Field Performance of Full Depth Precast Concrete Panels in Bridge Deck Reconstruction

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There have been many applications of full depth precast and precast, prestressed concrete deck panels for bridge rehabilitation in North America. This paper presents the findings obtained through an investigation conducted in the United States to evaluate the field performance of bridge deck panels. The states included in the investigation were Illinois, Connecticut, Virginia, Maryland, Iowa, California, New York, Alaska, Ohio, Pennsylvania, and Washington, D.C. The investigation mainly consisted of a visual inspection of bridges selected through an earlier phase of the overall study. The inspection process entailed a general visual search for any problems associated with the joints between adjacent precast concrete panels as well as the connection between the deck and supporting system and the condition of the overlay system. The investigation also included discussions with state engineers regarding the design, construction, and performance of the precast concrete panels. This paper is part of an investigation carried out for an Illinois Department of Transportation rehabilitation program.

Full depth precast concrete systems have been used successfully in North America in the rehabilitation and replacement of deteriorated bridge decks, as documented by Issa et al.[1-3] and other researchers.[4-18] The first paper[1] involved the identification of systems constructed using the full depth precast system for an Illinois Department of Transportation (DOT) rehabilitation program. This process was carried out through the collection of literature and, especially through a comprehensive questionnaire survey that was sent to every department of transportation and other agencies throughout the United States and parts of Canada. Hence, the first paper[1] essentially summarized the significant results of that survey.

This paper presents the results of a field investigation which was carried out by a research team from the University of Illinois at Chicago starting in September 1993 and concluding in May 1995. Selected bridges were inspected in Illinois, Connecticut, Virginia, Maryland, Iowa, California, New York, Alaska, Ohio, Pennsylvania, and Washington, D.C. The objective of this evaluation phase is to determine the structural behavior of full depth precast, prestressed concrete panels for bridge deck replacement.

The field investigation began in Illinois with a visit to the Bayview Bridge in Quincy. A visual inspection of the bridge was conducted with help from the Illinois, Missouri and Federal DOTs. The visit to Connecticut involved the visual inspection of Bridge 03200 in Waterbury, where a general inspection of the bridge was made.

The field investigation process continued in Virginia and included visits to two bridges in Culpeper and Fairfax. In both cases, engineers from each respective district of the Virginia DOT accompanied the team on the site visit. In addition, other visits were conducted in order to examine the types of overlays being implemented in Virginia. A major bridge [5 miles (3 km) long] was then inspected in Maryland.

Two inspection visits were made to the Burlington Bridge in Iowa in an effort to witness the construction work. A visual inspection of the bridge was conducted while the second half of the bridge was still under construction. The Seneca Bridge in Illinois was also inspected along the guidelines set forth by the sponsors.

Two bridges were selected in California. The High Street Overhead Separation Bridge is fracture critical and will be replaced in the near future while the Oakland-San Francisco Bay Bridge is in fair condition; however, the structure is currently being retrofitted for earthquakes.

The inspection process resumed with three bridges that are maintained by the New York State Thruway Authority. After conducting a visual inspection of these bridges, it was observed that the transverse joints were in unsatisfactory condition with evident leaking. The process of bridge deck investigation continued through the inspection of five more bridges that are maintained by the New York DOT. The bridges maintained by the New York DOT were in the same condition as those maintained by the New York State Thruway Authority. It was noticed that low volume bridges that were designed to carry moderate loads were in better condition than bridges that were carrying high traffic volumes.

Several bridges in Alaska were also inspected. These inspections included the visual inspection and investigation of 19 bridges maintained by the Alaska Department of Transportation and Public Facilities.

The inspection process began with the Chulitna Bridge. The visual inspection revealed that the transverse joints were leaking. The process of bridge deck investigation continued through the inspection of 18 more bridges that are located on the Dalton Highway. This highway is unpaved so the deck surfaces were not overlaid. The biggest cause of any adverse effects was the lack of post-tensioning in the longitudinal direction of the structure to secure the tightness of the joints.

The investigation resumed in Ohio and Pennsylvania where five bridges were inspected. The inspection process began with the Dublin 0161 Bridge in Columbus, Ohio. This bridge is unique in that its supporting system is a reinforced concrete arch with cross beams. This structure is very stiff and no major problems were encountered with the bridge deck panels.

The investigation continued with four bridges in Pennsylvania. The type of connection used in the bridge played a major role in evaluating the

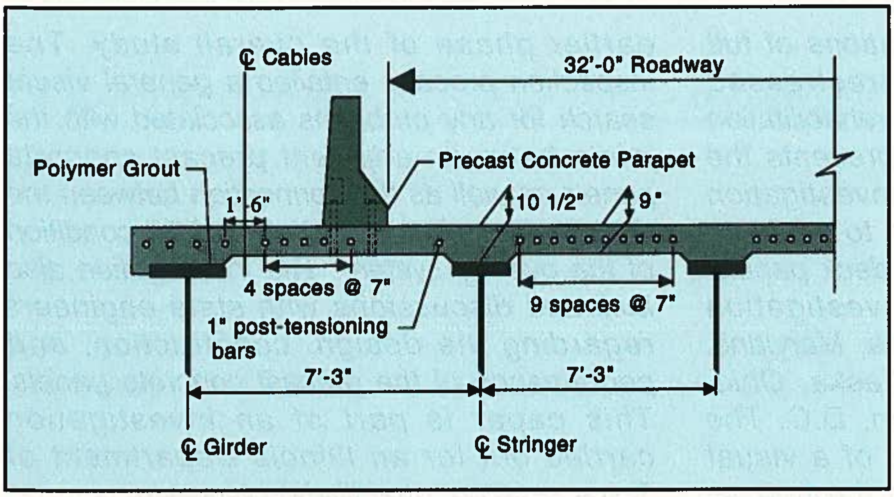


Fig. 1. Precast concrete deck section of Bayview Bridge, Quincy, Illinois.


Fig. 2. Condition of bridge deck viewed from below, Seneca Bridge, Lasalle County, Illinois.

performance of these bridges. It was found that in some of the bridges, the deck and the supporting structure were not acting compositely and, as a result, many problems were detected at critical locations in the deck.

The overall field investigation will conclude in May 1995, when selected bridges will be visited in Alabama, Maine, Massachusetts, and Toronto, Canada.

ILLINOIS DOT

Two of the bridges inspected in Illinois, the Bayview Bridge and the Seneca Bridge, are under the jurisdiction of the Illinois Department of Transportation.

Bayview Bridge over the Mississippi River, Quincy

The investigation of this bridge concentrated on structural performance. The main areas of investigation were the precast decks, joints between adjacent precast panels and connections between the slab, and supporting system.

The Bayview Bridge was built in 1986 and opened to traffic in 1987. It is located on the Illinois-Missouri border. The bridge was temporarily closed to traffic for 2 months due to the Mississippi River flood in 1993. Mainly affected were the approach spans, especially those on the Missouri side.

The structure consists of 14 continuous approach spans and two simple transition spans. The deck for the approach spans is overlaid with 9 in. (229 mm) thick cast-in-place concrete. The main river structure consists of a three-span cable-stayed bridge unit with a 9 in. (229 mm) thick precast concrete deck, covered with a $1^3/_4$ in. (44.5 mm) waterproofed bituminous wearing surface.

The full width, full depth precast concrete deck panels are 46 ft 6 in. (14.2 m) wide and vary from 9 to 11 ft (2.7 to 3.4 m) in length. Post-tensioning bars, spaced at 7 in. (178 mm), had an initial tensioning stress of 105 ksi (723 MPa). A minimum concrete strength of 3500 psi (24.1 MPa) was specified. Three to five panels are post-tensioned to form a group. Fig. 1 shows a section of the precast concrete deck.

The investigation was chiefly concentrated on the main river spans. It was found that one side of a precast concrete span experienced debonding. The bituminous sand-seal used was acting as a leveling binder. As a result, a $^1/_2$ in. (12.7 mm) layer of a bituminous compound was placed.

In general, the decks are performing satisfactorily. Also, no problems were encountered in the joints, although some rusting is visible, indicating leaking had occurred. The butt joints between adjacent precast concrete panels are performing adequately and no cracking or leaking is apparent. It is believed that this type of joint is satisfactory for the existing bridge system because the cable-stayed precast, prestressed deck is in compression.

Seneca Bridge, Lasalle County

The Seneca Bridge was built in 1932 and consists of 13 spans. The total span length is 1510 ft 3 in. (460 m). Spans 1 through 5 and 10 through 13 are approach spans, and Spans 6 through 9 are interior truss spans. In 1986, the existing concrete bridge deck was replaced with a $6^1/_2$ in. (165 mm) thick precast, prestressed concrete slab deck. All the precast concrete panels are match cast with epoxy adhesives in between the units. The deck replacement was performed in sections.

Full two-way traffic was maintained throughout the construction in accordance with special outlined provisions. Bridge closure was permitted in 10-hour periods, Sunday through Thursday, from 7:00 p.m. to 5:00 a.m.

The precast concrete panels used have a concrete compressive strength of 5000 psi (34.5 MPa). A 2 in. (51 mm) thick minimum Class I concrete overlay, with a waterproofing membrane system, covers the panels. The existing beams are spaced 5 ft 6 in. (1.7 m) apart. The connection between the precast concrete deck and supporting system varies in accordance with the type of span. Two high strength $^3/_4$ in. (19 mm) diameter by 10 in. (254 mm) bolts are used in the shear connector pockets for the approach spans. On the other hand, four high strength $^3/_4$ in. (19 mm) diameter bolts are used for the truss spans.

Smooth prestressed bars 1 in. (25 mm) in diameter, 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), are used. In addition, 1 in. (25 mm) diameter, deformed prestressed bars, Grade 150, initially stressed to 45,000 psi (310 MPa) are used. Eight of these bars are spaced at 2 ft 10 in. (864 mm) across the bridge width.

The bridge deck shows random cracks at the approach spans. The match cast joints between the precast, prestressed panels are leaking, as shown in Fig. 2. This type of joint is not effective in this type of construction. The limited contact area does not allow enough bond area in the joint for the grout. Signs of corrosion are also apparent on the steel supporting system.

CONNECTICUT DOT

The 03200 Waterbury Bridge, under the jurisdiction of the Connecticut Department of Transportation, was built in 1965 and reconstructed in 1989. The 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. Between the piers, the girders are straight.

Because the bridge is only 27 ft 6 in. (8.4 m) wide, full width precast concrete panels 8 ft (2.4 m) wide, 26 ft 8 in. (8.1 m) long, and 8 in. (203 mm) deep are used. The shear connector blockouts are rectangular 18 x 5 in. (457 x 127 mm) at the top and trapezoidal from top to bottom. The spacing for these blockouts is 2 ft (610 mm) on center for each slab. Three $^7/_8$ in. (22 mm) welded stud shear connectors are placed in each blockout. A standard shear key configuration (female-to-female type) filled with high strength non-shrink grout is used for the transverse joints.

An arbitrary stress of 150 psi (1.0 MPa) is used for the simple spans. This was significantly increased to 300 psi (2.1 MPa) in the three-span continuous portion of the bridge to account for the unusually large composite dead load and live load stresses. The amount of

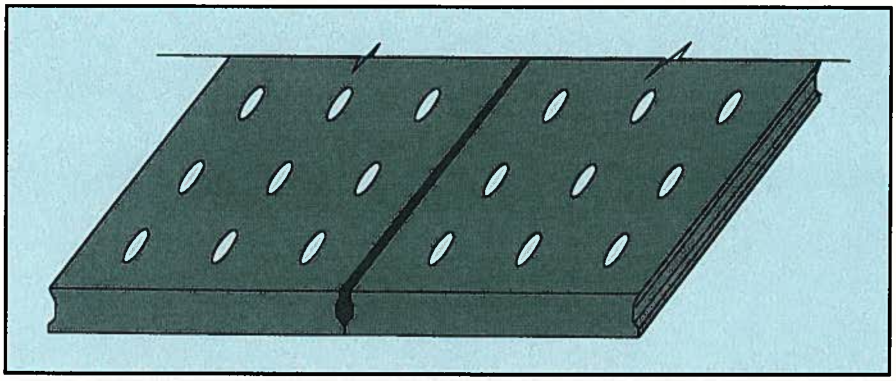


Fig. 3. Layout of precast concrete panels for Route 229 Bridge, Culpeper, Virginia.

Fig. 4. General view of William Preston Jr. Memorial Bridge, Chesapeake Bay, Maryland.

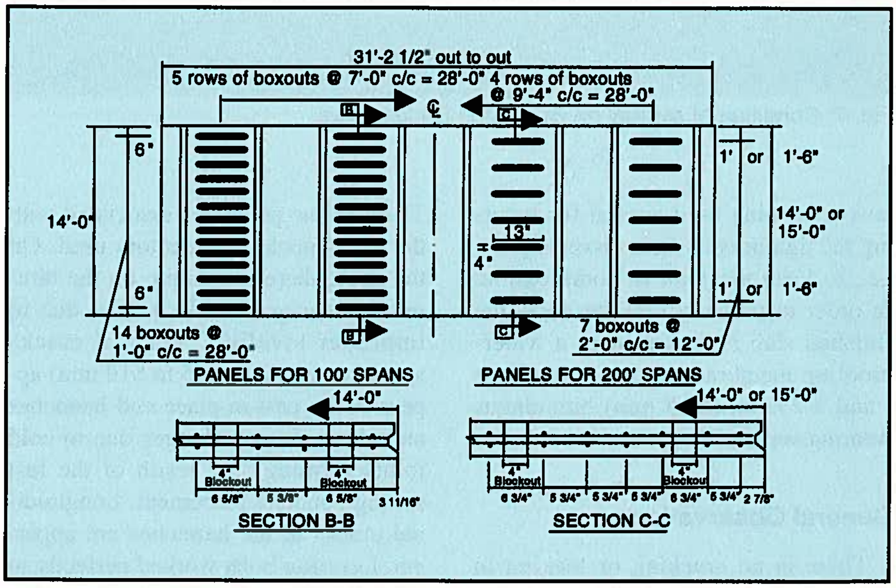


Fig. 5. Panel layout for William Preston Jr. Memorial Bridge, Chesapeake Bay, Maryland.

Fig. 6. Cracking and leaching in William Preston Jr. Memorial Bridge, Chesapeake Bay, Maryland.

Fig. 7. Condition of overlay on Burlington Bridge, Iowa.

post-tensioning used is vital for securing the tightness in the transverse joint, i.e., to keep the joint in compression. In order to properly seal the deck, the finished slab is topped with a waterproofing membrane system with Class I and a 2½ in. (63.5 mm) bituminous wearing surface.

General Observations

There is no cracking or leaking in the deck, indicating that the transverse joints are performing satisfactorily. There are no problems associated with the shear pocket connectors used. On the deck, there is a hump on the bituminous surface, at the joints, due to improper leveling. Vertical cracks spaced at 1 to 2 ft (305 to 610 mm) appear in the cast-in-place end haunches and along the top flanges due to cold joints forming as a result of the fast setting concrete placement. Longitudinal cracks at the haunches are apparent. Leveling bolts worked perfectly to solve the vertical differential problem. A flexible bituminous mixture was used where no leakage was reported. Some girders experienced section loss. However, in general, the bridge is performing satisfactorily.

Cost Analysis

Based on information provided by the bridge engineers at the Connecticut DOT, the following conclusions can be drawn:

1. The average cost for cast-in-place construction is about $45 per sq ft ($484 per m^2) (including demolition, parapets, and wearing surface).

2. The bid for the project was relatively high because of unforeseen construction costs as well as the penalty imposed ($5000 per day) when construction went beyond the allotted time for the completion of the project.

3. The costs for two similar spans, one using a cast-in-place replacement for 500 ft (152 mm) spans and the other implementing precast concrete replacement for 700 ft (213 m) spans, were $71 per sq ft and $75 per sq ft ($764 and $807 per m^2), respectively.

4. The cost on another bridge (Seymour Bridge) at the time of inspection was estimated at $30 per sq ft ($323 per m^2).

5. Precast concrete replacement is feasible for large scale projects.

VIRGINIA DOT

The Routes 229 and 235 bridges are under the jurisdiction of the Virginia Department of Transportation.

Route 229 Bridge Over Big Indian Run, Culpeper

This bridge was built in 1941 and rehabilitated in December of 1985. In the rehabilitation operation, precast concrete panels were installed on the bridge's existing steel beams. A detailed description of this bridge is provided by Issa et al.[1-2]

Construction was accomplished in two phases in order to allow traffic flow to continue without interruption. The bridge is currently in good condition. No leaking is apparent through the joints. A female-to-female type of shear key is used. The panels are in contact at the bottom part of the joint (see Fig. 3).

This is a standard configuration for slabs in Virginia. A 10 in. (254 mm) wide layer of Class II waterproofing membrane covers the joints as a precautionary measure to prevent any leakage. Nevertheless, it is recommended that this type of joint have at least a $^{1}/_{4}$ in. (6.4 mm) opening at the bottom to allow for any misalignment or dimensional growth of panels.

The precast concrete panels are not post-tensioned longitudinally to secure the tightness of the joints. Uniform transverse cracks spaced approximately 1 ft (0.305 m) apart can be seen near the interior girder. Also, there is some leaching at the ends. The overlay is not in very good condition because it was not replaced.

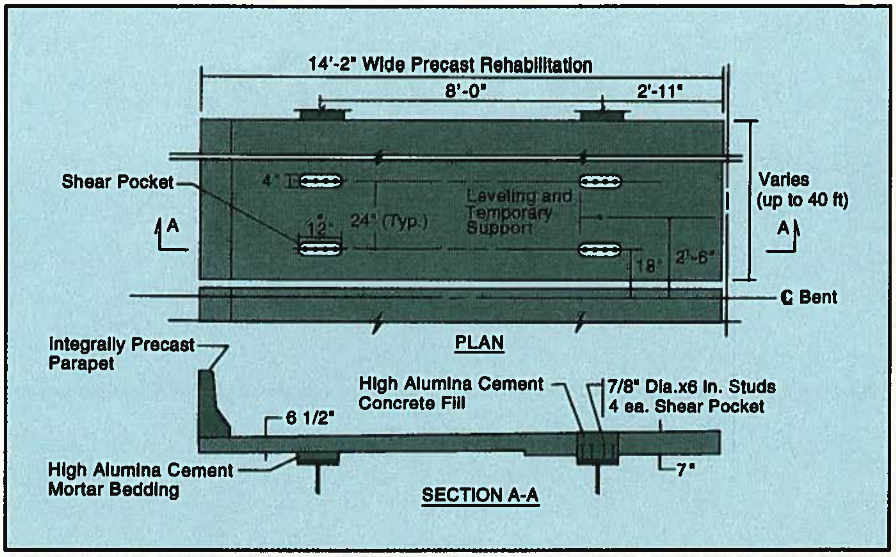


Fig. 8. Typical precast concrete deck panel for High Street Overhead Separation Bridge, California.

Route 235 Bridge Over Dogue Creek, Fairfax

This bridge was built in 1932 and rehabilitated in 1982. For the past 13 years, no major repairs have been made to the bridge. A detailed description of the bridge is included in Refs. 1 and 2. The structure is rated fracture critical. The precast concrete panels are not prestressed and the panels are not post-tensioned longitudinally.

There is some leakage at the joints, which had the same configuration as the previous bridge (Route 229 bridge). However, Class I waterproofing covers the entire deck, without any consideration given to the location of the joints. Cracking and rusting can be observed from underneath the bridge, especially at the joints between the precast panels.

For the past 10 years, no major rehabilitation has been performed on the bridge. The structure is performing satisfactorily. The overlay, as in the Route 229 Bridge, needs some repair. The wearing surface has transverse and random cracks while the deck has transverse cracks and efflorescence through the deck at the construction joints.

Fig. 9. Condition of deck and supporting system viewed from below, High Street Overhead Separation Bridge, California.

Types of Overlay

The Virginia DOT reports that several types of overlays are currently being used in their rehabilitation projects.

The standard overlay in the state has always been latex modified concrete. Recently, however, EP-5 (epoxy) has been used, along with aggregates of the same size, and applied as an overlay in the range of $1^{3}/_{4}$ to $2^{1}/_{4}$ in. (44.5 to 57 mm) thick.

In other instances, state engineers have specified silica fume as the overlay, where the minimum required thickness is $1^{1}/_{4}$ in. (32 mm). This form of overlay is beneficial and more effective than latex modified concrete. The application cost of silica fume is \$600 per cu yd (\$785 per m^3) while latex modified concrete costs \$900 to \$1000 per cu yd (\$1177 to \$1308 per m^3). Therefore, silica fume is 40 percent more cost effective. Silica fume is also less sensitive to temperature changes than latex modified concrete.

Epoxy Concrete Overlay

Prior to placing the epoxy concrete overlay, the entire deck surface is cleaned by shotblasting or other specified cleaning methods. The purpose here is to remove deteriorated asphaltic material, oils, dirt, rubber, curing compounds, paint carbonation, laitance, weak surface mortar and other potentially detrimental materials that

Fig. 10. Condition of deck and joints in High Street Overhead Separation Bridge, California.

Fig. 11. Section of deck replaced due to earthquake, Oakland-San Francisco Bay Bridge, California.

may interfere with the bonding or curing of the overlay.

The epoxy overlay is applied in two separate courses. The rate for Course 1 requires a minimum of 2.5 gal per 100 sq ft surface area (9.5 liters per 9.3 m²), and application of aggregate at a minimum of 10 lbs per sq yd (5.4 kg/m²). The second rate requires a minimum of 5.0 gal per 100 sq ft surface area (18.9 liters per 9.3 m²), and application of aggregate at 14 lbs per sq yd (7.6 kg/m²).

The epoxy mixture is uniformly applied to the surface of the bridge deck with a squeegee or paint roller. The bridge deck temperature must be above 60°F (16°C) at the time of application. The dry aggregate is then applied to completely cover the epoxy mixture within 5 minutes. The second course of epoxy is then applied over the layer of aggregate. Each course of epoxy concrete overlay is cured until vacuuming or brooming can be performed without tearing or damaging the surface. Traffic or equipment is not permitted on the overlay surface during the curing period.

Type EP-5 is a low modulus patching, sealing, and overlay adhesive with an elongation of at least 10 percent. The first course of application for this material is 1 gal per 75 sq ft (3.8 liters per 7 m²), while the second coat of epoxy resin is 1 gal per 50 sq ft (3.8 liters per 4.6 m²). In between, 11 lbs per sq yd (6 kg/m²) of sand is applied. Brooming is not performed until the epoxy resin has cured sufficiently to prevent tearing.

High Molecular Weight Methacrylate

High molecular weight methacrylate (HMWM) is used for crack sealing and treatment of concrete surfaces. The HMWM can fill cracks ¼ to ½ in. (6.4 to 12.7 mm) in depth and cracks with a greater depth depending on the amount of deleterious material in the crack and the width of the crack. Shotblasting may be necessary to clean the decks prior to placing the HMWM. Application of this polymer material usually requires a lane closure of up to 24 hours. This material is used on decks with a tined texture and cracks that are so numerous and randomly oriented that grouting and sealing or epoxy injection are not practical.

Silica Fume Concrete

Silica fume is a very fine material consisting primarily of noncrystalline pozzolanic silica produced by electric arc furnaces as a by-product of the production of metallic silicon or ferrosilicon alloys; it is also known as condensed silica fume or microsilica. If the overlay is to be placed on newly cast concrete with a surface that is clean and free of curing compound or other chemicals, light sandblasting or shotblasting is required to remove the laitance. The overlay should not be placed until the new concrete has attained at least 90 percent of its design strength.

The surface of the base concrete should be in a saturated surface dry condition during placement of the overlay and should be wetted at least one hour before placement of the overlay. The cleaned and wetted surface is covered with a plastic to pre-

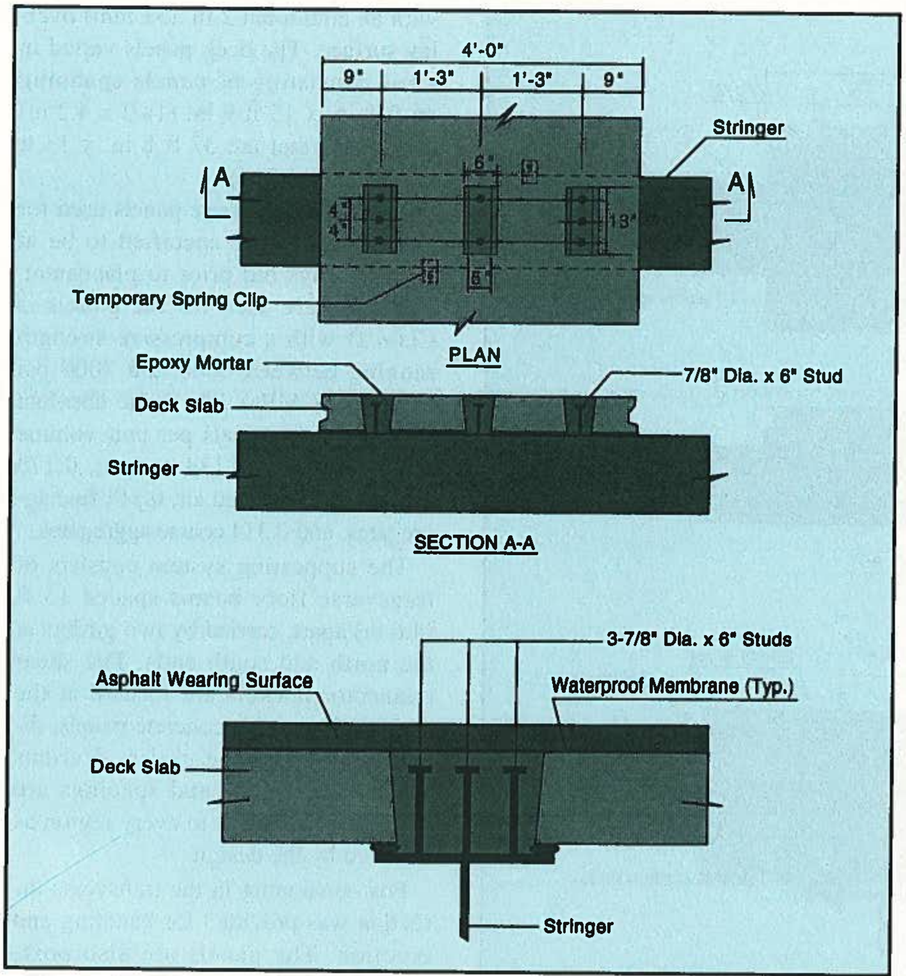


Fig. 12. Plan and section details of welded stud connection in Krumkill Road Bridge, Albany County, New York.

vent contamination prior to placement. Silica fume concrete is brushed on the surface and excess aggregate is discarded just prior to placement. The minimum thickness of the bridge deck overlay must not be less than 1$^1/_4$ in. (32 mm). Silica fume concrete does not bleed as much as normal concrete and is more susceptible to plastic shrinkage cracking.

Latex Modified Concrete

Latex modified concrete (LMC) achieves a compressive strength of 3000 to 3500 psi (20.7 to 24.1 MPa) within 2 to 3 days, which is sufficient for traffic in the case of overlays. For applications requiring high early strength, Type III cement can be used. The smaller particles associated with Type III cement react quickly in LMC and permit LMC overlays to be opened to traffic in 24 hours, with no loss in the ultimate properties of the concrete.

At one-day cure, the tensile strength of the LMC bond exceeds 100 psi (689 kPa). The normal curing procedure for LMC is one day of moist cure followed by air drying for the remainder of the cure time. The low permeability of LMC contributes to the impermeability of the cured concrete and mortar by resisting infiltration of moisture and gases. LMC produces concrete with a modulus of elasticity that is 15 percent lower than comparable conventional concrete, i.e., LMC can take more strain and is not as brittle as conventional concrete.

MARYLAND TRANSPORTATION AUTHORITY

The William Preston Jr. Memorial Bridge (Bay Bridge) over the Chesapeake Bay is under the jurisdiction of the Maryland Transportation Authority. The bridge was built in 1952 and consists of two lanes in each direction.

Fig. 13. Fracture and spalling at transverse joint in Krumkill Road Bridge, Albany County, New York.

A general view of the bridge is shown in Fig. 4. The deck for most of the spans was replaced with precast concrete panels that vary in sizes in order to fit the geometric requirements. The bridge was completely closed to traffic for 6 months in order for the replacement process to begin.

The panel layout and configuration of the pockets are shown in Fig. 5. Each span consists of four full width precast concrete panels. The overlay for the deck consists of a 2 in. (50.8 mm) layer of latex modified concrete in addition to the 6 in. (152 mm) deck.

The deck was inspected from the top surface and from underneath using a catwalk positioned directly below the deck. Diagonal and map cracking occur on both sides of the deck. The diagonal cracking is attributed mainly to handling and the absence of any prestressing tendons in the precast concrete panels. The panels are post-tensioned in the longitudinal direction to secure the tightness of the joints.

At one location on the deck, the latex modified concrete did not adhere at the joints between the precast concrete panels and the supporting system, causing popouts in the concrete. The engineers at the site also noticed corrosion problems associated with excessive chloride contents. The top 2 in. (50.8 mm) layer of the deck has an excess of chlorides, prompting the re-

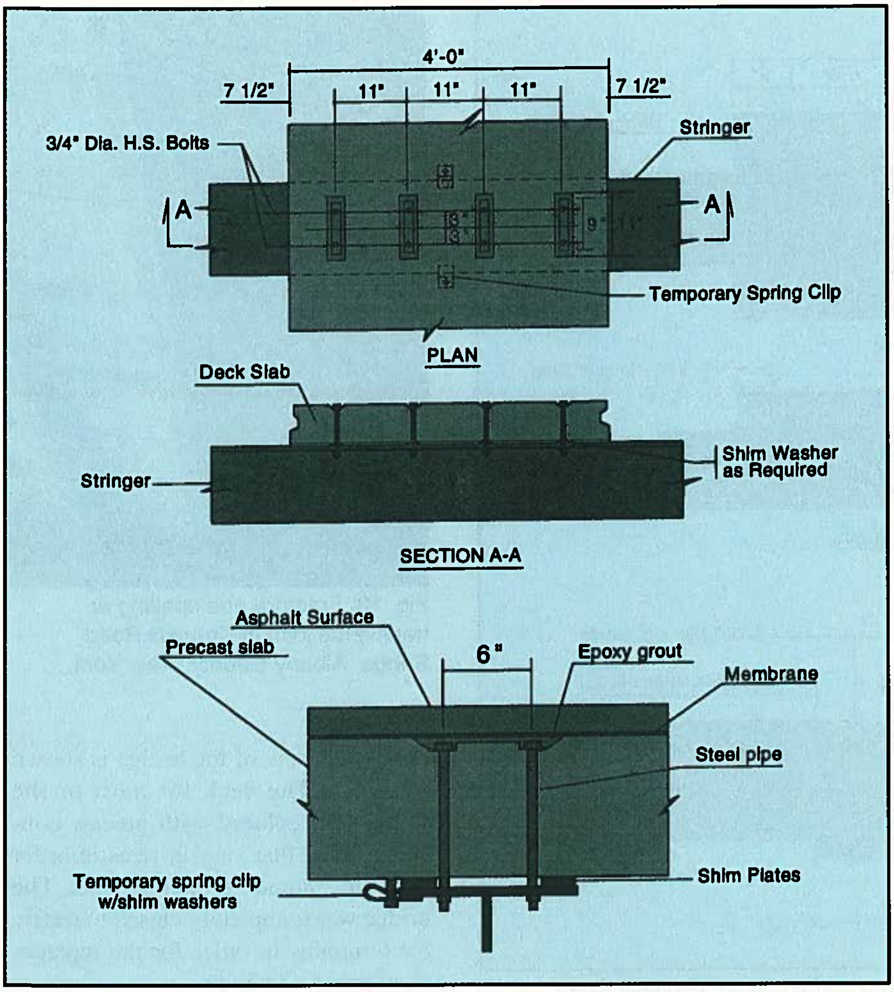


Fig. 14. Details of bolted connection in Amsterdam Interchange Bridge, Montgomery County, New York.

moval (milling) of the top 2 in. (50.8 mm) of the deck and replacing it with latex modified concrete.

Leaching through the joints between precast concrete panels is a major problem; numerous locations underneath the deck show deposits and stains (see Fig. 6). This problem is attributed to the type and configuration of joint used. The closed joint end (bottom) does not allow for any size irregularities in the panels or any dimensional growth. The problem is currently being treated by patching the openings in the joints with a caulking material.

There is also some spalling, in addition to some exposure of steel. Furthermore, signs of corrosion are noticeable below the deck.

IOWA DOT

The Burlington Bridge is under the jurisdiction of the Iowa Department of Transportation. The bridge is located on the Illinois-Iowa border and provides vehicular passage across the Mississippi River. The planning process for construction of the cable-stayed Burlington Bridge started in 1990. It began with the construction of the tower supporting the cables while construction of the precast, prestressed concrete deck panels was initiated in the spring of 1992. One-half of the bridge was completed and opened to traffic in October 1993. The other half of the bridge was opened to traffic in August 1994.

The structure consists of two spans, 660 and 405 ft (201 and 123 m), while the entire width of the bridge is 87$^1/_2$ ft (26.7 m). The deck for the approach spans is 10 in. (254 mm) thick cast-in-place concrete. The main river structure consists of two cable-stayed spans with a precast concrete deck, covered with a 2 in. (51 mm) layer of low slump dense concrete. The precast concrete deck is 10 in. (254 mm) thick with an additional 2 in. (51 mm) overlay surface. The deck panels varied in sizes consisting of panels spanning 46 ft 8 in. x 13 ft 9 in. (14.2 x 4.2 m) and panels that are 37 ft 8 in. x 13 ft 9 in. (11.5 x 4.2 m).

The precast concrete panels used for construction were specified to be at least 60 days old prior to placement. The concrete used for the panels is Class D with a compressive strength ranging between 6000 and 7000 psi (41 and 48 MPa). The basic absolute volumes of materials per unit volume of concrete are: 0.134 cement, 0.178 water, 0.06 entrained air, 0.314 fine aggregates, and 0.314 coarse aggregates.

The supporting system consists of transverse floor beams spaced 15 ft (4.6 m) apart, carried by two girders at the north and south ends. The shear connector pockets are located at the ends of the precast concrete panels, directly over the edge girders. Various types of locations and spacings are used, corresponding to every region as required by the design.

Post-tensioning in the transverse direction was provided for handling and erection. The panels are also post-tensioned in the longitudinal direction with an initial post-tensioning force of 89 and 166 kips (396 and 739 kN) for the 1 and 1$^3/_8$ in. (25 and 355 mm) diameter thread bar, respectively. The 1 in. (25 mm) diameter post-tensioning bars, spaced at 1 ft 4$^3/_4$ in. (425 mm), are used in all typical panels while the 1$^3/_8$ in. (35 mm) diameter bar is only used for some panels, as imposed by the design requirements. The post-tensioning process involved stressing three panels at each interval.

The joints between adjacent precast panels are 15 in. (381 mm) and are filled with cast-in-place concrete. The shear pockets and transverse and longitudinal joints between the precast panels are grouted with Class D concrete. Type III cement is used for the cast-in-place joints. The space under the slab units, i.e., between the slab panels and the top of the edge girders as well as between the slab panels and floor beams at the leveling devices, is filled with high flow, high strength, early loadbearing, non-shrink grout.

The 2 in. (51 mm) overlay used on the bridge is low slump, dense Class O concrete, with basic absolute volumes of materials per unit volume of concrete as follows: 0.156 cement, 0.16 water, 0.06 entrained air, 0.312 fine aggregates, and 0.312 coarse aggregates. Because of superimposed loads, such as machines causing vibrations on the deck, fatigue cracking occurred. These cracks were sealed with methacrylate. The poor condition of the overlay is depicted in Fig. 7.

A corrosion inhibitor admixture (DCI) was incorporated into the concrete used to fabricate the precast concrete deck panels [142 panels, containing 2557 cu yd (1955 m³)], cast-in-place deck units, closure panels [839 cu yd (641 m³)], concrete median, barrier rails [467 cu yd (357 m³)], and the shear connector pockets in the precast slabs [24 cu yd (18 m³)]. The corrosion inhibitor admixture is a solution of 29 to 32 percent by weight of calcium nitrite and water with a unit weight of at least 10½ lbs per gallon (1.26 kg/liter).

In general, the decks on the completed side of the bridge are performing well. Also, the project engineer at the site indicated that no major problems have been encountered with the joints. No leaking was observed in the precast concrete deck because longitudinal and transverse post-tensioning kept the bridge deck in compression.

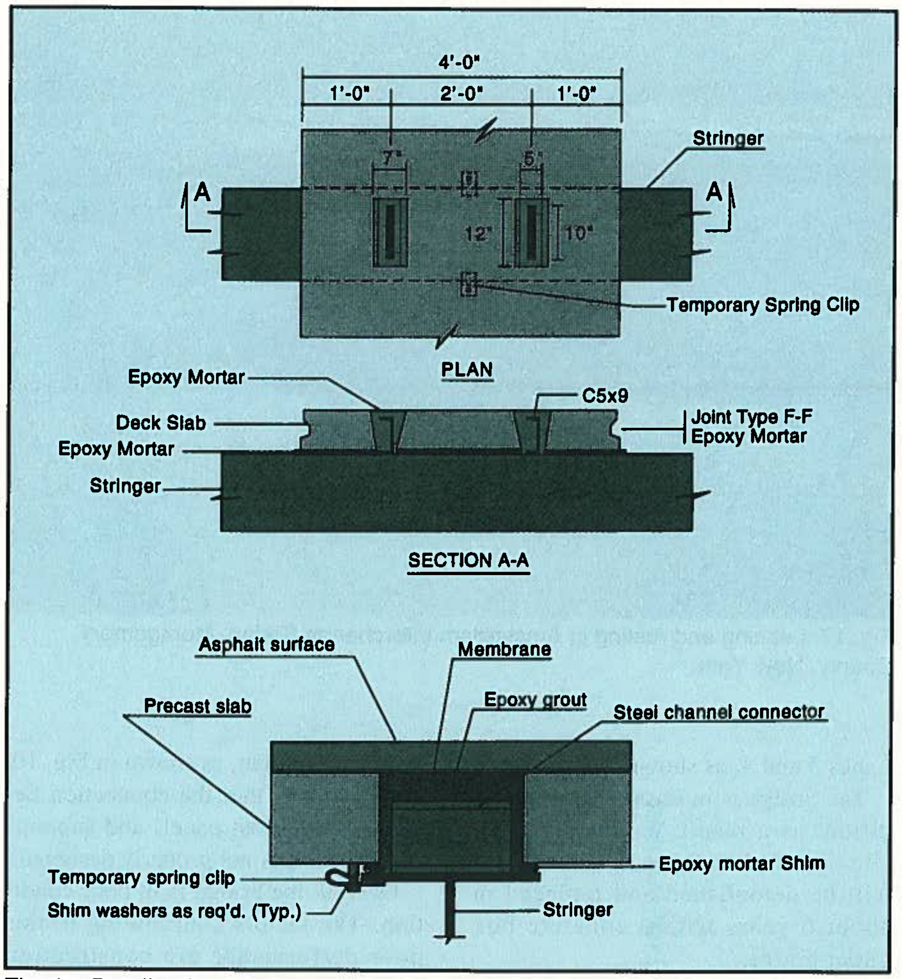


Fig. 15. Details of welded connection in Amsterdam Interchange Bridge, Montgomery County, New York.

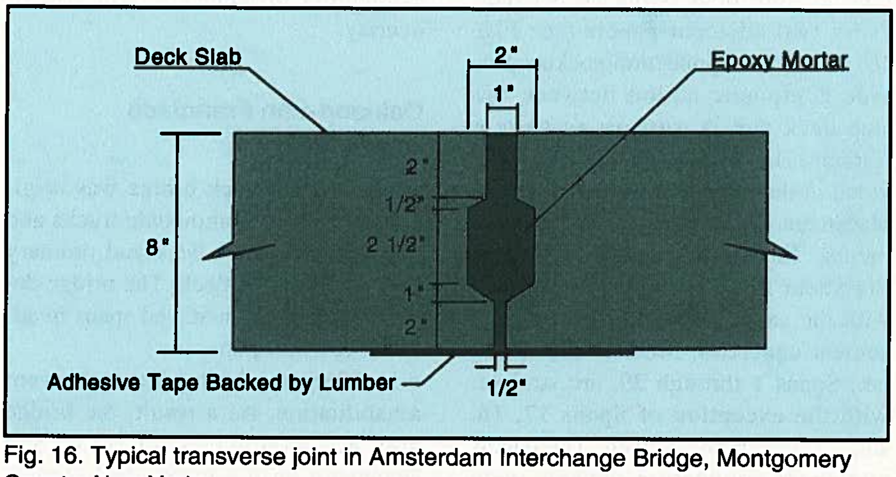


Fig. 16. Typical transverse joint in Amsterdam Interchange Bridge, Montgomery County, New York.

CALTRANS

The High Street Overhead Separation Bridge and the Oakland-San Francisco Bay Bridge are under the jurisdiction of the California Department of Transportation (CALTRANS).

High Street Overhead Separation Bridge

This bridge was widened on the left and right sides in 1955 and 1963, respectively. The structure currently consists of four lanes. The fourth lane on Spans 1 through 29 of the 30-span bridge was replaced with precast concrete deck panels in 1978 (see Fig. 8). Traffic flow was maintained during the rehabilitation process. The slab units were placed in direct contact with the girders. No haunches were provided to allow for any dimensional irregularities and expansion. Longitudinal joints are provided between the slabs of Lanes 3 and 4 in Spans 1 through 29 of the bridge.

The structure is rated fracture critical. The deck (mainly the cast-in-place deck), shown from the underneath in Fig. 9, exhibits cracking, leaking, leaching, and rusting. The deck has transverse cracks in addition to radial cracks emanating from the 12 x 4 in. (305 x 102 mm) shear connection pockets, as shown in Fig. 10. Typically, these blockouts are spaced 2 ft 6 in. (762 mm) apart. Spalling is also apparent, particularly between

Fig. 17. Leaking and rusting in Amsterdam Interchange Bridge, Montgomery County, New York.

Lanes 3 and 4, as shown in Fig. 10.

The bridge is in unsatisfactory condition; as a result, engineers at the site stated that the entire structure will be demolished and replaced in about 6 years with a concrete box girder bridge.

Post-tensioning was not provided to secure the tightness of the joints between adjacent panels. A 9 in. (229 mm) closure pour is placed between every two adjacent panels (see Fig. 10) while stud connection pockets provide composite action between the slab deck and its supporting system (girders). Leveling bolts are also provided in the panels to allow for proper placement of the precast concrete elements. The closure pours as well as the shear stud pockets are grouted with the same material (high alumina cement concrete). Most of the spans, i.e., Spans 1 through 29, are straight, with the exception of Spans 17, 18, and 19, which are skewed. The panels and shear connection pockets were prepared to account for these structural requirements.

Prior to placement of the new deck in 1978, the overlay was stripped off the entire bridge (in the vicinity of Spans 1 through 29) and has not been replaced. As a result, cracking problems are clearly evident in the deck.

We were informed that over the past 6 months, the shear stud pockets have been popping out, as shown in Fig. 10. This suggests that the connection between the precast panels and supporting system was not properly designed.

Overall, the bridge is in poor condition. The factors contributing to this poor performance are construction procedures and type of design (e.g., no post-tensioning), type of connection between precast concrete panels, unavailability of haunches, and lack of overlay.

Oakland-San Francisco Bay Bridge

This double deck bridge was originally built to accommodate trucks and trains on the lower deck and ordinary cars on the upper deck. The bridge design includes cable-stayed spans in addition to truss spans.

In 1960-61, the bridge underwent rehabilitation. As a result, the bridge now accommodates traffic to San Francisco on the upper deck and to Oakland on the lower deck. Trains no longer have access to the bridge because the right two lanes of the lower deck were converted for regular traffic. These two lanes were replaced with precast concrete deck panels (lightweight concrete). The bridge deck was originally surfaced with epoxy asphalt pavement in 1964 as part of the reconstruction of the bridge. Because of wear, the deck was resurfaced with epoxy asphalt in 1974 (upper deck) and 1977 (lower deck).

The deck shows some cracking and leaching; however, the cast-in-place areas of the deck exhibit more cracking and leaching than the precast areas. The cast-in-place top deck shows widespread spalling below the deck.

In 1989, a severe earthquake (Loma Prieta earthquake) hit the area. As a result, a small section of the lower deck of the bridge was damaged and the entire width in that section (see Fig. 11) was replaced with precast concrete panels. However, that area has not been overlaid. Closure pours [12 in. (305 mm)] are provided between adjacent precast concrete elements. The bridge was closed for a period of one month while construction took place on the deck.

Every weekday one lane of the bridge is closed (either the far right lane or the far left lane) as part of the bridge maintenance program (see Fig. 11). Closures for the lower deck take place between 8:00 a.m. and 2:30 p.m., while the upper deck closures occur between 10:30 a.m. and 2:30 p.m. On Fridays, only one lane of the lower deck is closed.

NEW YORK STATE THRUWAY AUTHORITY

The Krumkill Road Bridge, Amsterdam Interchange Bridge and the Harriman Interchange Bridge are under the jurisdiction of the New York State Thruway Authority.

Krumkill Road Bridge, Albany County

This 50 ft (15.2 m) long single-span, six-lane mainline throughway bridge spans over Krumkill Road in Albany County. The bridge consists of two structurally separate spans supported on common abutments. Each structure carries two active traffic lanes. The remaining lane will be used when widening of the bridge is required. This extra lane was effectively used to detour traffic during construction.

To make the deck fully composite with the structural steel, welded headed studs were provided. Fig. 12

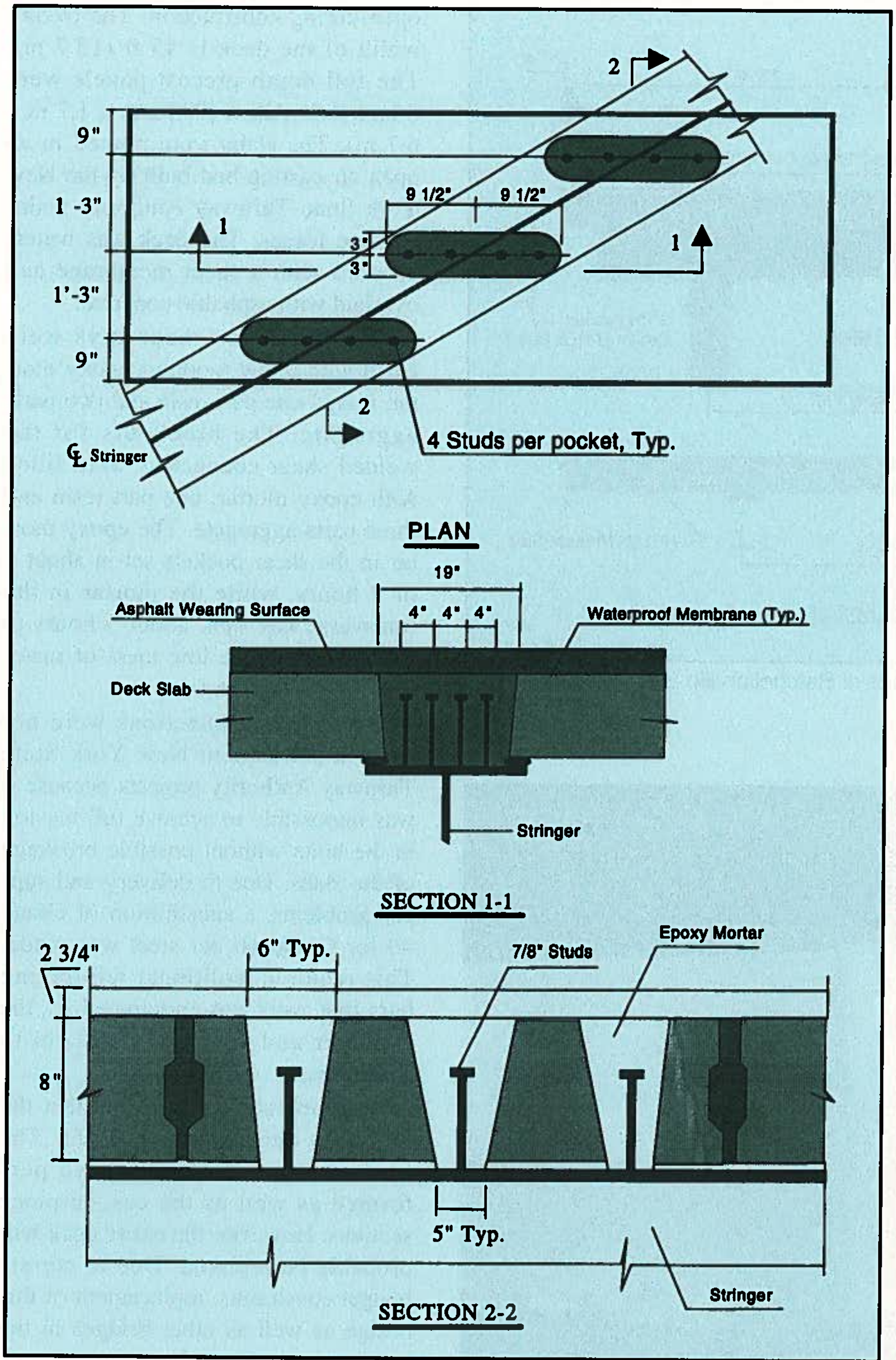


Fig. 18. Plan and section details of blockouts and transverse joints in Harriman Interchange Bridge, Orange County, New York.

Fig. 19. Randon cracking and spalling in Harriman Interchange Bridge, Orange County, New York.

Fig. 20. Minor leaking in Vischer Ferry Road Bridge, Schenectady County, New York.

shows the plan and section details of the welded stud connection. Precast concrete panels, $7^1/_2$ in. (191 mm) thick and 5 ft 2 in. (1.6 m) long, of two different widths, were used. The 42 ft (12.8 m) wide panels were placed over six stringers and the 21 ft (6.4 m) wide panels were placed over three stringers. A 3 ft (914 mm) wide cast-in-place longitudinal joint was provided over continuous reinforcing bars extending from the adjacent panels. The deck is overlaid with a membrane and 6 in. (152 mm) of asphalt. Cracks over the reinforcing bars were detected in the precast concrete panels during construction; these cracks were subsequently sealed with epoxy.

The most noticeable problem with the bridge is the fracture and spalling at the transverse joints, as shown in Fig. 13. For safety purposes, the chunks of concrete from the spalling were removed to protect the roadway underneath. The fracture and spalling of the concrete deck is mainly attributed to the absence of any post-tensioning in the longitudinal direction. As a result, the joints are not tight and leakage occurs regularly.

The beams show signs of rusting as a result of the leakage through the transverse joints. Cracks are also apparent in the overlay.

Amsterdam Interchange Bridge, Montgomery County

This bridge was set up as an experimental project during the fall of 1973 and the spring of 1974. The bridge

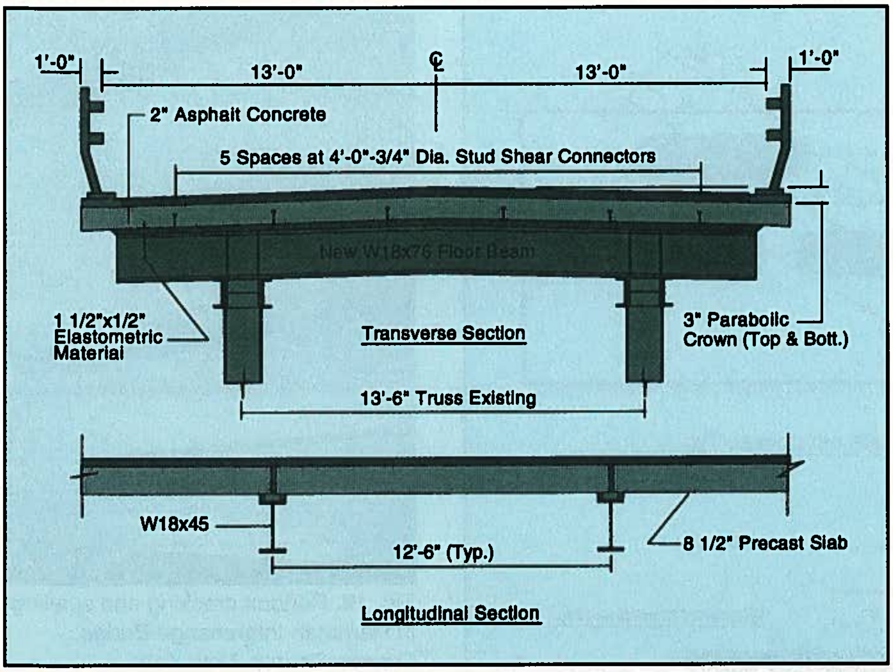


Fig. 21. Transverse and longitudinal sections of Batchellerville Bridge, Saratoga County, New York.

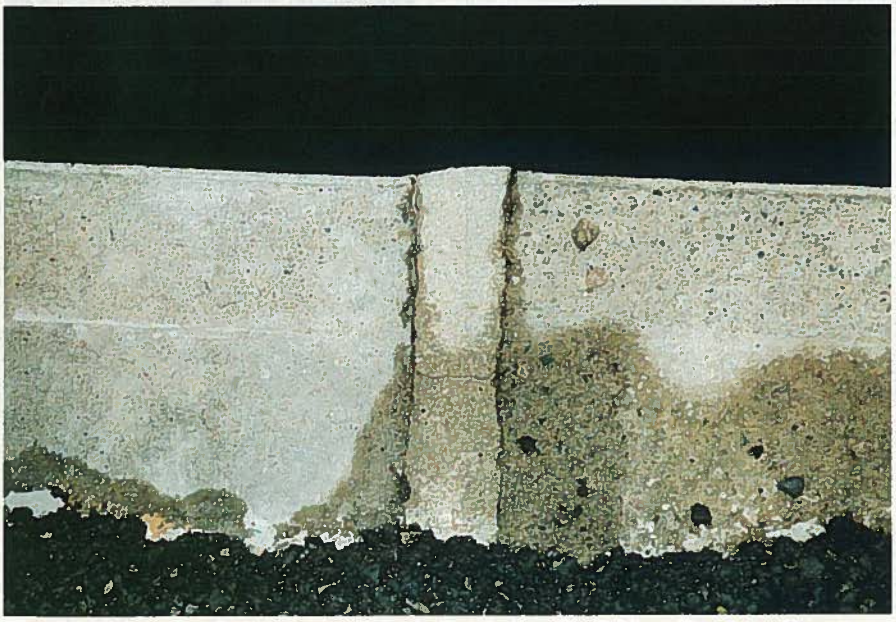


Fig. 22. Debonding in transverse joint in Batchellerville Bridge, Saratoga County, New York.

was originally built in 1954. The objective of this prototype project was to evaluate the effectiveness of both welded and bolted connections that were designed to act compositely with the steel girders. The bridge has two lanes consisting of four spans: 33, 59, 66, and 60 ft (10, 18, 20, and 18 m) long, respectively. A cast-in-place concrete slab was used to replace the deteriorated deck.

The precast concrete panels were installed on only one-half of Span 2 due to constraints on the availability of resources and weather. Seven panels were placed in each lane, three using bolted connections and four with welded connections. Details of both types of connections are shown in Figs. 14 and 15.

Fig. 16 shows the configuration of the panel-to-panel joints. A staged construction sequence was used to maintain at least one lane of traffic open during construction. The overall width of the deck is 45 ft (13.7 m). The full depth precast panels were 8 in. x 4 ft x 22 ft (203 mm x 1.2 m x 6.7 m). The slabs were poured in an open air casting bed built by the New York State Thruway Authority maintenance forces. The deck was waterproofed with a sheet membrane and overlaid with asphaltic concrete.

The transverse shear keys were filled with a low modulus epoxy mortar, mixed one part resin and two parts aggregate. The blockouts for the welded shear connectors were filled with epoxy mortar, one part resin and three parts aggregate. The epoxy mortar in the shear pockets set in about 1 or 2 hours, while the mortar in the transverse key took about 5 hours to set because of the low mass of material in the long thin joint.

The bolted connections were not used in subsequent New York State Thruway Authority projects because it was impossible to achieve full tension in the bolts without possible breakage of the slabs. Due to delivery and supply problems, a substitution of Grade 40 for Grade 60 bar steel was made. This required additional reinforcing bars that were not anticipated by the designer and crowded the form in some areas.

Engineers at the site stated that the bridge is due for replacement. The precast concrete panels have performed as well as the cast-in-place sections. However, the entire deck will probably be replaced. Due to current budget constraints, replacement of this bridge as well as other bridges in the area has been delayed.

Major problems include spalling, cracking, and leaking. Rusting has occurred as a result of the leaking, as shown in Fig. 17. All the major problems appear to be initiated at the joints. Post-tensioning in the longitudinal direction was not provided to secure the tightness of the joints.

An inspection of the bridge in August 1993 revealed that the joint and transverse cracks near the joint in Spans 1 to 3 were filled with a hot poured type of sealer material and that no evidence of any recent joint leakage was reported thereafter. All the spans exhibited $^1/_2$ to 1 in. (12.7 to 25 mm) wide transverse cracks at both sides of the joint in the wearing surface. Also, 1/8 to 1/4 in. (3.2 to 6.4 mm) wide longitudinal cracks were observed in the middle of the bridge running the entire span length. The asphalt shows minor rutting with minor random cracking.

Harriman Interchange Bridge, Orange County

This structure is a three-span [each 75 ft (22.9 m) long], two-lane ramp. The connection details are similar to those of the Krumkill Road Bridge. The roadway is on both vertical and horizontal curves. Because this is a curved, super-elevated bridge, the precast panels are skewed and are not level on the beam flanges. Therefore, the epoxy mortar bed is thicker on one edge of the flange than the other. Fig. 18 shows the plan and section details of the blockouts and transverse joints.

Reflective cracks were observed on the asphalt where the transverse joints are located. Spalling at the joints is spread widely at the low side of the structure. Random cracking and spalling are apparent from underneath the deck (see Fig. 19). The lack of post-tensioning is once again the apparent cause of the adverse conditions encountered at the bridge.

NEW YORK DOT

The Vischer Ferry Road Bridge, Batchellerville Bridge, Route 155 Bridge, Kingston Bridge, and Cochecton Bridge are all under the jurisdiction of the New York State Department of Transportation.

Vischer Ferry Road Bridge, Schenectady County

This structure was originally designed for H15 loading because this bridge provides vehicular transportation for four homes in the town. The cost of replacing the deck amounted to $300,000; the rehabilitation process took approximately one season, i.e., 6 months. The residents were transported back and forth prior to and after each day's work, which consisted of replacing two full width panels per day.[14]

During construction, the bridge was closed to traffic between 10:00 a.m.

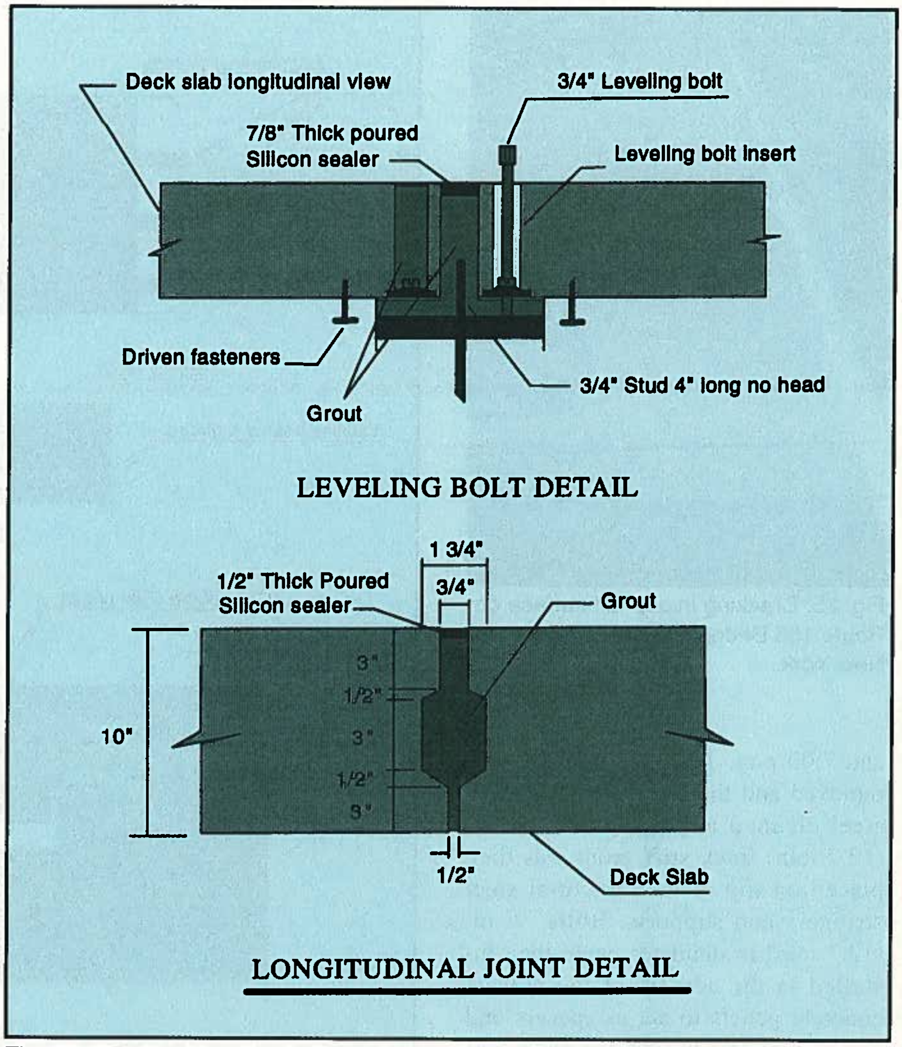


Fig. 23. Details of leveling bolts and shear key in Route 155 Bridge, Albany County, New York.

Fig. 24. Leaking at the transverse joints in Route 155 Bridge, Albany County, New York.

Fig. 25. Cracking in asphalt surface on Route 155 Bridge, Albany County, New York.

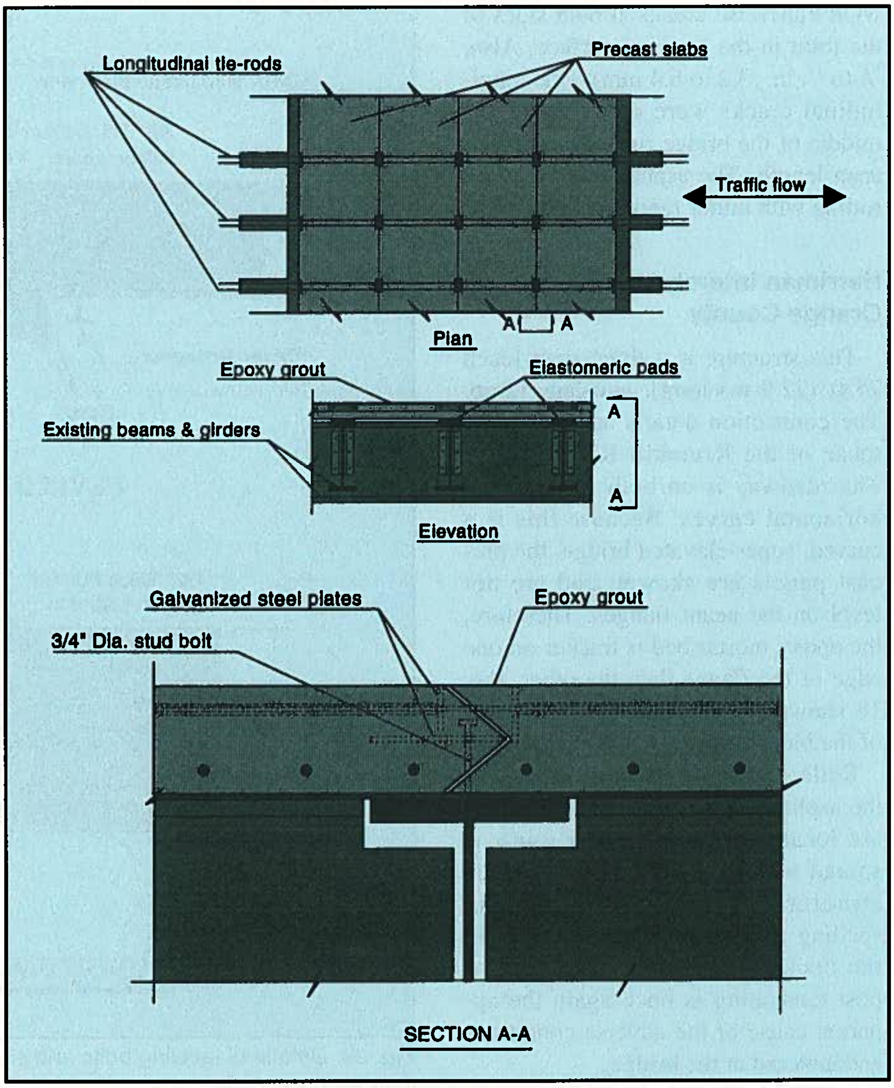


Fig. 26. Plan and elevation of deck along with connection in Kingston Bridge, Ulster County, New York.

and 7:00 p.m. The existing deck was removed and the top of the structural steel cleaned and primed. A $^1/_2$ in. (12.7 mm) thick stiff grout was then placed on top of the structural steel stringers and supports. Bolts, $^1/_2$ in. (12.7 mm) in diameter, were then installed in the corners of the precast concrete panels to act as spacers and also to facilitate the lifting of the panels with a crane.

Post-tensioning was not provided longitudinally to furnish tightness in the joints between the precast concrete panels. Minor leaking was observed from underneath the deck, as shown in Fig. 20. The low volume of traffic observed on the bridge contributes to the good condition of the deck.

Batchellerville Bridge, Saratoga County

This long bridge spans 3075 ft (937 m). The bridge is significant because it is the only direct route to a remote community. The community was given the option of a staged construction with a long construction period or complete closure of the bridge for 6 months. The community opted for the 6-month bridge closing, with a provision for ferry service during that time.[14]

The full width precast concrete panels were placed over newly installed floor beams. The crown of the roadway was built into the panels using curved panels. Transverse and longitudinal sections of the rehabilitated structure are shown in Fig. 21. Because the transverse slab joints are located over the floor beams, the panel length varies from 11 ft 8 in. to 13 ft (3.6 to 4.0 m) depending on the spacing of the floor beams.

Construction started on April 30, 1982, and ended on October 8, 1982, a week ahead of schedule. This project demonstrated the combined cost and time effectiveness achieved through the application of precast concrete slabs in large-scale bridge deck replacement. Due to the uniform spacing of the floor beams, the rehabilitation process was carried out efficiently.

Because the bridge was not designed to support heavy loads [15 ton (13.6 ton) capacity] and the town needs to incorporate a landfill in the area, the capabilities of the structure need to be upgraded. Therefore, it is expected the bridge will be replaced in the near future or, as an alternative option, a new bridge will be built near the first bridge.

This structure is performing well. The only problem appears to be some minor debonding in the joints between the precast concrete panels and some cracking in the asphalt (see Fig. 22). The deck appears to be in good condition from the underneath and above. The community of Saratoga currently owns this 41-span bridge. However, the state will take over responsibility for the bridge prior to redecking or replacing the structure. The low traffic volume and load limitations have contributed towards keeping the structure in excellent condition.

Fig. 27. Leaking underneath bridge deck, Kingston Bridge, Ulster County, New York.

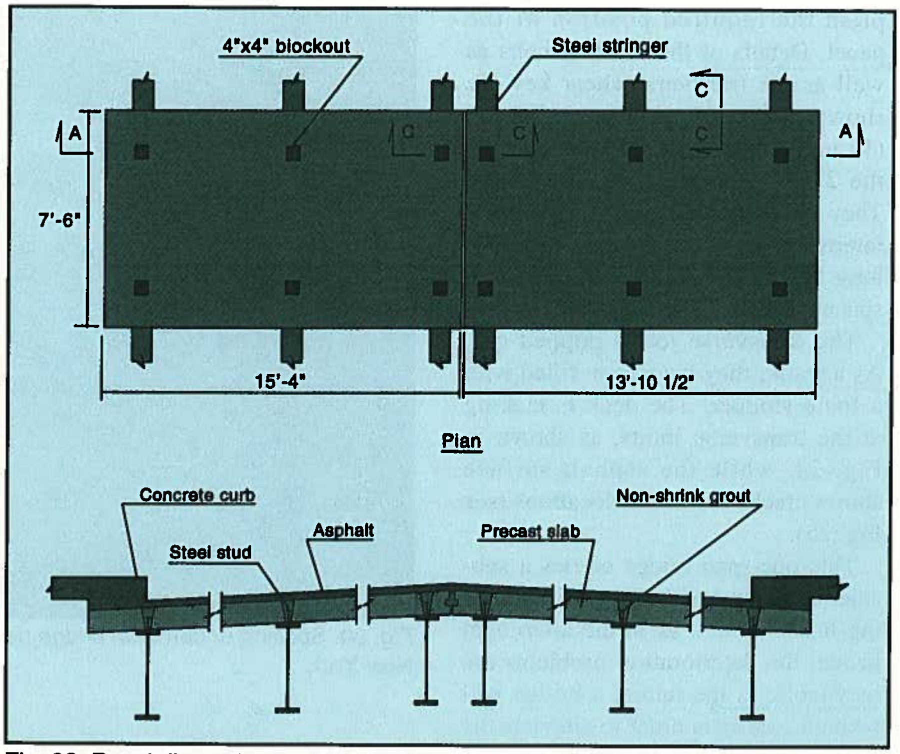


Fig. 28. Panel dimensions and typical section, Cochecton Bridge, Sullivan County, New York.

Route 155 Bridge Over Normanskill, Albany County

The Route 155 Bridge over Normanskill state highway was built in 1928 in the town of Guilderland. Two previous contracts were fulfilled of which the first was the original bridge construction in 1931 and the second was the bridge deck resurfacing in 1972. The rehabilitation was performed on the bridge as a "stop-gap" temporary fix until it is rebuilt.

The replaced area was 101 ft 10 in. (31 m) long and 25 ft $5^1/_2$ in. (7.8 m) wide. Stage I of the construction process consisted of closure of 14 ft 3 in. (4.3 m) of the full width of the bridge, leaving 9 ft 9 in. (3 m) as a traveling lane. In Stage II, the work commenced on the other side of the roadway, leaving a 9 ft 11 in. (3 m) width for traffic.

Two types of precast concrete panels were used 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 the framing system [transverse girders with a spacing of 12 ft 6 in. (3.8 m) held by two trusses at the north and south bounds].

The typical $1^3/_4$ in. (12.7 mm) female-to-female longitudinal shear key is filled with non-shrink cement grout. Every panel has four leveling bolt sleeves at the four corners to accom-

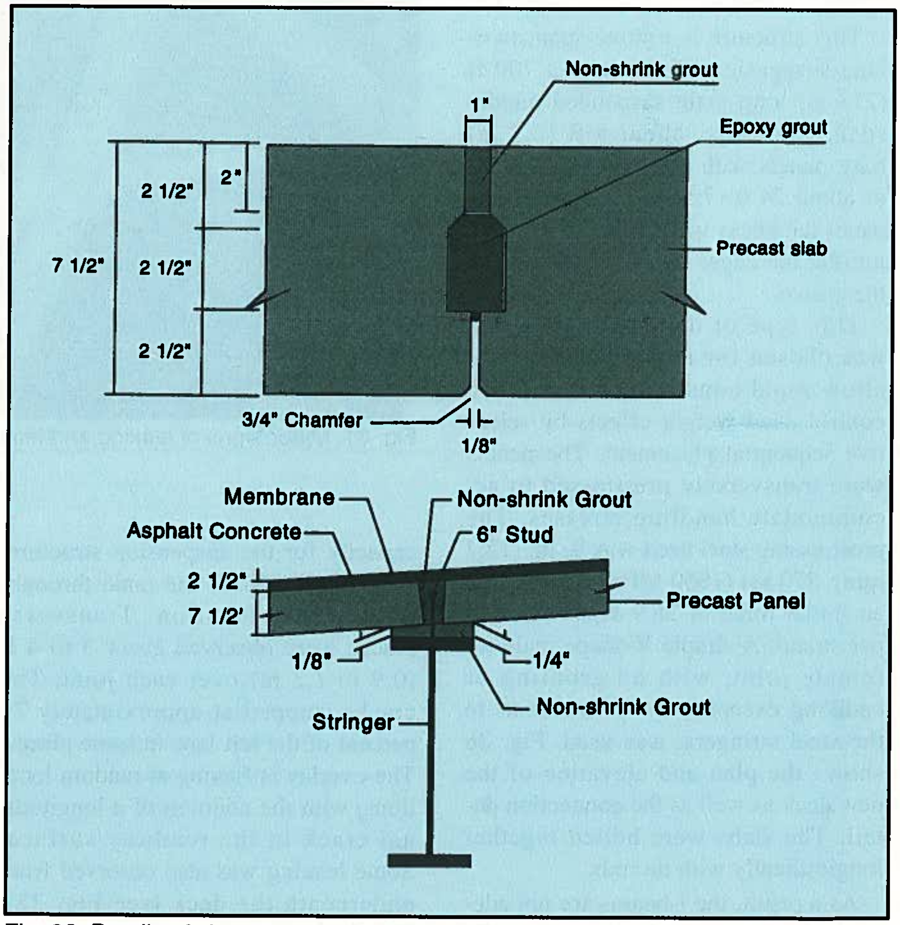


Fig. 29. Details of shear key and slab-stringer connection, Cochecton Bridge, Sullivan County, New York.

plish the required position of the panel. Details of the leveling bolts as well as the transverse shear key are shown in Fig. 23. Non-headed $^3/_4$ in. (19 mm) shear studs were installed in the 2 in. (51 mm) transverse joints. They are 4 in. (102 mm) long for the intermediate panels and 1 in. (25 mm) long for the end panels with a typical spacing of 1 ft 3 in. (381 mm).

The transverse joints popped out. As a result, they have been filled with a foam stopper. The deck is leaking at the transverse joints, as shown in Fig. 24, while the asphalt surface shows cracks at random locations (see Fig. 25).

This one-span bridge carries a substantial amount of traffic in the morning hours as well as in the afternoon; hence, the deterioration problems are inevitable. In the future, a bridge will be built nearby in order to alleviate the amount of traffic on this bridge.

Fig. 30. Spalling underneath bridge deck, Cochecton Bridge, Sullivan County, New York.

Fig. 31. Minor signs of leaking and leaching, Chulitna River Bridge, Alaska.

Kingston Bridge on Wurtz Street Over Rondout Creek, Ulster County

This structure is a three-span, two-lane suspension bridge with a 700 ft (213 m) long main suspended middle span. Typically, about 9 ft (2.7 m) long panels with full roadway widths of about 24 ft (7.3 m) were used. The panel thickness varies from 6 in. (152 mm) at the edges to 7 in. (178 mm) at the crown.

This type of deck reconstruction was chosen for two reasons: (1) to allow rapid construction and (2) to control dead weight effects by selective sequential placement. The panels were transversely prestressed to accommodate handling stresses. The prestressing steel used was $^1/_2$ in. (12.7 mm), 270 ksi (1860 MPa) strands with an initial force of 28.9 kips (129 kN) per strand. A simple V-shape male-to-female joint, with no grouting or caulking except at the connections to the steel stringers, was used. Fig. 26 shows the plan and elevation of the new deck as well as the connection detail. The slabs were bolted together longitudinally with tie rods.

As a result, the I-beams are not adequate for heavy loads, prompting the postage of only 5 tons (4.5 tons) as a capacity for the suspension structure. A nearby bridge is the main throughway in that location. Transverse cracks were observed every 3 to 4 ft (0.9 to 1.2 m) over each joint. The cracks stopped at approximately 75 percent of the left lane in some places. The overlay is flaking at random locations, with the addition of a longitudinal crack in the roadway surface. Some leaking was also observed from underneath the deck (see Fig. 27). This leakage has caused the concrete adjacent to the joints to spall.

Cochecton Bridge Over Delaware River, Sullivan County

This structure is a three-span, two-lane truss bridge with a total span length of 675 ft (206 m). The panels are $7^1/_2$ in. (191 mm) thick, 7 ft 6 in. (2.3 m) long and about half of the roadway width. A bituminous wearing surface along with a waterproofing membrane system were provided.

Fig. 28 shows the panel dimensions and typical bridge section. Details of a transverse slab shear key and slab-stringer connection are shown in Fig. 29. These transverse joints were filled with mortar consisting of one part Type II portland cement to two parts mortar sand. Traffic was maintained by way of staged construction. Reflected cracks appeared along the longitudinal joint that were patched later.

The deck is generally in fair condition, while transverse cracking is observed on the top surface of the deck at uniform distances, i.e., at every other panel joint. Some spalling was observed from underneath the deck, as shown in Fig. 30.


Fig. 32. Minor debonding at transverse joints, Chulitna River Bridge, Alaska.

ALASKA DOT

The Chulitna River Bridge and the Dalton Highway Bridges are under the jurisdiction of the Alaska Department of Transportation and Public Facilities.

Chulitna River Bridge

This 790 ft (241 m) long bridge was rehabilitated in 1992. The existing deck was removed as part of a stage construction and replaced with full width, full depth precast concrete panels. The width of the bridge was consequently changed from 34 ft to 42 ft 2 in. (10.4 to 12.9 m). There were three types of panels used in accordance with geometric allowances, as shown in Refs. 1 and 2.

A 2 in. (51 mm) asphalt overlay with a waterproofing membrane was placed on the new deck. The width of the waterproofing membrane was 18 in. (457 mm) minimum, centered over all the joints. The transverse joint between the precast panels is a female-to-female type; however, post-tensioning was not provided to secure the tightness of the joints.

The panels were connected to the stringers by shear pockets and grouted with magnesium phosphate concrete. A stopper was used prior to placing the grout. However, the panels over the truss elements were connected by a bolted connection because the truss flanges are too narrow for a grouted connection.[1,2]

No equipment or any other significant loads were allowed on the panels from the time grouting commenced until the grout was cured and attained a compressive strength of at least 3000 psi (20.7 MPa). Traffic was allowed in the adjacent lane at 5 miles per hour (8 km/hour) while the grout was curing. After the precast panels were placed and adequately supported, bolted, and the edges of the haunches formed, the haunches, grout pockets, and joints were grouted.

The grout used was high strength, quick set grout designed for rapid, high strength development. Its consistency is such that the grout can be easily pumped and can be used at the temperatures specified without bonding agents or curing compounds.

The magnesium phosphate grout[18] for application at an ambient temperature below 40°F (4.4°C) conformed with the following:

1. Minimum application temperature of 15°F (-9.4°C)

2. Self leveling and easily pumpable

3. Compressive strength at one hour of 2000 psi (13.8 MPa) minimum and at 3 hours of 5000 psi (34.5 MPa) minimum

4. Flexural strength at 6 hours of 300 and 600 psi (2.1 and 4.1 MPa) at 24 hours

5. Slant shear bond strength at 6 hours of 300 psi (2.1 MPa), at 24 hours of 600 psi (4.1 MPa), and at 7 days of 1500 psi (10.3 MPa)

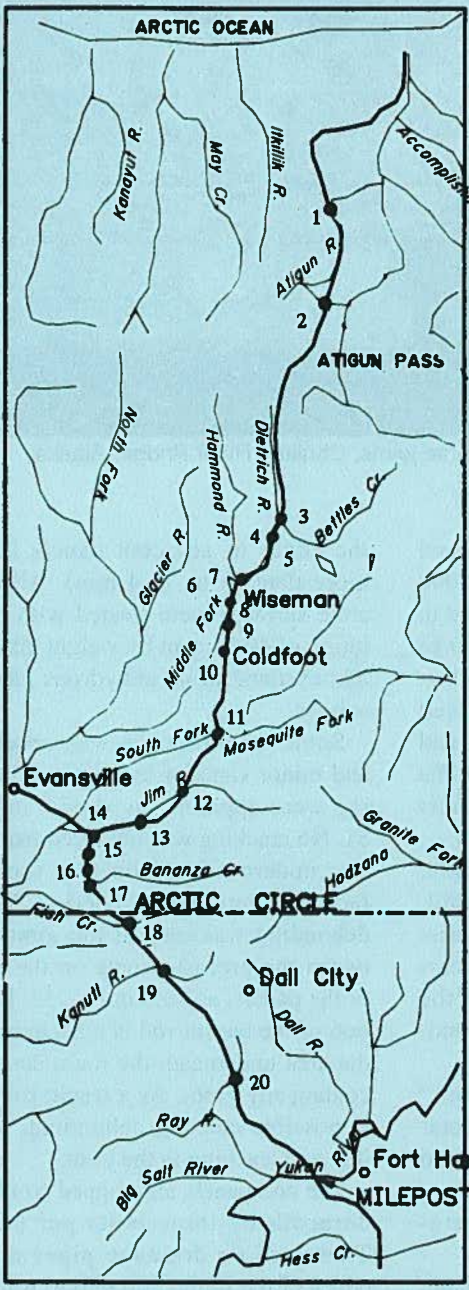
The deck panel units were produced and placed so that the differences in elevation between the top surfaces of the edges of adjacent panels is not more than 1/4 in. (6.4 mm). All concrete surfaces were treated with a solution of 40 percent by weight alkyltrialkoxysilane in an anhydrous alcohol solvent.

Some efflorescence was observed and minor signs of leaking and leaching were apparent, as shown in Fig. 31. No cracking was observed from either underneath the deck or top surface, i.e., bituminous overlay. Minor debonding was seen at the joints between the precast panels on the edge of the panels, as seen in Fig. 32. Location of the backer rod is inadequate, as the area underneath the foam does not contain any grout. As a result, rotation is possible causing debonding, leaking, and leaching in the joint.

The end panels are clipped from underneath by three bolts per panel. There are no drainage pipes at the edges of the bridge; therefore, water is seeping from the edges of the bridge, causing some leaching. Overall, the steel and concrete deck are in good condition. The bituminous overlay surface is also in good condition.

Dalton Highway Bridges

This project consists of 18 bridges. A list representing the bridges and their locations, mile posts, and total span lengths is shown in Fig. 33. The existing timber decks were replaced. These decks were supported on either



MAP DES	MILE-POST	BRIDGE No.	BRIDGE NAME	BRIDGE LENGTH
1	273.1	1333	Atigun River 2	331.83
2	253.2	1439	Atigun River 1	91.83
3	207.0	1337	Dietrich River	211.10
4	204.5	1284	Middle Fork Koyukuk River 4	121.83
5	204.3	1283	Middle Fork Koyukuk River 3	151.10
6	190.8	1282	Middle Fork Koyukuk River 2	271.83
7	190.6	1336	Hammond River	151.10
8	188.5	1261	Middle Fork Koyukuk River 1	331.83
9	187.2	1335	Minnie Creek	121.83
10	175.1	1332	State Creek	91.83
11	156.1	1260	South Fork Koyukuk River	421.83
12	144.1	1437	Jim River 3	180.38
13	141.1	1436	Jim River 2	121.83
14	140.1	1259	Jim River 1	121.83
15	135.1	1258	Prospect Creek	121.83
16	125.7	1256	North Fork Bonanza River	120.38
17	124.7	1257	South Fork Bonanza River	91.10
18	114.0	1255	Fish Creek	120.38
19	105.7	1338	Kanuti River	151.83
20	79.1	1334	No Name Creek	91.83
	0.0		JCT with Elliott Highway	

Fig. 33. Locations, mile posts, and total span lengths, Dalton Highway Bridges, Alaska.

steel stringers or timber floor beams. The rehabilitation process included the installation of full-width, full-depth precast, prestressed concrete deck panels on new steel stringers and pile caps. The replacement procedure imposed a stage construction to maintain traffic flow during construction.

The sizes of the stringers varied due to the difference in span lengths, i.e., either 30 or 60 ft (9.1 or 18.3 m). All precast panels were 9½ in. (241 mm) thick at the centerline of the roadway and 7½ in. (191 mm) thick at the edges with a typical length of 27 ft 5⅜ in. (8.4 m) and two typical widths of 4 ft 10 in. and 5 ft 7 in. (1.5 and 1.7 m). Details of the panels are presented by Issa et al. in Refs. 1 and 2. Fig. 34 shows a typical overview of the deck panels.

Traffic was allowed to pass when the deck units were placed but not grouted; traffic was stopped when grouting commenced. Traffic speed was limited to a maximum of 3 miles per hour (5 km/hour) while the grout is not placed. The same specifications apply for these bridges as for those in the Chulitna Bridge rehabilitation project. Deck panels were produced and placed so that the difference in elevation between the top surfaces of the edges of adjacent panels in place is not more than $^3/_{16}$ in. (4.8 mm).

The prestressing strands were $^1/_2$ in. (12.7 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 (1302 MPa) and the effective stress after all losses was 149 ksi (1027 MPa). The prestressing strands were typically spaced at 1 ft 3 in. (381 mm).

The joint between the precast concrete panels was the female-to-female type, which is similar to that used in the Chulitna River Bridge rehabilitation project. The shear pocket size was 7 x 5 in. (178 x 127 mm) with two studs $^7/_8$ x 6 in. (22 x 152 mm) in each pocket for the 30 ft (9.1 m) spans; the pocket size was 12 x 5 in. (305 x 127 mm) with three studs of the same size for the 60 ft (18.3 m) spans. However, there were a number of exceptions to those criteria. Fig. 35 is a typical view of the deck condition from underneath.

The main problems encountered on the bridges in this rehabilitation project are attributed to the fact that no post-tensioning was provided longitudinally to secure the tightness of the transverse joints. There is no overlay on the surface of the deck because the road is not paved; therefore, it is not practical to provide an overlay over the panels.

Fig. 34. Typical overview of deck panels, Dalton Highway Bridges, Alaska.

Fig. 35. Typical underside view of deck condition, Dalton Highway Bridges, Alaska.

Condition of Bridges

In general, there was no leaking through the panels. However, the transverse joints at the supports experienced cracking and loss of material, as shown in Fig. 36. Typically, the top surface of the deck shows cracking at almost all the transverse joints, with the most severe cracking at the supports. Some bridges, such as the Kanuti River Bridge and the Middle Fork Koyukuk River 3 Bridge, had cracks between the pockets, as shown in Fig. 37.

The decks for the Fish Creek Bridge, Jim River 2 Bridge, and Middle Fork Koyukuk River 1 Bridge contain patching between the pockets, as shown in Fig. 38. Most of the transverse joints between the precast concrete panels are experiencing debonding, as shown in Fig. 36, while some of the bridges (North Fork Bonanza River Bridge) contain minor spalls in the deck, as shown in Fig. 34.

The Minnie Creek Bridge showed signs of leaking, while all spans contain splitting at the joints (see Figs. 34 and 36). The Middle Fork Koyukuk River 2 Bridge was reported as damaged because one of the piers has settled. Currently, travel on the bridge has been reduced to one lane with a 5 miles per hour (8 km/hour) restriction posted. Many of the transverse joints contain cracking and loss of material, especially those at the supports (see Fig. 36).

The transverse joint at the junction between the two 60 ft (18.3 m) spans is severely split, as shown in Figs. 34 and 36. The last two 30 ft (9.1 m) spans appear to be in good condition. The rubber material (stopper) that is provided to hold the grout is not efficient because it holds an undesirable volume. More grout and less stopper would contribute towards providing more composite action.

Fig. 36. Cracking and loss of material at transverse joints, Dalton Highway Bridges, Alaska.

Fig. 37. Cracks between shear pockets, Dalton Highway Bridges, Alaska.

Fig. 38. Patching at shear connecting pockets, Dalton Highway Bridges, Alaska.

OHIO DOT

The Dublin 0161 Bridge is under the jurisdiction of the Ohio Department of Transportation.

Construction of the Dublin 0161 Bridge started in 1986. This skew bridge consists 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 a height clearance of about 50 ft (15 m). The bridge has a concrete arch with cross beams as its deck supporting system. A two-phase construction procedure was adopted to replace the old deck.

The full depth precast concrete panels consist of panel lengths of 12 ft 1½ in., 9 ft 10½ in., 9 ft 6½ in., 9 ft 5½ in., and 10 ft 1 in. (3.7, 3.0, 2.9, 2.9, and 3.1 m), a panel width of 28 ft (8.5 m), and varying depths. Fig. 39 shows a typical panel layout. A typical finished roadway section at the arch crown in addition to a close-up of the tie-down connection are shown in Fig. 40. Fig. 41 shows the transverse and longitudinal panel joints, along with typical sections in the transverse and longitudinal directions.

Non-prestressed steel was furnished as panel reinforcement 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 (6.9 MPa). Panels are supported on elastomeric bearings and are anchored down to floor beams using dowel bars. All the mild steel reinforcement is epoxy coated and the prestressing strands are polymer coated. The unit stress for the precast, post-tensioned deck panels is 2200 psi (15 MPa) in compression (service load) and 444 psi (3.1 MPa) in tension (construction Phase II). Epoxy mortar material is used for the joints between the adjacent precast concrete panels.

The rigidity of the structure is attributed to the concrete supporting system. The deck was designed to be noncomposite, yet the bridge superstructure is very rigid. Random cracking was found in the overlay. There are no signs of any leaking, leaching, or debonding in the deck.

PENNSYLVANIA TURNPIKE COMMISSION

The B-501 Bridge, B-552 Bridge, NB-216 Quakertown Interchange Bridge and NB-750 Clark Summit Bridge are under the jurisdiction of the Pennsylvania Turnpike Commission.

B-501 Bridge, Somerset County

This simple span bridge is an exit ramp for the Pennsylvania Turnpike at Somerset. The joint between the precast concrete panels is shown in Fig. 42, while the slab-to-stringer connection is shown in Fig. 43.

The superstructure has extensive surface corrosion with rust laminations in the bottom flanges of all members. Some tie-downs are missing or loose. Leakage is excessive at the joints between precast concrete panels, as shown in Fig. 44. This is attributed to the inadequate connection between the deck and supporting system as well as the configuration of the transverse joint. The overlay contains some cracks. In some cases, patching was done on the top surface; however, some cracks are still seen propagating from the patching area.

B-552 Bridge, Everett

This narrow, one-lane bridge is simply supported. The deck is in good condition because the bridge is only used by a private community. The

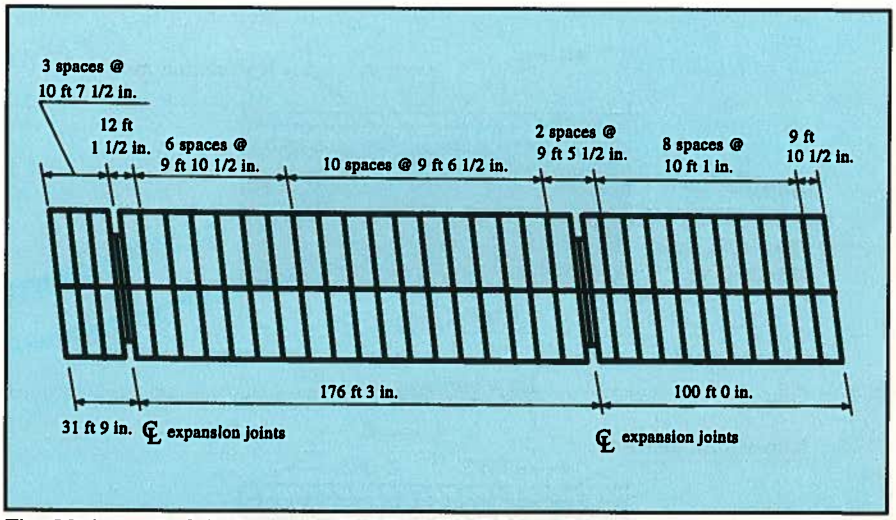


Fig. 39. Layout of deck panels, Dublin 0161 Bridge, Ohio.

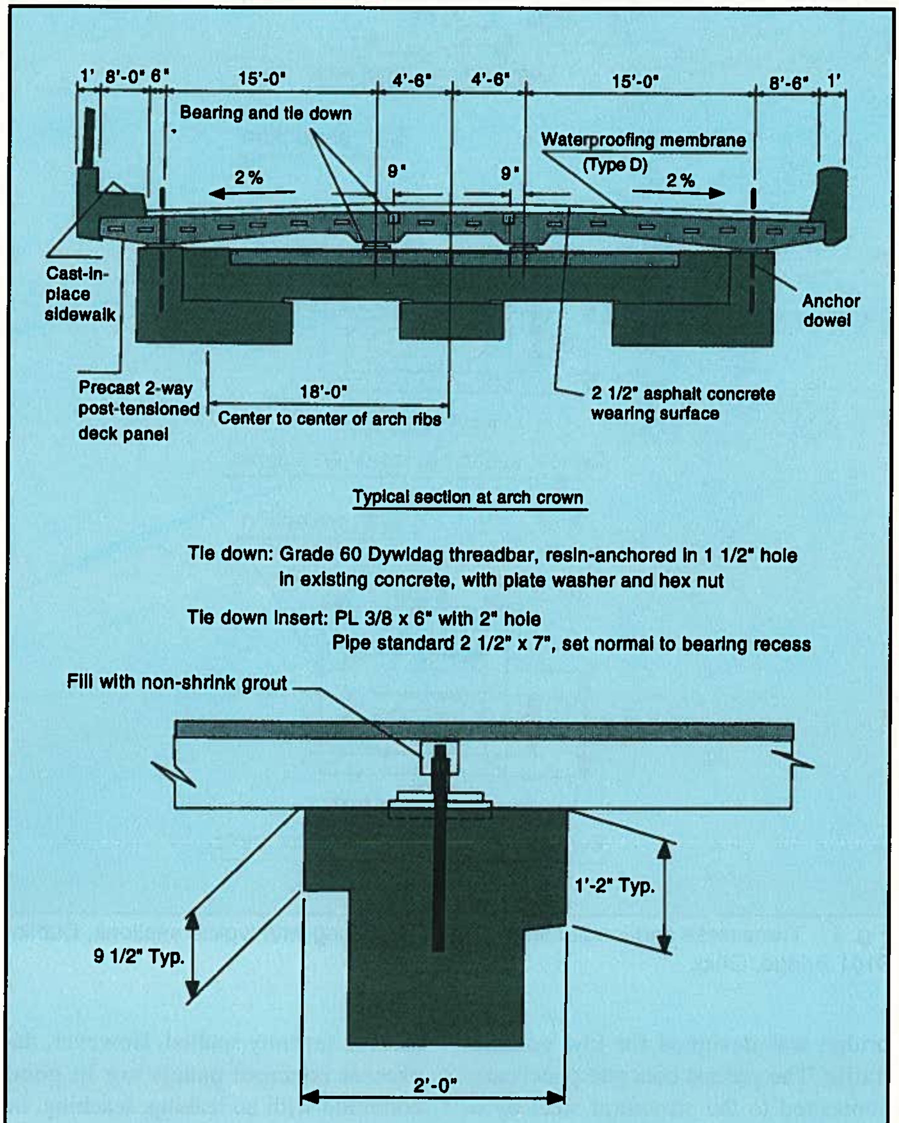


Fig. 40. Finished roadway section and details of tie-down and insert, Dublin 0161 Bridge, Ohio.

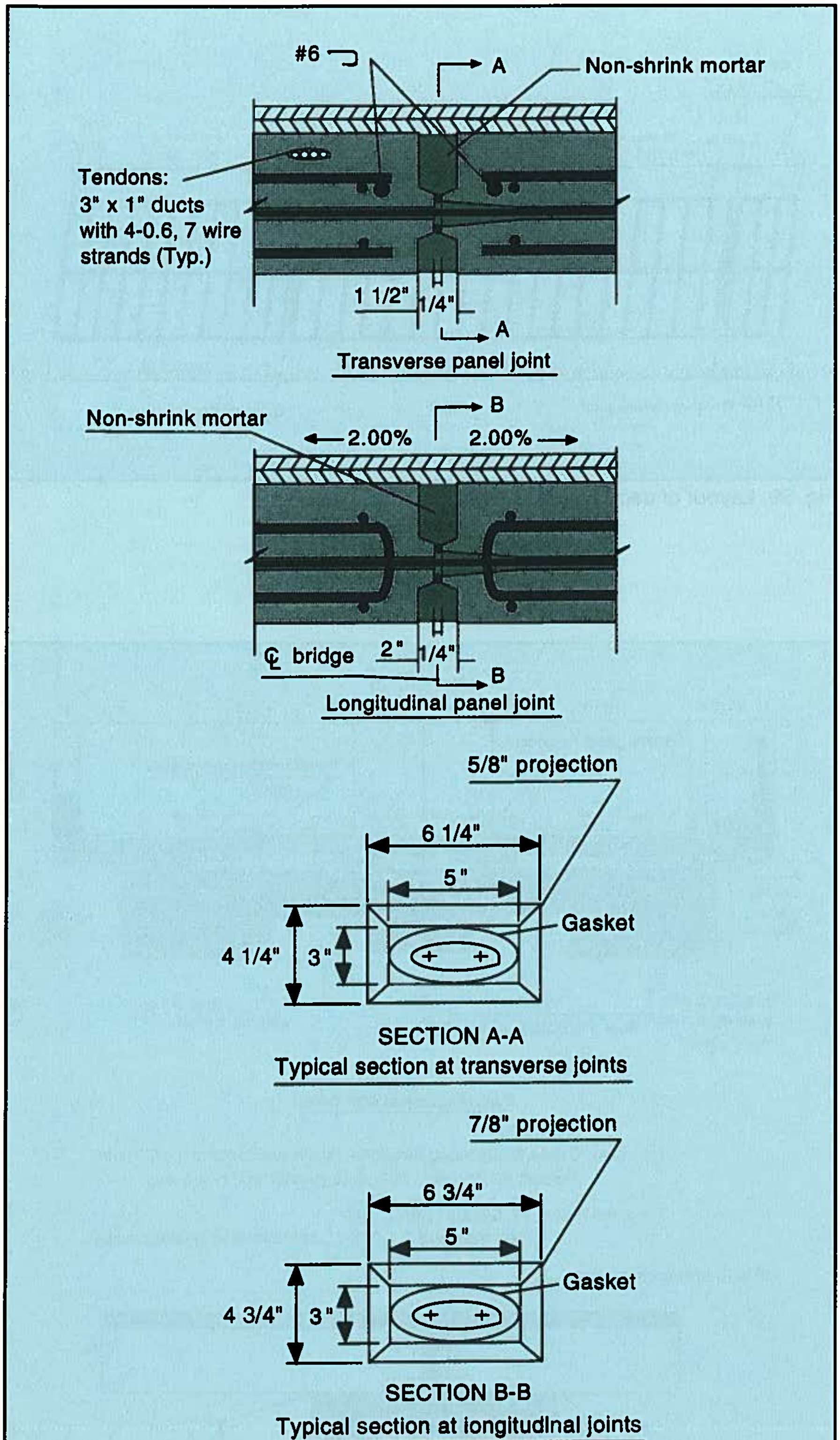


Fig. 41. Transverse and longitudinal panel joints along with typical sections, Dublin 0161 Bridge, Ohio.

bridge was designed for low volume traffic. The precast concrete panels are connected to the structural steel by a tie-down, as shown in Fig. 45.

The existing steel supporting system is extensively corroded and the abutment is severely spalled. However, the precast concrete panels are in good condition with no leaking, leaching, or cracking observed during the investigation. Efflorescence is present along the slab and curb joints as well as along the top flanges of the floor beams. The wearing surface shows some light wear with areas of aggregate exposure; however, there is no significant deterioration apparent. Overall, the precast concrete deck is in good condition; nevertheless, the supporting system is in unsatisfactory condition.

NB-216 Quakertown Interchange Bridge, Bucks County

This is a suspended cantilever system with a composite deck in the suspended span and a noncomposite deck in the cantilever span. The bridge serves as an interchange exit for the Pennsylvania Turnpike.

The precast concrete panels are $6^1/_2$ in. (165 mm) thick, with a varying haunch thickness, 7 ft $7^1/_2$ in. (2.3 m) long, and 17 ft 6 in. (5.3 m) wide, and cover one-half the width of the structure. Existing bulb angle shear connectors were left in place as the old slab was removed in 1981. The slab panels with shear pockets were cast with sufficient precision so that the precast slab fitted properly when placed. Elastomeric strips are glued to the top of the flanges to contain the epoxy mortar that provides uniform bedding of the precast concrete panels. Fig. 46 shows the connection details.

The transverse joints are pulled together using nominal longitudinal post-tensioning. In addition to providing rapid erection, the construction of the bridge described above has proven efficient and cost effective compared to conventional deck replacement methods.

The latex modified concrete overlay contains transverse cracks running the entire width of the bridges. These cracks are located at each deck panel joint. Some of these cracks have been patched with asphalt. The approach roadway needs to be paved to provide a smooth transition to the bridge deck.

The cantilever spans contain heavy spalling and cracking within distances of 18 in. (457 mm) (see Fig. 47). However, the main span only contains some cracking. The precast concrete deck panels show water marks at every joint with extensive cracking and delaminations at approximately

75 percent of these joints (see Fig. 47). There are corrosion and rust laminations below the deck joints and along the bottom flanges of the stringers. The pin and link details exhibit rust laminations and rust packing around the lower pins at both the expansion and fixed joints. The tie-down connections do not provide enough composite action to guard against any vibrations, causing deterioration in the deck.

NB-750 Clark Summit Bridge, Lackawanna County

This ten-span, 1627 ft (496 m) long bridge consists of two parallel structures carrying two lanes each way and a clearance of 49 ft (14.9 m). In 1980, precast concrete panels were chosen for the replacement of the deteriorated deck. They were selected because of the necessity to maintain traffic on one-half of the bridge while redecking the other half. It was also feared that vibrations from the traffic could interfere with the proper concrete setting, especially at the juncture of the new decks.

The panels are typically $6^3/_4$ in. thick and 7 ft (2.1 m) long with a full roadway width of 29 ft (8.8 m), and weighing 18,000 lbs (8164 kg) each. Elastomeric strips and epoxy mortar grout are used for bedding over existing stringers. Non-shrink cement grout was placed at the transverse joints and nominal longitudinal post-tensioning was used. The connection of the slab to the stringer is similar to that used on the Somerset Bridge.

The bridge slabs were not designed for composite action with the stringers, although it is likely that some composite behavior resulted from this detail. Several of these tie-downs are either loose or missing as a result of vibrations caused by the heavy traffic volume.

The overlay shows cracking at every panel joint. Patching is widespread on the surface, while cracking continues to emanate from these patching areas. Viewed from the underside, the deck is cracking and spalling at every joint (see Fig. 48) due to the inadequate type of connection between the deck and supporting system.

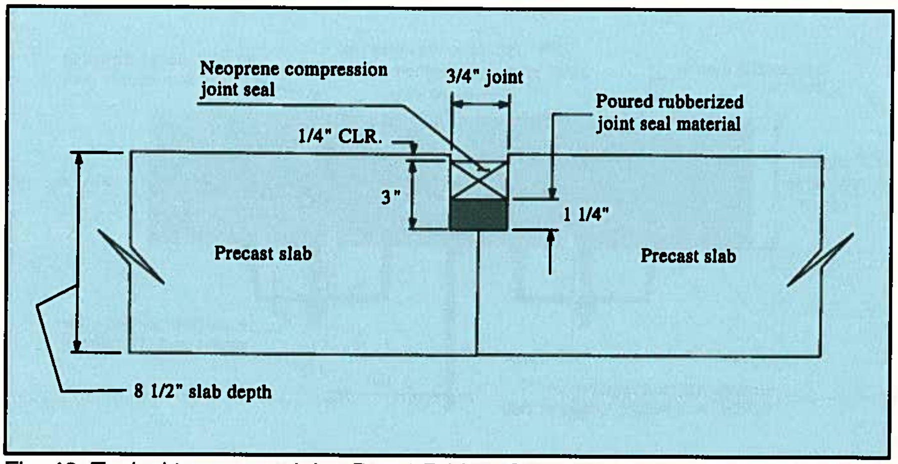


Fig. 42. Typical transverse joint, B-501 Bridge, Somerset County, Pennsylvania.

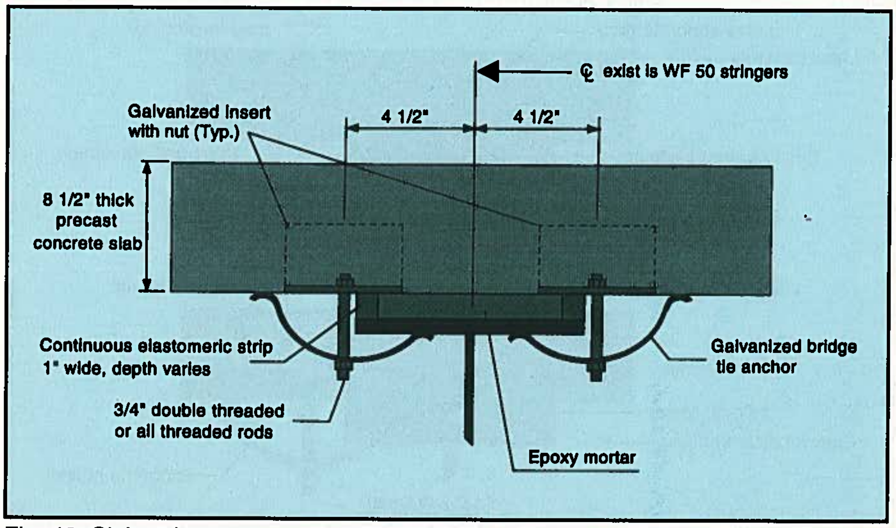


Fig. 43. Slab-stringer connection, B-501 Bridge, Somerset County, Pennsylvania.

Fig. 44. Excessive leakage at transverse joints, B-501 Bridge, Somerset County, Pennsylvania.

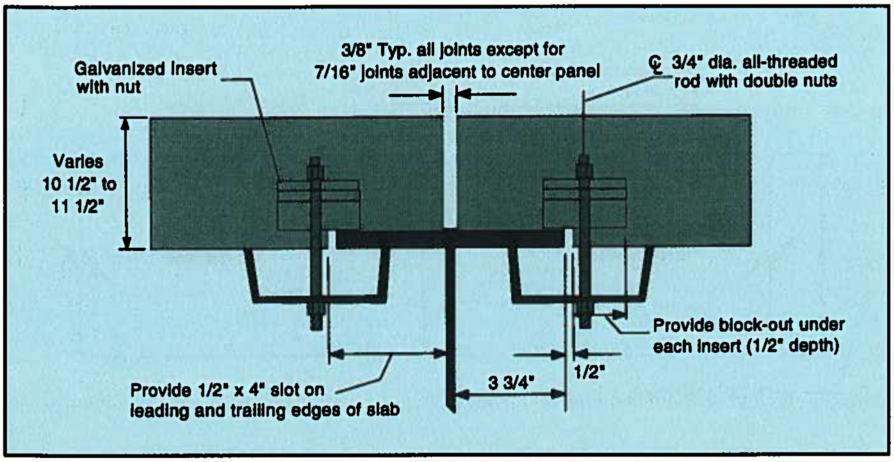


Fig. 45. Slab-stringer connection, B-552 Bridge, Everett, Pennsylvania.

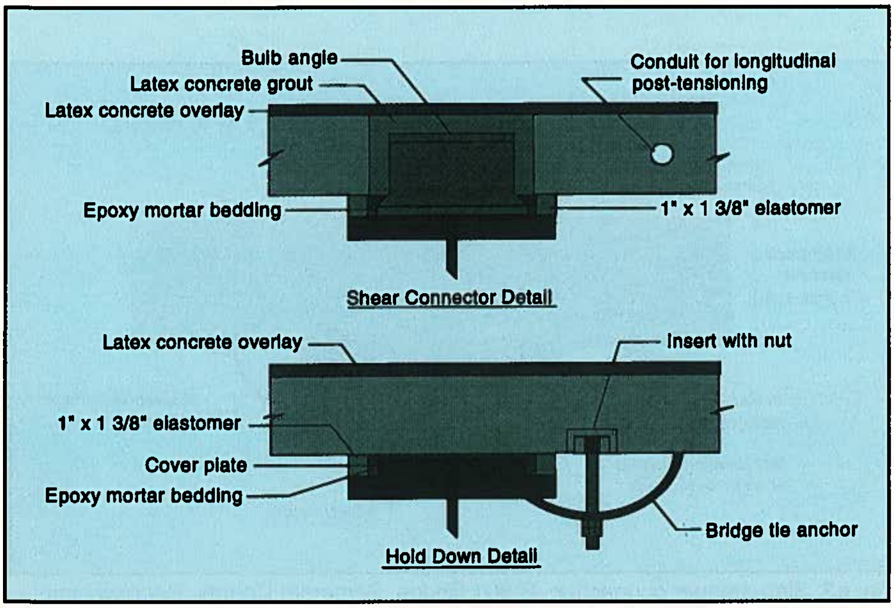


Fig. 46. Shear connector and hold down details, NB-216 Quakertown Interchange Bridge, Bucks County, Pennsylvania.

Fig. 47. Heavy spalling and cracking, NB-216 Quakertown Interchange Bridge, Bucks County, Pennsylvania.

Fig. 48. Cracking and spalling, NB-750 Clark Summit Bridge, Lackawanna County, Pennsylvania.

CONCLUSIONS AND RECOMMENDATIONS

The visual inspection process carried out throughout the past 18 months has been extremely informative in terms of design and construction techniques used by the various DOTs throughout the nation. The visits have aided in the determination of the best system to be implemented in future applications of full depth precast or precast, prestressed concrete bridge deck replacement. The advantages and disadvantages of each aspect of design for the system can be seen clearly.

The following conclusions and recommendations can be made as a result of the investigations conducted in Illinois, Connecticut, Virginia, Maryland, Iowa, California, New York, Alaska, Ohio, and Pennsylvania:

1. Precast concrete panels are an efficient and economical means for replacing deteriorated bridge decks in our nation's highway system.

2. The investigation has shown that precast concrete panels have an excellent performance record. In cases where the performance has not been good, it can be attributed mainly to the type of connection between the slab and supporting system, configuration of joint between adjacent precast panels, construction procedures, lack of longitudinal post-tensioning, and materials used.

3. Shear studs can be used for the connection between the precast concrete panel units and the supporting system through shear connection pockets. However, proper construction procedures must be maintained to obtain a satisfactory design, i.e., haunches must be provided to allow for any dimensional irregularities or volume changes.

4. The shear key between precast concrete panels may be of a female-to-female type with at least a $^1/_4$ in. (6.4 mm) opening at the bottom to allow for any panel size irregularities. Some states used this type of joint; however, the panels were in contact at the bottom. This configuration causes leaking if the panels do not fit perfectly. The tongue-and-groove joint is not practical because of difficulties encountered with the grouting process. The type of joint, which includes a direct contact of the precast panels (butt joint), is not effective because it causes leakage through the joint as a result of the deck being put into tension. The proposed type of shear key is very effective because it compensates for many of the problems indicated. High strength polymer grout can be used for the proposed type of joint so that the the post-tensioning operation can follow immediately.

5. The precast panels should be post-tensioned longitudinally to secure the tightness of the joints, to keep the joint in compression, and to guard against leakage.

6. The precast concrete panels may be designed for transverse flexure with mild steel reinforcement, prestressing strands, bonded post-tensioning strands, or a combination of each, depending on the size of the panels used. In general, precast panels need a sufficient amount of transverse prestress to avoid cracking during handling of the slab units.

7. A waterproofing membrane system may be used, as demonstrated by all the DOTs.

8. An overlay is essential to keep the deck in good performing condition and to provide a smooth ride. The most widely used type of overlay was found to be latex modified concrete. Silica fume concrete is currently in use as an overlay because it is more cost effective and less sensitive to temperature changes.

9. Regular maintenance is necessary to keep the bridge deck in satisfactory condition and to prolong the lifetime of the structure.

10. The selection of shear pockets and spacing of the pockets depends on the configuration of the supporting system, i.e., whether the beams or girders are in the longitudinal or transverse direction.

11. Precast concrete supporting systems are less flexible than steel in the design of bridge superstructures. Fewer problems were encountered with bridge decks supported on concrete elements.

12. The amount of vehicular volume significantly affects the structural behavior of the bridge. A bridge carrying small loads does not indicate the feasibility of constructing this type of system in future applications.

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