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RAPID REPLACEMENT OF BRIDGE DECKS

FINAL REPORT

APPENDICES

Prepared for
National Cooperative Highway Research Program
Transportation Research Board
National Research Council

Center for Infrastructure Research
University of Nebraska

July 1997

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APPENDIX A

LITERATURE REVIEW

Partial Deck Removal (1):

Partial removal is defined as removal of deteriorated concrete before repair (e.g., by patching or overlay). The objectives in removing part of a concrete member are to salvage the sound portions and to remove the existing concrete quickly and economically without damage to the concrete to be left in place. Partial removal is not limited to the removal of deteriorated concrete. Removal may also include curb removal before widening of a deck or the removal of the surface concrete from a deck slab before construction of an overlay.

The methods for partial removal are divided conveniently into either hand-held percussive tools, hydro-demolition, and thermal techniques.

Breakers. The most widely used tools for partial removal of concrete from a concrete component are hand-held percussive breakers, commonly known as jackhammers. Originally powered only by compressed air, hydraulics, electric, or gasoline powered hand-held breakers are now available.

Needle Scalers. Needle scalers are designed primarily for such applications as removing paint from metal or cleaning weld seams. Needle scalers are especially well suited for use on an uneven surface because the needles conform to the contour of the work. In concrete work, they are mainly used to remove small quantities of concrete in areas where access is difficult and special care is required. The tools can be fitted with a chisel point so that the concrete can be removed to a desired amount using the needles.

Water Jet Removal. Water jets operate without abrasives, and at low pressures they can be used for the partial removal of concrete. They consist of relatively lightweight wands or lances connected by suitable hoses to a high-pressure pump and water supply.

Hydro-demolition. Hydro-demolition simply means the process of using water to demolish concrete. The process consists of applying a water jet on a large scale. In practice, hydro-demolition is used mainly for surface preparation and removal of deteriorated concrete from bridge decks as an alternative to scarifying and chipping. A number of advantages are claimed by the manufacturers of hydro-demolition equipment: (1) the process is naturally selective and removes all unsound concrete; (2) the reinforcing steel is cleaned but not damaged; (3) the process does not induce microcracks in the existing concrete; (4) surface preparation is not required; (5) no dust and minimal noise or vibration are produced; and (6) the process is rapid and costs are reasonable.

Thermal Techniques. Oxyacetylene cutters are used routinely in combination with other methods in concrete removal operations to cut exposed reinforcing bar and prestressing steel. The use of thermal techniques to cut concrete are less common and require equipment capable of generating much higher temperatures than a mixture of oxygen and acetylene. This technology is relatively new, however, likely to advance rapidly than other traditional methods, such as saws and percussive tools.

Thermal Flame Lance. Thermal lances have been used to cut metals for many years, but the technology has only recently been applied to cutting concrete. Thermal lances have no vibration and low noise levels. They can be used where access is difficult and the process is not hampered by embedded steel. They also can be used under water. The major disadvantage is that costs are higher than mechanical methods under normal conditions.

Jet Flame Lance. A jet flame lance uses a flame producing 5800°F to 6300°F (3204°C to 3482°C) from a mixture of kerosene and oxygen accelerated to supersonic speeds in order to cut concrete. The process has the same advantages as the thermal flame lance but has a higher cutting speed. However, the molten slag generates fumes and presents fire and safety risks.

Other Thermal Cutting Techniques. Other thermal cutting techniques, such as laser beam, electron beam, and arc heating, are all at an experimental stage and require further development in order to overcome problems, such as high capital costs, large energy requirements, and high saw cutting speeds.

Electric Heating of Reinforcing Steel. The electric heating of the reinforcing steel can be used to cause the concrete cover to delaminate above the steel so that it can be removed by mechanical means. The electric heating can be by either direct or induced methods. Experiments in Japan have shown that the direct heating of reinforcement can be used to remove the concrete cover. It has disadvantages, such as the expense of induction heaters and power requirements that are very high especially for thick concrete covers.

Direct Heating of The Concrete. Concrete can be broken by thermal strain. Thus, the direct heating of a concrete surface is one method of partially removing concrete or preparing a concrete surface for repair. The use of a direct flame was investigated in Europe as a method of cleaning a concrete surface. It was discovered that the thickness of the concrete layer removed was largely a function of the speed at which the burner was moved and the moisture content of the concrete. The advantages of microwave heating are that it is easy to control and generates no noise or vibration (except when the concrete fractures). However, there are serious disadvantages to be overcome before the method can be used in the field. These include ensuring the safety of workers, protection of communications from interference, and the availability of required high-output magnetrons.

Complete Removal (1):

For reasons of economy and efficiency, there is a natural tendency to use much larger equipment for the complete removal of concrete than for partial removal.

Equipment varies for full removal of concrete bridge decks. Pneumatic breakers have been used for almost a century, whereas other equipment, such as hydro-demolition equipment, is relatively new. Although the range of available equipment is vast, it is rare that different types of equipment can be used interchangeably. The optimum choice of equipment for a particular application is determined by (1) the quantity and quality of concrete to be removed, (2) time available to complete the work, (3) type of concrete component and its accessibility, (4) cover to the reinforcement, and (5) restrictions with respect to vibration, noise, dust, and disposal of the debris.

Equipment used for the complete removal of a concrete component includes saws, drills, breakers, splitters, crushers, ball and crane, and blasting. Saws are often used to cut the deck into sections, but care must be taken not to cut the top flanges of bridge girders. Breakers are usually machine-mounted and can be used on most components. Restrictions are often placed on the size and manner for the operation of these breakers because of the high risk of damage to the remainder of the structure.

Sawing. The most common type of saw blade used for cutting concrete is the wet-cutting diamond blade. This blade is made by welding, or silver-brazing, diamond-impregnated segments to the perimeter of a steel core. For cutting on horizontal surfaces, the wet-cutting diamond blades are normally used when they are mounted on self-propelled gasoline or diesel-powered saws. Wet-cutting diamond blades (Fig. 2.1) work

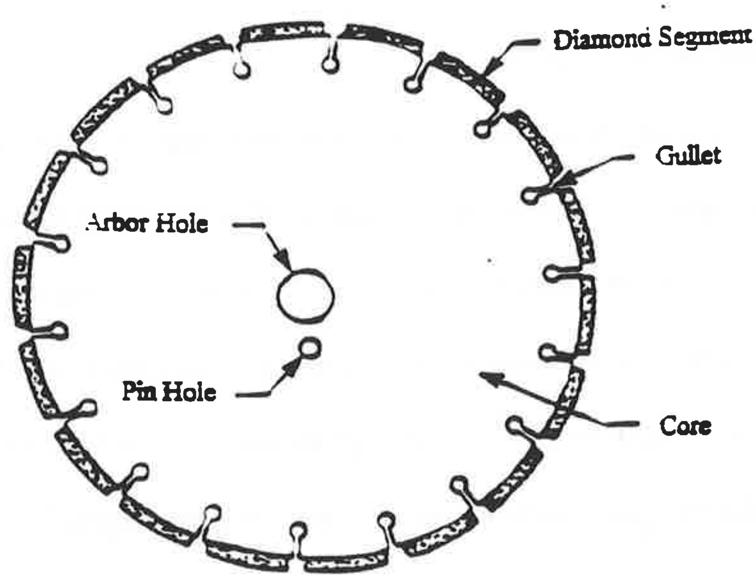


Fig. A.1. Wet-Cutting Diamond Saw Blade

best under a steady, even pressure without the blade being forced beyond its cutting capacity.

Dry-cutting blades are made by laser-welding diamond impregnated segments to the steel core. Most dry-cutting blades are designed for use with portable, low-horsepower machines where the operator can control the cutting speed of these saws, which in turn control the amount of heat generated.

The most common application of sawcutting for concrete removal in bridge work is to cut the bridge deck into slabs and lift them out by crane. This method of removal is relatively rapid and minimizes falling debris. It is often used in conjunction with redecking operations utilizing prefabricated deck panels. This method works well in night work, allowing the bridge to be kept open during the day.

Diamond Wire Saw. A different method of sawcutting employs a diamond wire cutting system. This relatively new technique has few applications for bridges. In one

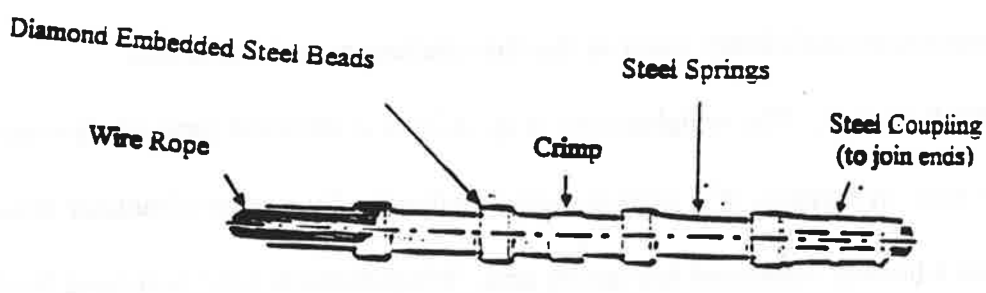


Fig. A.2. Diamond Cutting Wire

example on Long Island, New York, the technique was used to remove the bridge deck, spandrel beams, and pier caps of a 14-span structure during a period of 19 days.

There are many types of diamond beads and wire assemblies available and many more are under development. The most common type is shown Fig. A.2.

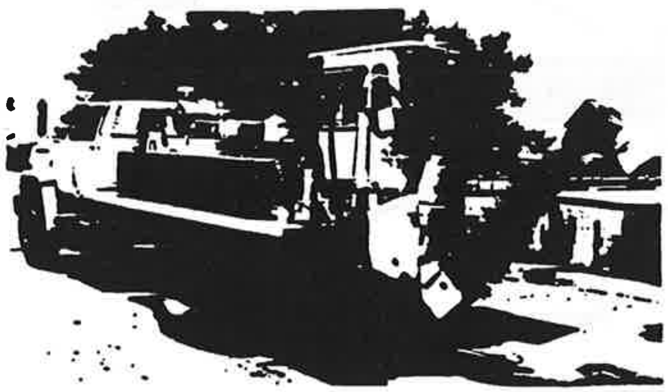
Breakers. Machine-mounted hydraulic breakers are a very common piece of equipment for the demolition of concrete. The breakers, which come in a wide range of sizes, are usually mounted on a hydraulic boom of an excavator, thus resulting in a machine that has both reach and maneuverability. Pneumatically powered breakers are also available.

Damage to the top flanges of steel and concrete girders by rig-mounted breakers is the second most common form of damage reported for removing concrete bridge decks. Unfortunately, there is no clear boundary between the size of breaker which will cause damage to concrete remaining in place, and that which will adequately demolish only the concrete to be removed.

For deck removal, two different techniques are possible. The breakers can be used to punch out the perimeter of a section of the deck slab, the reinforcement cut with a torch, and the slab section lifted out. Alternatively, the breaker can be used to break the concrete into pieces sufficiently small so that the reinforcement is separated.

Whiphammer. The whiphammer (Fig. A.3) is a different form of rig-mounted percussive tool. It consists of a truck-mounted hydraulically operated hammer attached to the end of a heavily restrained leaf spring arm. Whiphammers have been used for both bridge deck and parapet wall removal.

The concept of the whiphammer is basically that of a headache ball. The ball is mounted on the end of a leaf spring type whip that hydraulically moves the headache ball up and down in a whipping motion. The striking force can be adjusted to within a fraction of an inch in order to allow the impact desired. The movement of the spring arm mounted headache ball is relatively simple. It moves from side to side within the area directly behind the truck, and moving horizontally with the truck. It has a vertical control which varies the depth to which it strikes (Fig. A.4).



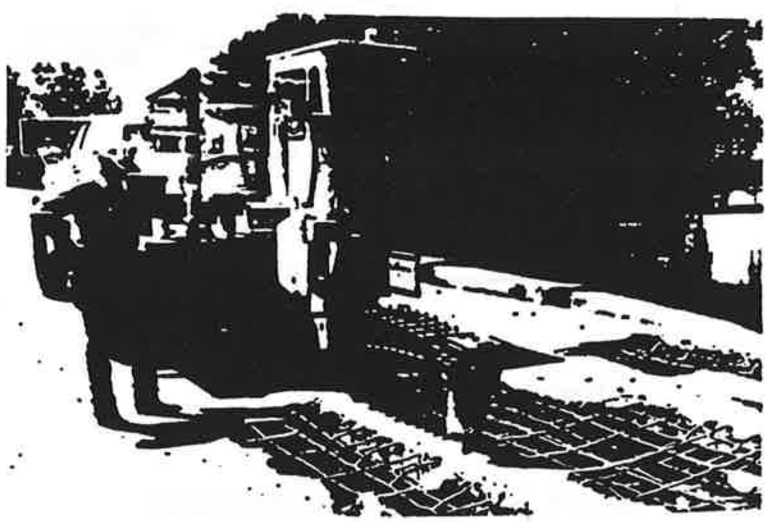
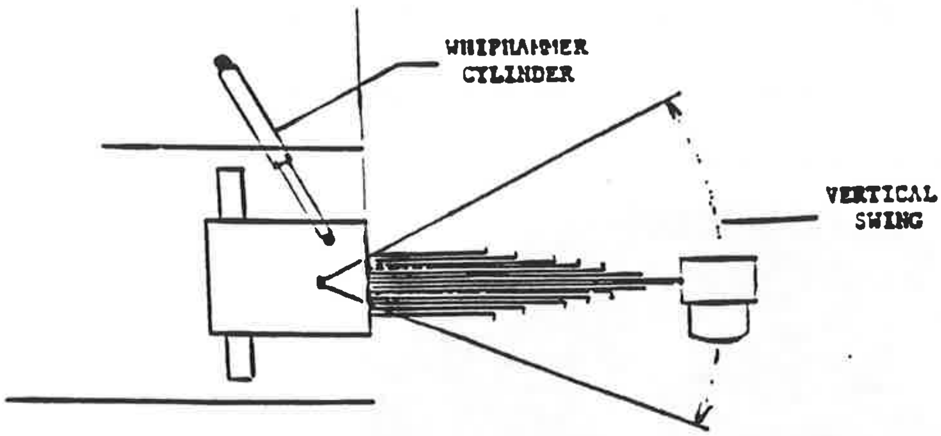
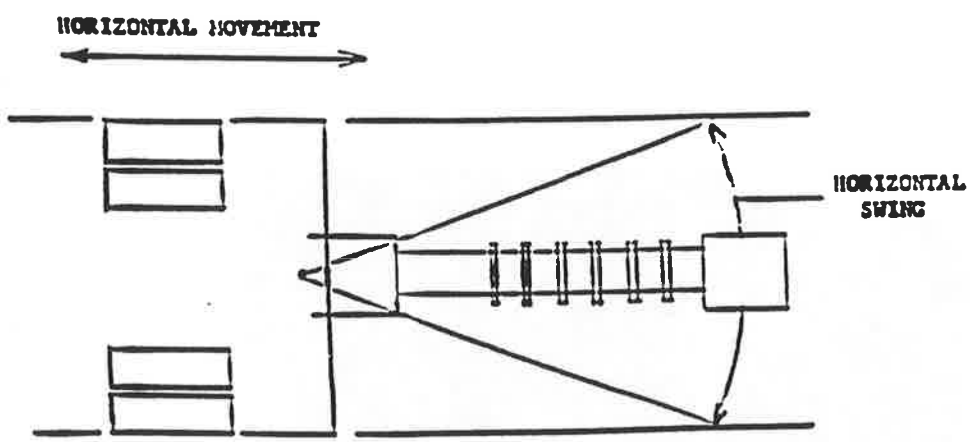


Fig. A.3. Whiphammer Equipment

Whiphammer equipment is relatively new, and the experience on bridge decks is therefore limited. Only a few states have reported using a whiphammer. Idaho DOT stated that it was the preferred method of deck removal, but also noted that some damage to stringers had occurred. Others reported excessive vibration and damage to the beams.



Vertical Adjustment



Horizontal Movement

Fig. A.4. Whiphammer Adjustment

Crusher. Concrete crushers remove pieces of concrete by applying opposing forces on either side of a concrete member. Crushers range in size from 86 lbs (39 kg) (suitable for hand-held operation) to units weighing upwards of 7 tons (6,350 kg) that must be mounted on excavators in the 40 ton (36,300 kg) range.

The models used for the removal of concrete in bridges typically have a jaw opening in the range of 14 to 33 in. (350 to 840 mm) and exert a force at the center of the blade of between 100 and 422 tons (890 to 3755 kN). In addition to producing little noise, dust, or vibration, rig-mounted crushers have the advantages of safety and high production rates. The equipment easily cuts through reinforcing bars, eliminating the need for torch cutting.

Water Jet Cutting. Water jets operating at a pressure 40,000 to 60,000 psi (280 to 410 MPa) can be used to cut plain concrete. However, for cutting reinforced and prestressed concrete, an abrasive must be introduced into the water jet and lower water pressures can be used. There are three groups of abrasives: mineral, metal, and artificial (ceramic).

It has been found that for members less than 12 in. (300 mm) thick, it is better to adjust the cutting speed to achieve penetration through the member in a single cut rather than making several passes at higher cutting speeds. For members more than 12 in. (300 mm) thick, better results can be obtained by making several passes. Water jetting has the advantages of being free from vibration and dust, while permitting a controlled cut without damage to the existing concrete. A major disadvantage of water jet use is the risk of maiming the operator or a bystander.

Formwork for Cast-In-Place Systems

The effort of constructing formwork for a cast-in-place bridge deck is one of the most cost consuming and the most expensive tasks that goes into the construction of a bridge deck. According to Kiewit Construction, approximately 55%-60% of the cost of a cast-in-place bridge deck goes into the labor and materials necessary to form the bridge deck. With the increasing cost of lumber in recent years, this percentage is rising. The other factor which is important is the time required to form the deck. The ideal estimate is around 20% of the overall time of the project is consumed in forming. In addition, no other operations can proceed until the forming operations are sufficiently advanced to allow placement of reinforcing bar to begin.

However, new form accessories are available in the market to speed up construction and save money. There are many companies who manufacture reusable, adjustable form hangers that reportedly speed the placing and stripping of bridge deck formwork. Since many of these forms are reusable, their initial cost can be spread over more projects. Some of the companies which are producing this kind of system are Economy Forms Corp., Des Moines, IA, and Symons Corp., Des Plaines, IL. Borg Adjustable Joist Hanger Co. of Wayzata, MN, is manufacturing a special kind of hangers. The top lip is from 1/2" (12.5 mm) steel with a milled slope on the top of the embedded portion to allow easy removal without greasing or other special steps. The adjusting range allows direct use of 2x6's through 2x12's. Modern Bridge Forming Company, St. Louis, MO, has introduced an innovative system that dramatically streamlines the concrete forming process for bridges and elevated highways. This system is made of steel forms which are reusable, can be adjusted to fit girders of different widths and depths and can be used for both steel and concrete girders. The whole system is mechanized for easy operation. The other reusable bridge forming sytem is Road Atlas system by Borode, Inc., Dallas, Tx. It is a modular system made of fiber glass construction. It has two basic sizes that conveniently covers spans from 3 ft 1 in. to 5 ft 7 in. (940 mm to 1700 mm) and 5 ft 7in. to 8 ft 2 in. (1700 mm to 2500 mm). Literature shows that a 4 ft x 8 ft (1220 mm x 2440 mm) section can be installed in 10 mins. with two persons.

Prestress Supply Inc., Lakeland, FL, has a new system for quick and easy positioning of stay-in-place concrete panels. This system uses a new steel framing which allow seal strip underneath the panel to open before placing over the girder. The seal strip work as grout barrier. Overall, the manpower crew can be reduced drastically with this system.

Full-Depth Precast Deck System

Literature review was performed for different full-depth precast deck systems. The following pages summarizes some of the projects studied. Biswas (49) and Issa (50) have done extensive study on precast full-depth sytems.

Project: Pintala Creek Bridge, Montogomery County, Alabama (1973)
Span: Four 34 ft (10 m) long spans
Features: Full width precast panels of 26 ft (7.8 m) wide, 7 ft (2 m) long and 6.5 in. (165 mm) thick with a complete curb, non-composite deck, no skew or superelevation
Deck to girder connection: 1/2 in. (12.5 mm) bolt was used as hold down device
Comments: 1980 inspection showed no evidence of leakage

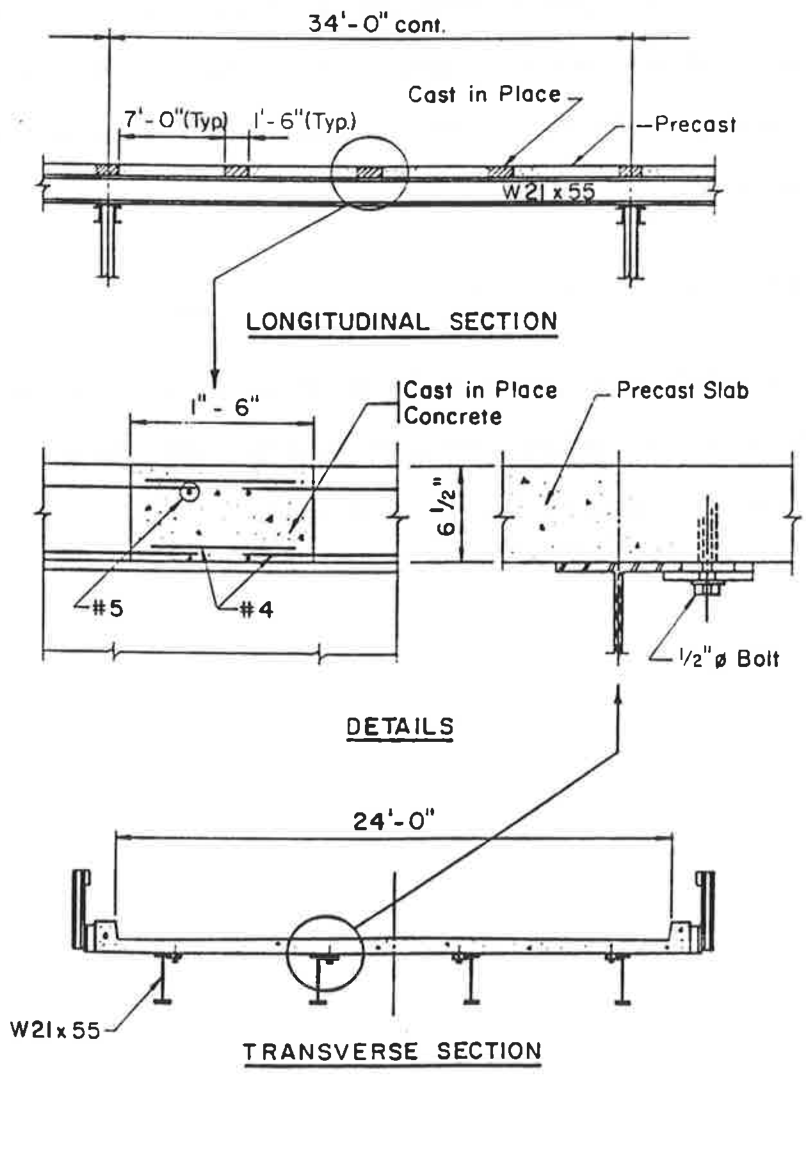


Fig. A.5 Longitudinal and Transverse Sections

Project: Kosciuszko Bridge, Brooklyn-Queens Expressway, New York (1971)

Span: Not reported

Features: Non-composite system, precast panels 6 1/2 in. (165 mm) thick, 5 ft to 8 ft (1.5 m to 2.4 m) long and 34 ft (10.2 m) wide with 1/4 in. gap between adjacent panels

Deck to girder connection: 3/4 in. (19 mm) dia bolt was used as hold down device, 1/4"x4"x6" (6.25 mm x 102 mm x 152 mm) Neoprene pads were used for bearing

Comments: No problem were seen during inspection

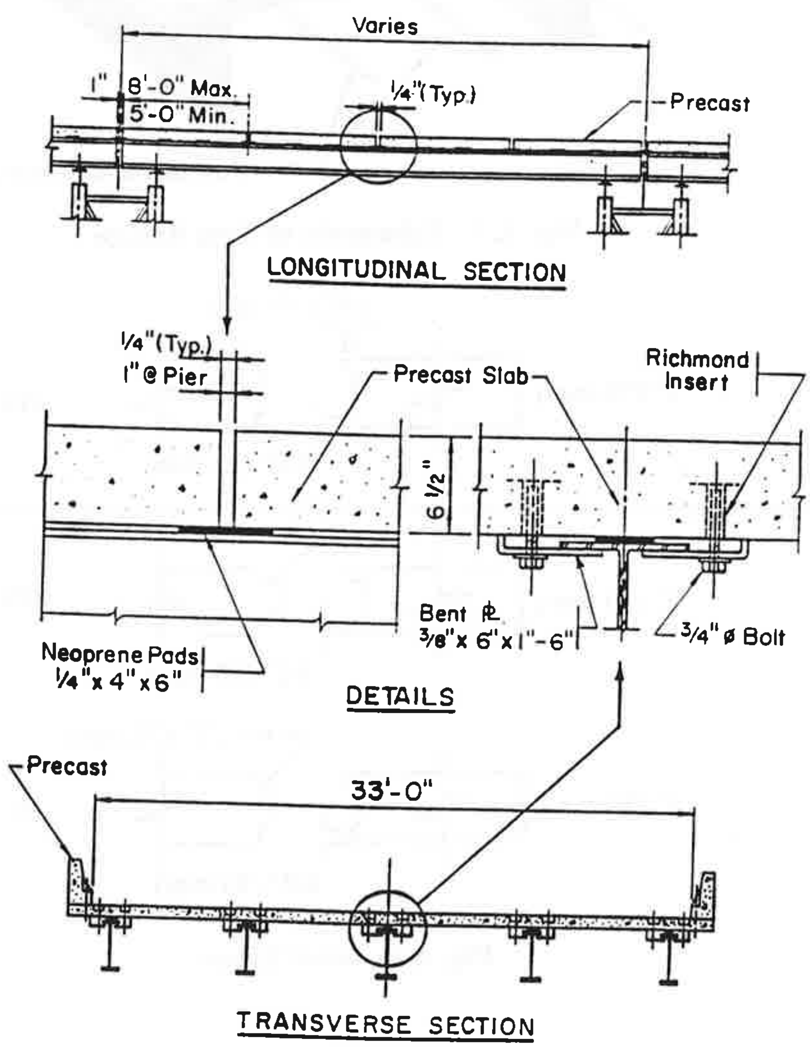


Fig. A.6 Longitudinal and Transverse Sections

Project: Purdue University and Indiana State Highway Commission
Span: Experimental two span system
Features: 3" (75 mm) thick precast panels with post-tensioning system. Different transverse joint details were evaluated.
Deck to girder connection: Not reported
Comments: No problems were reported

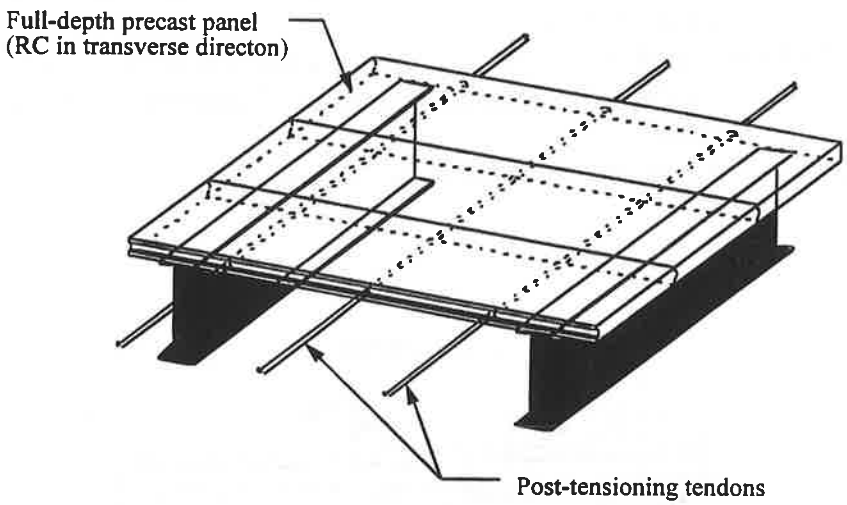


Fig. A.7 Schematic of Test Bridge

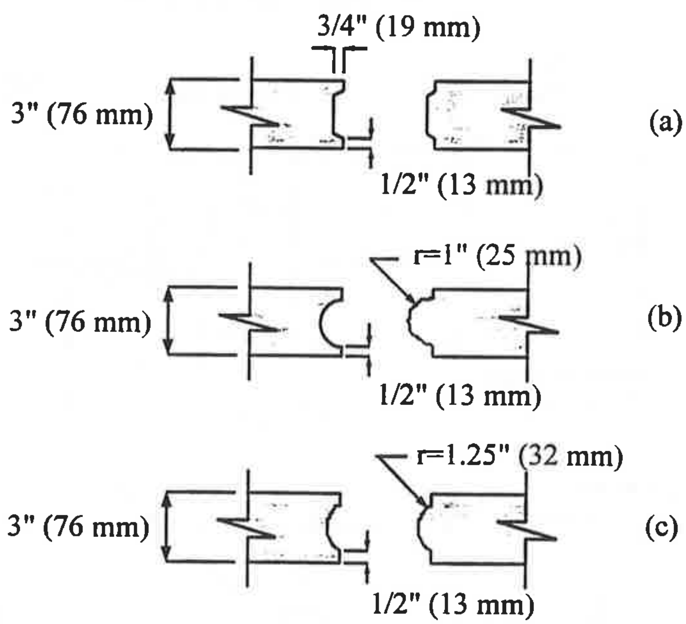


Fig. A.8 Joint Types

Project: Bloomington bridge, Indiana (1982)
Span: 125 ft (37.5 m) single span
Features: 4 ft (1.2 m) precast prestressed panels with tongue-and-groove joint, post-tensioning (90 ksi (620 MPa after all looses) in the longitudinal direction. Construction was completed in 47 days
Deck to girder connection: Railroad tie down clips were used for hold down
Comments: Cracking, spalling, and leakage at the panel joints

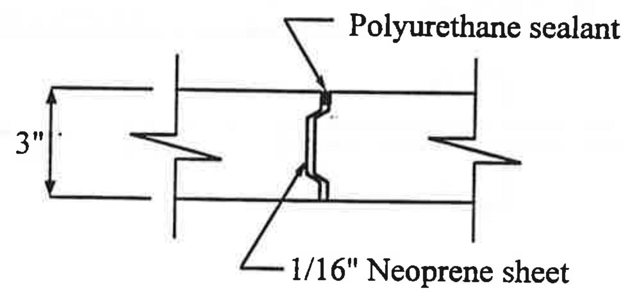


Fig. A.9 Joint Detail for Prototype Bridge

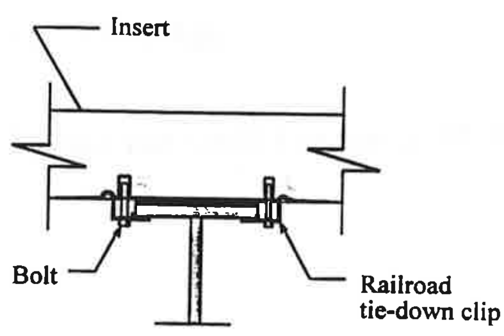


Fig. A.10 Tie-down Connection for Indiana Experimental Bridge

Project: Big Blue River Bridge, Knightstown, Indiana (1970)
Span: Three spans bridge, 70, 60, 70 ft (21, 18, 21 m)
Steel girders @ 6 ft (1.8 m)
Features: Full width panels, 4 ft (1.2 m) long and 39 ft (12 m) wide
Tranversely pretensioned and longitudinally post-tensioned
Deck to girder connection: Not reported
Comments: No problems were reported

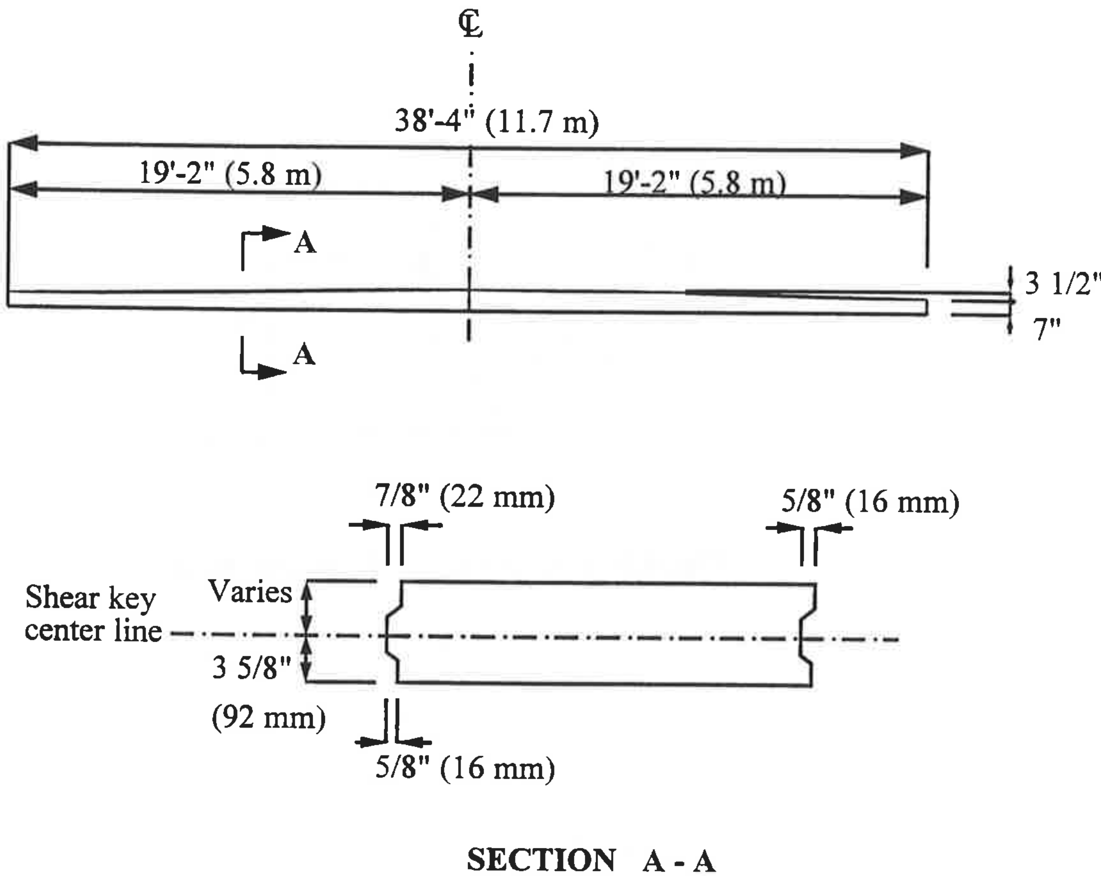


Fig. A.11 Elevation and Shear Key Detail for Knightstown Bridge

Project: Hannover Viaduct, Germany
Span: Not reported
Features: Composite system. precast deck on steel box girders, the panels were about 18 ft (5.4 m) wide
Deck to girder connection: Epoxy morter and high strength bolts were used for composite connection.
Comments: No problems were reported.

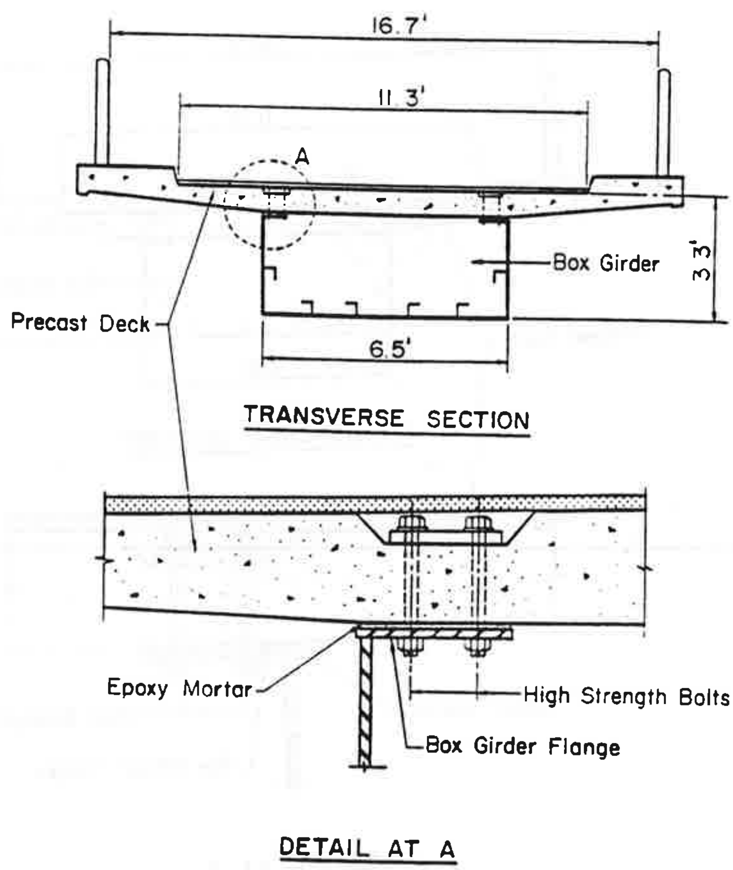


Fig. A.12 Transverse Section and Detail

Project: Emil-Schulz Bridge, Germany
Span: Not reported
Features: Composite system, precast deck on steel box girders. The panels were 47.3 ft (14.2 m) wide
Deck to girder connection: Shear studs in combination with spirals were used for composite action.
Comments: No problems were reported.

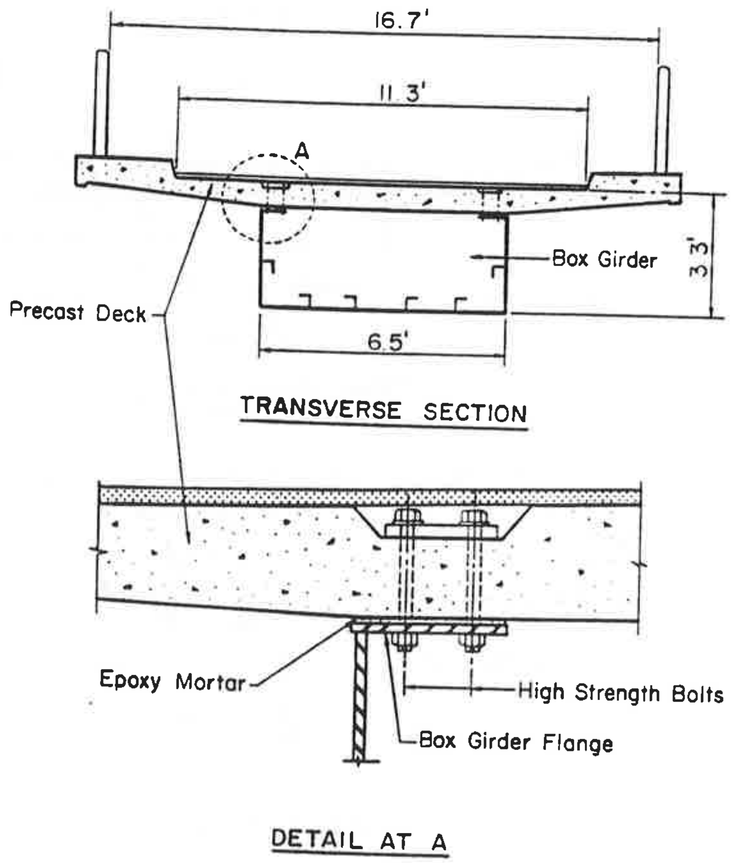


Fig. A.13 Transverse Section and Detail

Project: Amsterdam Interchange Bridge, New York (1973)
Span: Four simple spans, 33, 59, 66 and 60 ft (10, 18, 20 and 18 m)
Features: Staged construction, precast panel dimensions were 8 in. (203 mm) x 4 ft (1.2 m) x 22 ft (6.7 m)
Deck to girder connection: Two types of connections were used, standard 5x9 in. channel & bolted connection in epoxy morter.
Comments: The May 1980 inspection revealed no evidence of any problem in performance for the different methods of connections.

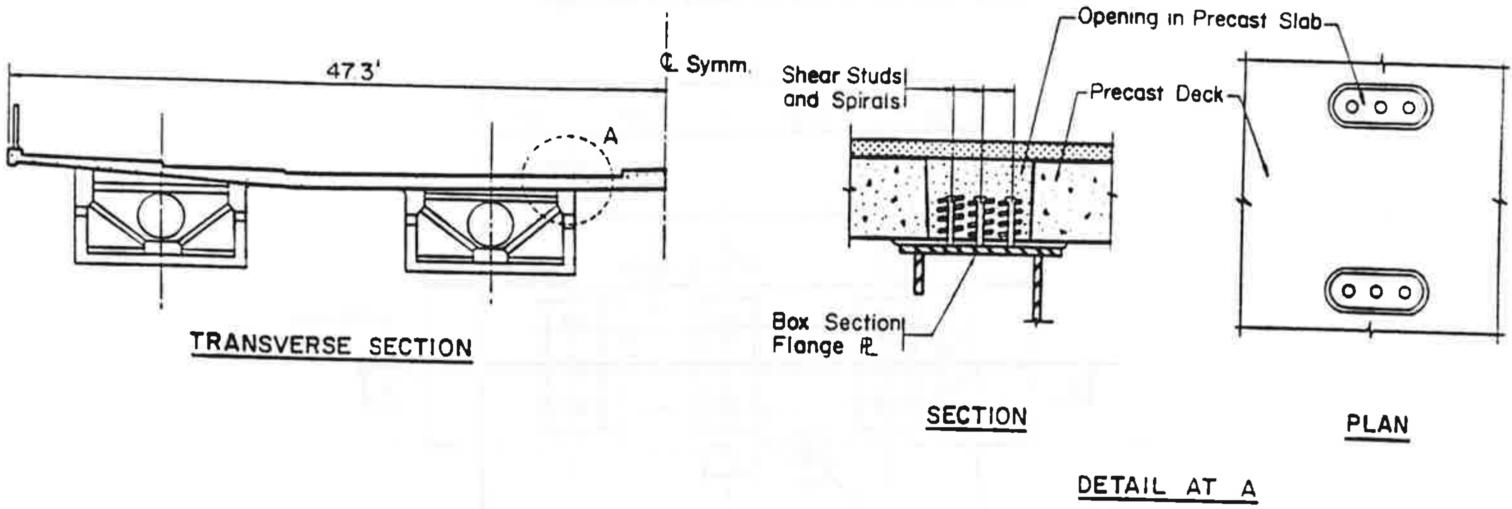
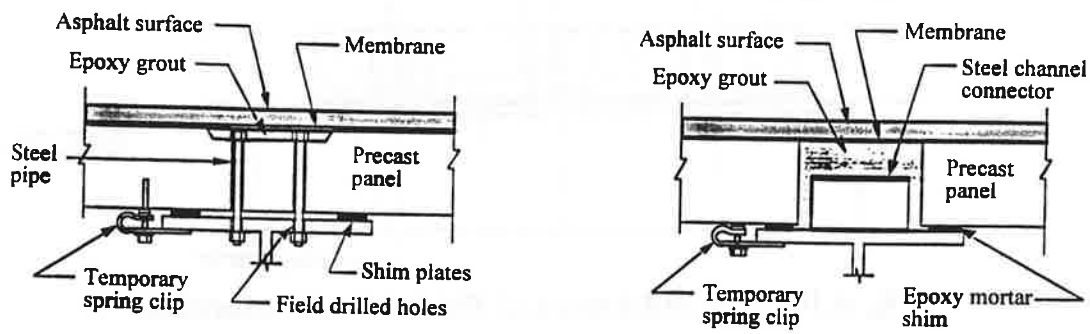


Fig. A.14 Plan and Section Views of Bolted Shear Connection



(a) Bolted Connection

(b) Welded Connection

Fig. A.15 Connection Details for New York Thruway Experimental Bridge

Project: Krum Kill Road Bridge, New York (1977)

Span: 50 ft (15 m) single span

Features: Precast panels, 7.5 in. (190 mm) thick and 5 ft (1.6 m) long, of two different widths, 42 ft (13 m) and 21 ft (6.5 m) are used.
3 ft wide longtitudinal joint at the crown was cast over continuity reinforcing bars extending from both panels.

Deck to girder connection: Welded shear studs and epoxy morter casted in shear pockets

Comments: During construction, cracks over the reinforcing bars were detected in the precast panels. The cracks were treated with a penetrating epoxy sealer. The may 1980, inspection indicated satisfactory performance although several joints have shown sign of leakage.

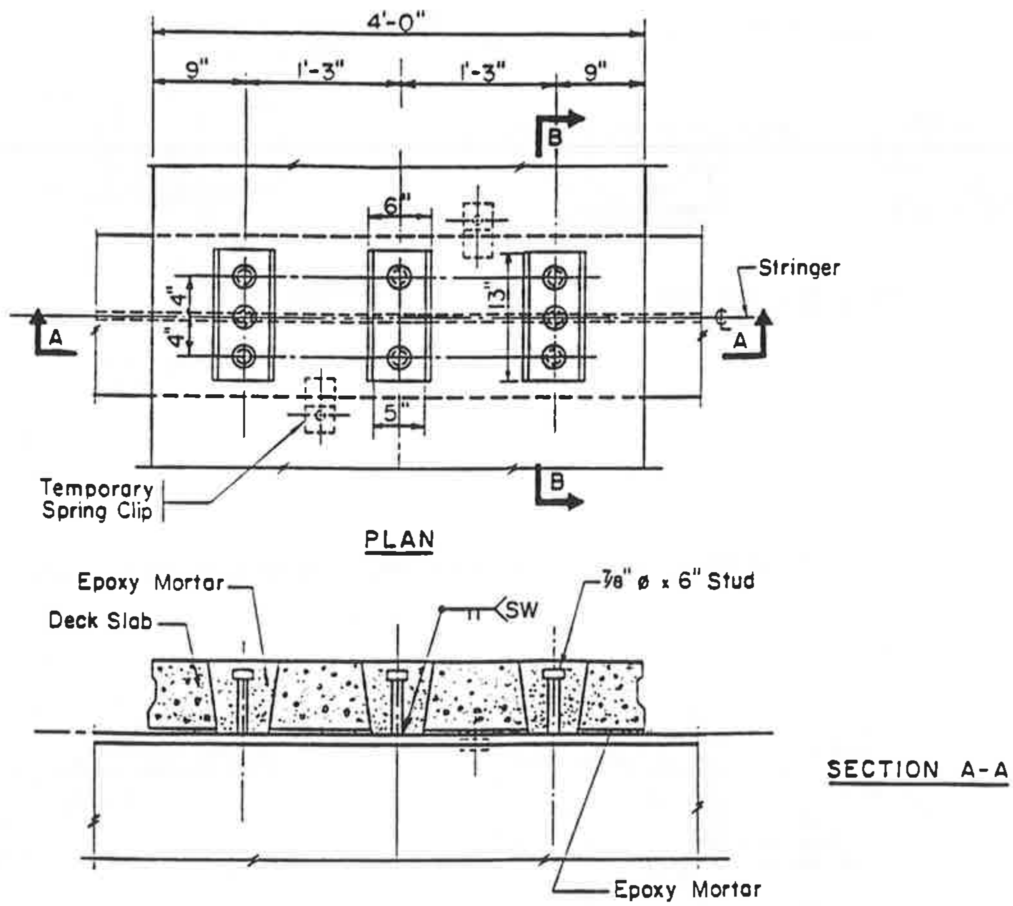


Fig. A.16 Plan and Section of Welded Stud Connection

Project: Harriman Interchange Ramp, New York (1977)
Span: Three span, 75 ft (23 m) each.
Curved bridge
Features: Full width precast panels of 8 in. (203 mm) thick, 4 ft (1.2 m) long and 54 ft (16.5 m) wide.
Deck to girder connection: Welded shear studs in the blockouts
Comments: Since this was a curved super-elevated bridge, the precast panels would not level on the beam flanges. Therfore, the epoxy morter bed wasthicker on one edge of the flange than the other.

All details are similar to Amsterdam Interchange Bridge

Project: Bridge No. 1 over Rondout Creek, Kingston, New York (1974)
Span: Three spans, 700 ft long main suspended middle span.
Features: Precast panels 6-7 in. (150 mm-175 mm) thick 9 ft. (2.7 m) wide. the panels were transversely prestressed to accommodate handling stresses. A simple V male-female joint between panels.
Deck to girder connection: $^3/_4$ in. (19 mm) dia stud with a 1/2 in. (12.5 mm) thick elastomeric bearing pad was used at the interface between the bottom of slab and top of stringer.
Comments: No problem were reported.

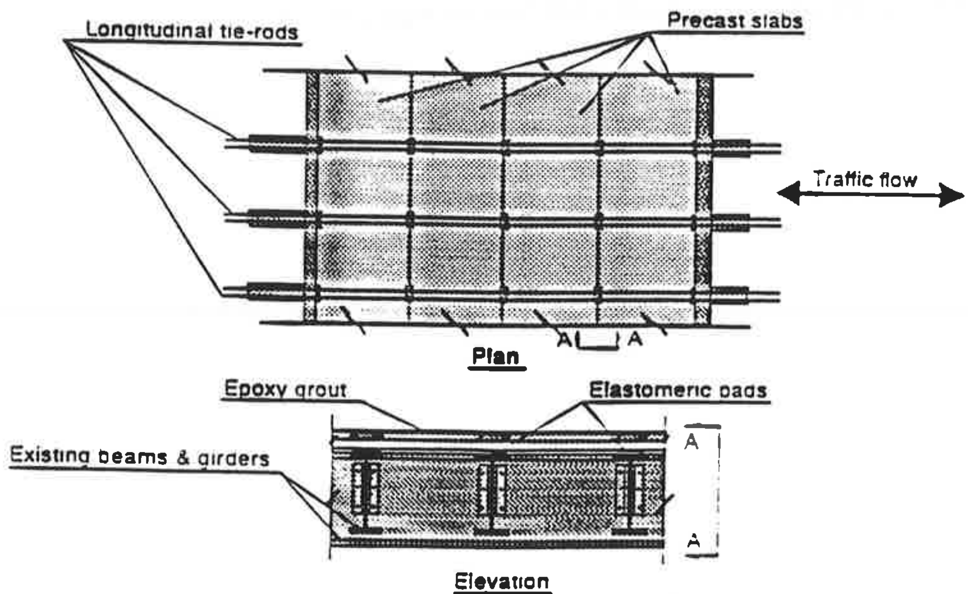


Fig. A.17 Plan and Elevation Views of the New Deck

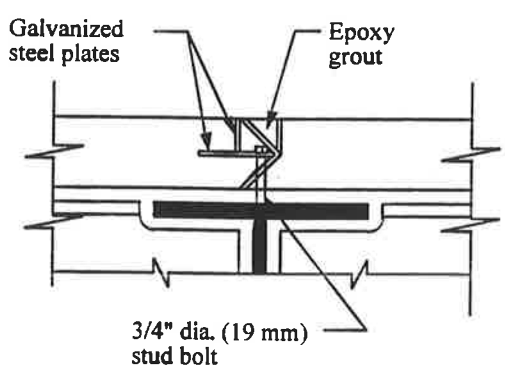
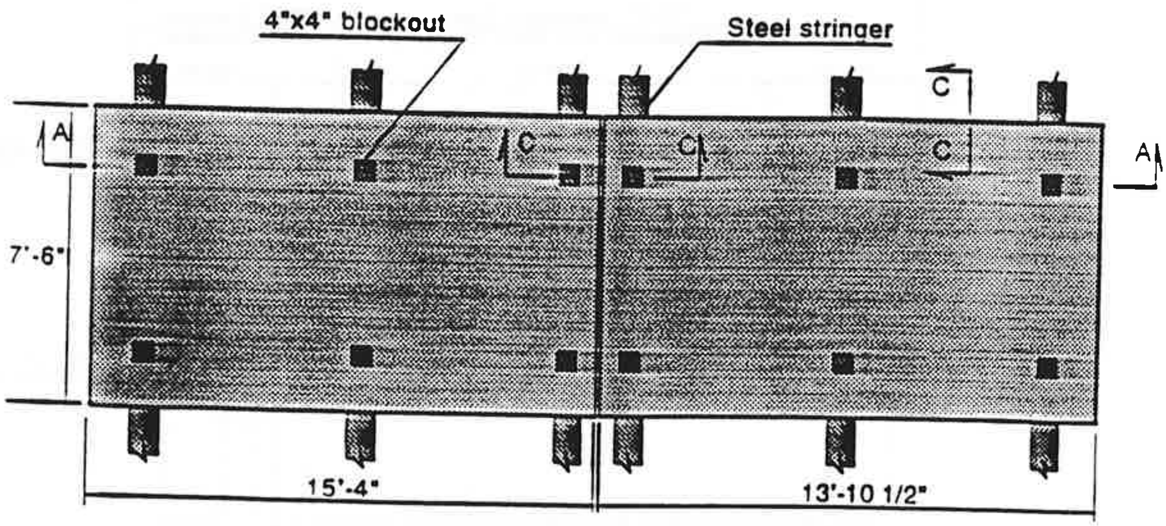


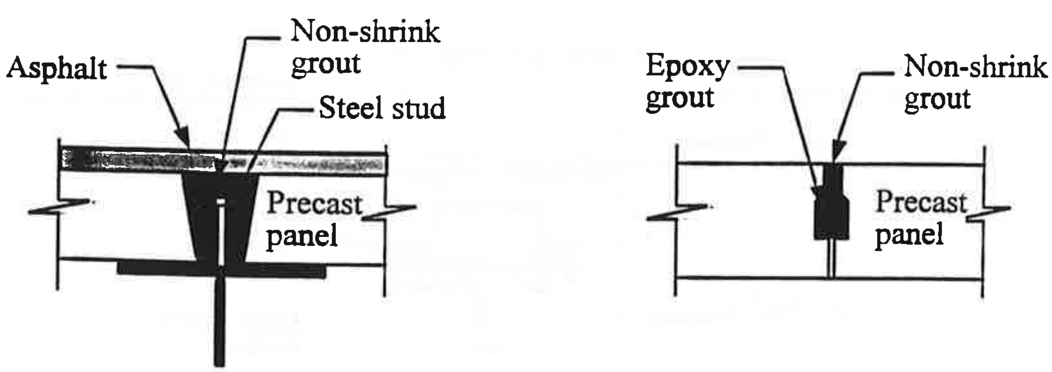
Fig. A.18 Joint Detail of Rondout Creek Bridge

Project: Bridge No. 6 over Delaware River, New York (1978)
Span: Three spans of total span length of 675 ft (205 m)
Features: Precast panels of 7.5 in. (190 mm) thick, 7.5 ft (2.3 m) long and half roadway width.
Deck to girder connection: 7/8” dia x 4” (22 mm x 100 mm) shear studs and non-shrink cement grout.
Comments: Reflective cracks along the longitudinal joint



Plan

Fig. A.19 Deck Plan View and Panel Dimensions



(a) Connection Detail **(b) Transverse Joint Detail**

Fig. A.20 Details for Delaware River Bridge

Project:	Quakertown Interchange Bridge, Pennsylvania (1981)
Span:	Suspended cantilever system with 75 ft (22.5 m) suspended span
Features:	Composite deck in the suspended span. Non- composite deck in the cantilever span. Precast panels of 6.5 in. (165 mm) thick, 7.25 ft (2.3 m) long and 17.5 ft (5.3 m) wide were used. Nominal longitudinal post-tensioning.
Deck to girder connection:	1" x $1^3/_8$" (25 mm x 35 mm) elastomeric strips were glued to the top of the flanges to contain the epoxy mortar to provide uniform bedding of the panels. Latex concrete grout in the shear pockets.
Comments:	No problems were reported.

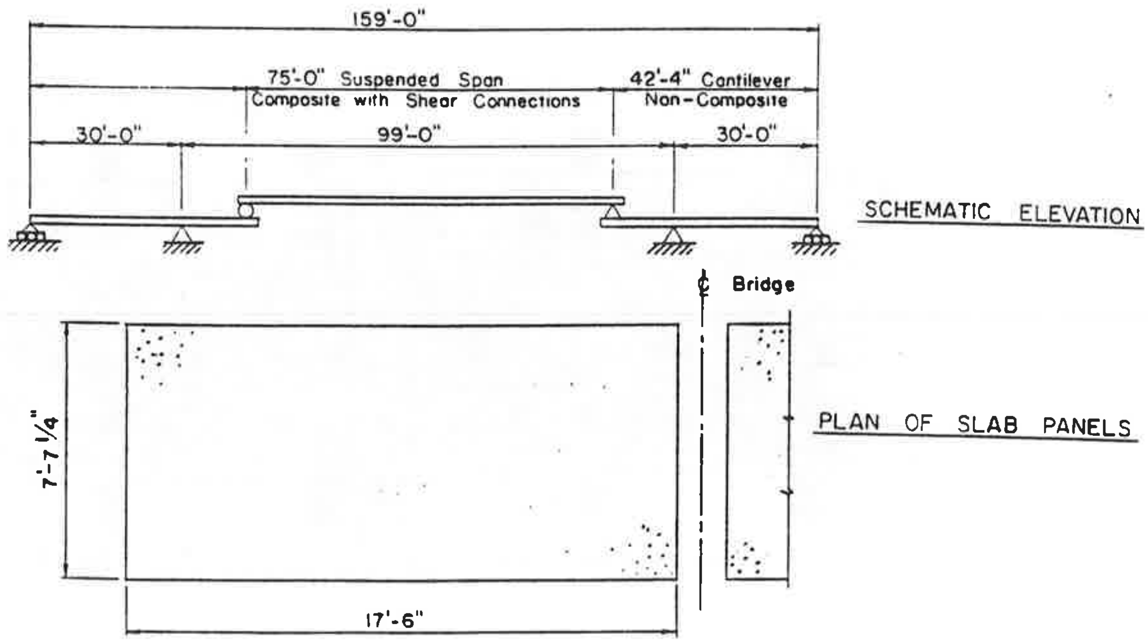


Fig. A.21 Schematic Elevation and Panel Dimensions

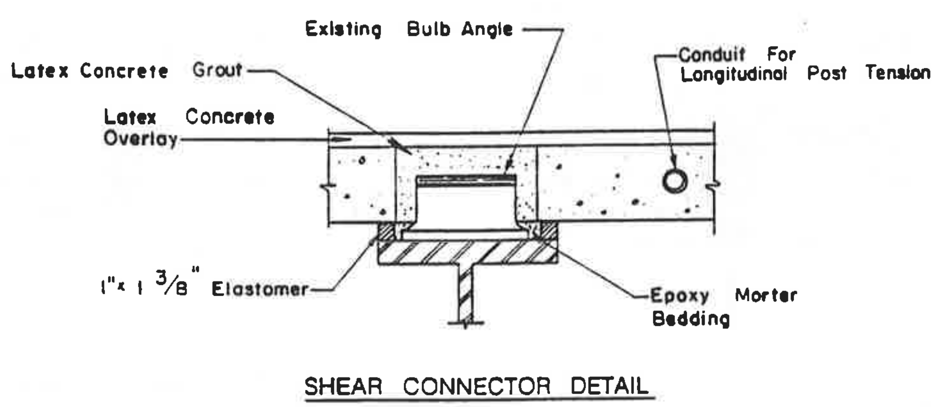


Fig. A.22 Connection Detail

Project: Connecticut River Bridge (1982)
Span: 224 ft (68 m) typical interior span with a total bridge length of 1224 ft (373 m)
Features: Precast panels were transversely pretensioned and longitudinally post-tensioned.
Lightweight aggregate concrete was used.
Deck to girder connection: Welded studs and Non shrink cement grout which was used as bedding and in stud pockets
Comments: No problems were reported

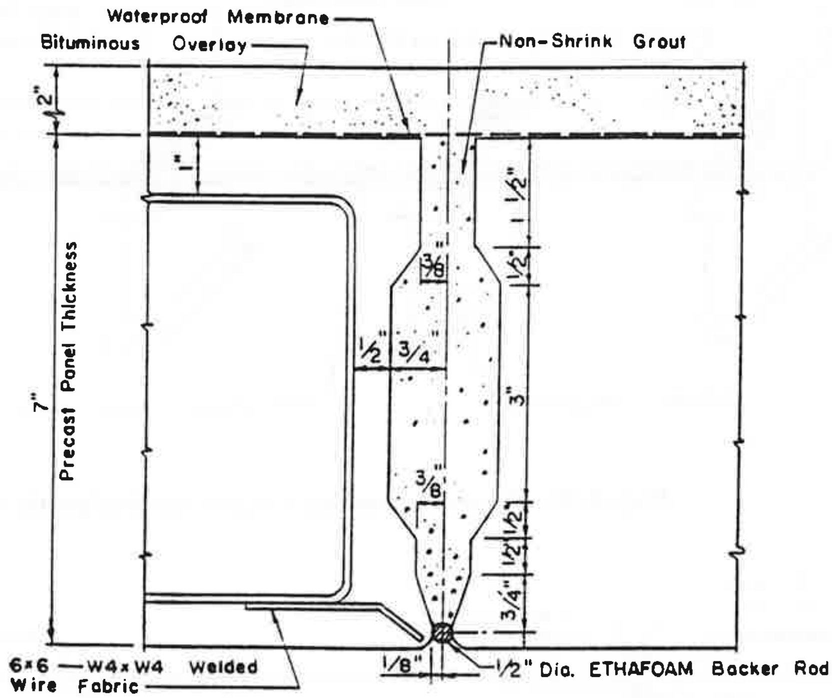
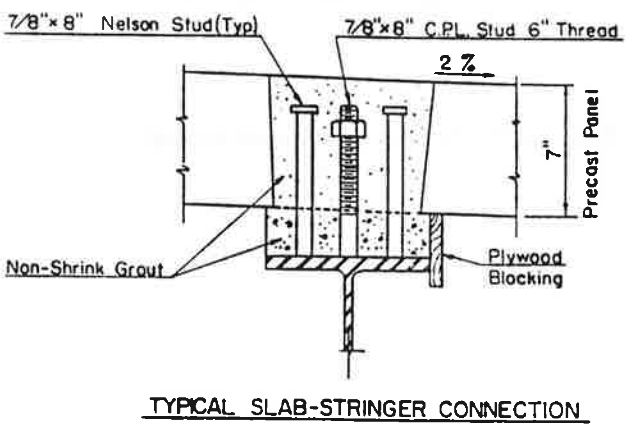


Fig. A.23 Typical Transverse Joint Details



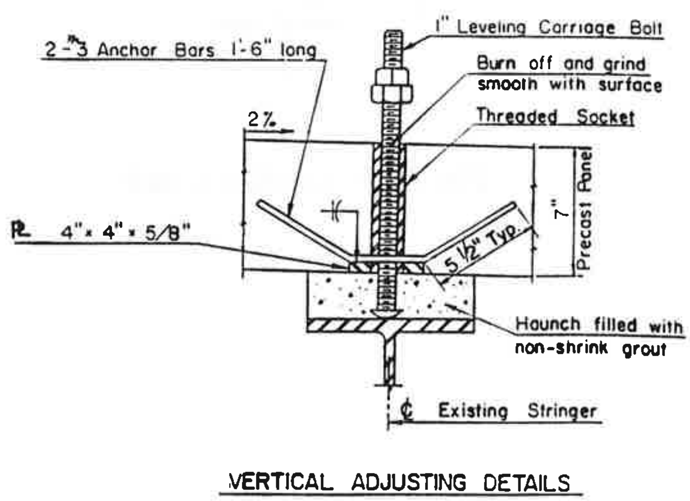


Fig. A.24 Connection Details

Project:	Interstate 84- Connecticut Route 8 Interchange in Waterbury, Connecticut
Span:	Six spans, 700 ft (213 m) long
Features:	Reinforced concrete panels of 8 ft (2.4 m) wide and 26 ft-8 in. (8.2 m) long. Longitudinal post-tensioning of 150 psi (1.0 MPa) Leveling bolts for adjusting the eleveation.
Deck to girder connection:	Three studs per row. Non-shrink grout for stud pockets and panel joints.
Comments:	No problems were reported.

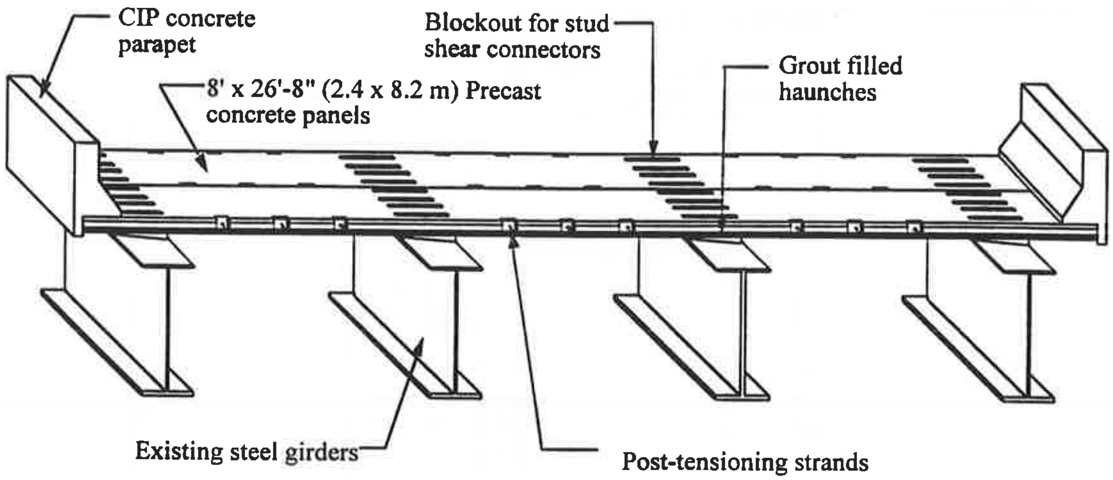


Fig. A.25 Typical Precast Panels for Bridge 03200

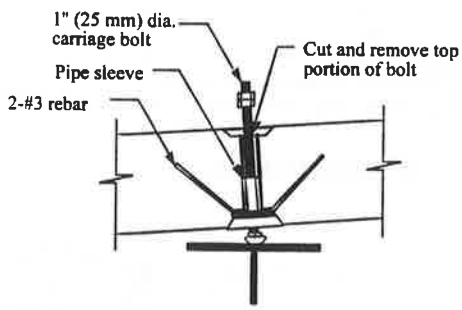


Fig. A.26 Leveling Detail

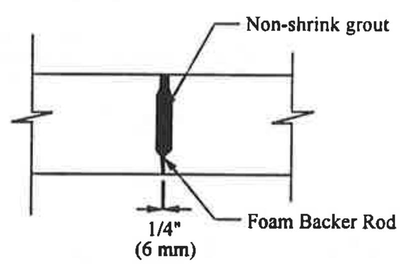


Fig. A.27 Transverse Joint Detail

Project: Queen Elizabeth Way-Welland River Bridge, The Ministry of Transportation of Ontario, Canada

Span: Redecking of lanes of a total of 954 ft (290 m) long and 40 ft (12.2 m) wide.

Features: Panels dimensions were 43.5 ft (13.3 m) wide, 8 ft (2.4 m) long and 8.9 in. (225 mm) thick. Longitudinal post-tensioing of 435 psi (3.0 MPa)

Deck to girder connection: Shear studs in blockouts

Comments: No problems were reported

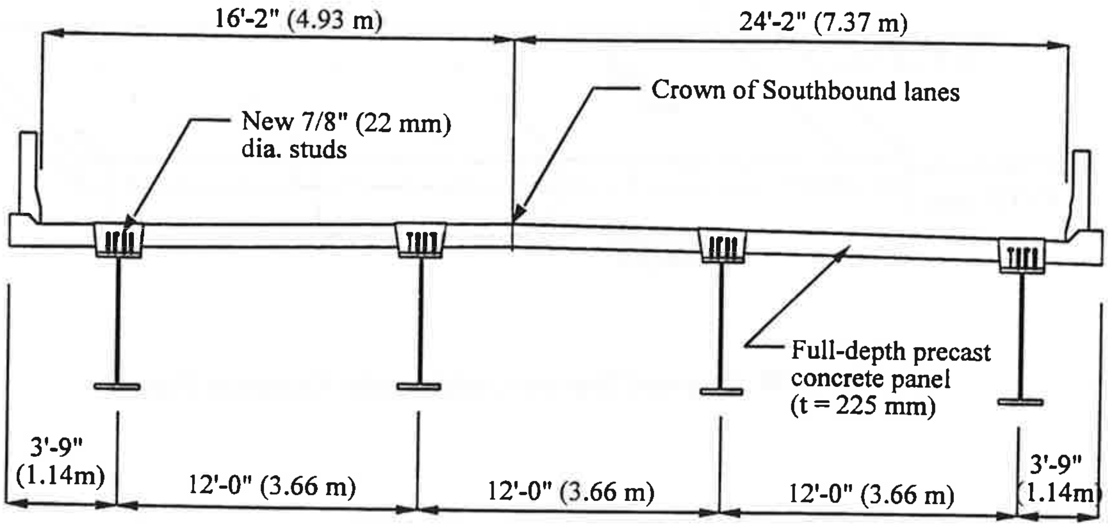


Fig. A.28 Typical Cross Section

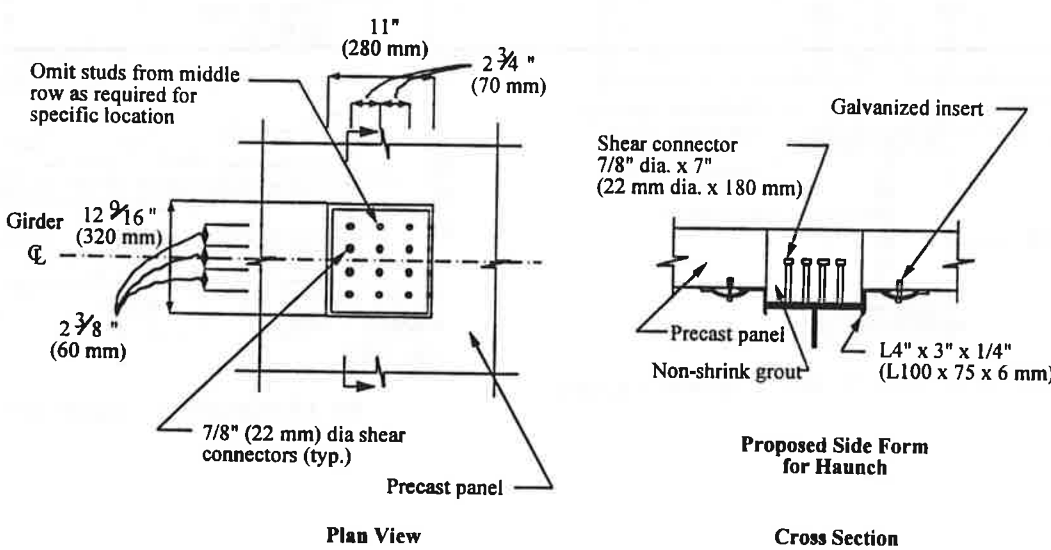


Fig. A.29 Connection Detail

Project: The Woodrow Wilson Memorial Bridge on I-95 over the Potomac River
Span: 5900 ft (1800 m)
Features: Panel dimensions were 10 to 12 ft (3.0 to 3.7 m) long, 47 ft (14 m) wide and 8 in. (203 mm) thick. Longitudinal and transverse post-tensioning
Deck to girder connection: Studs as shear connectors
Comments: No problems were reported

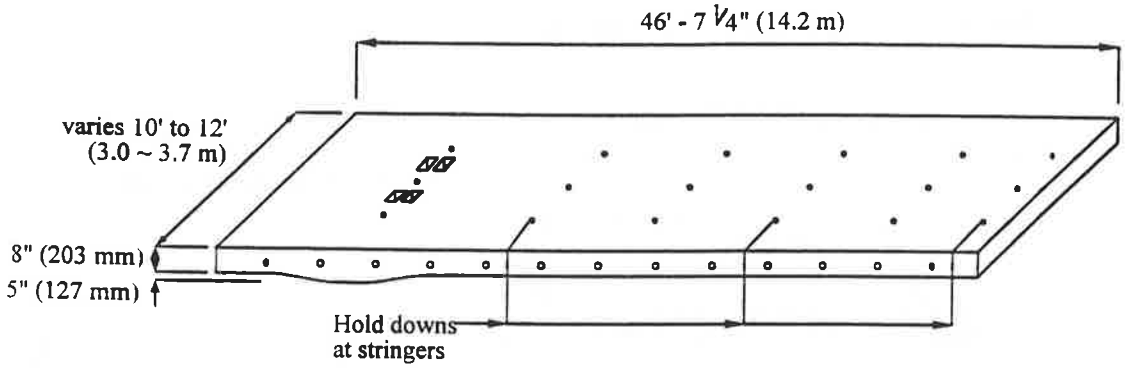


Fig. A.30 Typical Precast Lightweight Concrete Panel

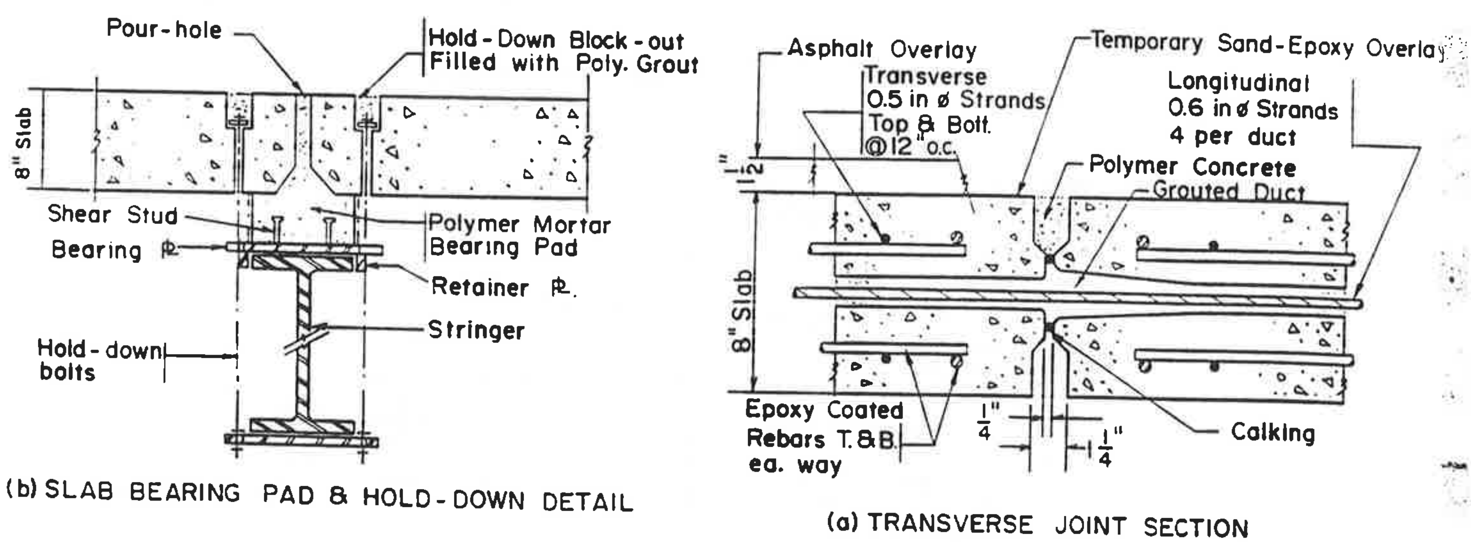


Fig. A.31 Typical Joint

Project: Lake Koocanusa Bridge Rehabilitation, Lincoln County, Montana

Span:

Features: Composite deck slab, Two half width precast panels of 15.58 ft (4.75 m) and 20.58 ft (6.30 m) wide and 8 ft (2.4 m) long.

Deck to girder connection: Used shear studs as connector. Shear pockets are grouted with high strength gorut

Comments: No problems were reported

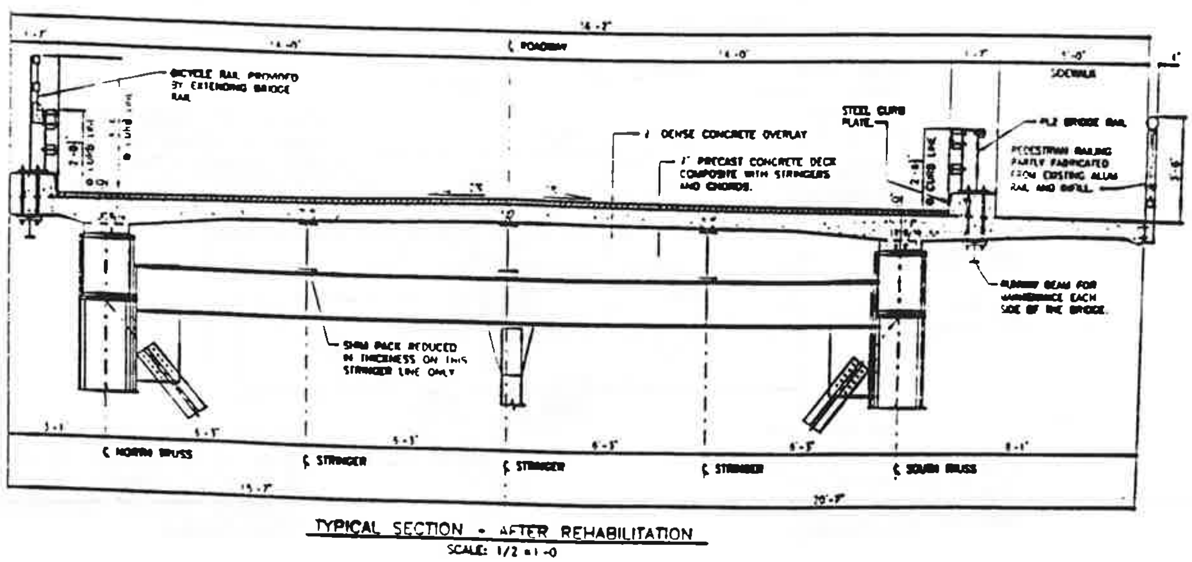


Fig. A.32 Cross Section of the New Deck Slab

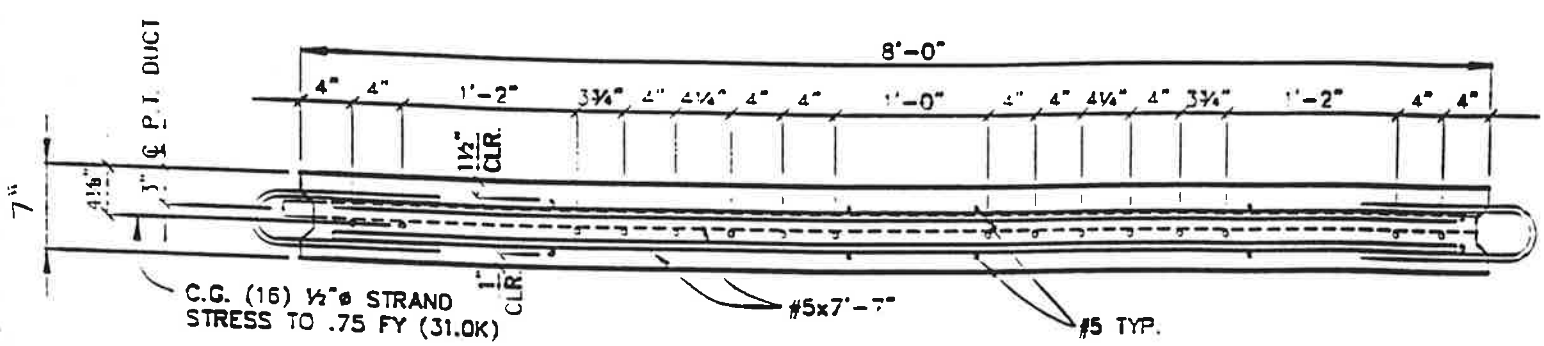


Fig. A.33 Prestressed Precast Panel Cross Section

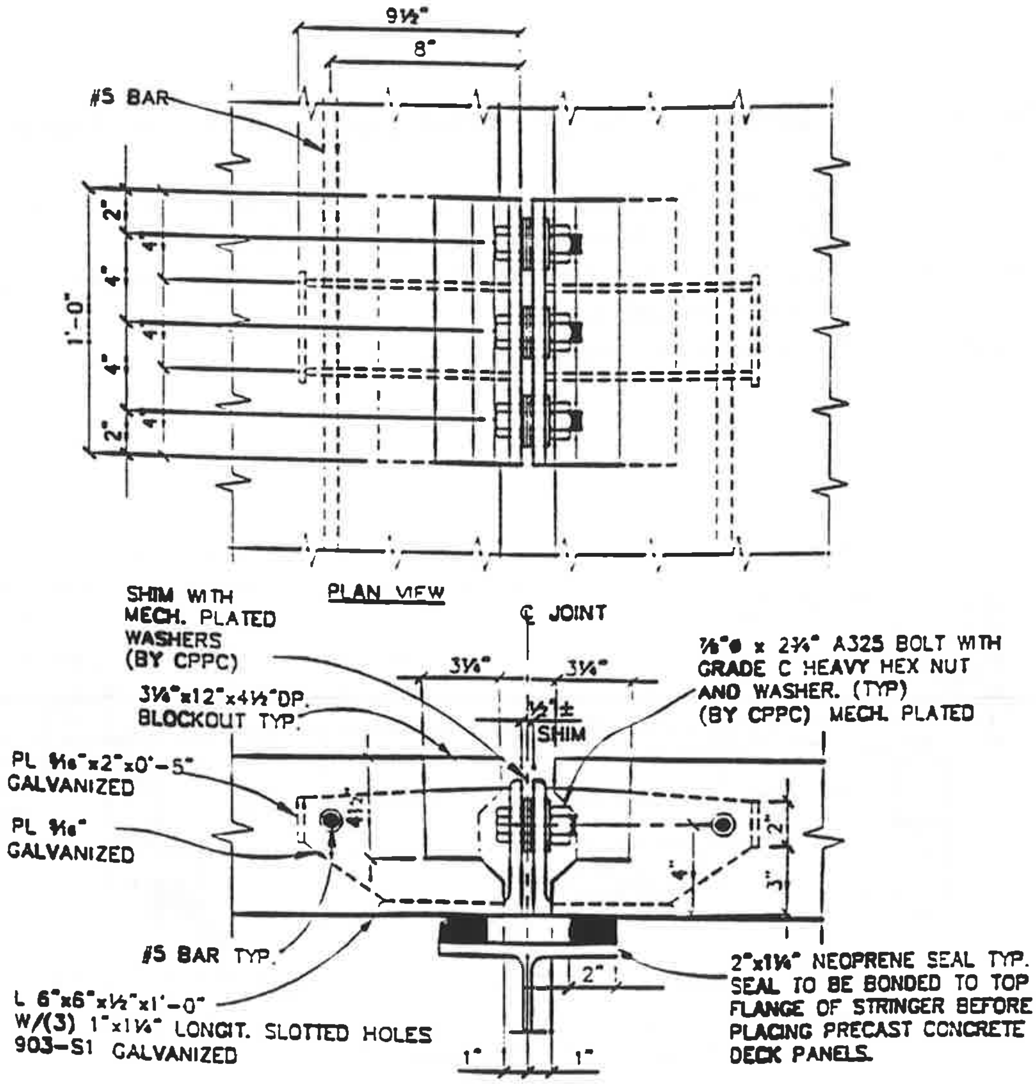


Fig. A.34 Longitudinal Connection between the South and the North Panels

Miscellaneous Deck Systems

Steel Grid : A steel grid bridge flooring system (*51*) mainly consists of a flat bearing bar (main bar) in transverse direction with secondary bars placed perpendicular to the main bars, as shown in Fig. A.35. Steel grid floors are either welded or bolted to the supporting members to achieve composite action. Steel grid can be filled with concrete either at the shop or at the construction site to create a smooth riding surface. Concrete filled steel grid floors have been installed on bridges for over 50 years.

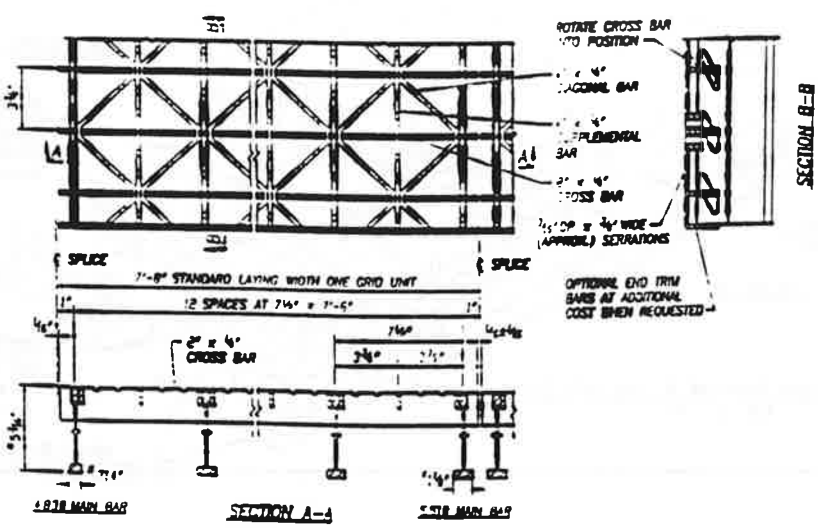


Fig. A.35 Steel Grid Deck System

Exodermic Bridge Deck : An Exodermic bridge deck (*52*) consists of a fabricated steel grid for the bottom portion and a reinforced concrete slab for the top portion, as shown in Fig. A.36. A part of the steel grid portion extends upward into the reinforced concrete in order to achieve a composite deck.

An Exodermic system can be either cast-in-place or precast. Embedding the shear connectors in the concrete haunch area allows Exodermic decks to be made composite with the steel girders. The haunches can be poured together with the reinforced concrete deck when a cast-in-place deck is used, or separately when using a pre-cast deck. The advantages of an Exodermic deck system include light weight, rapid erection, and simplified construction staging.

The Exodermic deck system has been used since 1984 (53). Kentucky DOT, Chicago DOT, and others have recently selected the use of an Exodermic deck for bridge rehabilitation.

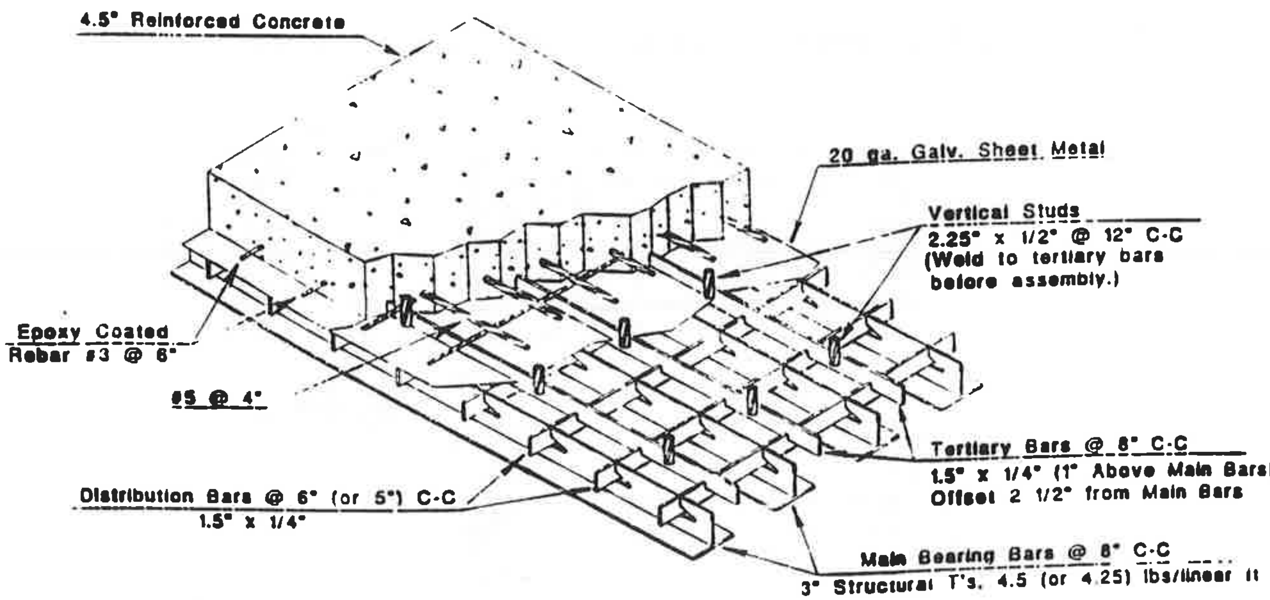


Fig. A.36 Exodermic Bridge Deck System

Inverset : The New York State DOT solved a bridge repair problem by using the "Inverset deck system" (*54*). In this system, a fabricated beam pair, shown in Fig. A.37, is inverted and supported as shown in Fig. A.38. Cross-members spaced at regular intervals on top of the beams support matching cross members below the beams. When concrete is cast in the forms, the combined weight of the forms and the concrete produces a prestressed effect on the beams as shown in Fig. A.39a. When the cured unit is turned over, a stress configuration shown in Fig. A.39b results. The compression stress in the bottom flange reverses to near zero and compression stresses result in the concrete deck.

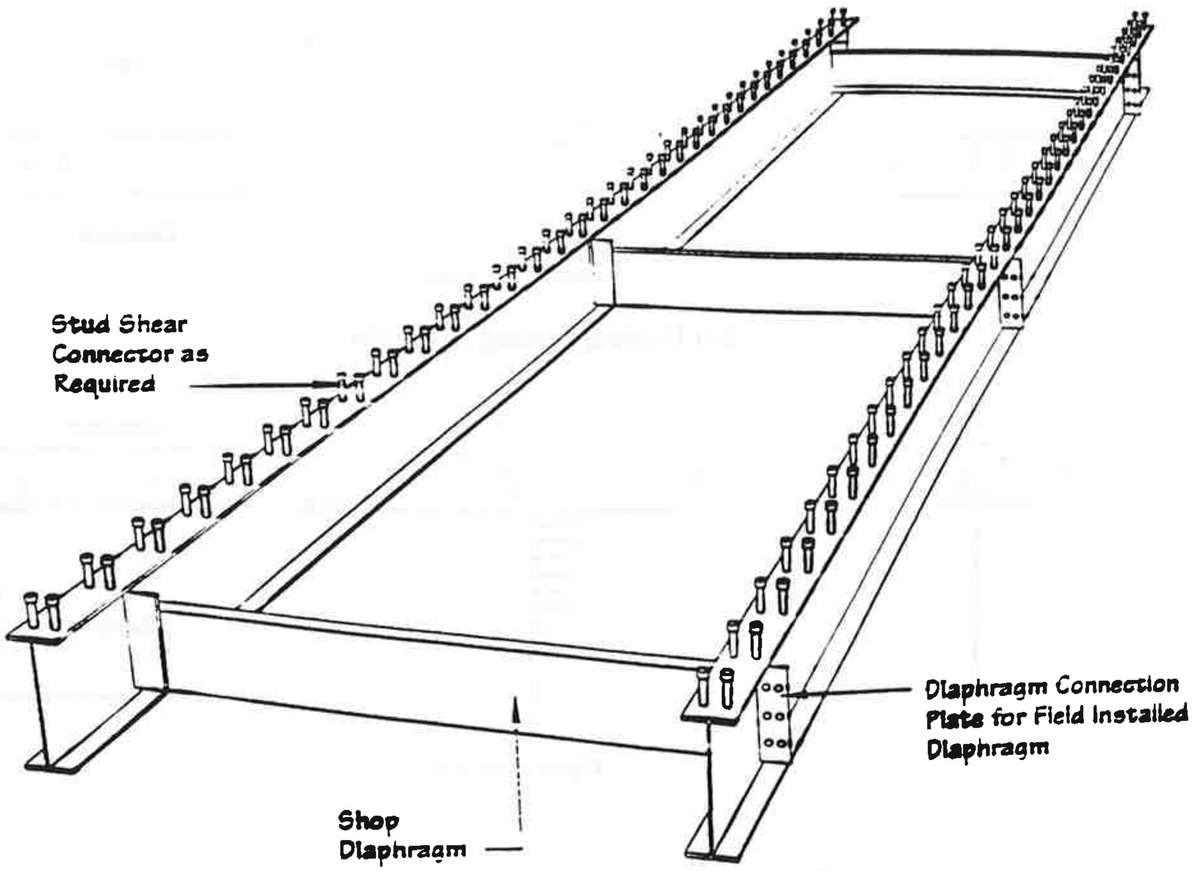


Fig. A.37 Prefabricated Beam Pair

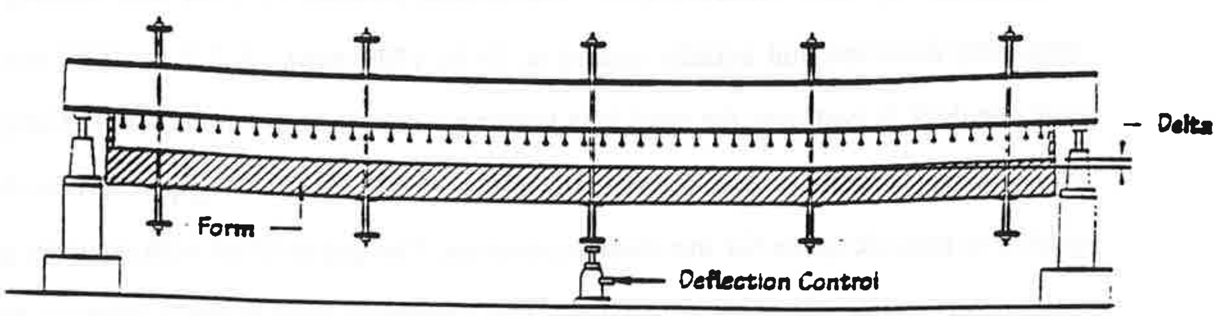
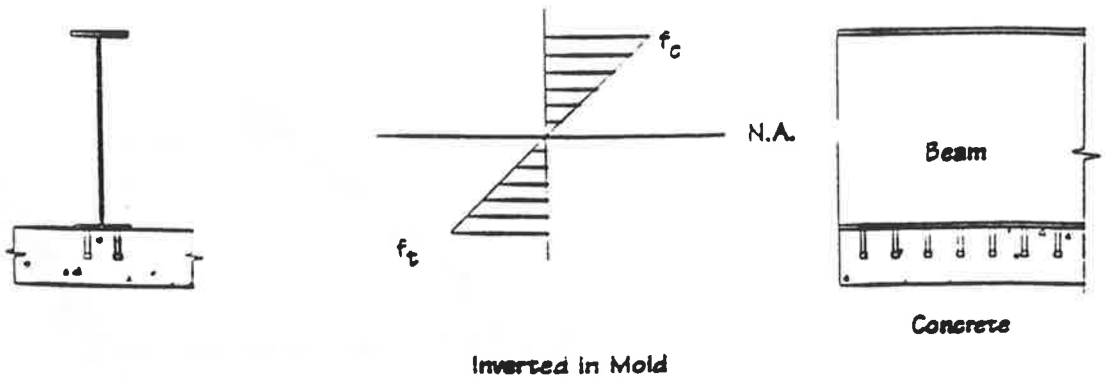
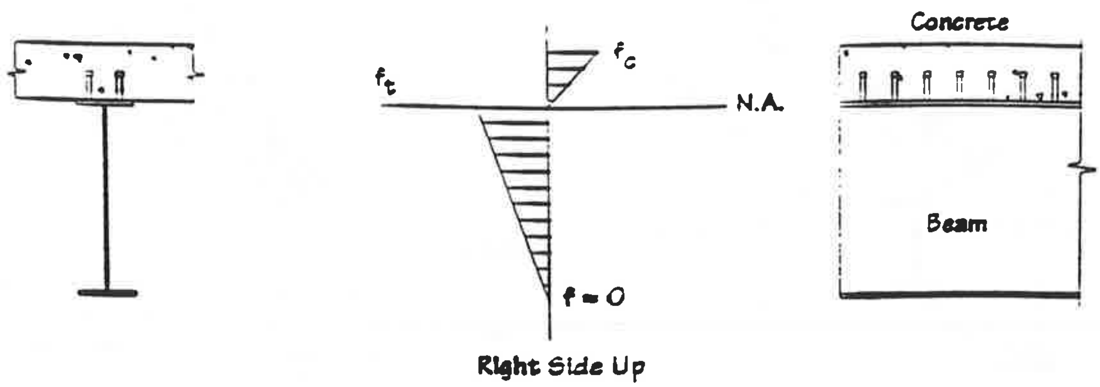


Fig. A.38 Inverted System ready for casting Concrete



(a) During casting cocnrete



(b) After turning the system right side up

Fig. A.39 Resulting Stresses

IKG Greulich Bridge Flooring system : A new bridge deck system is under development by IKG Greulich (*51*). The system consists of steel bars running in the transverse direction and equally spaced at 16 in. (400 mm). A 3.5 in. (87.5 mm) thick concrete deck is cast over the steel bars forming a composite section of a total height of 8 in. (200 mm), as shown in Fig. A.40 and Fig. A.41. A full width gap is formed over the girders to provide space for the shear connectors. The gap is filled with rapid set grout or with concrete after the deck is installed. The transverse joint between adjacent panels is also grouted. Fig. A.42 shows the two alternatives that can be used to form the transverse joint. The deck system is post-tensioned longitudinally over the supporting girders.

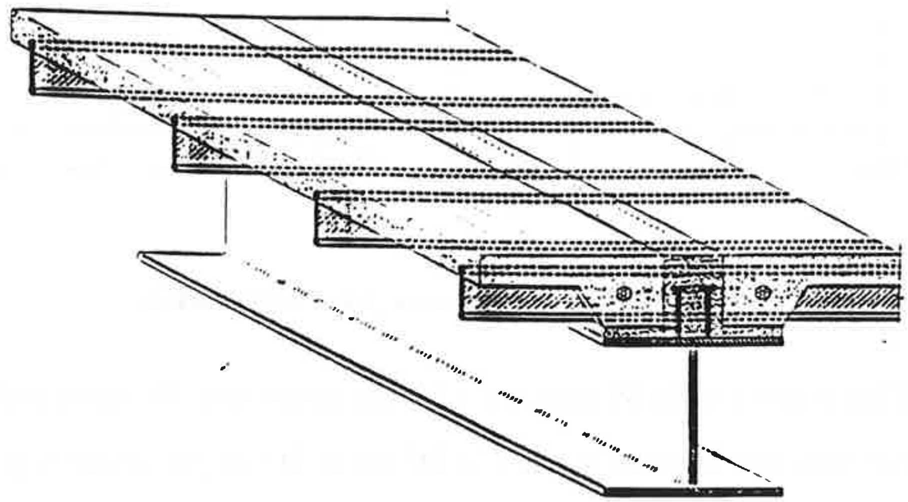


Fig. A.40 Isometric View of the IKG Greulich Deck System

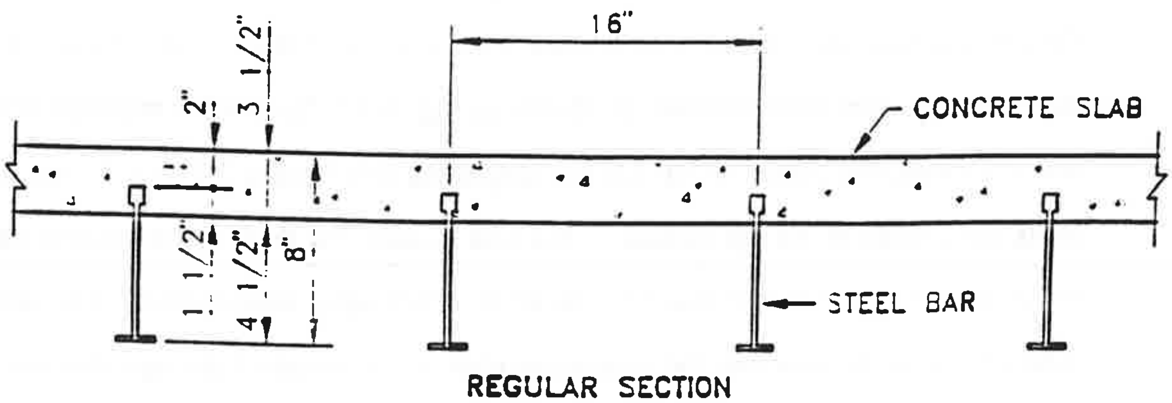


Fig. A.41 Cross Section of the IKG Greulich Deck

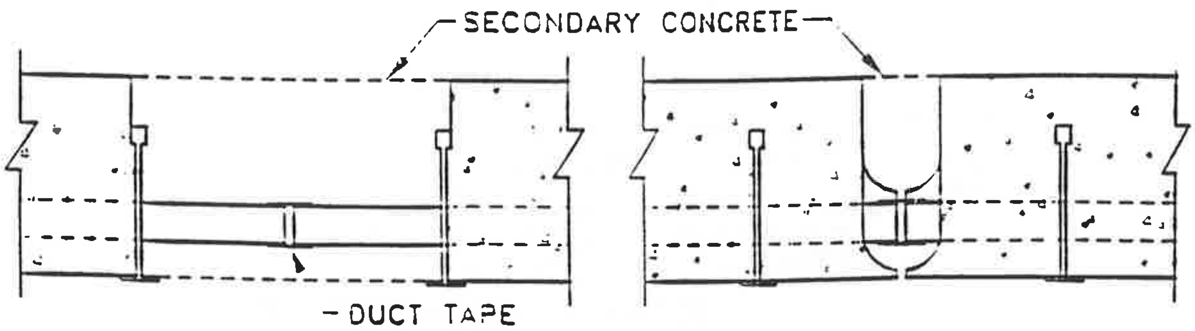


Fig. A.42 Transverse Joint Alternatives

This system weighs 55 pounds (25 kg) per square feet. The estimated cost is $8.00 per square foot, and installed in place is $13.00 to $14.00 per square foot. Using post-tensioning reduces the possibility of cracks, especially over the transverse joints. The full width gap and the optimized spacing between the transverse steel bars make the system attractive for reconstruction projects.

EFFI-DECK System : A new bridge deck system is under development by The Fort Miller Co., Inc. The system consists of a 5 in. (127 mm) reinforced concrete slab supported on steel tube sections, as shown in Fig. A.43. In order to maintain composite action between the concrete slab and the supporting tube beams, 3/4 in. (19 mm) diameter studs are welded to the top surface of the tube beams. The deck system covers the entire width of the bridge and is provided in 10 ft (3050 mm) wide sections. The composite action between the deck and the supporting girders is maintained through blockouts in the deck. To connect the precast panels in the transverse direction, blockouts at the girder lines are provided.

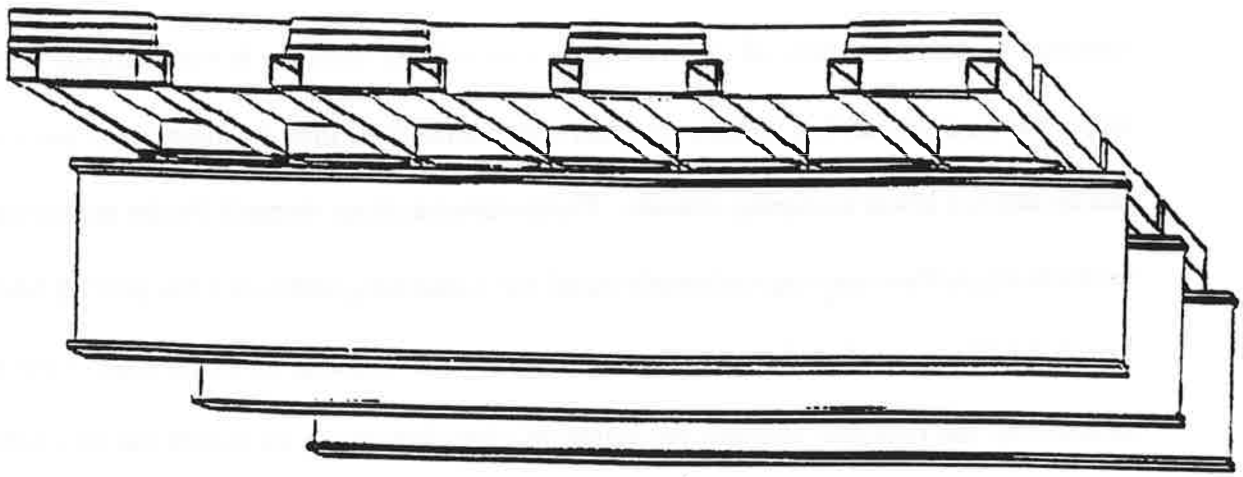


Fig. A.43 EFFI-DECK System

Concrete-Girder-to-Deck Connection

Precracked vs. Uncracked Interfaces: Several parameters can affect the horizontal shear transfer strength across the interface, such as the concrete strength and the strength, size, and spacing of reinforcing bars crossing the shear interface.

A pre-existing crack along the shear plane can reduce the ultimate shear transfer and increase the horizontal slip. Hofbeck et al (33) studied the effect of this parameter on both precracked and uncracked sections. Hofbeck et al (33) concluded in their study that for 4000 psi (27.6 MPa) normal weight concrete with clamping stresses between 200 psi (1.4 MPa) and 1000 psi (6.9 MPa), the reduction in shear strength was a constant 250 psi

(1.7 MPa). For lower values of clamping stress the reduction was higher. However, for clamping stresses above 1000 psi (6.9 MPa), the strength of the initially uncracked interface specimens increased at a very slow rate with an increase in the clamping stress, while the strength of the initially cracked specimens continued to increase at the same rate as that for lower clamping stresses. The horizontal shear strength for the cracked and uncracked interface was approximately equal for a clamping stress of 1340 psi (9.2 MPa). Fig. A.44 illustrates the above. Specimens with a pre-existing interface crack were not affected by the concrete strength for values of clamping stress up to 600 psi (4.1 MPa). For higher values of clamping stress, concrete strength influenced the horizontal shear strength. The shear transfer strength was also a function of the clamping stress. When A432 grade reinforcement was used, changes in strength, size, and spacing of the reinforcement did not affect the horizontal shear strength for the same clamping stresses.

The direct tension stresses parallel to the shear plane reduced the shear transfer strength for an initially uncracked interface, but did not reduce the shear strength of specimens with an initially cracked interface, Mattock et al. (36). It is also an adequate assumption to include any externally applied compressive stresses on the shear interface to the clamping stress for calculating the ultimate transfer shear strength of both initially cracked and uncracked specimens. The shear transfer for a pre-cracked interface, with a moderate amount of reinforcement, was developed primarily by the friction resistance of the two interfaces sliding and the dowel action of the reinforcement crossing the interface. When large amounts of reinforcement, or sufficient externally applied compressive stresses normal to the shear plane were provided, the crack in the shear plane "locked up" and shear strength was developed as in initially uncracked specimens.

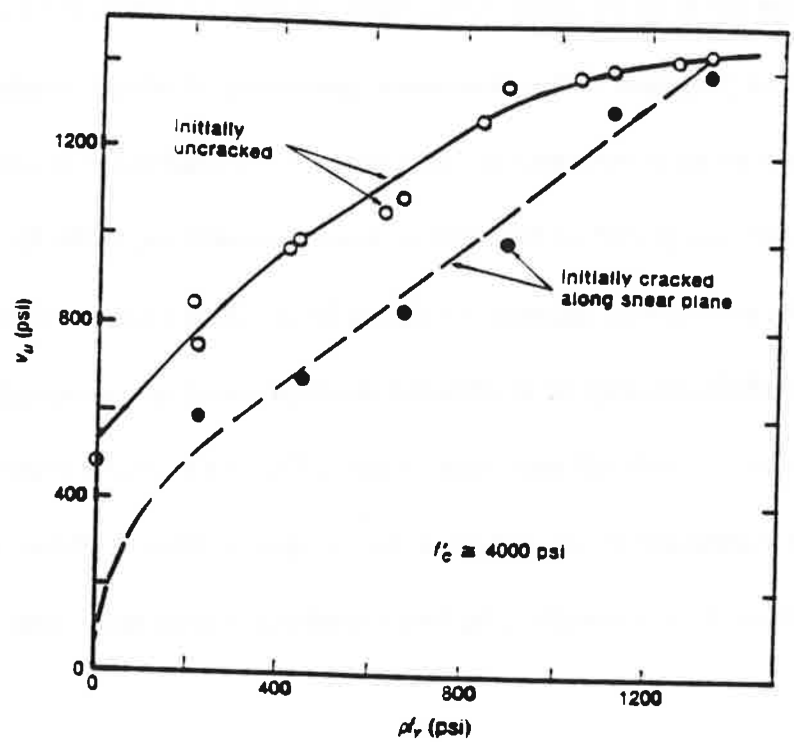


Fig. A.44 Hofbeck, Ibrahim, and Mattock Equation adopted from Hofbeck et al. (1969)

The effect of dowel action of reinforcement crossing the interface on shear resistance for both uncracked and precracked specimens was studied by Hofbeck et al. (33) and Mattock et al. (36). Dowel action of reinforcing bars crossing the shear plane was insignificant for initially uncracked specimens; however, it was found to be substantial for specimens with a pre-existing crack along the interface. The shear-friction theory was found to be adequate in estimating the shear strength for the case of precracked specimens with a friction coefficient of $\mu = 1.4$. For the cases of an uncracked interface, this theory was found to be conservative.

Push-off Tests vs. Beam Tests: Push-off specimens have been used in the majority of the testing programs rather than beam specimens. Push-off specimens were found to be relatively easier to construct and less expensive comparing to beam tests. For push-off tests, no limit was placed on the relative slip at the interface, while for beams, limiting the slip was the criterion for defining the failure load. Shear transfer behavior of the interface in a composite beam may be in between the behavior of an uncracked push-off specimen and a precracked push-off specimen. Push-off tests can give a reasonable indication of the actual composite beam behavior for horizontal shear. However, interface shear strength should be governed by slip limits established from beam tests.

Horizontal Shear Strength Design equations: Prior to 1966 the shear strength of the interface was assumed to vary directly with the amount of crossing reinforcement. The first parabolic function for the horizontal shear strength at the interface was introduced by Birkeland and Birkeland (28) as follows:

$$v_n = 33.5\sqrt{\rho_v f_y} \quad \text{(psi)} \quad \text{(A.1)}$$

$$v_n = 2.78\sqrt{\rho_v f_y} \quad \text{(MPa)}$$

Mattock (36) proposed equation (A.2) which found comparing to Birkeland et al. (28) to be equally applicable to the general case of both shear and direct stress acting across a shear plane.

$$v_n = 400 + 0.80\,\rho_v\,f_y \quad \leq 0.3 f_c' \text{ (psi)} \qquad \text{(A.2)}$$

This equation was modified by Mattock (55) to include the effect of concrete strength (A.3):

$$v_n = 4.5 f_c'^{\,0.545} + 0.8\,(\rho_v\,f_y + \sigma_n) \quad \text{(psi)} \qquad \text{(A.3)}$$

$$v_n = 0.467 f_c'^{\,0.545} + 0.8\,(\rho_v\,f_y + \sigma_n) \quad \text{(MPa)}$$

$$v_n \leq 0.3 f_c'.$$

Equation (A.3) was found by Walraven et al. (56) to be adequate for low and intermediate concrete strengths. For concrete strength higher than 5,000 psi (34.5 MPa) the equation was found conservative.

A revision to the shear-friction provisions of the PCI Manual on Design of Connections for Precast Prestressed Concrete was introduced by Shaikh (57). The proposed revisions were a result of an extensive review of the available research. The revisions were considered to be more realistic and would generally lead to a more economical design. The suggested equations (A.4) and (A.4a) are as follows:

$$A_{vf} = \frac{V_u}{\phi\,f_y\,\mu_e} \qquad \text{(A.4)}$$

$$v_n = \rho_v\,f_y\,\mu_e$$

Where μ_e is the effective coefficient of friction calculated as:

$$\mu_e = \frac{1000\lambda^2\mu}{v_u} \quad \text{(psi)} \qquad \text{(A.4a)}$$

$$\mu_e = \frac{6.9\lambda^2\mu}{v_u} \quad \text{(MPa)}$$

The coefficient, λ, is a constant used for the effect of concrete density given as:

$\lambda = 1.0$ for normal weight concrete

$\lambda = 0.85$ for sand lightweight concrete

$\lambda = 0.75$ for all lightweight concrete.

The recommended shear friction coefficients, μ, are given as follows:

$\mu = 1.4$ for concrete to concrete, cast monolithically.

$\mu = 1.0$ for concrete to hardened concrete, $^1/_4$ in. (6.4 mm) roughness.

$\mu = 0.4$ for concrete to concrete, smooth interface.

$\mu = 0.6$ for concrete to steel.

Equations (A.4) and (A.4a) were combined into equation (A.5) by Loov et al. (34) to produce a parabolic equation for v_u with respect to the clamping force as:

$$v_u = \lambda\sqrt{1000\phi\rho_v f_y} \leq 0.25 f_c' \lambda^2 \text{ and } 1000\ \lambda^2 \quad \text{(psi)} \qquad \text{(A.5)}$$

$$v_u = \lambda\sqrt{6.9\phi\rho_v f_y} \leq 0.25 f_c' \lambda^2 \text{ and } 6.9\lambda^2 \quad \text{(MPa)}$$

Shaikh (1978) also suggested a minimum area of shear friction reinforcement, A_{vf}, crossing the interface equivalent to a clamping stress, $\rho_v f_y$, of 120 psi (0.83 MPa).

For precracked shear interface, Walraven et al. (57) suggested the following equation:

$$v_n = C_3 (0.007 \rho_v \ f_y)^{C_4} \quad \text{(psi)} \quad \text{(A.6)}$$

The coefficients, C, are function of the concrete strength for 6 in. (150 mm) cubes. If f_c' is assumed equal to 0.85 of the cube concrete strength, as suggested by Loov (34), C, can be determined by (A.7):

$$C_1 = 0.878 f_c'^{\ 0.406} \quad \text{and} \quad C_2 = 0.167 f_c'^{\ 0.303} \quad \text{(A.7)}$$

$$C_3 = 16.8 f_c'^{\ 0.406} \quad \text{and} \quad C_4 = 0.0371 f_c'^{\ 0.303}$$

Hsu and Mau (58) introduce a simple equation which is adequate for both initially cracked and uncracked shear interfaces, as follows:

$$\frac{v_u}{f_c'} = 0.66\sqrt{\omega} < 0.3 \quad \text{(A.8)}$$

Where $\omega = \rho_v f_y / f_c'$.

In order to be consistent with other equations, this equation can be rewritten as (A.9):

$$v_u = 0.66\sqrt{\rho_v \ f_y \ f_c'} < 0.3 \ f_c' \quad \text{(A.9)}$$

An extensive study on horizontal shear strength of composite concrete beams with a roughened interface was conducted by Loov and Patniak (34). Based on the test results, combined with other published results of similar tests, a parabolic equation was introduced to express the transfer shear strength of the interface. Equation (A.10) by Loov et al. (34) was considered a modification of the one used in the PCI Hand Book.

$$v_u = 0.5\sqrt{(15+\rho_v f_y) f_c'} < 0.25 f_c' \quad \text{(psi)} \quad \text{(A.10)}$$

$$v_u = 0.5\sqrt{(0.1+\rho_v f_y) f_c'} < 0.25 f_c' \quad \text{(MPa)}$$

Fig. A.45 compares equation (A.10) with the test results.

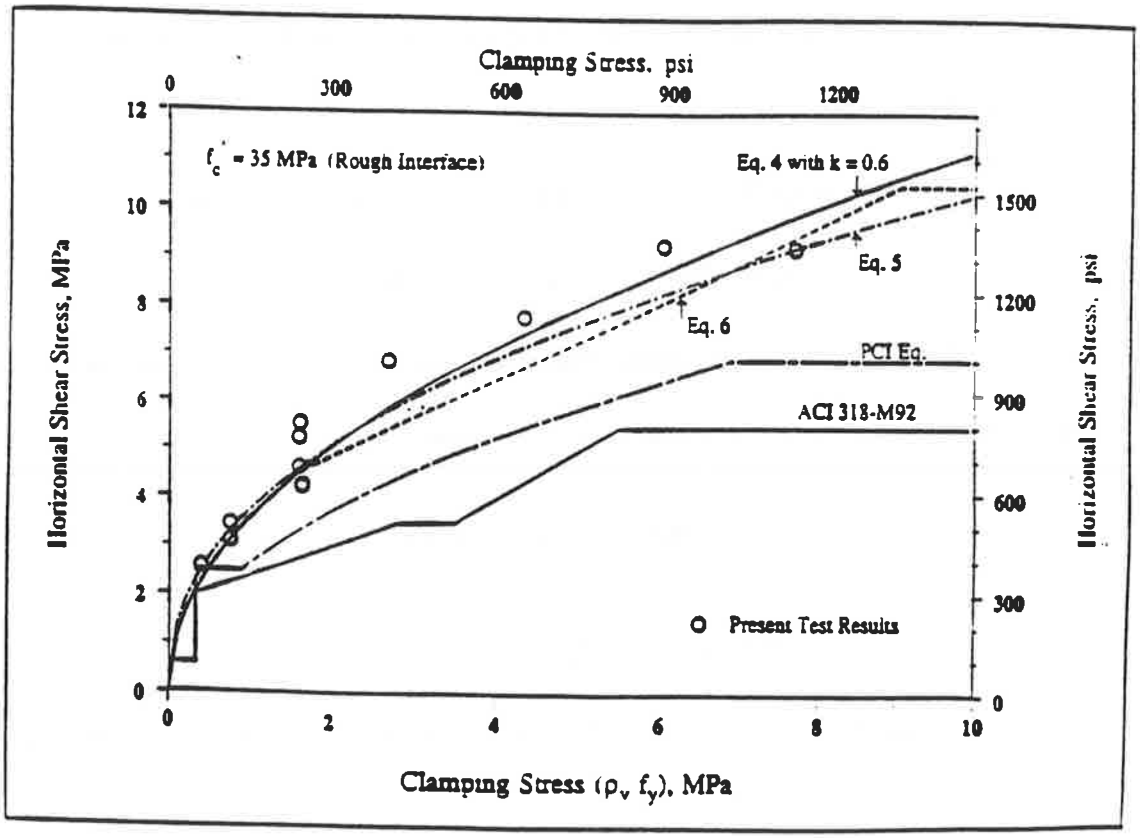


Fig. A.45 Loov and Patniaks Equation adopted from Loov and Patniak (1994)

Steel Girder-to-Deck Connection

Pushout Tests vs Beam Tests: Pushout tests of steel studs were used by most researchers rather than beam tests to determine the behavior and load carrying capacity of stud shear connectors. Pushout tests were used by Viest (39), Slutter and Driscall (41), Slutter and Fisher (42), Naithani et al. (43), and Deric Oehlers (59) in their experiments.

The pushout tests were used to evaluate the properties of the connectors themselves, because the load per connector cannot be measured directly in a beam test. Also, the pushout specimens were considered inexpensive compared to the beam tests.

The results of pushout specimens can be taken as conservative approximations of ultimate strength of shear connectors in beam, the differences that affect the test results as indicated by Slutter and Driscall (41) are the following: (i) No direct stress can exist in the pushout specimens as it is the case in the concrete slab of a beam; (ii) Slab peel-away from the steel section at the top of the pushout specimen is due to the stress distribution. (iii) Eccentricity of loading can occur in pushout specimens either because of slight errors in fabrication or nonuniformity of the concrete which often results in low values of the average ultimate strength per connector; (iv) The amount of reinforcement in the pushout specimens should be greater than the beam specimen to attain comparable values of the ultimate strength of connectors.

Configuration of the Pushout Specimens: In some of pushout specimens, the shear connectors experienced moment due to eccentricity in the test specimen, as shown in Fig. A.46. Slutter and Driscall (41) indicated that four of their specimens failed during testing due to eccentricity of loading and insufficient transverse reinforcement of the topping. Therefore; Viest (39) and Naithani et al. (43) developed new pushout specimens in which the shear transfer at the steel and concrete interface does not experience any applied moments, as shown in Figs. A.47 and A.48.

Schemes of Loading for Ultimate Strength Test: Applying the load continuously until shear failure occurs is the most common scheme of loading. In another scheme of loading used by Viest (39) the load was applied to a certain level, removed, and the residual slip recorded. This process was repeated until the specimen failed, thus determining the residual slip of the composite connection. Similarly, Slutter and Driscall (41), their loading procedure during testing was to load the beam to working load in several increments and to unload to zero. Working load was arbitrarily taken as half of the predicted ultimate load. The dead load was cycled ten times between working load and zero, then the member was loaded to ultimate load.

Splitting Induced by Shear Connectors in Composite Beams: The concentrated load which the connector applies to the slab induces three modes of cracking of the slab. The ripping action of the concentrated load on the slab causes lateral cracks extending from the sides of the slab. Shear cracks occur near the compressive zone which may affect the triaxial compression-bearing zone, which has an effect on the dowel strength. The dispersal of the concentrated load to regions of uniform longitudinal stress induces a region of large lateral tensile stresses in front of the triaxial compression zone, which has a very high effect on splitting the concrete in front of the connector.

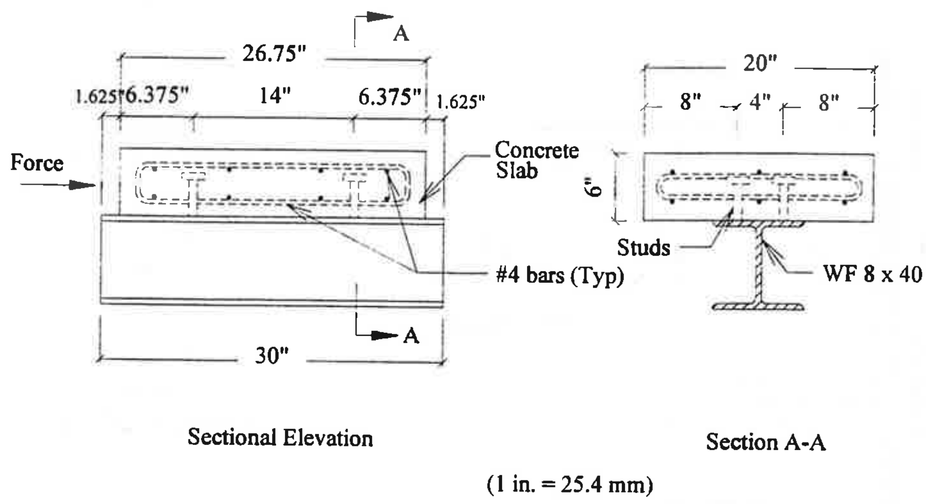


Fig. A.46 Slutter and Fisher Pushout Specimens Reproduced from Slutter and Fisher (1966)

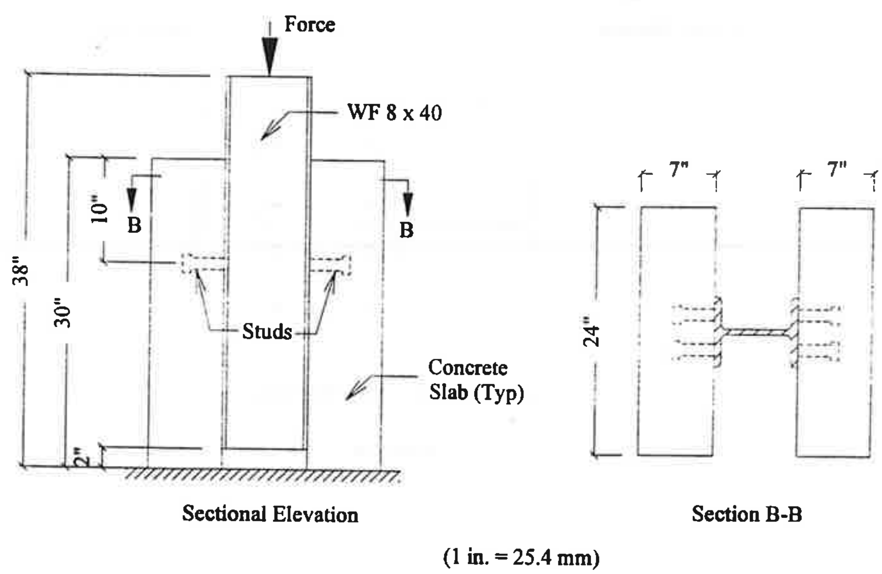


Fig. A.47 Viest Pushout Specimens Reproduced from Viest (1956)

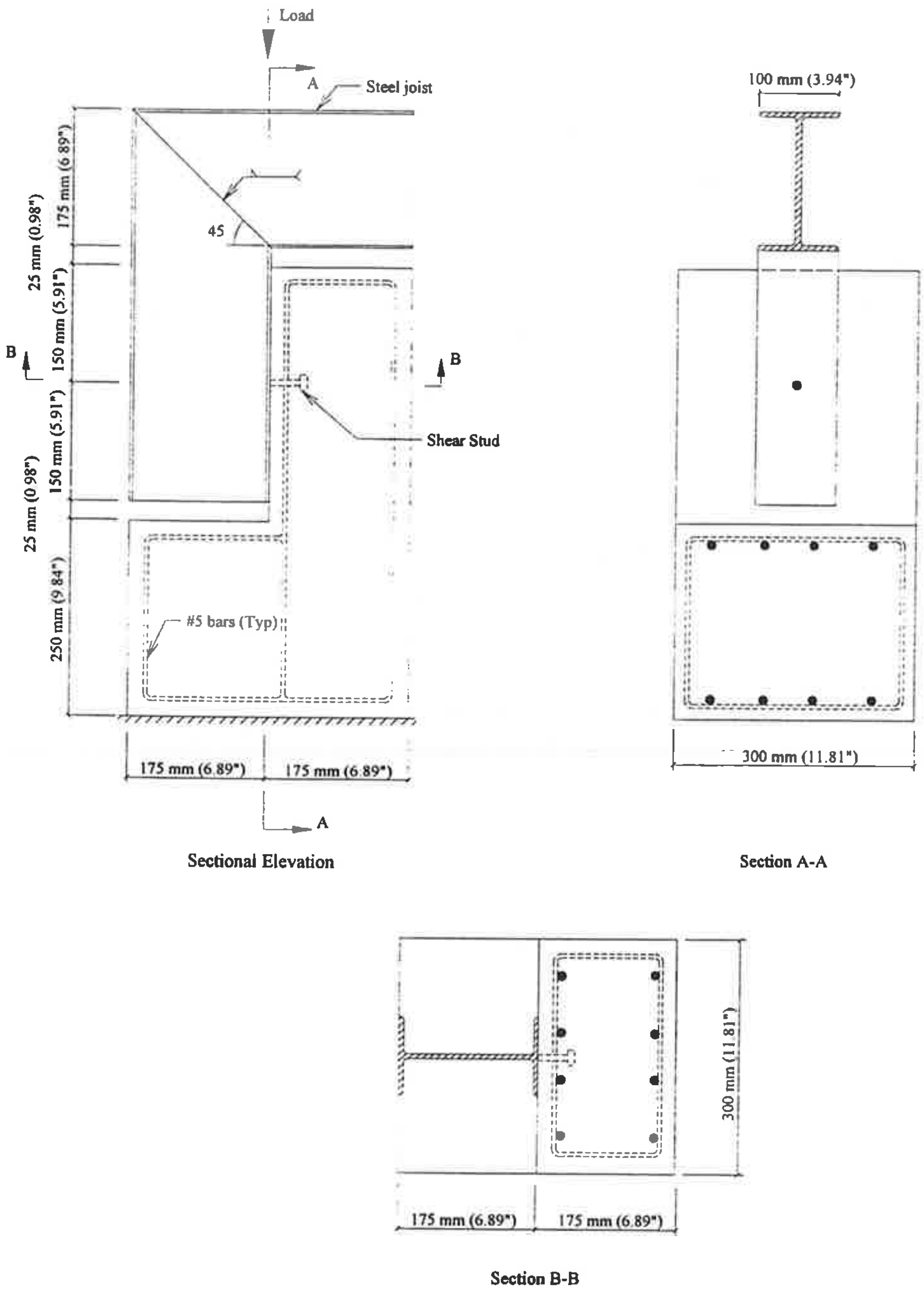


Fig. A.48 Naithani, Gupta and Gadh Pushout Specimens Reproduced from Naithani, Gupta and Gadh (1988)

The propagation of this split then induces splitting behind the connector, and also relieves the triaxial restraint to the bearing zone, hence inducing dowel failure through compressive failure of the concrete. Fig. A.49 shows the tensile cracking induced by the concentrated load.

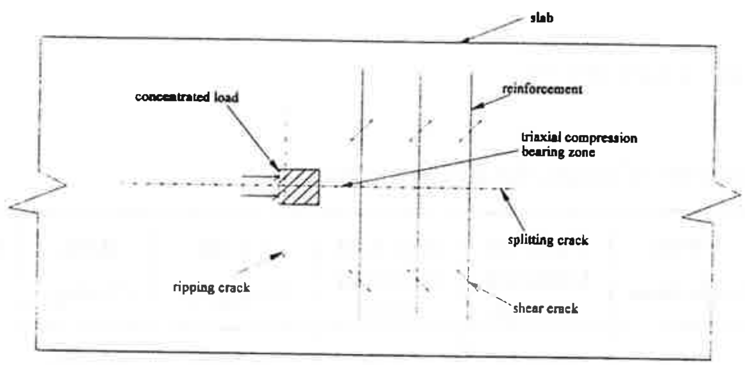


Fig. A.49 Tensile Cracking Induced by Concentrated Load Reported from Oehlers (1989)

Oehlers (59) investigated the influence of the transverse reinforcement on the post-splitting strength and stiffness. He found that splitting of the slab caused a reduction in load which was more gradual in the reinforced specimens than in those of the unreinforced specimens. This gradual reduction would suggest that there is some ability to redistribute the shear load in reinforced slabs after splitting.

Oehlers (59) experimental program showed that straight, fully anchored, transverse reinforcement placed in front of heavily loaded single connectors did not increase the splitting strength after splitting. However, the transverse reinforcement was found to limit the length of the split and allow a gradual reduction in the shear load after splitting.

APPENDIX B

ANALYSIS OF SURVEY RESULTS

B.1. Criteria to determine decks to be replaced.

Question (Owners): What criteria do you use to determine which decks are replaced?

Response Rate: 41/49 (83.7%)

(% of Respondents who use the listed methods)

Criteria	Visual Inspection	Chloride Contents Analysis	Half Cell Potential Test	Chain Dragging	Deck Coring	Sounding	Other*
Rate (%)	73%	24.4%	17%	24.4%	19.5%	9.8%	

* Radar inspection and Impact echo, Thermography, Electrical potential, etc.

B.2. Average age of decks to be replaced.

Question (Owners): What is the average age of bridge decks that require either partial or full replacement in your area?

Response Rate: 40/49 (81.6%)

Average Age	Decks on Concrete Girders		Decks on Steel Girders	
	Partial Removal	Full Removal	Partial Removal	Full Removal
11-15 years	2.4%	0%	4.8%	0%
16-20 years	29%	4.8%	8.3%	2.4%
21-25 years	24%	10%	37%	8.3%
> 26 years	27%	68%	19.5%	73%

B.3. Deck removal methods.

Question (Owners and Contractors): Which methods are used for removing concrete bridge decks?

Response Rate: Owners 40/49 (81.6%); Contractors 16/18 (88.8%)

(% of Respondents who answered affirmatively)

Removal Methods	**Respondents**	**Concrete Girders**		**Steel Girders**	
		Partial Removal	**Full Removal**	**Partial Removal**	**Full Removal**
Boom-Mounted Breaker	Owners	14%	42%	14%	58%
	Contractors	47%	53%	47%	80%
Hydraulic Jaws	Owners	2%	16%	2%	16%
	Contractors	20%	27%	33%	47%
Sawing	Owners	23%	42%	28%	49%
	Contractors	33%	53%	53%	73%
Hand-Held Hammer	Owners	44%	21%	49%	19%
	Contractors	67%	47%	80%	53%
Water Jet	Owners	26%	12%	28%	7%
	Contractors	27%	6%	27%	20%
Pressure Bursting	Owners	7%	0%	7%	0%
	Contractors	0%	7%	13%	7%
Crane & Ball	Owners	0%	9%	0%	9%
	Contractors	7%	20%	7%	20%
Blasting	Owners	0%	2%	0%	2%
	Contractors	0%	0%	0%	0%

Other Methods (1) Mechanical Hammer (2) Plaining Machine (3) Shot Blasting (4) Roto-Mill (5) Whip Hammer		(1) Full Removal (2) Partial Removal (3) Partial Removal (4) Partial Removal (5) Full Removal	(1) Full Removal (2) Partial Removal (3) Partial Removal (4) Partial Removal (5) Full Removal

B.4. Problems in the use of removal methods.

Question (Owners and Contractors): What problems are you having with these methods and what can be done to improve them?

Response Rate: Owners 23/49 (47%); Contractors 12/18 (67%)

Removal Methods	Problems
Boom-Mounted Breaker	1. Extensive vibration and excessive noise. 2. Damage to the top flange of steel girders (cracking and micro-cracking). 3 Expensive to clean up. 4. Damage to vehicle and worker.
Hydraulic Jaws	1 Over breakage and damage to flange of girders. 2. Slower than other methods.
Sawing	1. Cutting into top flange of steel girder. 2. Cutting off reinforcement bars in concrete girders. 3. Need to control slurry.
Hand-Held Hammer	1. Very expensive and slow.

Water Jet	1. Very expensive and difficult to control for partial removal of deck. 2. Need to control waste water and environmental considerations.
Pressure Bursting	1. Expensive and slow.

B.5. Techniques used in replacing existing bridge decks.

Question (Owners): Rapid deck replacement depends on methods acceptable to a variety of owners dealing with a variety of conditions throughout the United States. Do you use any of the following techniques in replacing EXISTING decks in your areas?

Questions (Designers): In designing a NEW bridge deck system, do you consider any of the following items in order to enhance future rapid deck replacement?

Response Rate: Owners 41/48 (85%); Designers 20/28 (71.4%)

(% of Respondents who answered affirmatively)

Methods	Owner's Experience with the Methods	Designer's Willingness to use the Methods	Descriptions
PERMANENT SIP FORMS			
Precast Prestressed Conc. Panel	25.6%	27%	Size: (1) 8' wide (nominal) Minimum 2'-0", (2) 4' to 11'-6" length, (3) 3.5" to 4.25" thickness.
Precast Reinforced Conc. Panel	5%	23%	
Corrugated Metal SIP Deck	35%	45%	
MODULAR DECK SYSTEM			
Precast Prestressed Conc. Panel	12%	5%	Modular deck system was identified as the first priority element which needs to be studied to develop an optimized system for rapid deck replacement.
Precast Reinforced Conc. Panel	5%	0	
Exodermic System	7%	0	
Pretensioning or Post tensioning in Deck	11.7%	18.2%	

DECK PROTECTION METHODS			
Polymer Coating	21%	41%	Other types of protection methods: (1) Superplasticizers to create a dense concrete mix and resist chloride penetration. (2) Silica Fume Overlays.
Water Barrier Layer	32.5%	27%	
Asphalt	23%	27%	
Cathodic Protection	19%	23%	
Linseed Oil	14%	32%	
Epoxy Coated Re-bar	12%		
Micro-silica Concrete	9%	32%	
Latex Modified Concrete	12%		
SHEAR CONNECTORS			
Rebar, Dowel, or Metal Studs	72%	82%	
Bolt in Drilled Holes	2%	9%	
Chemical Adhesive	7%	0	

B.6. Influencing factors.

Question (owners and designers): In determining the efficiency of **EXISTING** bridge deck replacement systems, please indicate your view of the importance of the following factors. (1 being the least important, 10 the most important)

Response Rate: Owners 39/49 (79.6%); Designers 25/26 (96%)
(% of Respondents' view of importance of each factors)

Items	Owner % (Rank)	Designer % (Rank)	Average % (Rank)
1. Girder Material	50(15)	58(14)	54(15)
2. Deck Material	63(6)	66(8)	64.5(5)
3. Composite & Non-composite Design	49(17)	64(10)	56.5(13)
4. Method of Removal & Installation	61(9)	67(5)	64(8)
5. Equipment and Level of Skill Required	51(12)	71(4)	61(9)
6. Environmental Requirements	50(14)	51(18)	50.5(18)
7. Traffic Control During Construction	73(2)	64(10)	68.5(2)
8. Safety Requirements	65(4)	72(3)	68.5(2)
9. Contractors Availability and Experience	51(12)	66(7)	58.5(12)
10. Volume of Traffic and Importance of Crossing	64(5)	65(9)	64.5(5)

11. Sources of Deterioration	50(15)	57(15)	53.5(16)
12. Relative Initial Cost	63(6)	59(13)	61(9)
13. Life Cycle Cost	62(8)	67(6)	64.5(5)
14. Contractor's Incentives to Speed Up	48(18)	54(17)	51(17)
15. Night Construction	39(19)	50(19)	44.5(19)
16. Structural Performance	74(1)	80(1)	77(1)
17. Cost of Bridge Partial/Full Closure	58(10)	63(12)	60.5(11)
18. Possible Future Replacement	56(11)	55(16)	55.5(14)
19. Innovative Features	35(20)	47(20)	41(20)
20. Measures to Protect New Deck	68(3)	72(2)	65(4)

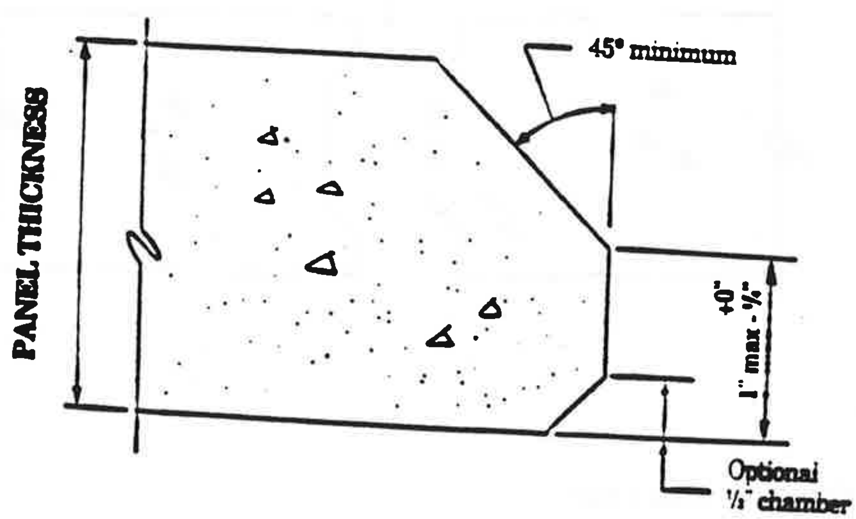
B.7. Types of transverse joints and their performances.

Question (owners): What types of transverse joints do you use in connecting precast prestressed concrete panels and what are their performance?

Response Rate: 16/49 (32.6%)

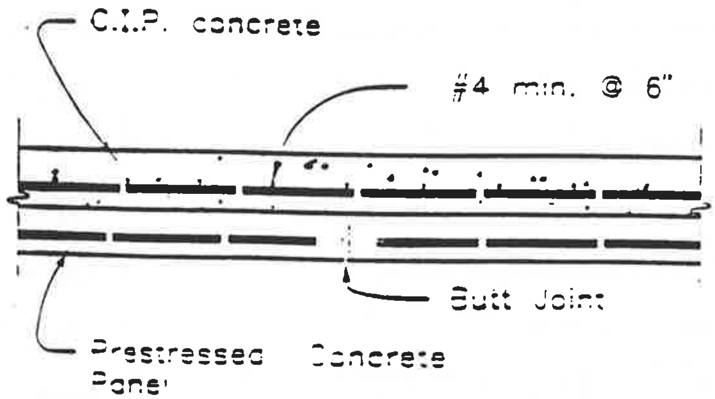
<u>Detail 1</u>

Source: Georgia DOT



Detail 2

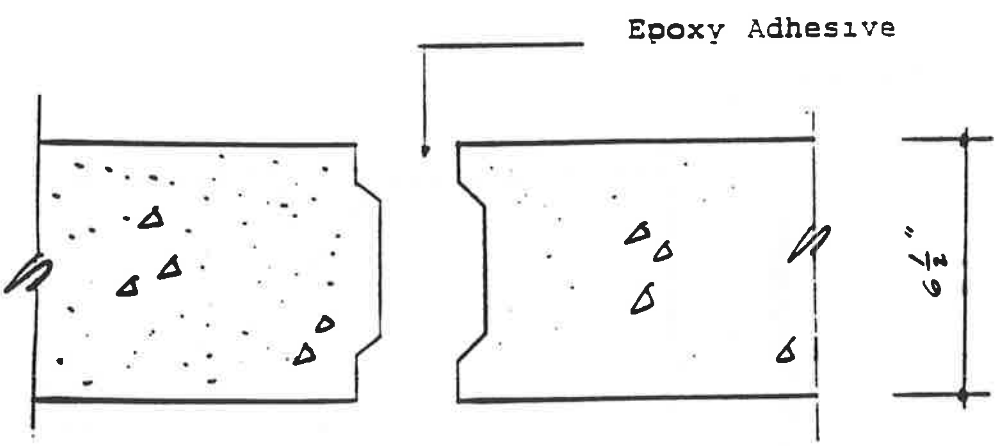
Source: Colorado DOT



Colorado DOT utilizes a butt joint. In the above detail, cracks reflect through the partial cast-in-place deck. Colorado DOT has increased the minimum top longitudinal reinforcing steel to #4 bars @6" o.c. from #5 bars @18" o.c. to control cracks.

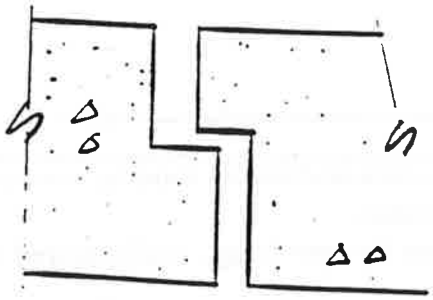
Detail 3

Source: Illinois DOT

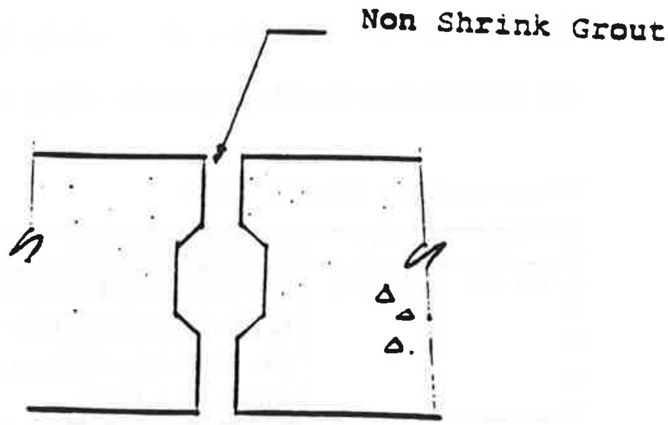


Detail 4

Source: New York State DOT



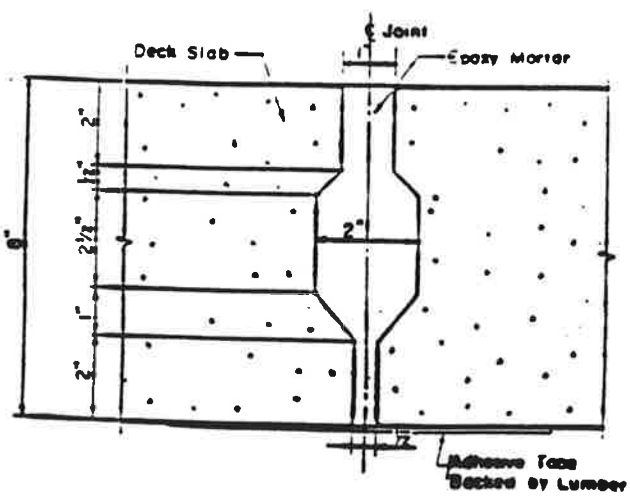
Lap Joint
(The joint was ineffective and is not recommended for future projects)



(The joint has performed satisfactorily)

Detail 5

(Source: Mrinmay Biswas. Journal of Transportation Engr., Vol. 112, No. 1, ASCE, 1986,)



B.8. Willingness to consider future rapid deck replacement.

Question (owners): Would you be willing to spend an additional amount on NEW bridge construction that would enhance the ability to use rapid deck replacement in the future?

Response Rate: 38/49 (77.5%)

Yes	No	Specific Comments
44.7%	10.5%	(1) Contingent on approval by state and federal funding sources. (2) It depends on cost-effectiveness. (3) Moderate additional moneys for new design when assured of future pay back.

B.9. Modification of the codes and specifications.

Question (owners): How should the codes and specifications be modified to enhance future rapid deck replacement? Please comment on your reasons.

Respondent	Specific Comments on Codes and Specification Modification
Owners (Response Rate: 16/49, 32.6%)	(1) Allow wider decks for future replacement. (2) A method for staging deck replacement should be detailed on the plans for new bridge structures. (3) If deck replacement is to be considered a viable option, the codes will need to be modified to require a longer service life for the substructures. (4) It should address damage to the top flange of prestressed girders. (5) Specify that all non-conventional designs, such as cable stayed and post-tensioned segmental, include in the provisions of the original plans details as to how decks must be removed and replaced. (6) Develop suggestions that make rapid deck replacement possible and easier. (7) Transverse and longitudinal joints of main concern. Most designers do not take casting tolerances into account when detailing precast.
Contractors (Response Rate: 5/18, 27.8%)	(1) Eliminate maintenance of traffic. (2) When building new structures, it would be helpful to widen abutments and piers to allow the future placement of additional girders for future widening. (3) Concrete curing time requirements are often the longest single item in a deck replacement schedule. Changes in this area would help significantly.
Designers (Response Rate: 11/26, 42.3%)	(1) Allowance for the widening of new decks for future replacement. (2) Provide more guidelines and a wide range of acceptable replacement methods, based on field experience and research. (3) AASHTO subsection 9.20. 4.5.c: Ties for Horizontal Shear ("All beam shear reinforcement shall extend into cast-in-place deck slabs. Extended shear reinforcement may be used in satisfying the minimum tie reinforcement"). (4) Consider less restrictive studs and stirrups.

B.10. Recommendations for Priority of developing rapid deck replacement system.

Question (Designers): In order to develop an optimized system for rapid deck replacement, further research of the following techniques may be necessary. Please give your order of preference and comments on reasons (1 being top choice)

Response Rate: 25/26 (96%)

Research Items	**Shear Connector**	**Modular deck system**	**Pretensioning or post tensioning in deck**	**Deck surface protection method**	**Other***
Total Points	62	45	65	60	
Ranking of Preference	3	1	4	2	

* Rideability: Rideability is the performance condition that needs to be measured to determine if the system is successful.

B.11. Recommendations for suggested details for deck replacement.

Question (designers and contractors): Fig. B.1 and B.2 are girder-deck joints suitable for the replacement of EXISTING bridge decks. Please comment on any possible improvements or problems anticipated.

Question (designers and contractors): Fig. B.3 and B.4 are girder-deck joints suitable for the NEW bridge deck design, which may facilitate future rapid deck replacement. Please comment on any possible improvement or suggestions.

Response Rate: designer 15/26 (57.7%); contractor 14/18 (77.7%)

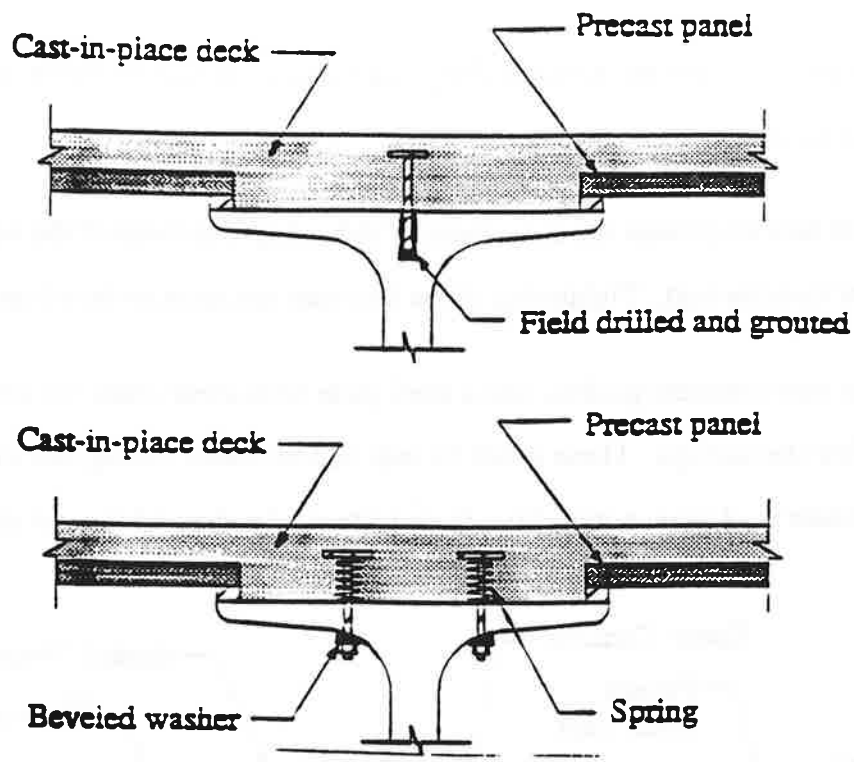


Fig. B.1. Suggested Details in Survey for Concrete Girder-Deck Joint for the Replacement of Existing Deck.

Suggestions for improvements.

(1) Replace spring with washer welded to stud and eliminate post tensioning.

(2) Drilling would result in considerable spalling of the bottom of top flange area. It would be very time consuming and expensive to field drill flanges. Both designs in Figure A.1 may result in hitting existing reinforcing steel when attempting to field drill holes.

(3) First design would perhaps be better than second design in the replacement of existing bridge deck.

(4) The second design looks good for new beam fabrication.

(5) Consider a bond breaker on bolt with hand tightened nut on top of flange. Taper the head of stud, at bottom, to minimize air entrapment under the head.

(6) Prepare a good bond between the precast panel and CIP concrete (roughening surface of panel or some type of shear connector)

(7) Find how to prevent the movement of slab relative to beam if the hole around the bolt is larger than the bolt. Tightening of the bolt may not seem to be a dependable solution.

(8) For new concrete girders, use a steel plate with shear studs on both sides instead of extending the stirrups. These could be cast into the beam during fabrication. It would be much easier to clean concrete from these instead of under and around stirrups.

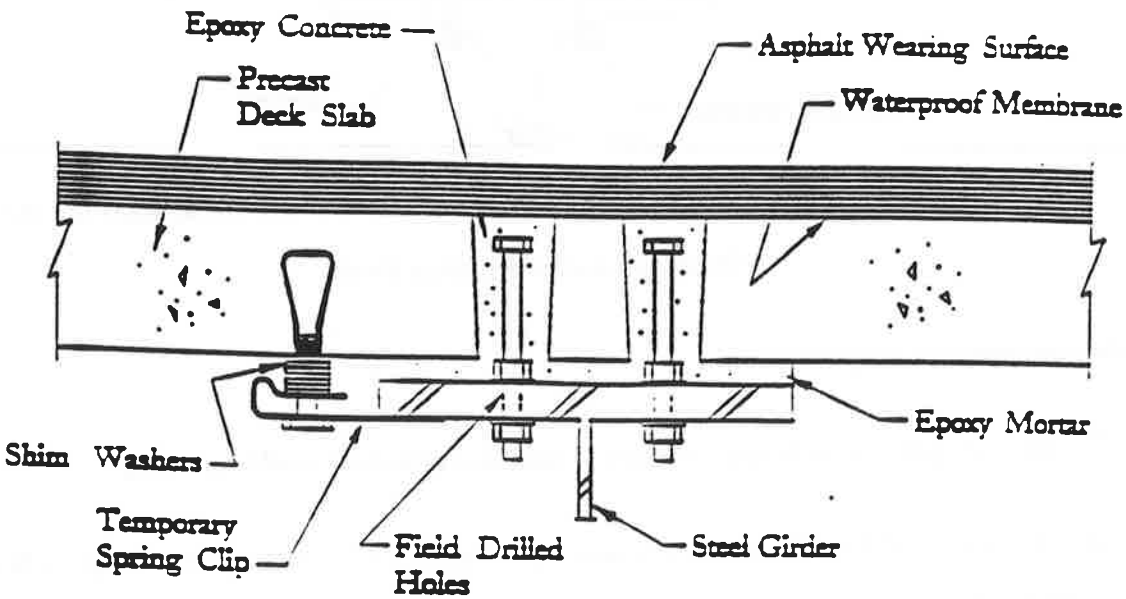


Fig. B.2. Suggested details in Survey for Steel for Steel Girder-Deck Joint for the Existing Deck.

Suggestions for improvements.

(1) Field-drilled holes are not recommended, since they could reduce the capacity of the section in negative moment areas. Need to check loss of section and stress in top flange due to the drilling of new holes.

(2) Field drilling is a more complicated operation than currently used welded steel studs. This method may be more expensive to install than welded shear studs. Also, when it comes to time to remove the nuts, the thread will be either filled with paint or rust making it very difficult to remove the nuts.

(3) After removal of the nuts, the bond between the steel girder and the epoxy mortar will prevent the deck from being lifted off in large pieces.

(4) This system appears to create a weak line in the deck, where the holes for pouring the epoxy concrete around the shear studs are. Why not pour a homogenous concrete for the entire deck, including around the studs?

(5) Water-proof membrane acts as slip coat for trucks on grade or near intersection with braking action.

(6) A thin plastic strip might be considered to retain mortar at edges of flanges and also support the precast deck slab. It will be helpful for placing and adjust the grade of panel.

(7) It may be necessary to place panels before drilling bolt holes to assure accurate location of bolt holes. In this case, access to the bottom nuts may be difficult. If panels are slightly blocked up, regular grout may be adequate. Drill tapered holes in the girder flange and make shear studs have a friction fit.

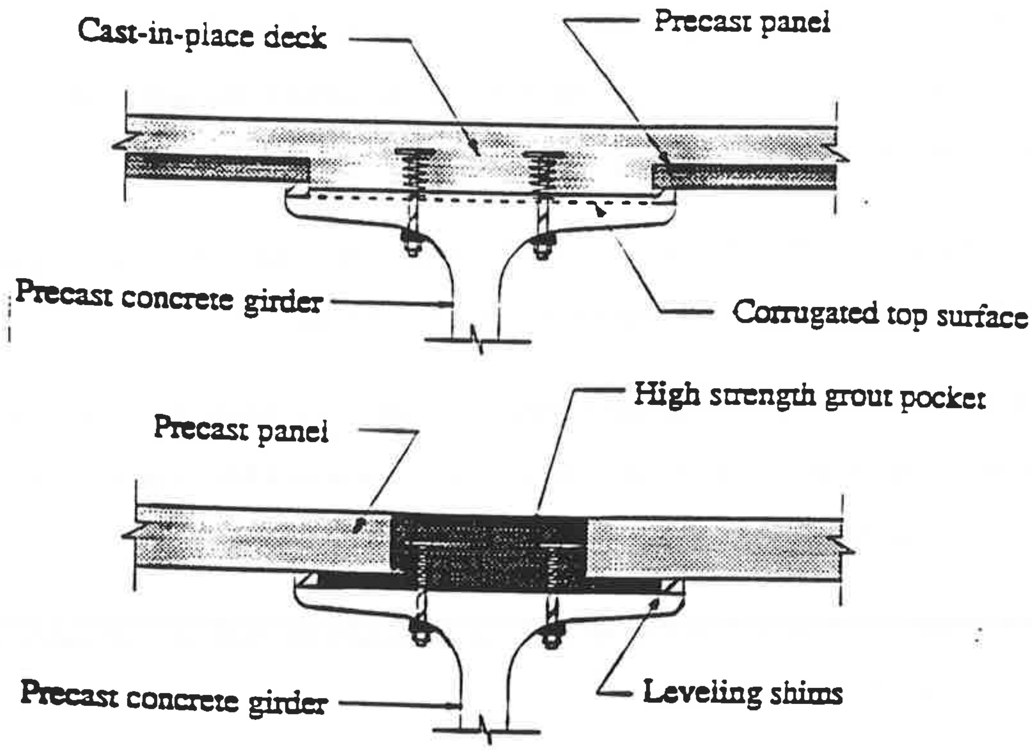


Fig. B.3. Suggested Details in survey for Concrete Girder-Deck Joint for the New Deck Design Suggestion for Improvements

(1) Consider using an elastomer for shims. Shims must account for variable height and inherent roughness of top of precast girders.

(2) Delete spring. Use nut and washer on top of girder.

(3) The pocket geometry needs to be modified to provide for a mechanical interlock between the grout and the precast panel.

(4) Use epoxy grout in the pocket. Unless non-shrink cement is used in the grout pocket, a longitudinal crack will be developed at the joint between the panel and grout.

(5) The full depth panel would need to be field ground for rideability. Some modest post tensioning may be required to keep the crack tight.

(6) This method may require code revision. Shear capacity of high strength bolts in this application is uncertain. Current codes make no allowance for vertical load along the joint. This method may be excessively conservative.

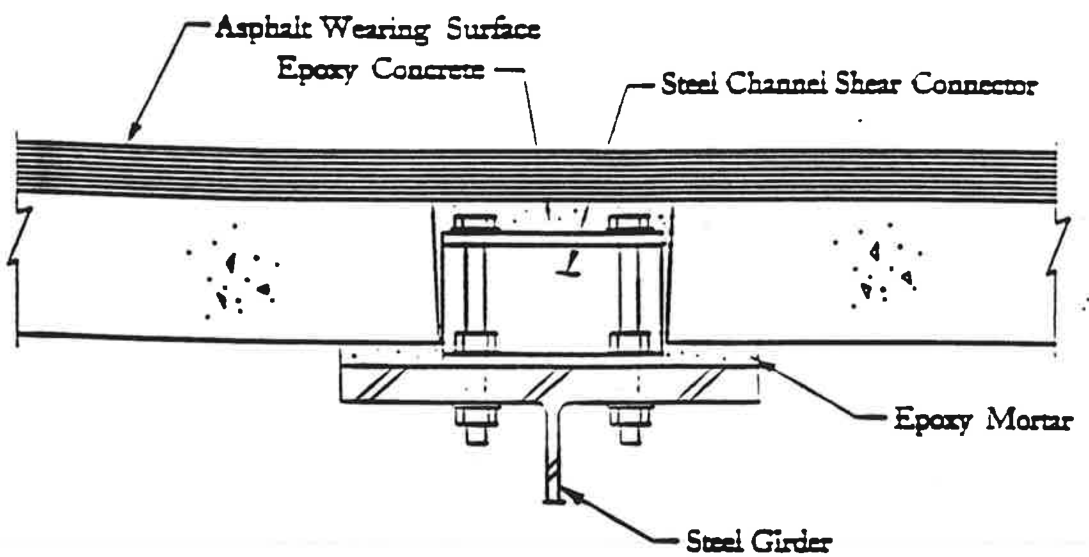


Fig. B.4. Suggested Details for Steel Girder Deck Design for New Girder Deck.

Suggestions for improvement.

(1) The channel connector interrupts the continuity of the concrete longitudinally over the girder. It does not allow rebar through. This detail appears less functional than Figure B1-2.

(2) Considerable amount of fabrication is required for the channel unit with fairly good quality control.

(3) Because of the taper, the deck would have to be broken at the shear connectors or it could not be removed without first removing the shear connectors.

(4) This is too complicated to use. Need to have contractors ensure constructability. The formed pocket has to match the holes in the fabricated girders. Will the normal tolerance accommodate this system?

APPENDIX C

SUMMARY OF INTERVIEW RESULTS WITH DOTS

C.1. Interview Performance

DOT	Date	Location	Participants
California	June 16, 1994	CALTRANS Bridge Division, Sacramento, California	CALTRANS: Tom Harrington, Jim Simon, Jeff Goronea, Mike Lee, Joe Gallippi, Thomas P. Wood HDR: Bill Dowd, Conrad Bridges
Georgia			Information lost
Illinois	June 15, 1994	Illinois Department of Transportation, Springfield, Illinois	Illinois DOT: Ralph Anderson, Iraj Kasper, Randy Jackson, Lou Hassis, Rich Hahn, Tom Domagalski, and Salah Khayyat University of Illinois at Chicago: Mohsen A. Issa University of Nebraska: Maher K. Tadros, Mantu C. Baishya
Minnesota	June 21, 1994	Minnesota Department of Transportation, Roseville, Minnesota	Minnesota DOT: R.A. Elasky, Dan Anderson, Dave Reinsch, Russ Noreen, Don Flemming HDR Eng.: Bill Dowd, Chuck Gonderinger
Missouri	May 24, 1994	Missouri Highway & Transportation Department, Jefferson City, Missouri	MHTD: Allen Laffon, Fred Martin, Alan Trampe, Tom Keith, Shyam Gupta, Roney L. Haden, Craig Lenning UNL: Maher K. Tadros, Mantu C. Baishya, John H. Ock, Darin Splittgerber
Nebraska	May 4, 1994	Mahoney Park, Nebraska	NDOR: Lyman Freemon, Marvin Lech, Ron Korinek, Mo Jamshidi, Claude Oie, Ken Gottula, O. Bumanis UNL: Maher K. Tadros, Mantu C. Baishya, John H. Ock Kiewit Co.: Jerry Thoendal HDR Engr.: Phil Rossbach, Hussein Khalil

New York	June 29, 1994	New York DOT, Bridge Division, Albany, New York	NYDOT: Dave Beal, George Christian, Tom Moon, Jim O'connell, Arun Shirole, William Winkler UNL: Maher Tadros Kiewit Co. : Sharad Mote
Texas	May 3, 1994	Texas DOT Bridge Division, Austin, Texas	Texas DOT: Ron Bailey, Charles C. Terry, Bob Cochrane, Dev Tulsiani HDR Engr.: Luis Ybanez, Bill Dowd
Virginia	June 30, 1994	Virginia DOT Bridge Division , Richmond, VA	VirginiaDOT: W.F. Dotson, Steve D. Edwards, Mark Farnsworth, Malcolm Kerly, W.D.(Bill) McDowall, Michael Sprinkel, Frank W. Waldrop UNL: Maher Tadros Kiewit Co.: Sharad Mote
Washington	June 2, 1994	Bridge and Structure Branch, Olympia, WA	Washington DOT: Myint Lwin, Ed Henley, DeWanye Wilson, Patrick Clarke HDR Engr.: Phil Rossbach, Santosh Kuruvilla

C.2. Findings from Interviews

Agenda Item #1: Definition of needs and desires of the DOT for rapid replacement of bridge decks, philosophy of deck life versus deck cost, and speed of deck replacement versus quality of product

1. California DOT

(1) Contractors are generally granted only a limited time for lane closures. These times include an 11-hour window for the state and 7 hours for the Los Angeles area.

(2) CALTRANS would like to see bridge deck replacement systems which allow bridges to act as originally designed as far as load distributions to members is concerned.

(3) There is a need to develop precast slab-bearing details such that a high quality connection is achieved with uniform support throughout length of girders.

(4) Details are needed for longitudinal construction joints which achieve a quality connection and provide for load distributions which are equivalent to that of the original design.

(5) Though it may not be a part of our research contract, CALTRANS Division of Structures feels that it may be appropriate to evaluate the long-term benefits and the short-term consequences of complete closures of sections of freeway to perform major maintenance and deck replacements within an accelerated construction period. This may be a method to get a higher quality product and lower construction costs with a limited duration of traffic impact. The downside is the significant traffic impacts during the closure period.

(6) CALTRANS has had good success with thin bonded polyester overlays. These materials are limited to thicknesses of 3" or less. They are also very expensive. It would be advisable to identify high strength, rapid-curing concrete, not unlike the polyester overlays, which could be used for full-depth deck replacements which are also economical.

2. Georgia DOT

Information lost

3. Illinois DOT

(1) IDOT utilizes staged construction for repairing bridge decks. They divide and repair the bridge in three portions: first, they pour the concrete in the outer two sides; after the concrete in those sides is set, they place the concrete in the center portion.

(2) IDOT does not use high strength concrete. For precast concrete, they use 6000 psi concrete; and cast-in-place concrete is generally 4000 psi.

(3) IDOT is reluctant to use high early strength concrete. Also, IDOT is interested in using precast deck modules to save construction time, but unsure about performance of joints in precast construction.

(4) IDOT considers prefabricated construction with pretensioning and post-tensioning can help speed up construction. Incentive and disincentive clauses can be introduced in the specifications to speed up the construction process. Whatever method is used in replacing bridge decks, it should be kept simple.

4. Minnesota DOT

(1) Need to minimize lane closures during peak hours.

(2) Need details for rapid replacement of existing decks on steel and concrete girders.

(3) Need systems which will work with complex geometry (curve, flared, and transition superelevations). Complex geometry generally coincides with locations of high traffic volumes.

(4) Greatest needs are in the metropolitan areas and on mainline interstates and other freeways.

(5) Would like to have accelerated construction practices for 10,000 to 15,000 sq. ft. mainline bridges.

(6) Need details for major river crossings where detours would be long.

(7) An optimum solution would be to have details for overnight bridge deck replacements. However, MnDOT seriously doubts whether this is possible.

(8) Would like to have details for even small bridges that could be done during a weekend closure period.

(9) MnDOT basically has a large inventory of existing bridges that were overlayed approximately 20 years ago. These structures are starting to show signs of deck deterioration such that they will require deck replacements in the not too distant future. This assessment is made based upon the original chloride ion problems detected in these decks in the 1960s. In 1965, decks were surveyed and few problems were detected. In 1969, a similar survey showed major problems developing in the decks. In 1973, surveys identified critical problems.

5. Missouri DOT

(1) Time delays and customer satisfaction are the key issues in a bridge deck rehabilitation job. Political, tax, and educational issues are also important to the design of a project as well as engineering issues.

(2) MHTD prefers initial investment for quality rather than repair in the future and is not willing to substitute quality for cost. In the design phase of bridge construction, pressures are present for a less-expensive product; however, there is a trend for a change in a higher quality product and the willingness to pay for it. Contractor incentives can cause a loss in construction quality. From the contractors' point of view, quality is one of the considerations but completeness is more important for incentive purposes.

(3) MHTD has performed a user cost analysis in some projects. However, it is not used on every project.

6. Nebraska DOT

(1) Quality is the top priority. NDOR would not sacrifice quality to shorten construction time.

(2) There is no specific criteria to determine decks to be replaced. It depends on case-by-case basis.

(3) NDOR usually does not consider user cost in deck replacement projects.

7. New York DOT

(1) NYDOT tries to extend the useable life of a bridge deck for as long as possible.

(2) Will not substitute decisions that will compromise quality. Effort is concentrated on increasing the life of the deck.

8. Texas DOT

(1) Texas DOT considers the use of rapid deck replacement on a case-by-case basis when there is significant pressure from the traffic perspective.

(2) If standard details exist, rapid deck replacements would be used more often and also be considered on unplanned deck replacements that are identified during the maintenance and construction phases of projects.

9. Virginia DOT

(1) A major portion of the decks are replaced due to deterioration and/or as part of an improvement project.

(2) With the options of either replacement or repairing, VDOT would prefer to replace a bridge deck if costs of repair exceed 60% of the cost of replacement.

(3) Quality suffers when traffic is placed on repaired surfaces prematurely.

10. Washington DOT

(1) Washington DOT began a proactive program in the early 1970s to identify bridges with deck deficiencies. They are currently about 80% complete with their deck overlay program. Overlays are targeted for structures with delamination ranging up to 30-40% of the deck area.

(2) They are not redecking cast-in-place boxes or other structures where the integral connection of the superstructure to the deck would require scaffolding or other supplementary support.

(3) They always look at overlaying a bridge as a first option. Overlay systems utilized include a contractor option of latex modified or micro silica overlays. The use of asphalt overlays is currently used only to extend the life of a bridge for a few more years if it is scheduled for replacement.

(4) They have no interest in sacrificing quality for speed of deck replacement unless it is driven for political reasons. Rideability of the bridge deck surface, particularly immediately following the repair work, must be good quality.

Agenda Item #2: Description of current practices employed by DOT in the replacement of bridge deck replacement with high traffic volume and problems experienced

1. California DOT

(1) CALTRANS currently uses a multiple-tiered approach to deck rehabilitation:

a) Decks with cracks and up to 20% of the deck area spalled with no significant chloride ion concentration problems - treat deck with methyl methacrylate crack filler, cure for four hours and reopen to traffic.

b) Decks that are basically structurally sound with more extensive spalling and/or high chloride concentrations in the upper portions of the deck - remove surface concrete up to 1" in depth and overlay with a polyester overlay having a 3/4" minimum, 3” maximum thickness: cure for four hours and reopen to traffic (dry conditions are required for this procedure).

c) For decks which are not structurally sound and those with high chloride concentrations throughout the depth - replace the deck with a "deck-on-deck" reconstruction. This scheme involves leaving the existing deck in place, placing new shear connectors from the girder up above the existing deck surface and casting a new structural deck on top of the existing deck. This method may require girder strengthening and lane closures of several days.

d) A last resort is to remove and replace existing deck. This system is used only when the deck-on-deck scheme will not work. Multiple day lane closures are required for this scheme.

(2) CALTRANS has employed a rapid bridge deck replacement utilizing a precast slab method. This method was used on the High Street Overhead modification project. Plans for this project were provided by CALTRANS. On this particular structure, slab sections were precast on site. Slab sections were detailed to be full length of the spans which were normally 50' in length. After reviewing the as-built plans, it was noted that the details were revised such that precast slabs were half the length of the spans. The closure pours as well as the stud pockets were placed with high alumina cement concrete. The project

was not considered successful because severe and numerous deck cracks developed; precast slabs ended up as point supported at every cast-in-place stud blockout.

(3) For the deck-on-deck systems, additional shear connectors are placed between existing studs or shear connector bars. Care is taken to expose only every other line of shear connectors during the demolition process. For steel girder options, hydro demolition is used to expose the heads of the existing shear connectors with new studs being piggy-back welded to the existing studs. The work area is patched in order to re-establish the existing shear connector prior to exposing the adjacent rows of shear connectors. On concrete structures, new shear connectors are drilled and grouted into the girders through the existing deck.

(4) CALTRANS has observed that hydro-demolition works very well, except when water is allowed to pool in a removal zone like a shear connector pocket. Standing water absorbs the energy of the water blast and makes it ineffective. Water must be freely drained away from these areas for the hydro-demolition to work properly.

(5) CALTRANS does not use latex modified concrete or high density low slump concrete overlays for their bridges. They feel that the epoxy coated rebar in the entire top and bottom layer plus the entire length of stirrups that project into the slab provides an adequate protection system for their decks. This coupled with timely maintenance of the deck should eliminate the need to replace decks in the future.

2. Georgia DOT

Information lost

3. Illinois DOT

(1) IDOT prefers to save shear studs during the demolition process. While metal deck can be used as a formwork for cast-in-place concrete decks, problems exist in inspecting the bottom of a deck. New steel forms can be effectively used as formwork; however, overloading can cause a separation between the steel formwork and cast-in-place concrete.

(2) Cracking at the panel joints has been the main problem with the use of precast panels.

(3) There is a problem with moisture leakage in the Seneca bridge. The bridge was constructed during the nighttime. IDOT feels that artificial lighting is not as good as sunlight during the daytime.

4. Minnesota DOT

(1) Traffic control requirements are approaching a policy that allows no lane closures during peak hours. On an I-94 pavement overlay project, the mainline was closed to traffic over a weekend with traffic diverted to a frontage road. It was felt that this was a good system where adequate frontage roads exist.

(2) Some bridges on I-94 east of St. Paul near the 3M plant were overlayed in a period of one week for each of the two directions. This was not a bridge deck replacement but an overlay which involved expansion joint replacements and a PC concrete overlay. Type 3 cement was used and heating blankets were used for curing. Curing time was reduced to 18 hours prior to reopening to traffic. The traffic control plan reduced traffic to one lane during construction. Both halves of an individual bridge were completely overlayed in a one-week period. MnDOT was concerned about opening to traffic this quickly, however, they have not seen any problems with the structures to date.

(3) In 1975, twin bridges on I-35E were overlayed. The concrete quality was so poor that on one structure, heavy dental work was done to remove poor concrete around the upper mat of steel. It was decided to overlay the other twin bridge, foregoing the extensive amount of removal of concrete around the top mat of steel. To date, both bridges have performed about the same, even though some of the chloride concentrations at the top mat of steel were as high as 2,000 parts per million.

(4) The old truss bridge at Prescott was redecked one lane at a time during weekend closures. A precast slab on stringer system was used between each set of floorbeams. This resulted in a joint above each floorbeam. Drawings for this project have been requested from MnDOT.

(5) On a small bridge crossing Minnehaha Creek, MnDOT considered filling in the median to provide extra bridge widths for traffic maintenance. There were objections to this scheme since it was viewed as providing additional future lanes which were not authorized.

(6) On some I-35W bridges, extra width was built into the structures to provide traffic maintenance during construction. These were in areas where the ADT was in the 100,000 vpd range.

(7) Where possible, the number of through lanes on divided roadways are reduced so that bridges can be redecked half at a time. This scheme is preferred over crossovers since user cost models for crossovers appear to be higher than those where traffic is maintained within the same directional lanes.

(8) The user cost modeled within the Pontis program assigns higher user costs for accidents and delays for crossover installations than for lane reduction methods. Clemson University has a contract to model user costs for AASHTO. MnDOTs traffic

information system may be used for Pontis input in the future to provide better user cost data.

(9) On the I-94 Hudson Bridge, MnDOT estimated that the user costs for the detour were in the range of $9,000 to $18,000 per day.

(10) MnDOT would like to have as much work done as possible in off-peak hours. However, noise problems are a big concern in the urban areas during nighttime hours.

(11) MnDOT Bridge is not willing to adopt substandard quality practices for the sake of speed of construction. There are pressures to sacrifice product quality for the sake of time.

(12) With the limited financial resources, MnDOT feels it is inappropriate to pay a substantial cost premium for rapid deck replacement schemes for the sake of traffic convenience. If additional funding is not provided, MnDOT cannot afford premiums in the range of 40% of construction cost for traffic maintenance requirements.

(13) MnDOT has a problem using epoxy-coated rebar in conjunction with uncoated prestressing steel. If a precast system was developed, MnDOT would prefer non-prestressed deck panels with all epoxy-coated mild reinforcing steel.

(14) Laminated timber panels with an asphalt overlay may be acceptable for low volume roadways but are not considered a viable option for high volume traffic. Timber panels tend to leak and do not hold up well under high traffic volume.

5. Missouri DOT

(1) MHTD does not use metal stay-in-place forms because of the extra dead load applied. Corrosion is not an issue regarding the use of metal stay-in-place forms.

(2) The use of stay-in-place forms and precast panels limit the inspection of moisture saturation through the concrete deck.

(3) MHTD specifies weight limits on hammers used for deck demolition. Since this limitation can cause the contractors additional expenses and time delays, they are open to trying other removal methods available with more freedom on the procedures used to obtain the end result desired.

(4) In some instances, environmental concerns, such as collecting the demolished deck debris, are of major concern. These concerns may be of more importance than money and speed for the deck replacement project.

(5) MHTD currently uses a 3" precast panel with 5-1/2" cast-in-place concrete on top. No full depth panels are used. The depth of the precast panels and cast-in-place concrete have changed from initial depths of 3-1/2" and 5" in order to reduce cracking in the overlay at panel joints.

6. Nebraska DOT

(1) NDOR agrees that noise can be a problem during nighttime construction.

(2) Concrete finishing during nighttime is of better quality because it is easier to see the ripples in the concrete because of the shading of light. These same advantages cannot be achieved in asphalt construction because of its dark color.

(3) NDOR usually does not require the contractor to use special equipment. Means and methods for deck removal should be the contractor's decision and should be based on their familiarity with specific equipment.

7. New York DOT

(1) Most of the NYDOT deck replacement has been "cast-in-place" with SIP steel forms filled with EPS (expanded polystrenene) to reduce the weight. Deck removal equipment and methods are up to the contractor except in the zone directly over the girder where the State limits the demolition to a 25 lb. hand hammer. Hand demolition over the top flange is slow and expensive.

(2) Stud removal is optional. Most contractors opt for stud removal and replacement.

(3) NYDOT has not had any problems with the performance of SIP metal deck forms.

8. Texas DOT

(1) TXDOT allows three different types of forming systems: steel stay-in-place forms, concrete stay-in-place forms, and removable forms, in order to give contractors maximum flexibility.

(2) TXDOT uses a detail called a "poor boy" joint. This involves using simple span prestressed girders with the deck continuous over the top of the bents. A construction joint is always required at the centerline of the bents which increase the time to replace the deck. They permit the use of a Zip Strip contraction joint insert and allow the deck to be poured in a continuous fashion over more than one span. This reduces the total deck casting time.

(3) There are no significant concerns regarding longitudinal construction joints on bridges with active traffic. Recent research reports confirm that there are little or no problems due to traffic vibration.

(4) TXDOT utilized "Pyrament" concrete in US 79/84 bridge project over the Trinity River, where TXDOT performed an emergency project for partial and full depth deck

replacement. Construction sequences of using pyrament concrete are: (a) the concrete is mixed on the site and transported to the patch area in power buggies; (b) when the concrete is dumped into the patch area, the plastic concrete will have often already set into the shape of the bucket; (c) a type of monomer is sprayed onto the concrete to allow the concrete to become workable again; (d) the concrete obtains the required strength within four hours and the traffic can be placed on the patches at the end of the work day.

9. Virginia DOT

(1) Recommends loosening of diaphragms to reduce the problems associated with differential deflections during multi-phased construction of a bridge deck.

(2) VDOT allows metal deck forms, but inspection from the bottom is impossible.

(3) Whip hammers, which are used to punch holes through a deck to be replaced, should be kept away from girder flanges.

(4) Most of the studs are saved; a hand held hammer has to be used. Choice of demolition equipment depends upon cost and environmental acceptability.

10. Washington DOT

(1) In general, a majority of deck replacements in WADOT are on old steel bridges which are usually non-composite. Typically, these bridges experience considerable deterioration of the top flanges as well as general deterioration of the structural steel members near the joints. Due to the loss in section of the structural steel, attempts have been made to eliminate joints and possibly make the steel structures composite when redecking them.

(2) Methods using ultrasonic testing prior to deck removal have been attempted to determine the extent of deterioration of structural steel members; however, it has not

produced good results. Consequently, WADOT has been using core holes above the stringer to floor beam connections so that coupon samples of the steel stringers can be taken prior to redecking operations. They have redecked some prestressed girder bridges but the older noncomposite steel bridges make up the largest number of redecking projects. Primary problems associated with prestressed girder bridges include damage to existing prestressed girders and shear stirrups during removal operations.

(3) WADOT does not generally consider cost benefit analysis when deciding which decks to replace. Life cycle costs are considered but they have found that the results of these vary depending upon the interest rate assumed. In general, if the cost to rehabilitate or redeck a bridge in today's dollar exceed 35-40% of the total cost of replacing a bridge, it may be cheaper in terms of life cycle costs to totally replace the structure.

(4) WADOT is receptive to nighttime work since it provides a more conducive environment for replacing concrete. It was noted that nighttime quality tends to go down since the primary inspection staff of the DOT and the supervisory staff of the contractor are often absent during nighttime pours. The same is true for work done on Fridays, holiday, and weekends.

(5) In general, many of the bridges on the west coast of the state are over inland bays and waterways, therefore, have a low salinity content. These bridges are not in an aggressive marine environment as might be expected.

(6) Washington historically has not used partial depth precast concrete deck panels. They have used some steel SIP deck forms in the past but currently are not using them due to concerns with respect to corrosion and inspection.

(7) WADOT is currently working on a project called the Lewis and Clark Bridge over the Columbia River at Longview. This bridge utilizes a 32' clear roadway on which they

are going to try to maintain two lanes of active traffic during construction. Restriction to a single lane of traffic may be allowed between 8 p.m. and 5 a.m. with total closures every other weekend. Options that have been considered include utilizing short closure periods with full depth precast deck panels. The concerns with the full depth panels include the cracking of grout placed over the top of the girders to build up the haunches when the grout exceeds 1/2" thickness. An additional concern is the alignment of longitudinal and transverse ducts for post-tensioning due to differential movements between adjacent sections.

(8) WADOT has a lot of experience with grid type steel decks. In general, these types of decks do not seem to last as long as conventional cast-in-place decks. Connection details are plagued with fatigue problems and they lose their skid resistance more rapidly than a conventional cast-in-place deck. Many of their steel grid decks have had an overlay added to prevent cupping and to provide a more skid-resistant surface. Many of the applications for steel grid decks have been for movable bridges.

Agenda Item #3: Description of any practices employed by DOTs to allow new bridges to be redecked more rapidly in the future.

1. California DOT

(1) CALTRANS has used bolts as shear connectors on some bridges in the past. On at least one structure, the Antioch Bridge, the nuts on the bottom of the top flange which hold the shear connectors in place became loose from traffic vibration. It was felt that these were either not high strength bolts or were not fully torqued at installation.

(2) With the use of epoxy coated reinforcing steel and a good bridge maintenance policy, CALTRANS does not feel it is necessary to detail bridges for rapid bridge deck replacements in the future.

(3) CALTRANS Traffic Maintenance Policy:

- Traffic impacts must be addressed in the planning stage of all projects.
- Traffic management plans are prepared by CALTRANS for all projects.
- A multiple step planning process is used:

  a) Public information

  b) Incident management for accidents, disabled vehicles, etc.

  c) Motorist information on job site

  d) Construction specifications and design (These are generally negotiated between the design group and traffic engineering.)

  e) Demand management strategies which include alternate modes of transportation

  f) Alternate route strategies

2. Georgia DOT

Information lost

3. Illinois DOT

IDOT uses a multi-girder design for new bridges. This design will allow for three girders under each traffic lane and will help in the future redecking of individual lanes.

4. Minnesota DOT

(1) MnDOT does not feel epoxy-coated rebar is the solution to all deck deterioration problems. The use of a double protection system involving all epoxy-coated steel in the deck plus a high density, low slump overlay may extend the life of the original decks. MnDOT, however, does not feel that there is such a thing as a 75-year deck.

(2) Total superstructure replacement may be more viable than deck replacements to deal with future deck deterioration problems. There was a bridge constructed for Northern States Power Company which utilized bulb-T superstructure members which were transversely post-tensioned together. An overlay was placed on the deck after post-tensioning.

(3) Any deck systems involving post-tensioning may be difficult to widen in the future.

(4) The rapid-curing concrete materials referenced in the SHRP manual for patching and repairing of decks seem appropriate for small area, partial deck replacement projects.

5. Missouri DOT

No design consideration is given for future rapid replacement of the decks.

6. Nebraska DOT

No design consideration is given for future rapid replacement of bridge decks. In Nebraska, there are very few bridge sites that would dictate extreme procedures to maintain full traffic volumes. Usually, partial maintenance of traffic or alternate detour routes with full closure of the bridge being redecked are the preferred choices. Also, the majority of cases require rehabilitation of other structural elements or total bridge replacement.

7. New York DOT

(1) One innovation being developed for the NYDOT is "Inverdeck" which is an outgrowth of the "Invertset" system.

(2) Exodermic decks are lighter than the original decks and quicker than CIP to install.

8. Texas DOT

No design consideration is given for future rapid replacement of bridge decks.

9. Virginia DOT

(1) One bridge was built with full depth precast panels. The cast-in-place portion of this deck is the weak link of the bridge. There were cracks at every blockout for the cast-in-place concrete on Route 235.

10. Washington DOT

(1) WADOT pays particular attention to grid deck structures with respect to future replacement. Steel grid decks do not perform as well as cast-in-place concrete decks and are subject to continual maintenance due to the fatigue of connections.

(2) Segmental concrete bridges are usually built with additional empty post-tensioning ducts to facilitate future redecking or maintenance operations.

(3) Several general practices are used to extend the life of an existing bridge deck. Epoxy-coated reinforcing is utilized only for the top mat of most cast-in-place decks. The bottom mat is also epoxy-coated in marine environments. A clear cover of 2.5" is utilized for the top mat of reinforcing. The use of cathodic protection has generally not been used, but may be considered necessary for high investment bridges, particularly bridges of segmental construction which do not accommodate easy deck replacement.

Agenda Item #4: Identification of specific items that the DOTs would like to see incorporated in the NCHRP study.

3. Illinois DOT

(1) IDOT would like to see the use of composite materials for bridge deck replacement. Also, they are interested in the use of non-conventional prefabricated materials, such as fiberglass, carbon fiber, etc.

(2) They would like to see the investigation of skid resistance and fatigue problems in bridge deck design. IDOTs concerns with wider girder spacing (fewer girder lines) in new design projects may result in problems with future deck replacement under stage construction. This may result in two girder lines supporting the staged construction under the traffic which could cause instability problems. They are interested in seeing wider spacing for the abutments and pier caps. IDOT is incorporating this idea into new design projects.

4. Minnesota DOT

(1) Life cycle cost considerations.

(2) Deck replacements should be considered for several types of girders.

(3) Details would be appropriate for precast deck replacement systems including both match cast decks as well as grouted joint systems.

(4) Actual field testing of the details proposed in our research project is recommended.

5. Missouri DOT

(1) MHTD is interested in partial repair of bridge deck panels when needed.

(2) During staged construction, forces and deflections from the side under traffic are transferred to the section being replaced prior to completion of curing. MHTD has loosened diaphragms in the past, when attempting to reduce any possible problems for deck replacement.

6. Nebraska DOT

(1) Wide spacing of girders is a factor to be considered in controlling traffic flow and enhancing the speed of replacement of bridge decks. Wider spacing of girders helps in speeding up demolition as long as there are at least four girders to allow one direction to be closed while the other is being redecked.

(2) The quality of construction with full-depth precast panels may not be satisfactory in providing a smooth riding surface.

(3) In extending deck life, high density overlays can be an attractive alternative. However, high performance concrete is gaining popularity and will be more economical than expensive overlays. Prestressing and post-tensioning with high strength concrete will be a useful method for extending deck life to match the girder life.

7. New York DOT

(1) Include FRP applications in your study. Look at new materials. Keep working on prefabricated systems until you get the problems solved and get these systems to be competitive.

(2) Precast panels may have elevation and camber problems.

8. Texas DOT

(1) TXDOT would like to see clear and complete standard details for rapid bridge deck replacement systems.

9. Virginia DOT

(1) Use a minimum of five girders to allow closure of one side of a bridge for maintenance, while traffic is maintained on the other side.

(2) Value engineering can force you to have wider girder spacing. Value engineering should take long term maintenance into account.

(3) Corrosion of the top flange lifts the slab off of the top flange even if studs are used.

(4) The heads on studs seam unnecessary. Is there research done regarding this?

10. Washington DOT

(1) WADOT would like to see an inventory of the types of systems available for rapid deck replacement along with pros and cons of each system. Any new system developed must address the continuity of structures, rideability of the surface and water-tightness of joints.

(2) The primary consideration is low maintenance. Steel grid type decks require frequent maintenance to repair fatigue prone details.

C.3. Comments on Developed Conceptual Details

Options	DOT	Comments
A (See Fig. c.1)	Illinois	(1) The nut may become unscrewed due to mechanical vibration. (2) It may be difficult to remove a slab portion where there is a grade change.
	Missouri	(1) There may be a problem with the horizontal shear resistance of the connection.

	New York	(2) Fatigue testing should be required for the uplift resistance mechanism (1) You do not have much uplift resistance. Favor the idea of headless studs. Studs would be epoxy-coated for bond breaking. Instead of underside clips, consider threaded stud with a nut at about 4 ft. spacing to resist the uplift. Then you can back drill from the top, remove the nut, and lift the deck..
B (See Fig. C.2)	Illinois Missouri New York	(1) A similar connection was used in Senaca bridge in Illinois. (2) A collection of moisture would make it difficult to unscrew the nuts. (3) Composite nuts and bolts can be used for the connection. (1) It is doubtful to use lubrication between the deck and the girder for removal. (2) With lubrication, a large clamping force is required. (3) It is possible to use stainless steel connection components to resist corrosion. In general, option B is preferred to option A. (1) Holes in top flange would be more expensive than option A.
C (See Fig C.3)		
D (See Fig. C.4)	Missouri New York	There will be concerns regarding the forming of the girder to allow for the placement of the metal angles for the connection. The angle may shift in the formwork when precasting the girder. (1) If you have an insert and screw a bolt in from the top, it might be more efficient. The part of the bolt above the girder could be sleeved or epoxy coated to facilitate debonding and deck removal.
E (See Fig. C.5)	Illinois Missouri	(1) The connection is very labor intensive. (2) The use of composite materials is desirable to avoid rust. (1) Concerns for the life span of a girder if the deck is replaced several times. Missouri's main concern for the girder would be the impact damage instead of salt corrosion. (2) MHTD would like to achieve a 20 to 30 year life from a bridge deck. (3) Provisions need to be provided to break the bond between the bolt and sleeve in the girder.
F (See Fig. C.6)	Illinois	(1) Fatigue may be a problem. (2) Composite materials may be used for this details. (3) Welding of a metal block to the side of a flange is expensive.

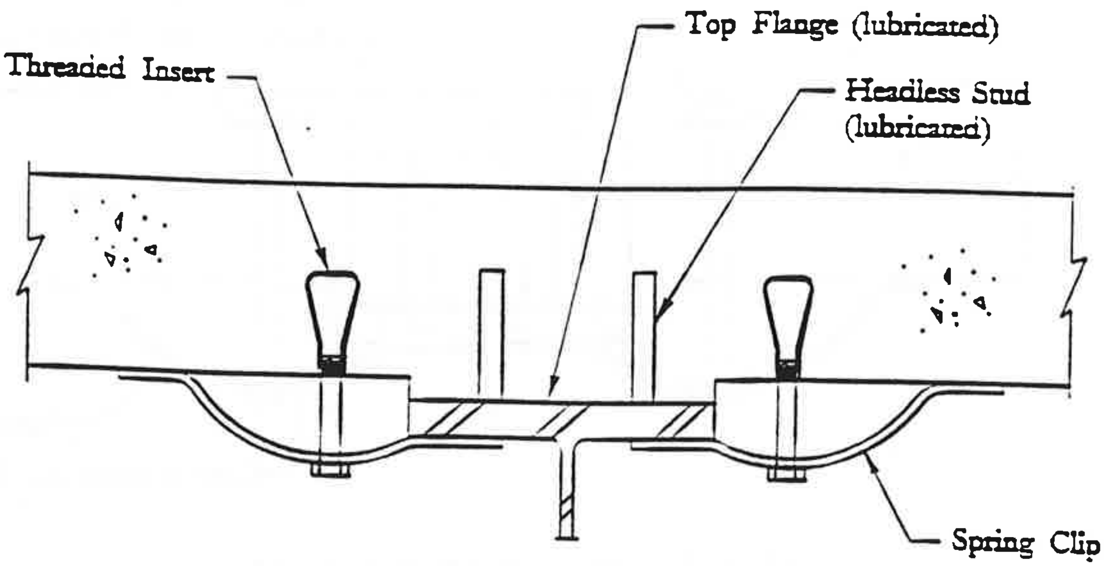


Fig. C.1. Cast-in-Place Deck on Steel Girder

New or Replacement Decks

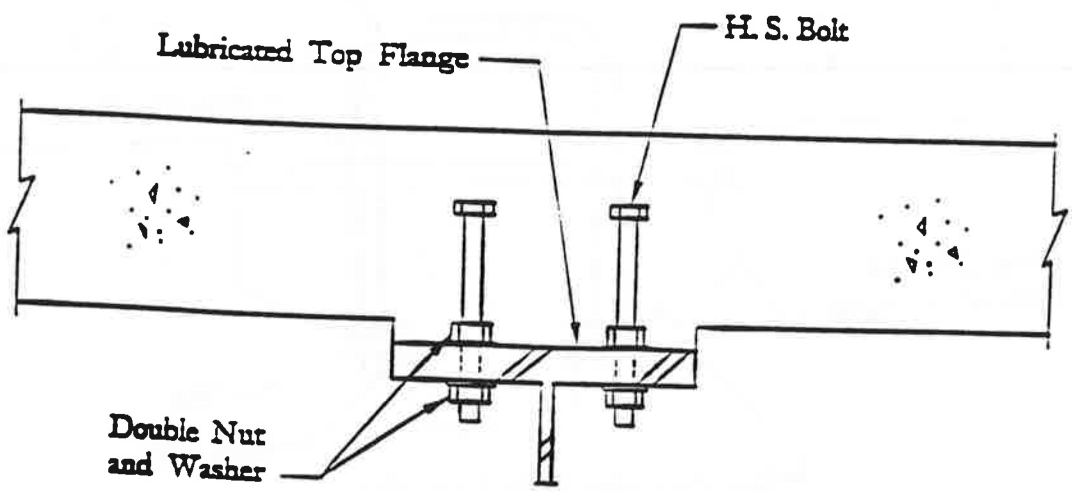


Fig. C.2. Cast-inPlace Deck

New Deck

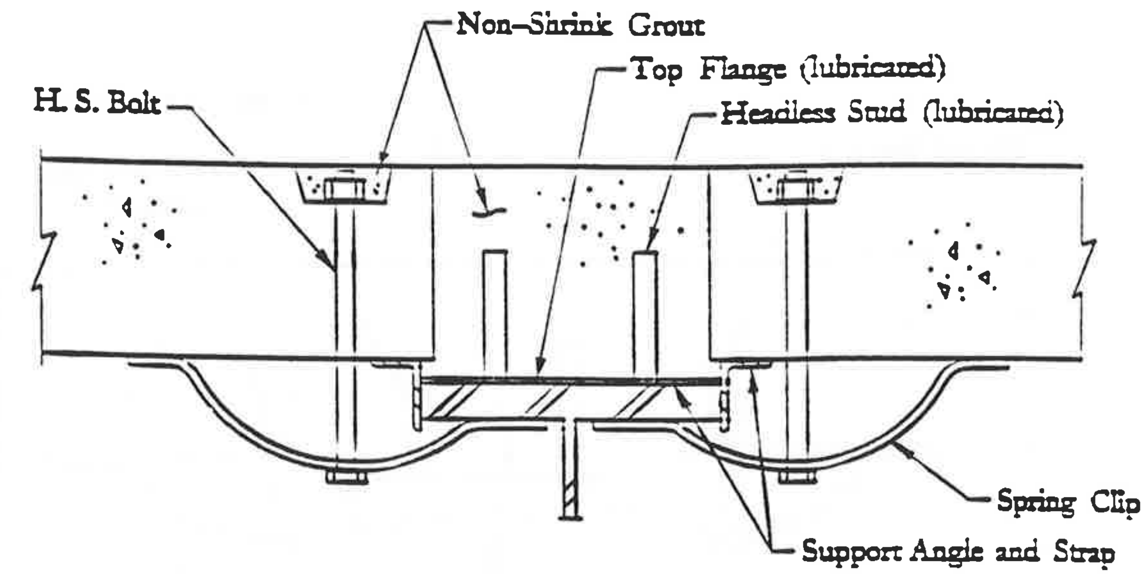


Fig. C.3. Prefabricated Deck on Steel Girder

New or Replacement Deck

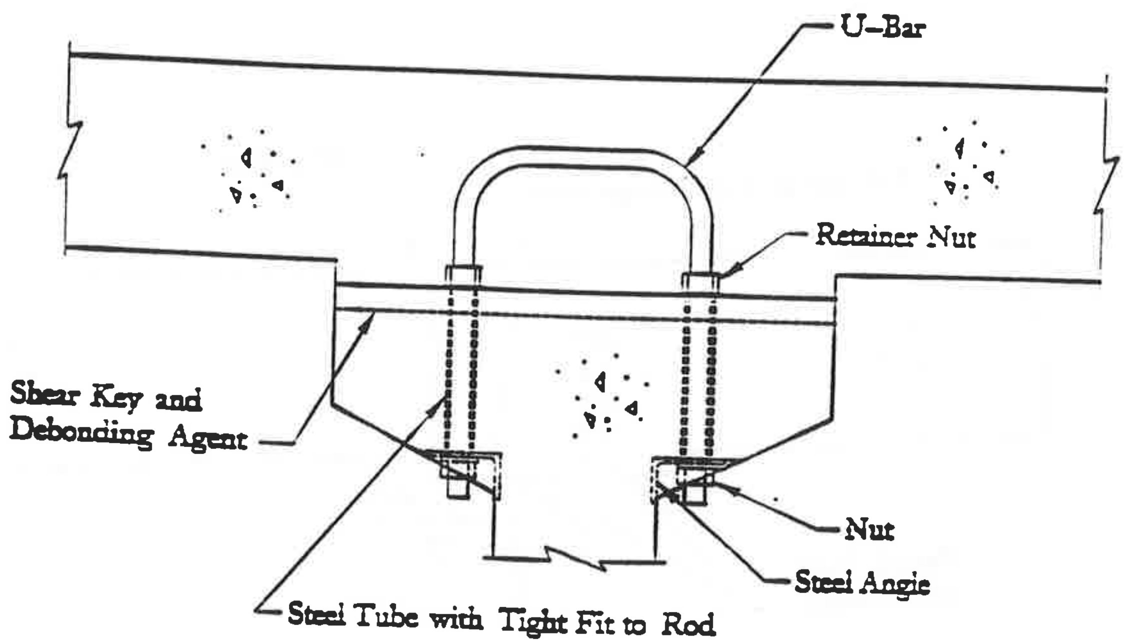


Fig. C.4. Cast-inPlace Deck on Precast Girder

New Deck

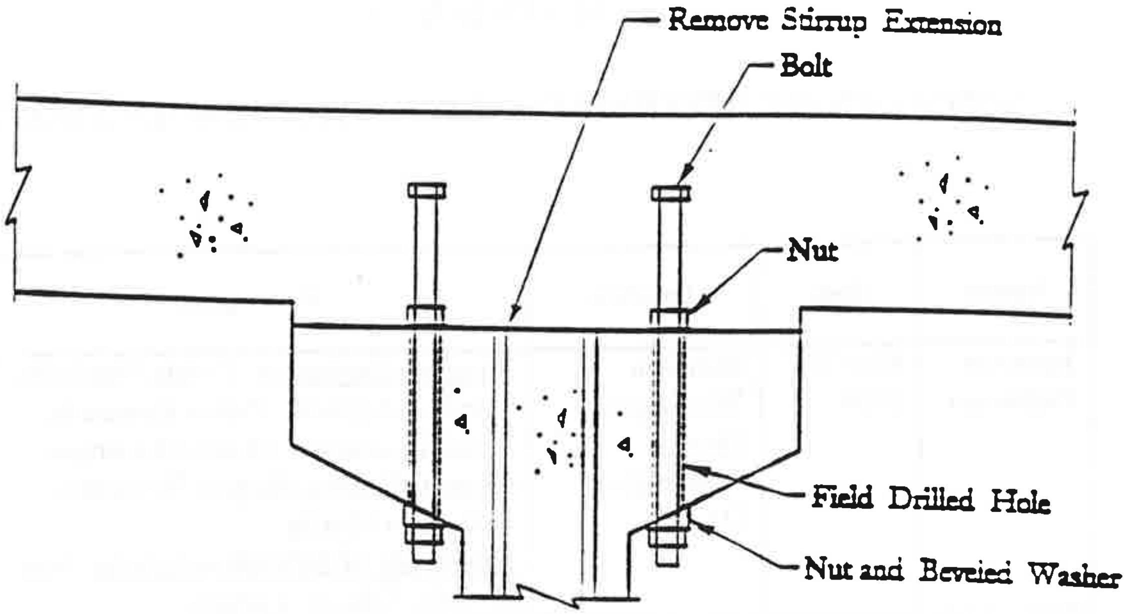


Fig. C.5. Cast-in-Place Deck on Precast Girder

Replacement Decks

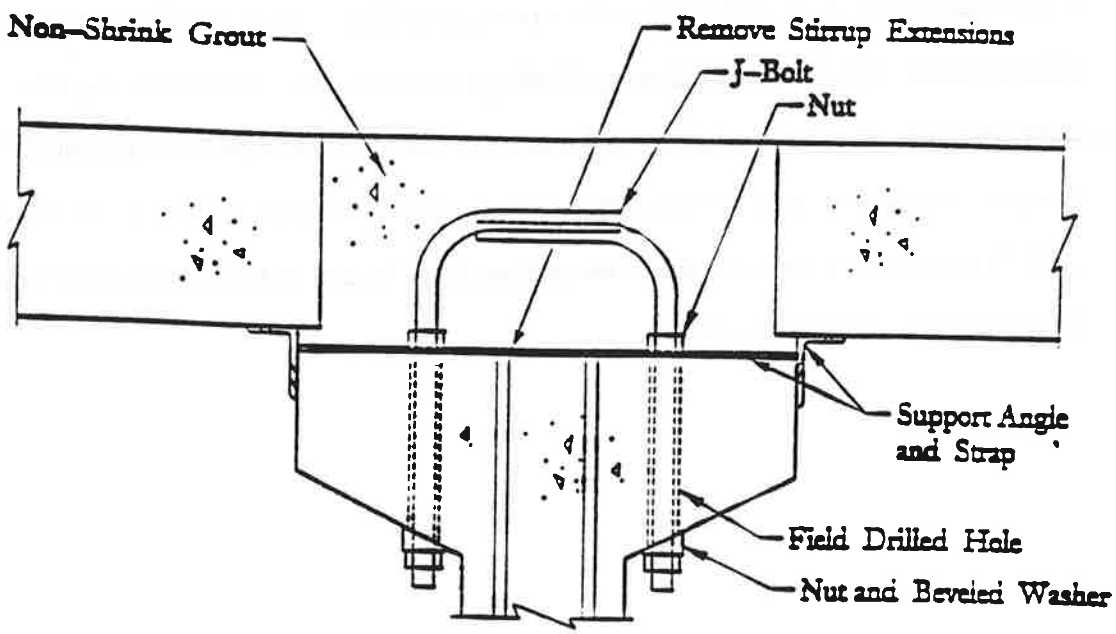


Fig. C.6. Prefabricated Deck on Precast Girder

New or Replacement Deck