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Implementation of Full-Width Precast Bridge Deck Panels: A Synthesis Study

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16. Abstract

In this synthesis study, information related to full-depth precast deck systems has been collected and analyzed in order to facilitate implementation of full-depth precast deck systems in the state of Indiana. Nine full-depth precast deck systems were identified in the literature review. For girder bridges or similar, a full-depth precast, prestressed deck system (New England Region System) is recommended to be implemented. For bridges with decks spanning in the longitudinal direction, where no precast, prestress deck systems are available, the Exodermic™ Deck System is suggested to be implemented. This system is also an alternative to the New England Region System for implementation in girder bridges. The Effideck™ System, of proprietary nature, is recommended as an alternative for bridges with decks spanning in the longitudinal direction. None of the systems, however, has been implemented over concrete or precast girders.

It is also suggested for implementation the use of 1 ¼” diameter headed studs as connectors for steel girders or beams in cases of cast-in-place decks, stay-in-place decks, or full-depth systems. The use of these larger studs in a single row over the girder web reduces the effort required for deck removal and also the probability of damaging the girder top flange and the larger stud.

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TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

CHAPTER 1
LITERATURE REVIEW

1.0 Introduction

This chapter has three sections. First, an overview of findings related to full-depth precast deck systems in the last 10 years is presented. The overview includes brief descriptions of full-depth precast deck systems found and also relevant information related to their components. Information is classified into three major groups: precast, prestressed concrete deck systems, steel grid deck systems and other composite systems.

Next, findings related to deck/girder connections are presented. Even though each deck system has its own type of connectors, research has been, and is still being done to improve connections, especially from the standpoint of deck removal. Is convenient to have this information separated from deck systems itself, because new type of connections could be applicable to different deck systems, and also extend their use to other type of bridges.

Finally, a detailed description of full-depth precast deck systems developed or most used in the last ten years is presented. Each description typically includes information about the precast panels, joints between adjacent panels and features related to the applications of each system. An appendix of figures is included at the end of the report.

1.1 Overview of full-depth precast deck systems

In this section, a summary of findings related to recently developed or existing full-depth precast deck systems is presented. The summary includes brief descriptions of full-depth precast deck systems and relevant research done to improve their use. Information is classified into three major groups: precast, prestressed concrete deck systems, steel grid deck systems and other composite systems, which include proprietary deck systems.

1.1.1 Precast, prestressed concrete deck systems

Full-depth precast and precast, prestressed concrete decks have been used in the USA for rehabilitation and new construction of bridges for three decades. They have been used in all types of bridges with different profiles, such as skewed, superelevated, and crowned (Issa et al., Feb. 1995). The inclusion of prestressing is to protect panels from cracking during handling and installation and/or reduce panel thickness and, therefore, weight.

A full-depth precast, prestressed concrete deck system was developed by the Connecticut Department of Transportation, based on successes of previous work done by several agencies. The system was suitable for girder bridges and consisted on precast concrete panels pre-tensioned in the transverse direction and post-tensioned in the longitudinal direction. Post-tensioning was applied uniformly throughout the entire bridge width.

Precast panels had pockets over the girders for placement of shear connectors. The system was successfully tested in two major deck replacements, a bridge in Waterbury, CT, in 1989, and two bridges in Seymour, CT, in 1994 (Culmo, 2003). Fig. 1 shows the deck replacement of a bridge in West Hartford, CT, in 2000.

A survey conducted by Issa et al. found that seven states (Indiana, New York, Maryland, Illinois, Washington, Connecticut and Alaska) had used full-depth precast, prestressed concrete panels in bridge decks (Feb. 1995). An investigation to evaluate the field performance of full-depth panels in bridge decks concluded that problems found could be attributed mainly to the type of connection between slabs and supporting system, the configuration of joint between adjacent panels, construction procedures, the lack of longitudinal post-tensioning, and the materials used (Issa et al., June 1995).

Issa et al. (1998) carried out a quantitative study to determine the amount of post-tensioning needed in the longitudinal direction of girder bridges. The model used in this study was very similar to the system developed in Connecticut. They concluded that for simply supported bridges, a minimum post-tensioning stress level of 200 psi is needed, and for continuous bridges, a minimum of 450 psi is required at interior supports, and 200 psi at midspan.

A new full-depth precast, prestressed concrete deck system was developed under NCHRP Project 12-41 “Rapid Replacement of Bridge Decks” (Tadros and Baishya, 1998). The system consisted of concrete panels pre-tensioned in the transverse direction and post-tensioned in the longitudinal direction. Main features of the system were that panels had a non-prismatic stemmed section, to optimize the system in terms of weight and amount of reinforcement, and that post-tensioning was applied over the girders. After the full-scale prototype of the proposed system was tested, some recommendations to improve the system were published (Yamane et al., 1998). Fig. 2 shows an overview of this system.

Another precast, prestressed concrete deck system called the Nudeck was developed under the same NCHRP Project 12-41 (Badie et al., 1998). Although it was originally a continuous stay-in-place panel system, the Nudeck evolved later to the NU-Deck, a full-depth precast, prestressed panel system with the inclusion of longitudinal post-tensioning over the girders. Main characteristic of the NU-Deck System is that panels have open channels over the girders for the placement of the post-tensioning strands and shear connectors. Fig. 3 shows a typical NU-Deck panel for the construction of the Skyline Bridge in Omaha, NE, in 2003.

In 2002, the PCI New England Region Technical Committee approved design guidelines for the use of full-depth precast deck slabs in new construction or in replacement of existing bridge decks in the New England Region (PCINER, 2002). These design guidelines were based on the details developed by the Connecticut Department of Transportation (Culmo, 2003).

Issa et al. (2003) evaluated the performance of four different types of grout materials at transverse joints between precast panels. The four products were Set® Grout, Set® 45, Set® 45 HW (for hot weather) and Emaco® 2020 (polymer concrete), which were tested in 36 full-scale specimens prepared with female to female joints. The polymer concrete was found to be the best material for transverse joints in terms of strength, bond and mode of failure. However, since this material is very expensive, the use of Set® Grout was recommended at transverse deck joints due to its ease of use and satisfactory performance. In special cases, where joints are subjected to excessive stresses or quick resumption of traffic is critical, the proper application of a more expensive polymer concrete was recommended.

1.1.2 Steel grid deck systems

Grid reinforced concrete decks have a long history dating from the construction of the Oakland Bay Bridge in the early 1930's. The importance of this history is that many of the grid reinforced concrete decks installed in the 1930's are still in service today, located in some of the most severely corrosive environments in the country. Moreover, in most cases where the decks have been replaced, the problems have been in the bridge structure under the deck, not the deck itself. Upper limits of service of grid reinforced concrete decks have not been reached (IKG Greulich, 1991).

There are many different grid designs that have been used and that are still being used in bridge deck construction. However, steel grid deck systems are usually classified in three different types: open grids, full-depth concrete filled grid decks, and half-depth concrete filled grid decks. Figs. 4, 5 and 6 show examples of the three types of decks.

L. B. Foster, American Bridge Manufacturing and Interlocking Deck Systems International are companies that have their own grid designs. Greulich Bridge Flooring Systems, with a large history in bridge decking, have recently been bought by L. B. Foster (2002).

In the early 1980's, another type of steel grid deck system called the Exodermic™ Bridge Deck was developed by Neal Bettigole, a consulting bridge engineer and head of a mid-sized firm headquartered in New Jersey. Defined as a 'composite unfilled steel grid', the exodermic deck is comprised of a reinforced concrete slab on top of, and composite with, an unfilled steel grid. Fig. 7 shows the original Exodermic™ Deck System (EBDI, 1998).

In the early to mid 1990's the original Exodermic™ Deck System design was revised and the shear transfer mechanism was simplified to obtain a more efficient, more economical, and easier to install deck system. The revised design was successfully used for the first time in the deck replacement of the Tappan Zee Bridge in New York in 1998 (Caltrans, 2003) and has been, since then, the standard for all Exodermic decks (EBDI, 2003). Fig. 14 shows the revised Exodermic™ Deck System.

In the last years, Interlocking Deck Systems International has introduced a new technology called weld-less steel bridge decking. They have six different grid designs that do not require welding (Figs. 8 to 13). Two of them, the 5-inch, half-filled concrete (Fig. 10) and the Tee-Lok (Fig. 11) have been successfully fatigue tested at the University of Pittsburgh. These weld-less deck systems can also be classified as open grids, full-depth concrete filled grid decks, and half-depth concrete filled grid decks (IDSI, 2003).

1.1.3. Other composite deck systems

In the early 1980’s, the Inverset™ Bridge System was developed by Stanley Grossman, P.E., of Norman, OK, who presently holds a patent on the system. The system is defined as a precast, pre-compressed, composite superstructure made up of steel beams and a concrete slab, which act as a composite unit. The beams are prestressed using a unique upside-down casting technique that uses the force of gravity to prestress steel beams. Inverset™ units can be used either in the longitudinal or the transverse direction of the bridge (TFMC, 1998). This system has been used in more than 145 bridges in over 12 states since the first bridge was installed in Cleveland County in Norman, OK, in 1982 (TFMC, 2002). Fig. 15 shows a typical Inverset™ unit.

The Effideck™ System was developed in the second part of the 90’s and is proprietary to The Fort Miller Co., Inc. Effideck is a precast deck system consisting of modular deck panels using a relatively thin (typically 5” thick) concrete slab supported by closely spaced hollow structural section steel tubes bonded to the underside of the slab. Effideck™ panels can span either in the transverse direction over bridge girders or stringers or in the longitudinal direction over floor beams (TFMC, 2002). This system has been successfully used in New York and Vermont in at least four bridges since 1997. Fig. 16 shows a typical Effideck™ panel.

1.2 Deck/girder connections

In this section, findings related to deck/girder connections in the last years are presented. Information is classified in two major groups: deck/concrete girder connections and deck/steel girder connections.

1.2.1 Deck/concrete girder connections

Current practice for connections between precast concrete girders made composite with concrete decks consist on developing horizontal shear through roughening the top flange surface and extending reinforcing bars from the girder, usually the shear reinforcement, into the concrete deck (Fig. 17).

A new connection system was developed at the University of Nebraska under NCHRP Project 12-41 “Rapid Replacement of Bridge Decks” in 1998. The system proposed was a debonded interface with shear keys formed into the girder and steel connectors at wide spacing. A debonding agent was applied to the hardened concrete using a brush or hand-held sprayer (Tadros and Baishya, 1998). Figs. 18 and 19 show the proposed debonded system and the steel connectors.

Full scale tests were performed to compare the performance of a conventional bonded system with the new unbonded shear key system. After the tests, 10 ft sections of the deck from both systems were removed with jackhammer. The tests performed provided excellent results for both composite action and deck removal.

The debonded surface with less congestion of reinforcement enhanced the deck removal. At sections away from the steel connectors, removing concrete deck from the debonded composite member resulted in 50% in time savings, while around the steel connectors, resulted in 20% time savings. No damage to the debonded shear keys and the steel connectors was noticed (Tadros and Baishya, 1998).

The Nebraska Department of Roads (NDOR) selected two bridges in the Douglas County, NE, to compare the performance of a conventional roughened interface system and the new debonded shear key system. For this project the concept of projecting the shear keys above the standard girder top flange surface was modified to have the shear keys below the top flange surface (Figs. 20 and 21). In this way the system was suitable for different types of decks like CIP systems, SIP systems and full-depth precast panels. Shear connectors were epoxy coated U-shaped bars embedded in the girder web and extended into the concrete deck slab (Tadros et al., 2002). Fig. 22 shows a top view of the debonded shear system.

The conclusions for this study were that the new system had the advantages of facilitating future deck removal, protecting shear connectors against corrosion, protecting the girder top flange from damage during deck removal and optimizing the design for horizontal shear. The system was found to perform well and had no detrimental effects on composite action or bridge stiffness. The incremental cost of using the new system was expected to be minimal and should eventually turn into savings when reduction in time and labor during deck removal is accounted for (Tadros et al., 2002).

Research recently concluded in Virginia Tech studied the horizontal shear strength developed at the interface of two precast concrete members bonded by means of grout. Test parameters studied include different grout types, different haunch heights, and different amounts and types of shear connectors (Menkulasi, 2002).

Important conclusions of this study were that shear connectors should be developed at least 5 in. into the slab in order to prevent pry-out failures, post-installed hooked rebars are a very convenient type of shear connector and epoxy is a very convenient type of adhesive to provide the bond between the hooked rebars and the concrete. Also, tests with shear keys on the beam side found a significant increment in shear capacity of the specimens, and more study for this type of connection was recommended. Finally, equations for uncracked and cracked concrete interfaces were proposed to be used in horizontal shear design when precast panels are used (Menkulasi, 2002).

1.2.2 Deck/steel girder connections

The most common type of shear connector used for developing composite action between steel girders and concrete decks is a welded shear stud, which consists of a smooth shank with a flat round head. The stud is welded to the top flange of the steel girder using an efficient arc welding process. The most common diameters of steel shear studs used in composite bridge construction are ¾" and ⅞" with a height of typically 5" (Fig. 23), which are often positioned in two or three rows spaced at 5" to 18" (Tadros and Baishya, 1998).

Research conducted at the University of Nebraska under NCHRP Project 12-41 "Rapid Replacement of Bridge Decks" proposed a new 1 ¼" diameter shear stud system (Figs. 24 and 25). This stud provides approximately twice the capacity of a ⅞" diameter stud and would allow positioning in a single row over the girder web. This arrangement greatly reduces the amount of jackhammering (saw cutting could be done very close to the girder centerline), and the probability of damaging both the girder top flange and the larger studs. Also, the research team found that alternating headed and headless studs was adequate for anchorage to the concrete deck. This further facilitates deck removal (Tadros and Baishya, 1998).

The 1 ¼" studs used were standard SAE 1018 steel with estimated yield strength of 52 ksi, and ultimate tensile strength of 64 ksi. The steep chamfer of these studs was found to greatly facilitate the welding process. The amount of flux material used was twice that needed for conventional ⅞" studs. The gun used to weld ⅞" studs had to be modified. Welding 1 ¼" studs require a 3000 amp power source with an appropriate power supply cord to avoid overheating of the equipment. Such a power source is now available from commercial vendors. A device for field inspection was also developed. With all these modifications, stud welding can be appropriately performed (Tadros and Baishya, 1998).

Test results confirmed that 1 ¼" studs could be efficiently welded in the shop or in the field, conveniently inspected for quality of weld, and designed using standard procedures. Welding the 1 ¼" studs can be done at approximately the same rate as for ⅞" studs, thus, the total time required to weld the larger studs is about 50% of that to weld the smaller studs. The total material cost was comparable (Tadros and Baishya, 1998).

Limited testing was performed to determine the fatigue capacity of the 1 ¼" shear studs. Tests showed these studs had higher fatigue resistance than conventional ⅞" shear studs. Additional tests are needed to provide sufficient data to support changes to the fatigue resistance provisions in the AASHTO specifications (Tadros and Baishya, 1998).

Headless studs, as well as threaded studs, were used as connectors in a full-depth system developed under the same research project. After the system was tested, the panels were removed. While headless studs allowed for easy removal of the panels, threaded studs provided an unexpected resistance to removal of the panels from the girders (Yamane et al., 1998).

This system could be used with cast-in-place decks, stay-in-place decks, and full-depth systems with open channels over the girders, like the NU-Deck System, the Exodermic™ Deck System, or other steel grid decks. The system was implemented recently with the NU-Deck System in the Skyline Bridge (October, 2003), but with headed studs only.

1.3 Detailed description of full-depth precast deck systems

In this section, a detailed description of the full-depth precast deck systems developed or most used in the last ten years is presented. Each description typically includes information about the precast panels, transverse joints and features related to the applications of each system.

1.3.1 Precast, prestressed concrete deck systems

In this section, a detailed description of three different precast, prestressed concrete systems is presented. Systems included are the one adopted in the New England Region (NER System), the one developed under NCHRP Project 12-41 (NCHRP System), and the NU-Deck System.

1.3.1.1 New England Region System

The Connecticut Department of Transportation developed a full-depth precast concrete deck slab system based on successes of previous work done by several agencies (Culmo, 2003). The system was suitable for girder bridges and consisted of precast concrete panels pre-tensioned in the transverse direction and post-tensioned in the longitudinal direction. Post-tensioning was applied uniformly throughout the entire bridge width, and panels had pockets over the girders for placement of shear connectors. The system was successfully tested on two replacement projects using different construction approaches. The first involved a full closure of a bridge in Waterbury, CT, in 1989, and the second was done with partial closures during weekends in two bridges in Seymour, CT, in 1994 (Culmo, 2003). In 2002, the PCI New England Region Technical Committee approved design guidelines for the use of full-depth precast deck slabs in new construction or in replacement of existing bridge decks in the New England Region (PCINER, 2002). These design guidelines were based on the details developed by the CT-DOT (Culmo, 2003) and had already been used in several projects.

The following information is based on Report PCINER-02 (2002) and the information gathered in a visit to the CT-DOT and two bridges in Connecticut (2003).

1.3.1.1.1 Specifications

The recommended design concrete compressive strength is 6,000 psi, with a minimum of 5,000 psi, which was used in Seymour. At time of prestressing transfer it should be no less than 4,000 psi. All mild reinforcement (ASTM A615) shall be epoxy coated (ASTM D3963). Pre-tensioning and post-tensioning reinforcement are seven wire strand, Grade 270, low relaxation (ASTM A416).

1.3.1.1.2 Precast panels

Panels are laid out perpendicular to the girders and have block-outs for shear connectors. A typical panel layout plan for a skewed bridge is shown in Fig. 26. Panels can be cast with a skew for skewed bridges (Fig. 27) or in a trapezoidal shape for curved bridges (Fig. 28). Panel width is typically 10’ and panel length depends on the width of the bridge. In case of staged construction, two or more panels can be used in a single bridge transverse section with longitudinal joints between adjacent panels. Fig. 29 shows the cross section of one of the bridges in Seymour, where both the northbound lanes and the southbound lanes are supported by five girders each. Panel thickness for that project was 8” for a maximum spacing between girders between 8’ and 9’. Fig. 30 shows the deck plan for one span of the same bridge.

Transverse reinforcement for flexure may be designed with mild reinforcement, pre-stressing strands, bonded post-tensioning strands, or combinations of each. Pre-tensioning is the preferred method; however, mild reinforcement will most likely be required in the deck overhangs (PCINER, 2002). Bridges in Seymour, CT, and West Hartford, CT, were designed with a combination of pre-tensioning and mild reinforcement. However, the pre-tensioning reinforcement, which was not shown in the plans, was used only to ensure there are no tensile stresses in the precast slabs during handling and erection. The Contractor, who determines pre-tensioning methods, was responsible for the design of the transverse pre-tensioning (CT-DOT, 1992 and 1997). Fig. 31 shows the transverse reinforcement for a typical panel of the bridge in West Hartford.

Longitudinal reinforcement consists of mild reinforcement uniformly distributed in two layers throughout the whole panel and additional post-tensioning strands throughout the span to provide continuity between deck slabs. Post-tensioning ducts are located at mid-depth distributed between girders and run the entire bridge or between closure pours (Fig. 32).

1.3.1.1.3 Transverse joints and longitudinal post-tensioning

A female-to-female shear key is recommended at transverse joints (Fig. 33). Non-shrink grout is recommended as filler material. Block-outs are also provided for post-tensioning ducts connectors (Figs. 34 and 35).

The post-tensioning shall transmit a prestress of 200 psi minimum after all losses and all dead loads have been applied to the structure (PCINER, 2002). The final post-tensioning force per duct was indicated in the plans (46 kips/duct in Seymour and 45 kips/duct in West Hartford), but the Contractor was responsible to determine number and diameter of the strands (CT-DOT, 1992 and 1997).

In case of curved structures, longitudinal post-tensioning should run along the curve, and the design of post-tensioning should take into account the losses due to friction in the ducts (PCINER, 2002).

1.3.1.1.4 Deck/girder connection

Composite action between the deck and the supporting members is achieved with shear connectors placed in block-outs over the girders. In case of steel girders, shear connectors consist of welded studs. Fig. 36 shows typical block-out details. In case of concrete girders, hooked reinforcing dowels are recommended. Figs. 37 and 38 show details for both new construction and deck replacement. In both cases recommended dowels are independent of girder shear reinforcement. However, the system has not been used with concrete girders, yet.

Spacing of shear connector block-outs is typically 2' on center (Fig. 27). Design for variable horizontal shear can be accommodated by varying the number of shear connectors per block-out. In the bridge in West Hartford, each block-out had typically 4 welded headed studs (all ⅞” diameter).

1.3.1.1.5 Alignment

Transverse closure pours are used to account for construction tolerances and varying field conditions. A minimum 2’ cast in place closure pour is recommended (Fig. 39). In the bridge in Seymour, where all spans were designed as simply supported, plans show 2’ closure pours at both ends of the span (Fig. 30). In the same project, where weekend construction was used, another closure pour was considered at midspan (Figs. 30 and 40). This detail was included to give the contractor an option to remove and replace half a span and re-open for traffic on Monday mornings (Culmo, 2003).

Plans should include elevations of each slab (generally each corner of each slab) based on the required elevation of the slabs after all slabs are placed on a span. Fig. 41 shows a typical leveling device used to adjust the grade of the deck slabs after placement.

1.3.1.1.6 Crowned roadways

Longitudinal closure pours are used typically to accommodate cross slope changes in the deck. Fig. 42 shows a typical detail for crowned roadways. For narrow roadways, it may be possible to install the slabs level and crown the wearing surface (PCINER, 2002).

1.3.1.1.7 Parapets

Fig. 43 shows a typical parapet detail. Similar cast-in-place detail was used for bridges in Seymour and West Hartford.

1.3.1.1.8 Construction

A typical sequence of construction for each span is the following:

1. Place all precast panels on girders in a span.
2. Adjust leveling devices on deck slabs to bring slabs to grade.
3. Install longitudinal post-tensioning strands (un-tensioned) in ducts and seal joints between deck slabs.
4. Place polymer concrete between transverse joints.
5. Longitudinal strands can be stressed after the concrete in the transverse joints has attained a strength of 1000 psi.
6. Install shear connectors in all block-outs.
7. Form haunches between the top of the girders and the bottom of the deck slabs.
8. Grout all haunches and shear connector block-outs with quick setting non-shrink grout.
9. Cast closure pours with high-early strength concrete.

1.3.1.2 NCHRP System

A full-depth precast, prestressed concrete bridge deck system was developed and studied at the University of Nebraska under NCHRP Project 12-41 in 1998. The system consisted on precast, prestressed concrete panels, pre-tensioned in the transverse direction with indented strands and post-tensioned in the longitudinal direction with threaded bars, welded headless studs, welded threaded studs, grout filled shear keys and leveling bolts. A full-scale prototype of the proposed precast panel system was constructed and tested to confirm the feasibility of the design. An overview of the system is shown in Fig. 44.

The following information is based on the original NCHRP Report 407 (1998) and a later paper published the same year by Yamane et al.

1.3.1.2.1 Specifications

The specified compressive strength of the concrete for the precast panels is recommended to be between 6,000 and 10,000 psi. Design calculations for the specimen tested were made with a strength of 5,000 psi at transfer and a 28-day strength of 7,500 psi. For transverse pre-tensioning, 270 ksi low relaxation indented strands were chosen. Indented strands help reduce transfer and development length. The mild reinforcement bars and the welded wire fabric were Grade 75 steel (Tadros and Baishya, 1998).

1.3.1.2.2 Precast panels

Panels are laid out perpendicular to the girders and have block-outs for shear connectors. The precast panels cover the entire section of the bridge and have a non prismatic section to optimize the system in terms of weight and amount of reinforcement. In the regions between the girders, the precast panels have a multi-stemmed cross section whose geometry is determined by the arrangement of pre-tensioning strands for positive moments and to provide an adequate compressive zone for negative moments. Fig. 45 shows the cross section for the tested specimen. Maximum thickness of the panel is 8.1" and the minimum is 4.5". At girder locations, however, the section is uniform in order to accommodate post-tensioning strands (8.1"). The flat bottom surface provided at girder locations allows a simple grout stop to be installed between panels and girders. The void between the girders bounded by the transverse stems is formed by using adjustable void forms (Fig. 46).

Reinforcement in the transverse direction consisted of ½" indented strands. One layer of steel mesh reinforcement is provided in the upper slab for flexural performance of the slab between stems. Additional transverse reinforcement is provided at the overhangs to make up for the short development length of the pre-tensioning strands (Fig. 47).

1.3.1.2.3 Transverse joints and longitudinal post-tensioning

A female-to-female shear key was chosen at transverse joints (Fig. 48). A clear spacing of 0.4" is provided between panels for production and construction tolerances. A rapid-set non-shrink grout (Set® 45 by Master Builders Inc.) was chosen to fill the shear keys, based on a study performed by Gulyas et al. and a technical report from construction projects in Alaska (Yamane et al., 1998).

Post-tensioning tendons for this system are located approximately at mid-depth above the top flanges of girders. Details shown in Fig. 49 are for a staged post-tensioning. Block-outs are provided for anchorages and couplers at both transverse edges of panels. Post-tensioning design was based on the minimum compressive stress of 200 psi (Yamane et al., 1998).

1.3.1.2.4 Deck/girder connection

Shear connectors consisted of welded headless studs and welded threaded studs with nuts (Fig. 50). The short headless studs are welded on the top flange through grout pockets after panel erection. These connectors take only horizontal shear developed when load is applied to the composite girder. The long threaded studs are welded onto the top flanges through grout pockets in each panel to match openings in the panels. These studs resist horizontal shear and uplift. Block-outs were grouted through 1" diameter tubes. A precast concrete girder design would utilize either threaded inserts or studs grouted into the tops of girders as shear connectors (Tadros and Baishya, 1998).

1.3.1.2.5 Construction

A general procedure for deck replacement would be:

1. Remove old deck and install new shear studs before panels placement
2. Erect new precast panels
3. Adjust to grade by the use of leveling bolts and tie down by the threaded studs
4. Install longitudinal post-tensioning strands (untensioned) in ducts
5. Grout the keyways between panels with rapid set grout
6. Once the grout has cured, longitudinally post-tension
7. Grout the pockets over the top of the girders

1.3.1.2.6 Modifications

After the system was full-scale tested, few simplified details were suggested to improve panel productivity (Fig. 51). Post-tensioning ducts were moved away from the girder centerline to provide more space between them. Threaded studs were replaced by furnishing insert anchors and tie down bolts at the bottom surface of panels to tie down the panels to girders, to eliminate their field welding and facilitate deck removal (Yamane et al., 1998).

1.3.1.3 NU-Deck System

The Nudeck System was developed and tested at the University of Nebraska under NCHRP Project 12-41 'Rapid Replacement of Bridge Decks' in 1998. The system was first developed as a continuous stay-in-place precast prestressed panel system that eliminated major drawbacks of conventional stay-in-place precast panels, like need of forming the overhangs and reflective cracking at the transverse joints. In the next years the system went through some changes, the most important was that it evolved to a full-depth precast system. Post-tensioning in the longitudinal direction was also included. The new NU-Deck System was used for the first time in the construction of the Skyline Bridge in Omaha, NE, in October, 2003. Several papers have been published about this system, but none to date for the full-depth version.

The following information about the NU-Deck System is based mainly on a visit to the precast plant of Concrete Industries Inc. at Lincoln, NE, where the NU-Deck panels for the Skyline Bridge were fabricated (2003) and construction details for the Skyline Bridge (2002).

1.3.1.3.1 Specifications

Concrete used in the panels was self-consolidated with concrete strengths of 4,350 psi at transfer and 6,090 psi at 28 days. Steel strands specified for pre-tensioning and post-tensioning are uncoated, seven-wire, low relaxation, Grade 270 (A416). Initial pre-tensioning force was 30,979 lbs per strand. Specified initial post-tensioning force was

43,950 lbs per strand. Reinforcing steel was Grade 60 (ASTM A615). Spirals were high carbon spring wire (ASTM A227). All plates were hot dip galvanized and all chairs and spacers used in the fabrication were galvanized. Welded wire fabric shall conform to the requirements of ASTM A497. Strands at panel ends were removed to a depth of 1” inside the panel edge. Resulting pockets was grouted with high strength, non shrink grout.

1.3.1.3.2 Precast panels

Panels have a full length gap over the girders for shear connectors. Figs. 52 and 53 show the NU-Deck panel layout plan and a typical roadway cross section for the Skyline Bridge. The bridge is a two-span bridge with a total length of 213 ft. There are 26 typical panels and two end panels, supported by five steel girders spaced at 11.9 ft. The deck has a skew of 25º and is crowned. Each panel covers the entire width of the bridge. Anchorage of the post-tensioning strands makes the geometry for the end panels different from the typical panels. The typical panel width is approximately 7’ 9” (on the skew) and has a thickness of 6”. Fig. 54 shows a plan view of a typical panel. There is a full length gap of 1’ 1” at each beam position. It can be said that there are six blocks of concrete (four interior plus two overhangs) connected through the reinforcement.

The reinforcement in the transverse direction consists of 8 pre-tensioning strands (4 in the end panels) through the entire length of the panel. Figs. 55 and 56 show a detailed plan view with the actual shop drawings for a typical and an end panel. To maintain the gap over the girders, reinforcing bars are used in two layers. The bottom layer consists of 8 reinforcing bars through the entire length of the panel (4 for the end panels), and the top layer consists of 8 short pieces of reinforcing bars (4 for the end panels) at each beam position. At the external girders, the top bars are prolonged to the entire overhangs. Figs. 57 and 58 show a typical section and an overhang section for the typical panel, while Figs. 59 and 60 show the same for the end panels, respectively. The end panels have a double layer of welded wire fabric in the region where strands would cross the post-tensioning anchorages. Fig. 61 shows a section of the end panel at a post-tensioning duct position. At the overhangs of all panels, each two strands are confined by a spiral reinforcement to help in the development of the strands (Fig. 62). In the longitudinal direction, the panels have a single layer of reinforcement at mid-depth.

1.3.1.3.3 Transverse joints and longitudinal post-tensioning

A typical transverse joint detail for the Skyline Bridge is shown in Fig. 63 (2002). Exact dimensions of the keyway are shown in Fig. 64 (2003).

Post-tensioning tendons for this system are located above the top flanges of girders. In the Skyline Bridge each channel had 16 strands (15.3 mm diameter). Fig. 65 shows a detail of the post-tensioning channel (2002).

1.3.1.3.4 Deck/girder connection

Shear connectors used in the Skyline Bridge were 1 ¼" diameter headed studs placed in a single row. Fig. 65 shows the position of the shear connectors (2002).

1.3.1.3.5 Alignment

Closure pours at deck ends over the abutments are poured simultaneously with the concrete overlay. Fig. 66 shows the detail of the closure pour (2002).

The proposed leveling device for vertical alignment of the panels in the Skyline Bridge is shown in Fig. 67 (2002).

1.3.1.3.6 Crowned roadways

NU-Deck panels for the Skyline Bridge were crowned with an innovative technique. Panels had a hinge at the crown section. This consisted of a plastic rod over fiber board at the bottom (Fig. 68), which was crossed by the bottom strands and rebars, and a steel plate at the top, which was crossed by the top strands. Figs. 69 and 70 show the section at the hinge for both the typical and the end panel. At that section, there were also three pairs of short pieces of reinforcing steel at the top of the deck. One piece of each pair is at one side of the vertical steel plate covered with insulation in block-outs. Figs. 71 shows side views at the hinge section. Bottom reinforcement bars were covered with foam wrap and also there were small block-outs at each top strand position. Fig. 72 show the actual hinge section before concrete was poured.

The crown was formed after the concrete was poured. First, the panels were set over supports which had the final crowned shape. Fig. 73 shows a panel lifted by 8 anchor devices before being set over the supports. Second, the steel plate was removed with the top block-outs (Fig. 74). Then, all top strands were cut (Fig. 75). After the strands were cut, the panel took the desired crowned shape. Then, three steel plates were welded to each of the three top pairs or reinforcing steel to make the reinforcement continuous (Fig. 76). Finally, the block-outs were filled with a polymer modified repair mortar (Sealtight® Meadow-Crete®-GPS by W. R. Meadows). Fig. 77 shows a plan and a side view of the crown final position.

1.3.1.3.7 Parapets

Fig. 78 shows the detail for the barrier curb pedestal in NU-Deck panels for the Skyline Bridge. A mechanical bar splice was provided for barrier curb rebar attachment (2002).

1.3.1.3.8 Construction

The following sequence of construction, which includes two alternatives for casting the overlay, was shown in the plans for the Skyline Bridge:

1. The support system shall be installed on the top girder flange and adjusted so as to support the deck panels at the correct elevations.
2. The NU Deck panels shall be placed as shown in the panel layout. Care should be taken that the panels are in tight contact with the backer rod separating them and that proper alignment is achieved.
3. Post-tensioning strands shall be threaded through the post-tensioning channels and anchor system.
4. Transverse joints shall be grouted level with the tops of the deck panels, and then roughened to a 6-mm amplitude. Allow grout to attain a compressive strength of 6,000 psi before the post-tensioning.
5. Beginning at either end of the deck, tension two of the innermost strands in each post-tensioning channel.
6. Repeat step 5, tensioning from the centroid of the strand pattern outward in both horizontal directions, so as to maintain symmetry during the post-tensioning operation. Repeat until all strands at one end of the deck have been tensioned.
7. Re-tension all strands at the opposite end of the deck to the required force.
8. If post-tensioning channels are being filled at the time of overlay, ensure that all voids are filled during concrete placement (skip to step 10). If not, fill all voids in the post-tensioning channels with the prescribed material. Before the concrete sets, roughen the surface to an amplitude of 6 mm.
9. Thoroughly clean the top surface of the deck panels prior to placement of overlay, and spray them with water until they are saturated, allowing no accumulation or ponding.
10. Pour the concrete overlay.

In the construction of the Skyline Bridge, the concrete overlay was poured separately from the post-tensioning channels.

1.3.2 Steel grid deck systems

In this section, a detailed description of the Exodermic™ Deck System is presented.

1.3.2.1 Exodermic™ Deck System

An exodermic or ‘composite unfilled steel grid’ is comprised of a reinforced concrete slab on top of, and composite with, an unfilled steel grid. The basic exodermic concept, which evolved from traditional concrete-filled grids, was developed in the early 1980’s. The innovation was to move the concrete from within the grid to the top of the grid in order to make more efficient use of the two components. Putting the concrete on top also allowed the use of reinforcing steel in the slab to significantly increase the negative moment capacity of the design, and moved the neutral axis of the section close to the fabrication welds of the grid, reducing the live load stress range and the possibility of fatigue damage at the welds and punch-outs in the grid. Fig. 79 shows the original Exodermic™ Deck System design. Static and fatigue tests were conducted at both Lehigh and West Virginia Universities, and the first use was on the Garden State Parkway's Driscoll Bridge over the Raritan River as part of a reconfiguration and widening of the twin structures.

In the early to mid 1990's the Exodermic™ Deck System design was revised and the shear transfer mechanism was simplified to obtain a more efficient, more economical, and easier to install deck system (Fig. 80). Static and fatigue tests of the revised design were conducted at Clarkson University. The shear connecting mechanism was also verified by push-out and pull-out tests.

While Exodermic design is covered by US and Canadian patents, the availability of the Exodermic grid from multiple, independent, licensed suppliers allows it to be considered 'generic' in most jurisdictions (EBDI, 2003).

The following information can be found in the Exodermic Bridge Deck website (2003).

1.3.2.1.1 Specifications

Steel grid panels shall be furnished by American Bridge Manufacturing or L. B. Foster Company. The steel grid is typically ASTM A-36. Hot dip galvanizing has been specified for Exodermic decks for many years, and provides excellent protection from corrosion. Due to the 5.5 to 6.0 mils of zinc typically deposited on steel grids during hot dip galvanizing, coating life is expected to be at least 55 to 60 years. Concrete compressive strength is 4,000 psi minimum, and standard weight or lightweight concrete can be specified.

1.3.2.1.2 Panels

Exodermic decks, like all concrete filled grid decks, can be either precast or cast in place, and can span in either the transverse or the longitudinal direction of the bridge, being applicable for all type of bridges. Fig. 81 shows a typical panel layout for a girder bridge. Overall thickness of the system using standard components ranges from 6" to 9 ½". Total deck weights range from 39 to 74 pounds per square foot. Exodermic decks using standard components can span over 18 feet. Larger bearing bars and or thicker concrete slabs can be chosen to span considerably further. Exodermic grids can be fabricated according to different geometry requirements like skewed, trapezoidal, tapered, and also for crowned bridges. Fig. 82 shows the section of a crowned bridge, and Fig. 83 shows the detail for the respective cambered panels.

As the original Exodermic design was based on existing filled grid designs, horizontal shear connection between the reinforced concrete slab and the grid was provided by welding "tertiary bars" to the base grid. One inch of these tertiary bars extended up into the slab, and short vertical studs (generally #4 rebar) were welded to them (Fig. 79).

In the revised Exodermic design, the tertiary bars have been eliminated, and their function taken over by the extension of the main bars of the grid 1” directly into the slab. To aid in the engagement of the bars with the concrete, ¾” diameter holes are punched on 2” centers at the top 1” of the main bars. Horizontal shear flow is direct from the concrete slab to the main bars of the grid (Fig. 80). The revised design is simpler, less expensive and, with the elimination of the tertiary bars, contractors have more working room for installation of shear studs.

In positive bending, the concrete at the bottom in a standard reinforced concrete deck is considered ‘cracked’ and provides no practical benefit. Thus, the effective depth and stiffness of the slab is reduced, and the entire bridge has to carry the dead load of this ‘cracked’ concrete. In an Exodermic deck, essentially all of the concrete is in compression and contributes fully to the section, while the main bearing bars of the grid handle the tensile forces at the bottom of deck (Fig. 84). Exodermic decks can be substantially lighter than reinforced concrete slabs without sacrificing stiffness or strength because steel and concrete are used more efficiently.

In negative bending, a standard reinforced concrete deck handles tensile forces with the top rebar and concrete handles the compressive force at the bottom of the deck. Similarly, in an Exodermic design, the rebar in the top portion of the deck handles the tensile forces, while the compressive force is borne by the grid main bearing bars and the full-depth concrete placed over all stringers and floor beams (Fig. 85). Rebar can be selected to provide significant negative moment capacity for longer continuous spans and sizable overhangs.

In both cases, the neutral axis is located near the welds and punch-outs of the grid. This keeps the live load stress range low at these locations, generally eliminating fatigue as a limiting factor in design.

1.3.2.1.3 Joints

A typical female-to-female shear key is used between precast panels at transverse joints in bridges with deck spanning in the transverse direction, or at longitudinal joints in decks spanning in the longitudinal direction (Fig. 86). Fig. 87 shows the equivalent detail for cast-in-place construction.

Longitudinal joints are needed for transverse staged construction in girder bridges or similar. Joints are located over girders or stringers, and rebars are lapped with threaded bar connectors to provide continuity over the supporting members (Fig. 88). The same detail is used at transverse joints for decks spanning in the longitudinal direction.

1.3.2.1.4 Deck/girder connection

Exodermic decks are made composite with the steel superstructure by welding headed studs to stringers, floor beams, and main girders as appropriate, and embedding these headed studs in full-depth concrete. This area is poured at the same time as the reinforced concrete deck when the deck is cast-in-place (Fig. 89), or separately when the deck is precast (Fig. 90). Exodermic decks require no field welding other than that required for the placement (with an automatic tool) of the headed shear studs.

1.3.2.1.5 Alignment

For horizontal alignment closure pours at deck ends over the abutments are poured simultaneously with the full-depth concrete portions over the girders (Fig. 91).

A typical detail for height adjustment of the panels is shown in Fig. 92.

1.3.2.1.6 Parapets

Typical details for both cast-in-place and precast barriers are shown in Figs. 93 and 94, respectively.

1.3.2.1.7 Construction

A typical sequence of construction with precast panels would be:

1. Haunches are generally formed before placing deck panels on the bridge, using self-adhesive foam strips, galvanized sheet steel or structural angles.
2. Exodermic steel grid panels are placed and set to the required elevation using built-in leveling bolts.
3. Headed studs may be laid out and welded in position either before or after precast panels are landed.
4. Fill transverse joints between precast panels with rapid setting grout or concrete. Concrete should be properly consolidated with a 'pencil' type vibrator.
5. Rebar can be spliced between panels over supports either before or after transverse joints are filled.
6. Fill block-outs full depth and closure pours with rapid setting concrete. The use of 3/8" maximum coarse aggregate is recommended. Concrete should be properly consolidated with a 'pencil' type vibrator.

For cast-in-place construction, the steel grid panels act as stay-in-place forms, and little or no additional formwork is required. Concrete fills the haunch areas at the same time the finished riding surface is poured.

1.3.3 Other composite deck systems

In this section, a detailed description of two different composite systems is presented. One is a full-depth precast deck system, the Effideck™ System, and the other is a precast, pre-compressed, composite superstructure, the Inverset™ Bridge System. Both systems have a proprietary nature.

1.3.3.1 Effideck™ System

Effideck™ System is proprietary to The Fort Miller Co., Inc. and was developed in the second part of the 90's. Effideck is a precast deck system consisting of modular deck panels using a relatively thin concrete slab (typically 5" thick) supported by closely spaced hollow structural section (HSS) steel tubes bonded to the underside of the slab. Effideck panels can span either in the transverse direction between bridge stringers or in the longitudinal direction between floor beams.

The effectiveness of a 5" concrete deck, supported by closely spaced steel structural members was established by testing performed at Cornell University and the University of Oklahoma. Fatigue and composite beam tests were completed at the University of Nebraska (TFMC, 1998).

The following information is based mainly on a design assistance report (Ryan Biggs Associates, 1999), Effideck brochures (TFMC, 1998 and 2002) and Effideck panels shop drawings for two bridges (TFMC, 2000 and 2001).

1.3.3.1.1 Specifications

Specified concrete compressive strength is typically 5,000 psi, but higher strengths can be used. Tubes (HSS) are typically ASTM A500 Grade B steel. For most applications, the tubes are hot-dip galvanized. Shear studs are welded to the tubes prior to galvanizing. All reinforcing steel is conventional Grade 60 (ASTM A615) and should be either galvanized or epoxy coated. Galvanized bars are recommended over epoxy coated bars, and they are also more compatible with the galvanized steel tubes and studs.

1.3.3.1.2 Precast panels

In transverse applications, panel width is typically 10', but can vary depending on the requirements of each specific project. Panel length depends on the width of the bridge. In case of staged construction or large bridge widths, two or more panels can be used in a single bridge transverse section with longitudinal joints between adjacent panels. Fig. 95 shows the deck plan view, Fig. 96 shows the bridge cross section and Fig. 97 shows the deck during construction for a 49' long one-span bridge with transverse panels covering the entire width of the bridge (Little River Town, Stowe, VT, 2000).

In longitudinal applications, panel length depends on floor beam spacing and panel width depends on the width of the bridge. Fig. 99 shows part of the deck plan view and Fig. 98 shows the deck during replacement for a 151' long one-span bridge with three two-span longitudinal panels covering the width of the bridge and spanning continuously over three floor beams (Cascadilla Creek, Ithaca, NY, 2001).

Panel thickness varies depending on the project and is equal to the depth of the concrete slab plus the depth of the steel tubes. Slab thickness should not be less than 5”, and tube depth is typically between 2” and 6”. Tube width is typically 6” for edge tubes and 3” for interior tubes. Edge tubes are wider than interior tubes to provide additional strength and stiffness at the panel edges, and also to accommodate the panel-to-panel connections at transverse joints. Tube wall thicknesses are typically ¼”, but can be thicker or even thinner if stringer spacing is small, and if thickness is compatible with the diameter of the shear studs that are welded to the tubes.

Portions of the slab thickness are increased to full-depth over bridge stringers and at overhangs, for transverse installations, and over floor beams, for longitudinal installations, so that the bottom of the slab is flush with the bottom of the tubes (Fig. 100, section B). These full-depth portions increase the transverse bending strength and stiffen the panels during shipping and handling. The transition from the standard depth slab to the full-depth slab is provided with a 2 (horizontal) on 1 (vertical) slope to avoid stress concentrations (Ryan Biggs Associates, 1999).

For bridges on a skew, the panels (including the steel tubes) can be fabricated either perpendicular to the stringers, or parallel to the skew. For skews less than 35 degrees, it is recommended to fabricate the Effideck panels parallel to the skew. For skews greater than 35 degrees, it is recommended to fabricate them perpendicular to the bridge stringers and also use special tapered panels at the bridge ends (Ryan Biggs Associates, 1999).

For bridges with flared stringer spacings, or bridges that are horizontally curved, panels can be either of a uniform width or tapered, depending on the specific application.

Internal composite behavior between the concrete slab and the tubes is accomplished using headed studs. The studs are welded to the top of the tubes (Fig. 101) and to the sides of the tubes at the full-depth portions (Fig. 102). Shear studs typically have a diameter of ¾” or ⅞”. As a general rule, stud diameter should not exceed 3 times the thickness of the tube to ensure that ultimate failure occurs in the stud and not in the base metal. However, size and spacing of tubes is typically governed by fatigue criteria not ultimate strength, therefore bridges with low traffic volumes can often use smaller and fewer studs (Ryan Biggs Associates, 1999).

The reinforcing is placed in a single layer to provide sufficient cover. The top set of bars in the layer runs parallel to the tubes, providing strength for negative bending moment, while the bottom set of bars runs perpendicular to the tubes, helping resist punching shear and distribute loads, and providing strength during shipping and handling. In full-depth portions, an auxiliary bottom layer of reinforcing bars is typically placed (Fig. 103).

Tests on panels with 30” clear distance between tubes, 5” of concrete, and number 4 bars spaced 6” on center have demonstrated that this design is sufficient for HS 25 loading (Ryan Biggs Associates, 1999).

1.3.3.1.3 Joints

Joints perpendicular to the supporting members between adjacent panels have a full-length shear key (Fig. 104) and intermittent joint pockets. Joint pockets are spaced approximately one at every interior bay (Fig. 95) and one in each overhang. However, if the stringer spacing exceeds approximately 11', two pockets per bay may be preferred (Fig. 99). If the overhang length is approximately less than 18", the joint pocket in the overhang may not be necessary. After two adjacent panels are placed on the stringers, threaded studs are welded to the exposed tops of the steel tubes at each pocket and steel channels are installed in each pocket to fasten adjacent panels together. These clamp-down channels are set down into the pockets, and each is just long enough so that each end of it rests on the top edge of the two steel tubes exposed in the pocket. The width of the grout pockets is slightly more than clear distance between steel tubes (Figs. 105 and 106). The pockets and the shear key are filled with non-shrink grout.

Joints parallel to the supporting members should be located directly over them. There are two options for treating this joints which might occur between two panels or one panel and an existing concrete deck (Ryan Biggs Associates, 1999).

One option is to provide a grouted butt joint with no continuity of reinforcing across the joint. Elastomeric concrete is often used at the top of this joint. This is the option most applicable to joints between Effideck™ and existing decks. A second option is to provide continuity using lapped or mechanically spliced reinforcing, with an appropriate length block-out at the panel edge for making the splice.

Full-depth concrete should always be used for the panels in the vicinity of the joint. The space between the underside of the panel and the top of the stringer should always be fully grouted for the full width and length of the flange wherever a longitudinal joint occurs. Figs. 107 and 108 show the plan view and the cross section of a typical longitudinal joint, Fig. 109 shows a typical rebar splice detail and Fig. 110 shows an actual picture of the joint for the bridge in Ithaca, NY.

1.3.3.1.4 Deck/girder connection

Shear connectors consist of headed studs and typically two threaded studs that are attached directly to the stringer or floor beams from the top of the deck through pockets. Figs. 111 and 112 show the plan view and a section of the shear pocket and Fig. 113 a picture of two pockets at each side of a joint. The pockets are then filled with non-shrink grout.

1.3.3.1.5 Alignment

A typical leveling device used for vertical alignment of the panels is shown in Fig. 114.

1.3.3.1.6 Construction

A typical sequence of construction is:

1. Foam sealant is typically attached to the top edge of the supporting member prior to placing a panel. The weight of the panel slightly compresses the sealant to form a seal that prevents grout from escaping out.
2. Panels shall be lifted using appropriate lifting hardware to fit lifting inserts detailed on the shop drawings. Place back-walls before setting respective panels, place panels and adjust them to proper vertical position using adjusting bolts.
3. Install steel shims between the top of the supporting member and the bottom of the tubes. The shims are installed at every tube so that no gaps exist (Fig. 115). This is essential to prevent cracking of the panels during construction, particularly if the in-place panels are driven on prior to grouting.
4. Remove all adjusting bolts.
5. Connect adjacent panels as follows: add shims as required to align transverse joint pockets. As necessary, add variable thickness shims between C7 channels and tubes at transverse joint pockets. Install threaded studs, using C7 channel as a template to locate studs. Install C7 channel, washers and nuts in transverse joint pockets.
6. Install studs and hardware in pockets over stringers as follows: install threaded studs using C3 channel as template. Weld headed studs to the top stringer flange through each pocket. Install C3 channels, washers and nuts in pockets. (The panels are fastened down at each ground pocket with steel channels. A piece of steel channel is then fastened down with nuts screwed onto the threaded studs.)
7. Fill transverse joints and panel to panel pockets with non-shrink grout.
8. Fill all remaining pockets and all spaces between top of stringer and bottom of panels with non-shrink grout (which encapsulates the headed studs, threaded studs, and clamp-down channels).

1.3.3.2 Inverset™ Bridge System

The Inverset™ Bridge System was developed in the early 1980's by Stanley Grossman, P.E., of Norman, OK, who presently holds a patent on the system. The system is defined as a precast, pre-compressed, composite superstructure made up of steel beams and a concrete slab, which act as a composite unit. The beams are prestressed using a unique upside-down casting technique that uses the force of gravity to prestress steel beams. When the finished units are turned right side up, composite action occurs immediately. The composite section supports the superstructure as well as the live load and any superimposed dead loads placed upon the structure. If this phenomenon is fully utilized, lighter or shallower steel beams than those used with conventional cast-in-place methods may be used. Inverset units can be used either in the longitudinal or the transverse direction of the bridge.

The validity of this system was originally established by extensive testing at the University of Oklahoma between 1982 and 1985. Inverset units were studied under sustained load, repeated load and static loading to failure. The Inverset™ Bridge System has been used in more than 145 bridges in over 12 states since the first bridge was installed in Cleveland County in Norman, OK, in 1982.

The following information is based mainly on a manual for the Inverset Bridge™ System (TFMC, 1999).

1.3.3.2.1 Specifications

The concrete compressive strength is typically 5,000 psi. The reinforcing steel may be plain, epoxy coated, or galvanized. Steel used in the beams can be A709 Grade 50W weathering steel. For other steels like A36 or A572, where painting is required, it is applied under shop-controlled conditions, prior to shipment to the job site.

1.3.3.2.2 The Inverset™ casting method

In this method, a fabricated beam pair, like shown in Fig. 116, is inverted and supported as shown schematically in Fig. 117. Cross-members spaced at regular intervals on top of the beams, support matching cross-members below the beams that are form supports. This arrangement transfers the weight of the form, the concrete and the reinforcement uniformly to the beams. This uniformly distributed weight is the gravity prestressing force.

The key to the Inverset casting method is control of the deflection of the beams. Fig. 117 shows a deflection control, or "deflection stop", under the beams at midspan. The stop limits the total deflection of the beam which is directly related to the prestress in the beam.

To further illustrate the concept of the Inverset casting method, a stress diagram of a beam, as it sits in the form, upside-down, is shown in Fig. 118. The diagram shows compression in the top flange and tension in the bottom flange, as expected. When concrete is placed in the form, these stresses increase linearly until they reach the predetermined prestress level. The concrete hardens and cures while the beam is maintained at this deflection and stress. At this point, the concrete and the beam become a composite unit.

Once the concrete deck has reached design strength, the entire composite unit is turned right side up. In this position, the resultant stress configuration is shown in Fig. 119. The compression stress in the bottom flange (top flange in form) reverses to near zero and the tension in the top flange (bottom in form), stays about the same, since it is near the neutral axis of the new composite section. The concrete deck is in compression, as shown in Fig. 119, under its own dead load.

It is important to note that the inverted beams are positioned in the casting bed at elevations that mirror the right-side-up bearing elevations of the beams in their final position in the field. In this way, Inverset units are fabricated for specific locations on the bridge and will meet the superelevation and profile requirements for each location.

1.3.3.2.3 Inverset™ units

The uses of Inverset units are broadly categorized as longitudinal or transverse. Used as longitudinal units, they are designed and erected as the main superstructure elements (Fig. 120). Used transversely, Inverset units are erected as a transverse deck/floor-beam system, supported by some other superstructure element such as deck trusses, deck girders or through trusses (Fig. 121).

Longitudinal applications include simple span elements, continuous over one or more supports or erected as simple span and made continuous over one or more supports. Supports can be piers, abutments or floor beams.

Transverse applications are particularly useful in deck/floor-beam replacements where the existing deck girders or deck trusses are in good condition. It is also useful in new construction where it is necessary to design spans longer than the maximum allowable span for longitudinal Inverset (approximately 100'). In this case, appropriately sized plate girders are designed and erected as the primary supporting members and transverse Inverset units are made composite or non-composite with them.

The majority of Inverset bridges built to date have been single, simple spans. Most of these have been replacement bridges where new Inverset units were placed on existing abutments.

A cross-section of a typical longitudinal unit is shown in Fig. 122. Two beams are framed together with shop-installed diaphragms and with pre-drilled diaphragm connection plates for diaphragms that are later installed in the field. The deck overhang is typically 18" or as required by the project requirements. Units are cambered during the casting operation as required by their location on the bridge. Each unit is fabricated with a discrete camber, so that it matches adjacent units and meets the required vertical alignment. Commonly available shapes and cross-sections are shown in Figs. 123 and 124.

Inverset units are shipped individually to the job site and erected side by side as shown in Fig. 125. After erection of all units, each one is adjusted to the correct vertical and horizontal position and diaphragms are installed to make the steel frame continuous. Since the concrete deck is not continuous, the function of transferring load from unit to unit is taken by the short, stiff and efficient field-installed diaphragms. Grid analysis and full-scale load testing prove that the load is, in fact, transferred in this manner and that the deck discontinuity is not detrimental to effective load distribution. Reduction in the length of these diaphragms further enhances their performance.

A cross-section of a typical transverse unit is similar to a longitudinal unit, except that the typical 18" slab overhang is reduced (Fig. 126). The overhang must be sufficient, however, to allow enough room for field erection crews to install the connecting diaphragms.

Transverse units must be fabricated to meet the required cross slope. In case of crowned roadways, the top of the slab is cast at the required crown with constant thickness, except where it is haunched down to the beam (Fig. 127). The floor beams are level and are bolted to the main supporting member in a conventional manner.

Where the bridge deck is superelevated, constant-depth transverse units are used as shown in Fig. 128. Beveled sole plates are attached to the bottom of the Inverset beams to create a level bearing surface. To make up differences in main structural beam cambers and tolerances, and in depth of the units, steel shims are installed between the beveled sole plate and the top of the structural beam.

Due to fabrication and shipping considerations, the practicable limit on Inverset unit length is approximately 100'. Shipping considerations usually limit the unit width to 12'. Normally, each bridge is divided into a minimum number of equal width panels less than 12' wide. Different width units may be necessary for staged construction. Standard deck thickness is 7 ½".

Two layers of reinforcing steel are used to reinforce the deck (Fig. 122). The cover over the top layer of steel is 2 ½" and the cover below the bottom mat is 1".

1.3.3.2.4 Joints

A longitudinal section of a typical single simple-span bridge is shown in Fig. 129. Units may be fabricated with a "plain" end, as shown on the "A" end, designed to fit between two conventional backwalls or units may be fabricated with an integral backwall, as shown on the "B" end, which can also serve as a support for an approach slab. Bearings are made of standard materials and are designed to accommodate expansion or fixation, as required.

A longitudinal section of a typical multiple span bridge is shown in Fig. 130. These bridges are built by erecting simple-span units, making them continuous after erection as required. Figs. 131 and 132 show joint details for simple spans. Fig. 133 shows a joint detail for making the spans continuous over the pier. In any case, the bearing design must be consistent with the joint and fixation requirements.

A single longitudinal unit can span continuously over one or more supports. In this case, the compression in the deck created by the casting process reduces the chances of cracking over the pier by offsetting the tensile forces in the concrete due to service loads.

Longitudinal joints for longitudinal units are similar than transverse joints for transverse units. When units are placed adjacent to each other in the field, edges of the deck slabs, which are ‘keyed’, form a double-grooved keyway which is filled with non-shrink grout and elastic sealing material (Fig. 134).

1.3.3.2.5 Connectors

The concrete deck is made composite with the steel girders by stud shear connectors used in the same manner as conventional cast-in-place decks (Fig. 116).

1.3.3.2.6 Construction

A general procedure for deck replacement would be:

1. An accurate survey of the bridge bearings should be performed prior to installation of the Inverset™ units.
2. Inverset™ units are set on the bearings. The first unit is set as close as practical to its position. The rest of the units are set accordingly but with consideration given to how the units line up at joints, at fascia lines, and relative to each other.
3. After all the units are in place, the vertical match between the edges of each unit is checked to ensure a smooth riding surface. If edges do not match, shims are inserted between the bottom of the beam and the top of the bearing to provide a proper match.
4. After all adjustments have been completed, field diaphragms are installed.
5. At joints, foam backer rods are installed first, and then non shrink grout is placed and cured.

CHAPTER 2
IDENTIFICATION OF ISSUES AND NEEDS

2.0 Introduction

The objective of this chapter is to identify issues and needs for the implementation of full-depth precast deck systems. The chapter is divided in three sections. First, issues for implementation of full-depth precast deck systems are identified. Next, issues are identified with each full-depth precast deck system to elucidate possible solutions. Deck systems limitations and needs are also identified. Finally, a summary of these needs is included in this chapter.

2.1 Identification of issues

In this section, issues identified with the implementation of full-depth precast deck systems are classified in three groups: use, behavior/design and construction.

2.1.1 Use

2.1.1.1 Project type

The two generic types of project under this title are construction of decks for new bridges and the more common replacement of existing decks.

2.1.1.2 Bridge type

An important goal would be to have full-depth precast deck systems suitable to all different types of bridges. In general, from the standpoint of deck behavior, almost all bridges can be classified in two groups: bridges with decks spanning in the transverse direction, like girder bridges or bridges with stringers, and bridges with decks spanning in the longitudinal direction between floor beams.

2.1.1.3 Girder material

An important issue is the girder/beam material. While bridge full-depth deck panels have been used over steel girders, they have never been used over concrete girders. This issue has to deal with an appropriate type of connection between the deck and the girder. For cast-in-place concrete girders and precast, prestressed concrete girders, the appropriate connection design remains a big issue.

2.1.1.4 Deck geometry

Issues identified under this title include the use of bridge deck systems for crowned bridges, skewed bridges and curved bridges. In general, precast systems can be fabricated to fit different geometries. However, in the case of the full-depth precast, prestressed concrete deck systems, transverse pre-tensioning and/or longitudinal post-tensioning must be employed.

2.1.2 Behavior/design

2.1.2.1 Deck behavior

Transverse joints between adjacent panels in girder bridges are located perpendicular to the girders and make the deck not continuous in the longitudinal direction (deck's secondary direction). In decks supported by floor beams, longitudinal joints are placed in a similar situation making the deck not continuous in the transverse direction (deck's secondary direction). Joints are important because bridge deck performance is manifested in their behavior. Problems associated with joints include cracking, leaking and deterioration. Configuration of the transverse shear key, the type of filler material and the inclusion of post-tensioning are important issues.

Many deck systems for girder bridges are full-width and, therefore, continuous in the transverse direction (deck's primary direction). In some cases, however, a longitudinal joint is necessary for wider bridges or to make staged construction viable. In case of bridges with floor beam systems, however, since no panel can cover the entire length of the bridge, transverse joints are always present, and continuity in the main direction is hard to achieve. In this case, a detail for deck continuity is an important issue.

2.1.2.2 Girder behavior for positive moments

From the standpoint of design, the deck panel forms part of the compression flange of the underlying girder and it is desirable to achieve composite behavior between the deck and girder. This can be done with connection systems capable of developing horizontal shear between the two elements. Some cases where the performance of a bridge deck system has not been good were attributed to the type of connection between the deck and the supporting system. Main issues for deck/girder connections are type of connectors for both steel and concrete supporting elements, and type of grout to make the connection composite.

2.1.2.3 Girder behavior for negative moments

In this case, the deck is part of the girder tension flange, and lack of continuity of longitudinal reinforcement through transverse joints prevents composite behavior for negative moments between the deck and the girder. However, due to the presence of shear connectors, some amount of composite behavior can be developed, resulting in undesirable tensile stresses at the transverse joints. The use of post-tensioning in the longitudinal direction to achieve composite behavior is a key factor for design.

2.1.3 Construction

2.1.3.1 Staged construction

In deck replacement projects staged construction is a very important issue. Some bridges can not afford full closure with detours, and a solution to minimize impact to the public is a partial closure of the bridge during off-peak hours, reducing the number of traffic lanes and replacing the deck by stages.

2.1.3.2 Speed of construction

Speed of construction is a major issue in deck replacement projects. Reducing construction time in deck replacement projects helps reduce the impact to the user from the standpoint of travel times and traffic safety during construction. It is desirable that construction time be reduced to short periods with low traffic, where full or partial closure of the bridge is acceptable.

2.2 Identification of needs

Table 1 shows the nine full-depth precast deck systems identified in the Literature Review Chapter. Herein, issues identified in the section 2.1 will be analyzed for each full-depth precast deck system in order to identify alternate solutions. Full-depth precast deck systems limitations and needs will be identified.

2.2.1 Use

2.2.1.1 Project type

Table 2 shows that all these deck systems can be used for new bridges and deck replacement projects, except the Inverset™ Bridge System, which can not be used in deck replacement jobs. However, when the whole superstructure needs to be replaced, the Inverset™ Bridge System, as well as all the other systems, is available for use.

2.2.1.2 Bridge type

Table 2 shows that all these systems can be used for bridges with deck spanning in the transverse direction, like girder bridges. However, for bridges with deck spanning in the longitudinal direction over floor beams, precast, prestressed concrete deck systems, which have post-tensioning in that direction, can not be used. These systems need a different approach for this type of bridges.

2.2.1.3 Girder material

All these systems have been used (or at least tested) over steel girders, but none of them has been used over reinforced or prestressed concrete girders. The New England Region

System has connection details for concrete girders, but there is some concern about their use. In the last year, research on precast girder-cast-in-place deck connections (Tadros et al., 2002) and precast girder-precast deck connections (Menkulasi, 2002) has been carried out. The first one could be suitable for the NU-Deck System, but has not been implemented yet. The last one would be appropriate for the New England Region System, but its results are not in the public domain as of yet.

2.2.1.4 Deck geometry

In the New England Region System (NER System), panels can be precast with the skew or in a trapezoidal shape for curved bridges. For crowned bridges, however, the change in slope has been made through longitudinal cast-in-place concrete that requires forming. In the NCHRP System, its stemmed non prismatic section makes it impractical to use it for skewed, curved or crowned bridges. In the case of the NU-Deck System, panels for the Skyline Bridge were precast with a skew of 25º, and the crown was formed in plant with an innovative technique. This system, however, has not been projected for curved bridges.

While the NU-Deck System can adopt the solution of the NER System for curved bridges, it is also possible that the NER System adopts the innovative technique of the NU-Deck System for crowned bridges. This last situation would be a considerable improvement for the NER System, because forming, casting and curing of the longitudinal cast-in-place pouring could be avoided for crowned bridges.

The use of these precast concrete deck systems, with pre-tensioning and post-tensioning, in skewed, curved and crowned bridges will require the implementation of some limits, especially on the maximum angle of skew and radius of curvature in the horizontal plane. These limits are not available as of yet.

2.2.2 Behavior/design

2.2.2.1 Deck behavior

All the full-depth systems described consider female to female shear keys at the transverse joints with at least ¼” opening at the bottom to allow for panel size irregularities. This shear key has been successfully used in many projects. To fill the shear key, Set® Grout is easy to apply and, in a recently concluded research (Issa et al., 2003), has performed better than the commonly used Set® 45. In cases where traffic interruption is critical, a more expensive high early strength polymer concrete should be considered.

Another important issue to consider in the longitudinal direction is the inclusion of post-tensioning. Post-tensioning keeps joints in compression and prevents any leakage. The NER System has longitudinal post-tensioning distributed in the slabs. Analysis performed for this type of system found that a minimum prestress level of 200 psi is needed for simply supported bridges, while 450 psi is needed over the interior supports for continuous bridges to keep transverse joints in compression (Issa et al., 1998). The NCHRP System and the NU-Deck System have post-tensioning over the girders and that also helps keeping joints in compression. However, there is no analytical study to recommend a minimum prestress level to keep transverse joints in compression.

Table 3 shows the behavior of the full-depth precast deck systems in the transverse direction for girder bridges and similar. All systems can be continuous for both positive and negative moments, except the Inverset™ Bridge System, whose typical unit with two girders has the deck simply supported between them. In cases where a longitudinal joint is required, the Effideck™ System and the Exodermic™ Deck System can maintain continuity through rebar connectors, while the rest of the grid decks need the grids to be welded or bolted. The NER System needs a longitudinal cast-in-place pouring to maintain continuity. Table 4 shows the same information as Table 3, but for bridges with deck spanning in the longitudinal direction. In this type of bridges, precast, prestressed concrete deck systems are not applicable.

2.2.2.2 Girder behavior

Table 5 shows information about connections between decks and steel girders. Post-tensioning information is also included. Most of the systems use headed studs as shear connectors, except the NCHRP System, which uses headless studs. The Effideck™ System uses also threaded studs. Most of the systems have the whole length of the girder to place the studs, except the NER System, the NCHRP System and the Effideck™ System, that have pockets to place the connectors. Finally, only the precast, prestressed concrete decks have post-tensioning.

All these systems, except the open grid, can achieve composite behavior with the girders in the positive moment region. However, for negative moment regions (near piers or supports), only the systems that have post-tensioning can be considered to achieve composite behavior. In the case of the NER System, where the post-tensioning is mainly applied to the slabs, this may not be true. Table 6 shows basically the same information as Table 5, but for bridges with deck spanning in the longitudinal direction. In this type of bridges, precast, prestressed concrete deck systems are not applicable.

2.2.3 Construction

2.2.3.1 Staged construction

All systems except the NCHRP System and the NU-Deck System have details for transverse staged construction. In other words, these two systems need a full closure of the bridge for deck replacement projects (Tables 7 and 8). In the case of the NU-Deck System, however, a detail similar to that of the NER System could be used. The NER System has also a detail for longitudinal staged construction. This might be useful when a long span can not be entirely replaced in a short period of time (weekend) and the bridge has to be reopened until the next closure period.

2.2.3.2 Speed of construction

The Effideck™ System, the Inverset™ Bridge System, and all the steel grid deck systems (precast applications) can be used in projects that require night-time construction only (Tables 7 and 8). Precast, prestressed concrete systems, however, due to post-tensioning, are not as fast to construct. Replacement projects with the NER System have been done with weekend construction only (Stark et al., 1994).

2.3 Summary

Based on the review conducted of issues and needs regarding the possible implementation of full-depth precast concrete bridge deck panels, the following key items remain yet to be addressed:

- Use of full-depth precast deck systems over concrete or precast concrete girders
- Full-depth precast, prestressed deck systems for bridges with deck spanning in the longitudinal direction remain unavailable
- Establish practical limits for the skew angle and curvature radius for the use of full-depth precast, prestressed deck systems in skewed and curved bridges
- Determine the minimum amount of post-tensioning necessary in the NU-Deck System to maintain the transverse joints in compression

Table 1

Full-Depth Bridge Deck Panel Systems
Clasification

	Reinforced Concrete Slab	Width	Overhangs Forming
Precast, Prestressed Concrete Decks			
New England Region System	Precast	Full or Partial	No need
NCHRP System	Precast	Full	No need
NU-Deck System	Precast	Full	No need
Steel Grid Decks			
Open Grid Systems	Does not have	Full or Partial	No need
Full-Depth Concrete Filled Grid Systems	Precast or Cast-in-place	Full or Partial	No need
Half-Depth Concrete Filled Grid Systems	Precast or Cast-in-place	Full or Partial	No need
Exodermic Deck System	Precast or Cast-in-place	Full or Partial	No need
Other Composite Systems			
Effideck System	Precast	Full or Partial	No need
Inverset Bridge System*	Precast	Full or Partial	No need

* Superstructure System (include beams)

Table 2

Full-Depth Bridge Deck Panel Systems
Applications

	Type of Project		Type of Bridge		Girder Material		Geometry of Deck			
	New	Replacement	Transverse	Longitudinal	Steel	Concrete	Non-Skewed	Skewed	Curved	Crowned
Precast, Prestressed Concrete Decks										
New England Region System	A	A	A	N.A.	A	Not used yet	A	A	A	A
NCHRP System	A	A	A	N.A.	A	Not used yet	A	N.A.	N.A.	N.A.
NU-Deck System	A	A	A	N.A.	A	Not used yet	A	A	Not used yet	A
Steel Grid Decks										
Open Grid Systems	A	A	A	A	A	N.A.	A	A	A	N.A.
Full-Depth Concrete Filled Grid Systems	A	A	A	A	A	Not used yet	A	A	A	A
Half-Depth Concrete Filled Grid Systems	A	A	A	A	A	Not used yet	A	A	A	A
Exodermic Deck System	A	A	A	A	A	Not used yet	A	A	A	A
Other Composite Systems										
Effideck System	A	A	A	A	A	N.A.	A	A	A	A
Inverset Bridge System*	A	N.A.	A	A	A	N.A.	A	A	A	A

* Superstructure System (include beams)

A = Applicable
N.A. = Not applicable

Table 3

Full-Depth Bridge Deck Panel Systems
Decks Spanning in the Transverse Direction
Deck Behavior

	Transverse Reinforcement		Behavior	
	Top	Bottom	(+) Moment	(-) Moment
Precast, Prestressed Concrete Decks				
New England Region System	Rebar & Pretensioning Strands		C	C
NCHRP System	Pretensioning Strands		C	C
NU-Deck System	Rebar &/or Pretensioning Strands		C	C
Steel Grid Decks				
Open Grid Systems	Grid		C	C
Full-Depth Concrete Filled Grid Systems	Grid		C	C
Half-Depth Concrete Filled Grid Systems	Grid		C	C
Exodermic Deck System	Rebar	Grid	C	C
Other Composite Systems				
Effideck System	Rebar	Steel Tubes	C	C
Inverset Bridge System*	Rebar	Rebar	S.S.	S.S.

* Superstructure System (include beams)

C = Continuous
S.S. = Simply supported

Table 4

Full-Depth Bridge Deck Panel Systems
Decks Spanning in the Longitudinal Direction
Deck Behavior

	Longitudinal Reinforcement		Behavior	
	Top	Bottom	(+) Moment	(-) Moment
Steel Grid Decks				
Open Grid Systems	Grid		C	C
Full-Depth Concrete Filled Grid Systems	Grid		C	C
Half-Depth Concrete Filled Grid Systems	Grid		C	C
Exodermic Deck System	Rebar	Grid	C	C
Other Composite Systems				
Effideck System	Rebar	Steel Tubes	C	C
Inverset Bridge System*	Rebar	Rebar	S.S.	S.S.

* Superstructure System (include beams)

C = Continuous
S.S. = Simply supported

Table 5

Full-Depth Bridge Deck Panel Systems
Decks Spanning in the Transverse Direction
Girder Behavior

	Panel-Girder Connection		Longitudinal Post-Tensioning	Behavior	
	Connectors	Position	Position	(+) Moment	(-) Moment
Precast, Prestressed Concrete Decks					
New England Region System	Headed studs	Blockouts	Mid-depth between girders	C	C
NCHRP System	Headless studs	Blockouts	Mid-depth over girders	C	C
NU-Deck System	Headed studs	Open channel	Mid-depth over girders	C	C
Steel Grid Decks					
Open Grid Systems	Welds	Over girder	N.A.	N.C.	N.C.
Full-Depth Concrete Filled Grid Systems	Headed studs	Open channel	N.A.	C	N.C.
Half-Depth Concrete Filled Grid Systems	Headed studs	Open channel	N.A.	C	N.C.
Exodermic Deck System	Headed studs	Open channel	N.A.	C	N.C.
Other Composite Systems					
Effideck System	Headed & threaded studs	Pockets	N.A.	C	N.C.
Inverset Bridge System*	Headed studs	Over girder	N.A.	C	N.C.

* Superstructure System (include beams)

N.A. = Not applicable
C = Composite
N.C. = Non composite

Table 6

Full-Depth Bridge Deck Panel Systems
Decks Spanning in the Longitudinal Direction
Girder Behavior

	Panel-Girder Connection		Behavior	
	Connectors	Position	(+) Moment	(-) Moment
Steel Grid Decks				
Open Grid Systems	Welds	Over girder	N.C.	N.C.
Full-Depth Concrete Filled Grid Systems	Headed studs	Open channel	C	N.C.
Half-Depth Concrete Filled Grid Systems	Headed studs	Open channel	C	N.C.
Exodermic Deck System	Headed studs	Open channel	C	N.C.
Other Composite Systems				
Effideck System	Headed & threaded studs	Pockets	C	N.C.
Inverset Bridge System*	Headed studs	Over girder	C	N.C.

* Superstructure System (include beams)

N.A. = Not applicable
C = Composite
N.C. = Non composite

Table 7

Full-Depth Bridge Deck Panel Systems
Decks Spanning in the Transverse Direction
Construction

	Speed of Construction		Staged Construction	
	Overnight	Weekend	Transverse	Longitudinal
Precast, Prestressed Concrete Decks				
New England Region System	N.A.	A	A	A
NCHRP System	N.A.	A	N.A.	N.A.
NU-Deck System	N.A.	A	Not used yet	N.A.
Steel Grid Decks				
Open Grid Systems	A	A	A	N.N.
Full-Depth Concrete Filled Grid Systems	A	A	A	N.N.
Half-Depth Concrete Filled Grid Systems	A	A	A	N.N.
Exodermic Deck System	A	A	A	N.N.
Other Composite Systems				
Effideck System	A	A	A	N.N.
Inverset Bridge System*	A	A	A	N.N.

* Superstructure System (include beams)

A = Applicable
N.A. = Not applicable
N.N. = No need

Table 8

Full-Depth Bridge Deck Panel Systems
Decks Spanning in the Longitudinal Direction
Construction

	Speed of Construction		Staged Construction
	Overnight	Weekend	Transverse
Steel Grid Decks			
Open Grid Systems	A	A	A
Full-Depth Concrete Filled Grid Systems	A	A	A
Half-Depth Concrete Filled Grid Systems	A	A	A
Exodermic Deck System	A	A	A
Other Composite Systems			
Effideck System	A	A	A
Inverset Bridge System*	A	A	A

* Superstructure System (include beams)

A = Applicable

CHAPTER 3
COMPARISON OF SYSTEMS

3.0 Introduction

The objective of this chapter is to provide an analysis of the advantages and disadvantages for the use of full-depth precast deck systems. The chapter has two sections. In the first section, a summary of the advantages and disadvantages of each full-depth precast deck system is presented. In the last section, full-depth precast deck systems are compared from the standpoint of design, fabrication, installation and final in-service performance. Information about costs is also included.

3.1 Advantages and disadvantages of full-depth precast deck systems

In this section, the advantages and disadvantages of each full-depth precast deck system are summarized.

In general, the use of full-depth precast deck systems has the following advantages: increased quality, lower life-cycle costs, traffic impacts are minimized, construction work-zone safety is improved and construction is less disruptive to the environment. These advantages are common to all systems and are not listed again for each system.

Limitations in the use of the different systems, which were analyzed in the preceding chapter, are summarized in Tables 9 and 10. Table 9 shows the applicability of the different full-depth precast deck systems in bridges with deck spanning in the transverse direction considering different geometries and different construction timelines. Table 10, for bridges with deck spanning in the longitudinal direction, is much simpler because the systems applicable for this type of decks can be used for all geometries and with all construction timelines.

3.1.1 New England Region System

Based on the literature review and the performance of projects where the system was used, the following advantages and disadvantages can be identified:

Advantages

- All the materials used in the production of the panel are non-proprietary and ready available

- The use of pre-tensioning for transverse flexure helps reducing dead load by decreasing the panel weight

- Due to pre-tensioning, less care is needed in shipping and handling of panels before placement

- The use of post-tensioning maintains transverse joints in compression and assures continuity in the direction of post-tensioning

- Active prestressing in both directions results in superior performance compared to conventionally reinforced decks

- Ten years of experience with its use indicates very good results and research is still being carried out to improve its performance

Disadvantages

- Slow installation in comparison with other full-depth systems that do not have post-tensioning

3.1.2 NCHRP System

After this system was tested, researchers participating in its development concluded that the implementation of this system had the following advantages:

Advantages

- All the materials used in the production of the panel are non-proprietary and ready available

- Is cost competitive with other concrete panel systems yet it is 10 to 30% lighter in weight

- Panels can be rapidly produced, constructed and removed

- The two-way prestressing resulted in controlled cracking

- Grouted post-tensioned transverse joints between precast panels exhibit satisfactory performance under service load and fatigue load

- The system has very large flexural capacity and the potential for reducing flexural reinforcement (punching shear was the mode of failure in the test)

Disadvantages

- After five years of being developed, the system has not been used in an actual bridge. This may be because the system can only be used for non crowned, non-skewed, non-curved, steel girder bridges, or because the fabrication of the panels is considered difficult

- Slow installation in comparison with other full-depth systems that do not have post-tensioning

3.1.3 NU-Deck System

Based on the information gathered for this system, the following advantages and disadvantages can be identified:

Advantages

- All the materials used in the production of the panel are non-proprietary and ready available

- The use of pre-tensioning for transverse flexure helps reducing the section and weight of the panel

- The use of post-tensioning helps achieve composite behavior between the deck and the underlying girder

- Active prestressing in both directions results in superior performance compared to conventionally reinforced decks

- Placement of shear connectors is more flexible, and the possibility of placing them in a single row could make deck removal easier

Disadvantages

- Slow installation in comparison with other full-depth systems that do not have post-tensioning

- Has not been tested as a full-depth precast deck. The first implementation has just been concluded, and results of its performance are not in the public domain yet

3.1.4 Exodermic™ Deck System

According to Exodermic Bridge Deck, Inc., this system has the following advantages and disadvantages:

Advantages

- Making efficient use of the constituent materials can span up to 15’ between supports and more

- Can be much lighter without sacrificing strength, stiffness, ride quality or expected life. An Exodermic deck typically weighs 35% to 50% less than the reinforced concrete deck that would be specified for the same span

- When the main bars of the grid are oriented parallel to the main girders, the substantial steel in the grid and the top reinforcing in the concrete slab adds to the capacity of the main girders

- When using precast panels, can be erected during a short, nighttime work window, allowing a bridge to be kept fully open to traffic during the busy daytime hours

- Hot dip galvanizing of the grid provides over 50 years of protection from corrosion

Disadvantages

- Contractor unfamiliarity in some areas.

3.1.5 Effideck™ System

According to The Fort Miller Co., Inc, the Effideck™ System has the following advantages:

Advantages

- Utilizes standard stock materials that are ready available

- The system is very versatile. Because a large choices of tube sizes, wall thickness and spacing, decks that may be required to be thin (or thick) or unusually strong are easily accommodated

- Efficient connection details facilitate overnight installation. Panels set directly on steel shims may be used immediately upon erection. Although panels can not develop full composite action until the grout has been installed, ungrouted panels can be occupied by the crane and delivery trucks to set more panels

- All work performed from the top of the deck makes installation safer and faster without costly scaffolding

- Is positively bolted down to steel stringers and does not depend upon vibration-vulnerable grouted connections

- Savings in readily available materials, simple fabrication and rapid erection cut overall construction costs

Disadvantages

- Has a proprietary nature

3.1.6 Inverset™ Bridge System

According to The Fort Miller Co., Inc, the Inverset Bridge Deck System has the following advantages:

Advantages

- Reduced beam depth. Reduced superstructure depth provides more clearance underneath while maintaining upper profile

- Units can be picked up at any point and even rolled into place

- Faster installation that includes deck, beams and connectors. Often under a day or even in minutes, under traffic conditions

- Year-round installation, units may be installed in cold winter months, day or night

- Inverset decks are in compression under their own dead load, further decreasing chances of transverse deck cracking and or chloride intrusion

- The densest concrete ends up at the riding surface providing greater durability

Disadvantages

- Precompressed concrete can not be replaced in the field. Future redecking requires removal of the entire units

- Has a proprietary nature

3.2 Comparison among full-depth precast deck systems

This section focuses in the comparison of the deck systems from the standpoint of design, fabrication, construction and in-service performance. To accomplish this task, it was found convenient to divide the analysis in three parts. In the first one, the three full-depth precast, prestressed deck systems are compared. Since all these systems have prestressing in the transverse direction, post-tensioning in the longitudinal direction, and have also similar limitations, a more in-depth analysis can be made. With the same rationale, a second comparison was made among the steel grid deck systems. Finally, a comparison was made among precast prestressed deck systems, steel grid deck systems, and the proprietary systems, Effideck™ System and Inverset™ Bridge System.

3.2.1 Full-depth precast, prestressed deck systems

The differences among the NER System, the NCHRP System and the NU-Deck System are basically three. The cross section of a NCHRP System typical panel is stemmed and non-prismatic, while the other two systems have a prismatic rectangular section. The NER System has longitudinal post-tensioning distributed through the slab, while the other two systems have it over the girders. Finally, NU-Deck System panels have an open channel over each girder to place shear connectors, while the panels of the other two systems have pockets over the girders to facilitate placement.

3.2.1.1 Design

From the standpoint of design, the NCHRP System is the lightest system. The stemmed non-prismatic section is the result of the arrangement of the prestressing strands in the transverse direction in order to obtain an efficient section and reduce weight and amount of reinforcement. Also, due to the reduced weight, the amount of post-tensioning required in this system is less. However, to maintain an optimum design for other conditions of span length or load, the cross section has to be modified. In the case of the other two systems the prestressing design in the transverse direction is much simpler. Between the NER System and the NU-Deck System, there is no technical difference in the transverse design except that the NU-Deck System requires additional reinforcing bars over the girders to maintain the transverse prestressing and the gap over the girders.

In a comparison made by Yamane et al. (1998), for a 12’ span, based on a HS-25 truck loading, a precast reinforced concrete panel required a 9” thickness, with a weight of 110 psf, while the NCHRP System had a maximum thickness of 8.1” and a weight of 70 psf (more than 30% lighter). On the other hand, a typical NU-Deck panel for the Skyline Bridge (2003), for an 11’ span, had a uniform 6” thickness and a weight of approximately 75 psf. Considering that the span was a little shorter, the NU-Deck System was around 10% heavier than the NCHRP System. If we consider that for the same span the NU-Deck panel would weigh around 80 psf, that would mean that around 75% of the weight savings (from 110 to 80 psf) are due to the inclusion of prestressing, and 25% are due to the non-prismatic section.

In the case of the post-tensioning, the NER System has it distributed through the slab. This distribution is very adequate to keep the transverse joints in compression. Research has been done with this system to determine the minimum amount of post-tensioning required (Issa et al., 1998). In the case of the other two systems, the post-tensioning is over the girders. This location is more adequate for the girder to reach full composite behavior at the negative moment region. As stated before, the lighter NCHRP System requires less post-tensioning.

Finally, regarding the placement of shear connectors, the open channel in the NU-Deck System permits a more rational design, in which connectors could be placed with different spacing. Also, placing them in a single row would facilitate future deck removal. In the other two systems the connectors have to be placed in the predetermined pockets.

3.2.1.2 Fabrication

From the standpoint of fabrication, the NER System panels seem the easiest to fabricate. They are similar to simple pre-tensioned slabs with the inclusion of block-outs for the connectors. Post-tensioning ducts are included also. In case of the NU-Deck panels, special formwork is needed to maintain the gap over the girders position. These panels have more reinforcement to be placed. In the NCHRP System, special formwork is also needed for the stemmed non-prismatic section.

3.2.1.3 Construction

From the standpoint of construction, one of the most important issues is the speed of installation. Care needed in shipping, handling and erecting of the panels is inversely proportional to their stiffness. NCHRP System panels are the stiffest, while NU-Deck System panels are the most flexible. In case of replacement projects, existing connectors have to be replaced in almost all cases, even with the NU-Deck System, where existing connectors might interfere with the post-tensioning strands. The NU-Deck System offers the least difficulty in installing new connectors, while the NCHRP System is the most difficult. To fill the pockets with grout in the NER System is easier than in the NCHRP System, while the NU-Deck System requires more grout to be poured on field. In general, difference in speed of construction amongst these systems could be estimated to be not significant.

3.2.1.4 In-service performance

From the standpoint of in-service performance, the NER System has been used several times with very good performance. After 13 years of service, the last inspection file of the bridge in Waterbury, CT, whose deck was replaced in 1989, rated the structure condition of the deck with 7 (on a scale from 0 to 9) which means that it is in very good condition. The total underside deck deterioration was less than 1%. The overall deck rating (including other elements like overlay, curbs, parapet, etc.) was 6 (good) (AI Engineers, Inc., 2002).

After 7 years of service, the last inspection files for the two bridges in Seymour, CT, whose deck was replaced in 1994, rated the structure condition of both decks with 7 and the total underside deck deterioration was also less than 1% in both cases. The overall deck rating was also 7 for both bridges (Lichtenstein Consulting Engineers, Inc., 2001).

The NCHRP System has never been used in an actual bridge. When the system was tested at the University of Nebraska, the system showed good performance. In particular, two way prestressing resulted in controlled cracking, and the grouted post-tensioning joints exhibited satisfactory performance under service load and fatigue load.

The NU-Deck System has just (October, 2003) been used for the first time in the Skyline Bridge, Omaha, NE.

A summary of this comparison is presented in Table 11.

3.2.2 Full-depth steel grid deck systems

3.2.2.1 Design

In terms of deck weight, open grids have a typical weight range of 14 to 20 psf and are the lightest of all systems. Exodermic decks and half-depth concrete filled grid decks are comparable with typical weigh ranges of 50 to 75 psf and 60 to 75 psf, respectively. Full-depth concrete filled grid decks have a typical weigh range of 70 to 100 psf.

Exodermic decks thickness is typical between 6” and 10”, and since the concrete is on the top of the grid, is thicker than other steel grid decks. Therefore, they have higher flexural capacity and can span longer. Exodermic decks with standard components can span up to 16’ and more. On the other hand, concrete filled grid decks have thicknesses less than 6”, and can not span more than 10’. Open grids can not span more than 8’.

3.2.2.2 Fabrication

From the standpoint of fabrication there is not much difference amongst all the grid decks, except, obviously, that open grids do not employ concrete in their fabrication.

3.2.2.3 Construction

From the standpoint of construction, Exodermic decks do not require field welding, except for the placement of the shear studs. On the other hand, concrete-filled grid decks, where the grid is both positive and negative reinforcement, require time consuming welding or bolted details for continuity.

3.2.2.4 In-service performance

From the standpoint of in-service performance, open grids have several disadvantages that make this system attractive mainly in deck replacement job, where dead load can not be increased. Among these disadvantages are that they are fatigue prone, produce an unpleasant ride quality and allow debris and salt laden water through the grids. Performance of concrete filled grid decks has been excellent in terms of durability. Hot dip galvanizing provides over 50 years of protection from corrosion.

3.2.3 Comparison among all full-depth precast deck systems

3.2.3.1 Design

In terms of deck weight, the Exodermic Deck™ System and the half-depth concrete filled grid deck systems seem to be the lightest systems. Exodermic decks typically weight between 50 and 75 psf. The Effideck™ System, with the typical 5” slab, weighs around 75 psf. Precast, prestressed concrete deck systems with rectangular cross sections and thicknesses from 6” to 8”, weight between 75 and 125 psf. The Inverset™ Bridge System has typically a 7 ½” slab, but weight savings are due to reduced beam depths. Thus, a comparison should be done for the whole superstructure weight. Reduced superstructure depth in this system provides more clearance underneath the beams.

Table 10 shows information relative to thickness, weight and associated maximum spans for each deck system. A precast prestressed concrete deck system with uniform rectangular cross section with pretensioning design in the transverse direction (NU-Deck) requires 6” thickness at a weight of 75 psf to cover an 11’ span (Skyline Bridge). Exodermic decks with standard components can span up to 16’ between supports. Effideck panels with 8” depth tubes and 13” total thickness can cover a 17’ span (Bridge over Cascadilla Creek) with a weight comparable to the NU-Deck Skyline Bridge.

All these deck systems can ensure composite action for positive moments with the girders. However, only precast, prestressed concrete deck systems with longitudinal post-tensioning can provide composite action for negative moments with the girders.

3.2.3.2 Fabrication

The fabrication of the precast, prestressed concrete deck systems requires pre-tensioning. As stated before, the NER System seems to be the easiest to fabricate, because the NU-Deck System and the NCHRP System require special formwork for the gap over the girders and the non-prismatic section, respectively.

The fabrication process of the Exodermic™ Deck System and the concrete-filled steel grid deck systems typically entails the formation of the haunches and casting the concrete. The steel grid acts as formwork. The deck areas that will be in contact with the top flange of the beams are blocked out during the fabrication stage.

In the case of the proprietary systems, the fabrication of the Inverset™ units or the Effideck™ panels have to be coordinated with the Fort Miller Company, Inc.

3.2.3.3 Construction

From the standpoint of speed of construction, all these systems can be divided, basically, in two groups. The first group includes the precast, prestressed concrete deck systems, where post-tensioning operations make the systems slower. The most favorable scenario for them is staged weekend construction. This has been done, for example, on the bridges in Seymour, CT (1994), where partial closure of the bridge was allowed only during weekends. In these cases of staged construction during weekends, typically one span is done per weekend, and therefore, all spans have to be designed as simply supported. When staged construction is not allowed due to the reduced width of the bridge, entire deck closure during weekends would be the best option. This was done in a bridge in West Hartford, CT (2000). In case the entire deck closure is allowed also during weekdays, the replacement project can be done in less time, and spans could be designed as continuous. This was done in the bridge in Waterbury, CT (1989).

The second group includes all the other systems discussed in this chapter. The most favorable scenario for these faster systems is staged night-time construction. In this timeline, some lanes are closed, replaced with the new deck, and opened again to traffic in less than 10 hours. This has been done, for example, in the Tappan Zee Bridge, NY (1998), where the Exodermic™ Deck and Inverset™ Bridge Systems were used. When staged construction is not allowed, another option for these systems is full closure of the bridge overnight. Since these systems are faster than those of the first group, all the scenarios described earlier would be alternatives for the use of these systems.

3.2.3.4 Costs

Deck cost depends on many factors, such as project location, project size, distance between supports, design loads, time constraints, etc. In general, lower costs per square foot are obtained with bigger projects, smaller distances between supports, lower design loads, etc. To make a good cost comparison of the different deck systems, information from similar projects would be necessary. However, for most of these systems, that are relatively new, costs information is limited and not necessary for similar projects.

For the New England Region System, the following information of deck replacement projects with precast panels in Connecticut was obtained from the CT-DOT. For the bridges in Seymour, where more than 50 spans (> 160,000 square foot) were replaced with staged construction during weekends only, the average of the three lower bidders was $21 per square foot (1992). For the bridge in West Hartford, were of only three spans (5,000 square foot) were replaced with full closure of the bridge and weekend construction only, the average of the three lower bidders was $52 per square foot (1998).

For the Exodermic™ Deck System, the following information was obtained from Robert Bettigole, President of the Exodermic Bridge Deck, Inc. Except in a few areas such as New York City, or where there are unusually difficult constraints on how the work gets done; a rough estimate of cost would be between $35 and $38 per square foot. For example, deck cost for the Tappan Zee Bridge (Tarrytown, NY), where 13 deck-truss spans (250,000 square foot) were replaced with nighttime construction, was $35 per square foot. On the other hand, the cost of the deck in a bridge in Connecticut with only 5,600 square foot was $48 per square foot.

The NU-Deck System has just been implemented for the first time in a two-span girder bridge, the Skyline Bridge (Omaha, NE). According to information received from one of the investigators at the University of Nebraska involved in this project, deck cost was estimated around $20 per square foot. After the panels were erected the cost has not been confirmed yet.

Summarizing the information:

New England Region System	\$21-\$55 / ft^2 *
Exodermic™ Deck System	\$35-\$48 / ft^2 **
NU-Deck System	\$20 / ft^2 ***

* Based on three lower bidders of two projects
** Based on manufacturer's cost claim
*** Based on engineer's estimate of one project

Table 9

Applicability of Full-Depth Precast Deck Systems
Bridges with Deck Spanning in the Transverse Direction

<table>
<tr><td colspan="4"></td><td colspan="6">Steel Girders or Stringers</td></tr>
<tr><td colspan="4"></td><td colspan="3">Non-crowned</td><td colspan="3">Crowned</td></tr>
<tr><td colspan="4"></td><td>Rectangular</td><td>Skewed</td><td>Curved</td><td>Rectangular</td><td>Skewed</td><td>Curved</td></tr>
<tr><td rowspan="5">Deck Systems</td><td colspan="3">New Construction</td><td>New England Region
NCHRP
NU-Deck
Open Grid
Full-Depth Filled Grid
Half-Depth Filled Grid
Exodermic Deck
Effideck
Inverset Bridge*</td><td>New England Region
NU-Deck
Open Grid
Full-Depth Filled Grid
Half-Depth Filled Grid
Exodermic Deck
Effideck
Inverset Bridge*</td><td>New England Region
Open Grid
Full-Depth Filled Grid
Half-Depth Filled Grid
Exodermic Deck
Effideck
Inverset Bridge*</td><td>New England Region
NU-Deck
Open Grid
Full-Depth Filled Grid
Half-Depth Filled Grid
Exodermic Deck
Effideck
Inverset Bridge*</td><td>New England Region
NU-Deck
Open Grid
Full-Depth Filled Grid
Half-Depth Filled Grid
Exodermic Deck
Effideck
Inverset Bridge*</td><td>New England Region
Open Grid
Full-Depth Filled Grid
Half-Depth Filled Grid
Exodermic Deck
Effideck
Inverset Bridge*</td></tr>
<tr><td rowspan="4">Deck Replacement</td><td rowspan="2">Staged Construction</td><td>Overnight Construction Only</td><td colspan="6">Open Grid
Full-Depth Concrete Filled Grid
Half-Depth Concrete Filled Grid
Exodermic Deck
Effideck</td></tr>
<tr><td>Weekend Construction Only or During Longer Periods</td><td colspan="6">New England Region
Open Grid
Full-Depth Concrete Filled Grid
Half-Depth Concrete Filled Grid
Exodermic Deck
Effideck</td></tr>
<tr><td rowspan="2">Full Closure</td><td>Overnight Construction Only</td><td colspan="6">Open Grid
Full-Depth Concrete Filled Grid
Half-Depth Concrete Filled Grid
Exodermic Deck
Effideck</td></tr>
<tr><td>Weekend Construction Only or During Longer Periods</td><td>New England Region
NCHRP
NU-Deck
Open Grid
Full-Depth Filled Grid
Half-Depth Filled Grid
Exodermic Deck
Effideck</td><td>New England Region
NU-Deck
Open Grid
Full-Depth Filled Grid
Half-Depth Filled Grid
Exodermic Deck
Effideck</td><td>New England Region
Open Grid
Full-Depth Filled Grid
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NU-Deck
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Full-Depth Filled Grid
Half-Depth Filled Grid
Exodermic Deck
Effideck</td><td>New England Region
Open Grid
Full-Depth Filled Grid
Half-Depth Filled Grid
Exodermic Deck
Effideck</td></tr>
</table>

* Superstructure System (include beams)

Table 10

Applicability of Full-Depth Precast Deck Systems
Bridges with Deck Spanning in the Longitudinal Direction

		Steel Floor Beams
		Non-crowned or Crowned
		Rectangular, Skewed or Curved
Deck Systems	**New Construction**	Open Grid Full-Depth Concrete Filled Grid Half-Depth Concrete Filled Grid Exodermic Deck Effideck Inverset Bridge*
	Deck Replacement	Open Grid Full-Depth Concrete Filled Grid Half-Depth Concrete Filled Grid Exodermic Deck Effideck

* Superstructure System (include beams)

Table 11

Full-Depth Precast Prestressed Bridge Deck Systems
Comparison

Deck System	Design		Fabrication		Installation		Performance	
	Advantages	Disadvantages	Advantages	Disadvantages	Advantages	Disadvantages	Advantages	Disadvantages
NER System	* Transverse joints in compression						* Have been used several times	
NCHRP System	* Lightest system * Less post-tensioning required * Composite action for negative moments	* Most complicated to design		* Special formwork for the non prismatic multi-stemmed section	* Stiffer panels	* Most difficult to install shear studs		* Never used on an actual bridge
NU Deck System	* Composite action for negative moments	* Requires aditional bars over girders		* Special formwork needed to maintain gap over girders	* Easiest to place shear studs	* More flexible panels		* Have just been used for the first time

Table 12

Full-Depth Bridge Deck Panel Systems
Design

	Deck Thickness	Deck Weight	Span	Commentary
Precast, Prestressed Concrete Decks				
New England Region System	8"	100 psf	9'	Seymour Bridges (CT, 1994)
NCHRP System	8.1"	70 psf	12'	Specimen tested at NU (NE, 1998)
NU-Deck System	6"	75 psf	11'	Skyline Bridge (NE, 2003)
Steel Grid Decks				
Open Grid Systems	< 6"	14 - 20 psf	< 8'	
Full-Depth Concrete Filled Grid Systems	< 6"	70- 100 psf	< 10'	
Half-Depth Concrete Filled Grids Systems	< 6"	60 - 70 psf	< 9'	
Exodermic Deck System	6" - 10"	50 - 75 psf	16'	with standard components
Other Composite Systems				
Effideck System	7" - 13"	75 psf	17'	Bridge in Ithaca (NY, 2001)
Inverset Bridge System*	7 1/2"	95 psf	< 10'	with typical thickness

* Superstructure System (include beams)

CHAPTER 4
SUMMARY, CONCLUSIONS AND SUGGESTED IMPLEMENTATION

4.0 Introduction

This chapter is divided in three sections. First, findings reported in the first three chapters of the report are summarized. Next, conclusions drawn from the whole study are presented. Areas of additional work are also discussed. Finally, an implementation plan is suggested.

4.1 Summary of findings and needs

4.1.1 Literature Review

4.1.1.1 Full-depth precast deck systems

Nine different full-depth precast deck systems were identified: three precast, prestressed concrete deck systems, four types of steel grid deck systems and two proprietary deck systems.

Among the precast, prestressed concrete deck systems, one (NER System) has been used for more than ten years with very good results, one was only tested but not used in an actual bridge (NCHRP System) and the last one (NU-Deck System) has just been used for the first time.

From the four types of steel grid deck systems, three have been used for decades (open grids, full-depth and half-depth concrete filled decks). The ‘newest’ of these systems has almost 20 years in the market (Exodermic), although its grid design was improved five years ago.

One of the two proprietary systems has been used at least in four bridges during the last five years (Effideck™ System), and the other one (Inverset™ Bridge System) is actually a superstructure system (because it includes the beams), that has been on the market for 20 years.

4.1.1.2 Deck/concrete girder connections

A new connection system (Figs. 18 and 19) consisting of a debonded interface with shear keys formed into the girder and steel connectors at wide spacing was developed under NCHRP Project 12-41 (Tadros and Baishya, 1998). The system was modified to have the shear keys below the top flange surface to make it suitable for different types of decks like CIP systems, SIP systems and full-depth precast deck systems (Figs. 20 and 21). Two bridges in Nebraska were used to compare this system with a conventional roughened interface system. The conclusions of this study were that the new system had the advantages of facilitating future deck removal, protecting shear connectors against corrosion, protecting the girder top flange from damage during deck removal and optimizing the design for horizontal shear. The system was found to perform well and had no detrimental effects on composite action or bridge stiffness (Tadros et al., 2002).

Research recently concluded at Virginia Tech (Menkulasi, 2002) studied the horizontal shear strength developed at the interface of two precast concrete members bonded by means of grout. Results of this research are not in the public domain as of yet, but it is expected that will provide recommendations for the use of precast, prestressed concrete deck systems (NER System) over precast concrete girders.

4.1.1.3 Deck/steel girder connections

Research conducted under NCHRP Project 12-41 proposed a new 1 ¼” diameter shear stud system for deck/steel girder connections (Figs. 24 and 25). This stud provides approximately twice the capacity of a ⅞” stud and would allow positioning in a single row over the girder web. This arrangement reduces the effort required for deck removal and also the probability of damaging the girder top flange and the larger stud. To further facilitate deck removal, alternating headed and headless studs was found adequate for anchorage to the concrete deck (Tadros and Baishya, 1998). However, implementation of this system in the Skyline Bridge considered headed studs only.

4.1.2 Identification of issues and needs

4.1.2.1 Issues addressed

Main issues addressed for the implementation of full depth precast deck systems are:

- All systems except the Inverset™ Bridge System can be used for deck replacement projects
- All systems except the precast, prestressed concrete deck systems can be used in bridges with deck spanning in the longitudinal direction
- All systems except the NCHRP System can be used in skewed or curved bridges
- All systems except the NCHRP System and the open grids can be used for crowned bridges
- For transverse joints, female to female shear key with at least ¼” opening at the bottom is recommended. The use of Set Grout ™ is recommended to fill the shear key and, when traffic interruption is critical, high early strength polymer concrete is recommended (Issa et al., 2003)

- Post-tensioning keeps joints in compression and prevents leakage. For the NER System, a minimum prestress level of 200 psi is needed for simply supported bridges, while 450 psi is needed over the interior supports for continuous bridges (Issa et al., 1998)

- All systems except the Inverset™ Bridge System are continuous for both positive and negative moments

- All systems can achieve composite behavior with the underlying girders in the positive moment region. However, for negative moment regions only systems with post-tensioning can achieve composite behavior.

- All systems except the NCHRP System can be used with transverse staged construction

- Precast, prestressed concrete deck systems can be used with weekend construction, while the other systems can also be used with night-time construction.

4.1.2.2 Needs

The following key items remain to be addressed:

- Use of full-depth precast deck systems over concrete or precast concrete girders

- Full-depth precast, prestressed concrete deck system for bridges with decks spanning in the longitudinal direction remains unavailable. Examples of this type of bridge include Arch Bridges and Truss Bridges with transverse floor beams and no stringers

- Establish practical limits for the skew angle and curvature radius for the use of full-depth precast, prestressed deck systems in skewed and curved bridges

- Determine the minimum amount of post-tensioning necessary in the NU-Deck System to maintain the transverse joints in compression

4.1.3 Comparison of full-depth precast deck systems

4.1.3.1 Full-depth precast, prestressed deck systems

- NCHRP System is the lightest system and therefore requires less post-tensioning

- Post-tensioning helps NCHRP and NU-Deck Systems achieve composite action with girders in negative moment regions, and NER System have transverse joints in compression

- NCHRP is the most complicated to design
- From the standpoint of fabrication, panels in both NCHRP and NU-Deck System require special formwork

4.1.3.2 Full-depth steel grid deck systems

- Open grid is the lightest system of all but has several disadvantages like being fatigue prone, produce an unpleasant ride quality and allow debris and salt laden water through the grids, that makes this system attractive for use only in replacement jobs
- In terms of weight, the Exodermic Deck System is comparable to half-depth concrete filled grid decks and both are lighter than full-depth concrete filled grid decks
- Exodermic decks with standard components can span up to 16’, while none of the other systems can span more than 10’
- Exodermic decks do not require field welding, except for the placement of the shear studs

4.1.3.3 Comparison of all full-depth precast deck systems

- In terms of deck weight, Exodermic decks together with half-depth concrete filled grid decks seem to be the lightest systems. The Effideck™ System is typically the heavier one, but it is still lighter than all precast, prestressed concrete deck systems
- The Exodermic and the Effideck™ System can span more than 15’ between supports. Precast, prestressed concrete deck systems could probably span similar distances, but would require thicknesses (and weight) that have not been used yet
- The inclusion of post-tensioning gives precast, prestressed concrete deck systems an improved behavior at the cost of sacrificing speed of construction

4.2 Conclusions

4.2.1 New England Region System

The New England Region System has been successfully used for over ten years, only in New York, Connecticut and the States in the New England Region, but also in other states like Illinois, where the system is still being studied. The system has been implemented in deck replacement of crowned or non-crowned, rectangular, skewed or curved bridges with decks spanning in the transverse direction over steel supporting members.

4.2.2 NCHRP System

Five years after being developed, the NCHRP System has not been used in an actual bridge. This may be because the system can only be used for non crowned, non-skewed, non-curved, steel girder bridges, or because the fabrication of the panels is considered difficult. It is not recommended to implement the use of this system until more information on its in-service performance is available.

4.2.3 NU-Deck System

The NU-Deck System is a good alternative for a precast, prestressed concrete deck system. Compared to the New England Region System, it has some advantages and some disadvantages. Even though it has been used only in one bridge, it has the potential to be used in the same cases, with the same recommendations and limitations as the New England Region System. However, its first implementation has just been concluded and results of its performance are not in the public domain yet. It is recommended to wait for the results of the construction of the Skyline Bridge. A paper about it is expected to be published at the end of this year.

4.2.4 Exodermic™ Deck System

The Exodermic™ Deck System has been successfully used for almost 20 years. The revised design of this system has been the standard of the system since its first use in the Tappan Zee Bridge in 1998. This system, as the other steel grid decks, can be used either precast or cast in place in new construction or deck replacement of crowned or non-crowned, rectangular, skewed or curved bridges with decks spanning either in the transverse or in the longitudinal direction over steel supporting members.

Amongst steel grid decks, half-depth concrete filled decks could be an alternative to the Exodermic™ Deck System. Full-depth concrete filled decks are much heavier, and open grid decks would be recommended only to replace existing open grid decks.

4.2.5 Effideck™ System

The Effideck™ System has been successfully used in four bridges in the last five years. For rapid replacement of existing decks, this system seems to be a good alternative to the Exodermic Deck System or other steel grid decks, especially in the case of decks spanning in the longitudinal direction over steel floor beams of truss or arch bridges, where there are no other alternatives.

4.2.6 Inverset™ Bridge System

The Inverset™ Bridge System has been used in more than 145 bridges in over 12 states since 1982. This system offers great speed of construction because the units include deck, beams and connectors, but its use is limited to new construction or when the whole superstructure needs replacement. Applications of this system include crowned or non-crowned, rectangular, skewed or curved bridges with beams spanning in the longitudinal direction over piers or abutments or in the transverse direction as floor beams supported by trusses or arches. However, when this system is used, future replacement of the deck without replacing the beams would mean a reduction in the capacity of the system since the new deck will not have the pre-compression effect the original precast deck had.

4.2.7 Debonded interface with shear keys

This connection system was developed in 1998 for cast-in-place decks over precast concrete girders, and was modified in 2002 to make it suitable also for stay-in-place systems and full-depth precast deck systems (Figs. 20 and 21). This connection system has the advantages of facilitating future deck removal, protecting shear connectors against corrosion, protecting the girder top flange from damage during deck removal and optimizing the design for horizontal shear. The use of this connection system in a bridge with a cast-in-place deck in Nebraska was found to perform well and had no detrimental effects on composite action or bridge stiffness.

This connection system could permit the use of full-depth precast panels, like the NU-Deck System, the Exodermic™ Deck System or other steel grid decks, over precast concrete girders. This is something that has not been done yet and is convenient to wait until it is tested. However, this connection system could be implemented in precast girders with cast-in-place decks.

4.2.8 Interface of two precast concrete members

This research recently carried out in Virginia Tech, studied the horizontal shear strength developed at the interface of two precast concrete members bonded by means of grout. The connections studied in this project are suitable for the use of full-depth precast systems, like the New England Region System, that has pockets to connect the two precast elements. Recommendations for the use of the New England Region System over precast concrete girders are not available yet, but it is expected that the PCI New England Region Technical Committee provide recommendations in the near future.

4.2.9 1 ¼" diameter shear stud system

This system was proposed for deck/steel girder connections in 1998. Positioning these larger studs in a single row over the girder web reduces the effort required for deck removal and also the probability of damaging the girder top flange and the larger stud.

Also, the system proposed the alternate use of headed and headless studs. This system was implemented recently in the Skyline Bridge, but with headed studs only.

This system could be implemented with cast-in-place decks, stay-in-place decks, and full-depth systems like the Exodermic™ Deck System, or other steel grid decks, or the NU-Deck System. Connection pockets prevent its use in the New England Region System and the Effideck™ System.

4.3 Suggested implementation

Due to its successful performance over the years, it is recommended the implementation of the New England Region System in new construction or deck replacement of crowned or non-crowned, rectangular, skewed or curved bridges with decks spanning in the transverse direction over steel plate girders, hot rolled beams or stringers with any of the following construction schedules:

1. Closure of the entire bridge
2. Closure of the entire bridge during weekends only
3. Transverse staged construction
4. Transverse staged construction during weekends only

In cases 2 and 4, where deck replacement takes place during periods of 60 hours maximum and typically only one span per week can be replaced, girders have to be designed as simply supported. Exceptionally, if two short consecutive spans could be replaced in one weekend, girders could be designed continuous over one support.

For crowned bridges in cases 1 and 2, panels crown for this system can be formed like the NU-Deck panels of the Skyline Bridge. In cases 3 and 4, the longitudinal joint for the staged construction might prevent the use of such technique.

There are no practical limits for the skew or curvature radius of the deck. But suggested values for the skew angle are under 25º.

Due to its successful performance over the years and large range of applications, it is also suggested that the Exodermic™ Deck System be considered as a positive system for deck construction and/or replacement of crowned or non-crowned, rectangular, skewed or curved bridges with decks spanning either in the transverse or in the longitudinal direction over steel plate girders, hot rolled beams, stringers or floor beams with any of the following construction schedules:

1. Closure of the entire bridge
2. Closure of the entire bridge during weekends only
3. Closure of the entire bridge during nighttime only
4. Transverse staged construction
5. Transverse staged construction during weekends only

6. Transverse staged construction during nighttime only

Cases 1, 2, 4 and 5 for deck replacement are the same as with the New England Region System (including decks spanning in the longitudinal direction). Cases 3 and 6 are recommended for use in the situations where only nighttime construction is possible.

Although it has been used in few projects, the Effideck™ System could be used in all the cases the Exodermic™ Deck System can. This system is a good alternative to the Exodermic™ Deck System, or other grid decks, especially for decks spanning in the longitudinal direction over steel floor beams of truss or arch bridges, where there are no other options. However, since this is a proprietary system, The Fort Miller Co., Inc has to be contacted before implementing it.

It is also recommended the implementation of 1 ¼" diameter headed studs as connectors for steel girders or beams in cases of cast-in-place decks, stay-in-place decks, or full-depth systems, like the Exodermic™ Deck System or other steel grid decks, or for a future implementation of the NU-Deck System.

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APPENDIX 1

FIGURES

Fig. 1. Deck replacement using full-depth precast, prestressed panels in a bridge in West Hartford, CT (CT-DOT, 2000)

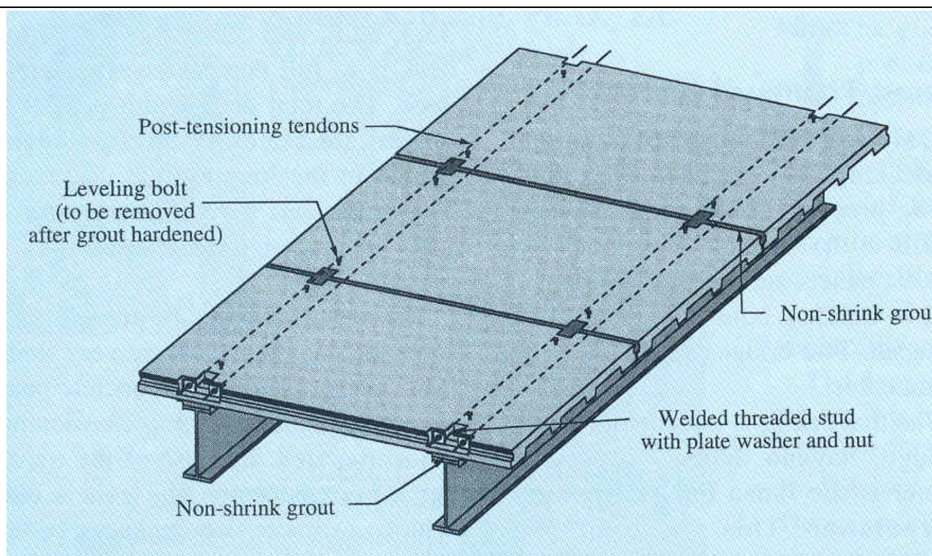

Fig. 2. Full-depth precast, prestressed deck system developed under NCHRP Project 12-41 (Yamane et al., 1998)

Fig. 3. Typical NU-Deck panel for a bridge in Omaha, NE (JDV, 2003)

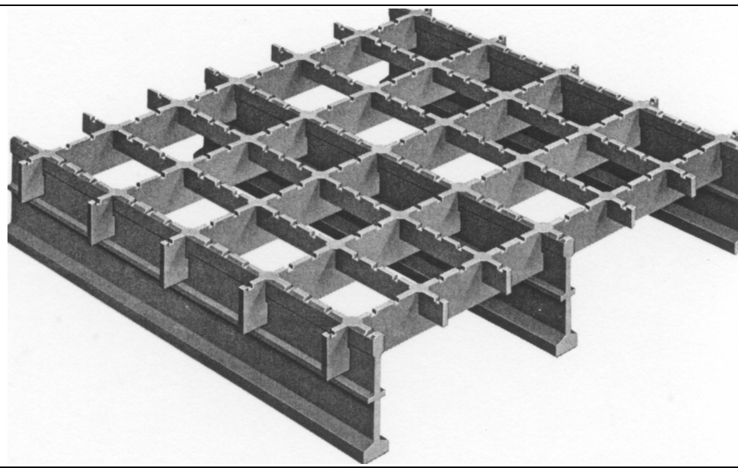

Fig. 4. Open grid deck (L.B. Foster, 1992)

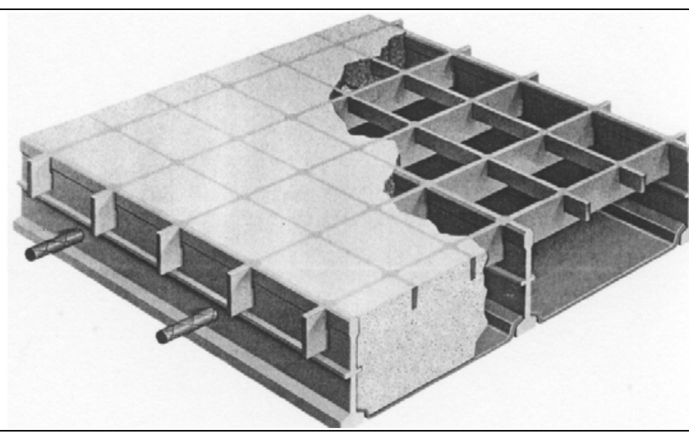

Fig. 5. Full-depth concrete filled grid deck (L.B. Foster, 1992)

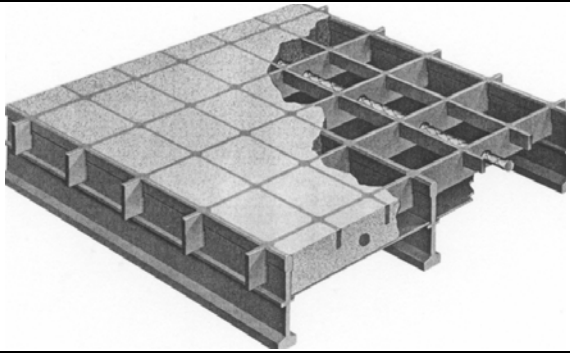

Fig. 6. Half-depth concrete filled grid deck (L.B. Foster, 1992)

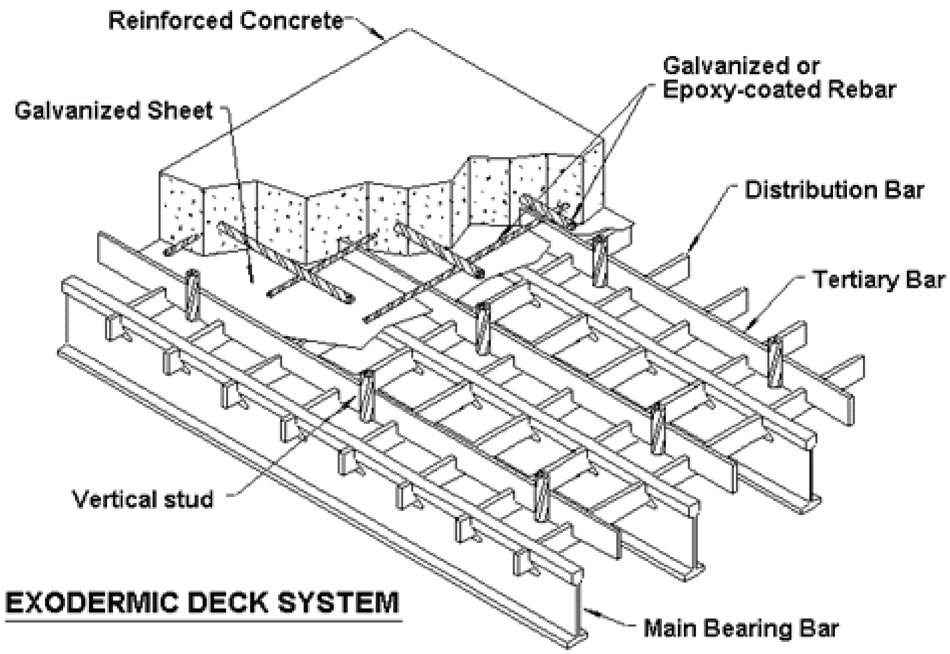

Fig. 7. Exodermic Deck System original design (Exodermic Bridge Deck, Inc., 1998)

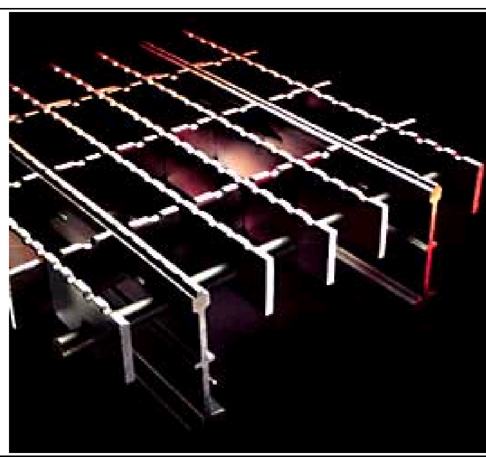

Fig. 8. Weld-less steel open grid deck (IDSI, 2003)

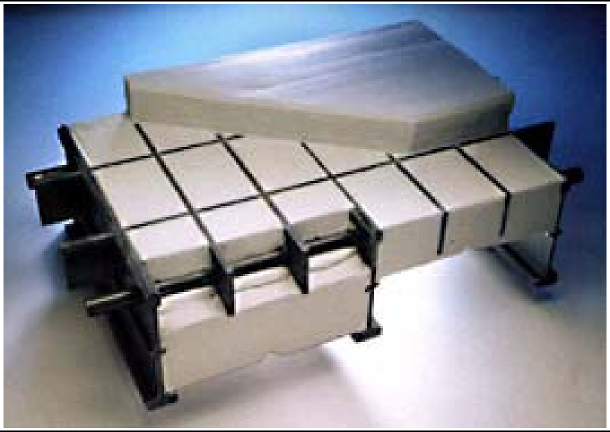

Fig. 9. Concrete filled weld-less steel grid deck (IDSI, 2003)

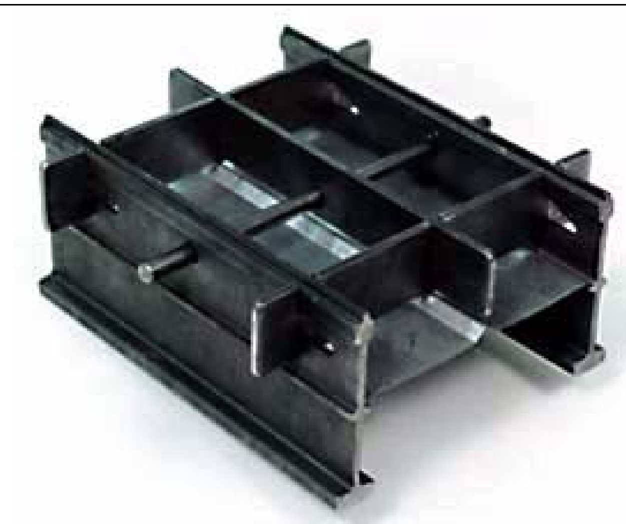

Fig. 10. 5-inch 5-piece weld-less steel grid deck (IDSI, 2003)

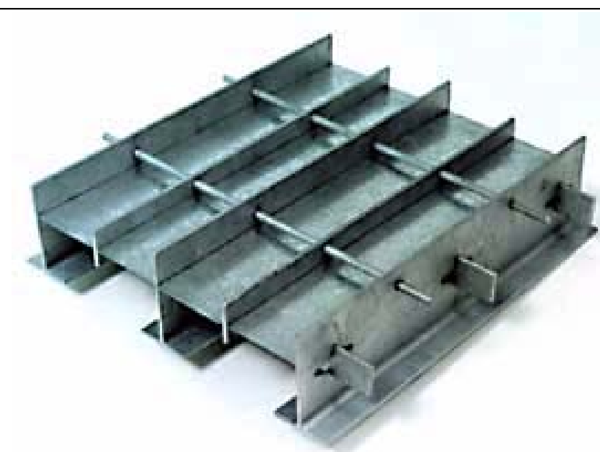

Fig. 11. Tee-Lock weld-less steel grid deck (IDSI, 2003)

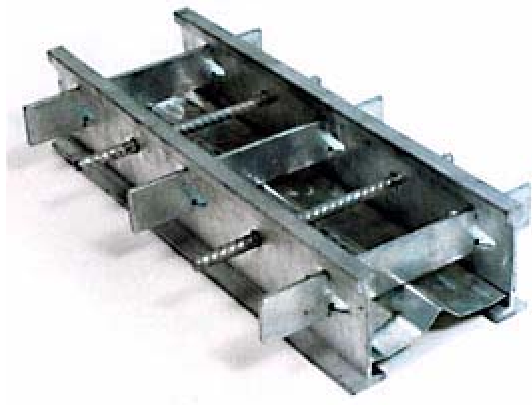

Fig. 12. 4.25-Inch 4-Piece weld-less steel grid deck (IDSI, 2003)

Fig. 13. Concrete overlayed weld-less steel grid deck (IDSI, 2003)

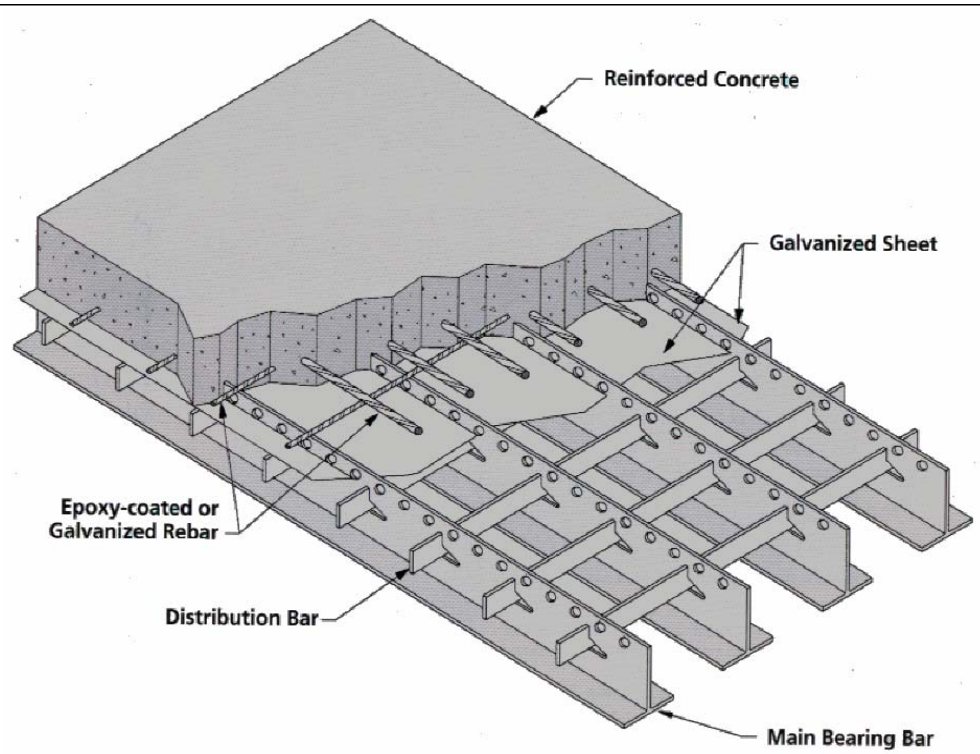

Fig. 14. Exodermic Deck System revised design (Exodermic Bridge Deck, Inc., 1998)

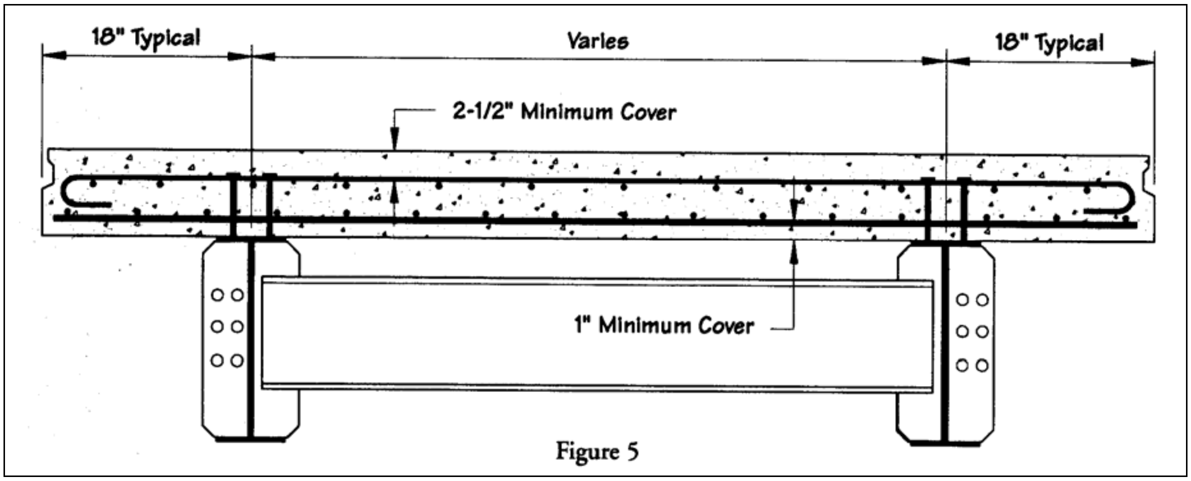

Fig. 15. Typical Inverset unit (The Fort Miller Co., Inc., 1998)

Fig. 16. Typical Effideck System panel (The Fort Miller Co., Inc., 1998)

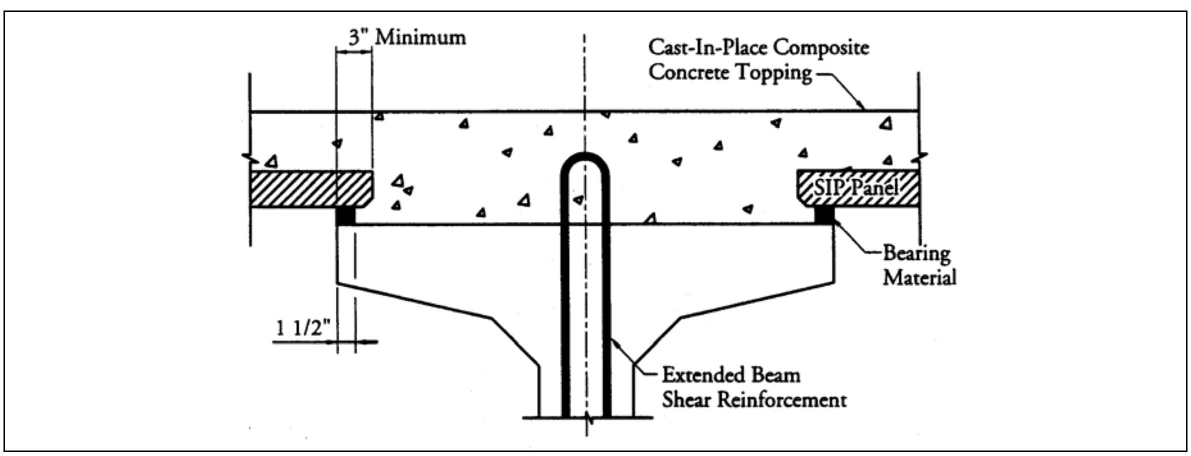

Fig. 17. Typical concrete girder/deck connection (PCI, 2000)

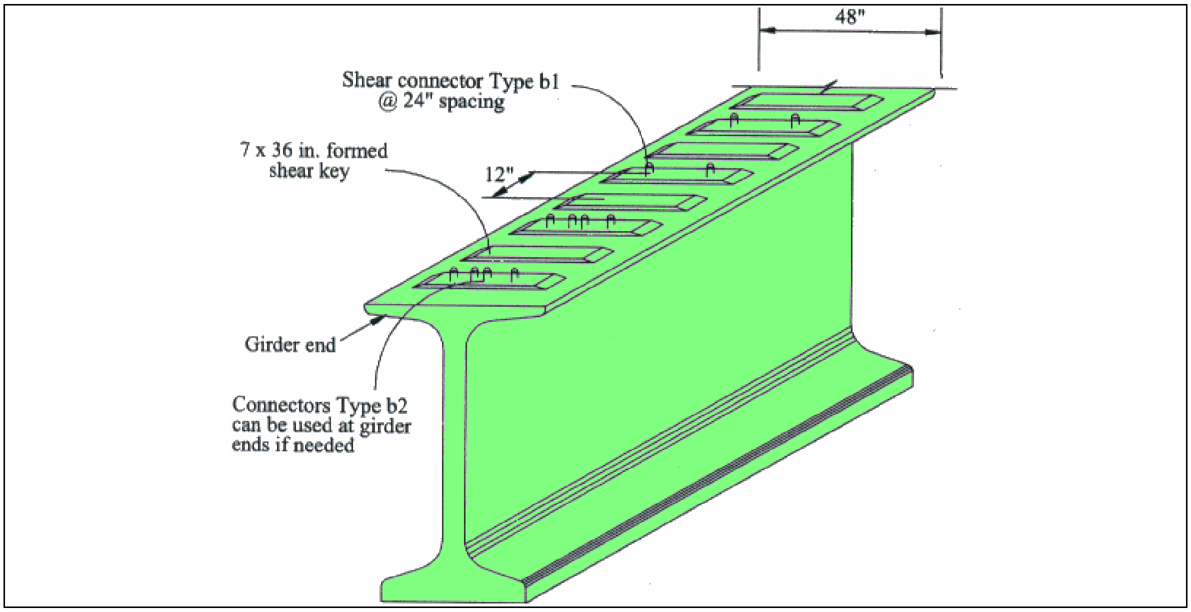

Fig. 18. Debonded shear key connection system (Tadros and Baishya, 1998)

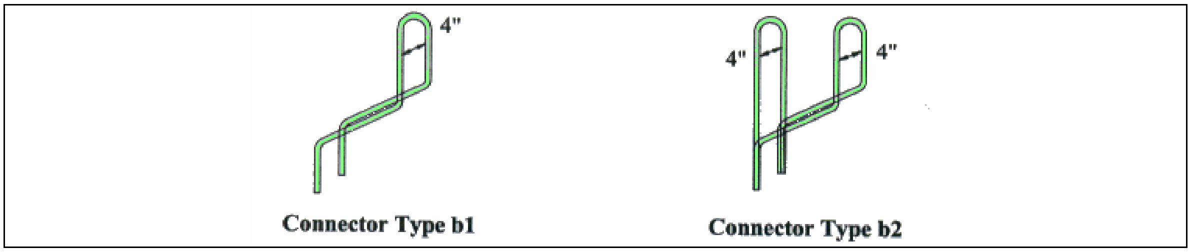

Fig. 19. Connectors for debonded system (Tadros and Baishya, 1998)

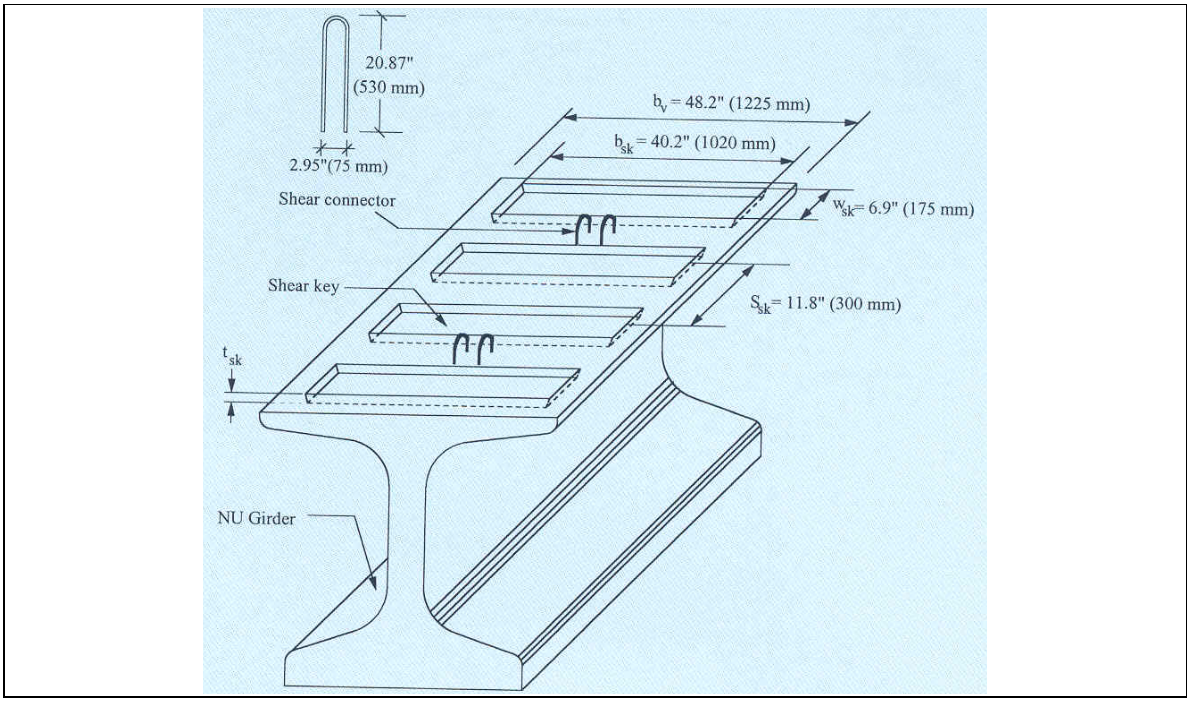

Fig. 20. Modified debonded shear key connection system (Tadros et al., 2002)

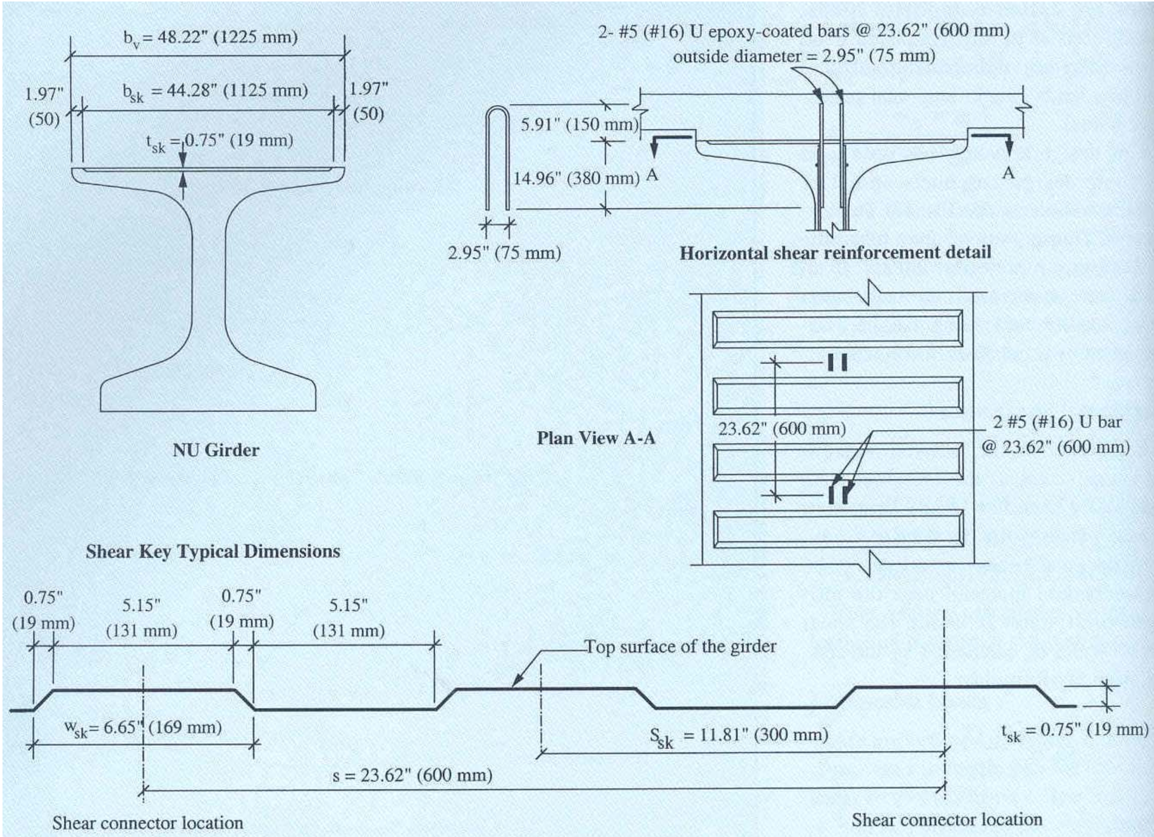

Fig. 21. Modified debonded shear key connection system (Tadros et al., 2002)

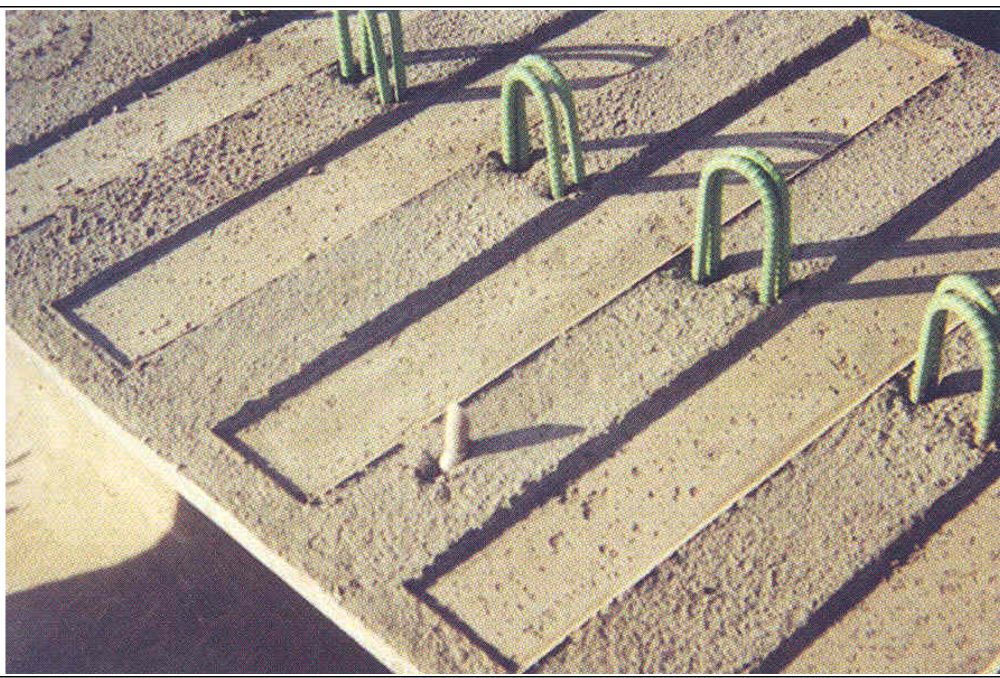

Fig. 22. Top view of debonded shear key connection system (Tadros et al., 1998)

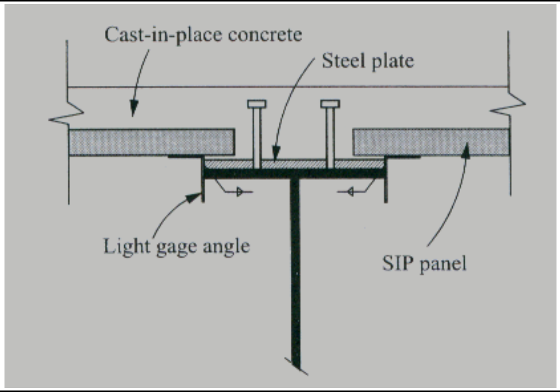

Fig. 23. Typical steel girder/deck connection (Tadros and Baishya, 1998)

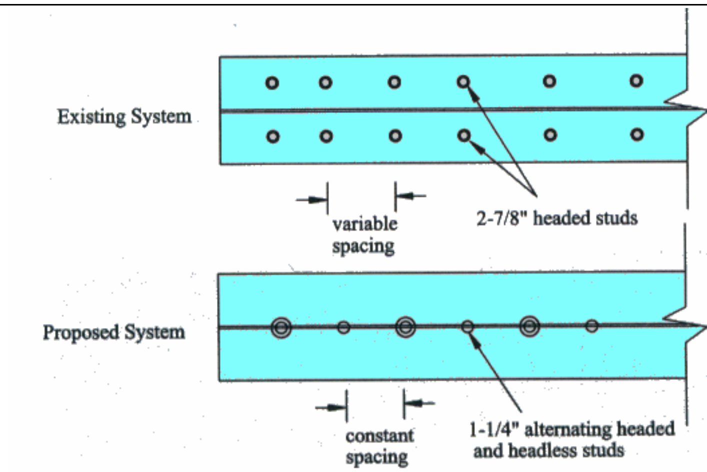

Fig. 24. Top view of existing and proposed steel girder/deck connection systems (Tadros and Baishya, 1998)

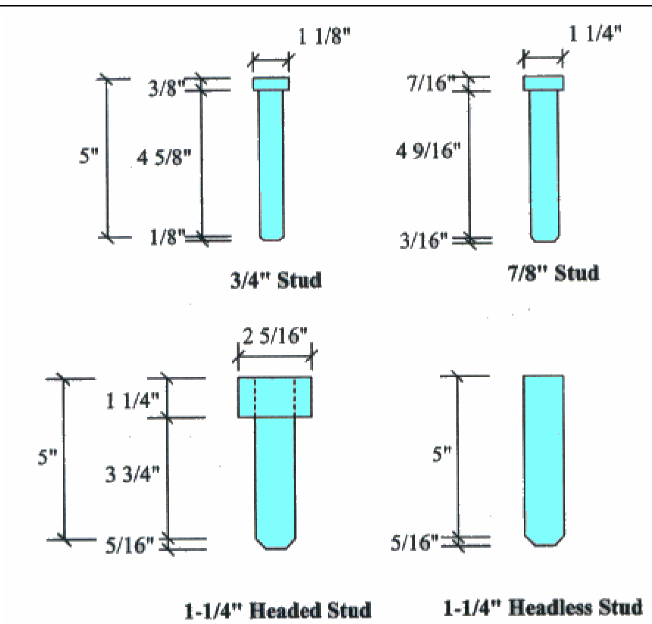

Fig. 25. Existing and proposed shear studs (Tadros and Baishya, 1998)

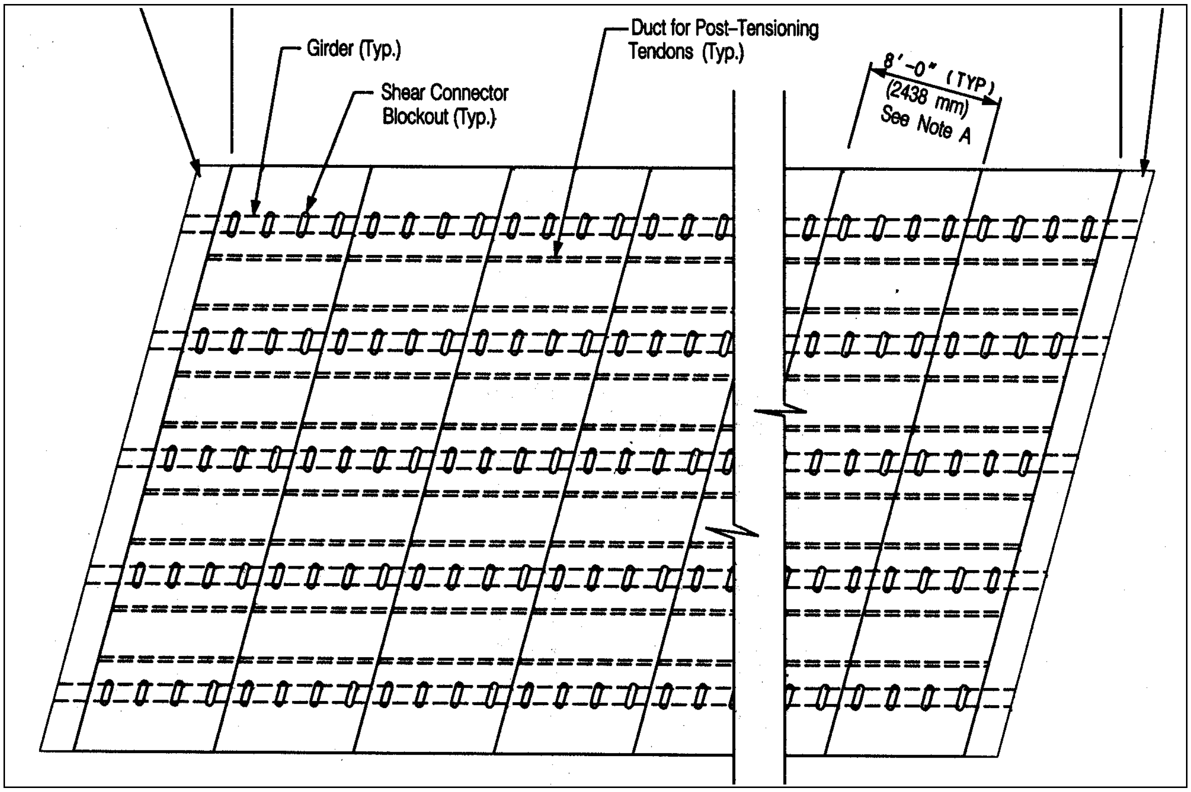

Fig. 26. Typical panel layout plan (PCINER, 2002)

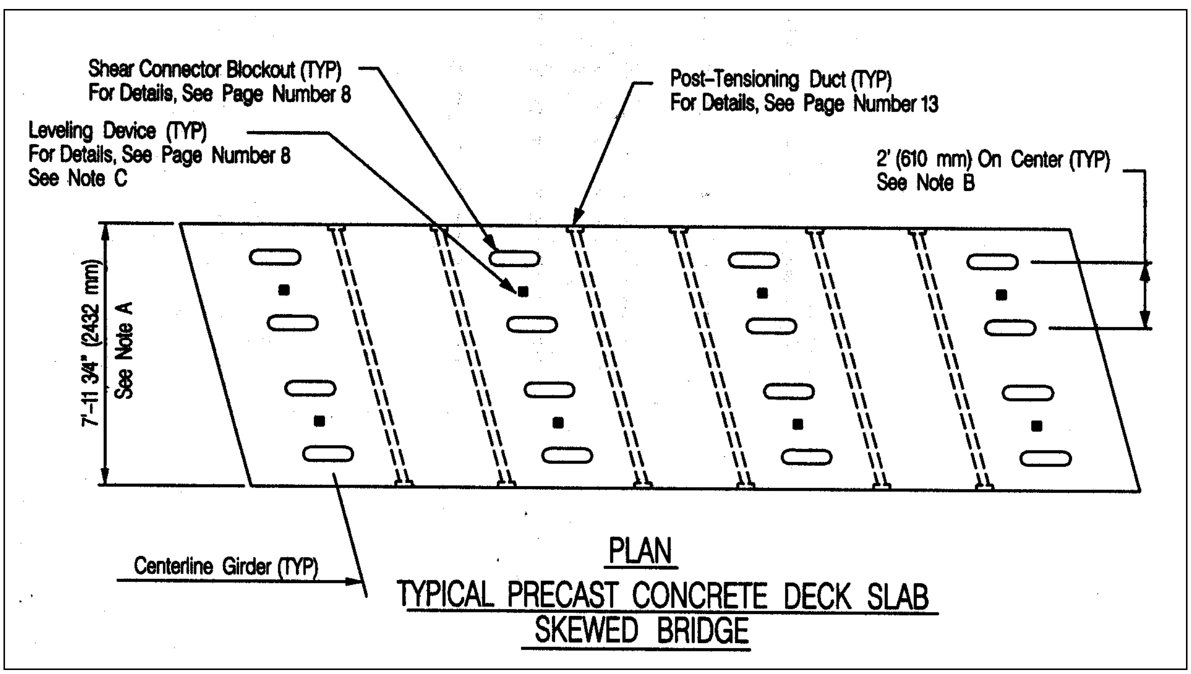

Fig. 27. Typical precast concrete deck slab for skewed bridges plan (PCINER, 2002)

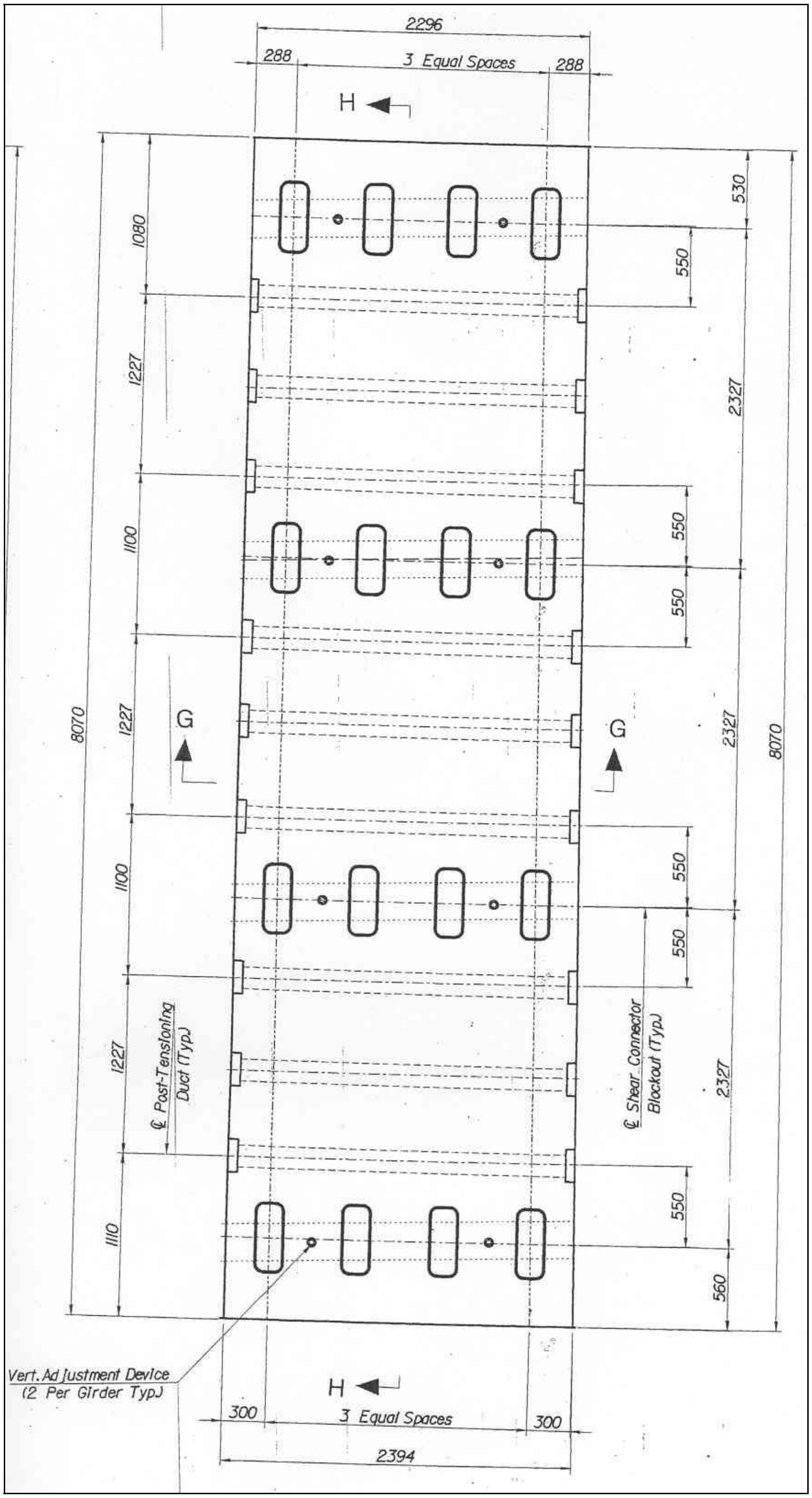

Fig. 28. Trapezoidal precast slab plan for a bridge in West Hartford, CT (CT-DOT, 1997)

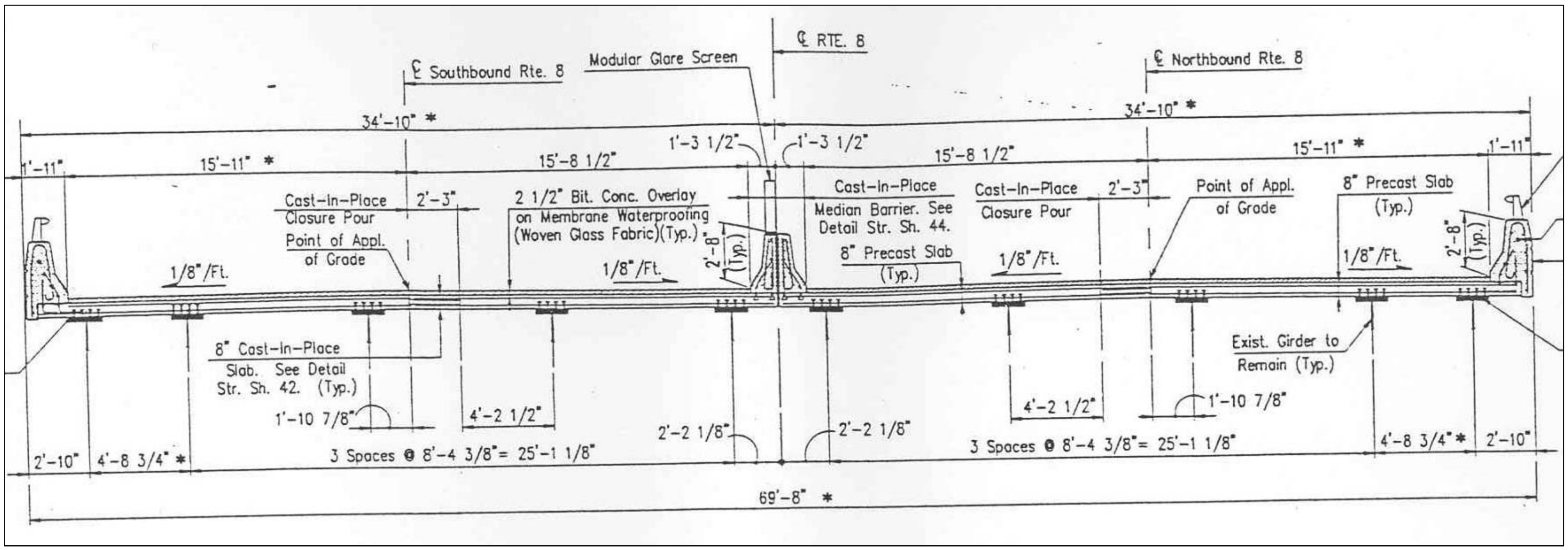

Fig. 29. Transverse cross section for one bridge in Seymour, CT (CT-DOT, 1992)

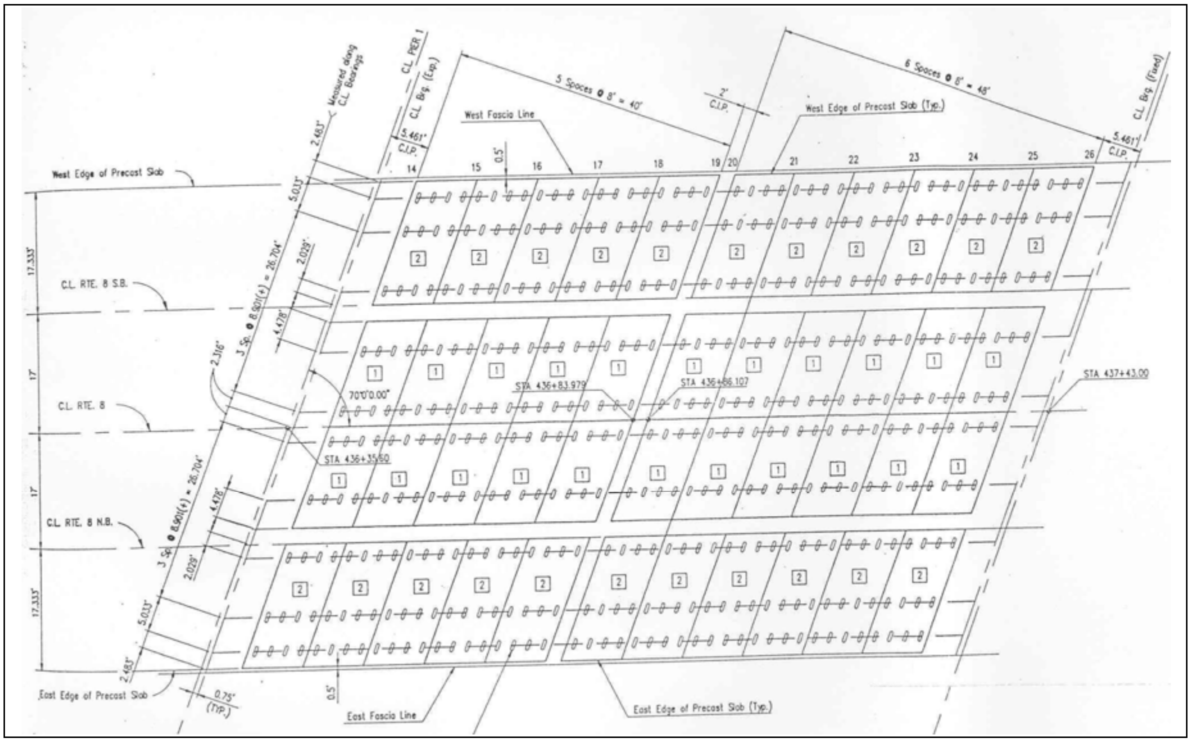

Fig. 30. Typical precast slab layout plan for one bridge in Seymour, CT (CT-DOT, 1992)

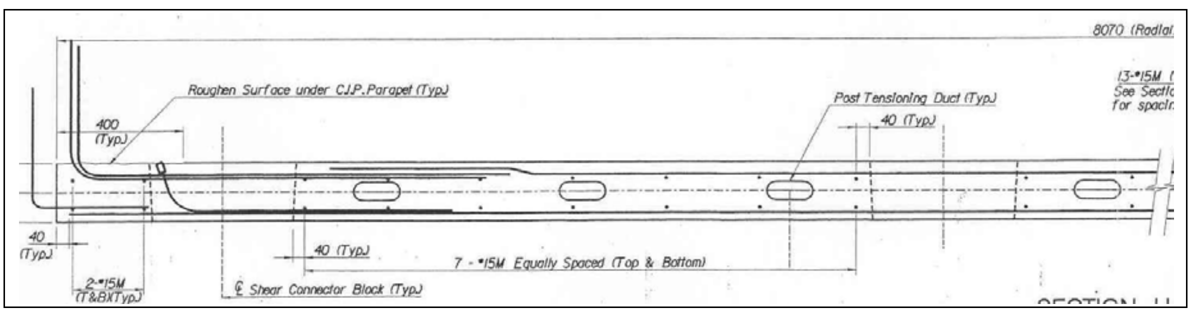

Fig. 31. Typical deck transverse reinforcement for a bridge in West Hartford, CT (CT-DOT, 1997)

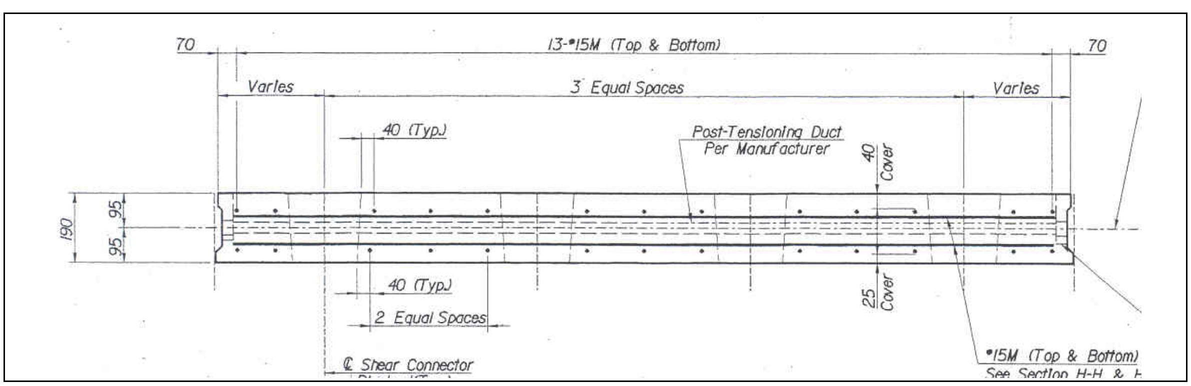

Fig. 32. Typical deck longitudinal reinforcement for a bridge in West Hartford, CT-DOT, 1997)

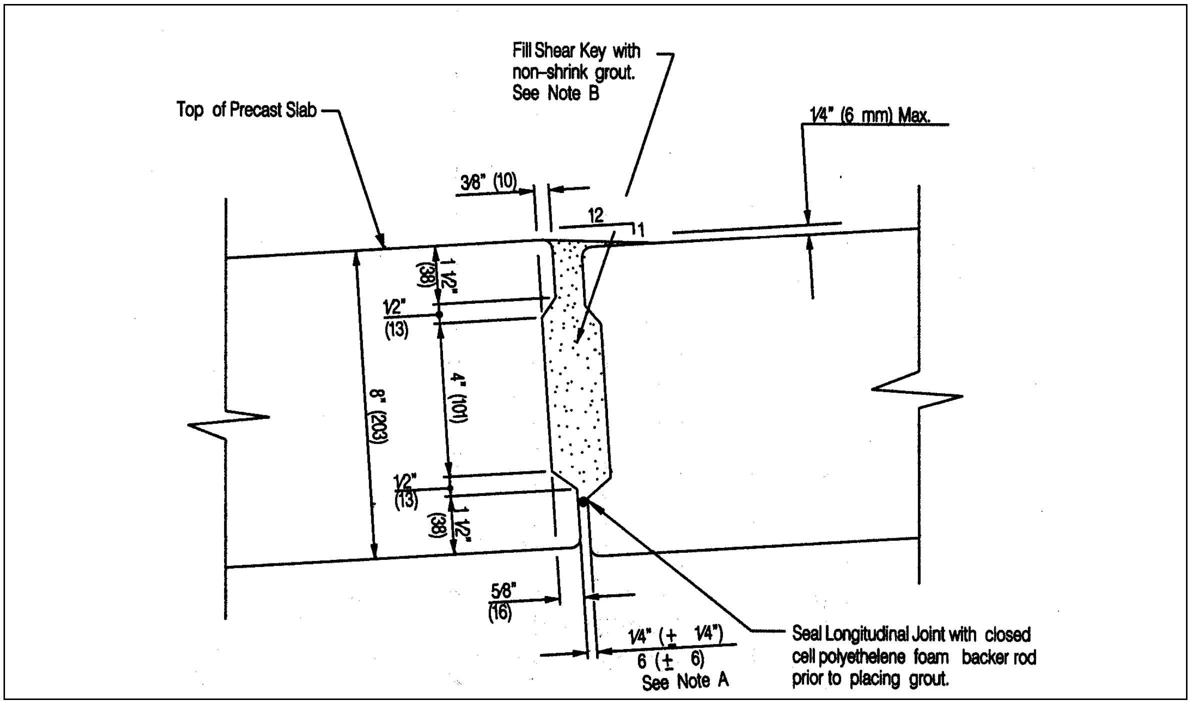

Fig. 33. Typical transverse shear key section (PCINER, 2002)

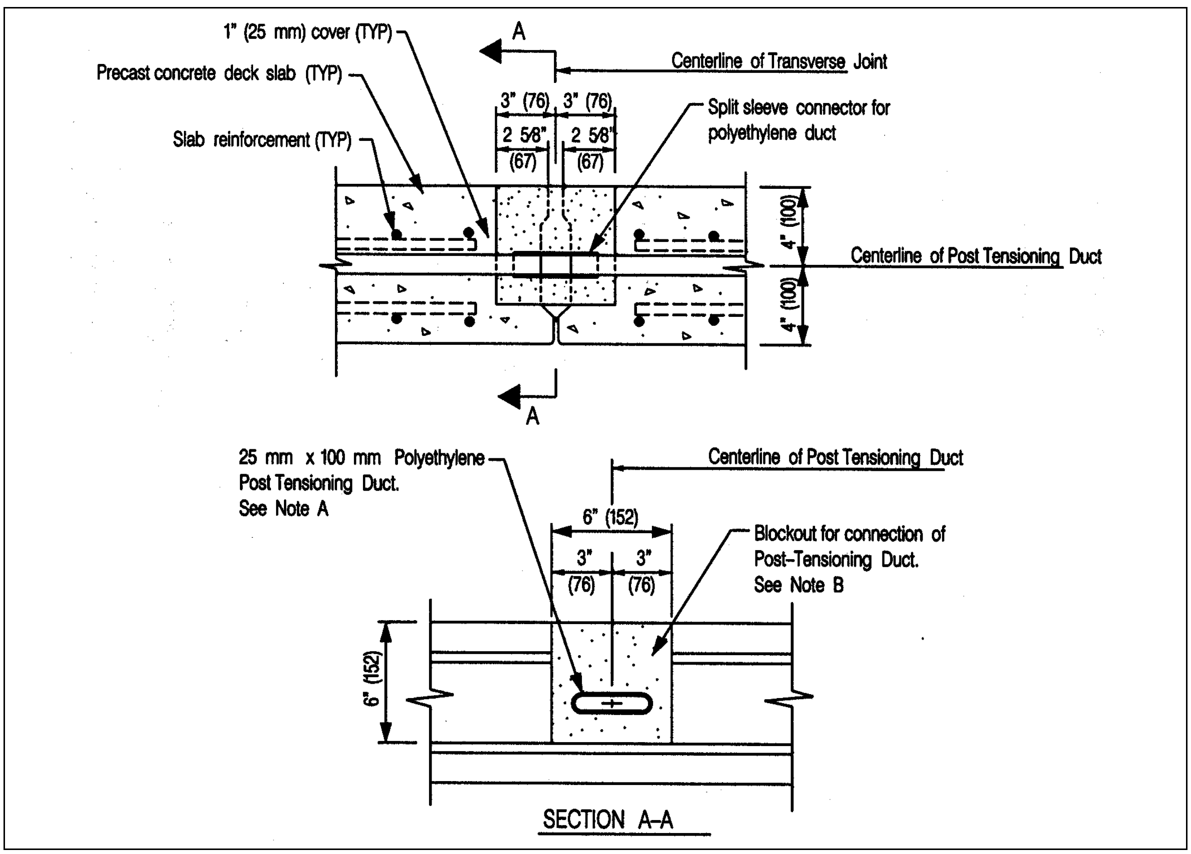

Fig. 34. Typical transverse joint section at post-tensioning duct (PCINER, 2002)

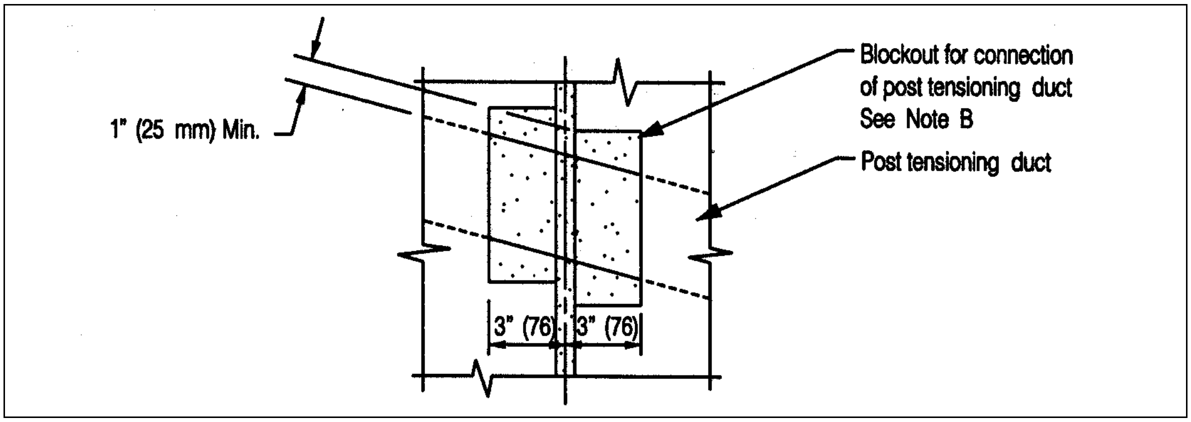

Fig. 35. Block-out for post-tensioning duct splice plan (PCINER, 2002)

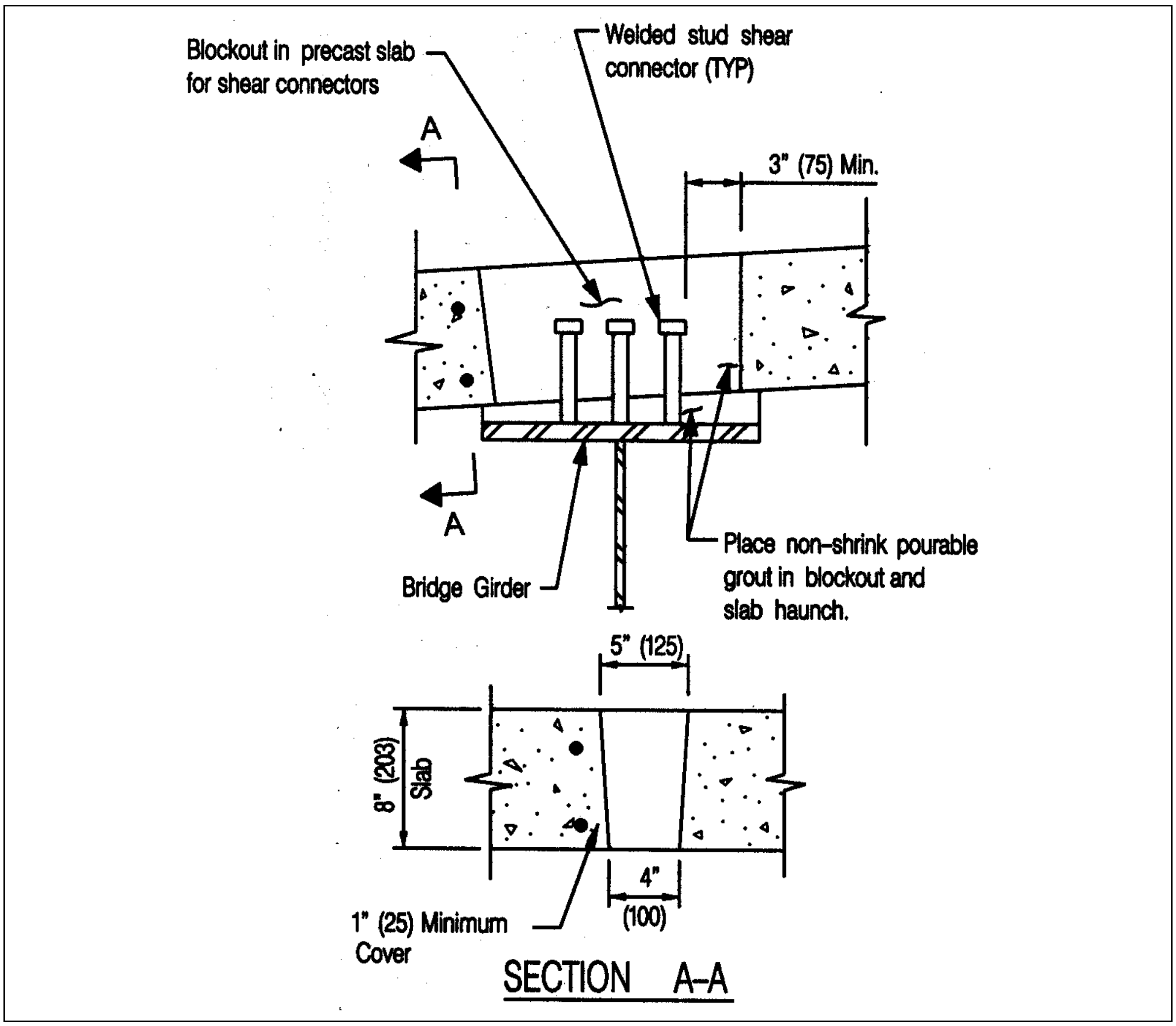

Fig. 36. Typical block-out for connectors section (PCINER, 2002)

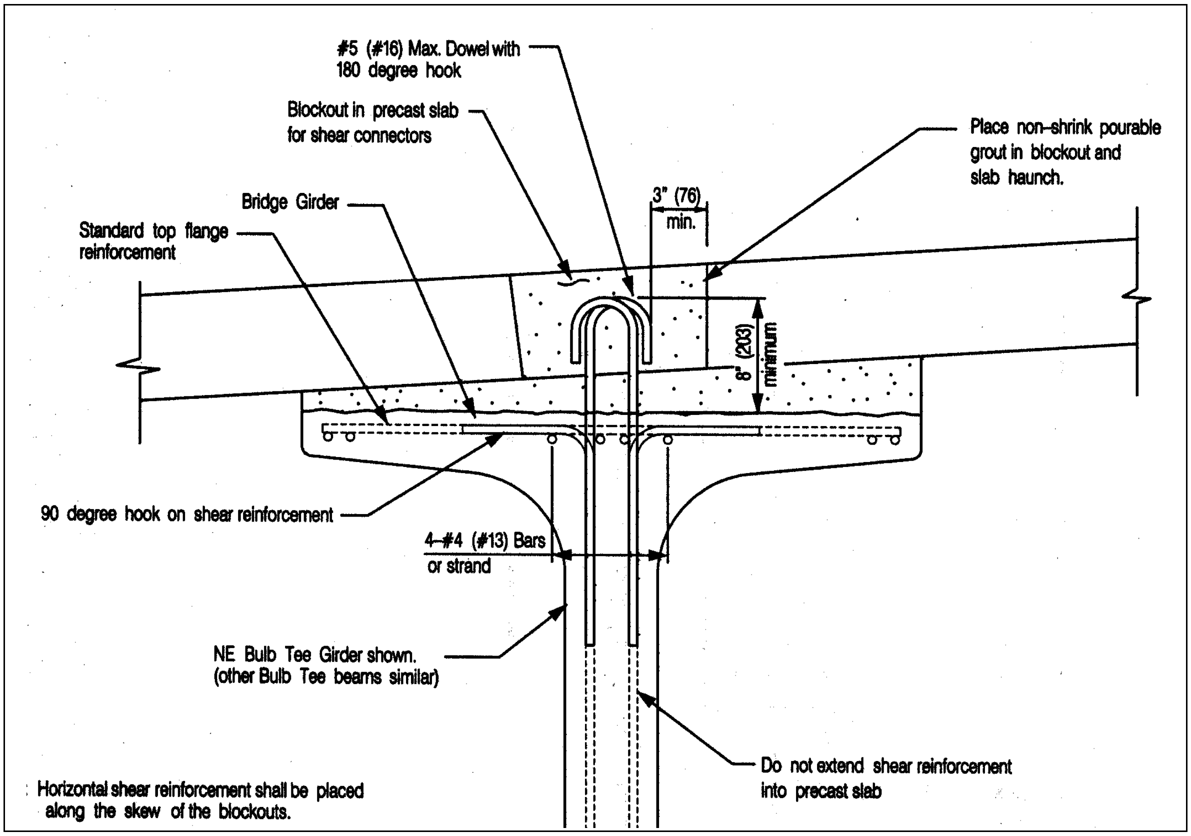

Fig. 37. Typical precast concrete girder/deck connection for new construction (PCINER, 2002)

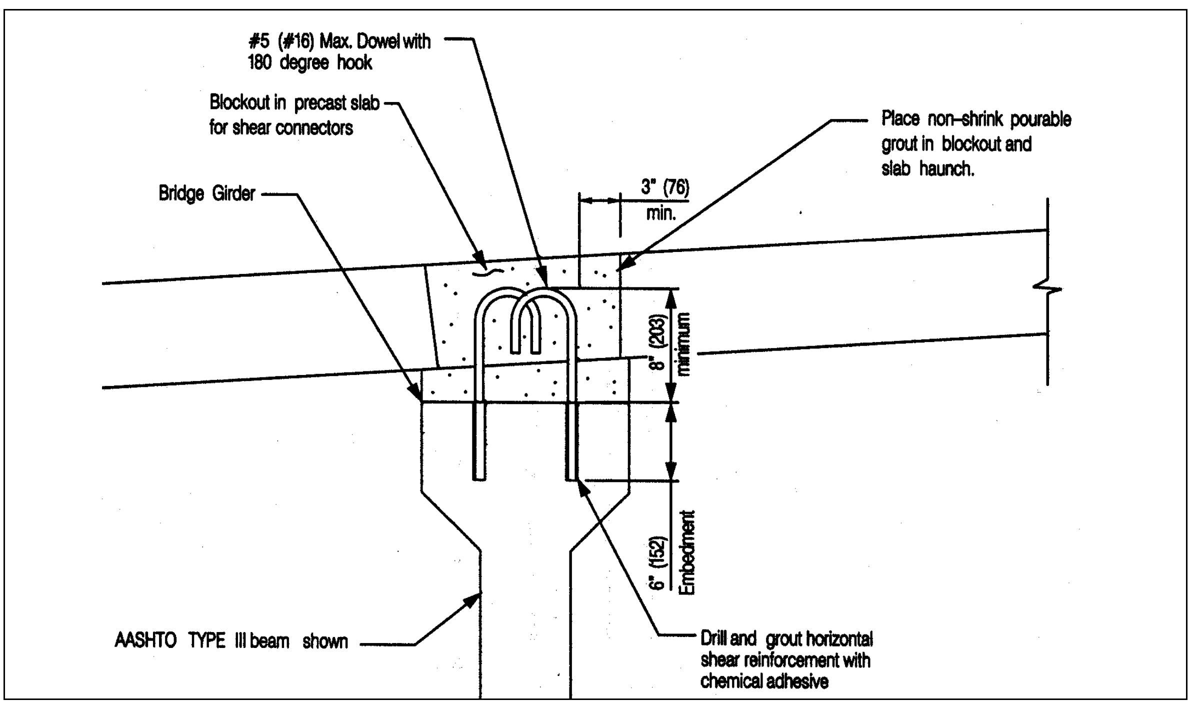

Fig. 38. Typical existing concrete girder/deck connection (PCINER, 2002)

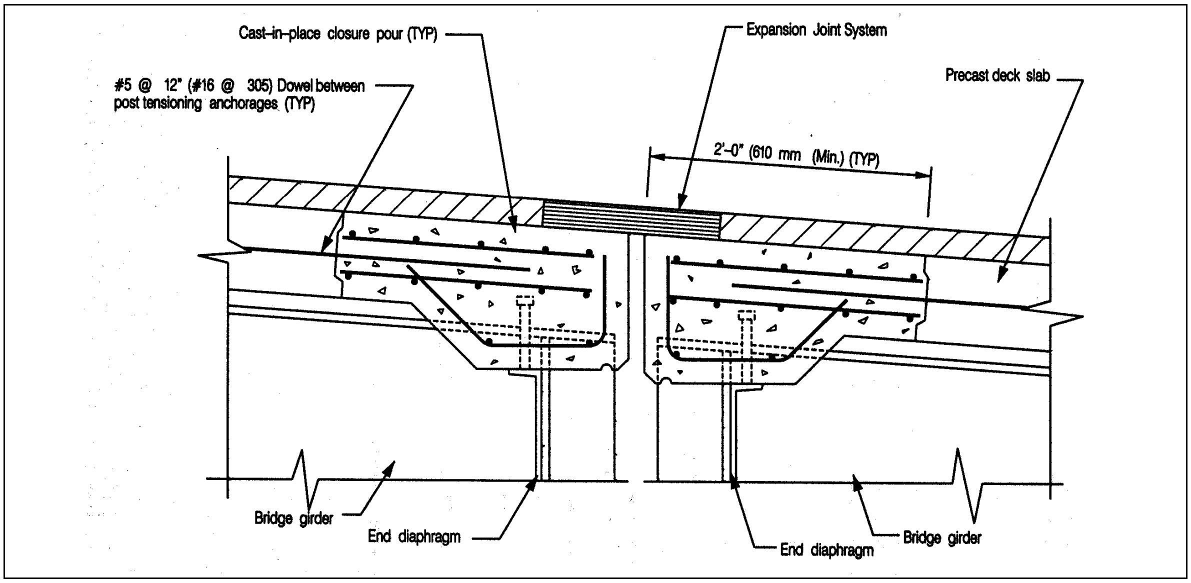

Fig. 39. Typical closure pour at deck ends (PCINER, 2002)

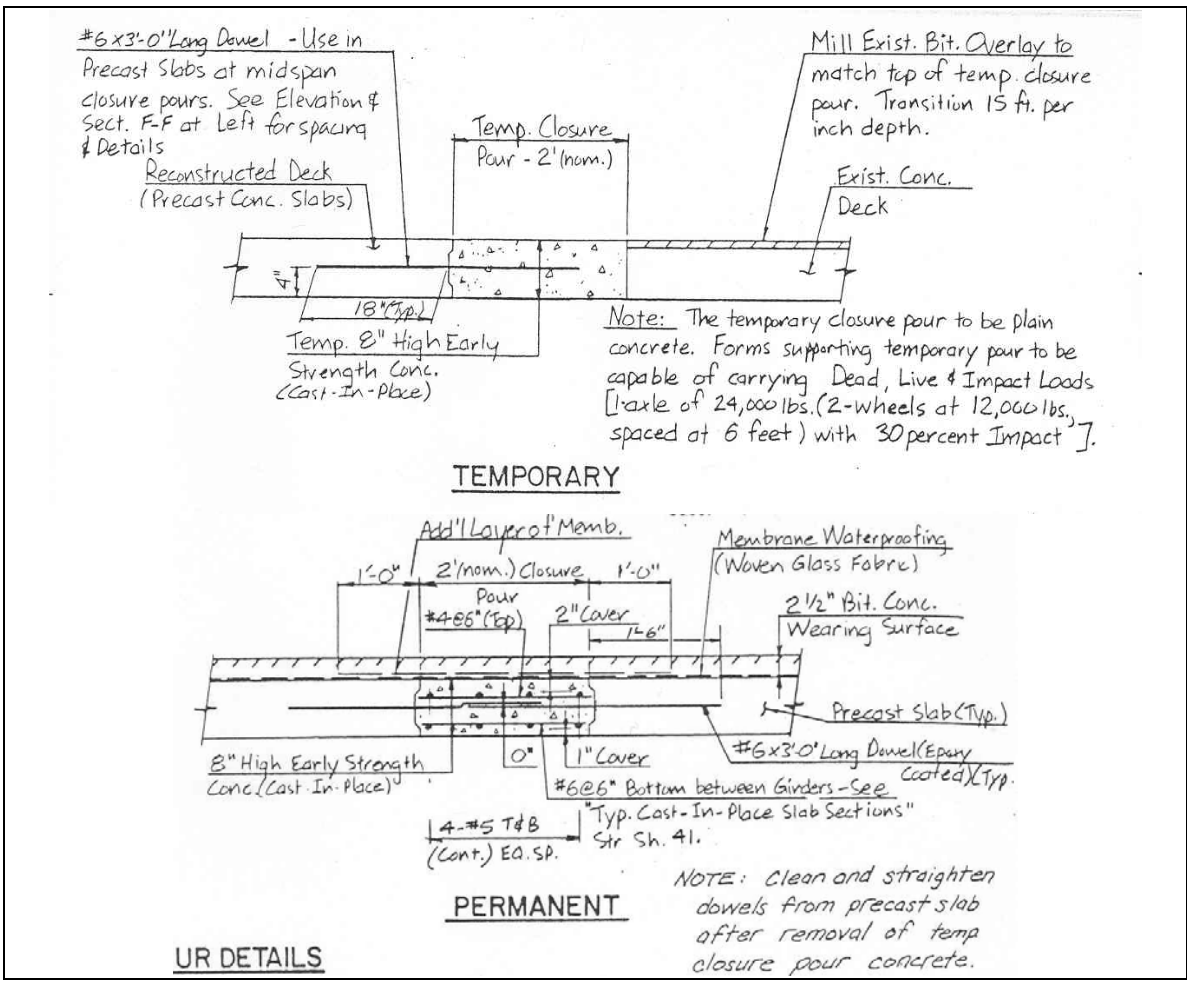

Fig. 40. Detail for longitudinal segmental construction for a bridge in Seymour, CT (CT-DOT, 1992)

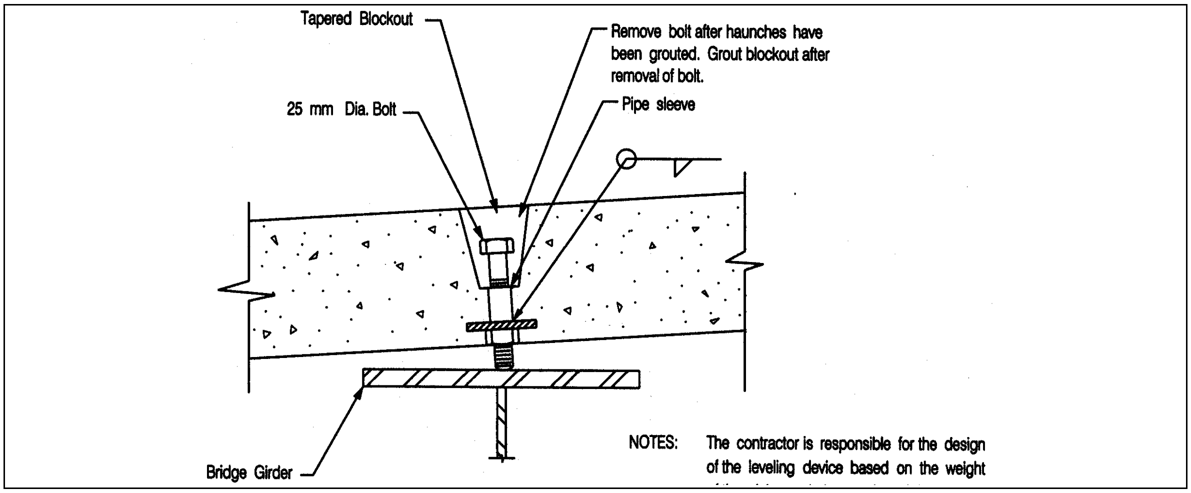

Fig. 41. Typical leveling device detail (PCINER, 2002)

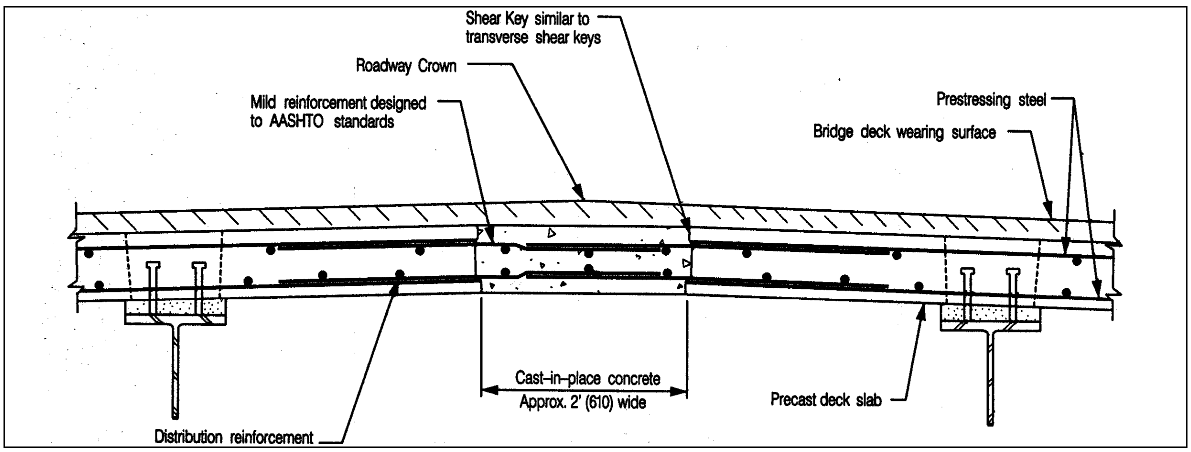

Fig. 42. Typical roadway crown section (PCINER, 2002)

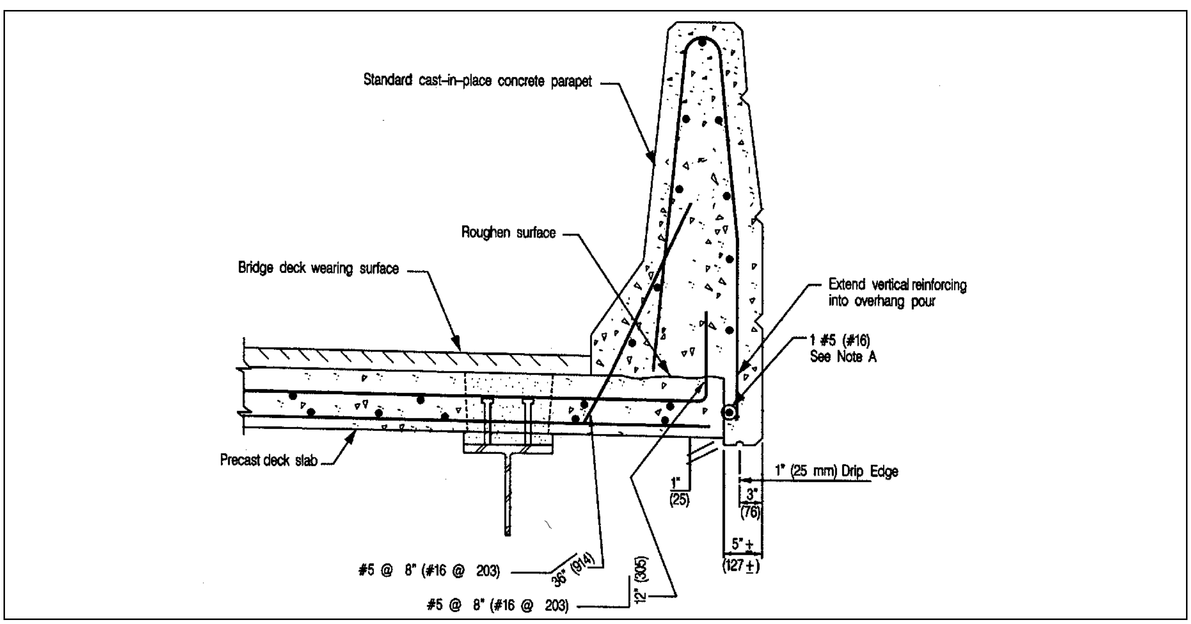

Fig. 43. Typical deck overhang and parapet section (PCINER, 2002)

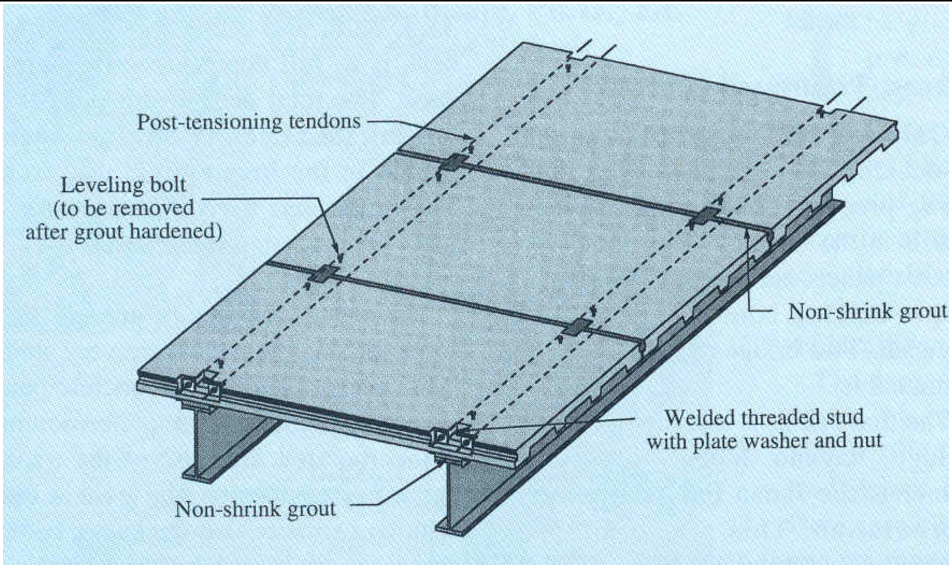

Fig. 44. Full-depth precast, prestressed deck system (Yamane et al., 1998)

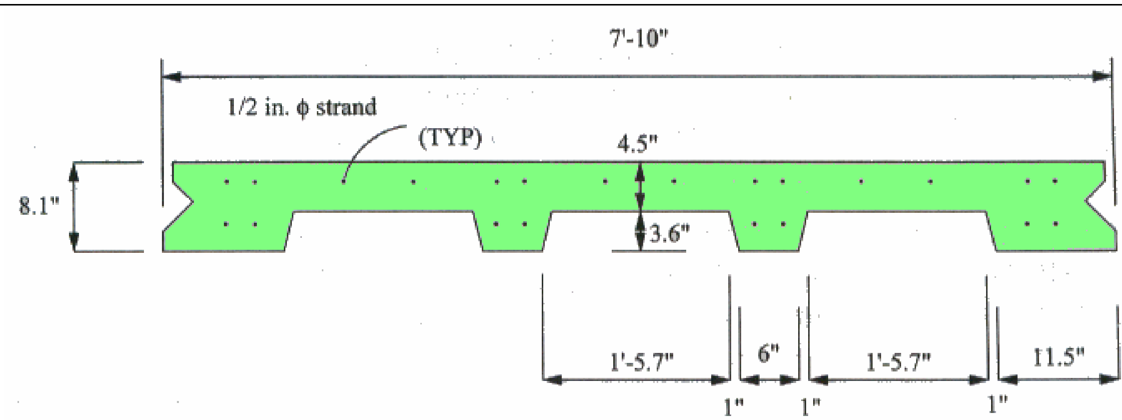

Fig. 45. Typical full-depth precast, prestressed panel section (Tadros and Baishya, 1998)

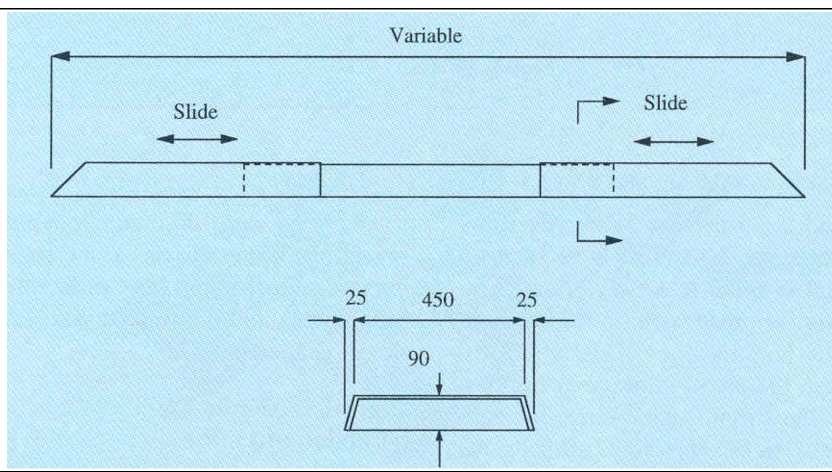

Fig. 46. Adjustable void form (Yamane et al., 1998)

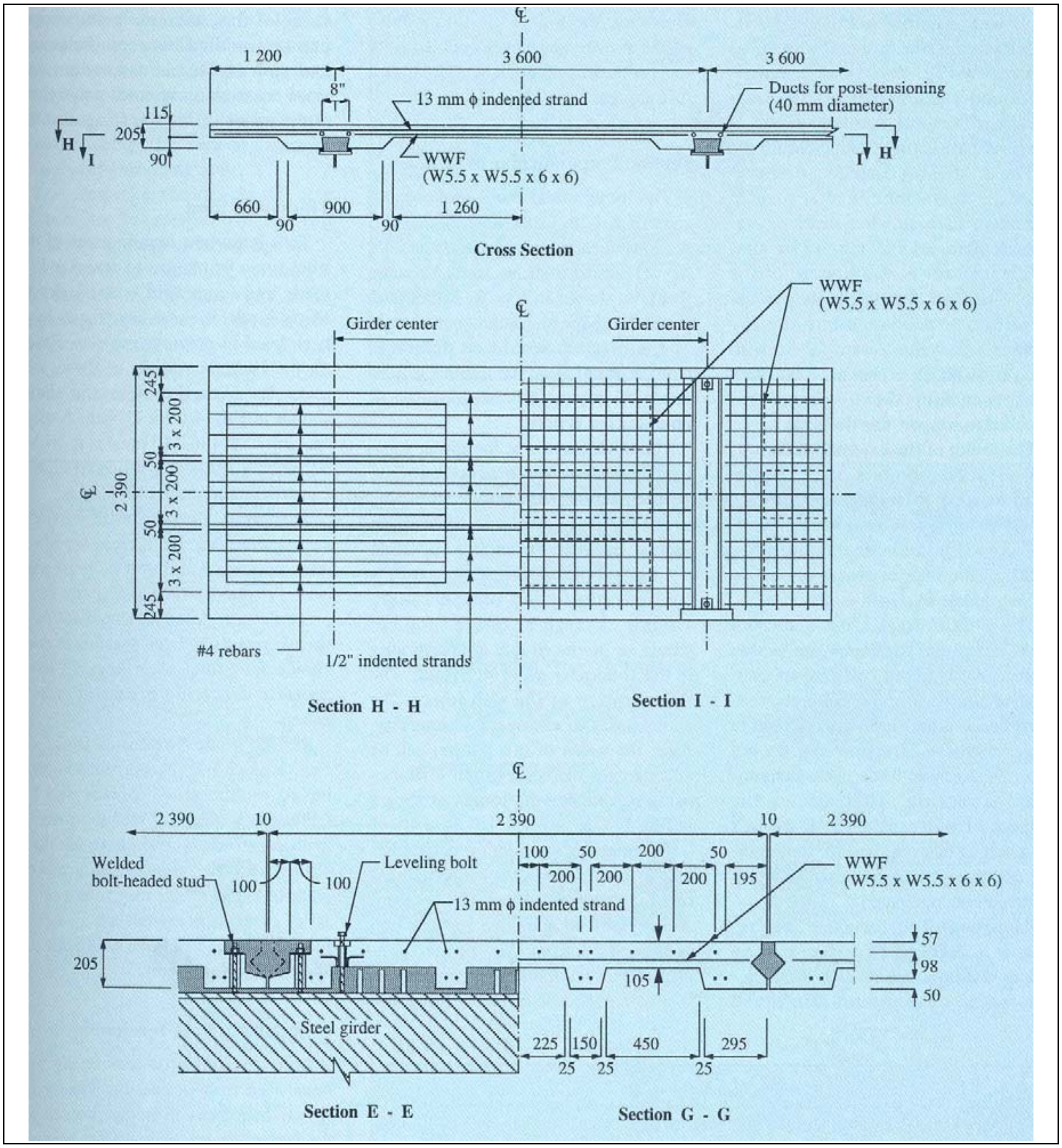

Fig. 47. Full-depth precast, prestressed deck system general view and steel arrangement (Yamane et al., 1998)

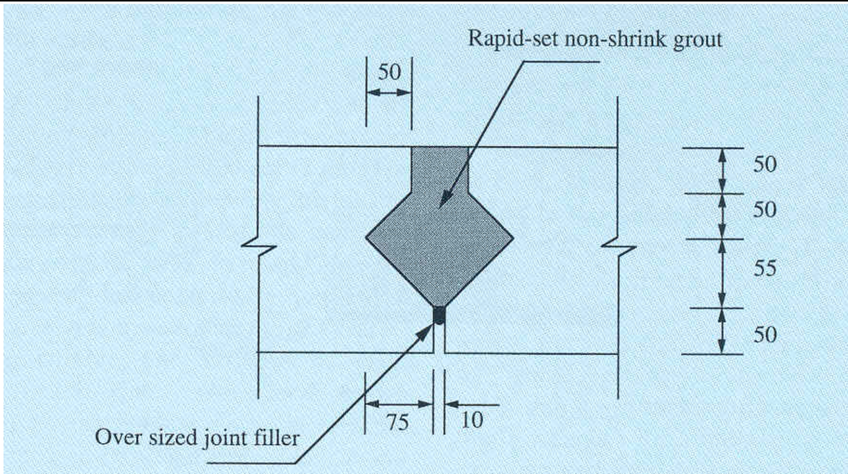

Fig. 48. Typical transverse joint shear key section (Yamane et al., 1998)

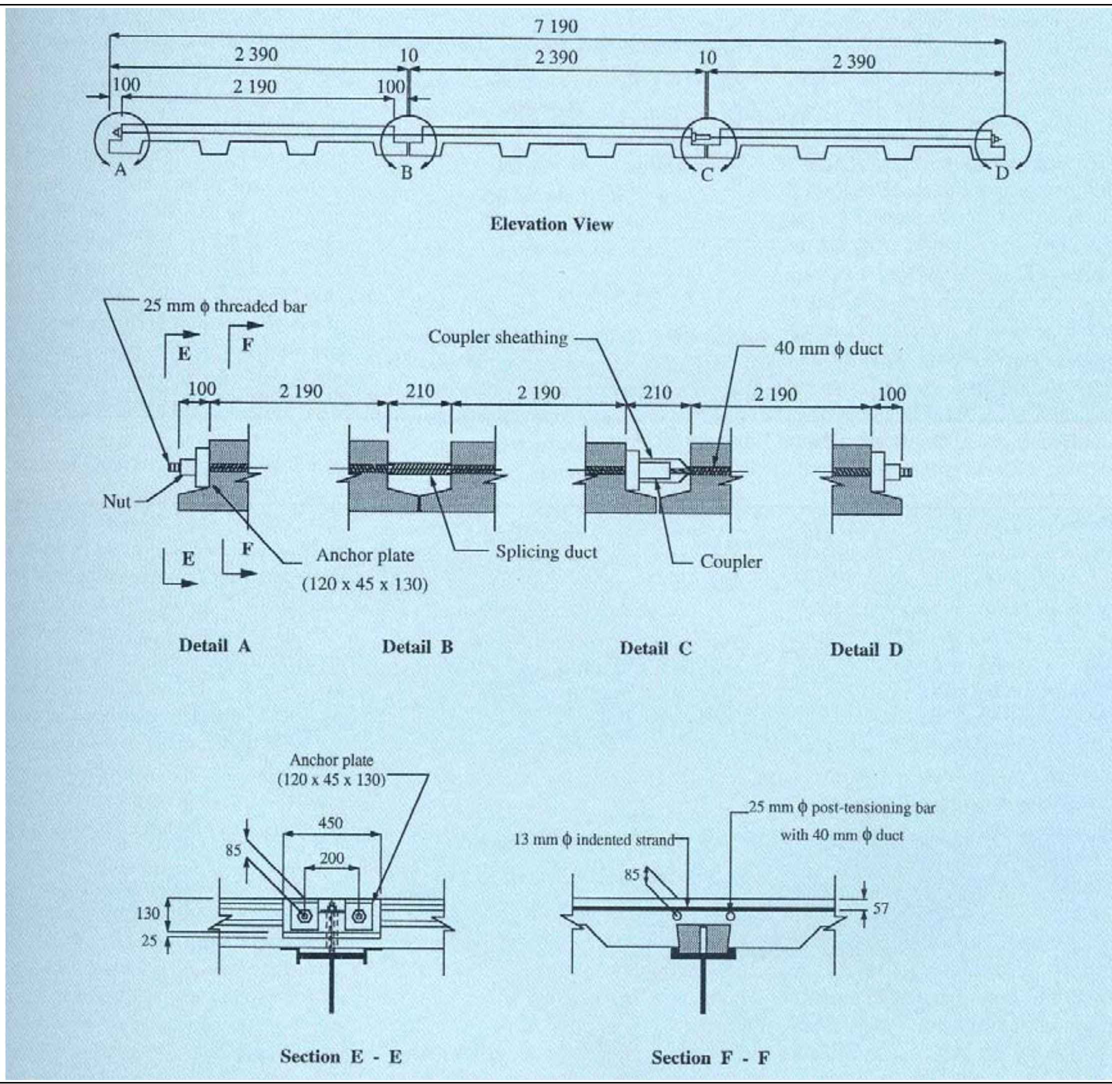

Fig. 49. Typical details for longitudinal post-tensioning (Yamane et al., 1998)

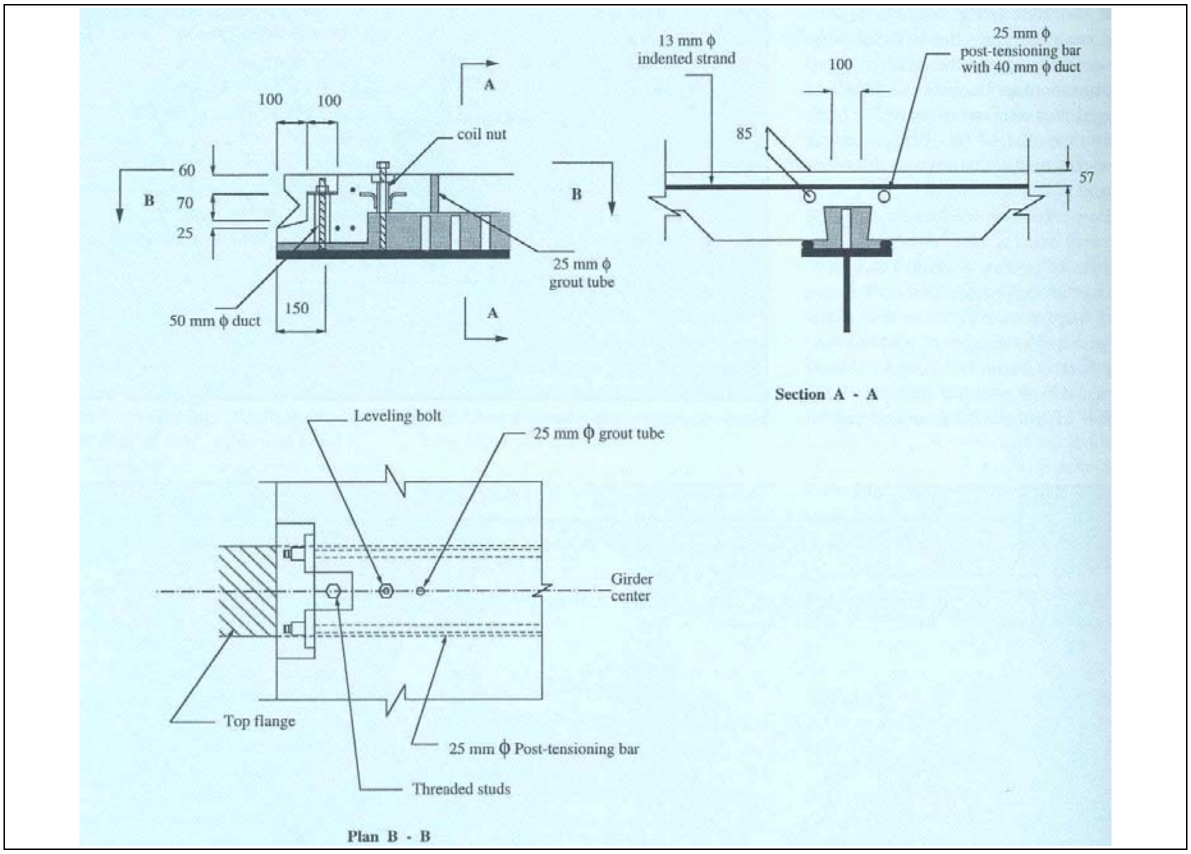

Fig. 50. Connection details at a girder location (Yamane et al., 1998)

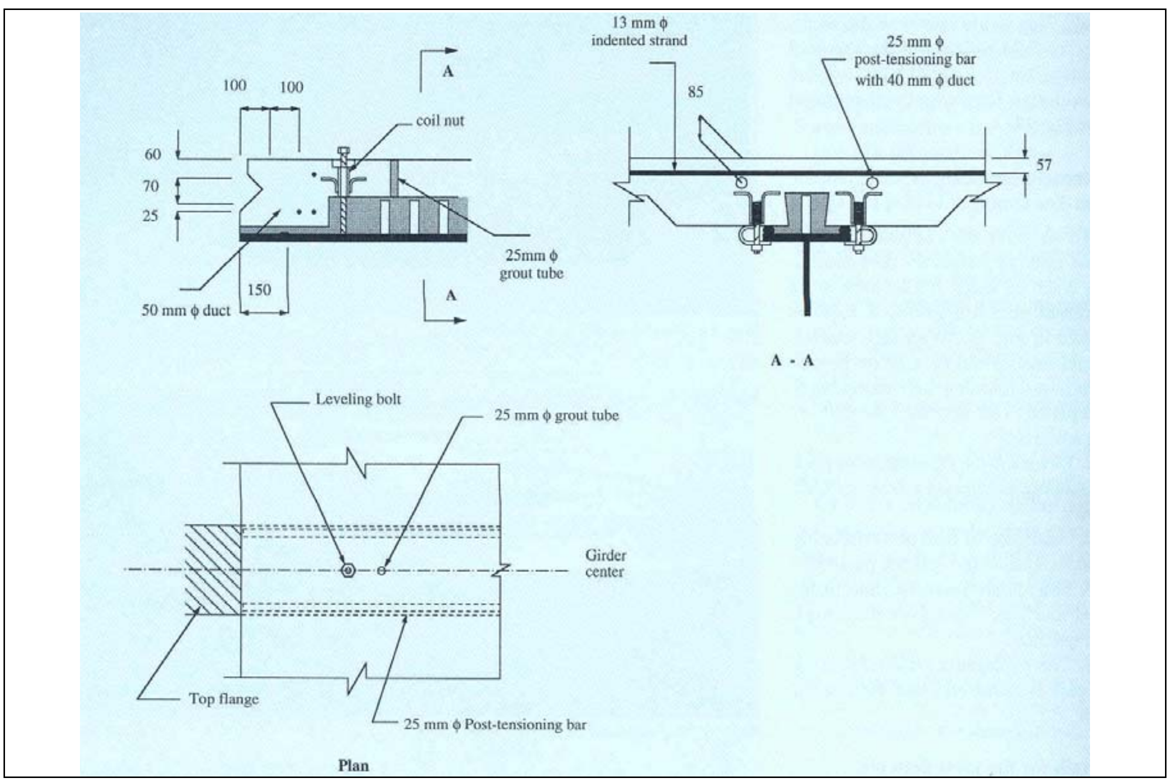

Fig. 51. Modified connection details at girder location (Yamane et al., 1998)

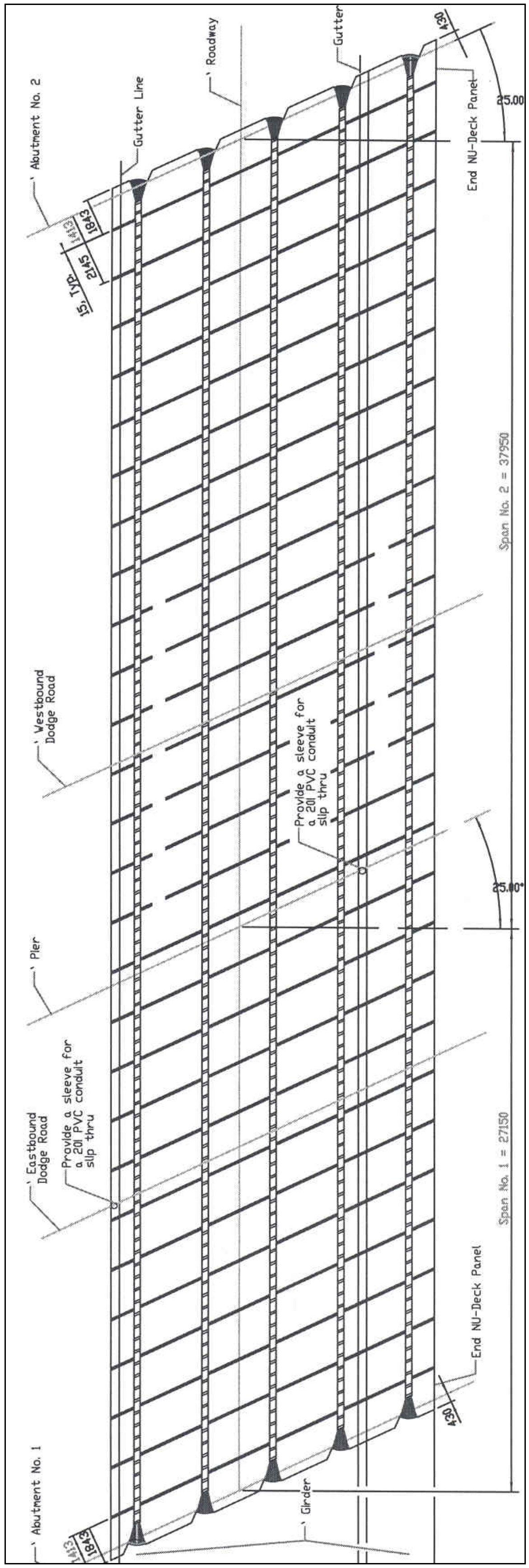

Fig. 52. NU-Deck panel layout for the Skyline Bridge (NDOR, 2002)

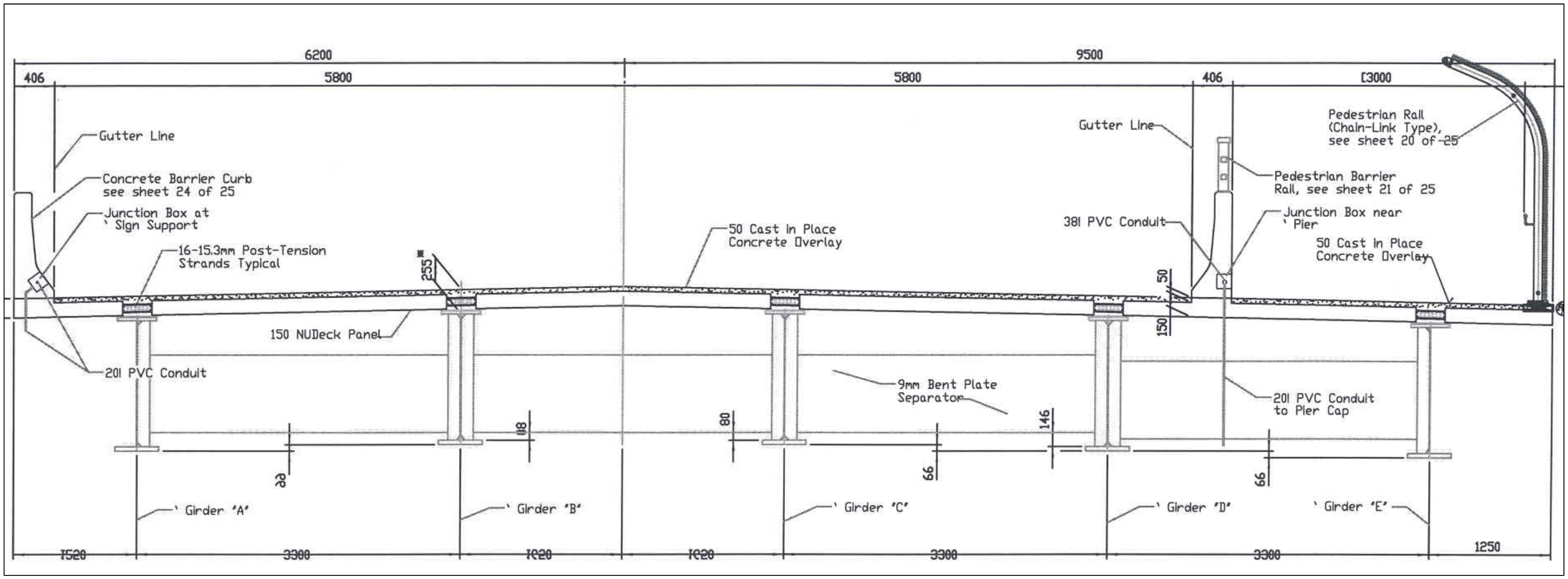

Fig. 53. Typical roadway cross section for the Skyline Bridge (NDOR, 2002)

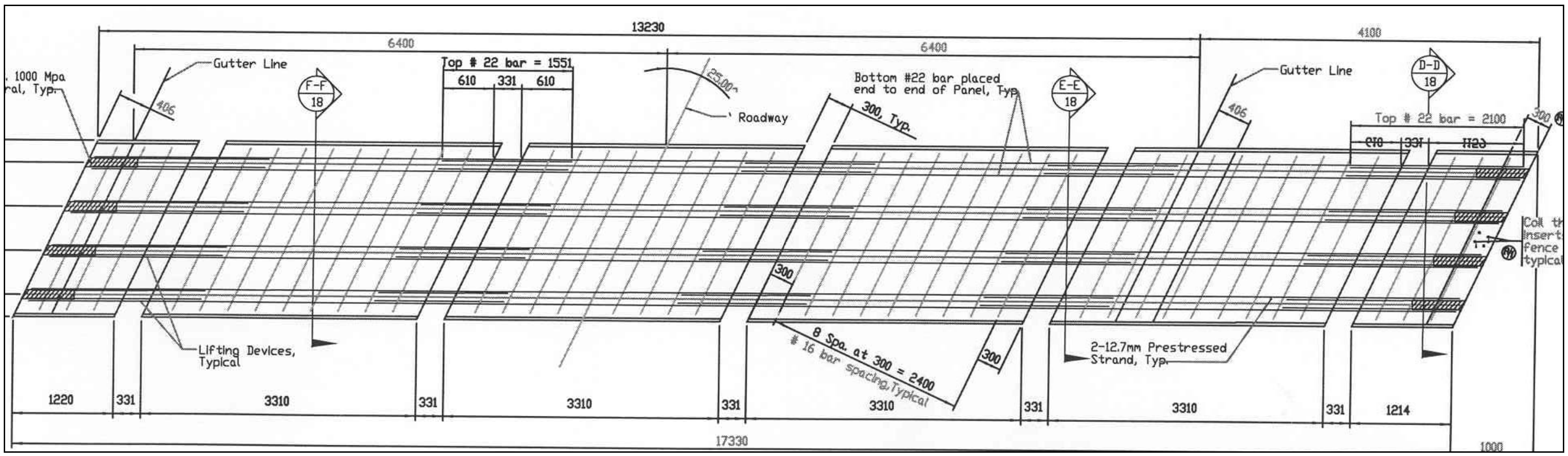

Fig. 54. Typical NU-Deck panel plan view (NDOR, 2002)

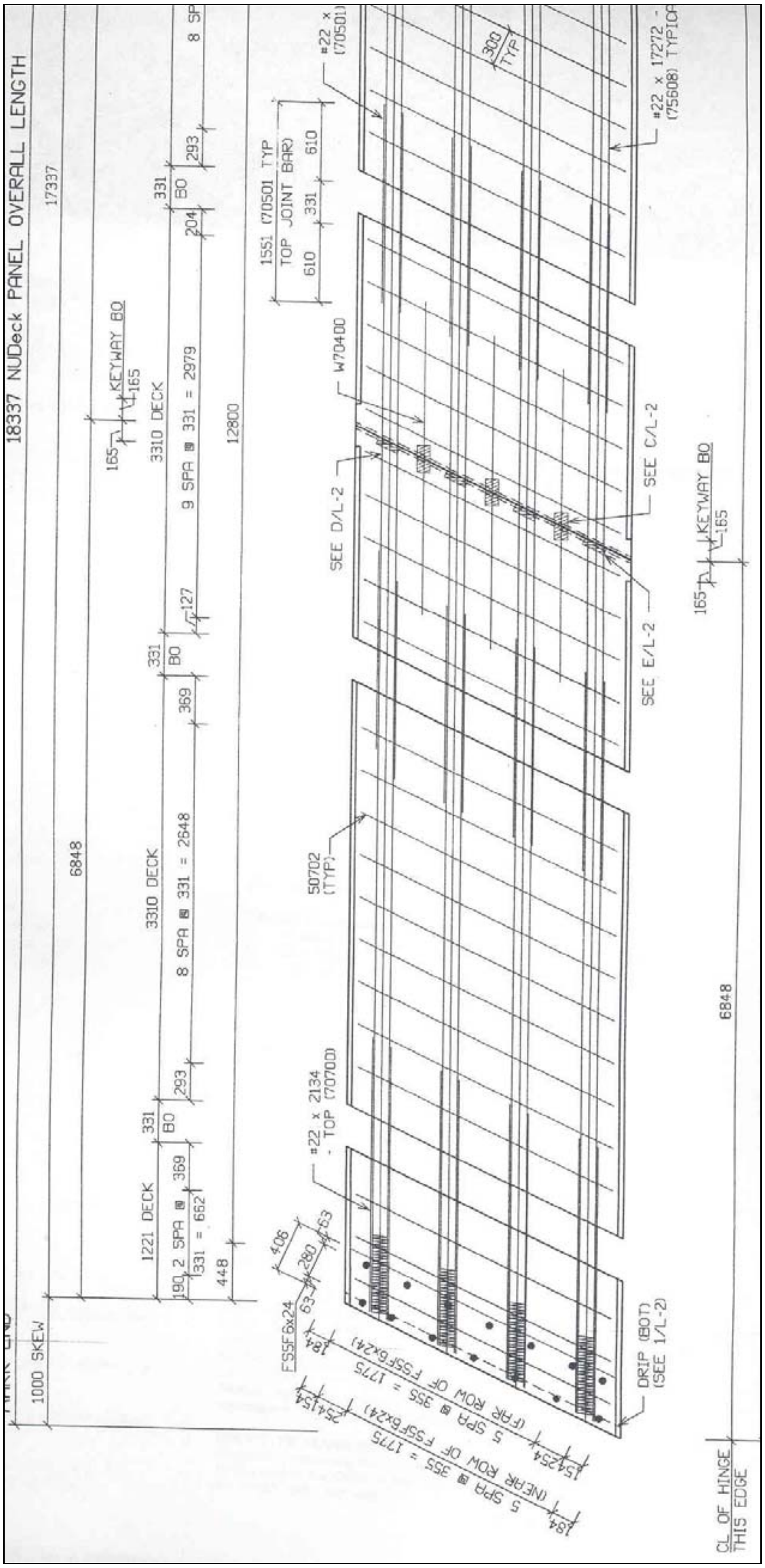

Fig. 55. Typical NU-Deck panel plan view (NDOR, 2003)

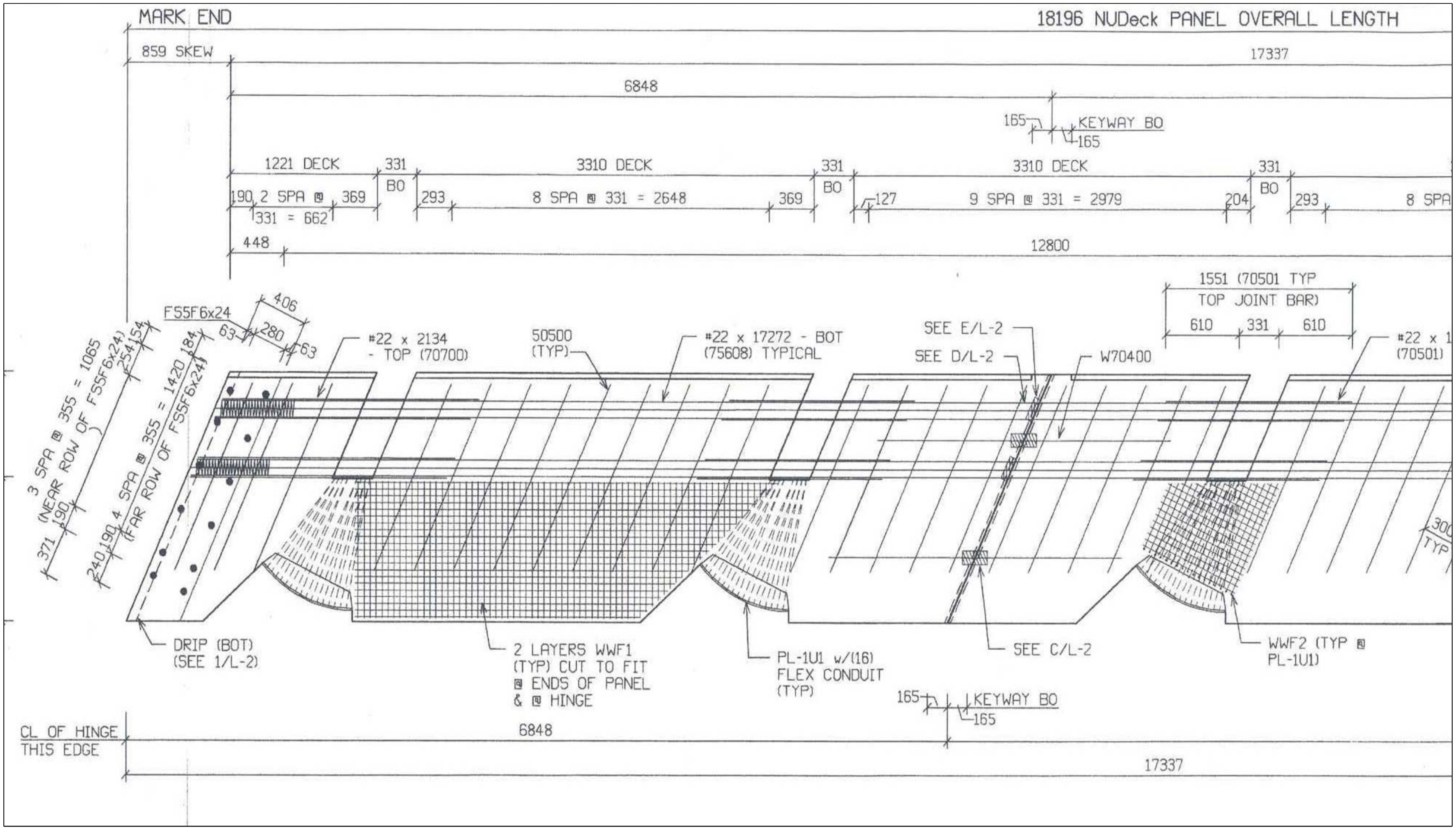

Fig. 56. NU-Deck end panel plan view (NDOR, 2003)

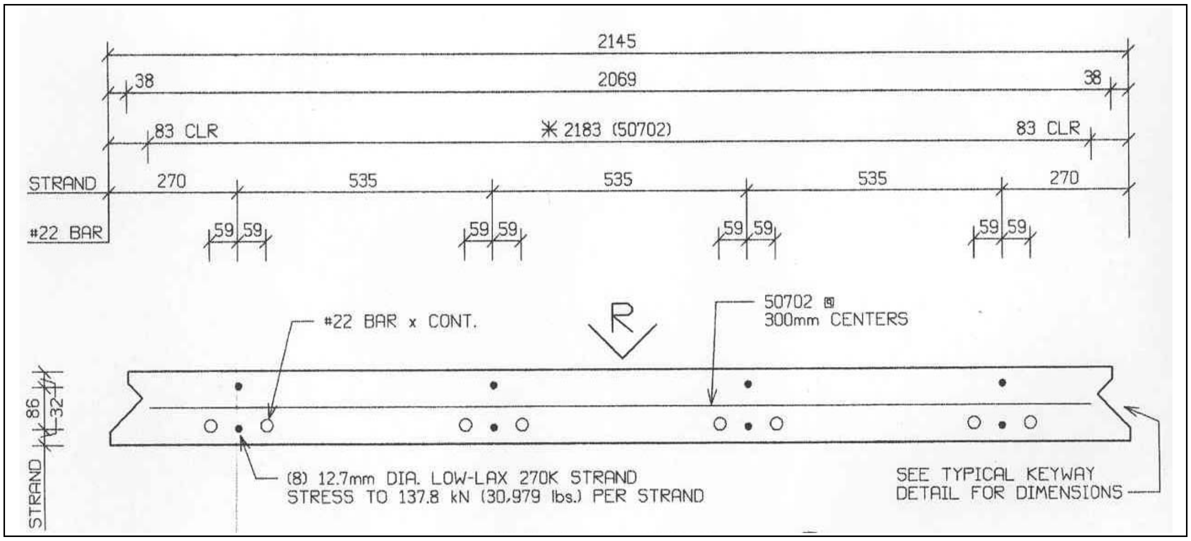

Fig. 57. Typical NU-Deck panel cross section (NDOR, 2003)

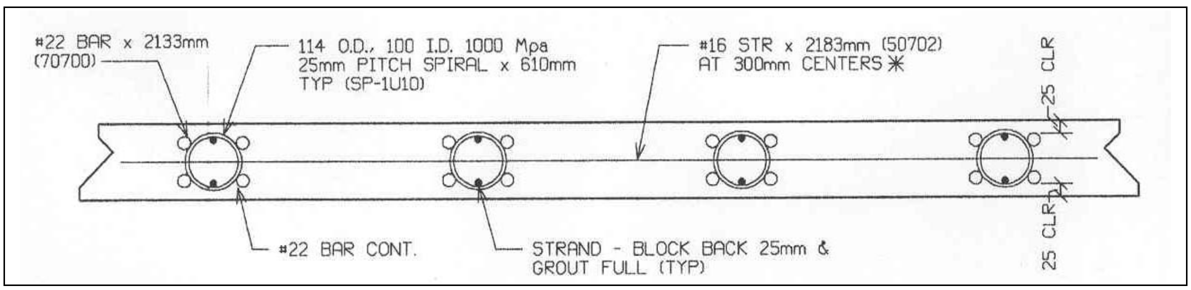

Fig. 58. Typical NU-Deck panel section at overhangs (NDOR, 2003)

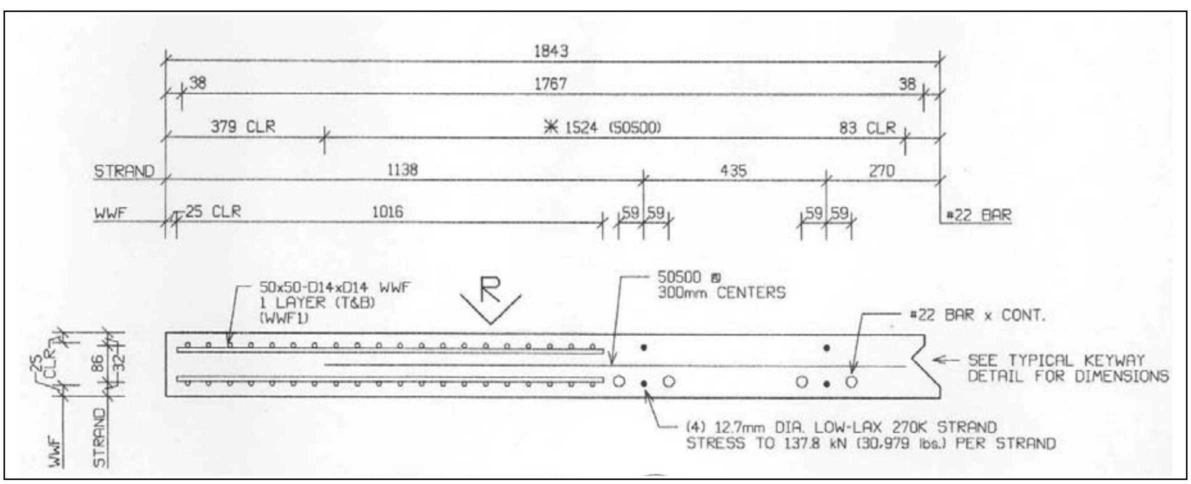

Fig. 59. NU-Deck end panel cross section (NDOR, 2003)

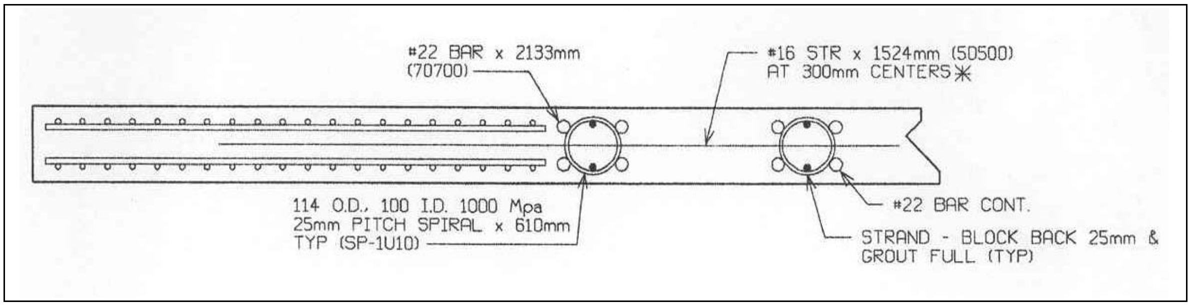

Fig. 60. NU-Deck end panel section at overhangs (NDOR, 2003)

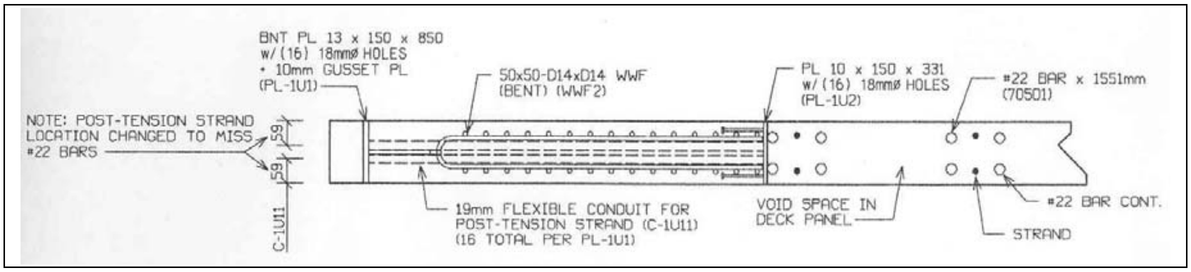

Fig. 61. NU-Deck end panel section at post-tensioning ducts (NDOR, 2003)

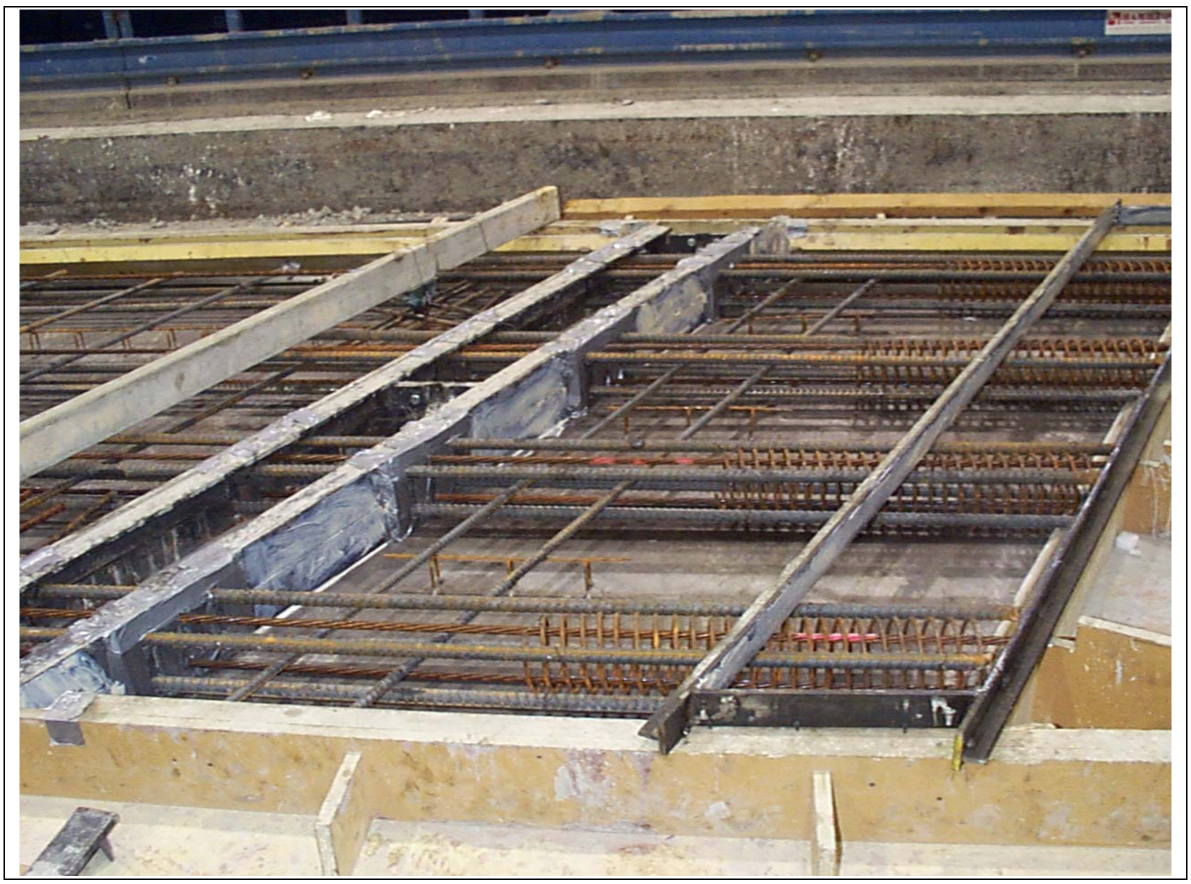

Fig. 62. Spiral confinement at panel overhang (JDV, 2003)

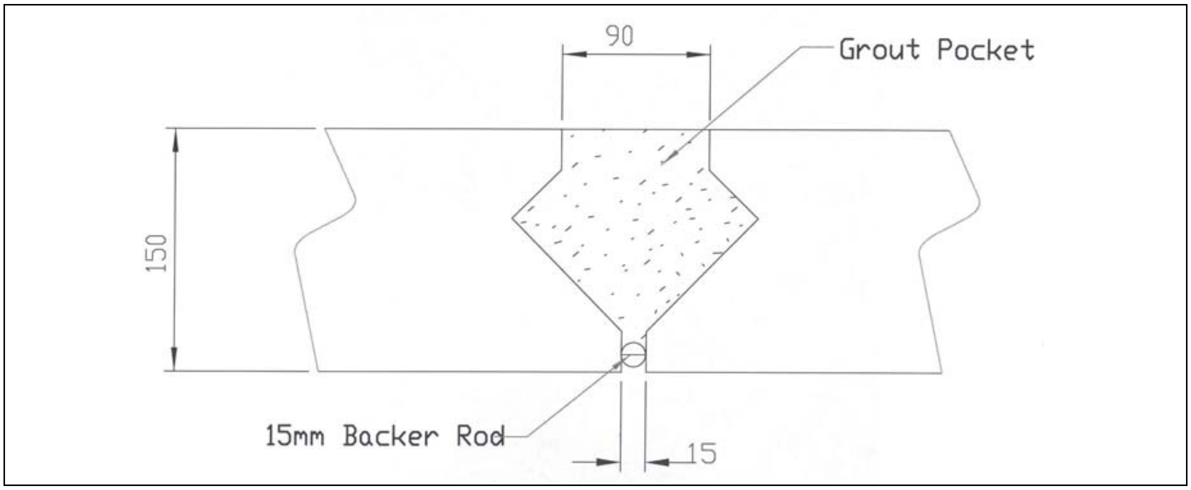

Fig. 63. Typical transverse joint detail (NDOR, 2002)

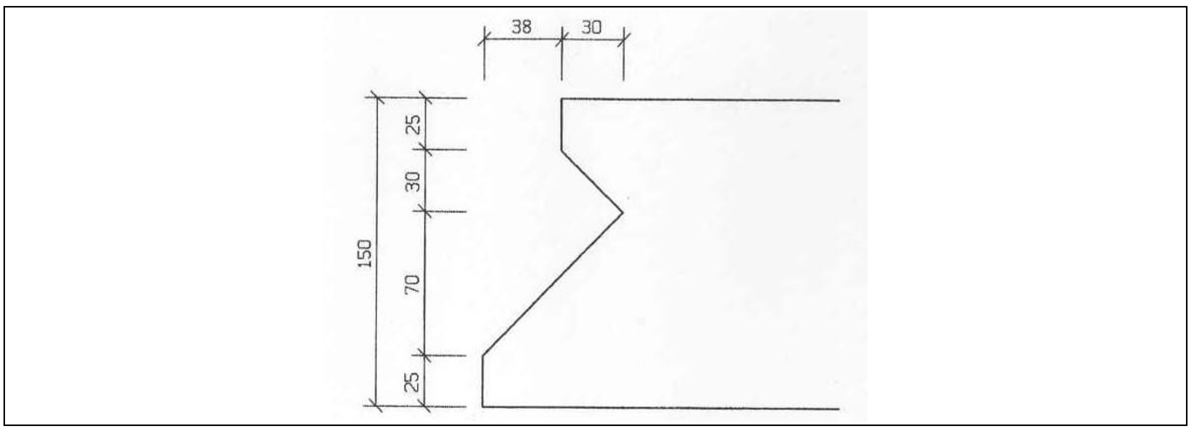

Fig. 64. Typical keyway dimensions (NDOR, 2003)

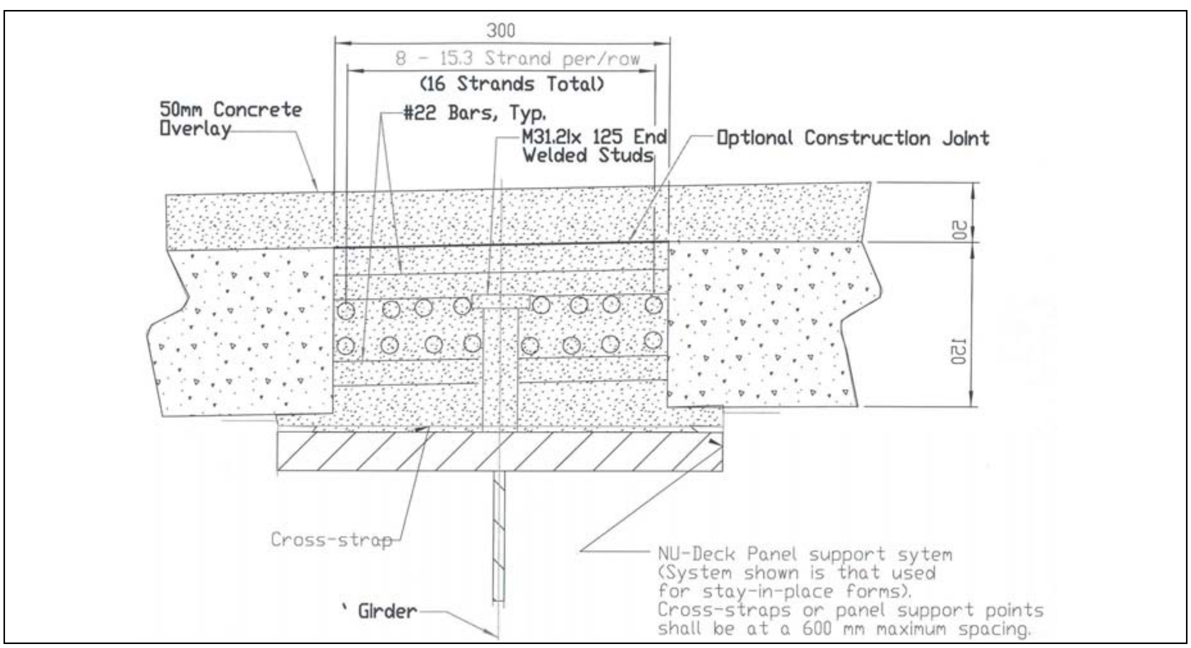

Fig. 65. Open channel and shear connectors over typical girder (NDOR, 2002)

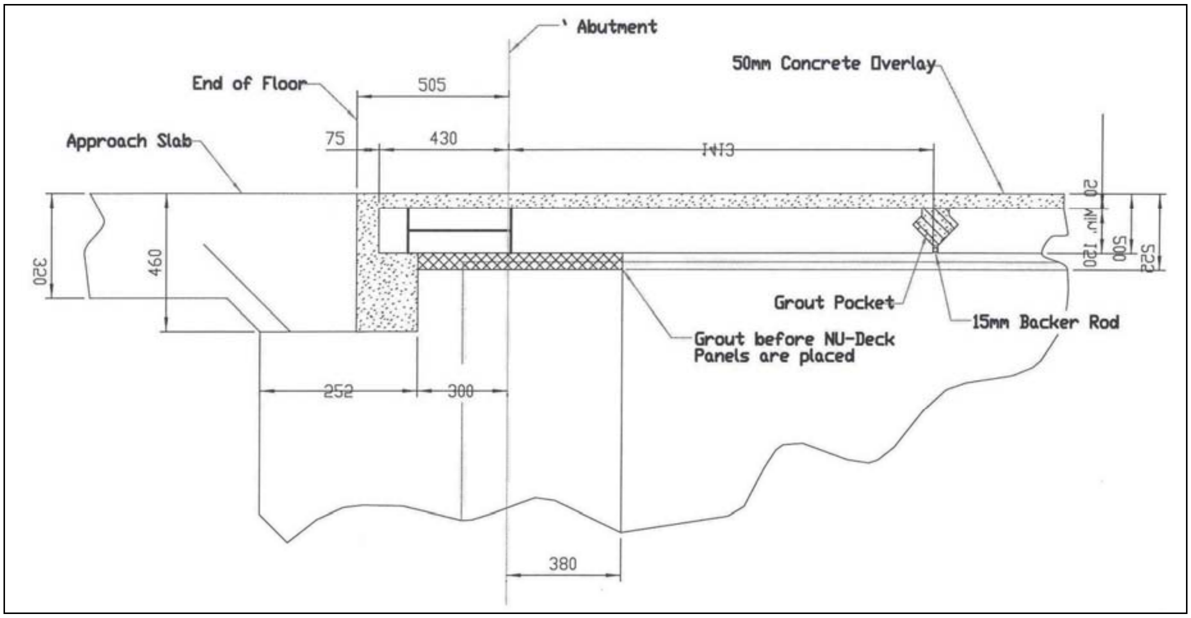

Fig. 66. Typical closure pour detail at deck ends (NDOR, 2002)

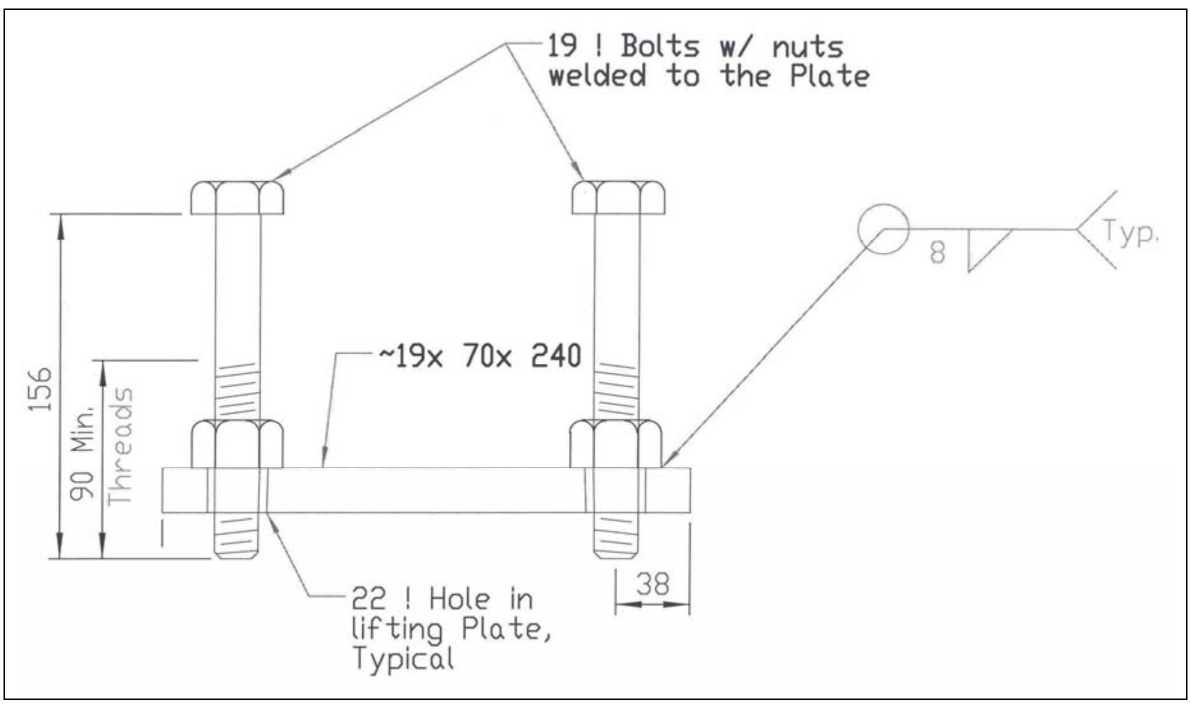

Fig. 67. Typical leveling device (NDOR, 2002)

Fig. 68. Plastic rod over fiber board at hinge section during fabrication of NU-Deck panels for the Skyline Bridge (Versace, 2003)

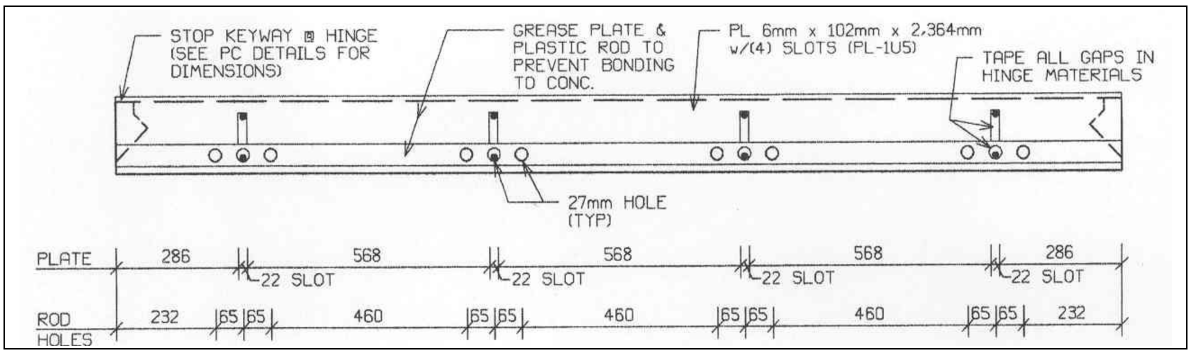

Fig. 69. Typical NU-Deck panel hinge section (NDOR, 2003)

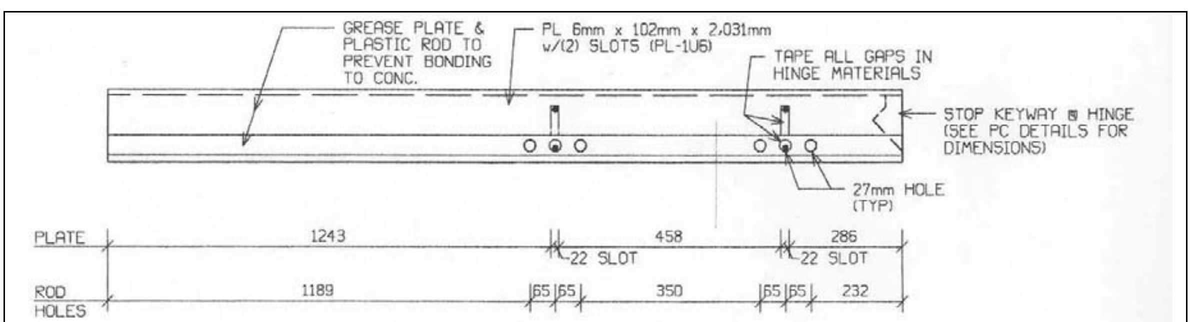

Fig. 70. NU-Deck end panel hinge section (NDOR, 2003)

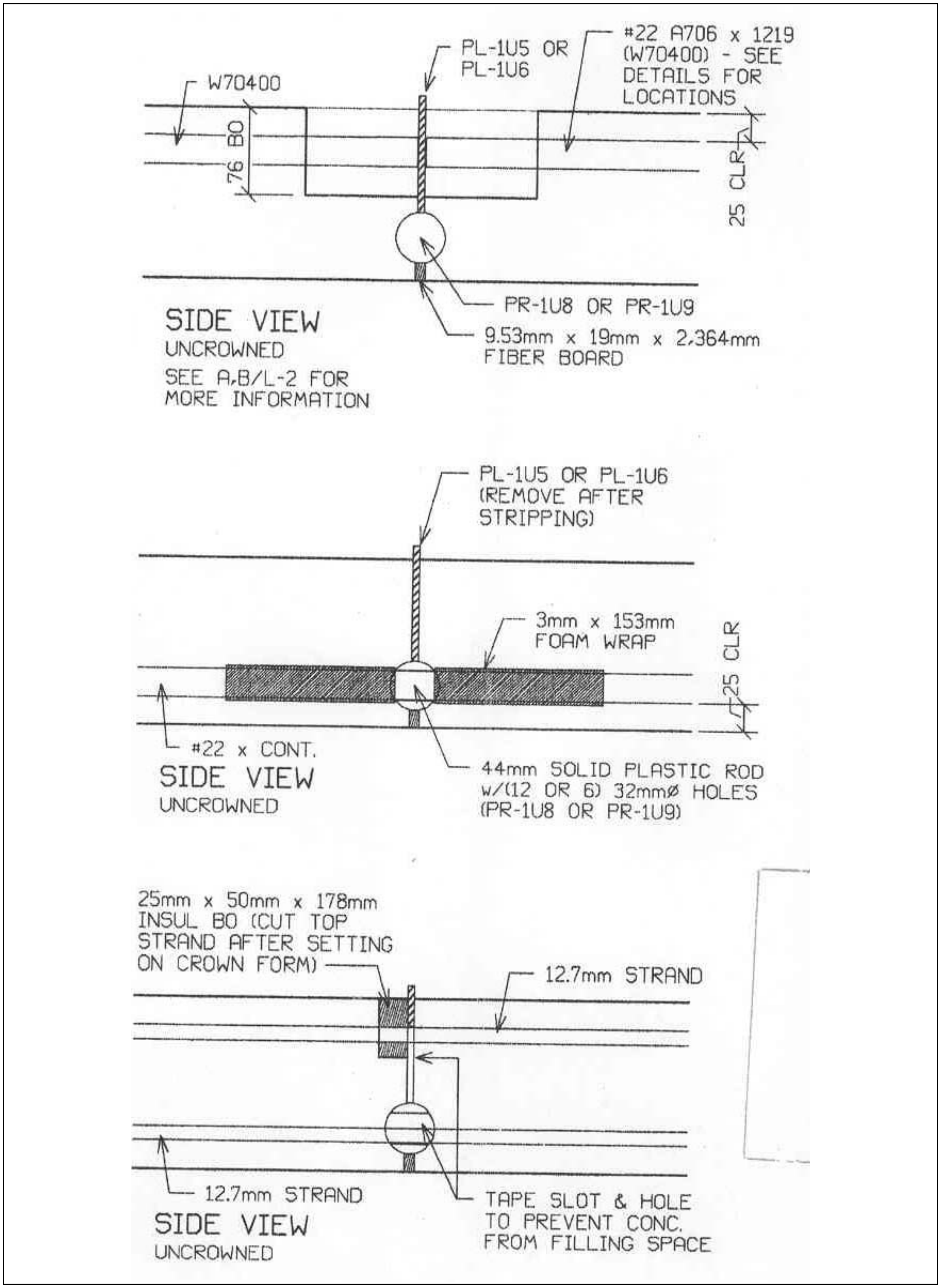

Fig. 71. Typical hinge section details (NDOR, 2003)

Fig. 72. Typical NU-Deck panel hinge section ready for concrete (Versace, 2003)

Fig. 73. Typical NU-Deck panel lifted with eight anchor devices before setting (Versace, 2003)

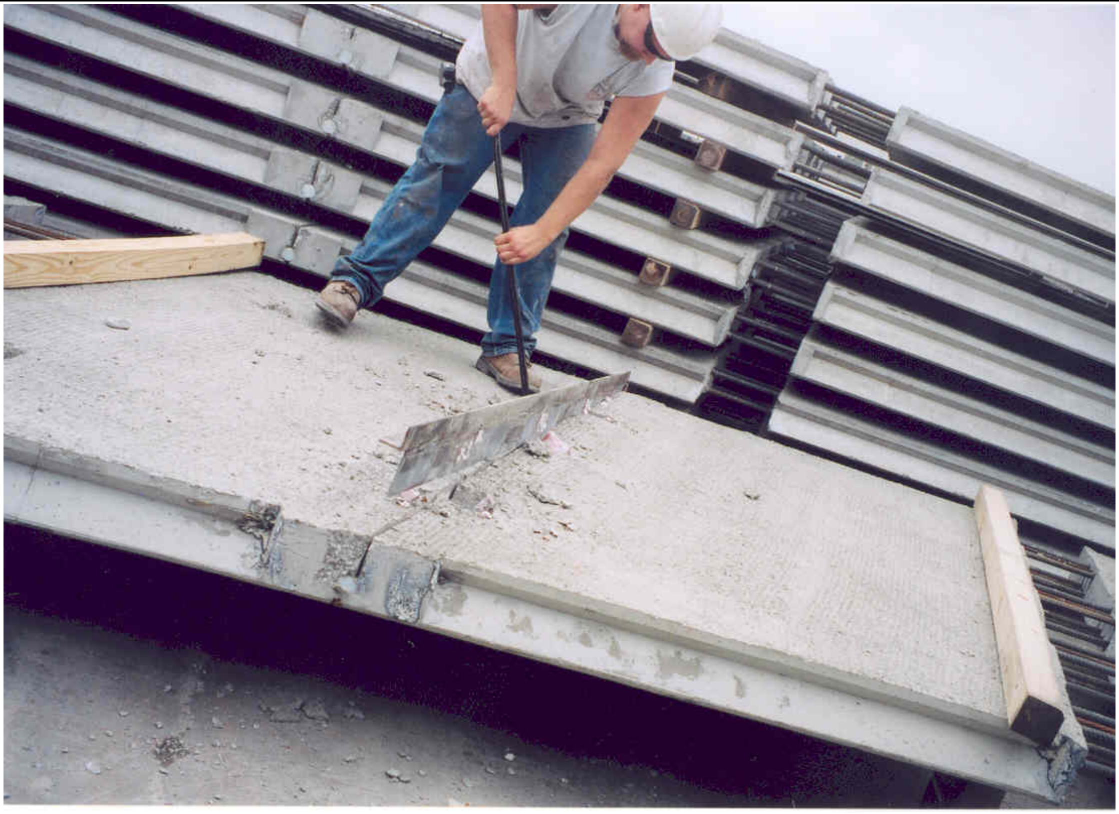

Fig. 74. Removal of plate at hinge section to form the panel crown (Versace, 2003)

Fig. 75. Cutting of top strands at hinge section to form the panel crown (Versace, 2003)

Fig. 76. Plates welded to reinforcing bars at hinge final position (Versace, 2003)

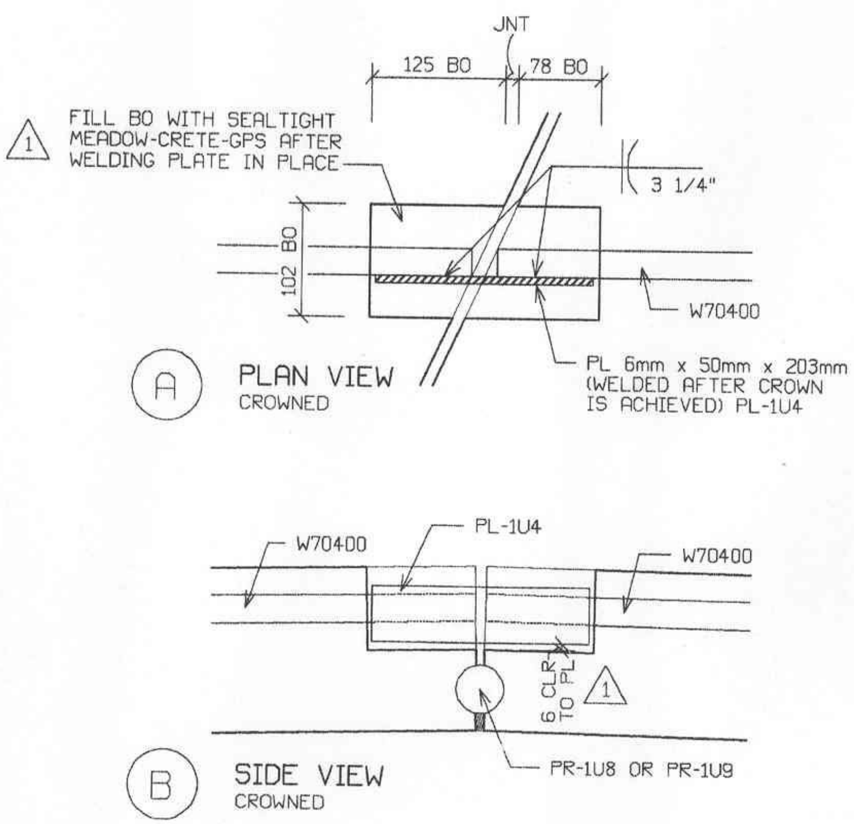

Fig. 77. Typical hinge section details after the crown is formed (NDOR, 2003)

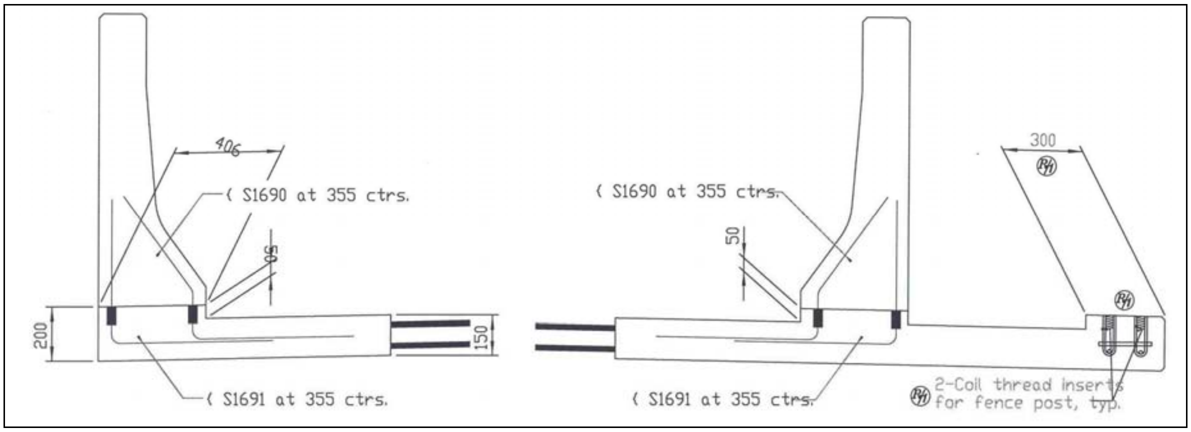

Fig. 78. Barrier curb pedestal in NU-Deck panels (NDOR, 2002)

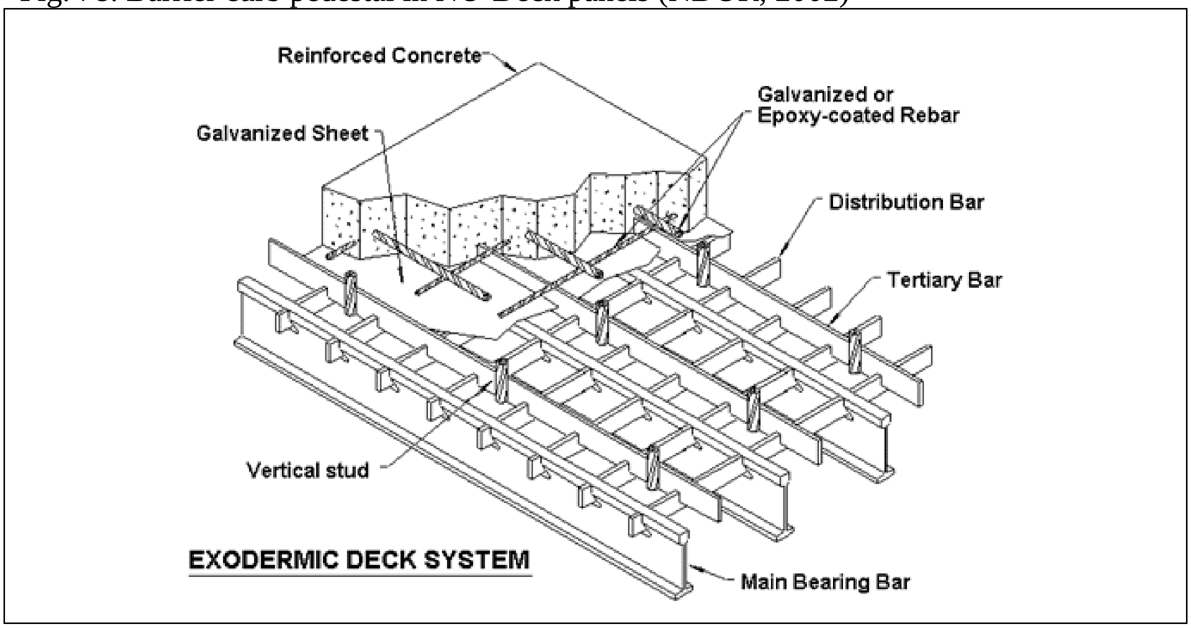

Fig. 79. Exodermic Deck System original design (Exodermic Bridge Deck, Inc., 1998)

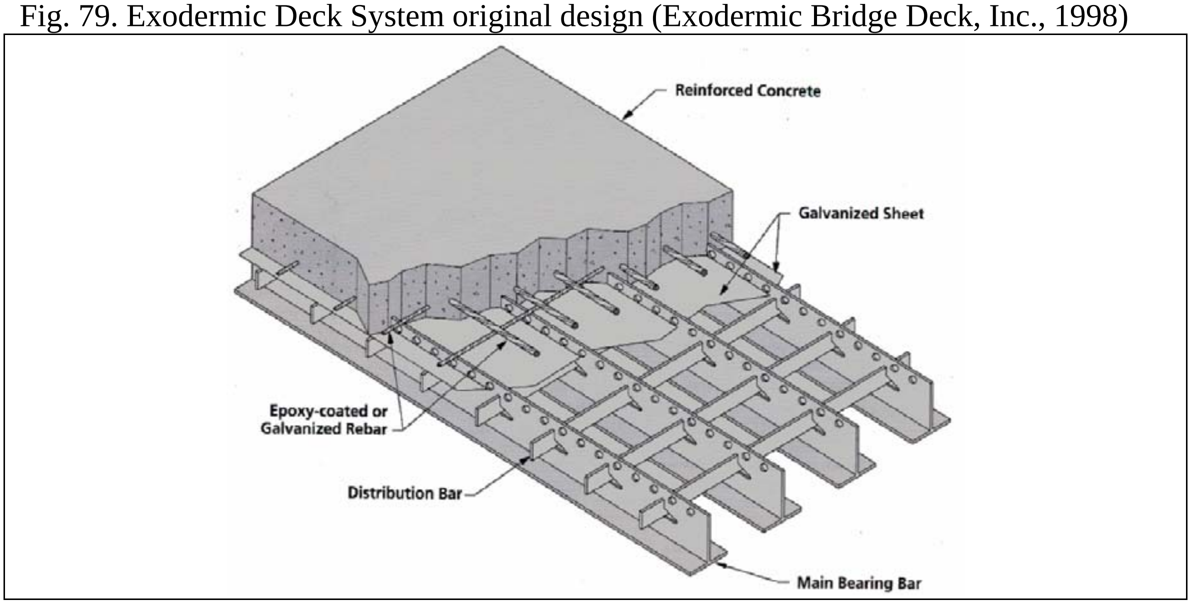

Fig. 80. Exodermic Deck System revised design (Exodermic Bridge Deck, Inc., 1998)

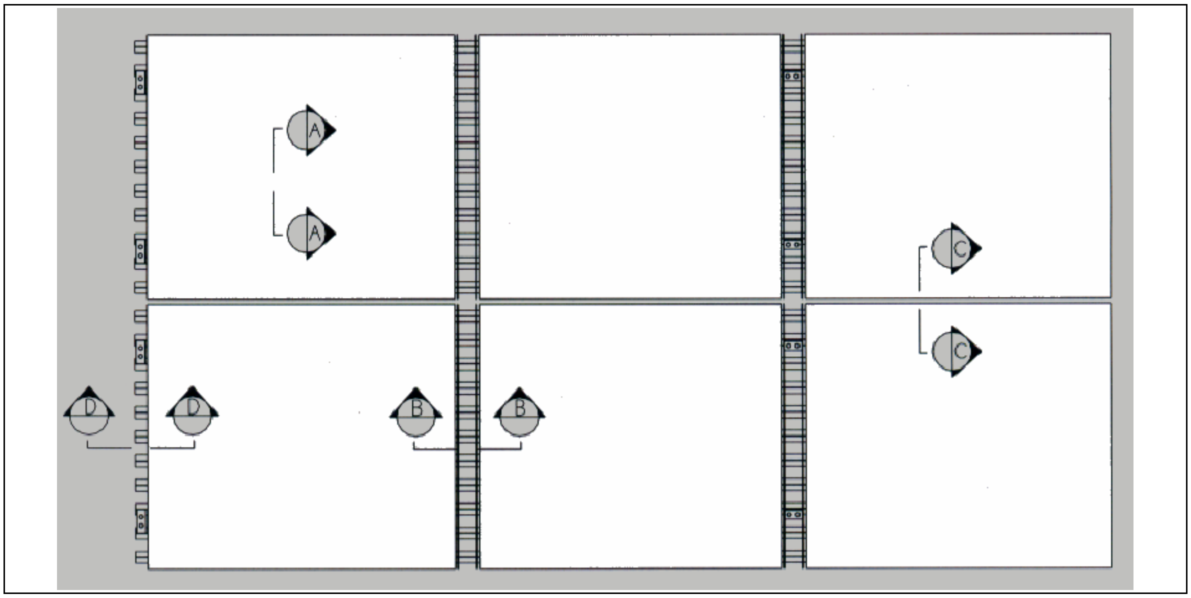

Fig. 81. Typical panel layout plan (Exodermic Bridge Deck, Inc., 1998)

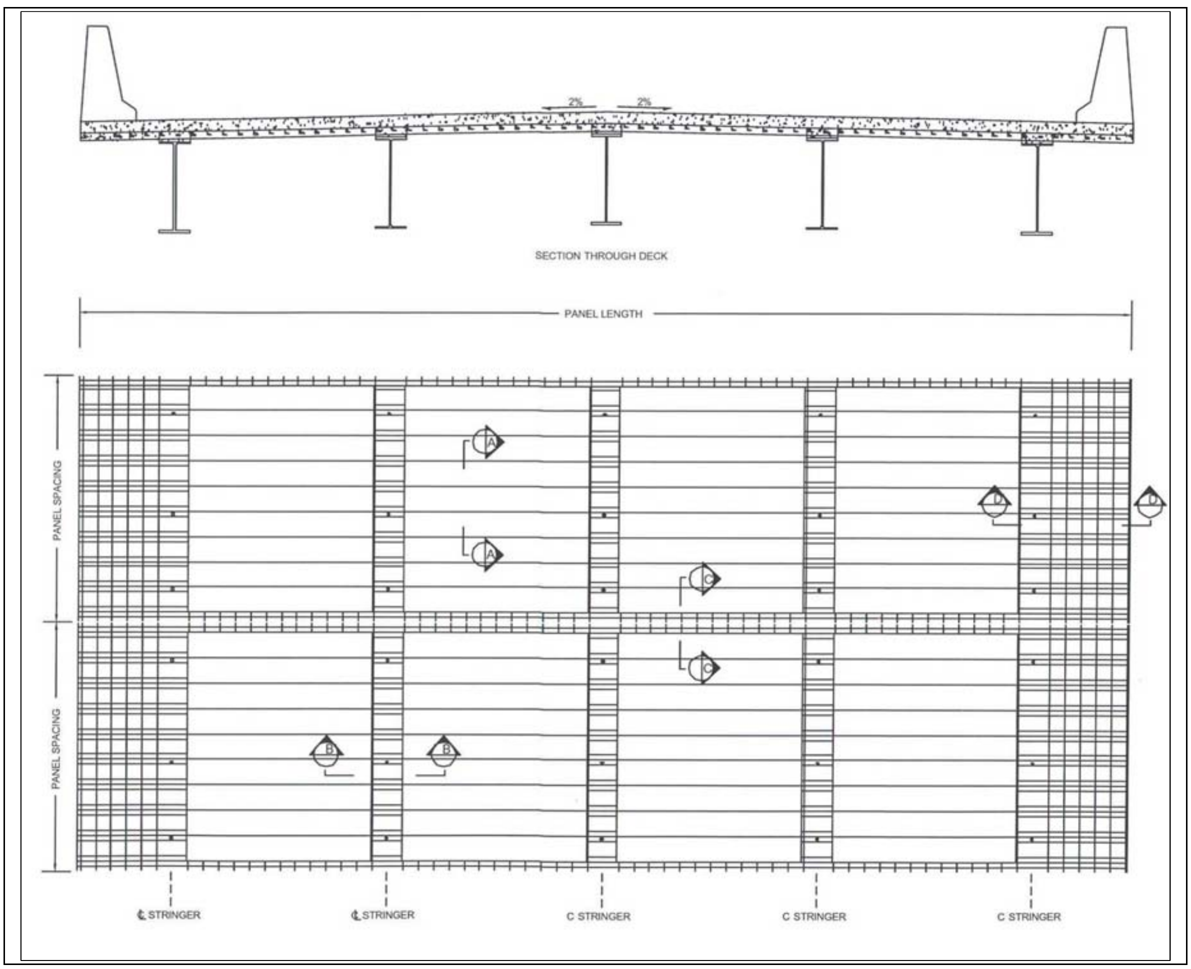

Fig. 82. Crowned bridge plan and section (Exodermic Bridge Deck, Inc., 2003)

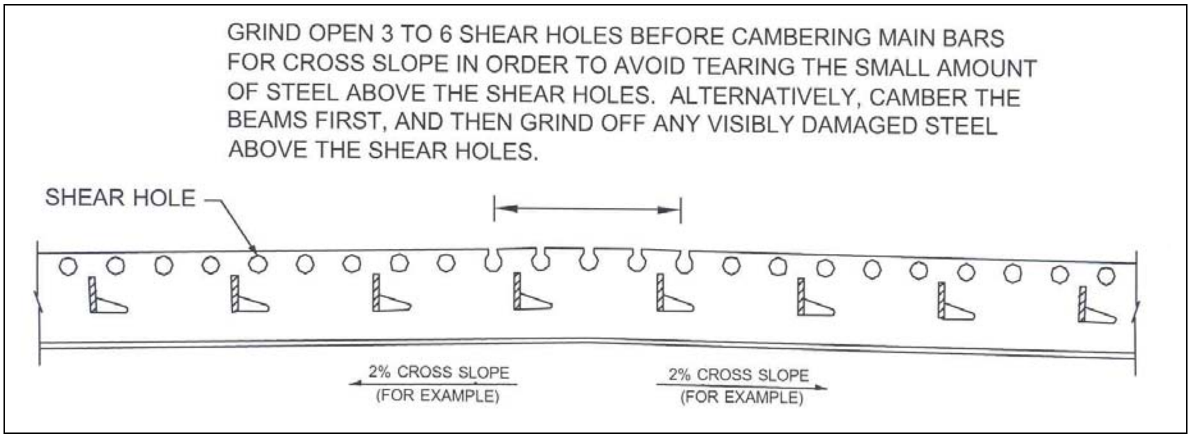

Fig. 83. Modification of shear holes for cambered panels (Exodermic Bridge Deck, Inc., 2003)

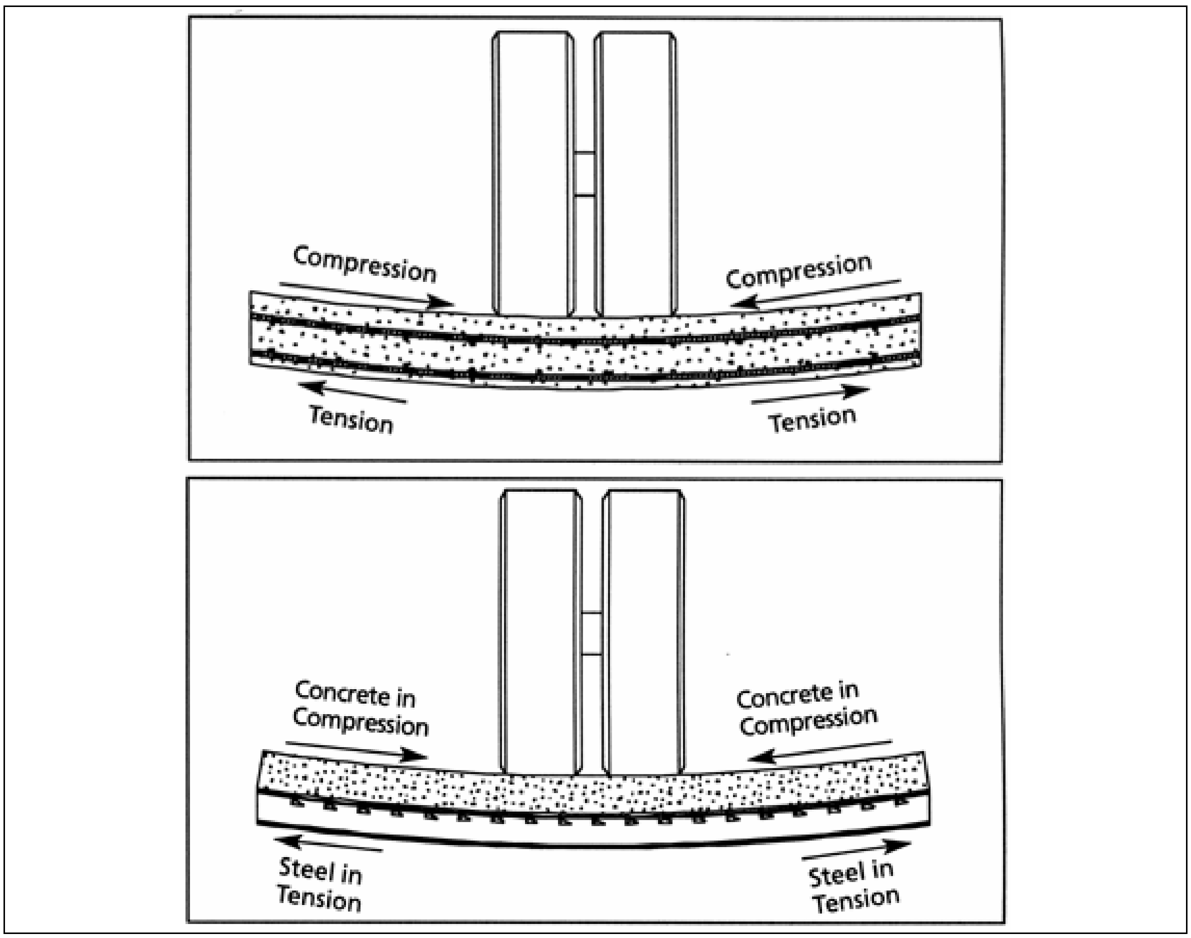

Fig. 84. Standard concrete slab and Exodermic deck in positive bending (Exodermic Bridge Deck, Inc., 1998)

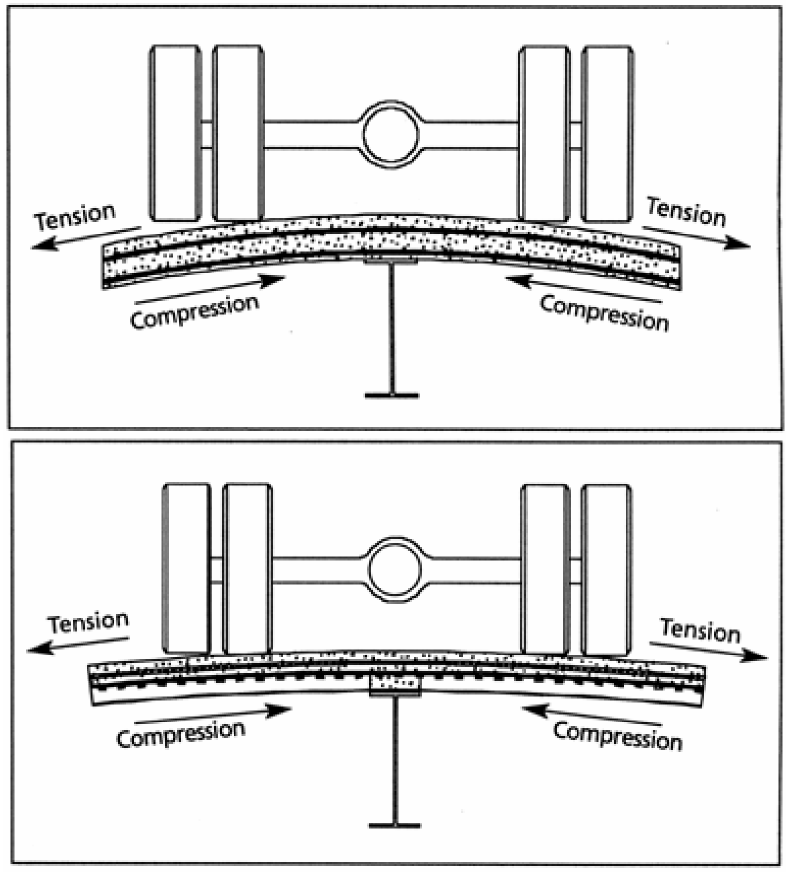

Fig. 85. Standard concrete slab and Exodermic deck in negative bending (EBDI, 1998)

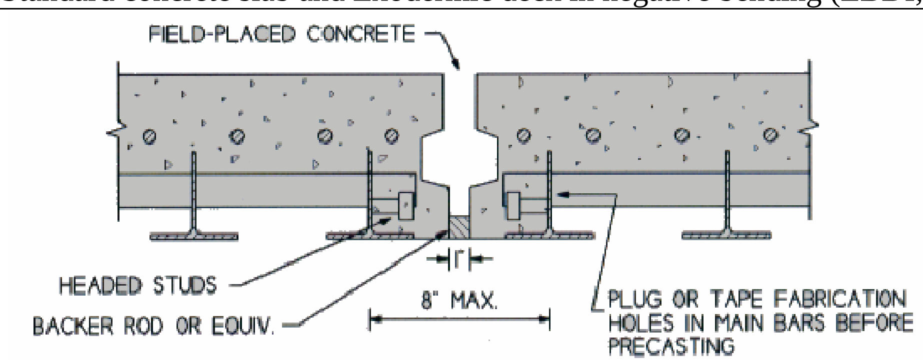

Fig. 86. Typical transverse joint detail for precast applications (EBDI, 1998).

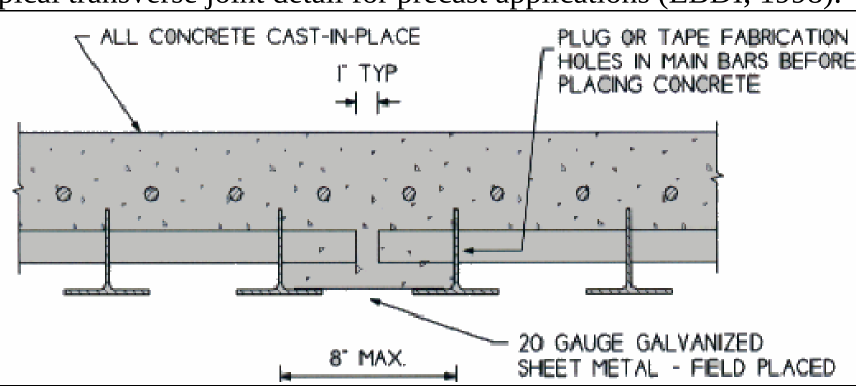

Fig. 87. Typical transverse joint detail for cast-in-place applications (EBDI, 1998)

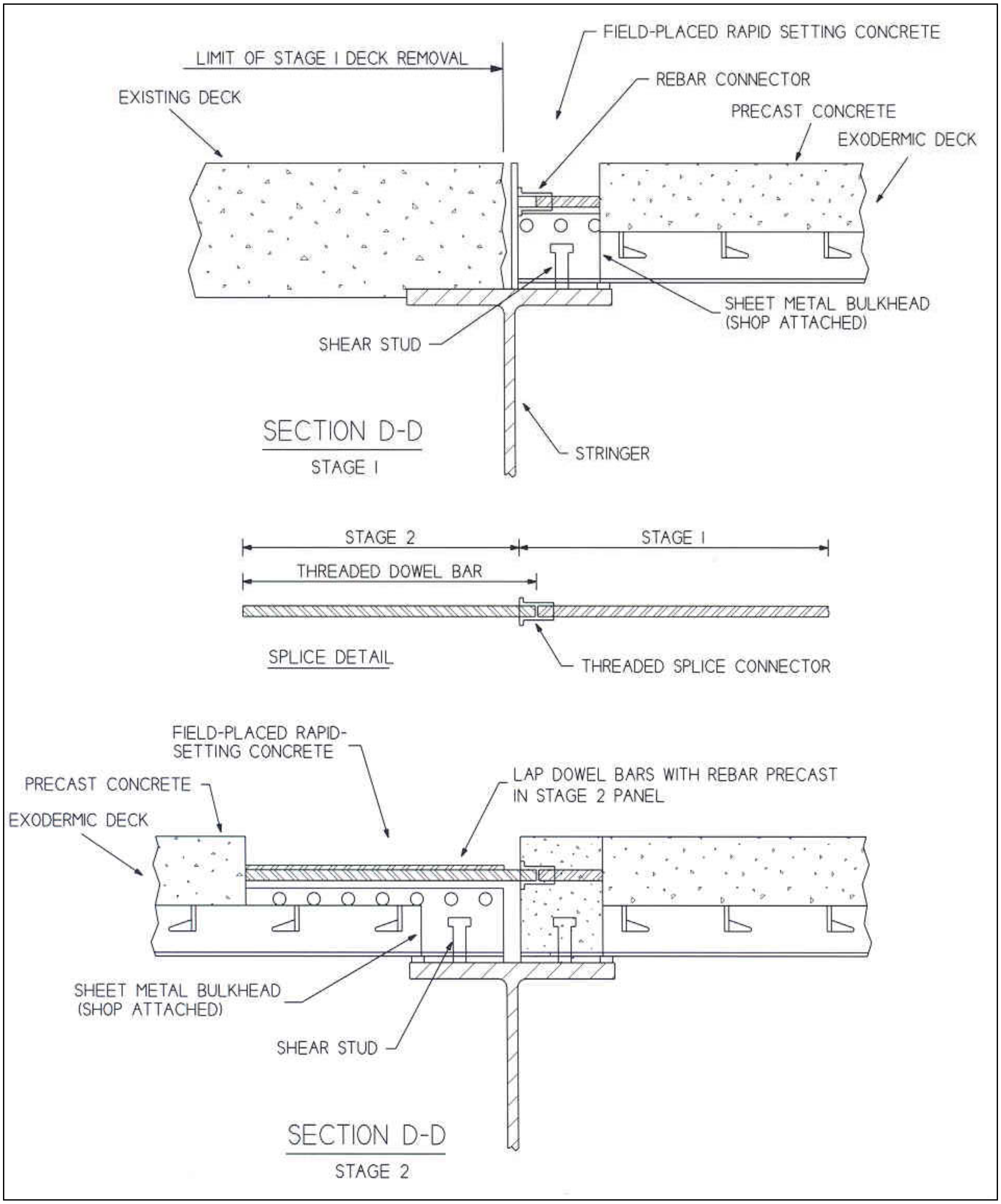

Fig. 88. Typical deck splice for staged construction (Exodermic Bridge Deck, Inc., 2002)

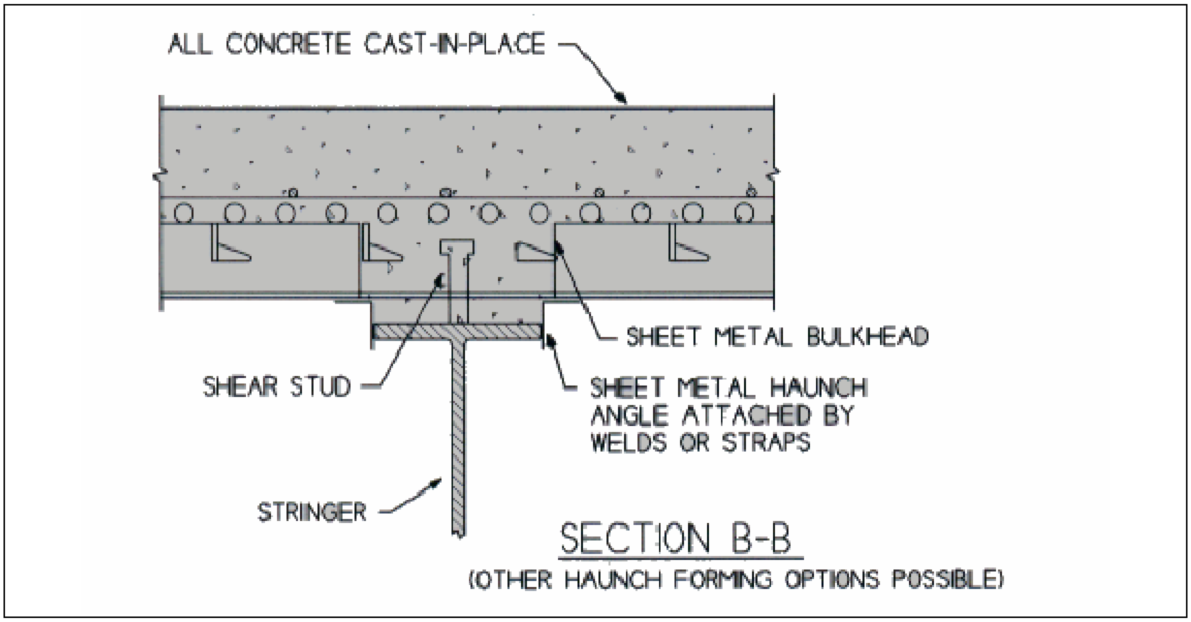

Fig. 89. Typical girder/deck connection for cast-in-place applications (Exodermic Bridge Deck, Inc., 1998)

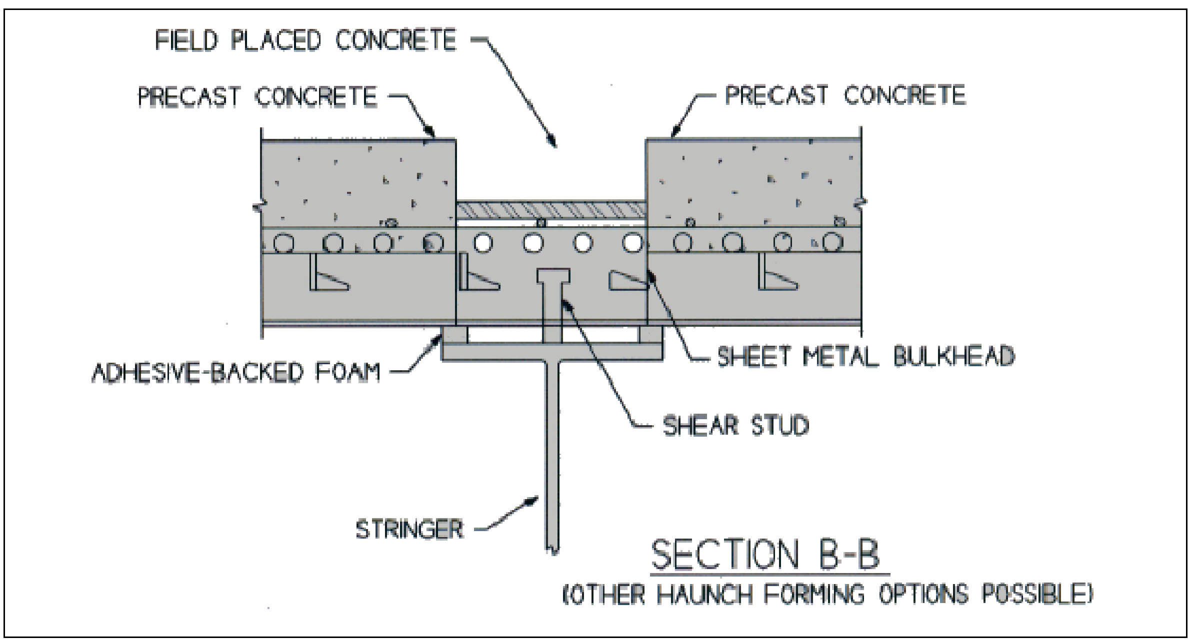

Fig. 90. Typical girder/deck connection for precast applications (Exodermic Bridge Deck, Inc., 1998)

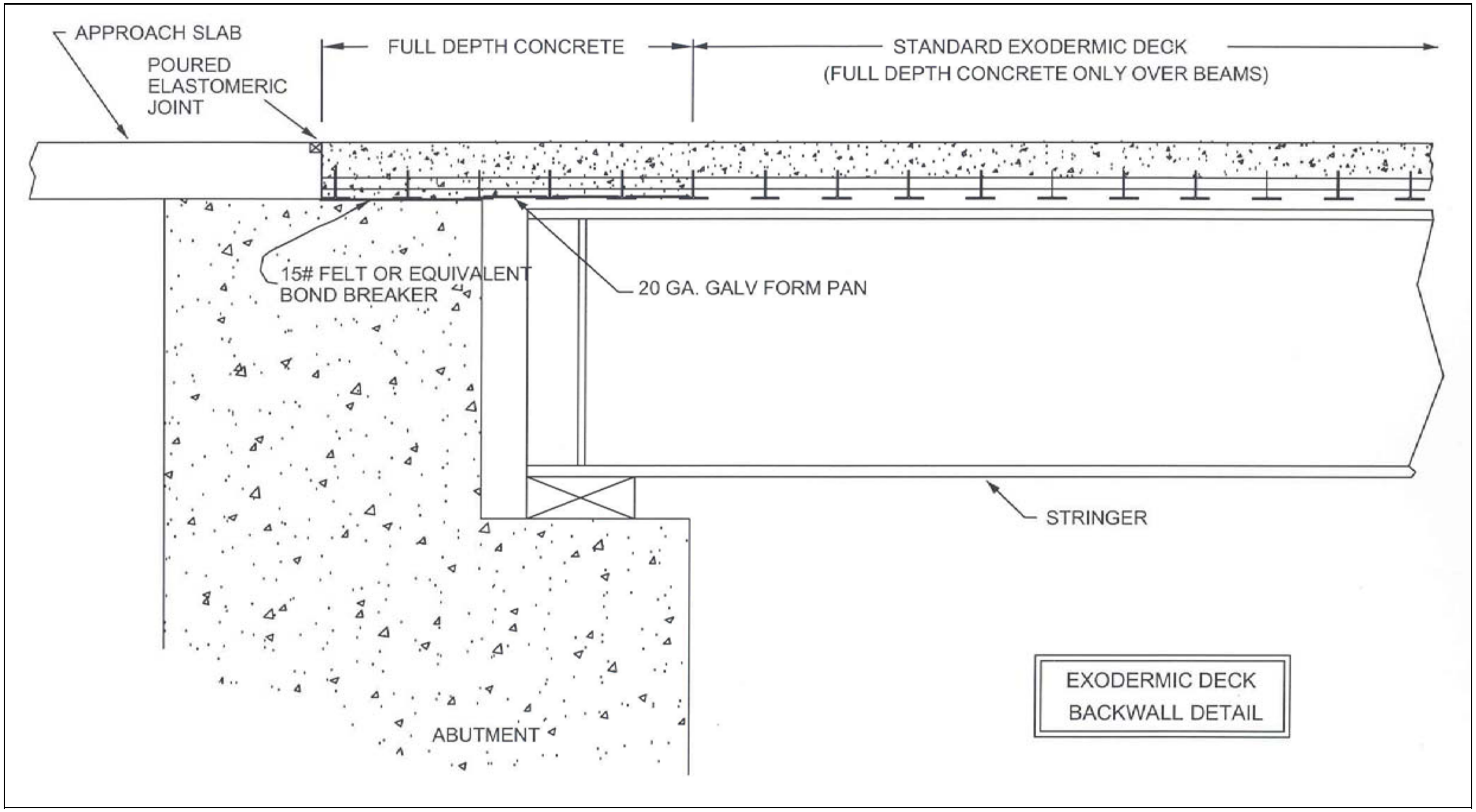

Fig. 91. Typical Exodermic deck backwall detail (Exodermic Bridge Deck, Inc., 2003)

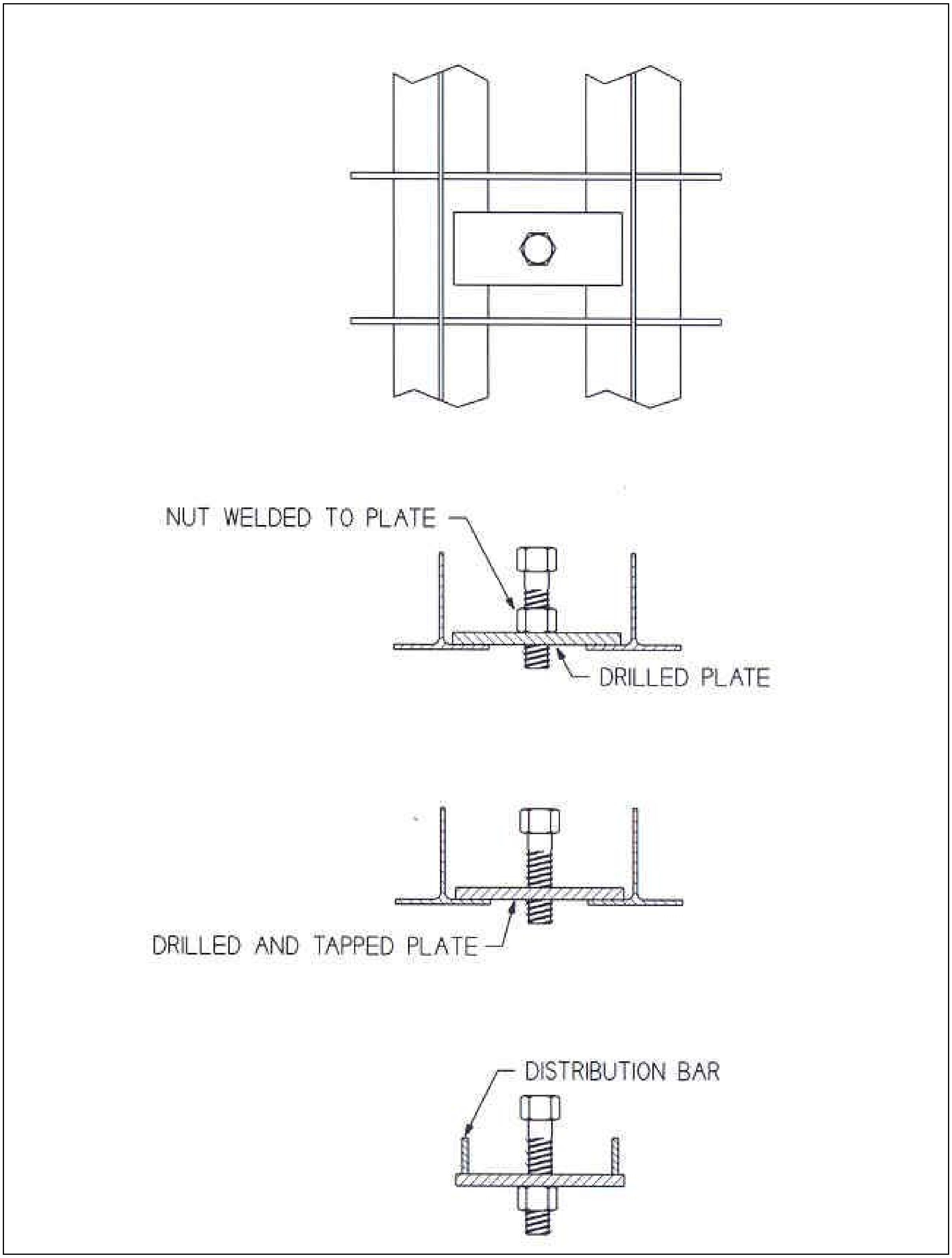

Fig. 92. Typical height adjustment details (Exodermic Bridge Deck, Inc., 2003)

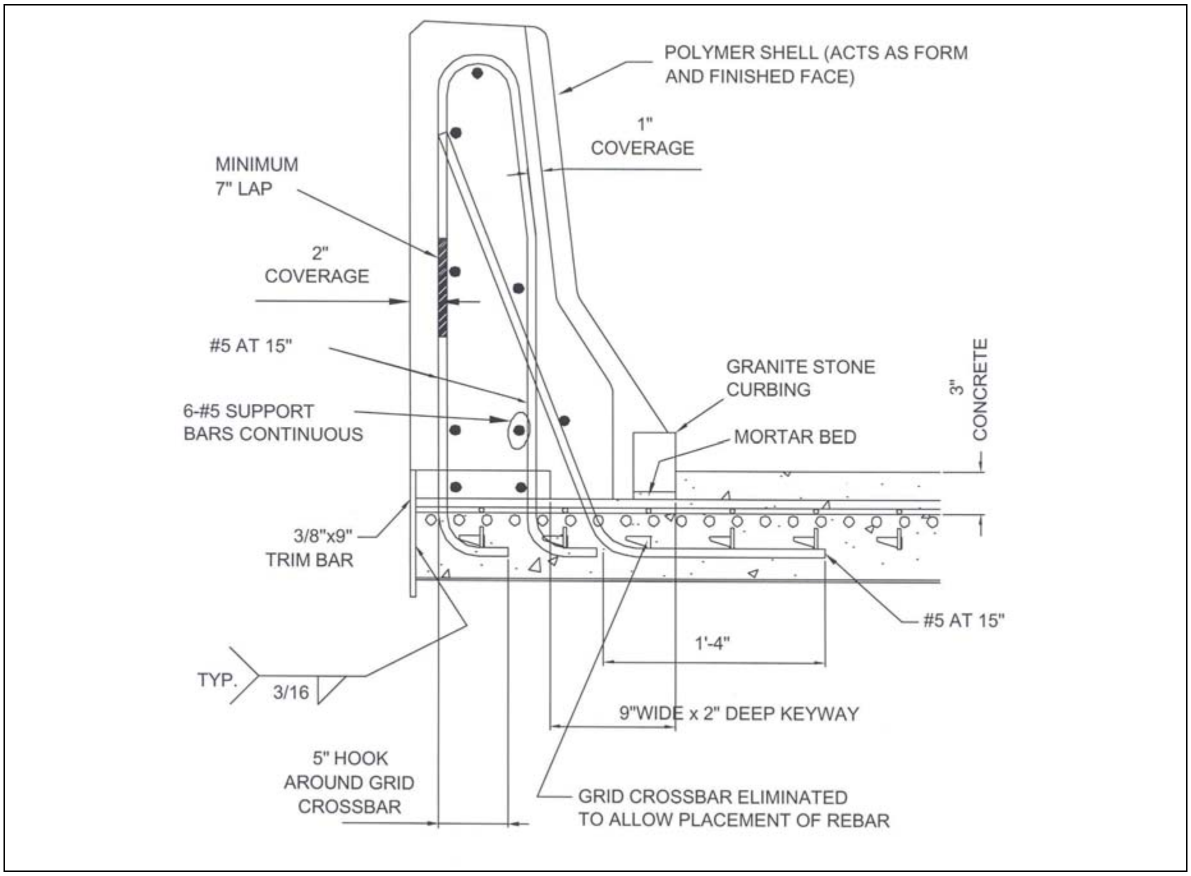

Fig. 93. Typical cast in place barrier detail (Exodermic Bridge Deck, Inc., 2003)

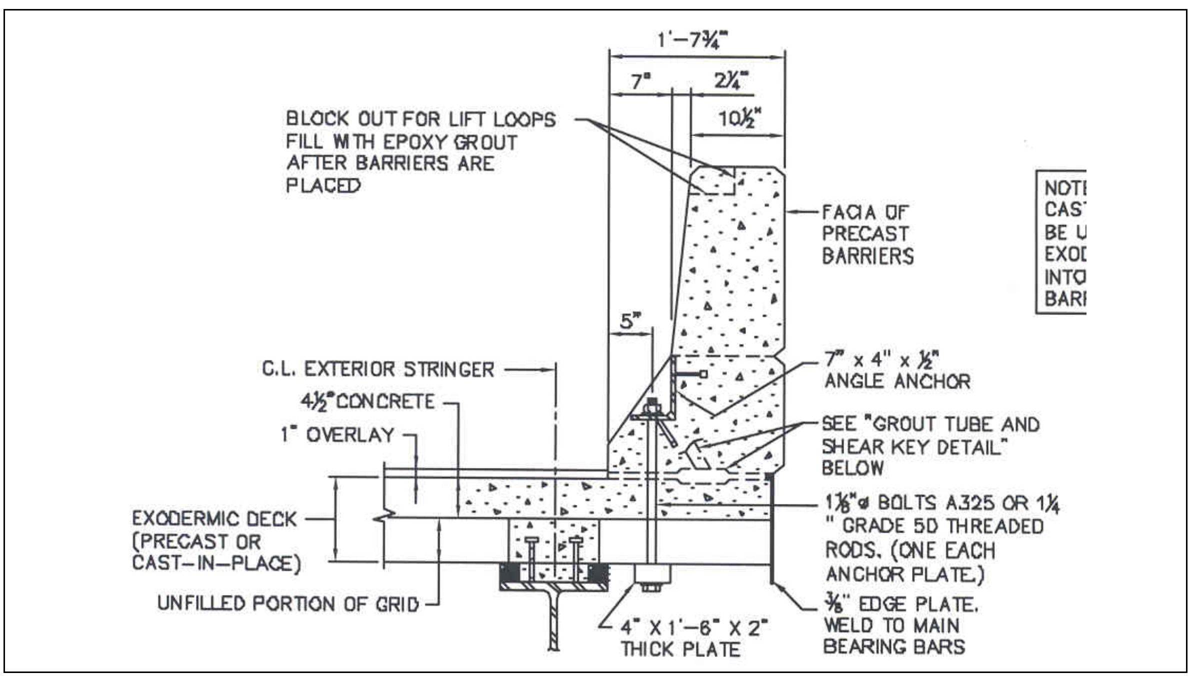

Fig. 94. Typical precast barrier detail (Exodermic Bridge Deck, Inc., 2003)

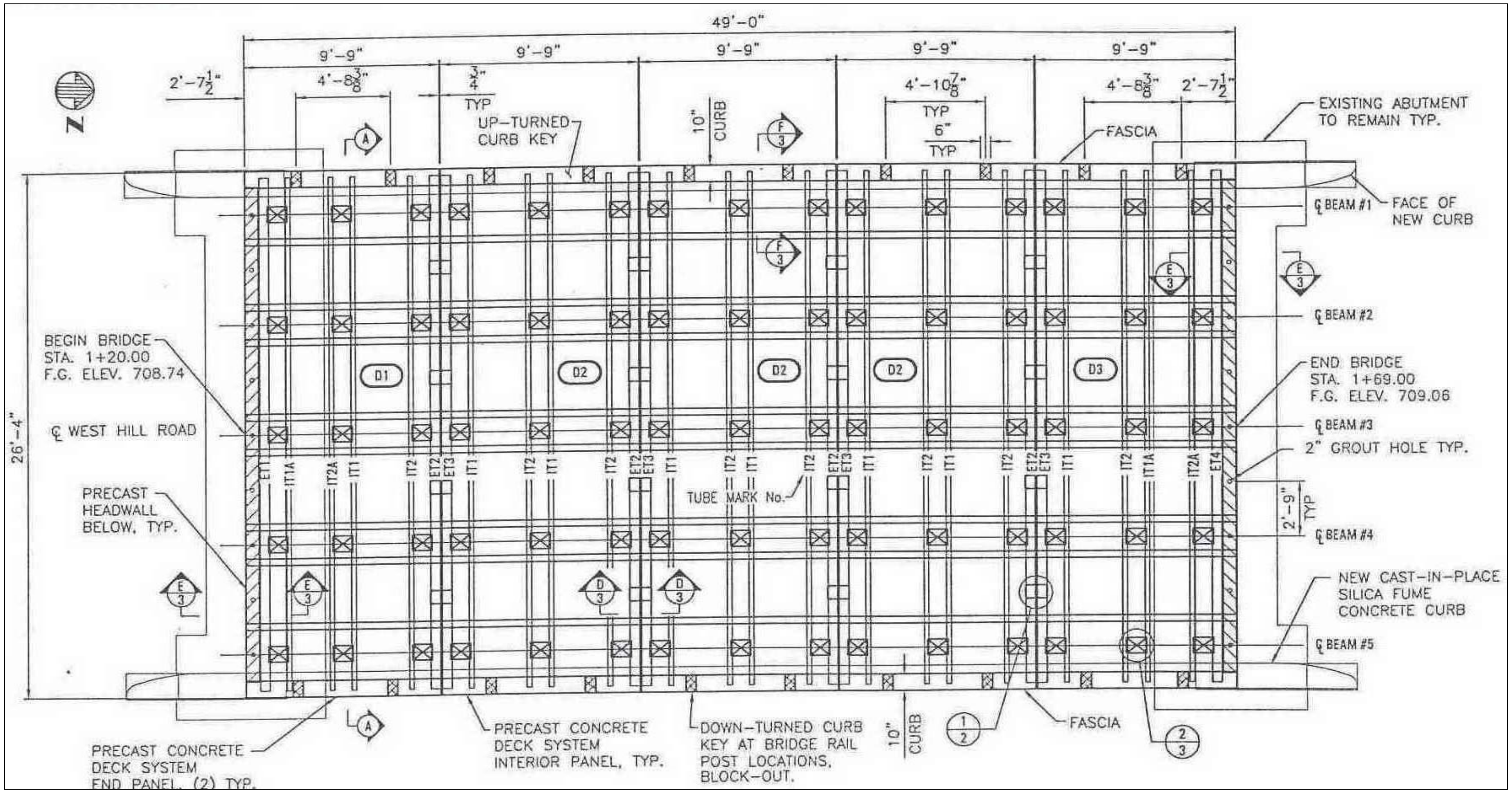

Fig. 95. Deck plan view of bridge over Little River Town, Stowe, VT (The Fort Miller Co., Inc., 2000)

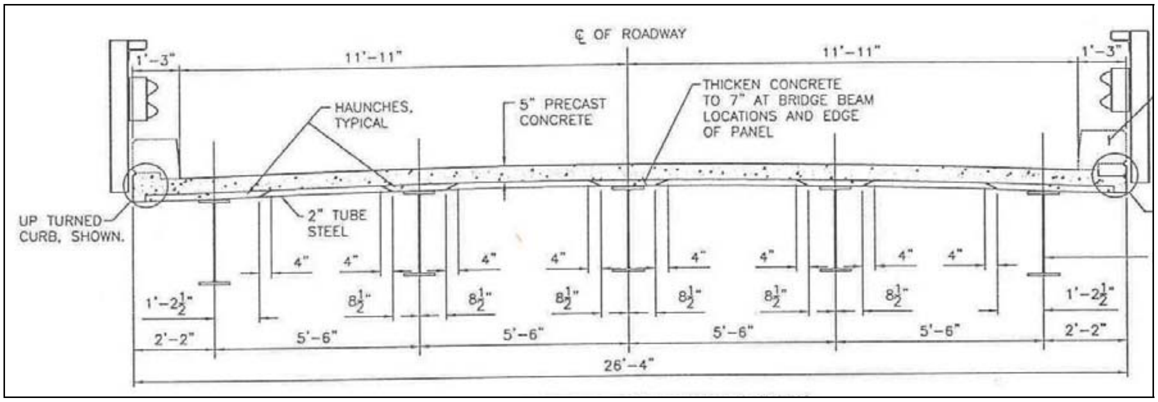

Fig. 96. Cross section of bridge over Little River Town, Stowe, VT (The Fort Miller Co., Inc., 2000)

Fig. 97. Deck construction of bridge over Little River Town, Stowe, VT (The Fort Miller Co., Inc., 2002)

Fig. 98. Deck construction of bridge over Cascadilla Creek, Ithaca, NY (The Fort Miller Co., Inc., 2001)

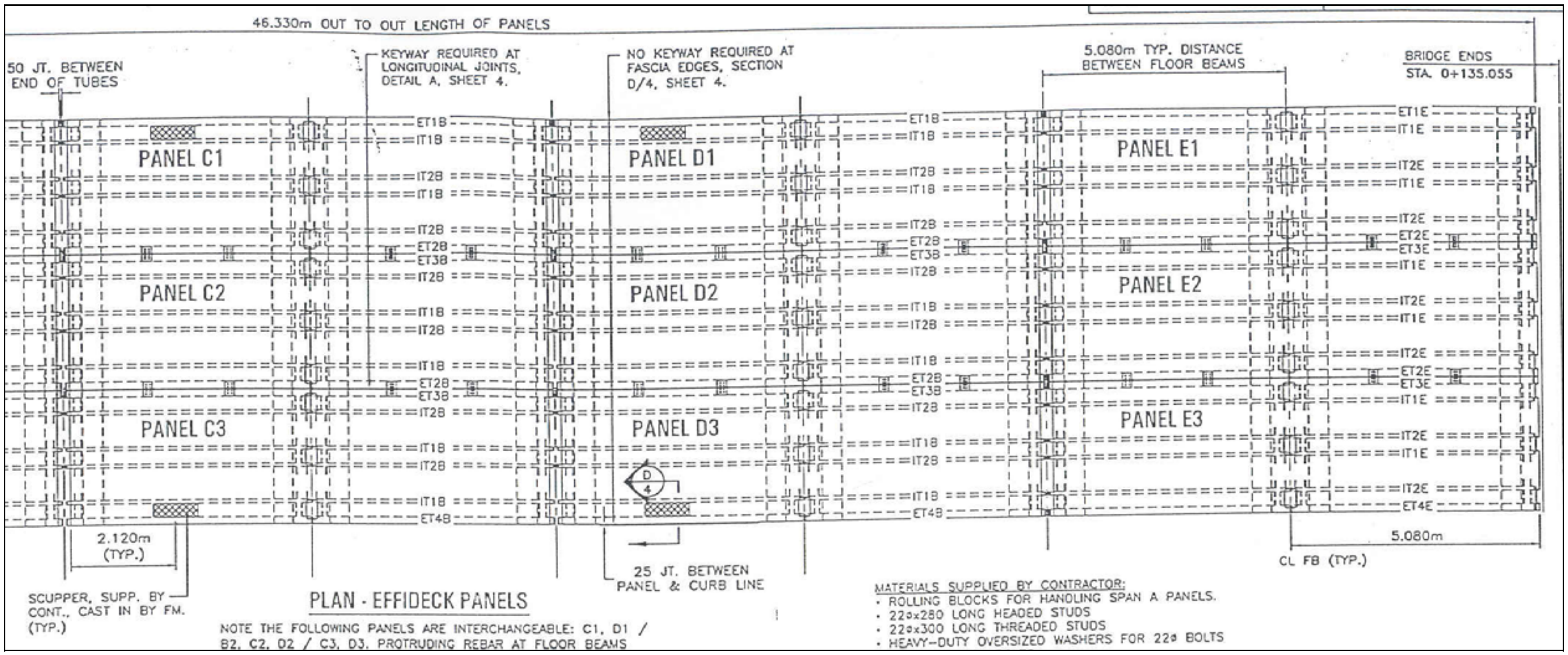

Fig. 99. Deck plan view of bridge over Cascadilla Creek, Ithaca, NY (The Fort Miller Co., Inc., 2001)

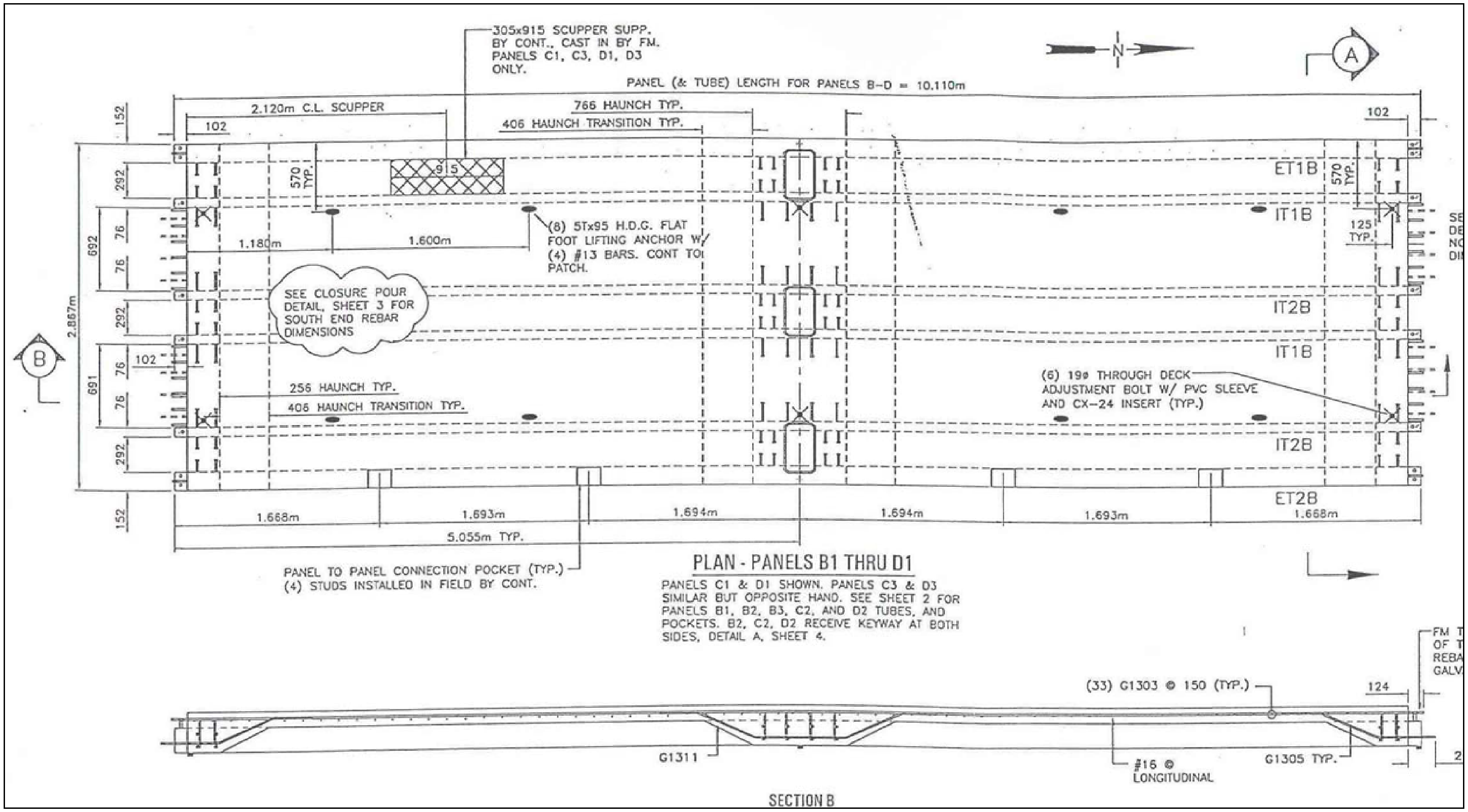

Fig. 100. Panel plan view and section for bridge over Cascadilla Creek, Ithaca, NY (The Fort Miller Co., Inc., 2001)

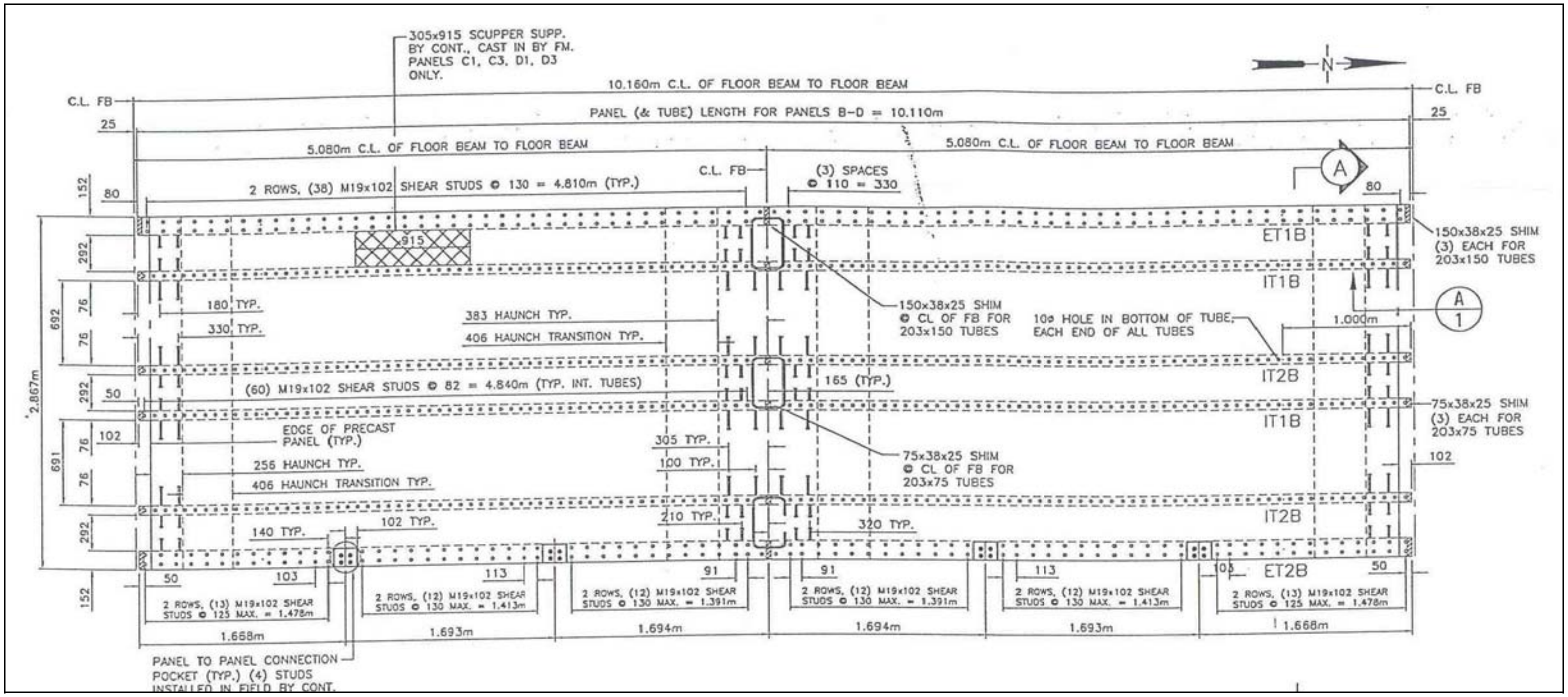

Fig. 101. Typical shear stud distribution in panels for bridge over Cascadilla Creek, Ithaca, NY (The Fort Miller Co., Inc., 2001)

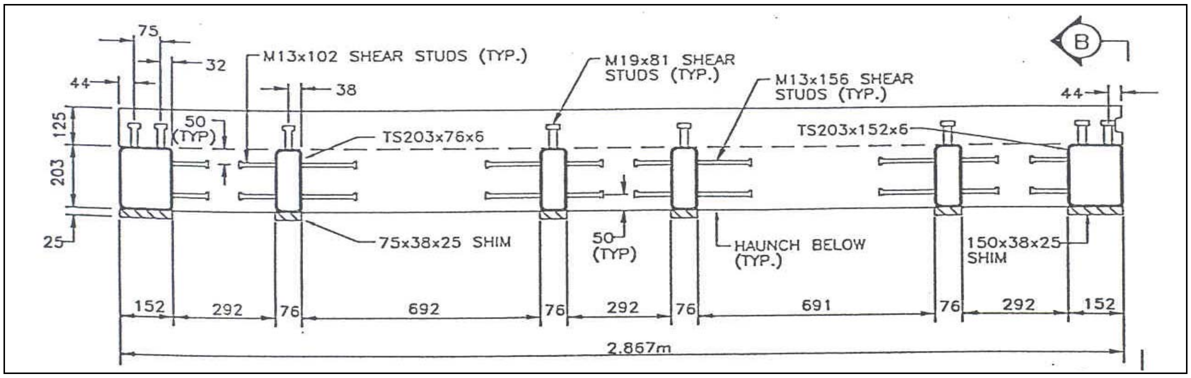

Fig. 102. Typical panel cross section at full-depth portions showing studs for bridge over Cascadilla Creek, Ithaca, NY (The Fort Miller Co., Inc., 2001)

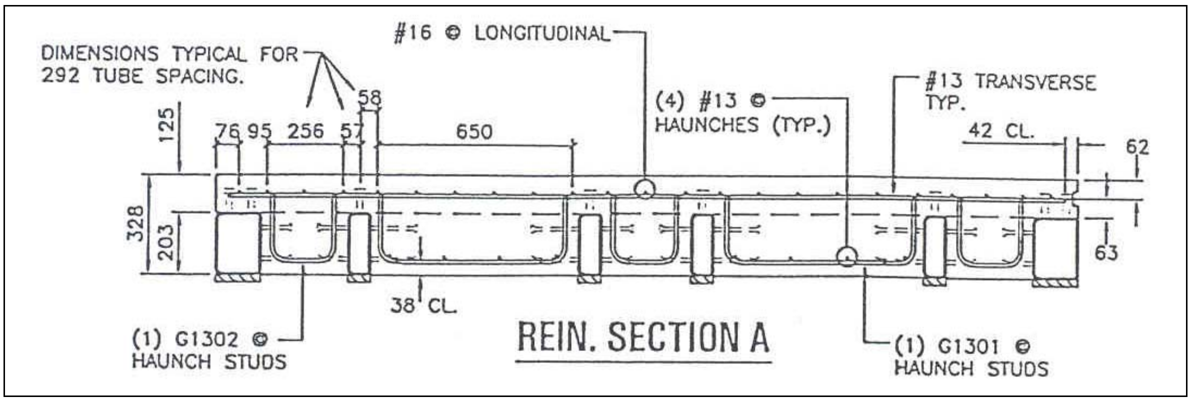

Fig. 103. Typical panel cross section at full-depth portions showing reinforcement for bridge over Cascadilla Creek, Ithaca, NY (The Fort Miller Co., Inc., 2001)

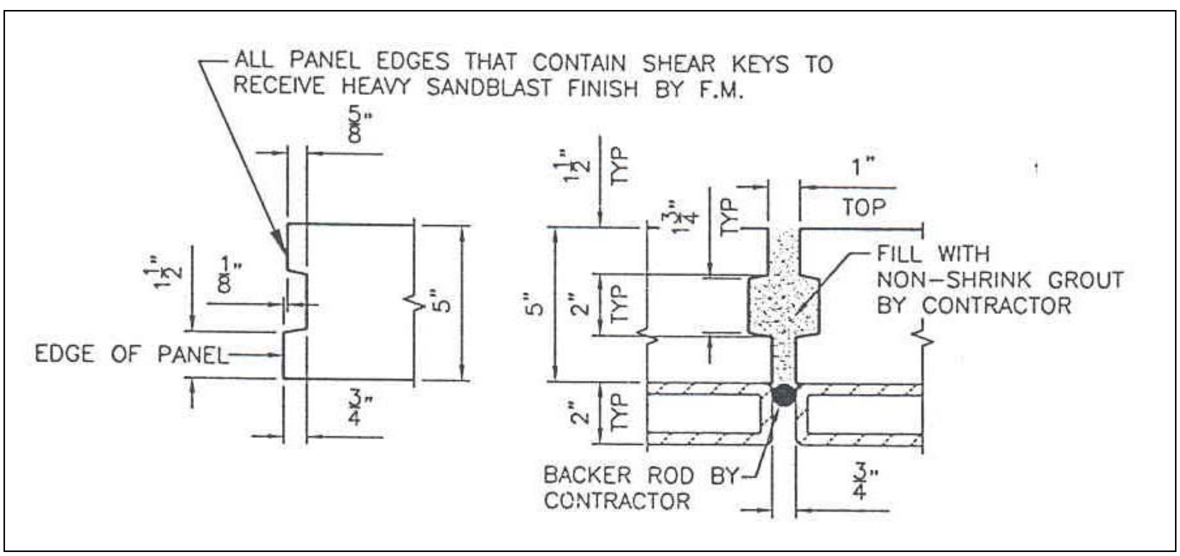

Fig. 104. Typical shear key detail for bridge over Little River Town, Stowe, VT (The Fort Miller Co., Inc., 2000)

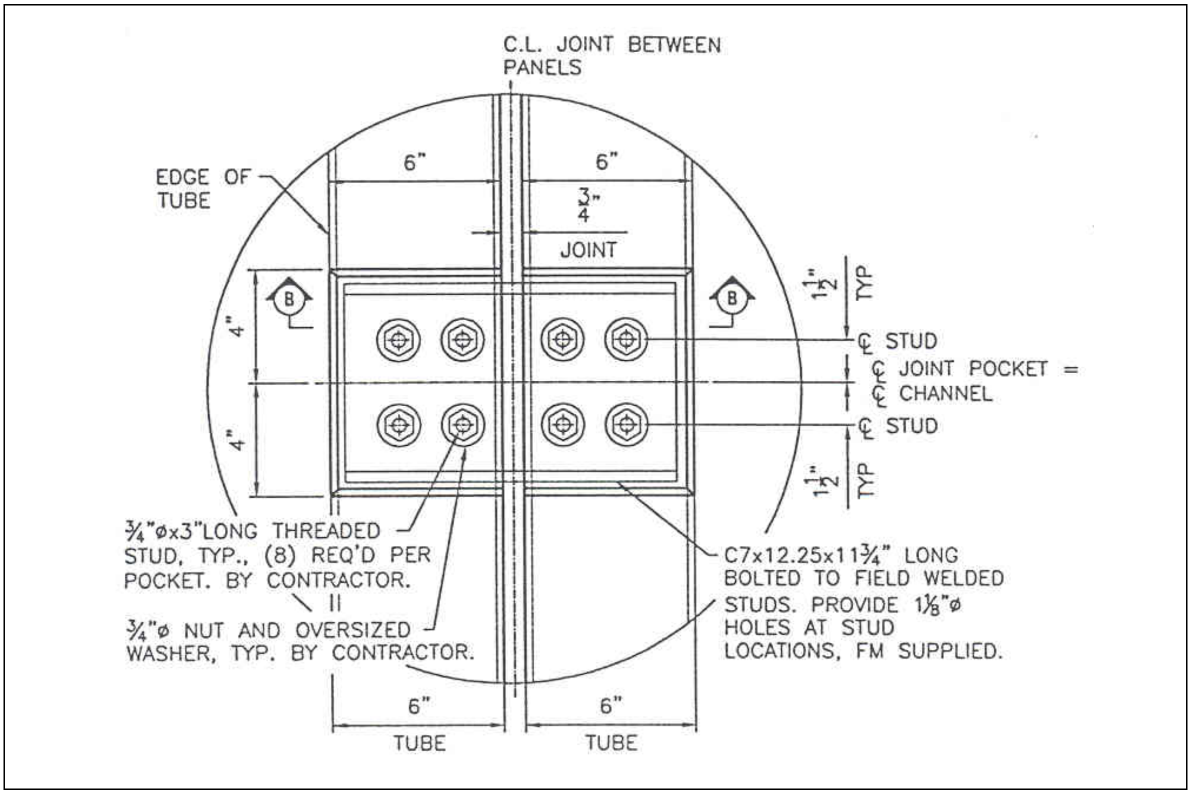

Fig. 105. Panel to panel connection plan view for bridge over Little River Town, Stowe, VT (The Fort Miller Co., Inc., 2000)

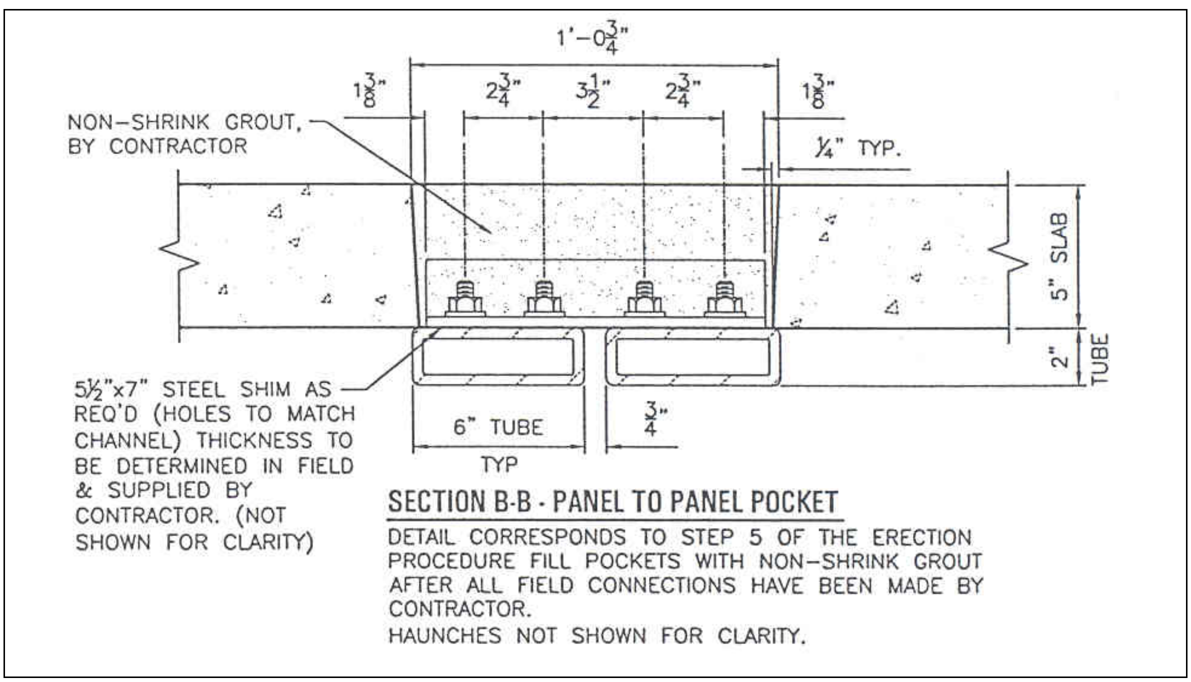

Fig. 106. Panel to panel connection section for bridge over Little River Town, Stowe, VT (The Fort Miller Co., Inc., 2000)

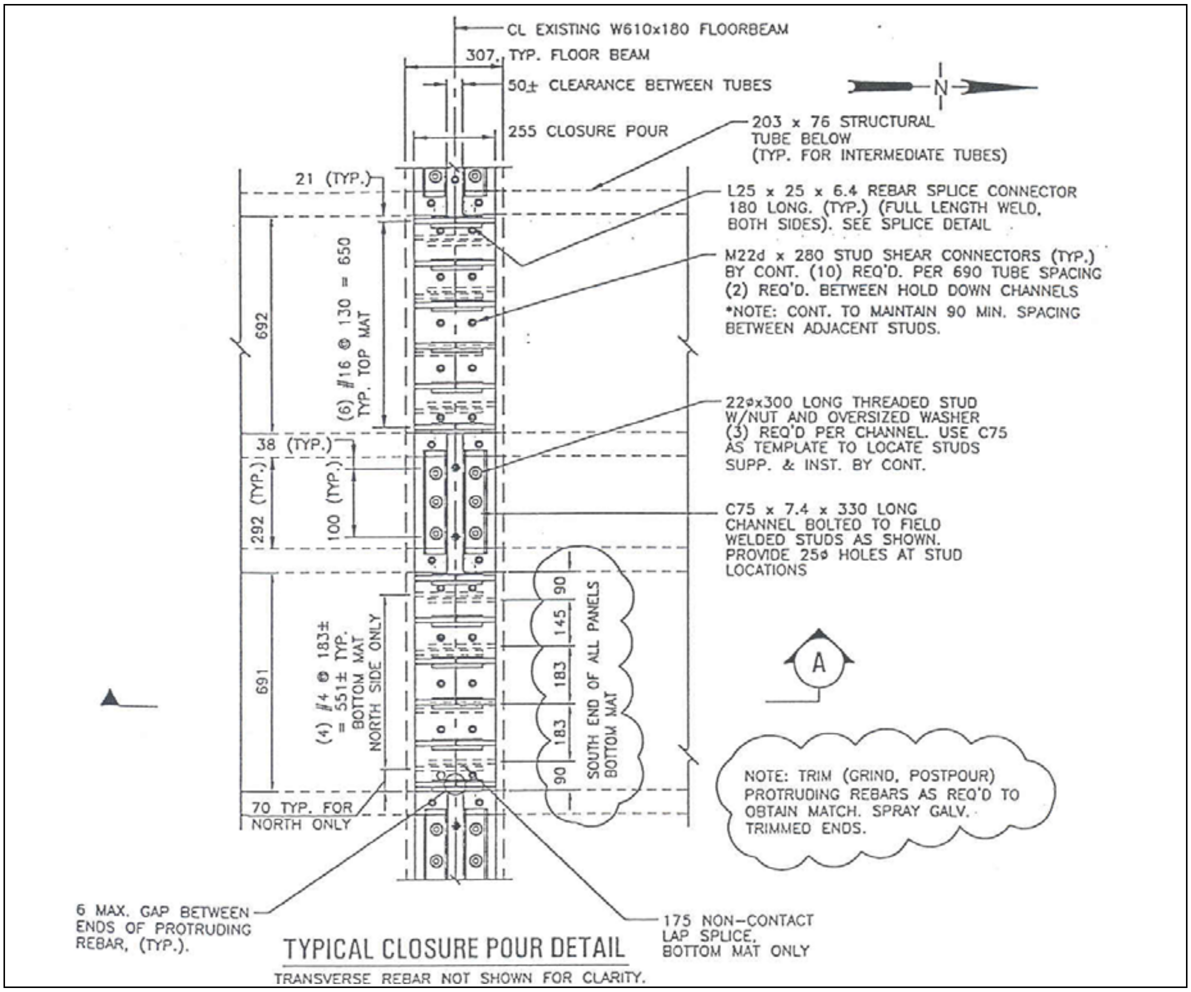

Fig. 107. Typical joint plan view for bridge over Cascadilla Creek, Ithaca, NY (The Fort Miller Co., Inc., 2001)

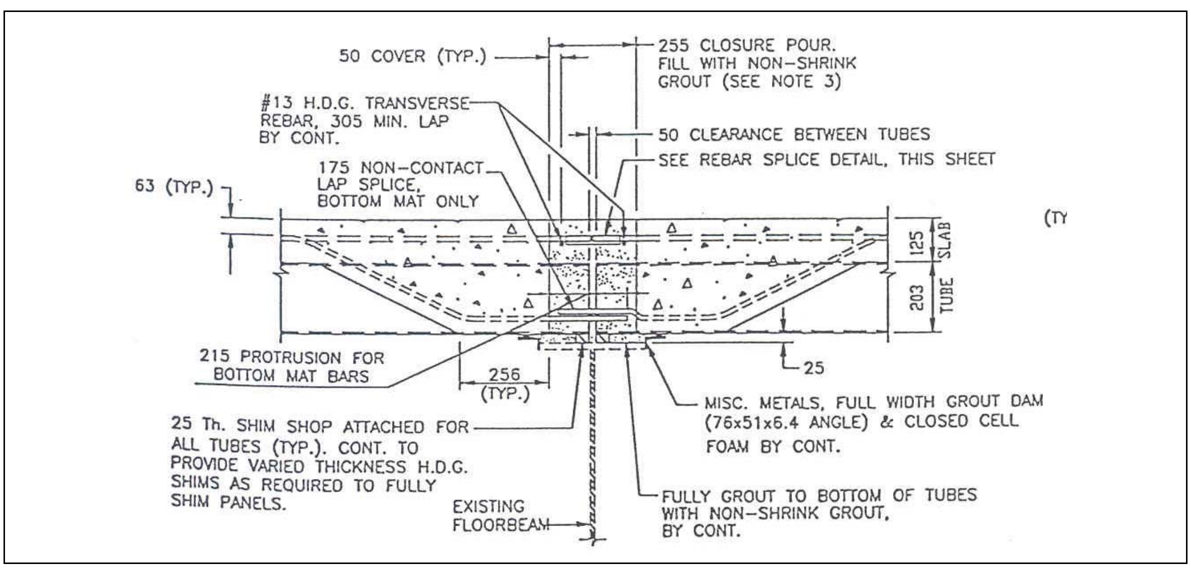

Fig. 108. Typical joint section for bridge over Cascadilla Creek, Ithaca, NY (The Fort Miller Co., Inc., 2001)

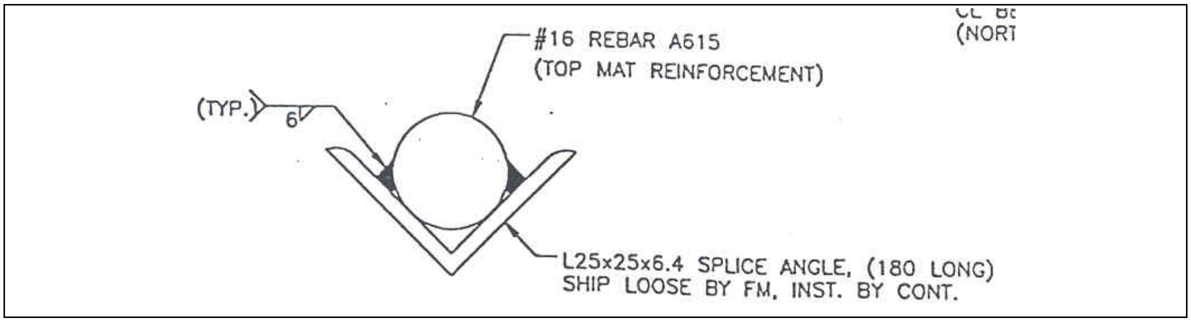

Fig. 109. Typical splice bar detail for bridge over Cascadilla Creek, Ithaca, NY (The Fort Miller Co., Inc., 2001)

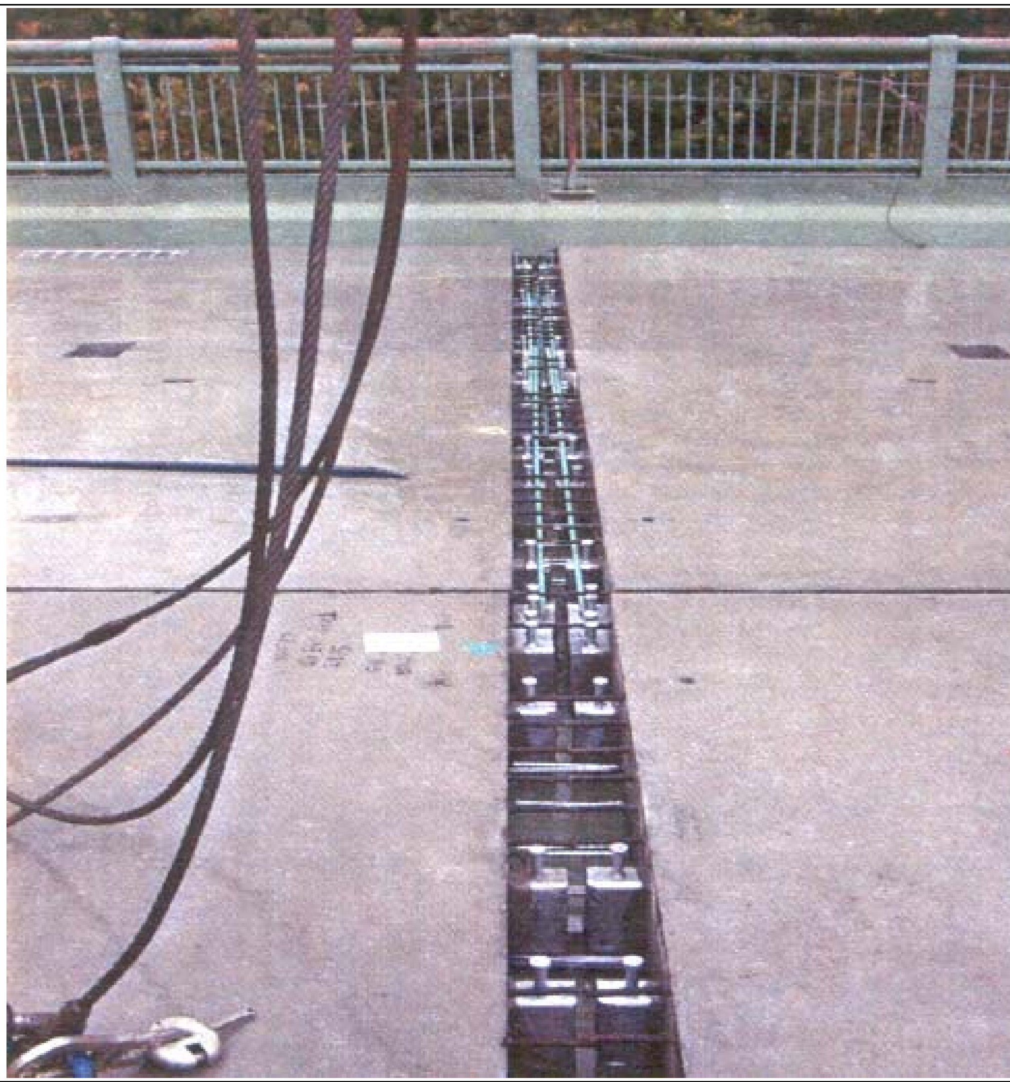

Fig. 110. Transverse joint over Cascadilla Creek, Ithaca, NY (The Fort Miller Co., Inc., 2001)

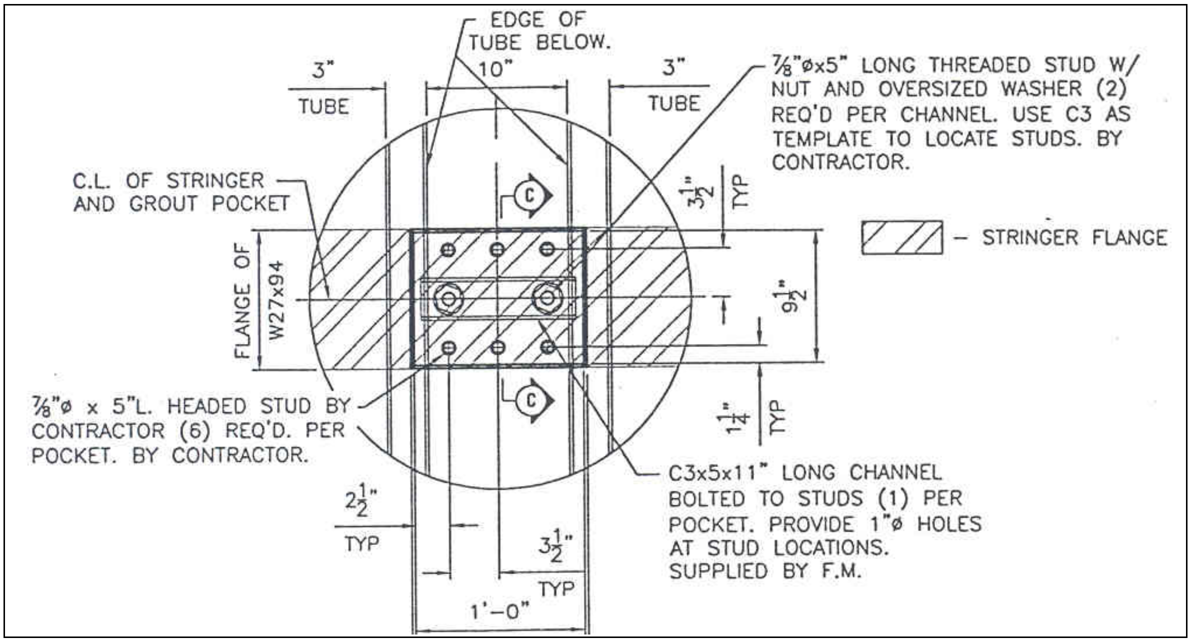

Fig. 111. Typical shear pocket plan view for bridge over Little River Town, Stowe, VT (The Fort Miller Co., Inc., 2000)

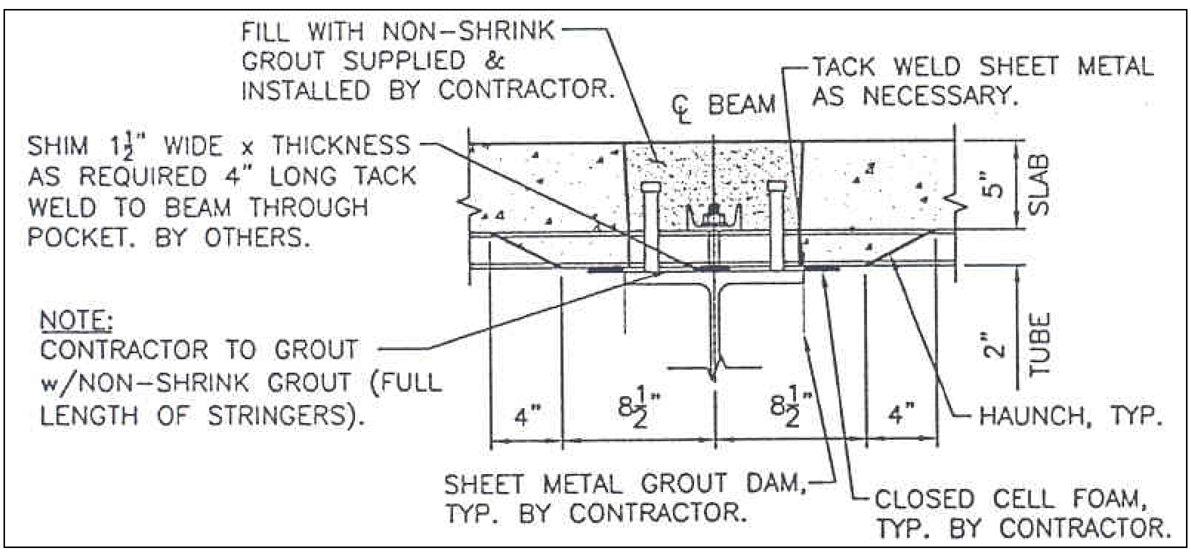

Fig. 112. Typical shear pocket section for bridge over Little River Town, Stowe, VT (The Fort Miller Co., Inc., 2000)

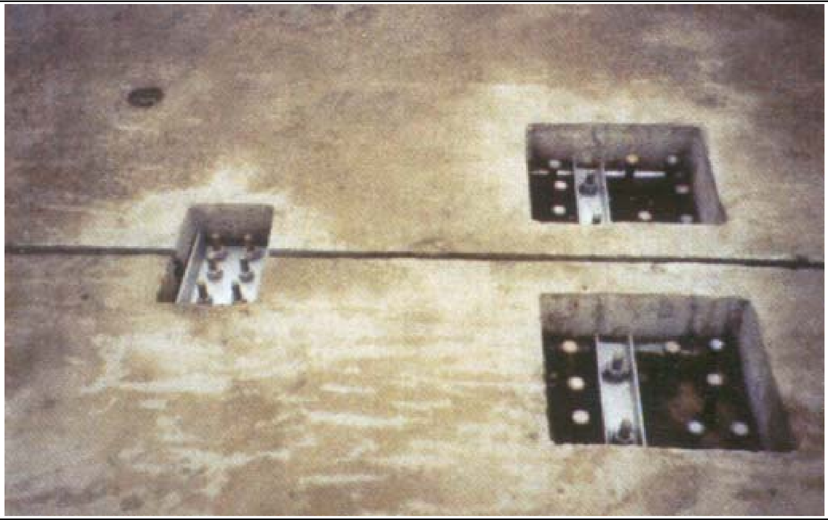

Fig. 113. Panel-to-panel pocket and shear pockets (The Fort Miller Co., Inc., 2002)

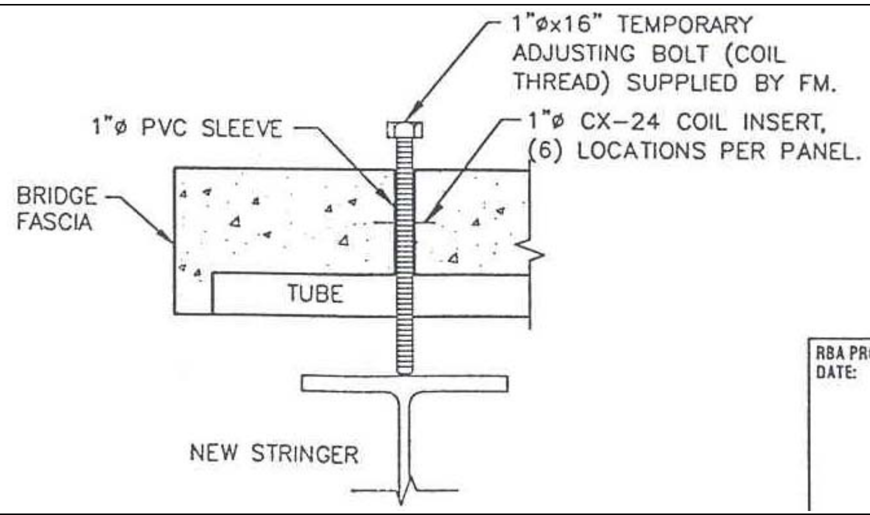

Fig. 114. Typical leveling device used for bridge over Little River Town, Stowe, VT (The Fort Miller Co., Inc., 2000)

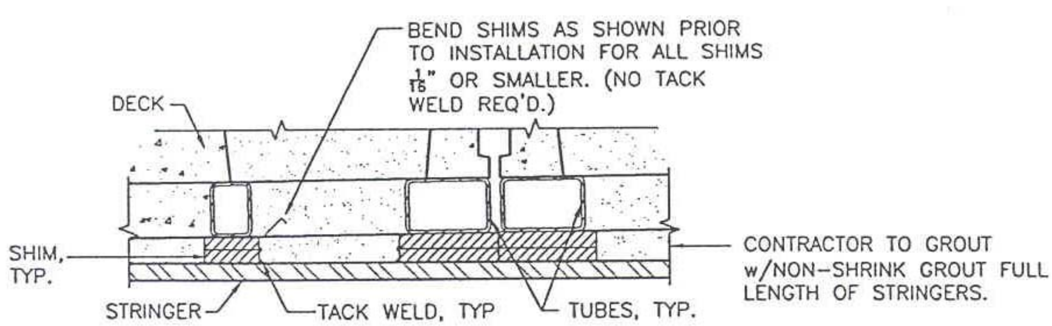

Fig. 115. Typical shim detail for bridge over Cascadilla Creek, Ithaca, NY (The Fort Miller Co., Inc., 2001)

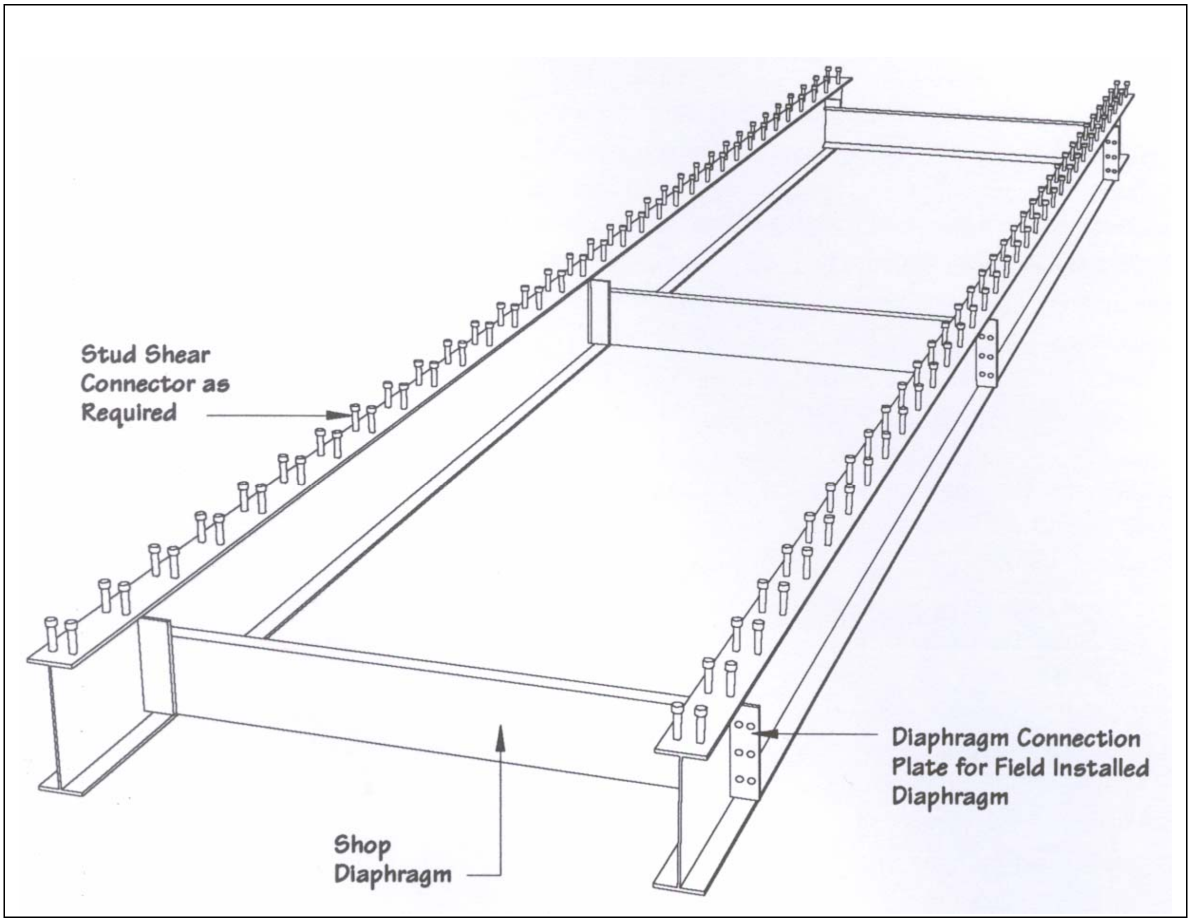

Fig. 116. Fabricated beam pair for an Inverset Bridge System unit (The Fort Miller Co., Inc., 1998)

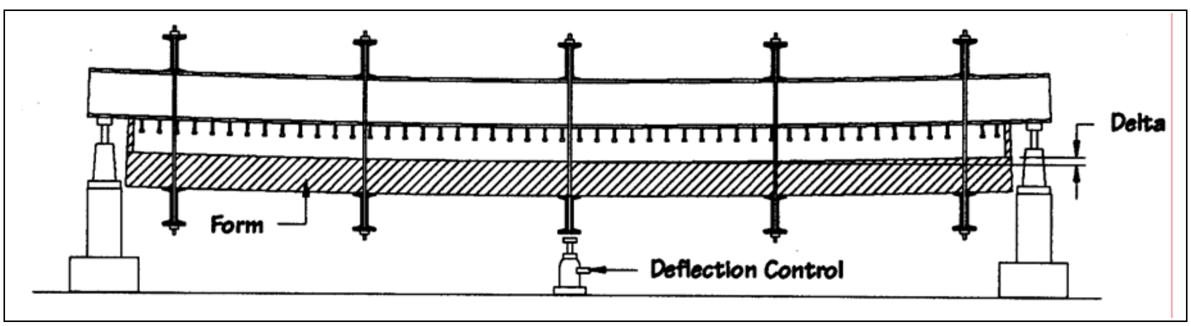

Fig. 117. Inverset Bridge System unit cast right side down (The Fort Miller Co., Inc., 1998)

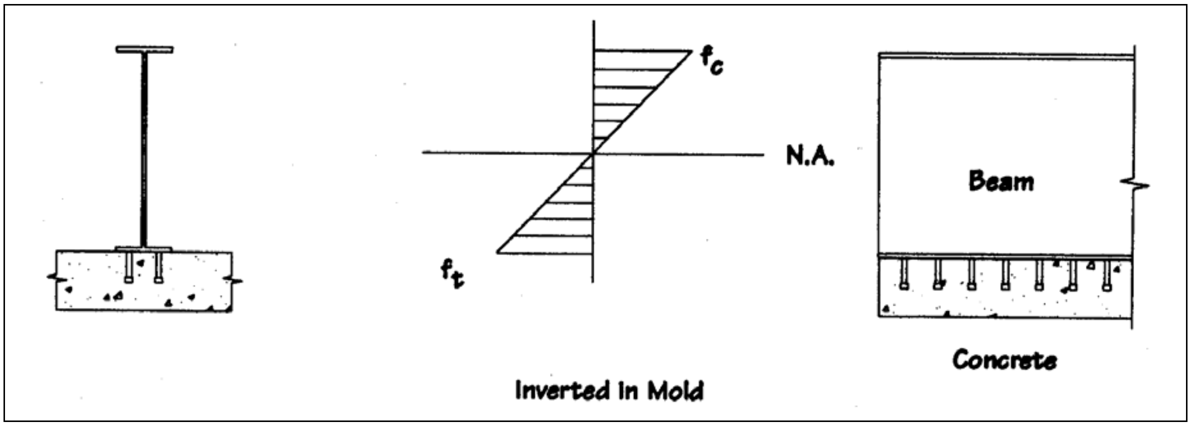

Fig. 118. Stress diagram of a beam upside down (The Fort Miller Co., Inc., 1998)

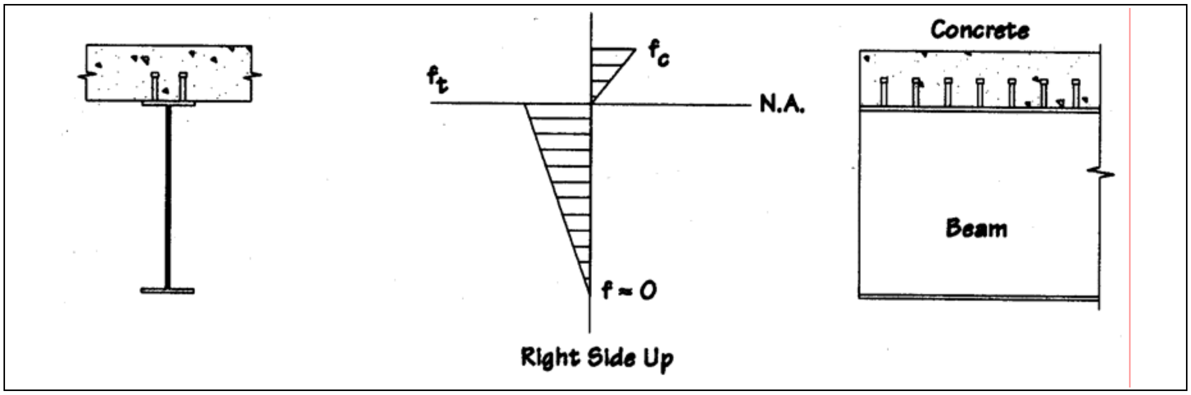

Fig. 119. Stress diagram of a beam after turned right side up (The Fort Miller Co., Inc., 1998)


Fig. 120. Longitudinal Inverset Bridge System unit (The Fort Miller Co., Inc., 1998)

Fig. 121. Transverse Inverset Bridge System units (The Fort Miller Co., Inc., 1998)

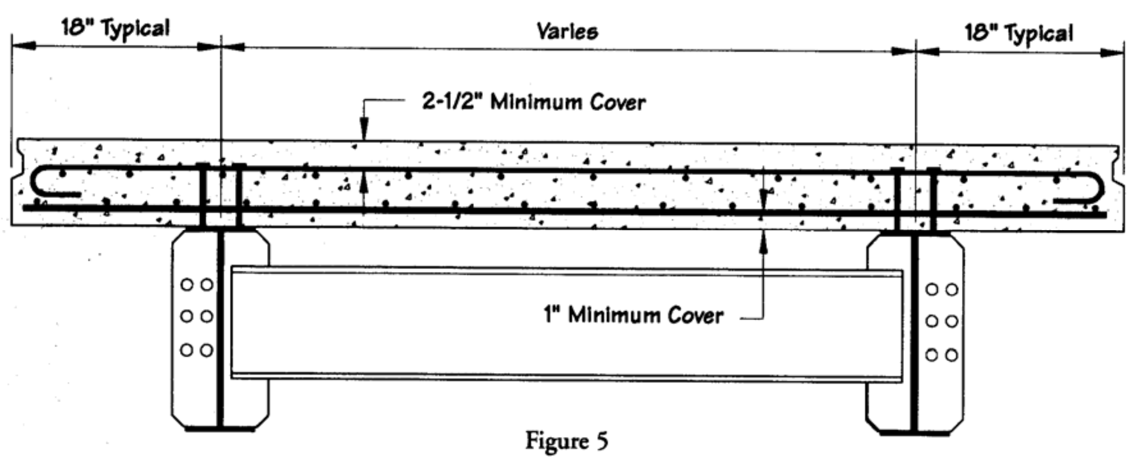

Fig. 122. Typical Inverset Bridge System longitudinal unit cross section (The Fort Miller Co., Inc., 1998)

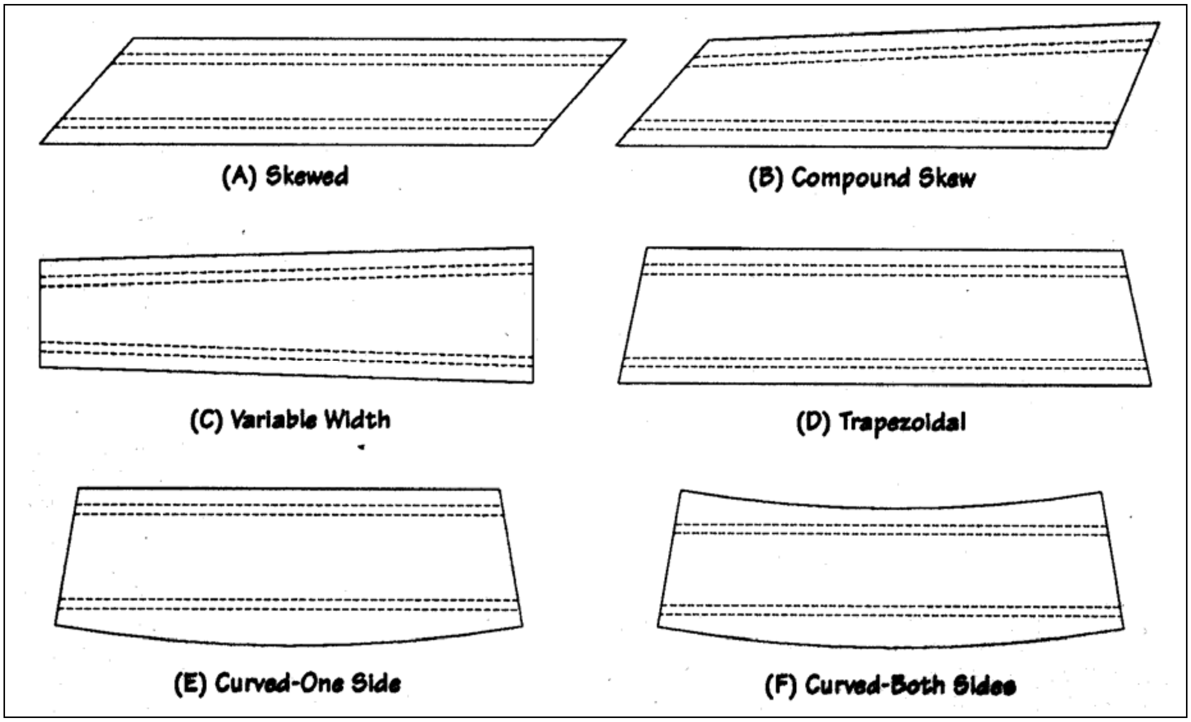

Fig. 123. Inverset Bridge System unit shapes (The Fort Miller Co., Inc., 1998)

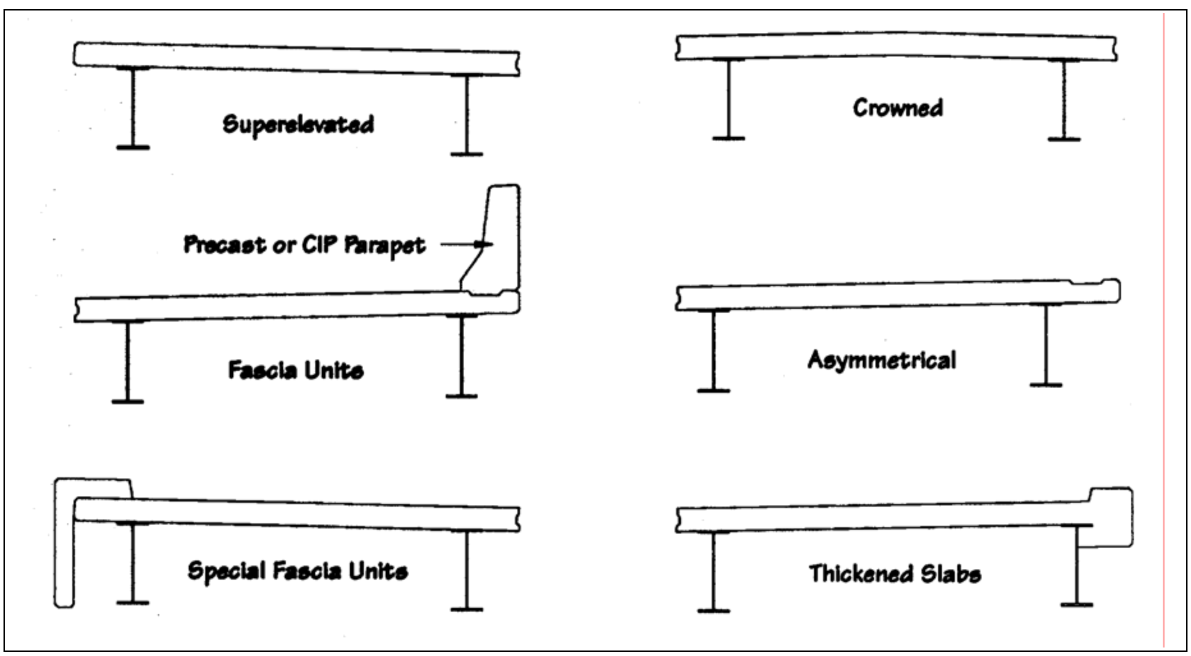

Fig. 124. Inverset Bridge System unit cross sections (The Fort Miller Co., Inc., 1998)

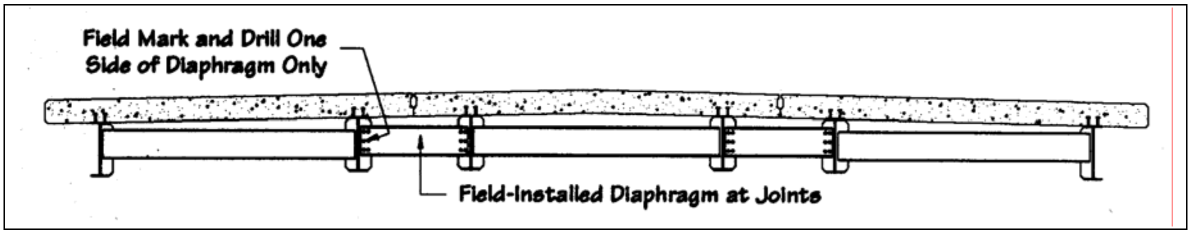

Fig. 125. Bridge cross section with three Inverset Bridge System units (The Fort Miller Co., Inc., 1998)

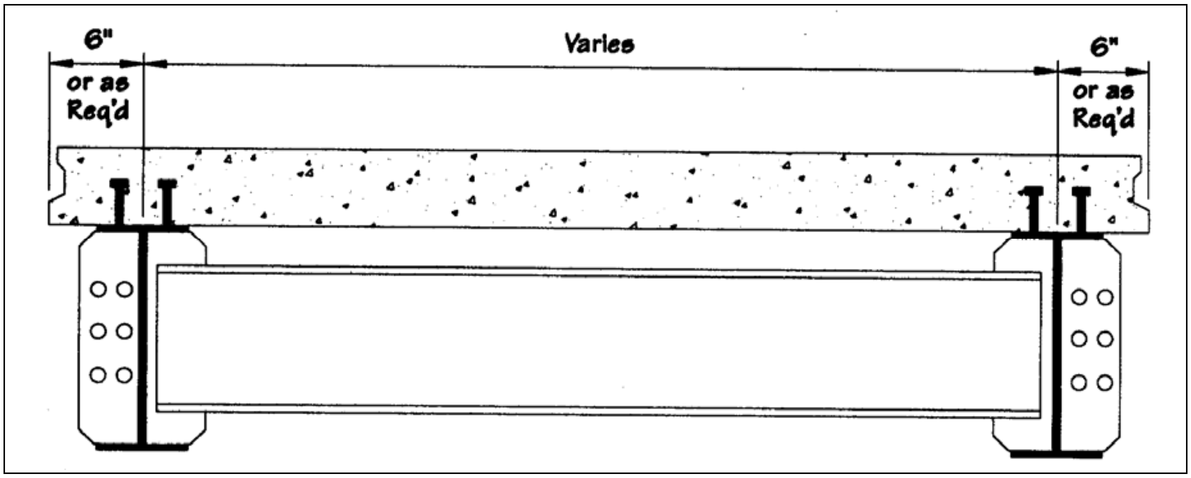

Fig. 126. Typical Inverset Bridge System transverse unit cross section (The Fort Miller Co., Inc., 1998)

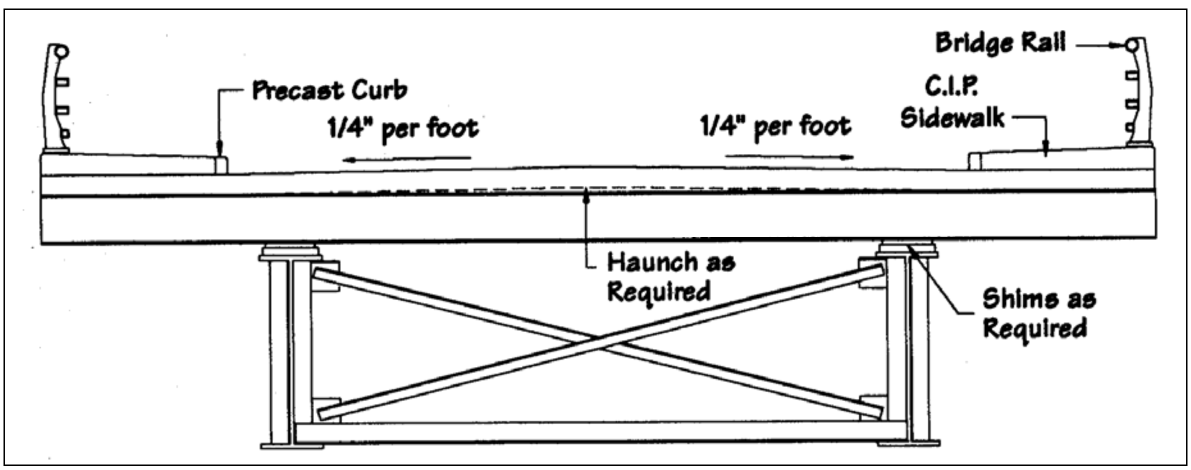

Fig. 127. Crowned Inverset Bridge System transverse unit (The Fort Miller Co., Inc., 1998)

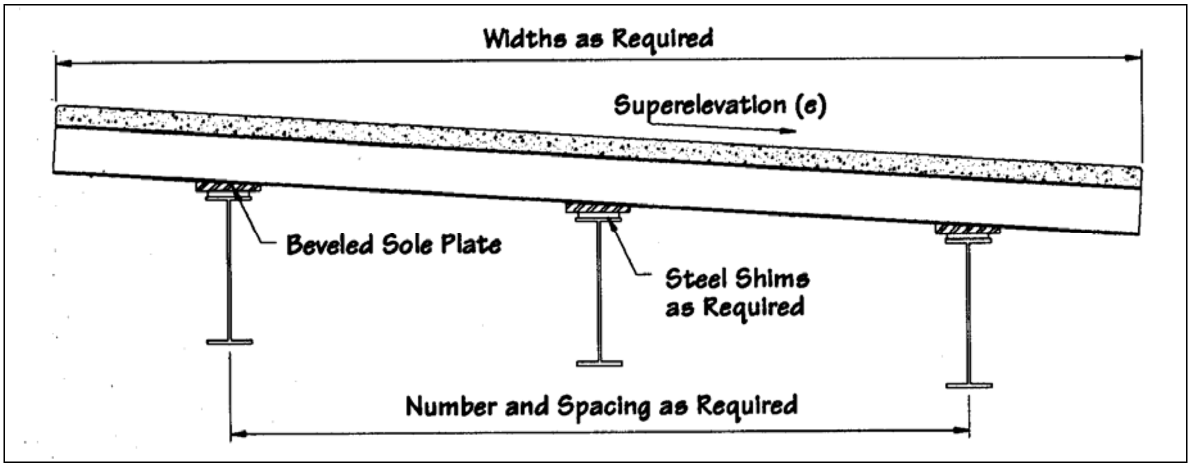

Fig. 128. Detail for super elevated decks (The Fort Miller Co., Inc., 1998)

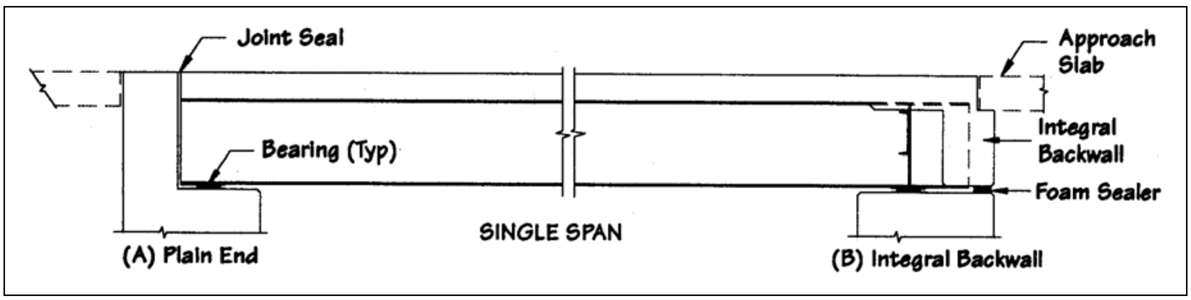

Fig. 129. Typical single span Inverset Bridge (The Fort Miller Co., Inc., 1998)

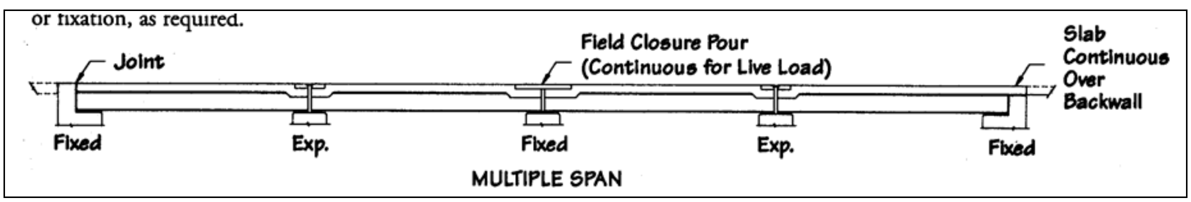

Fig. 130. Typical multiple span Inverset bridge (The Fort Miller Co., Inc., 1998)

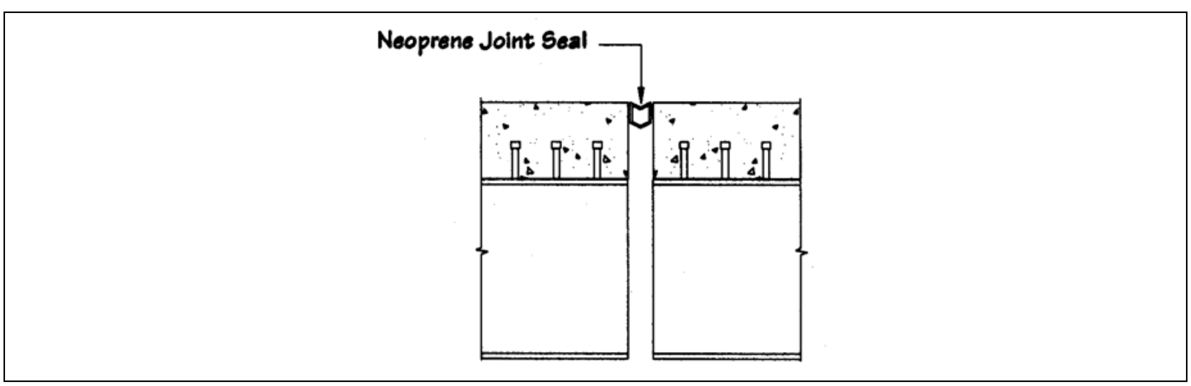

Fig. 131. Typical joint detail with neoprene (The Fort Miller Co., Inc., 1998)

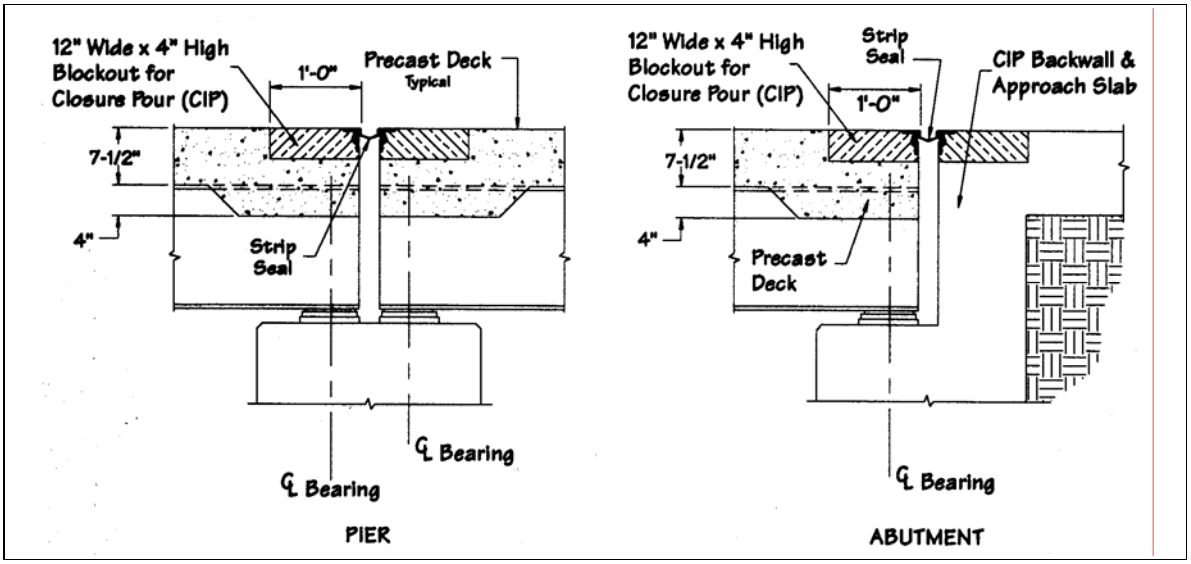

Fig. 132. Typical joint detail with strip seal (The Fort Miller Co. Inc., 1998)

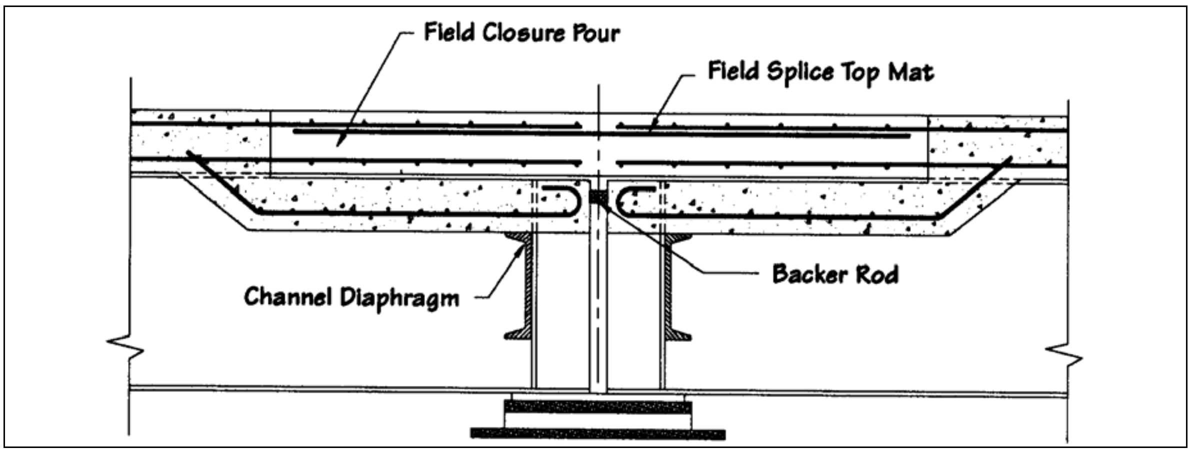

Fig. 133. Detail for continuous spans (The Fort Miller Co. Inc., 1998)

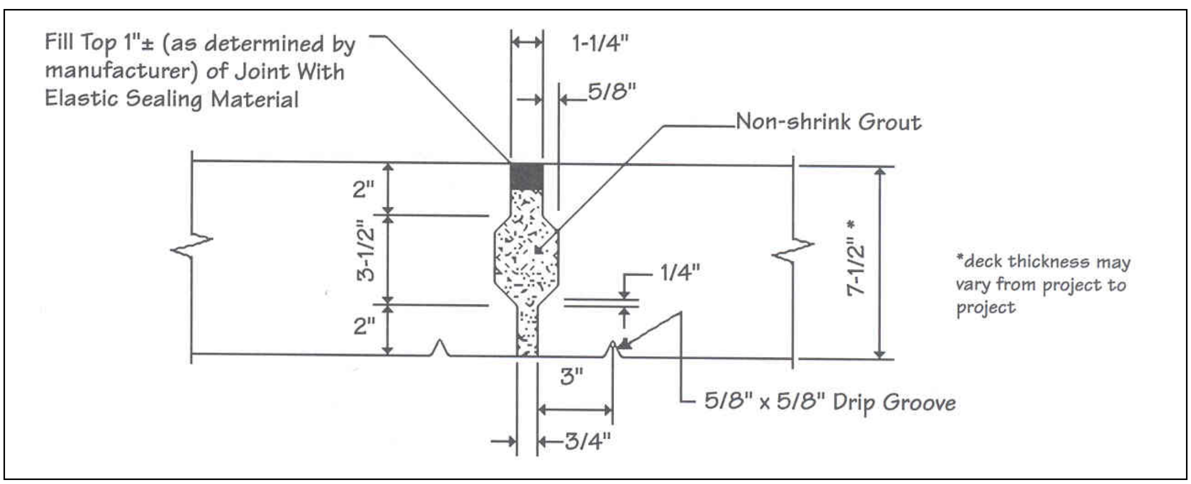

Fig. 134. Typical joint detail with strip seal (The Fort Miller Co. Inc., 1998)

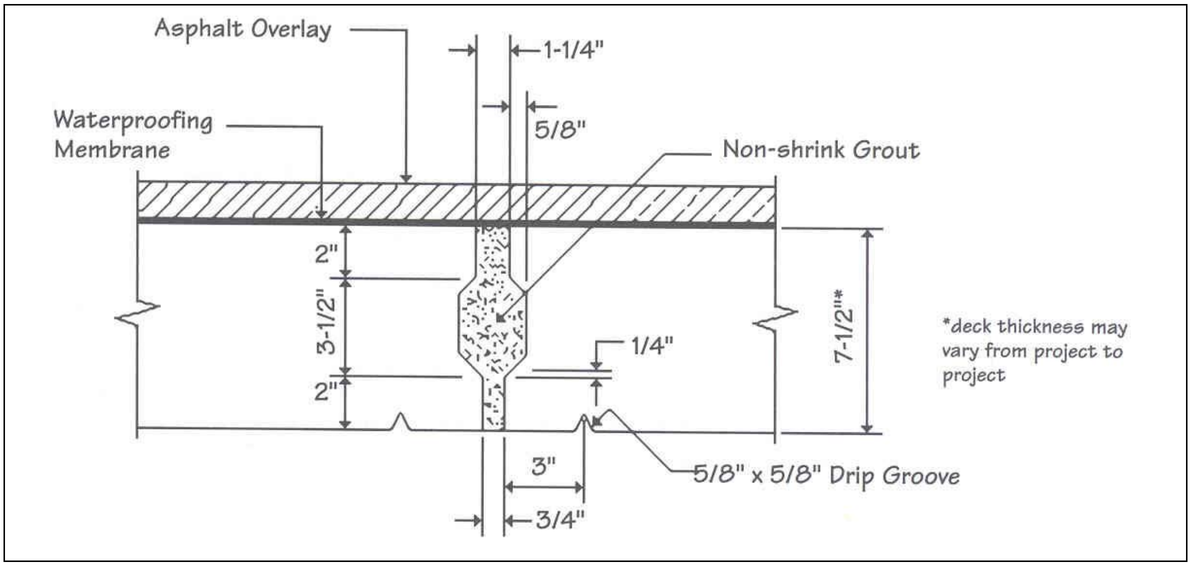

Fig. 135. Typical joint detail with asphalt overlay (The Fort Miller Co. Inc., 1998)