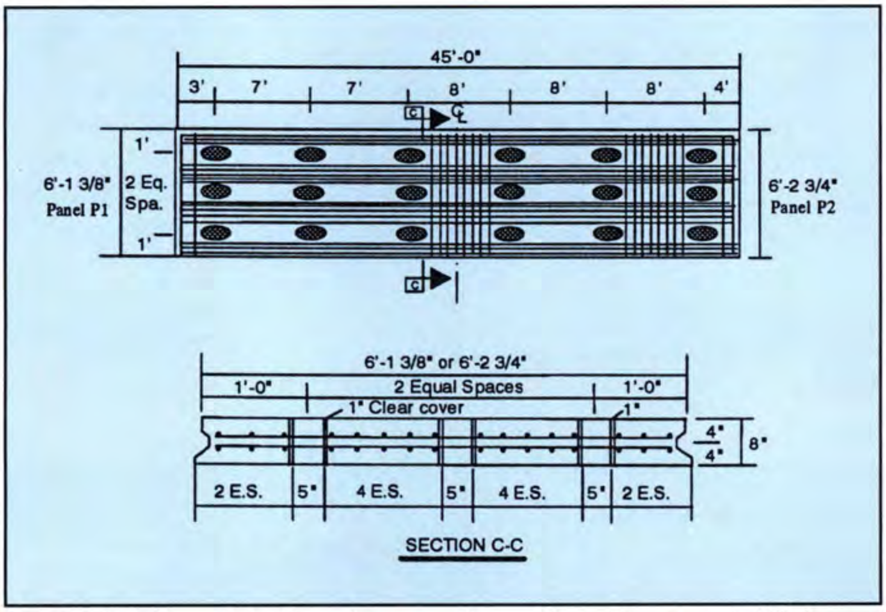

Fig. 33. Typical panel details and cross section — Texas Department of Highways and Public Transportation.

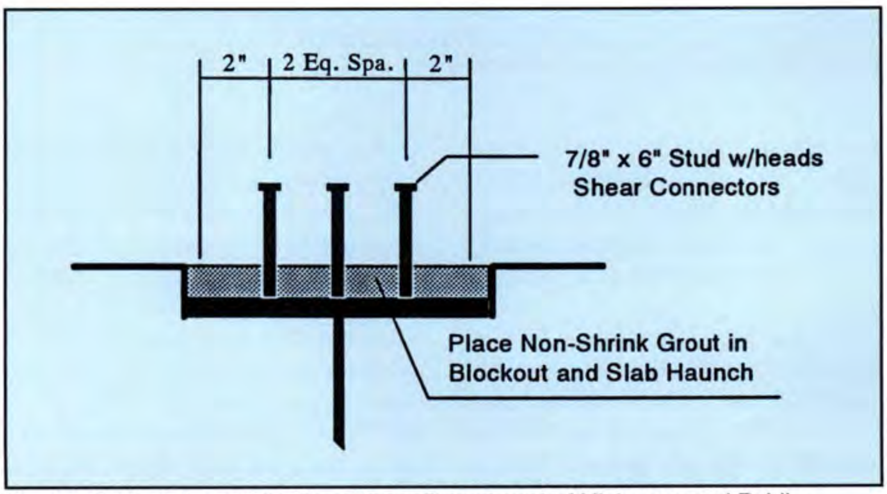

Fig. 34. Stud and grout detail — Texas Department of Highways and Public Transportation.

coated, while the prestressing strands were polymer coated. Epoxy mortar material was used for the joints between the adjacent precast panels.

MINISTRY OF TRANSPORTATION OF ONTARIO, CANADA

Welland River Bridge

The structure selected for redecking was the 18-span Welland River Bridge, which carries two southbound lanes near the city of Niagara Falls. The bridge was originally built in 1939 and consists of five continuous span units. The structure was noncomposite prior to the rehabilitation.

For comparison purposes, four of the five units were rehabilitated using cast-in-place concrete decks and only one unit of three spans at the south end with precast concrete decks. The three spans were $48^3/_4$ ft, 48 ft, and 48 ft (14.859, 14.63, and 14.63 m). The bridge width was $43^1/_2$ ft (13.26 m) with a variable bridge clearance and panel depth of 9 in. (225 mm).

The deck supporting system consisted of four lines of steel girders with sizes 33WF125 for the exterior girders and 33WF150 for the interior girders. The full depth precast panels consisted of a length of 79.7 ft (24.28 m) for the end panels and 79.3 ft (24.18 m) for the remaining panels.

Non-prestressed steel was used as reinforcement for the panels. The steel sizes were 15 at 10 in. (250 mm) longitudinally and 15 at 9 in. (230 mm) transversely [with 13.4 in. (340 mm) spacing at the openings for stud connectors]. The tendons consisted of 11 [four 0.6 in. (16 mm) diameter strands] tendons in the panels near the ends to 20 [four 0.6 in. (16 mm) diameter strands] tendons over the piers and center span. The strands were 0.6 in. (16 mm) with a strength of 58.4 kips (260 kN).

The joints between the adjacent precast panels were key joints with a nonshrink grout. Longitudinal prestressing was increased by about 33 percent to prevent the occurrence of any cracks. As a result, no problems were encountered during actual installation. A waterproof membrane with Class I bituminous surfacing was used as a protection system. Details of this study are presented in a paper by Farago et al.[13]

TEXAS DEPARTMENT OF HIGHWAYS AND PUBLIC TRANSPORTATION

The Department of Highways and Public Transportation of the State of Texas reported the reconstruction of a bridge (A. T. and S. F. Railway Overpass). The span length for this bridge is 50 ft (15 m) and the width is 45 ft (14 m). The bridge deck is supported on W36 x 150 I-beams. The project called for the replacement of the deck with a precast concrete deck as well as replacement of the two end beams with new W36 x 135 I-beams. These two beams were replaced and new diaphragms installed. The remaining four beams (W36 x 150) were preserved.

Two types of panels were used in the construction. Fig. 32 shows a plan view of the two panels considered. The end panels have a width of 6 ft $1^3/_8$ in. (1.9 m) and the interior panels 6 ft $2^3/_4$ in. (1.9 m).

The beams are spaced; two spaces at 7 ft (2.1 m) and three spaces at 8 ft (2.4 m). The distance from the end where the new beams were installed is 3 ft (0.9 m), and the distance from the other end is 4 ft (1.2 m). The shear connector openings were of beveled shape; however, openings may be 5 x 11 in. (127 x 279 mm) rectangular at the contractor's option. Details of a typical panel are shown in Fig. 33.

Studs of $^7/_8$ in. diameter x 6 in. (22 x 150 mm) with heads were end welded after all deck panels were placed. Fig. 34 shows the stud and grout detail. A female-to-female type joint was used between the deck panels, as shown in Fig. 35.

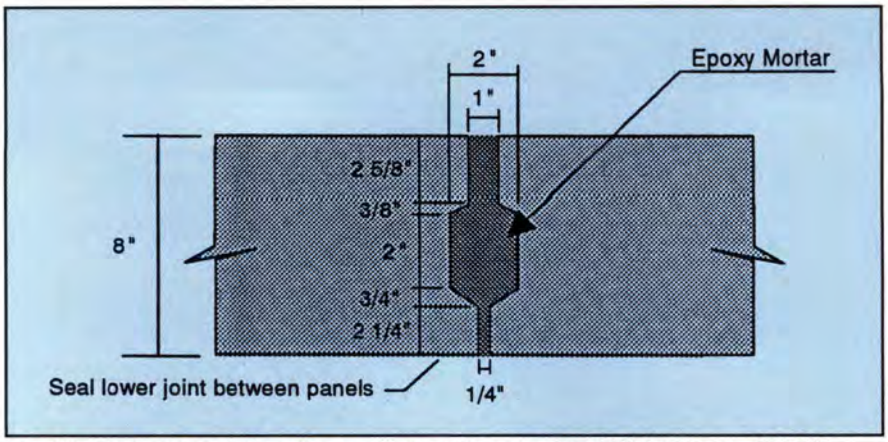

Fig. 35. Typical panel shear key — Texas Department of Highways and Public Transportation.

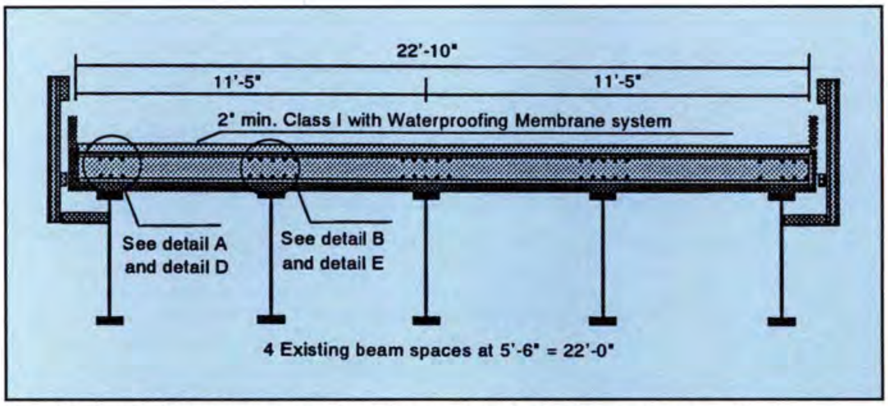

Fig. 36. Typical transverse sections for approach and truss spans — Seneca Bridge.

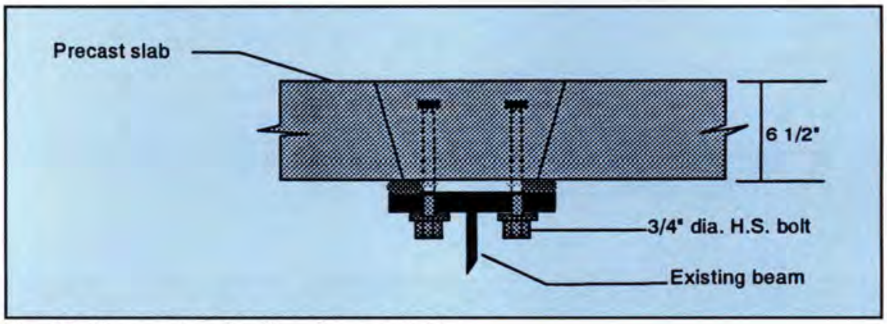

Fig. 37. Connection detail — Seneca Bridge.

ILLINOIS DOT

The Illinois DOT funded the Seneca Bridge. The structure was built in 1932 and consists of 13 total spans. The span lengths are 60 ft, 60 ft, 60 ft, 60 ft, 60 ft, 202 ft $1^3/_8$ in., 364 ft 4 in., 202 ft $1^3/_8$ in., 201 ft 9 in., 60 ft, 60 ft, 60 ft, and 60 ft (18, 18, 18, 18, 18, 62, 111, 62, 61, 18, 18, 18, and 18 m), with an overall span length of 1510 ft 3 in. (460 m). Spans 1 through 5 and 10 through 13 are approach spans, while Spans 6 through 9 are interior truss spans.

The four truss spans, along with the approach spans, had the existing concrete deck removed and replaced with a $6^1/_2$ in. (165 m) precast, prestressed slab deck. All precast slabs were match set, with the replacement being performed in sections. Full two-way traffic was maintained throughout construction, in accordance with special provisions. Bridge closure was permitted in a 10-hour period, Sunday through Thursday, from 7 p.m. to 5 a.m.

One in. (25 mm) diameter smooth prestressed bars, quenched and tempered to a minimum yield strength of 90,000 psi (620 MPa) and a maximum yield strength of 110,000 psi (758 MPa) were used. One in. (25 mm) diameter deformed prestressed bars of Grade

150, initially stressed to 45,000 psi (310 MPa), were also used.

The existing beams are spaced at 5 ft 6 in. (1.7 m). The connection between the precast deck and supporting system varies in accordance with the spans. Two high strength $^{3}/_{4}$ in. (19 mm) diameter bolts were used for the approach spans. On the other hand, four high strength $^{3}/_{4}$ in. (19 mm) diameter bolts were used for the truss spans. Typical transverse sections for both types of spans are shown in Fig. 36, while the connection between the concrete deck and beams is shown in Fig. 37.

The types of shear pockets for the end and interior panels, including also the approach and truss spans, are shown in Fig. 38. A male-to-female type joint was used between the deck panels, as shown in Fig. 39.

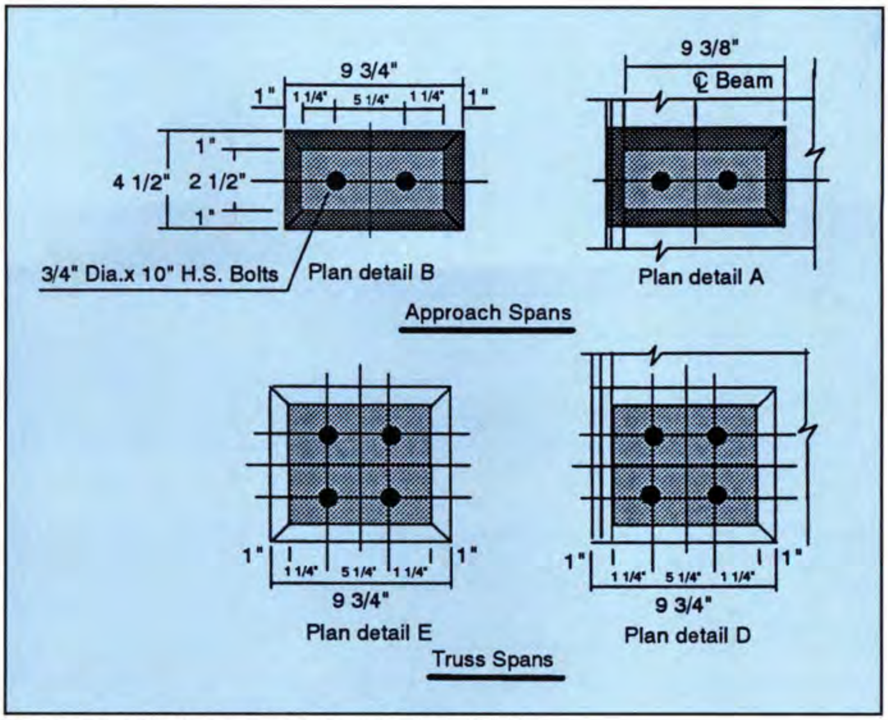

Fig. 38. Details of shear pockets — Seneca Bridge.

VIRGINIA DOT

The following two bridges are under the jurisdiction of the Virginia DOT.

Route 229 Bridge Over Big Indian Run Near Culpeper

This structure is a simple span bridge, 54 ft (16 m) long and 30 ft (9.1 m) wide. The existing steel rolled beams are 6 ft 3 in. (1.9 m) center to center. The two exterior beams, spaced 3 ft (0.9 m) from the end, are W33 x 125, while the interior beams are W33 x 132. Fig. 40 shows a half transverse section of the structure.

The project included the installation of precast deck panels on the bridge by the Virginia State Bridge Maintenance crew. Two phases of construction were carried out to maintain traffic flow, as shown in Fig. 41. The project was successfully completed on December 18, 1985. Six precast panels were installed at 8 ft (2.4 m), with a typical panel shown in Fig. 42.

The joints between adjacent panels were of female-to-female type, as shown in Fig. 43. The connection system between the slab and the beams consisted of $3^{7}/_{8}$ in. (22 mm) shear studs. The stud voids were filled with high early strength concrete and a non-shrink additive. Details of the stud voids are shown in Fig. 44.

Route 235 Bridge Over Dougue Creek, Fairfax

This structure consists of four 38 ft (12 m) spans and a width of 36 ft (11 m) face-to-face of rails. The bridge was originally built in January 1932. In January 1969, some minor repairs were performed on the structure; however, in February 1981, the bridge was redecked with precast deck panels. The panels were 7 ft (2.1 m) wide and 17 ft 11 in. (5.5 m) long for the end panels, and 7 ft 6 in. (2.3 m) wide and

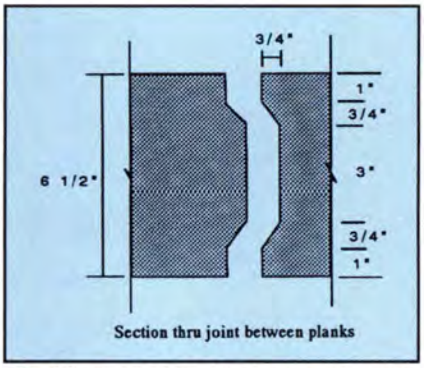

Fig. 39. Typical joint between adjacent precast panels — Seneca Bridge.

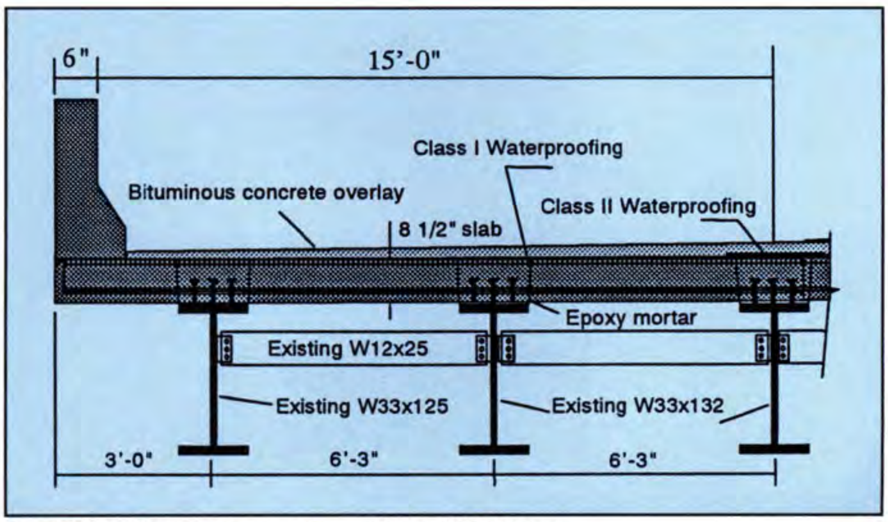

Fig. 40. Half transverse section — Route 229 Bridge.

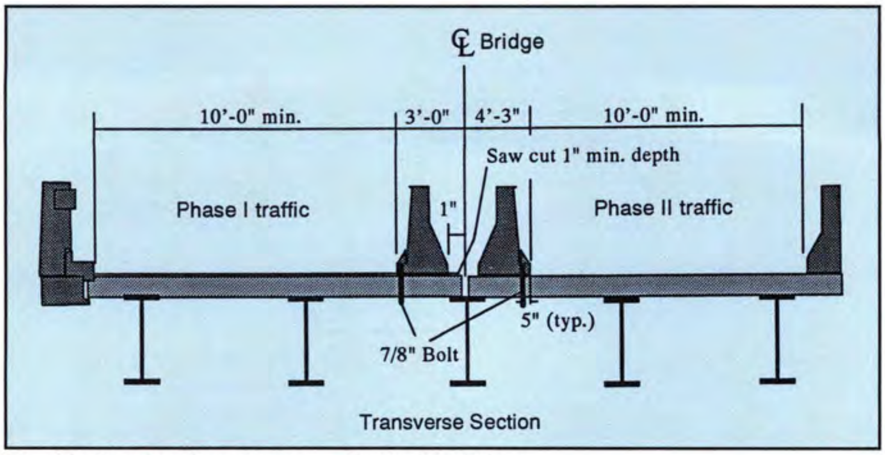

Fig. 41. Construction phases — Route 229 Bridge.

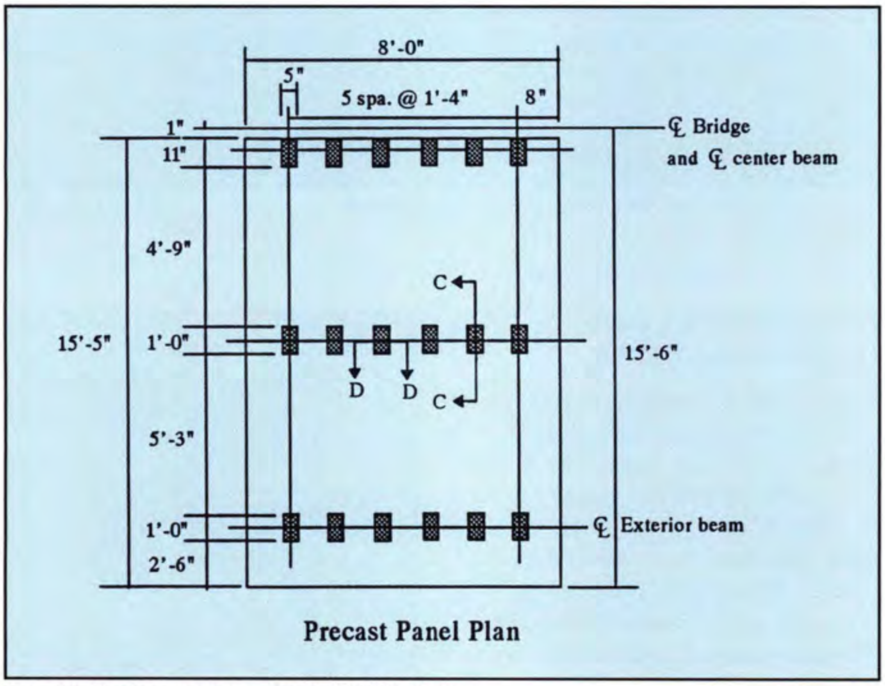

Fig. 42. Typical panel — Route 229 Bridge.

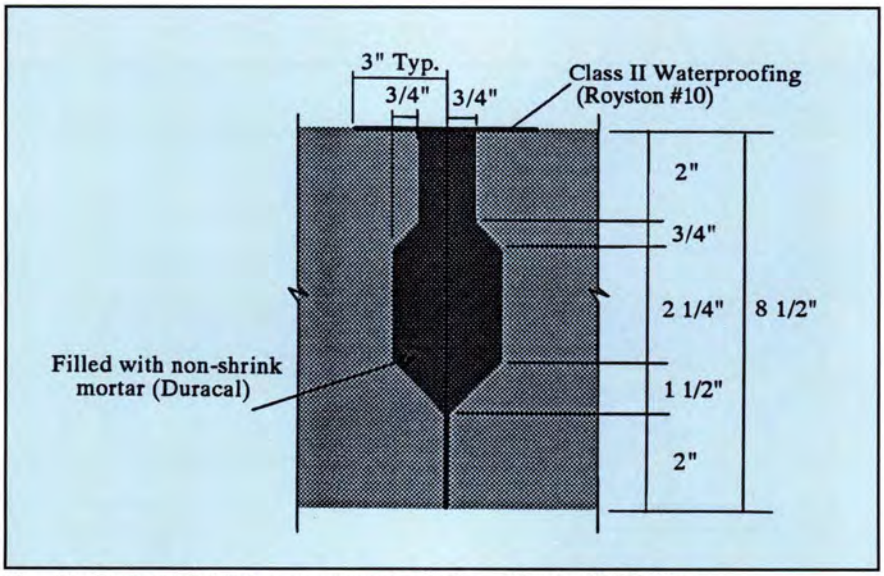

Fig. 43. Typical joint between precast panels — Route 229 Bridge.

17 ft 11 in. (5.5 m) long for interior panels, as shown in Fig. 45.

The existing interior steel rolled beams (W28 x 104) were preserved and cleaned, while new beams (W27 x 102) were installed at the ends. The beams are spaced 6 ft 4 in. (1.9 m) center to center, with the exterior beams at 3 ft 1 in. (0.9 m) from the ends. A two-phase construction process was employed to maintain traffic flow.

The joints between the precast panels were similar to those used in the Route 229 bridge, i.e., female-to-female joints (see Fig. 43); however, the dimensions are slightly different. These joints were filled with non-shrink mortar. Stud shear connectors, $^5/_8$ in. (22 mm) in diameter, were used between the precast slab and beams.

PRACTICAL IMPLICATIONS OF THE STUDY

Full depth precast and precast, prestressed concrete decks have now been used for the rehabilitation and new construction of bridge projects for two decades. This new concept of construction has been successfully applied to all types of bridges, including suspension bridges, cable-stayed bridges, truss bridges, and girder bridges, with different geometric profiles, such as skewed, superelevated, and crowned.

Durability and the reduced need for maintenance, ease and speed of construction, together with maintaining traffic without interruption are all advantages in using precast or precast, prestressed concrete deck panels.

Although the precast or precast, prestressed concrete deck is slightly more expensive than a cast-in-place deck alternate, the reduction in construction time and corresponding decrease in traffic delays will reduce the added expense to bridge users and owners alike.

CONCLUSIONS

The innovative features of using precast or precast, prestressed bridge deck construction are as follows:

1. Full depth precast and precast, prestressed concrete deck panels give the contractor the opportunity to fabri-

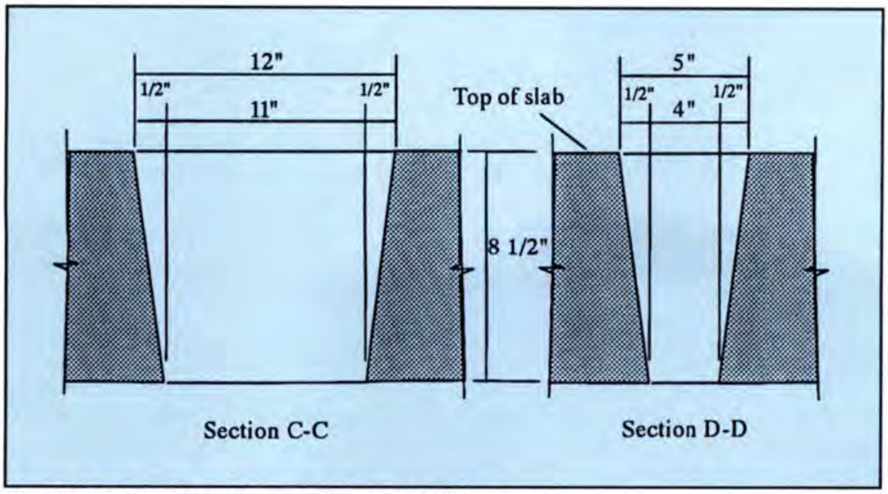

Fig. 44. Stud void sections — Route 229 Bridge.

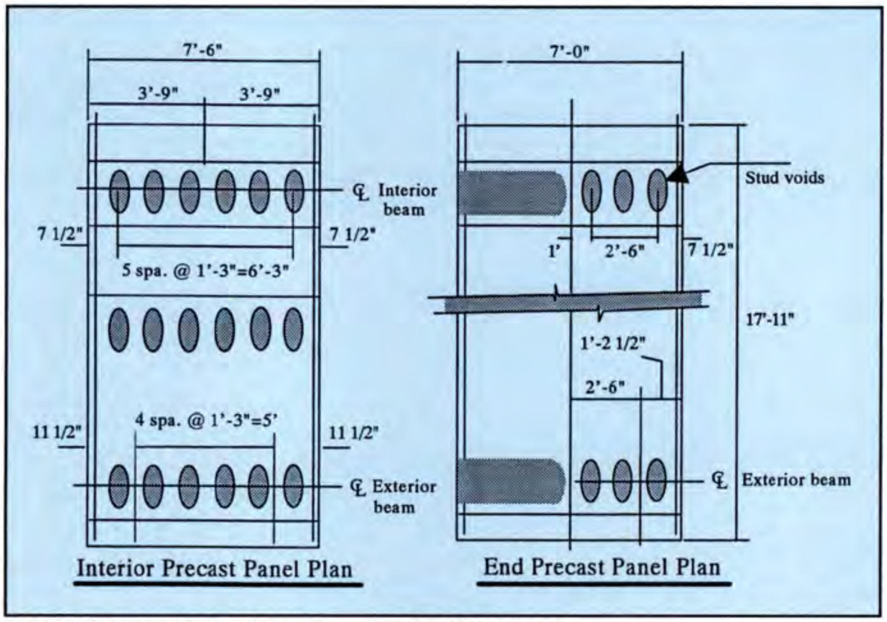

Fig. 45. Interior and end panels — Route 235 Bridge.

cate all the required units before demolition begins, and also give the owner the option to widen the deck to meet current safety requirements in an expeditious manner.

2. By adopting special techniques, it is possible to maintain uninterrupted daytime traffic flow during rehabilitation. A half width of the bridge is closed during the night and a specific portion of the old deck is removed and replaced during the same night to be ready for traffic the next day. This method may present some obstacles if the structure is a composite plate girder type bridge.

3. Adopting a grouting material, such as polymer concrete, to be used for the joints can provide the required strength in a very short time, e.g., f_c' of 4500 psi (31 MPa) in an hour. This is essential when it is necessary to open all lanes of the bridge for traffic in the rush hours.

4. Transverse prestressing is provided to protect the panels from cracking during handling and installation.

5. Longitudinal prestressing is provided to secure the tightness of the joints. Vertical shear and transverse bending moments may occur at the joints, especially in continuous span portions where significant composite dead load and live load stresses occur. To account for these stresses and to ensure that the transverse joint will remain in compression, post-tensioning in the longitudinal direction is provided in the system.

6. Special types of joints, used between adjacent panels, are filled with high strength, non-shrink grout to bond the two panels and to resist the vertical shear stresses in the joint.

7. Special types of shear connectors have been used between the panels and the supporting system to resist the horizontal shear stresses.

Research Goals

Finally, after identifying the ideal system, the research will focus on:

1. The best jointing system between the panels that can provide high flexural and shear resistance, full bond, and complete tightness.

2. The best connecting system between the deck and its supporting system that can provide high performance associated with adequate capacity to absorb the horizontal shear stresses between deck panels and support elements.

3. The overall structural behavior of a deck that is built using full depth precast, prestressed transverse concrete elements on longitudinal stringers.

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