NCHRP 12-65

FULL-DEPTH, PRECAST-CONCRETE BRIDGE DECK PANEL SYSTEMS

FINAL REPORT

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

Sameh S. Badie
The George Washington University,
Washington DC

and

Maher K. Tadros
Amgad F. Girgis
University of Nebraska-Lincoln,
Lincoln, Nebraska

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Sameh S. Badie, Assistant Professor of Civil Engineering, Civil and Environmental Engineering Department, The George Washington University, was the principal investigator. Coauthors of this report are Maher K. Tadros, The Charles J. Vranek Distinguished Professor of Civil Engineering, and Amgad F. Girgis, Research Assistant Professor, Department of Civil Engineering, University of Nebraska-Lincoln.

The following individuals provided assistance during various phases of the project: Walter Mesia, Nghi Nguyen, Parul Patel and Krissachai Sriboonma, graduate research students of The George Washington University, Karen A. Bexten, senior engineer and partner of Tadros Associates, LLC, and Carlos Encarnacion and Yuri V. Jukarev, graduate research students and Kelvin J. Lein, Senior Laboratory Technician of the University of Nebraska-Lincoln.

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NATIONAL SURVEY AND LITERATURE REVIEW

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A.1 NATIONAL SURVEY

Limited publications, on bridges built with precast panels between 1994 and the present time, were available in the literature. Therefore, the research team prepared a national survey to collect this information. The survey was sent to the state DOTs in the United States and Canada, members of the PCI Bridge Committee, members of the TRB A2C03 Concrete Bridges Committee, consulting firms and precast concrete producers. The following sections give the survey and a summary of its results.

A.1.1 Hard Copy of the Survey

TRB TRANSPORTATION RESEARCH BOARD

OF THE NATIONAL ACADEMIES

July 27, 2003

Subject: Survey of Bridge Owners
NCHRP Project 12-65
Full-Depth Prestressed-Concrete Bridge Deck Panel Systems

To Whom It May Concern:

The National Cooperative Highway Research Program (NCHRP) has contracted with George Washington University to conduct the subject study under the direction of Dr. Sameh Badie. The objectives of this study are to develop recommended guidelines for design, fabrication, and construction of full depth, precast-concrete bridge deck panel systems and to develop durable and rapidly constructed connections between panels.

Under this contract Dr. Badie is required to review relevant practice, performance data, research findings, physical test results, and other information related to the design, fabrication, and installation of full-depth, precast-concrete bridge deck panel systems. These requirements are being satisfied, in part, by means of the enclosed survey. Your cooperation in completing this survey will be appreciated.

NCHRP is sponsored by the American Association of State Highway and Transportation Officials in cooperation with the Federal Highway Administration and is administered by the Cooperative Research Programs Division of the Transportation Research Board; it was created in 1962 to accelerate research in acute problem areas that affect highway planning, design, construction, operations, and maintenance. Pooling state resources enables a concerted attack on the major problems faced by Member Departments.

Very truly yours,

David B. Beal, P.E.
Senior Program Officer, NCHRP

Enc:

THE NATIONAL ACADEMIES
Advisers to the Nation on Science, Engineering, and Medicine

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Phone: 202 334 2934
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SURVEY

National Cooperative Highway Research Program (NCHRP)

Project No. 12-65

"Full-Depth, Precast-Concrete Bridge Deck Panel Systems"

Name of the respondent: ____

Title: ____

Address: ____

____

Phone number: ____

Fax number: ____

E-mail: ____

Please, send all replies and questions to:

Dr. Sameh S. Badie

Civil and Environmental Engineering Department

The George Washington University

801 Twenty Second Street, NW

Academic Center, Suite 638

Washington DC 20052

Phone: 202-994-8803, Fax: 202-994-0127, E-mail: badies@gwu.edu

If you prefer to provide your response electronically, you can download this document from the following website

http://www.geocities.com/badies_2000/NCHRP_12_65_Survey

and send it back to

badies@gwu.edu

Please, respond by August 15, 2003

Introduction

This survey is part of the National Cooperative Highway Research Program (NCHRP) Project 12-65, "Full-Depth, Precast-Concrete Bridge Deck Panel Systems," sponsored by the National Academy of Sciences (NAS). The project contractor is the George Washington University (GWU), Washington DC. Your response to this survey is greatly appreciated.

Full-depth precast concrete panels provide an efficient, quick and reliable system for the construction of bridge decks, especially for deck replacement projects in high traffic areas. This is because no cast-in-place (CIP) concrete is needed except for small joints between the prefabricated pieces. In addition, the prefabricated pieces are produced under high quality control in precast concrete plants, which results in highly durable products. Also, precast concrete deck panels undergo little shrinkage, and creep at the time they are installed on a bridge. Further, the temperature drop experienced in cast-in-place concrete following the intense heat of hydration can create excessive thermal stresses that do not exist in pre-cured precast concrete.

Most currently used precast deck panel systems require use of longitudinal post-tensioning or composite concrete overlays, or both. This tends to slow down construction due to the post-tensioning and curing operations. It also reduces the flexibility required in over-night and weekend replacement projects.

The objectives of the research project are to:

(1) Develop and evaluate connection details for full-depth precast concrete deck systems that can be used with steel and prestressed concrete girders, (2) Develop recommended guidelines for design, fabrication, and construction of full-depth precast concrete bridge deck panel systems, and (3) Develop recommended specification language and commentary for the AASHTO LRFD Bridge Design Specifications and the AASHTO LRFD Bridge Construction Manual necessary to implement the recommended systems. ***The Emphasis of this project*** will be on systems that do not require longitudinal full-length post-tensioning or concrete overlays.

Q1: Has your organization used any full-depth precast concrete deck panel systems in highway bridges during the last 10 years?

Yes _____

No _____ (please, give reasons):

Incremental cost ______

Lack of specifications or guidelines ______

Unsatisfactory performance in the past ______

Other (specify) ____

Q2: Approximately, how many bridges, utilizing full-depth precast concrete panels, have you constructed during the last 10 years? ______

Q3: Approximately, how many square feet of full-depth precast concrete panels have you constructed in the past 10 years? _____ sq. ft

Q4: Of the bridges listed in answer to Questions 3 & 4, please, indicate the type of transverse (normal to traffic direction) reinforcement.

Pretensioned in the precast yard %_____

Post-tensioned in the field %_____

Conventionally reinforced %_____

Partially pretensioned and partially conventionally reinforced %_____

Other (specify)______%_____

Q5: How were the panels connected in the longitudinal direction (parallel to the traffic direction)?

Using longitudinal post-tensioning %_____

Splicing reinforcing bars using commercial mechanical couplers %_____

Using special mechanical devices %_____

Other (please specify) %_____

Q6: What is the percentage of the systems built compositely with the supporting girders? ______%_____

Q7: Did you use an overlay?

Yes _____ (if Yes, please, provide the overlay type and percent of decks)

Asphalt %_____ Thickness ______

Concrete %_____ Thickness ______

Other (specify) _________________%_____ Thickness ______

No ____ (If No, did you provide special treatment to the top surface of the precast panels to provide for ride-ability?

Yes ______ No ______

If yes, what type? Roughening in the precast plant during production ______

Grooving in the precast plant during production ______

Grinding in the field after construction ______

Sand blasting in the field after construction ______

Other (specify) ___________________________________

Q8: What is your overall evaluation of the performance of full-depth precast concrete deck panels?

Excellent ______

Good ______

Fair ______

Poor ______

Please comment and indicate whether or not you will use full depth precast deck panel systems again in future projects: ____

____

____

____

____

____

Q9: Have you developed guidelines or specifications for design, fabrication or construction of full depth precast concrete panel systems?

Yes ______ (please, attach a copy of the specifications)

No ______

Q10: Successful grouting of the panel-to-panel and the deck-girder joints is considered one of the key elements of having a durable and high performance deck. Have you developed specifications for the grout properties and the grouting process?

Yes ______ (please, attach a copy of the specifications)

No ______

Q11: In order to simplify the connection between the concrete deck and the steel girders and to facilitate deck removal in the future, the state of Nebraska has used 1¼ in. diameter steel studs successfully. One 1¼ in. steel stud is equivalent to two 7/8 in. studs. Do you see any problems with use of individual or clustered 1¼ in. steel studs with full depth precast deck panels.

Yes ______

No ______ (please, give reasons) ____

____

____

Q12: AASHTO Specifications stipulate a maximum spacing of the shear connectors between the girder and the deck of 24 inches. Relaxing this limit could simplify deck placement and removal. Do you see a need for research on the performance of shear connectors at 4, 6 or even 8 feet?

Yes ___ No ___

Please comment: ____

____

Q13: Please, provide the name, phone number and e-mail address of one person on your staff who can help in answering questions on issues related to design and construction with precast concrete deck panels.

Name: ____

Title: ____

Phone: ____

E-mail: ____

Q14: Are you interested in receiving a copy of the findings of this survey?

Yes ___ No ___

THANK YOU FOR YOUR HELP

A.1.2 Summary of the Survey Results

A.1.2.1 List Of States Where Full-Depth Precast Concrete Deck Panel Systems In Highway Bridges Have Never Been Used During The Last 10 Years And The Reasons Why This System Has Not Been Used

- Alberta, AB (CANADA): Incremental Cost and Lack of specifications and guidelines.
- Arizona Department of Transportation (AZDOT): Due to the construction issues and performance.
- California Department of Transportation (CADOT): Incremental cost and Lack of specifications or guidelines.
- Florida Department of Transportation (FLDOT): Tried half-depth panels and were unsatisfied.
- Hawaii Department of Transportation (HIDOT): Lack of specifications or Guidelines.
- Kansas Department of Transportation (KSDOT)
    - Building joints into a deck system (Even with an overlay – reflective cracks)
    - Long-term performance vs. speed of construction does not warrant their use. We would probably require post-tensioning and an overlay.
    - The ability to obtain a smooth riding deck without an overlay.
    - Future maintenance considerations.
    - Obtaining composite action with the girder.
    - Grout between the deck panels and the girders.
- Massachusetts Department of Transportation (MADOT): Expensive and the large number of joints.
- Maryland Department of Transportation (MDDOT)
    - They have used this type of design in the past and consider it a viable technique.
    - They have not had a suitable application in the last 10 years, however, and so have no nearest experience to relate.
- Minnesota Department of Transportation (MNDOT)
    - No experience.
    - Considering this design for future projects.
- Mississippi Department of Transportation (MSDOT): They have had satisfactory performance with CIP concrete decks.
- North Carolina Department of Transportation (NCDOT): Lack of specifications or guidelines.
- North Dakota Department of Transportation (NDDOT): Believe that cast in place decks provide better, maintenance-free service than precast panels.

- New Jersey Department of Transportation (NJDOT): They have selected CIP decks using HPC over precast due to cost and surfacing issues.
- New Mexico Department of Transportation (NMDOT): Lack of specifications or guidelines.
- Nevada Department of Transportation (NVDOT): Lack of Experience.
- Ohio Department of Transportation (OHDOT): Evaluated the use of full depth panels on an ODOT bridge study. They did not compete in cost with Spliced Deck Bulb-Ts.
- Ontario Department of Transportation (ONDOT)(CANADA)
    - They have used this system on one span of a three span bridge as a trial 10 years ago.
    - Because of higher cost they did not use this approach on other bridges.
- Oregon Department of Transportation (ORDOT): Concerned about surface smoothness.
- Tennessee Department of Transportation (TNDOT)
    - If used as new installation, could not be readily replaced while maintaining traffic in the future.
    - Incremental cost and Lack of specifications or guidelines.
- Washington Department of Transportation (WADOT): In the only job designed to date (in Washington State) as a full-depth precast deck slab, the contractors all bid on the CIP slab option as being less expensive.
- Wisconsin Department of Transportation (WIDOT): Incremental cost and Lack of specifications or guidelines
- Wyoming Department of Transportation (WYDOT): Concern with joint integrity, salts reaching the steel girders; no design or construction experience.

A.1.2.2 List Of States Where Full-Depth Precast Concrete Deck Panel System Has Been Used In Highway Bridges During The Last 10 Years.

<u>Alaska Department of Transportation (AKDOT)</u>

Q1: 2 bridges

Q2: 10000 sq. ft

Q3: 100% Conventionally reinforced.

Q4: 100% by Grout keys

Q5: 100%

Q6: Roughening in the precast plant during production

Q7: GOOD

Q8: Used in remote locations where CIP concrete is unavailable. Plan to use more in the future.

Q9: YES

Q10: YES

Q11: Perhaps, Precast deck panels are often HS concrete ($f_c^{'}$ > 6ksi). More studs are required for strength limit.

Q12: YES

Q13: YES

Q14: no answer

<u>California Department of Transportation (CALTRAN)</u>

On a phone interview, the research team found that CALTRAN used a hollow core slab system.

Q1: N/A

Q2: N/A

Q3: 100% Pretensioned in the precast yard (panels)

100% transversely Post-tensioned in the field

Q4: Only used for small simple spans with no longitudinal joints.

Q5: 40%

Q6: 100% Polyester Concrete overlay of 1 in. thickness.

Q7: GOOD

Q8: NO

Q9: YES. Use standard specifications.

Q10: NO. But in general we use 7/8” studs and they perform well, if 1¼” used the concrete slab has to be thick enough to carry the capacity of such stud.

Q11: YES. If such spacing is used, we require that a research done in this area before we use it.

Q12: YES

Q13: no answer

Q14: YES

<u>Colorado Department of Transportation (CODOT)</u>

Q1: 1 bridge

Q2: 17,400 sq. ft

Q3: 100% Pretensioned in the precast yard.

Q4: 100% by hoop dowel and closure pour

Q5: 0%

Q6: 100% Asphalt with waterproof membrane overlay of 3” thickness.

Q7: GOOD

Q8: YES, faster.

Q9: NO

Q10: NO

Q11: NO, if there is an evidence of testing and performance.

Q12: YES

Q13: YES

Q14: YES

Illinois Department of Transportation (ILLDOT)

Q1: 4 bridges

Q2: 15,000 sq. ft

Q3: 100% Conventionally reinforced.

Q4: 100% using longitudinal post-tensioning

Q5: 100%

Q6: 100% concrete of 2½” thickness.

Q7: GOOD

Q8: YES

Q9: YES

Q10: Welding the larger 1¼-diameter studs may be a problem.

Q11: NO

Q12: YES

Q13: no answer

Q14: YES

Kentucky Department of Transportation (KYDOT)

Q1: 2 bridges

Q2: 270,000 sq. ft.

Q3: 100% Conventionally reinforced.

Q4: 50% splicing reinforcing bars using commercial mechanical couplers

Q5: 100%

Q6: 100% latex overlay of 1¼” thickness.

Q7: GOOD

Q8: Don’t know, will continue to evaluate. Will probably use again.

Q9: NO

Q10: NO

Q11: NO

Q12: NO. Would not recommend spacing over 24”.

Q13: YES

Q14: YES

New Brunswick Department of Transportation (NBDOT, CANADA)

Q1: 1 bridge

Q2: 4255 sq. ft.

Q3: 100% Post-tensioned in the field

100% Conventionally reinforced.

Q4: 100% using post-tensioning

Q5: 0%

Q6: 100% Asphalt overlay

Q7: GOOD

Q8: NO

Q9: NO

Q10: YES

Q11: no answer

Q12: no answer

Q13: no answer

Q14: YES

New York Department of Transportation (NYDOT)

Q1: 5 bridges

Q2: 900,000 sq. ft.

Q3: 20% conventionally reinforced.

80% post-tensioned in the precast yard in the transverse direction.

Q4: 80% using longitudinal post-tensioning

Q5: 100%

Q6: 20% asphalt and 80% concrete overlay with water proofing membrane.

Q7: GOOD

Q8: Based on our experience full-depth precast concrete systems perform very well when the joints are leak proof. Post-tensioning across the joints seems to be the best way to achieve this. Overlays improve riding quality and may also improve durability.

Q9: YES

Q10: YES

Q11: NO. We have used clustered stud shear connectors and had no negative problems from it.

Q12: NO. Shear connector design and detailing is not the main problem with this system. Connections between the panels are the most vulnerable part of the system.

Q13: YES

Q14: YES

<u>Texas Department of Transportation (TXDOT)</u>

Q1: Only two projects in history of our department:

- SPUR 326 over ATOSF RR (1986) of our department

- A series of tied arches over US 59 on Houston (late 1990's):4 bridges (Huzard, Woodhead, Donlavy and Mindell st. over US 59)

Q2: 55,000 sq. ft. for 4 bridges mentioned above.

Q3: 100% pretensioned in the precast yard (Houston tied arches)

100% post-tensioned in the field (Houston tied arches)

The 1986 Lubbock project was conventionally reinforced in both directions.

Q4: 100% using longitudinal post-tensioning (Houston tied arches)

Q5: 0%. The Houston tied arches had precast panels that were attached to the tie of the two parallel arches. No supporting girders were involved. The 1986 Lubbock project was designed as a composite system.

Q6: 100% concrete overlay of 2" to 4" thickness

Q7: GOOD. Texas does not have the exposure condition as other states

Q8: They will be used where warranted due to difficult access, long length of repetitive structure or urban environments that require minimal traffic disruption. Where speed is not critical, the use of stay-in-place structural panels or steel deck forms will probably still be the preferred system. The NUDECK system is something we may explore.

Q9: YES

Q10: NO

Q11: NO. However, need to consider availability of installation equipment and possibility of more defects in the fused metal of large diameter studs.

Q12: YES. Fewer connections translate into less area exposed to durability problem. Need to ensure no serviceability problems are induced with wide spacing. Non-composite systems should be addressed.

Q13: YES

Q14: YES

Utah Department of Transportation (UTDOT)

Q1: 1 project under design and 2 projects are anticipated in near future.

Project #1 (single span) Deck and superstructure are precast, under design

Project #2 (4-span continuous, 45degree skew) Deck rehab, plans and details of this project will be received by the 1st week of October.

Project #3 (single span) Deck rehab, under design

Q2: Current designs are for conventionally reinforced and post tensioned in the field.

Q4: Second project will use longitudinal post tensioning

Q5: Will be 100% on both projects for seismic load transfer. Mill maximum ¼ in. of the panel.

Q6: Will be co-polymer overlay of 3/8 in. thickness.

Q8: Planning on continued implementation. Look forward to results of this survey.

Q9: YES

Q10: YES

Q11: NO, Do not see any problems with their use.

Q12: YES. Composite action and horizontal load transfer needs to be investigated for longer minimum spacing of shear connectors.

Q13: YES

Q14: YES

Virginia Department of Transportation (VADOT)

Q1: 2 projects. VADOT has used full depth precast panel systems in two projects. These are the I-95 James River Bridge Replacement in Richmond and the Route 7 Bridge over Route 50 in Fairfax County.

Q2: See the paper by Babaei et al.

Q4: See the paper by Babaei et al.

Q5: 100%

Q6: See the paper by Babaei et al.

Q8: Mixed results with partial deck panels.

Q9: YES

Q10: YES

Q11: NO, Do not see any problems with their use.

Q12: YES

Q13: YES

Q14: YES

Precast Concrete Producers

Central Pre-Mix Prestress Co., Washington State

Q1: Manufactured full-depth precast concrete panels for 10 bridges.

Q2: 130,000 sq. ft.

Q3: 90% pretensioned in the precast yard

10% conventionally reinforced.

Q4: 5% using special mechanical devices

90% by spliced Dywidag bars

Q5: 90%

Q6: 9% asphalt overlay of thickness 2”-3”

10%concrete overlay of thickness 2”-3”

Q7: Excellent. No complaints received

Q8: N/A

Q9: N/A

Q10: NO

Q11: YES. Forming pockets, shooting studs and grouting pockets is expensive.

Q12: YES

Q13: no answer

Q14: YES

A.1.2.3 Quantitative Summary Of The Survey

	Total number	Percentage
Total number of surveys sent	110	---
Response received	32 out of 110	35%
Never used full-depth precast deck panel systems before	22 out of 32	69%
Used this system in last 10 years	10 out of 32	31%
Do not see any problems with use of 1¼” diameter steel studs instead of 7/8” studs	9 out of 10	90%
A need for research on the performance of shear connectors at 4, 6 or even 8 feet	8 out of 10	80%
Interested in receiving summary of the survey	31 out of 32	97%

A.2 LITERATURE REVIEW

A.2.1 Introduction

Large-scale utilization of precast prestressed elements in bridge construction was started in North America in the late sixties and early seventies (*1*). This was a direct result of the significant amount of research conducted in the late sixties and the technological advances in the areas of fabrication and transportation of precast concrete elements. Until the mid seventies, use of precast concrete elements in bridges was relatively limited to the longitudinal girders, which are the main supporting elements of the superstructure.

Over the time, bridge designers continued to gain appreciation of the benefits of using precast elements, especially in the metropolitan cities where traffic volume has been growing at a fast pace. Road closures for extended periods, for rehabilitation of existing bridges or construction of new bridges, can cause serious inconvenience to the traveling public.

Public inconvenience and loss of income during bridge construction have been the driving motives to explore rapid construction methods. Precast concrete bridge elements can be used to effectively reduce construction time. A cast-in-place (CIP) concrete bridge deck slab represents a significant part of the time required to complete a superstructure of a bridge. It includes a time-consuming process of forming, reinforcing bar placement, concrete placement, and an extended period of moist curing. As a result, use of full depth precast deck panels has the potential of replacing CIP decks as a natural extension to the use of prefabricated girders in bridges.

In the following four sections, a summary is given of the literature review of bridges built with full depth precast concrete deck panels. For convenience, the summary is divided into three time periods: before 1973, between 1973 and 1994, and between 1994 and the present time. The goal of this summary is not to report all the bridges built with full depth precast panels, but to show the diversity of the panel-to-panel and panel-to-superstructure connection details.

A.2.2 Bridges Built With Full-Depth Precast Panels Before 1973

Biswas (2) gave a comprehensive report on use of full depth, precast concrete bridge deck panels in the United States. It included a limited coverage of projects before 1973. These bridges were in Alabama, Indiana and New York. Biswas (*2*) summarized his finding of applications of full depth precast panels in bridge deck construction prior to 1973 in the following points:

1. The deck girder systems were primarily noncomposite.
2. The spans did not have any skews, or superelevations (due to horizontally curved alignments).
3. More projects involved new construction than rehabilitation.
4. Fewer geometric fit-up problems were experienced with new construction than with replacement deck.
5. This method of construction was used for both temporary and permanent bridges.
6. The structures Biswas (*2*) reported on had performed well.
7. Minor problems were mainly due to partial failure of panel-to-panel joints.

Follow-up phone interviews were conducted between the research team and the designers in the agencies, where these bridges were built. The designers stated that they have believed that partial failure of the panel-to-panel connections were a result of lack of longitudinal post-tensioning and/or overlay.

A.2.3 Bridges Built With Full-Depth Precast Panels Between 1973 And 1994

Between 1973 and 1994, significant advances in the construction of full depth precast concrete deck panels were made. Many major bridges were built during this period with precast concrete panels. Some of them had spans over 1000 ft (305 m). Most of them were made composite with the beams. The following sections give a summary of some of these bridges.

A.2.3.1 California Department of Transportation (3)

The I-80 overpass project, in Oakland, California, was completed without disruption of traffic on the freeway. The bridge had 32 spans and some of them were skewed. Conventionally reinforced concrete panels were used to replace only the outside southbound lane. Figure A.2.3.1-1 shows the panel dimensions and details of the shear connections.

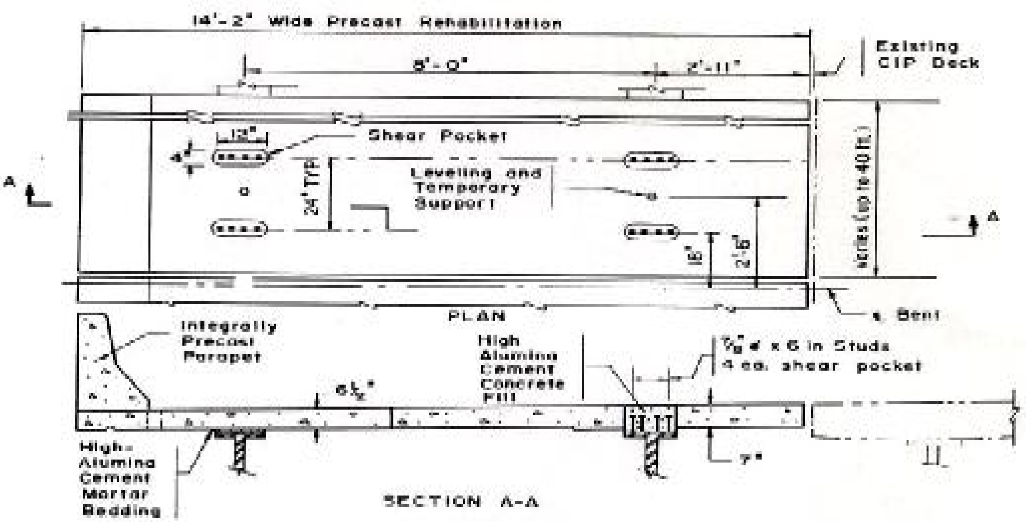


Figure A.2.3.1-1 Panel dimensions and cross section of the I-80 Overpass project

After the panels were temporarily seated, four shear studs were welded to the girders through each pocket. Using two-headed bolts, the panels were leveled and the pockets between girders and panels were filled with fast-setting concrete. No longitudinal post tensioning was used across the transverse joints between panels. Early high strength cement mortar was cast to fill the joints. Each day, the deck was removed in sections of 60 to 80 ft (18000 to 24000 mm) long by 12 ft (3700 mm) wide. The new panels were 30 to 40 ft (9 to 12 meters) long with oblong pockets for shear connectors.

A.2.3.2 Connecticut Department of Transportation (4)

The Connecticut Department of Transportation selected precast panels for redecking one of its bridges on I-84 - Connecticut Route 8 Interchange (Bridge 03200). This bridge was a six-span, composite plate girder bridge, which had a compound curvature and a vertical grade of approximately 7 percent. The bridge was 27 ft 6 in. (8400 mm) wide and approximately 700 ft

(213000 mm) long. Four different roadways and a pedestrian bridge were located beneath the structure, all of which had to remain in service during the redecking process.

The Connecticut DOT shut down the ramp for construction, using a nearby existing detour. Two major reasons for avoiding night closures and day openings were: (1) the superstructure was composite; therefore nighttime removal of existing deck would only be done in small amounts due to the difficulty of demolishing areas around existing shear connectors; and (2) night closure would leave a small non-composite section between the existing deck and new deck when the bridge would be reopened. This resulted in unacceptable overstresses in the girders of the non-composite section near mid-span.

Due to the fact the bridge was on a super elevated curvature with no crown, each panel was designed as a trapezoid with a uniform thickness of 8 inch (200 mm). The reinforced concrete panels were 8 ft (2400 mm) wide and 26 ft-8 in. (8200 mm) long as shown in Figure A.2.3.2-1. A minimal amount of prestressing force was applied transversely to prevent cracking during handling and installation.

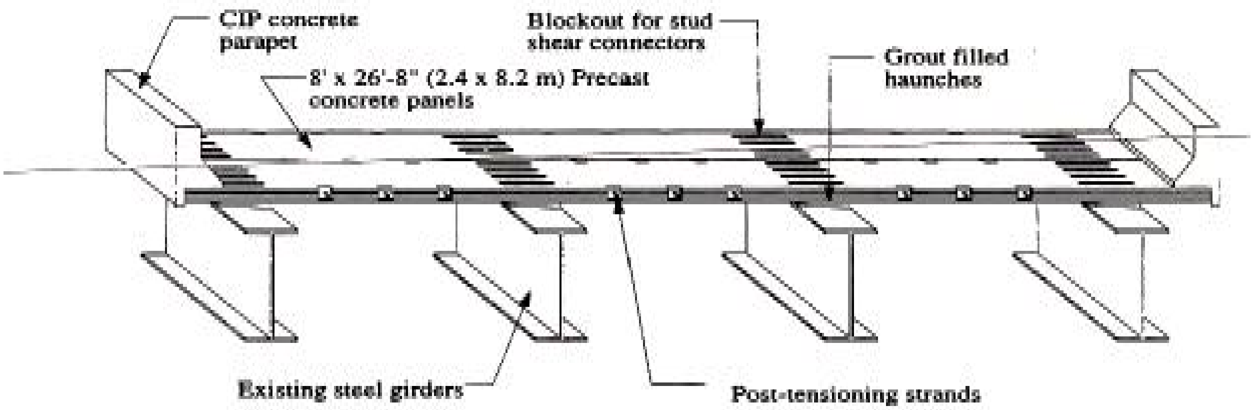


Figure A.2.3.2-1 Typical precast panels of Bridge 03200

A leveling bolt was used to adjust the elevation and grade of the panels. Grout pockets were provided for the shear connectors (Figure A.2.3.2-1). The transverse joints had a shear key filled with high strength non-shrink grout. This left a 1/4 in. (6 mm) gap at the bottom of the key to allow tolerance of the panel sizes (Figure A.2.3.2-2). For longitudinal post-tensioning, a stress of 150 psi (1.0 MPa) was chosen arbitrarily in the positive moment zone. However, the maximum post-tensioning stress of 300 psi (2.1 MPa) was needed to maintain the transverse joints in compression under dead and live loads in the negative moment zone.

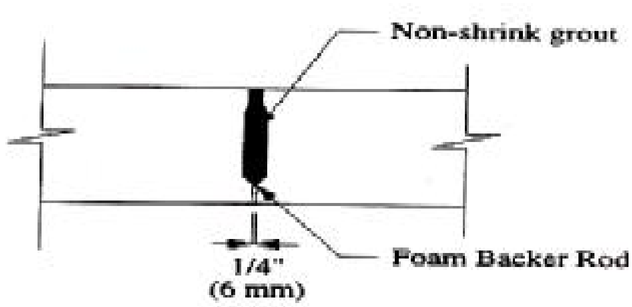


Figure A.2.3.2-2 Transverse joint detail

The roadways and pedestrian bridge beneath the construction area were shielded from falling debris. The precast panels were placed one span at a time and adjusted to grade using the leveling bolts, Figure A.2.3.2-3 and the post-tensioning strands were inserted in oversized ducts through the panels. The panels were post-tensioned after the grout in the transverse joint demonstrated the designated compressive strength.

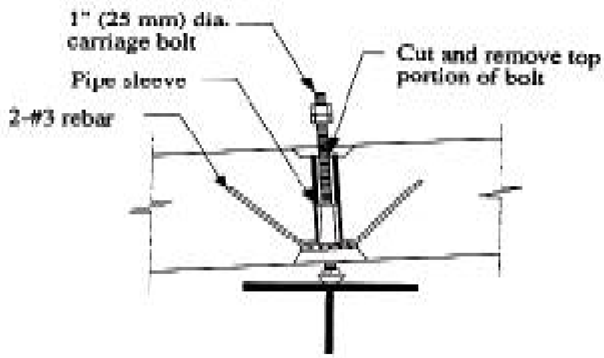


Figure A.2.3.2-3 Leveling bolt detail of Bridge 03200

Shear connectors were placed through the pockets in the panels and filled with flowable high strength non-shrink grout (Figure A.2.3.2-4). After installing the precast panels for one span, a small CIP closure section of the deck was poured.

Reconstruction of the entire bridge was accomplished in 48 calendar days, which included 5 days of minimal work due to inclement weather. Work was not performed on most weekends; however, some night work was done in demolition of the existing deck. The cost of the CIP deck including demolition, parapets, and wearing surface was approximately $71 per square foot and $75 for the precast deck. This difference in cost was acceptable because the main purpose for the use of precast panels was to reduce construction time and inconvenience of travelers.

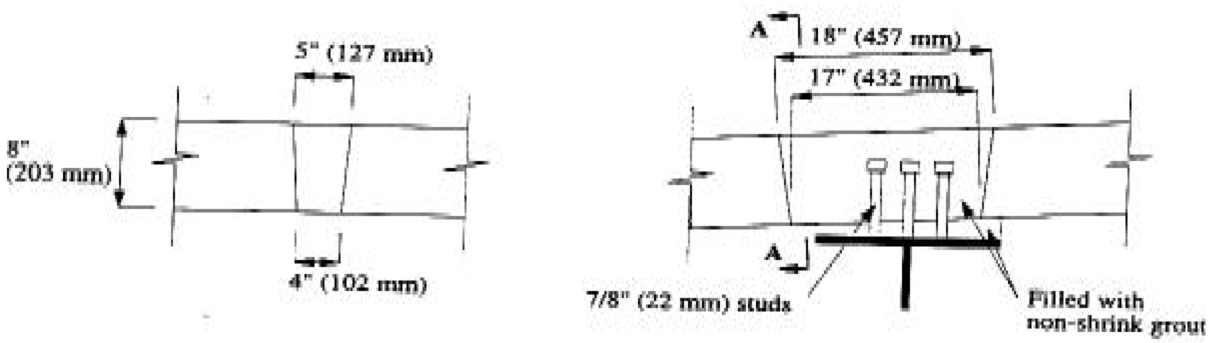


Figure A.2.3.2-4 Panel-to-girder connection detail

Six months after construction, inspections showed that the performance of the new deck was excellent. Leakage did not occur at the joints, nor was there cracking in the deck.

A.2.3.3 Indiana State Highway Commission (5)

Bloomington Bridge was a two-lane, 125 ft (38000 mm) span, pony truss. The timber deck in the bridge was replaced with the precast, prestressed panels normally 4 ft (1200 mm) wide with the tongue-and-groove joint shown in Figure A.2.3.3-1. The slabs were secured to the steel beams with railroad tie-down clips (Figure A.2.3.3-2) and then post-tensioned together to 90 psi (620 KPa). The joints had a neoprene sheet to provide for transfer of the post-tensioning across the joints.

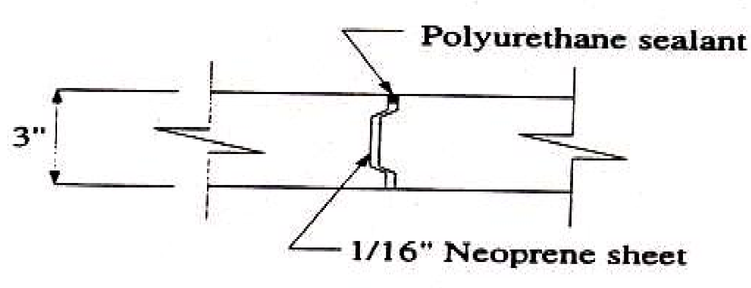


Figure A.2.3.3-1 Shear key detail of the Bloomington Bridge

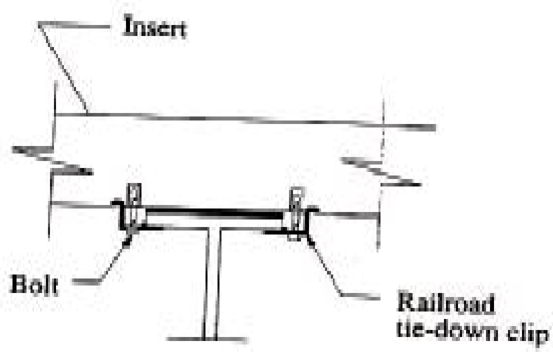


Figure A.2.3.3-2 Tie-down connection of the Bloomington Bridge

Cracking, spalling, and leakage at the panel joints were observed after five years of service. The elevation between slabs varied by as much as 1/4 in. (6 mm). Under repeated wheel loads, damage was experienced. The damage primarily consisted of cracking and spalling, primarily due to improper materials and application techniques. An inspection in 1980 showed that water leakage appeared to be a continuing problem. Some of the tie-down clips had corroded extensively, and one was completely destroyed.

The Knightstown Bridge was a new, three-span continuous steel beam bridge having spans of 70, 70, and 60 ft (21000, 21000 and 18000 mm). The deck slabs were 38 ft-4 in. (12000 mm) wide, 7 in. (180 mm) thick at each end, and 10½ in (270 mm) thick at the center as shown in Figure A.2.3.3-3. The shear keys and connections to the steel beams were identical to those of the Bloomington Bridge.

There were eight cracks approximately 2 ft (600 mm) long, perpendicular to the joints. Cracks appeared during the post-tensioning due to irregularities in the width of the joints at the top of the slab. The defective joints were less than 1/8 in. (3 mm) in width and half of them were completely closed after the slabs were post-tensioned together.

The only immediate effect appeared to be joint leakage because no sealant could be installed in the closed joint. However, a few months after the bridge was opened, the concrete in the vicinity of the closed joints began to spall. Six years later, an inspection noted that the joint sealant did not effectively bond to the edge of the slabs. In addition, many of the beam clips exhibited signs of severe corrosion due to the continual problem of joint leakage. Little evidence was found of surface spalling of the concrete.

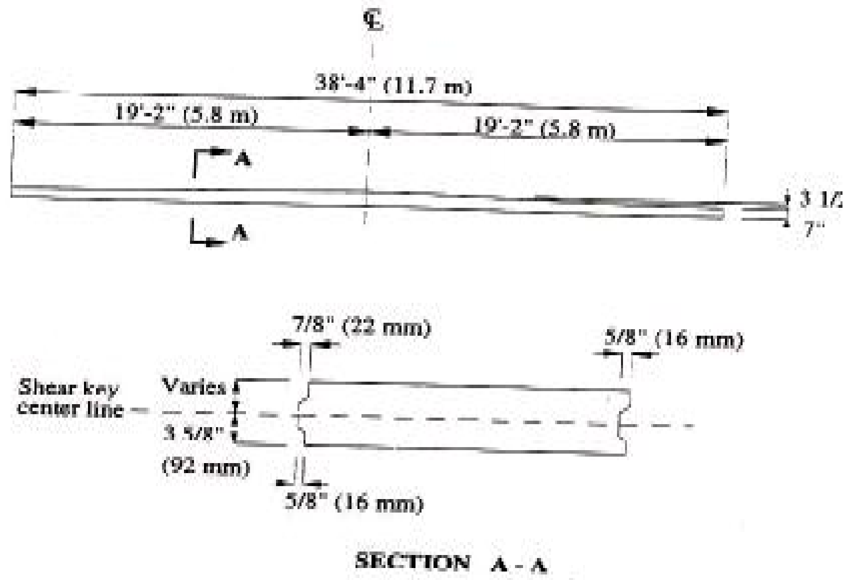


Figure A.2.3.3-3 Elevation and shear key details for Knightstown Bridge

A.2.3.4 Maryland State Highway Administration

The Woodrow Wilson Memorial Bridge spanned the Potomac River, south of Washington, D.C. (*6*). This bridge was constructed between 1959 and 1962. Peak daily traffic of this bridge exceeded 110,000 vehicles by 1979, and serious deterioration of the reinforced concrete deck was evident. The widening and replacement of the deck was completed with full-depth precast, lightweight concrete panels. The panels were post-tensioned transversely and longitudinally.

The original exterior girders and continuous stringers supported the precast panels. Cast-in-place polymer concrete was used as a bearing material on the top flanges. The Maryland and Federal Highway Administrations tested the methyl methacrylate polymer concrete and mortar before they approved the project. Specifications for the methyl methacylate polymer included 4,000 psi (27.6 MPa) compressive strength within one hour at temperatures from 20 to 100 F (-6.7 to 37.8 C) and 8,000 psi (55.2 MPa) at 24 hours. Polymer concrete was also used for transverse joints between panels and for closure pours at the end of longitudinally post-tensioned deck segments.

The top flange of the stringers and exterior girders were sandblasted and painted with zinc rich primer. Steel bearing plates, pad forms, and predetermined height shim packs were then set in place. After a precast panel was installed in position, polymer concrete was poured in the bearing pad forms through holes in the panel. After the polymer concrete gained 4,000 psi (27.6 MPa) compressive strength, the pad form and shim packs were removed.

The typical panel was 46 ft $7^1/4$ in. (14200 mm) wide, 10 to 12 ft (3050 to 3660 mm) long and 8 in (203 mm) thick with a 5 in. (127 mm) haunch at the exterior girder as shown in Figure A.2.3.4-1. The panels were pretensioned transversely at the precast plant and post-tensioned longitudinally at the construction site through segments 140 to 285 ft (42700 to 86900 mm) in length. This segment averaged 17 panels in length.

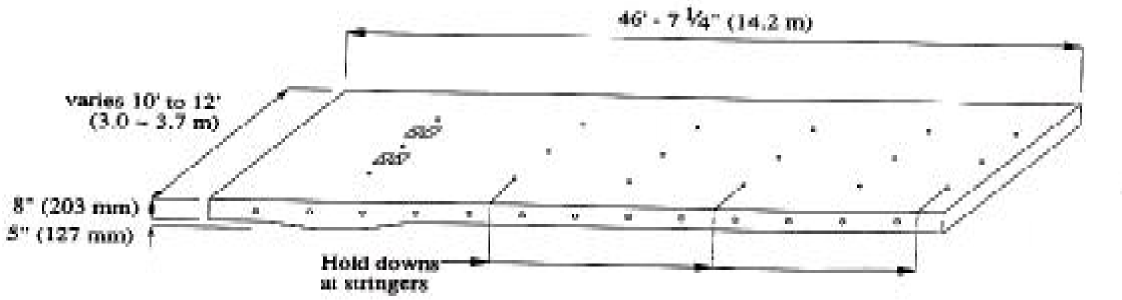


Figure A.2.3.4-1 Typical precast concrete panel of the Woodrow Wilson Memorial Bridge

Figure A.2.3.4-2 shows details of the panel-to-panel and panel-to-girder connections. Due to the geometry of the shear key, only the top half of the transverse connection between panels was grouted.

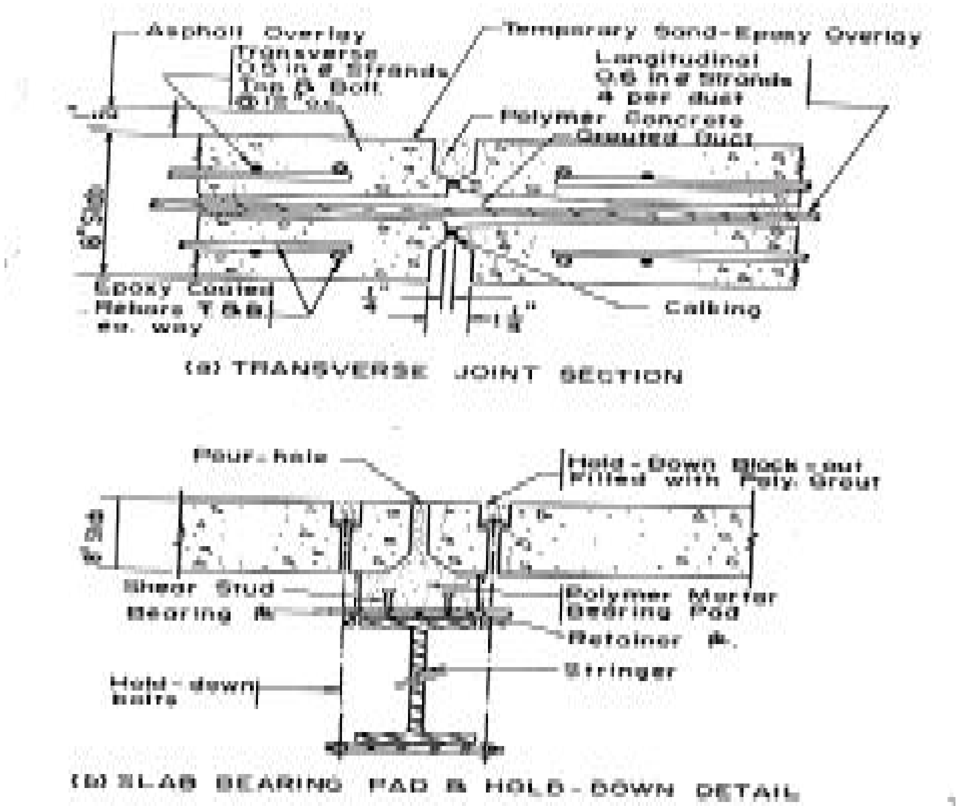


Figure A.2.3.4-2 Joint details of the Woodrow Wilson Memorial Bridge

Single lanes were maintained for the two-way traffic during nighttime deck replacement. The original six lanes were reopened prior to morning rush hour by using temporary open grid deck panels with steel extension barrier. The 1026 precast panels were set in 129 nights, and 350 calendar days were spent to replace the six-lane bridge deck which was over one mile (1.61 km) long. Recent inspection of the deck has shown deterioration of the panel-to-panel joints and spalling of the deck concrete.

A.2.3.5 New York State Department of Transportation (7)

The New York State Department of Transportation (NYDOT) used precast panels for redecking projects on several steel girder bridges.

The first project, a 1,040 ft (317000 mm) suspension bridge, spanned the Rondout Creek near Kingston, New York. The precast deck was selected for this bridge because of the need to load and unload the suspension bridge in a specific sequence, and due to the speed of construction. Each precast panel was 9 ft (2700 mm) wide and 24 ft (7300 mm) long. Panel thickness varied from 7 in. (178 mm) at the crown to 6 in. (152 mm) at the edges. The panels were transversely pretensioned to control handling stresses. The transverse joint used to connect the panels, was a simple V-shaped male-female dry joint tied together by rods, as shown in Figure A.2.3.5-1.

A three-span structure over the Delaware River between Sullivan County, New York and Wayne County, Pennsylvania, was 675 ft (206000 mm) in total length. Two panels connected at the crown-point were used across the bridge. The panels were connected by epoxy shear keys transversely and longitudinally. Horizontal shear connectors consisted of a single shear stud in each pocket, as shown in Figure A.2.3.5-2. Reflective cracks appeared along the longitudinal joint and were later patched. No longitudinal post-tensioning was applied to the deck and the bridge had reportedly performed well.

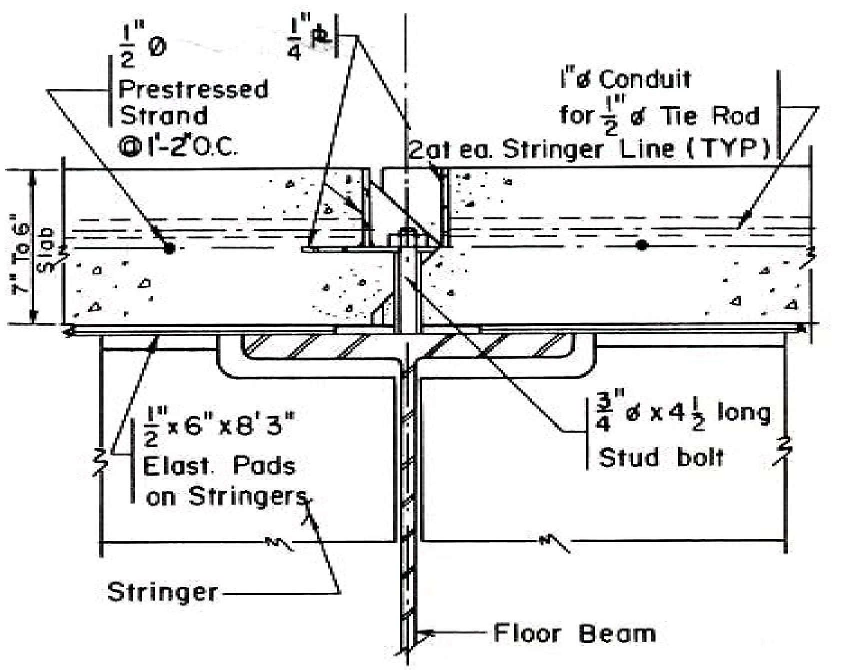


Figure A.2.3.5-1 Joint details of the Rondout Creek Bridge

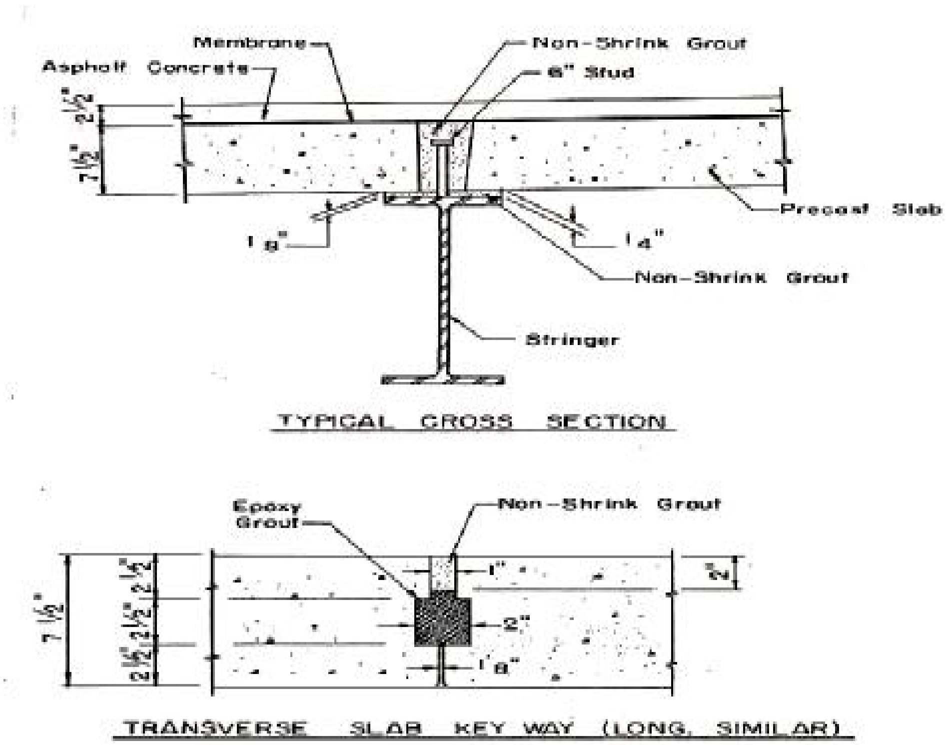


Figure A.2.3.5-2 Connection details of the Delaware River Bridge

In 1979, a three-span bridge was built in southern Erie County, Southwestern Blvd. Bridge over Cattaraugus Creek. This bridge used precast panels and was 540 ft (150000 mm) in length with three equal spans. The joints for the precast panels of the bridge were identical to that of the Delaware River Bridge. The shear connectors were single threaded stud in each pocket and they were used to tie down the panels to the steel girders, as shown in Figure A.2.3.5-3. A seating steel assembly made of vertical plates and angles was used to support the panels before grouting and adjusting the cross slope of the bridge.

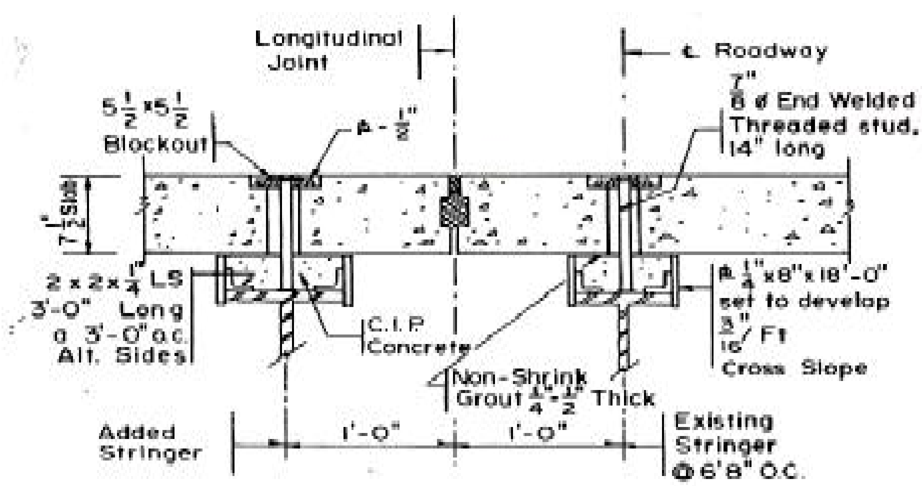


Figure A.2.3.5-3 Connection details of the Southwestern Blvd. Bridge over Cattaraugus Creek

In 1982, NYDOT replaced the cast-in-place deck of the Batchellerville Bridge with a precast panel deck system. The bridge had 21 spans with a total length of 3075 ft (938000 mm). The bridge superstructure was made of two arched steel trusses supporting cross-floor beams. Curved conventionally reinforced full-width precast panels were used because the roadway had a crown at its centerline,. Each panel was supported longitudinally by the two arched steel trusses and transversely by two adjacent cross-floor beams. Composite action with the superstructure was created by using steel studs welded to the cross-floor beams. No longitudinal post tensioning was used. The panels were connected in the longitudinal direction using welded steel plates. Figure A.2.3.5-4 shows the connection details. The precast panels were overlaid with a 2-in. (50 mm) asphalt concrete mix.

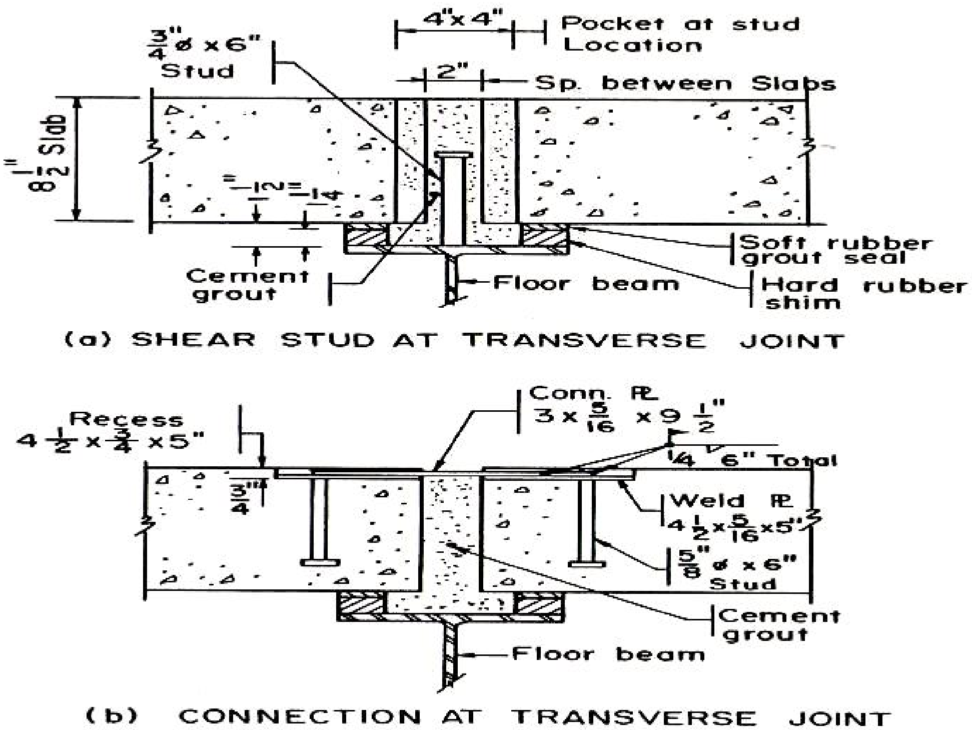


Figure A.2.3.5-4 Panel-to-panel connection details of the Batchellerville Bridge

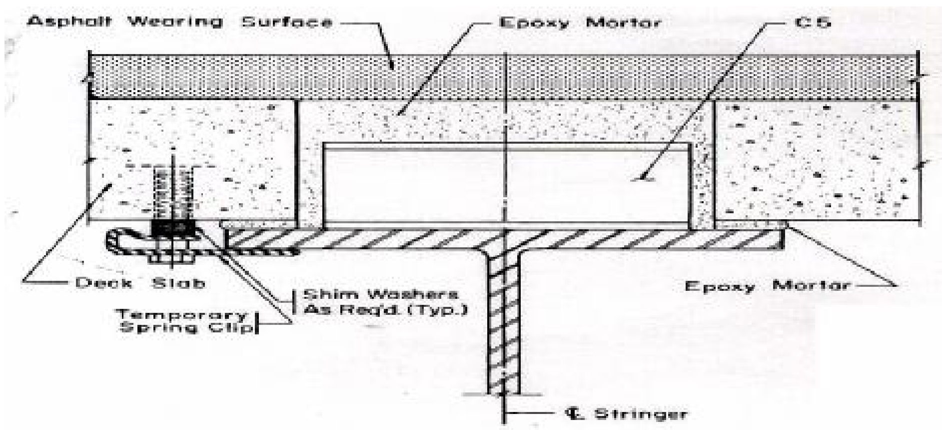
A.2.3.6 New York State Thruway Authority

The New York State Thruway Authority (NYSTA) had selected full-depth precast concrete decks for replacement of deteriorated concrete deck at three different locations. Some of the features common to all three bridges were: (1) conventionally reinforced precast panels were used, (2) the panels were made composite with the girders, and (3) Epoxy mortar, one part epoxy to two parts sand, was used at the joints.

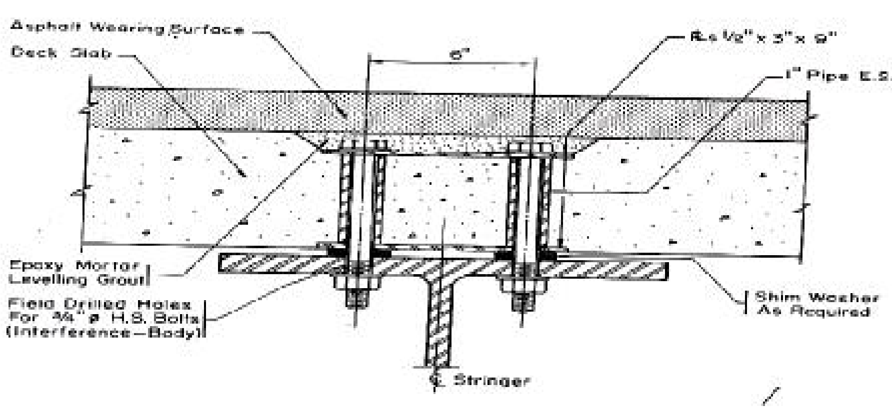
The first project in Amsterdam, New York (*8*), included an experimental installation of welded connectors and bolted connectors to evaluate the effectiveness of each connection. The bridge had four simple spans, 33, 59, 66 and 60 ft (10000, 18000, 20000, and 18000 mm). The deteriorated deck of one-half of the 66-ft (20000 mm) span was replaced by precast panels. Field welded standard channel sections were used as shear connectors on four panels, and a "dry" system detail using long high strength bolts was used on three precast panels, as shown in Figure A.2.3.6-1.

In the bolted connection, the panels were placed using steel shims for leveling. After the holes of the bolts were drilled in the top flange of the steel girder through the sleeves in the panels, high strength bolts were fastened. Achievement of full tension in the bolts could not be ascertained. Breakage of the precast slab due to excessive tensioning was expected. This connection detail was not used in any subsequent projects. In the welded connections, the panels were seated and channel connectors were welded to the steel girders. The pockets in the panels at connectors were filled with epoxy grout.

Six years later, there was no evidence of any difference in performance among the bolted panels, welded panels, and cast-in-place deck. The decks, which use a sheet membrane and asphalt overlay, did not exhibit any moisture leakage.



(a) Welded Connection



(b) Bolted Connection

Figure A.2.3.6-1 Connection details of the New York Thruway Experimental Bridge

The second bridge *(9)* was a single 50 ft (15000 mm) span overpass on the Thruway, crossed Krum Kill Road near Albany. 7½ in. (190 mm) thick conventionally reinforced precast panels were placed on the top flanges of the steel girders on which an epoxy mortar bed was provided. Headed studs were welded to the steel girders through the pocket openings in the panel. These pockets were then filled with epoxy mortar. Two precast panels were used across the bridge and were connected at the crown because the roadway had a crown. Reinforcing bars were extended from both panels in a 3-ft (910 mm) wide longitudinal joint at the crown and cast-in-place concrete was used to fill the joint. No longitudinal post tensioning was applied to the deck. Some cracks were developed in the panels and treated with epoxy sealer. Although several joints had shown signs of leakage, inspection of the bridge indicated that performance was satisfactory.

The third bridge (9) was located in the entrance ramp of the Thruway at Harriman, New York and consisted of three spans. The roadway was on horizontal curve as well as a vertical curve and all of the spans were skewed. Therefore, precast panels with different geometries were used. A greater thickness of epoxy on the top flange was used when compared to the first two bridges due to the super-elevated deck surface. The bridge exhibited no particular problems in construction. However, cracking and some leakage through the panels were detected.

A.2.3.7 Pennsylvania Department of Transportation

The replacement of the deteriorated deck of a non-composite Clark's Summit Bridge on the Pennsylvania Turnpike was completed for the Clark's Summit Bridge. It was necessary to maintain traffic on half of the bridge while redecking the other half; therefore, precast panels were chosen to prevent influence of traffic vibration to the CIP deck. Other reasons for using precast panels included: (1) elimination of dangerous field work; (2) reduction of redecking time from one-half to one-third; and (3) comparable cost to a CIP deck. The top flanges of the steel girders were sandblasted after removal of the old deck. Neoprene strips were attached to the top flange with epoxy as shown in Figure A.2.3.7-1. Epoxy grout was poured between strips with a slight overfill, and panels were placed on the neoprene strips. The inserts and bolts were located near the edge of the slab. Transverse joints had a shear key, which was filled with a non-shrink grout.

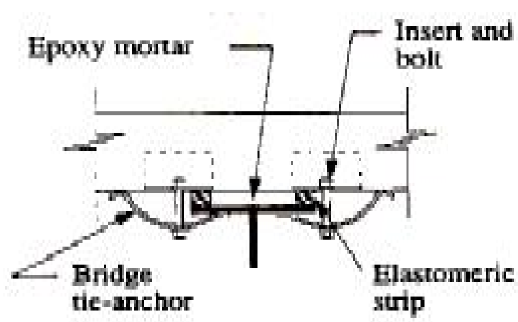


Figure A.2.3.7-1 Connection detail of Clark's Summit Bridge

A.2.3.8 The Ministry of Transportation of Ontario, Canada (10)

The Ministry of Transportation of Ontario had selected precast concrete panels for the Queen Elizabeth Way-Welland River Bridge. The main purpose for this selection was to reduce the time of reconstruction and inconvenience of the travelers.

The structure selected for redecking consisted of two southbound lanes, which total 954 ft (290780 mm) in length and 40 ft (12260 mm) wide. The existing reinforced concrete deck was $7^1/_2$ in. (190 mm) thick. The eighteen spans consisted of three four-continuous spans and two three-continuous spans. The south end three-continuous span received precast panels.

The precast panels were 43 ft 6 in. (13260 mm) long, 7 ft - 11 in. (2420 mm) wide, and $8^7/_8$ in. (225 mm) thick, as shown in Figure A.2.3.8-1. Each panel had twelve blockouts for the groups of shear connectors. Each group consisted of eight to twelve $^{7}/_{8}$ in. (22 mm) diameter shear studs as required for each specific location (Figure A.2.3.8-2).

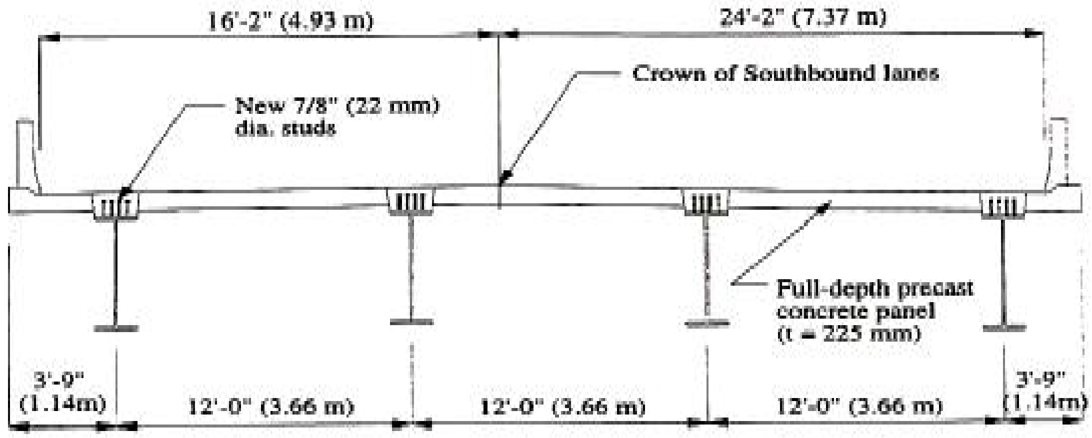


Figure A.2.3.8-1 Typical cross section of the Queen Elizabeth Way-Welland River Bridge

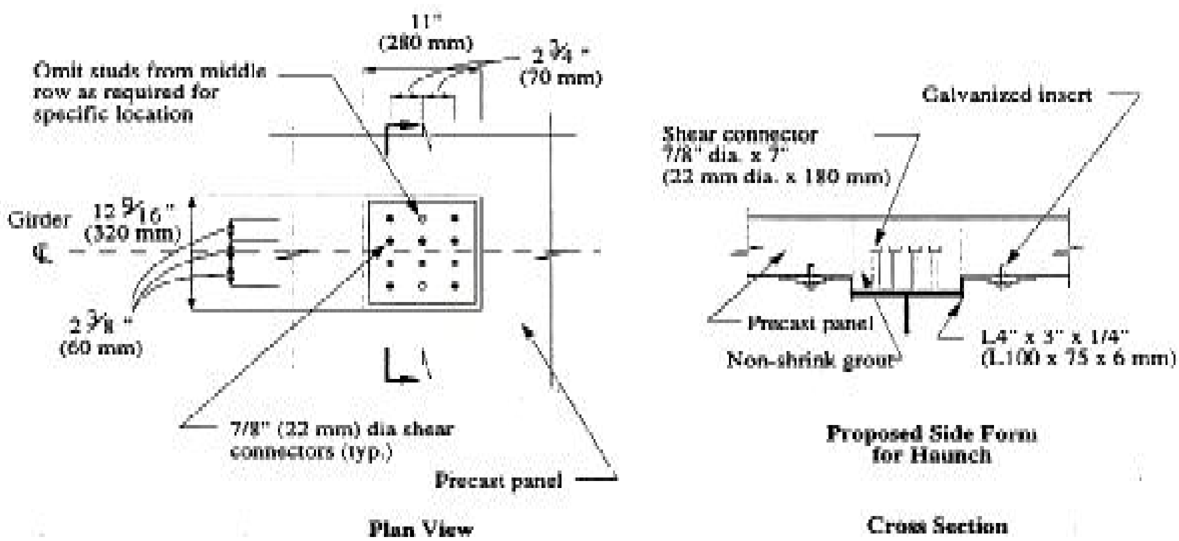


Figure A.2.3.8-2 Connection Detail of the Queen Elizabeth Way-Welland River Bridge

Polyethylene corrugated ducts were placed at mid-depth for post-tensioning. The concrete strength of the panel was 5000 psi (35 MPa) and epoxy coated reinforcing steel was used. Transverse prestressing was not applied because the deck cross-slopes made the system complicated. Approximately $2^{3}/_{8}$ in. (60 mm) of haunch was provided between the precast panels and top flanges in order to accommodate deflections of girders and cover plates at splices.

Eight $1^{3}/_{16}$ in. (30 mm) diameter leveling screws per panel were used to adjust for grade. Transverse joints were then filled with grout. Longitudinal post-tensioning was provided with four $^{5}/_{8}$ in. (15 mm) diameter strands, spaced at 1 ft - 8 in. (520 mm) on center over the intermediate supports and 3 ft 5 in. (1040 mm) in the end span. The final prestress at the intermediate supports was 435 psi (3.0 MPa), which secured uncracked and fully composite section in the negative moment regions.

After longitudinal post-tensioning, the shear studs were welded through the blockouts. The blockouts and haunches were filled with 5,000 psi (35 MPa) prebagged proprietary non-shrink grout. Figure A.2.3.8-3 shows the contractor's modified side-forms for haunch.

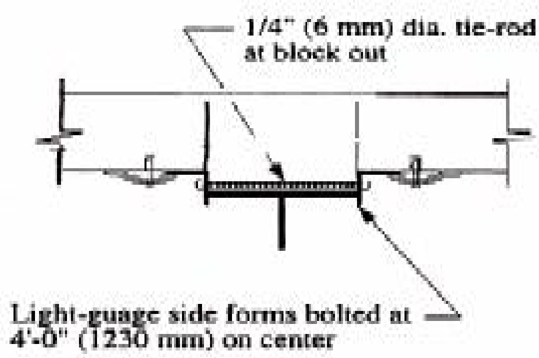


Figure A.2.3.8-3 Contractor's modified side-forms for the haunch

The cost of placing cast-in-place concrete deck was Canadian $200 per square meter, including the installation of shear connectors, but excluding the barrier walls, expansion joints, water proofing, and paving. Full-depth panels were installed for a comparable cost of Canadian $300 per square meter, which provided significantly faster construction and less inconvenience to travelers.

The project also included an extensive test program. The test program included: (1) laboratory tests for shear connectors; (2) load testing of a full size prototype assembly; (3) punching shear testing of the concrete deck in the trial assembly and also the finished bridge deck; and (4) measurement of prestress loss in the composite girders.

The testing program resulted in the following conclusions: (1) Shear studs may be welded in groups to maintain full composite action; (2) different height of shear studs in a group will improve the performance of the group of studs; (3) interface bond and friction may allow shear transfer for the serviceability and fatigue limit state; and (4) the transverse joint does not resist punching shear as well as the center of the panels. However, the panel withstood a load equal to six times the legal load limit and performed similar to a cast-in-place concrete deck in resisting punching shear.

A.2.3.9 The Japanese Highway Public Corporation

In Japan, there has been an increasing use of precast panels for bridge deck construction in order to solve labor shortage problems, obtain higher and consistent quality, and to accomplish rapid construction.

Shin-kotoni-kouka Bridge was a continuous five-span, non-composite plate girder bridge (*11*). The Japanese Highway Public Corporation decided to use precast prestressed concrete panels rather than reinforced concrete panels for new construction of Sapporo-Nishi bound lanes of the bridge. The main reasons for this selection included: (1) smaller thickness; (2) improved crack control; and (3) easier handling. A typical precast panel was 33 ft - 1 in. (10095 mm) long, 4 ft - 11 in. (1490 mm) wide, as shown in Figure A.2.3.9-1. The thickness varied from 7 in. (180 mm) at mid-span between girders to 10 in. (250 mm) at support on the girders. The panels were transversely prestressed at the precast site and longitudinally post-tensioned at the construction site. The construction sequence of placing the deck was similar to that of Welland River Bridge in Ontario, Canada and Bridge 03200 in Connecticut.

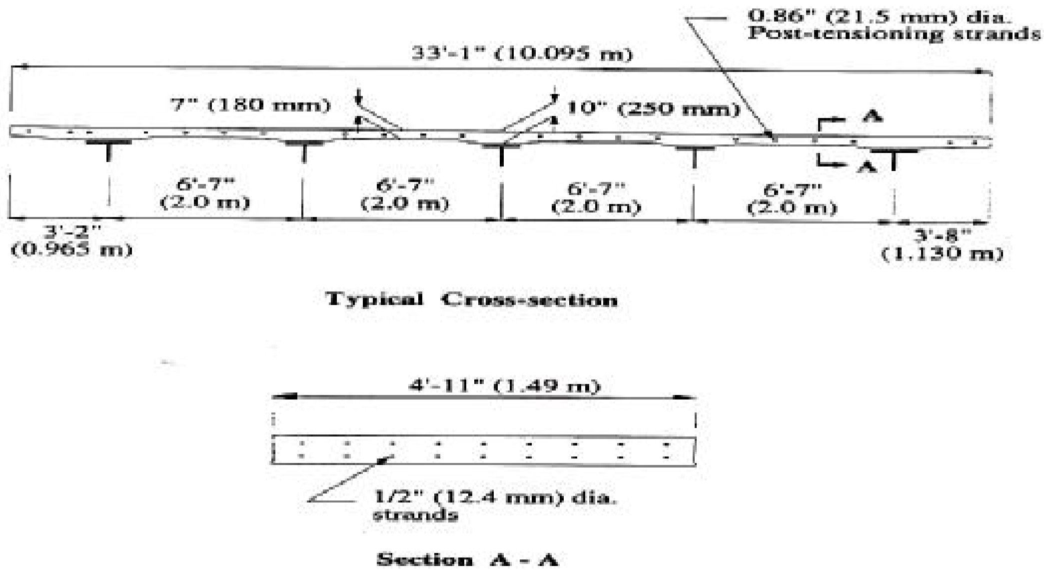


Figure A.2.3.9-1 Typical precast prestressed concrete panel of the Shin-kotoni-kouka Bridge

Suehiro Viaduct in Osaka, Japan

The Japan Highway Public Corporation had selected channel-shaped biaxially prestressed precast panels for a new construction of the Suehiro Viaduct on the Kansai International Airport Line in Osaka, Japan (*12*). The cross-section of precast panels is shown in Figure A.2.3.9-2. The bridge was a three-span continuous non-composite plate girder bridge consisting of two approximately 38 feet wide separate superstructures for each bound. Each superstructure consisted of five main girders at 7 ft - 2 in. and three spans of 123, 113, and 112 ft (37400, 34500, and 34150 mm). The construction was completed in October 1993.

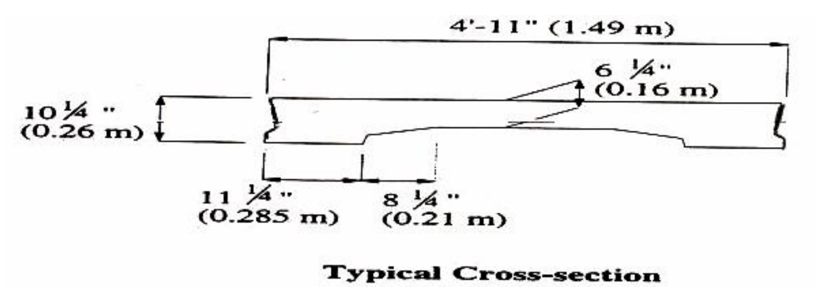


Figure A.2.3.9-2 Cross section of the Suehiro Viaduct precast panels

Three series of static tests and two series of dynamic fatigue tests for the precast decks were performed to prove their adequacy towards adoption in the Suehiro Viaduct. Static tests typically resulted in punching shear failure after the cracking spread over three panels. Ultimate strength of three panels with longitudinal post-tensioning was approximately 1.7 times higher than that of single panels. This proved the efficiency of longitudinal post-tensioning for load transfer over transverse joints. Overall, the system performed very well under both static and dynamic fatigue loading.

Precast panels were cast at a precast plant near the construction site. Erection of 72 panels for one of two parallel superstructures required only four days. Case study for CIP reinforced concrete deck was conducted to compare with the precast deck system. On-site work day and labor force of this system was approximately one-third of CIP deck. The cost of the system was much higher than CIP, however, the combined savings of construction time and labor force reduced this impact when the effect on the whole project was carefully studied.

A.2.4 Bridges Built With Full-Depth Precast Panels Between 1994 And Present Time

Most of the information presented in this section was collected from the responses to the national survey to collect this information. Some of the information was collected from conference papers and personal contact with design engineers of state DOTs.

A.2.4.1 Alaska Department of Transportation

Alaska Department of Transportation (ADOT) has had long and successful experience with full depth precast concrete bridge deck panels. It started about 20 years ago with the construction of Route FAP 65, where about 20 bridges were built using full depth precast panels on the route. A senior design engineer with ADOT has sent the following comments about the condition of these bridges:

"There is no significant cracking of the joints in the positive moment regions. Near the simply supported girder ends, there is some cracking but no leakage. None of the bridges have overlays at this time -- perhaps in the future we will install a waterproofing membrane and asphalt overlay. ADOT has found that full depth precast deck panel systems are cost competitive systems with cast-in-place (CIP) concrete decks, especially in remote areas where CIP concrete can cost as much as $2,000 (two thousands dollars) per cubic yard. Based on our long and successful experience with the precast deck systems, ADOT has decided to continue using precast concrete panels as needed."

During the past three years, ADOT used precast deck panels in two bridges, the Pedro Creek Bridge and the Kouwegok Slough Bridge. The following sections will give the details of these bridges in addition to the system used on Route FAP 65.

Dalton Highway Bridge, Route FAP 65

The structure was a multi span bridge, the longest span was 60 FT (18288 mm) and the shortest span was 30 ft (9144 mm). The superstructure of the old bridge (pier caps, girders and deck) was made of timber. Renovation of the bridge included casting concrete pier caps, installing steel girders and using precast deck panels, as shown in Figure A.2.4.1-1.

The new bridge had a total clear width of 27 ft (8230 mm) and a crown in the middle with a cross slope of 2 percent both ways. Variable thickness panels were used, where the top surface of the panels followed the road profile and bottom surface of the panels was made flat, as shown in Figure A.2.4.1-2. The panels were transversely reinforced only with ½ in. (12.7 mm) straight strands placed on two layers. In the longitudinal direction, #6 (M19) epoxy coated bars were used on two layers. The panels were provided with female shear key across their transverse edges, as shown in Figure A.2.4.1-3 and A.2.4.1-4. A ½ in (12.7 mm) gap was provided between panels to provide for production tolerance of the panels and polyethylene backer rods were used to protect grout from leaking during casting.

The precast panels were made composite with the supporting girders. 5x7 in. (127x178 mm) pockets were created in the panel to accommodate the steel studs, as shown in Figure A.2.4.1-5. The inside face of the pocket had a female shear key shape to enhance the interlocking effect between the panel and the grout. Because no post-tensioning or continuous conventional reinforcement was provided in the longitudinal direction of the deck, expansion joints were provided over piers. Also, no overlay was provided and the top surface of the panels was heavily broomed to provide the texture required for the riding surface.

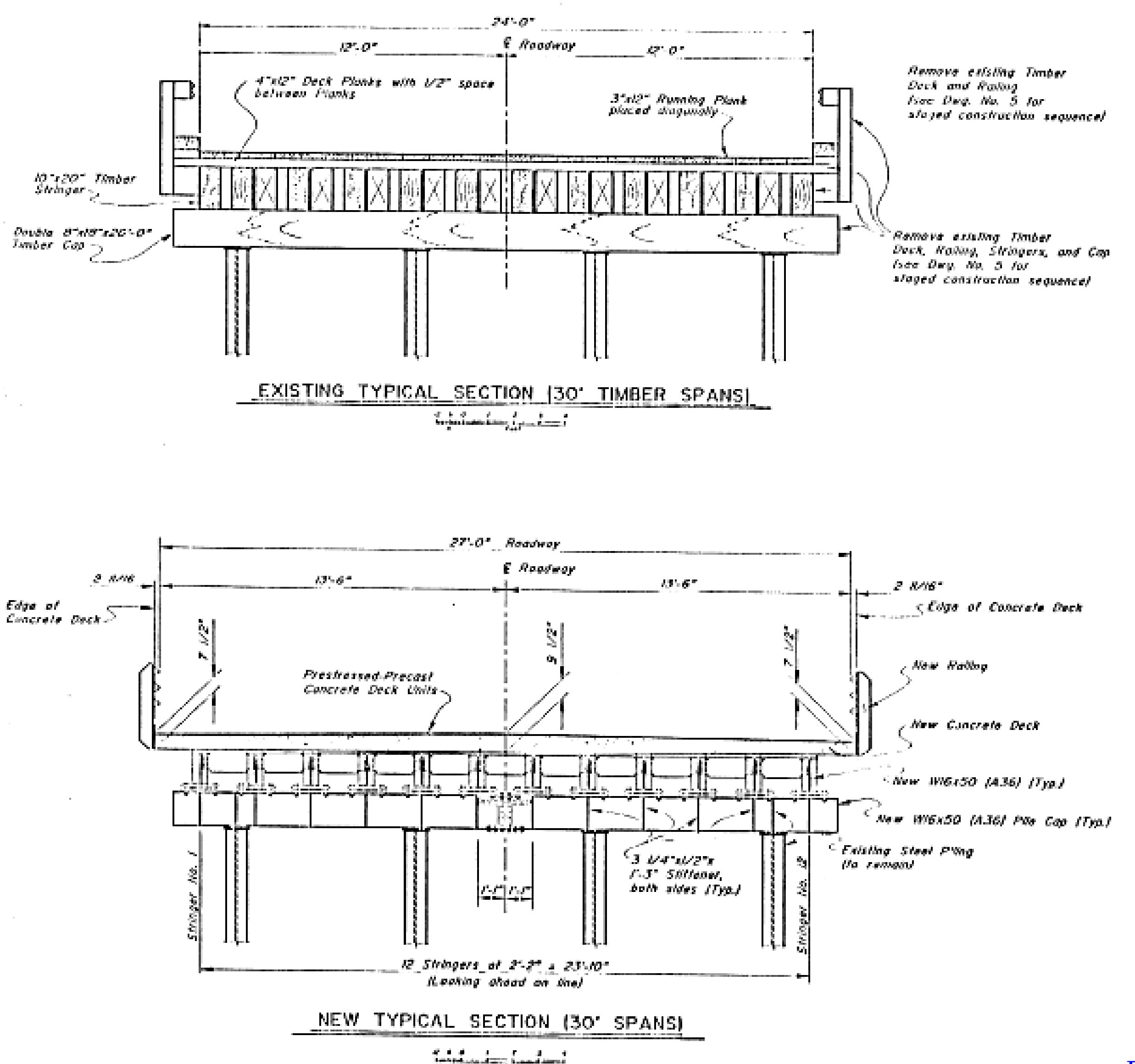


Figure A.2.4.1-1 Cross section of the old and new bridges

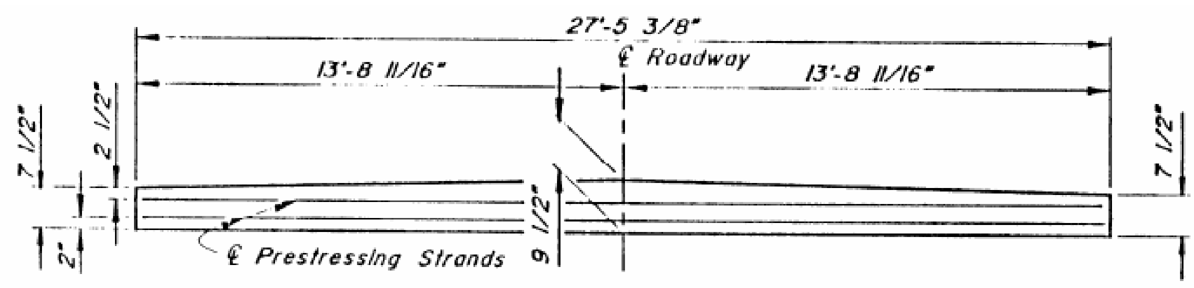


Figure A.2.4.1-2 Elevation view of the precast panel

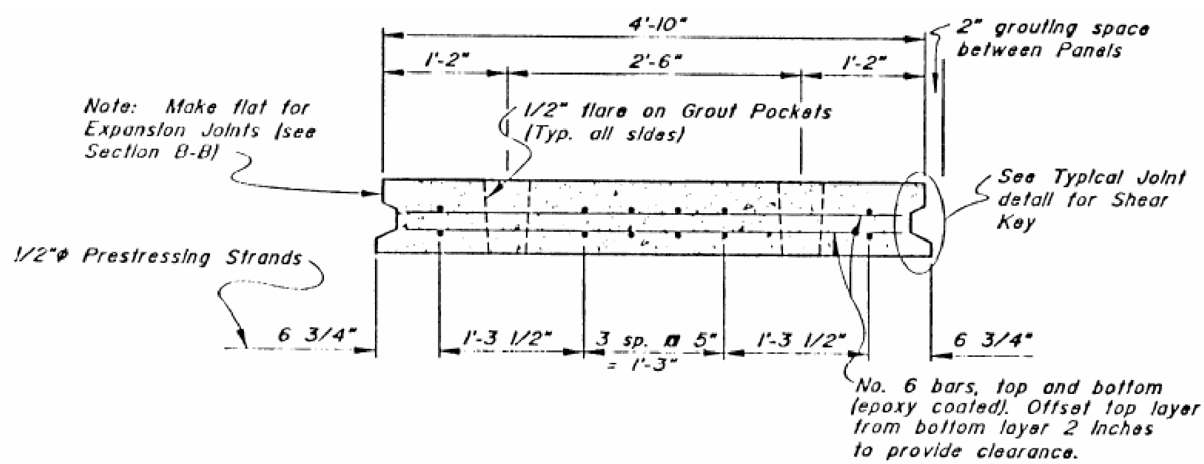


Figure A.2.4.1-3 Cross section of the precast panel

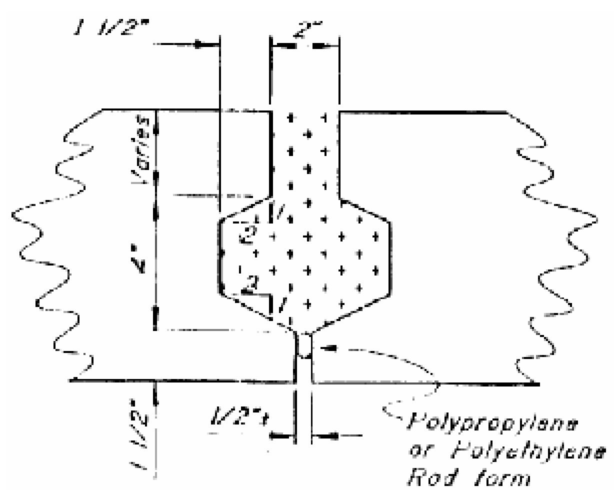


Figure A.2.4.1-4 Shear key details

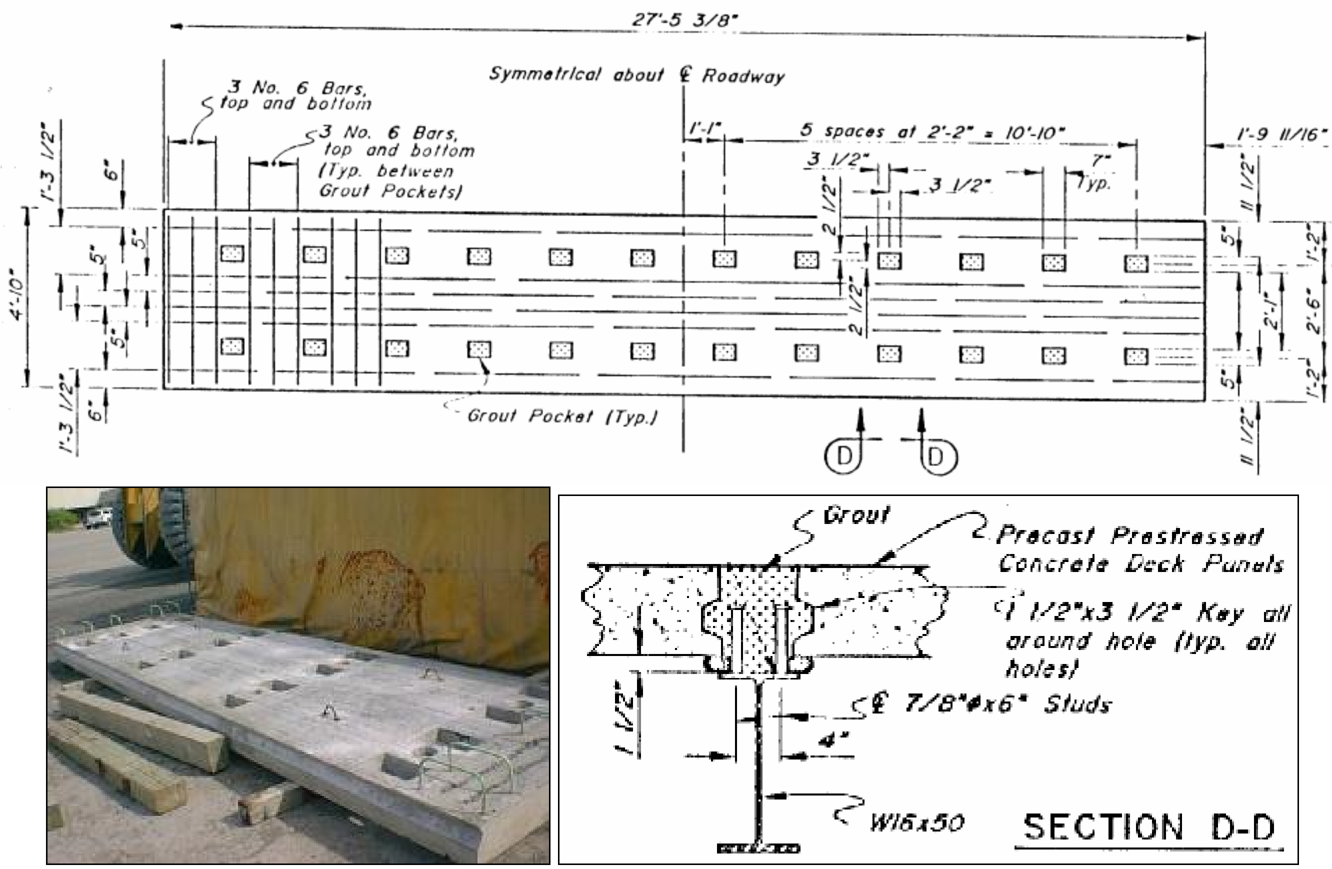


Figure A.2.4.1-5 Plan view and section d-d of the precast panel

Pedro Creek Bridge

The bridge had a 73 ft – 10 in. (22500 mm) single span on Pedro Bay Road. Total width of the bridge was 18 ft – 8 in. (5690 mm) and had one traffic lane. The superstructure consisted of four steel girders spaced at 4 ft – 11 in. (1500 mm), as shown in Figure A.2.4.1-6. 7½ in. (190 mm) thick precast panels made composite with the superstructure were used.

Figure A.2.4.1-7 shows the plan view of the precast panel. The panel was conventionally reinforced with two layers of #5 (M16) bars in the transverse direction and #4 (M13) bars in the longitudinal direction. Eight leveling screws per each panel were temporary used to support the panel. Polyethylene rods glued to the steel girder top surface were used as grout barrier as shown in Figure A.2.4.1-8. Neither longitudinal post-tensioning nor overlay was used on this bridge and the top surface of the panels was heavily broomed. A cement based non-shrink grout was used for grouting the transverse panel-to-panel joints and shear blockouts in the panels. The construction of the bridge was complete in 2001. Recent visual inspection of the deck has revealed that the transverse joints have satisfactory performance and are in very good condition, where no cracking or leaking has been observed.

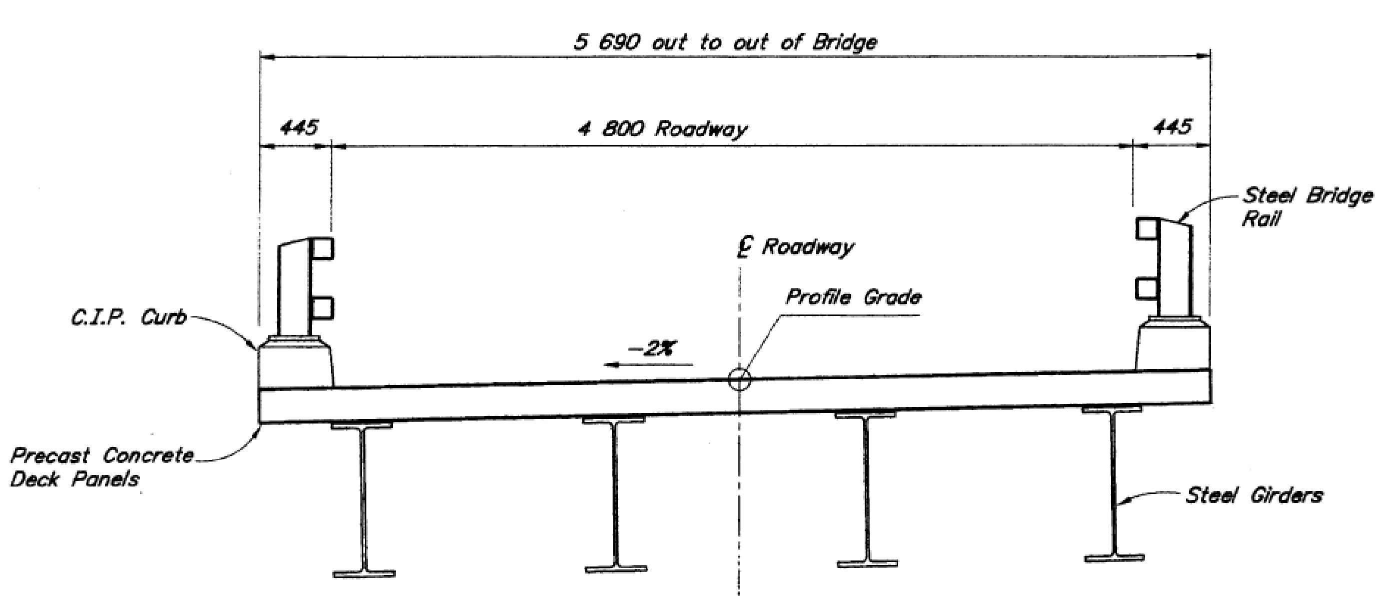
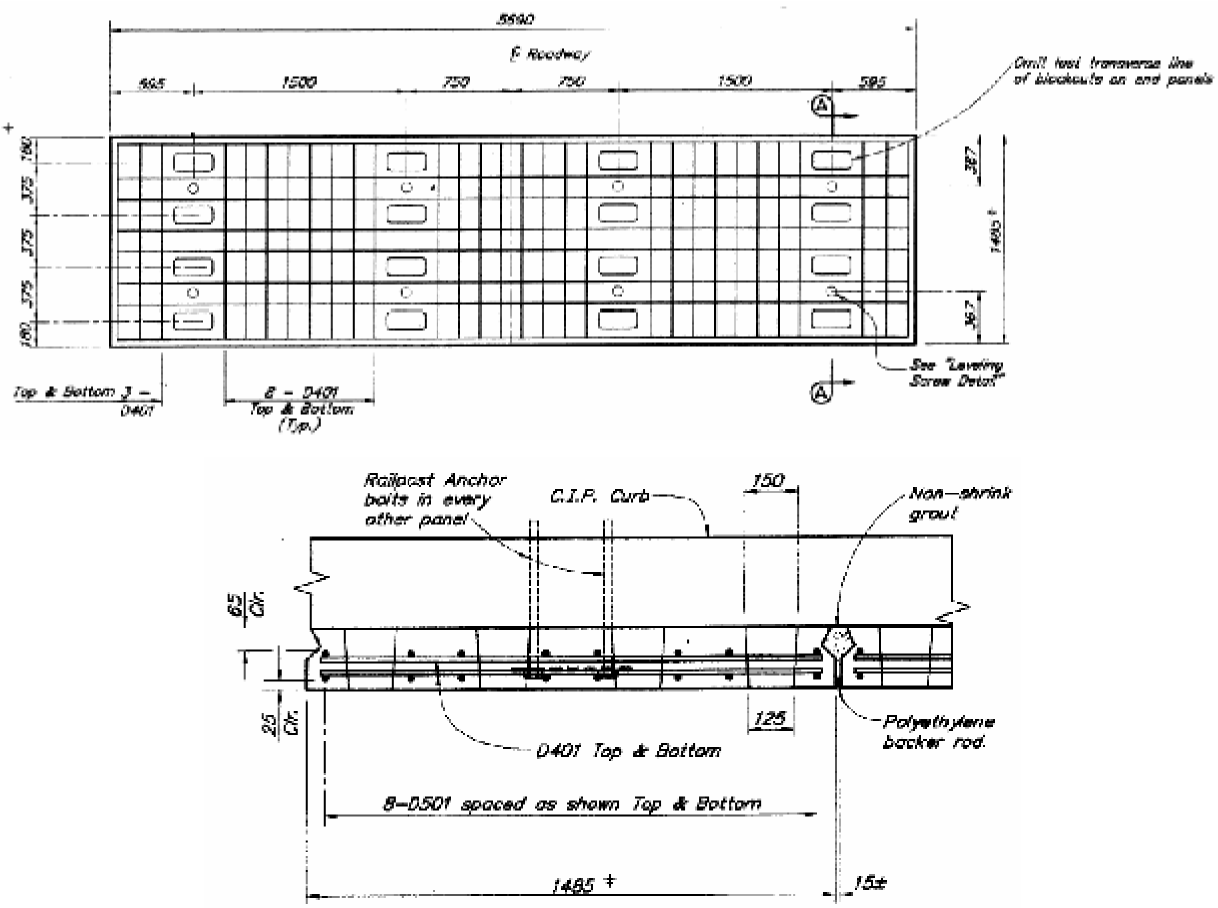


Figure A.2.4.1-6 Cross section of the bridge



SECTION A-A

Figure A.2.4.1-7 Plan view of the precast panel

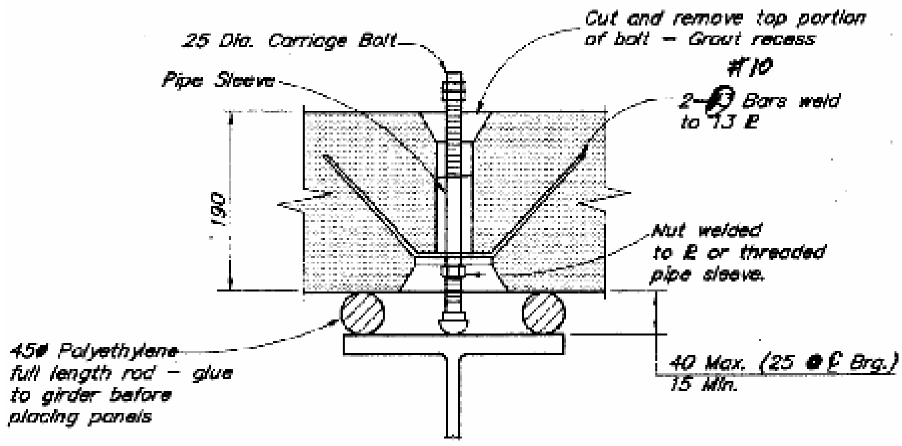


Figure A.2.4.1-8 Leveling screw detail

Kouwegok Slough Bridge

The bridge had three spans, 114, 144, and 114 ft (34800, 44000, and 34800 mm). The superstructure was made of five steel girders spaced at 5 ft – 5 in. (1650 mm) made composite with concrete deck slab. The bridge had a total width of 24 ft – 7 in. (7500 mm) with a crown in the middle, as shown in Figure A.2.4.1-9.

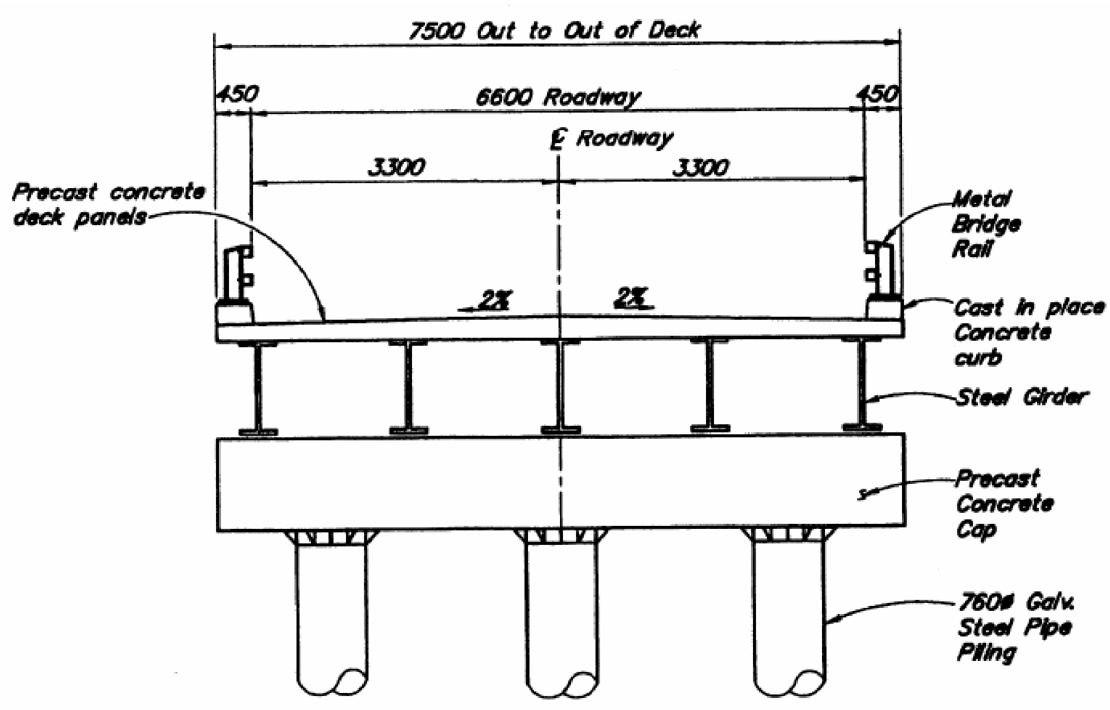


Figure A.2.4.1-9 Cross section of the Kouwegok Slough Bridge

A full depth precast panel system, similar to that used in Pedro Creek Bridge, was used on this bridge. Figure A.2.4.1-10 shows a plan view of the precast panel. The panel had a variable depth, 9.8 in. (250 mm) at the crown and 6.9 in. (175 mm) at both ends. The panel was transversely and longitudinally conventionally reinforced with two layers of epoxy coated reinforcing bars in each direction. Neither longitudinal post-tensioning nor overlay was used. As a result, an expansion joint was created at every pier. The precast panels were made composite with the steel girders by using grouted shear pockets that accommodated the shear connectors. A female-to-female shear key detail is used, as shown in Figure A.2.4.1-11.

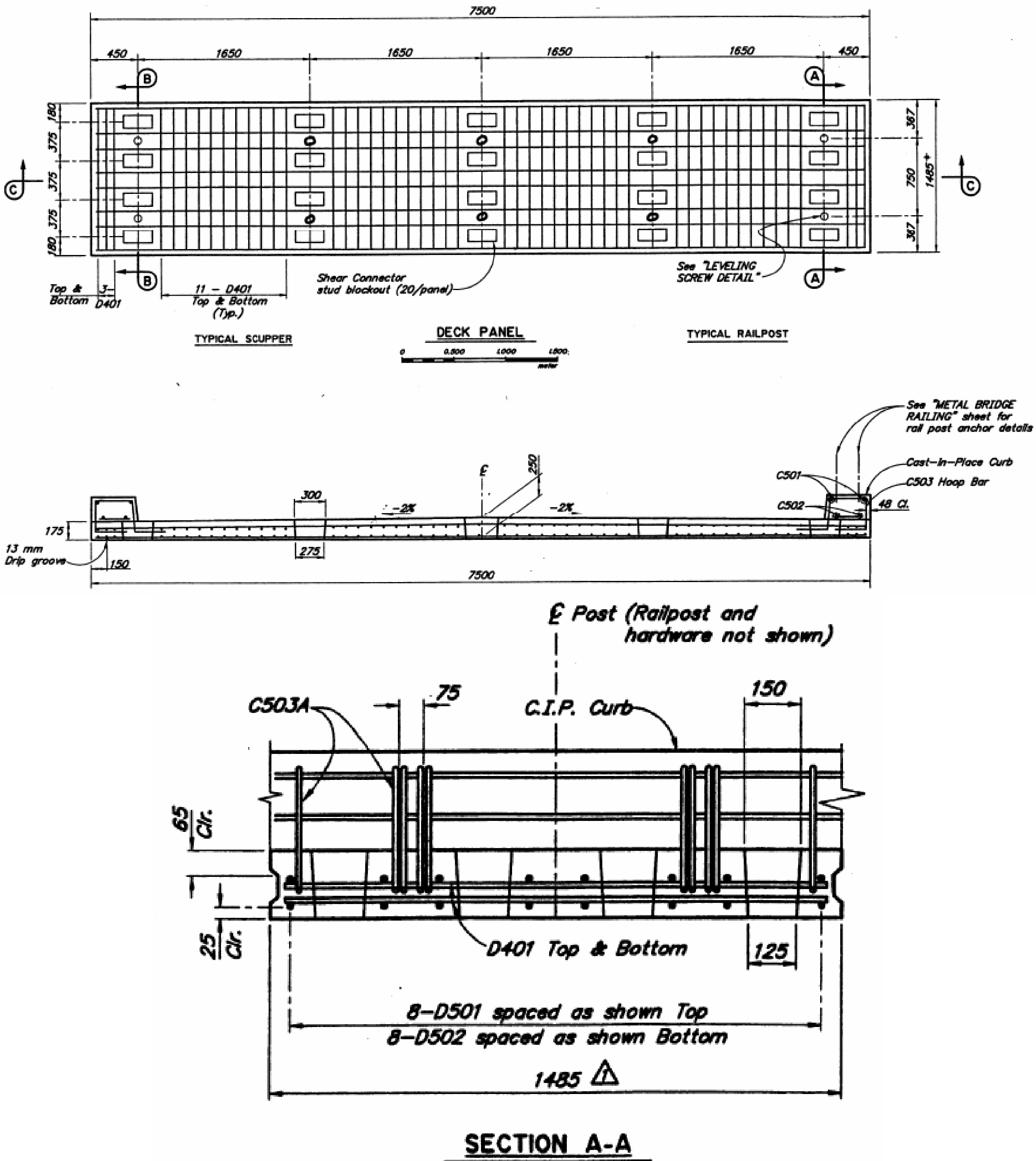


Figure A.2.4.1-10 Plane view of the precast panel

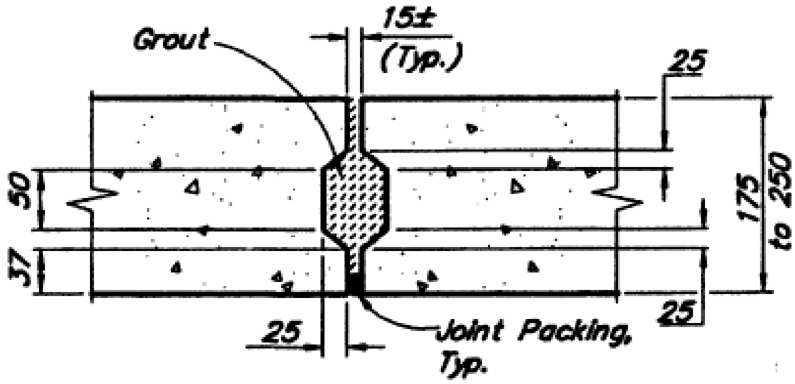


Figure A.2.4.1-11 Shear key details

A.2.4.2 Colorado Department of Transportation

Colorado Department of Transportation used a full depth precast panel system for the deck replacement and widening project of the Castlewood Canyon Bridge. The bridge had three spans and each span was made of two separate reinforced concrete arches spaced at 20 ft (6095 mm) in the transverse direction. The arches supported vertical posts with cross piers. The full depth precast panels extended in the direction of traffic between the cross piers, as shown in Figure A.2.4.2-1 & A.2.4.2-2.

Figure A.2.4.2-1 The superstructure system of the Castlewood Canyon Bridge

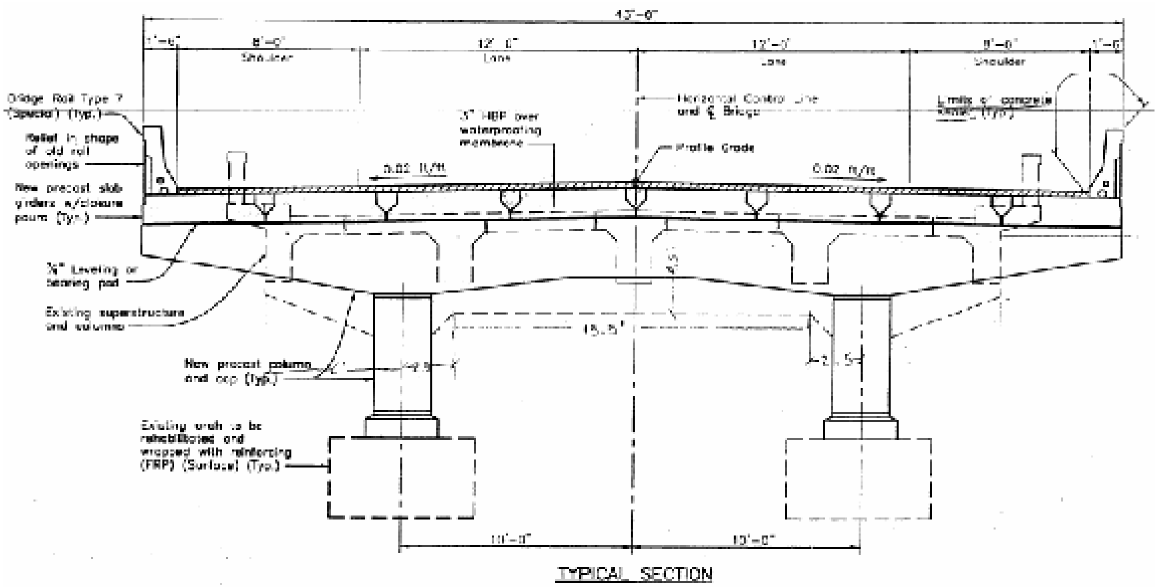


Figure A.2.4.2-2 Cross section of the Castlewood Canyon Bridge

The length of the precast panels ranged from 16 ft – 4 in. (4977 mm) at the peak of the arch to 38 ft – 4 in. (11683 mm) at the ends of the arch. The width of the panels was 5 ft – 4in. (1625 mm), as shown in Figure A.2.4.2-3.

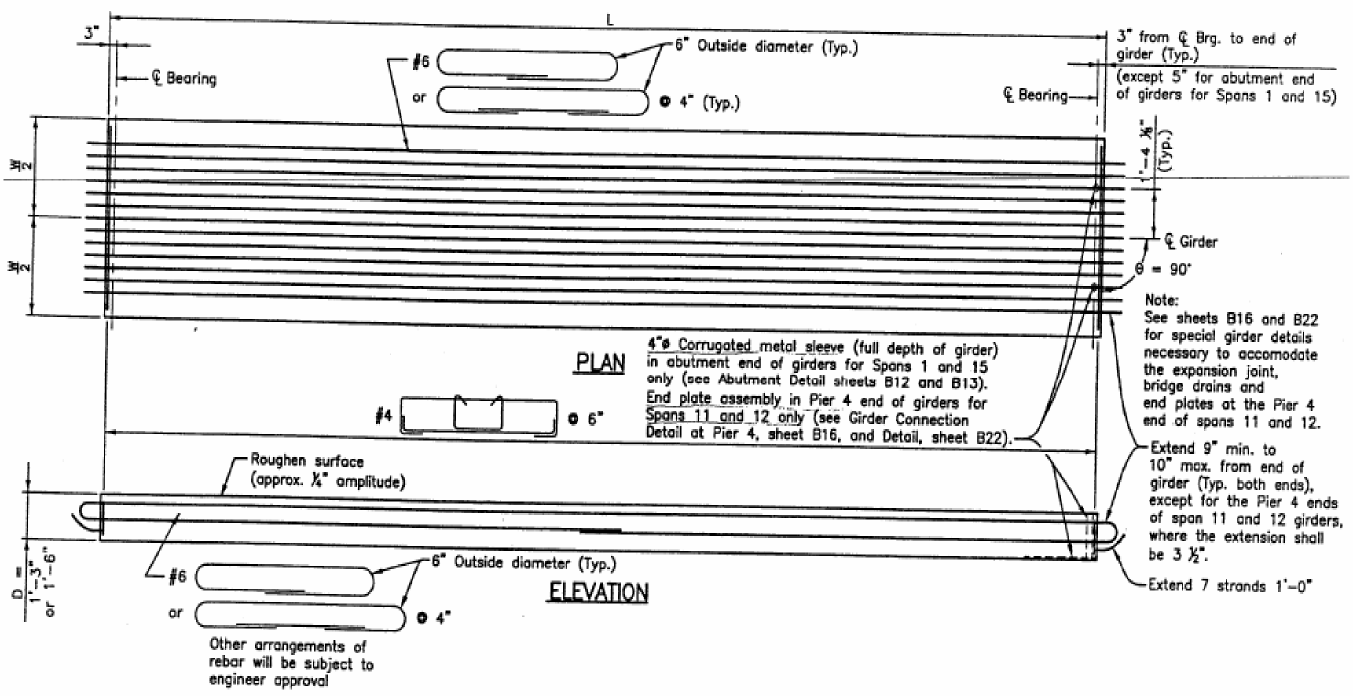


Figure A.2.4.2-3 Plan view and elevation of the precast panel

Eight panels were used to form the full width of the bridge. A 3 in. (76 mm) thick asphalt overlay with a waterproofing membrane was cast on the panels to protect the precast panels and provide for a smooth riding surface.

The panels were pretensioned in the longitudinal direction with 29- ½ in. (12.7 mm), 270 ksi (1.86 GPa) strands. Seven strands were extended 12 in. (304.8 mm) outside the panel at both ends and the rest of the strands were cut flush with the panel ends. In addition to the prestressed strands, the panel was reinforced with #6 (M19) bars at 4 in. (100 mm). These bars extended about 9 in. (228.6 mm) outside the panel at both ends. Normal strength concrete mix was used for the panel. Concrete strength at release and 28 days were 4 ksi (27.6 MPa) 4.5 ksi (31.0 MPa) respectively. The panels were made continuous in the longitudinal direction over the cross piers by overlapping the #6 bars. 2- #6 (M19) transverse bars were installed in the overlapped core and confined with #4 (M13) closed stirrups spaced at 12 in. (304.8 mm), as shown in Figure A.2.4.2-4.

Figure A.2.4.2-5 shows the cross section of an exterior and interior panels. Longitudinal shear keys were formed along the edges of the panel, as shown in Figure A.2.4.2-4. To connect the panels in the longitudinal direction, two #8 (M25) bars were installed in the shear key close to the top surface and confined with #4 (M13) bars spaced at 12 in. (304.8 mm).

Cast-in-place (CIP) concrete was used for the side barrier. The exterior panels were provided with shear connectors to connect the side barrier with the deck. Also the interface surface between the panel and the side barrier was roughened to ¼ in. (6 mm) amplitude to enhance the bond between the CIP concrete barrier and the deck.

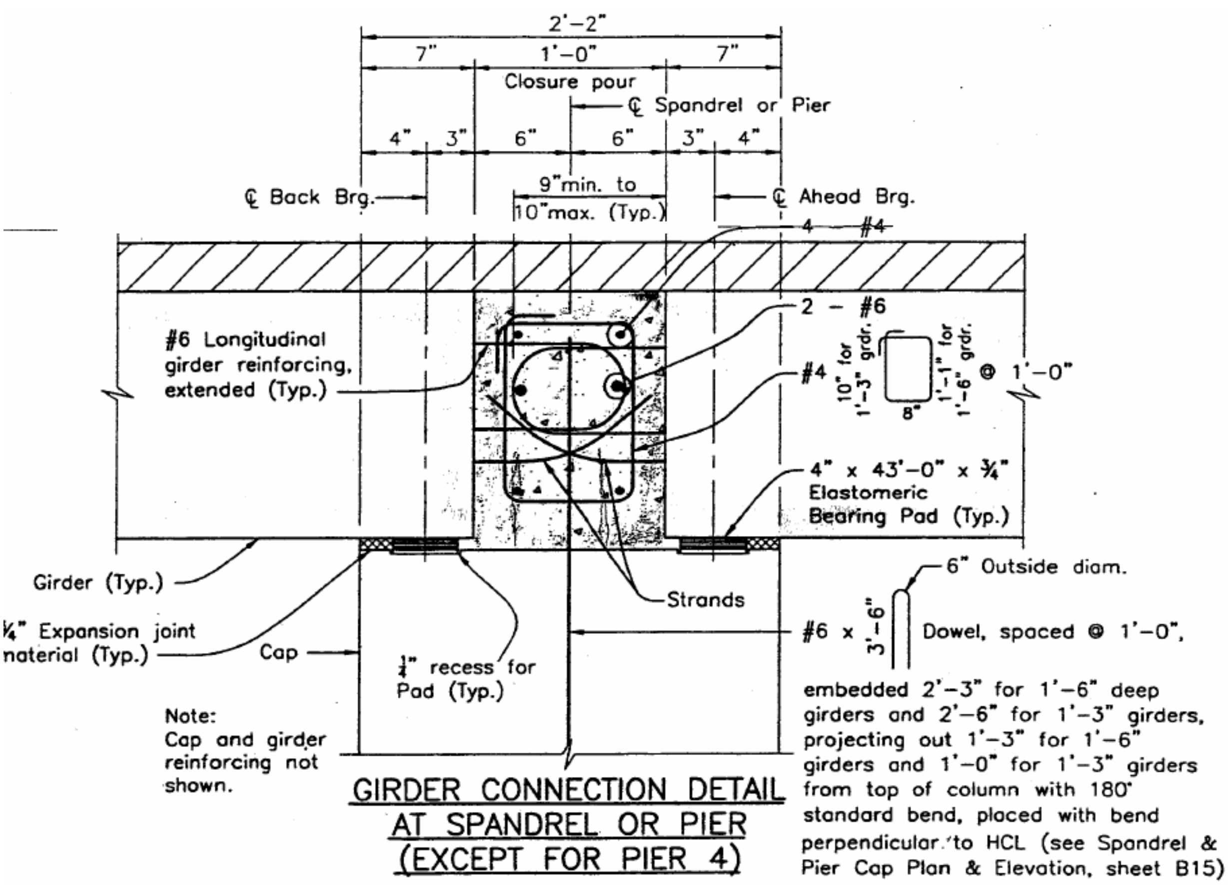


Figure A.2.4.2-4 Continuity detail over the cross piers

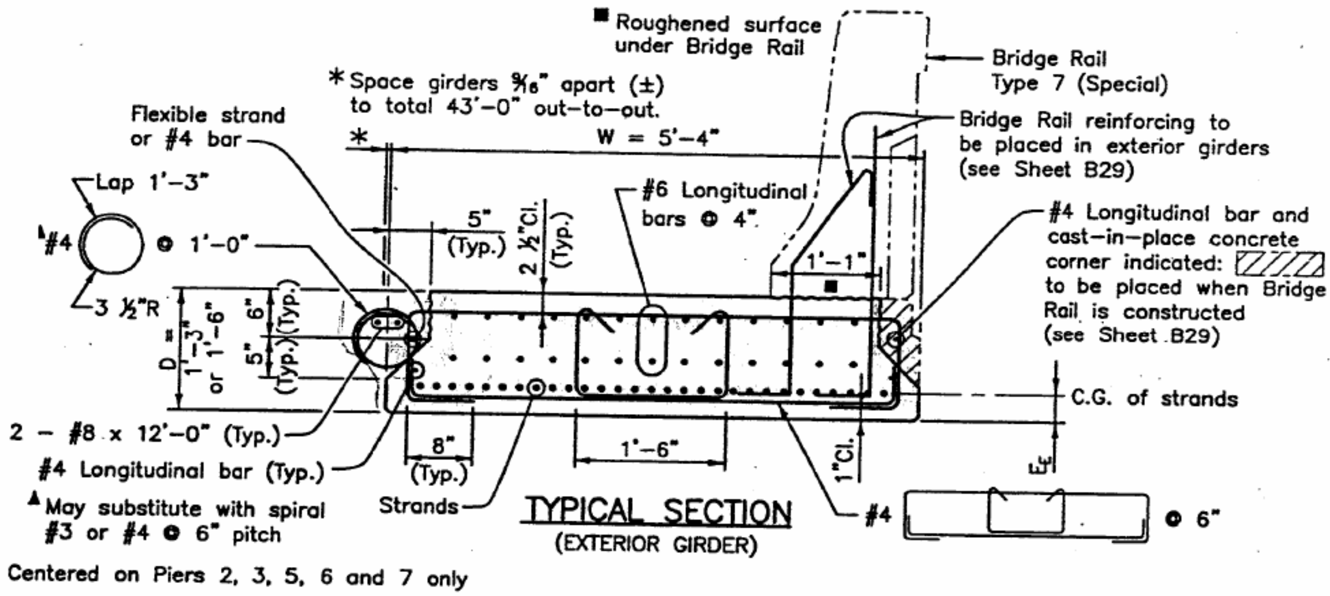


Figure A.2.4.2-5a Typical cross section of the exterior precast panel

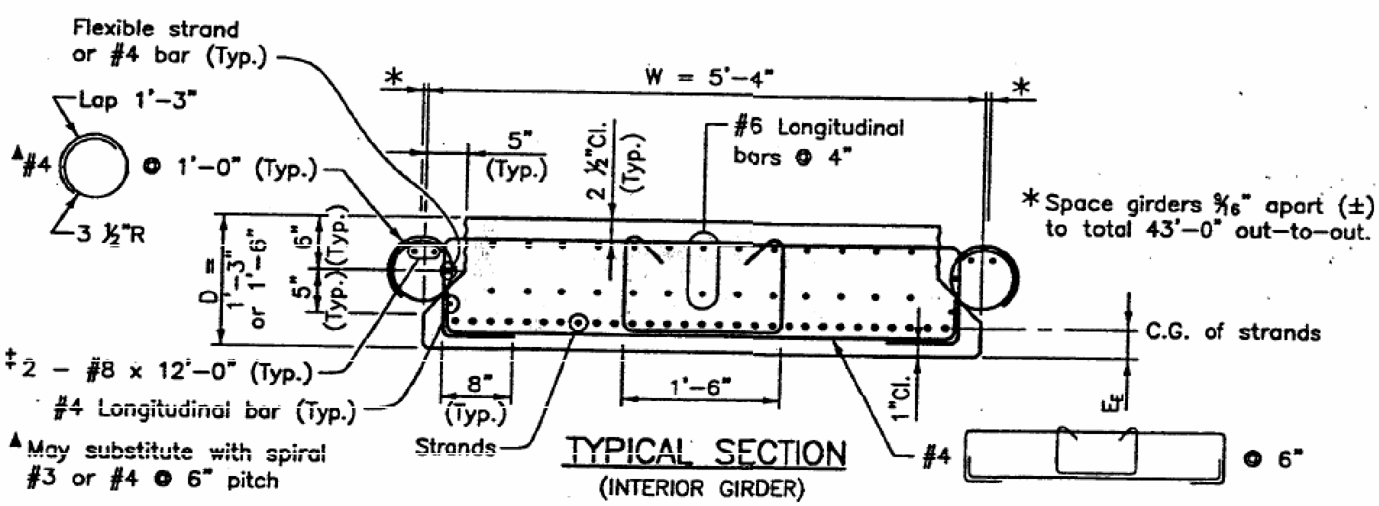


Figure A.2.4.2-5b Typical cross section of the interior the precast panel

A.2.4.3 District of Columbia, DC

Rehabilitation of Bridges over Dead Run and Turkey Run, George Washington Memorial Parkway, National Park Service

In 1995, the National Park Service Authority decided to replace the deck slab of two structures on George Washington Memorial Parkway. These are the Dead Run and the Turkey Run structures, as shown in Figure A.2.4.3-1.

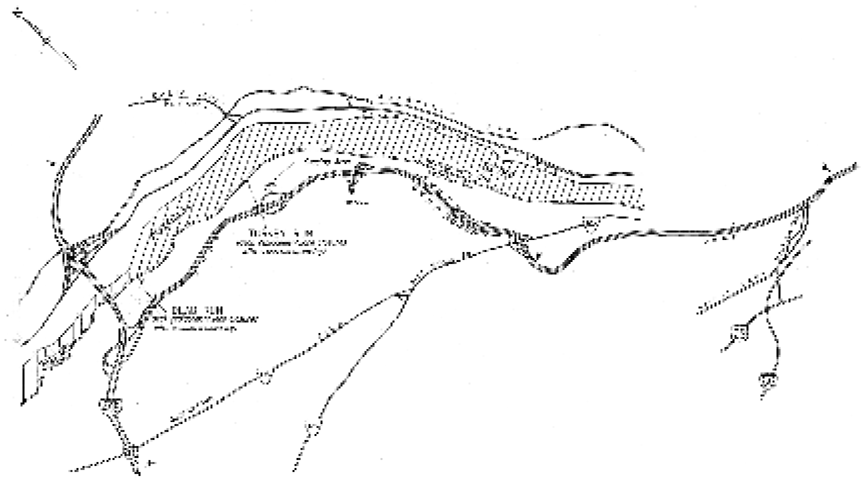


Figure A.2.4.3-1 Location of the Dead Run and Turkey Run Structures

The Dead Run Structure

The Dead Run Structure had two bridges, namely the northbound and southbound bridges. Both bridges had a slight curve in plan and 3.37% longitudinal slope, as shown in Figure A.2.4.3-2. The old deck was a 7.9 in. (200 mm) thick cast-in-place concrete deck made non-composite with the supporting girders. The northbound bridge had a 2.6 in. (65 mm) thick asphalt overlay with waterproofing membrane, while the southbound bridge had no overlay. Visual inspection of both bridges showed that the deck slab of the southbound bridge had significant concrete spalling and corrosion of the deck reinforcement, while the deck slab of the northbound bridge showed no signs of deterioration. As a result, the National Park Service called for full replacement of the deck on the southbound bridge and replacement of the asphalt overlay with a 1.2 in. (30mm) thick latex modified concrete overlay on the northbound bridge.

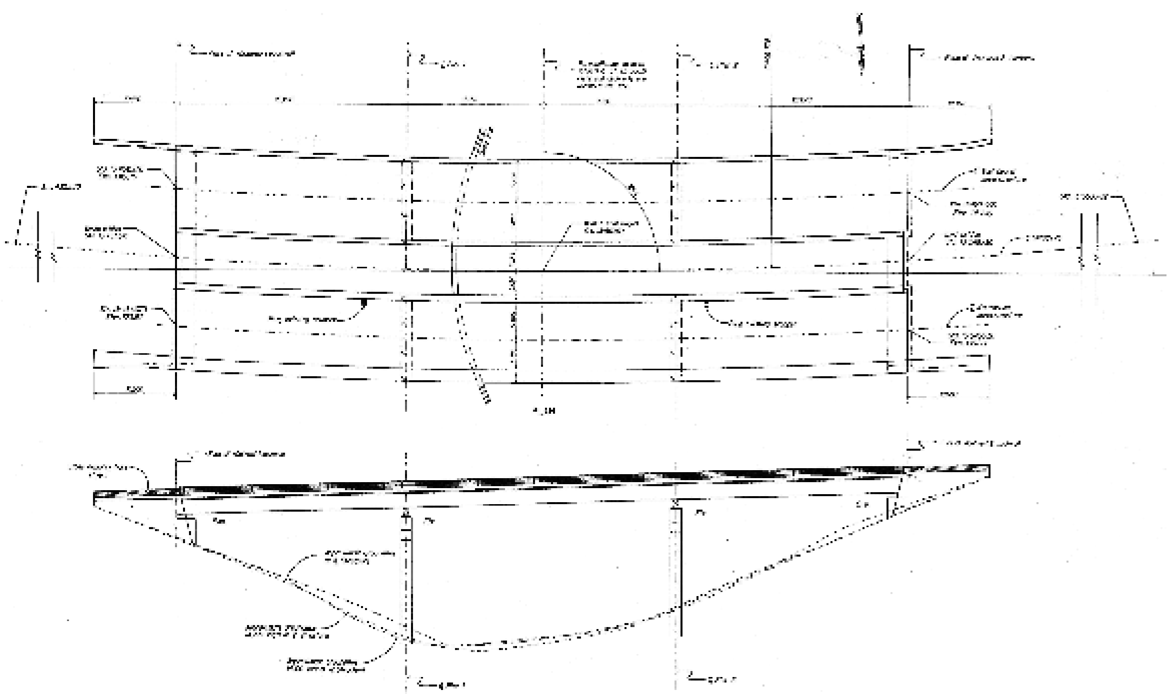


Figure A.2.4.3-2 Plan and elevation views of Dead Run and Turkey Run Structures

The southbound bridge had three spans, 95, 113 and 95 ft (29.24, 34.5, and 29.24 m) respectively. The superstructure was made of two exterior steel girders and two interior steel stringers. The interior steel stringers were supported by cross beams, which in turn were supported by the exterior steel girders. This arrangement resulted in a deck slab supported on four girders spaced at 9 ft (2.74 m) and two overhangs, 5 ft – 3 in. (1.6 m) each, as shown in Figure A.2.4.3-3. The old cast-in-place deck slab had a non-prismatic profile with haunches at the exterior girders.

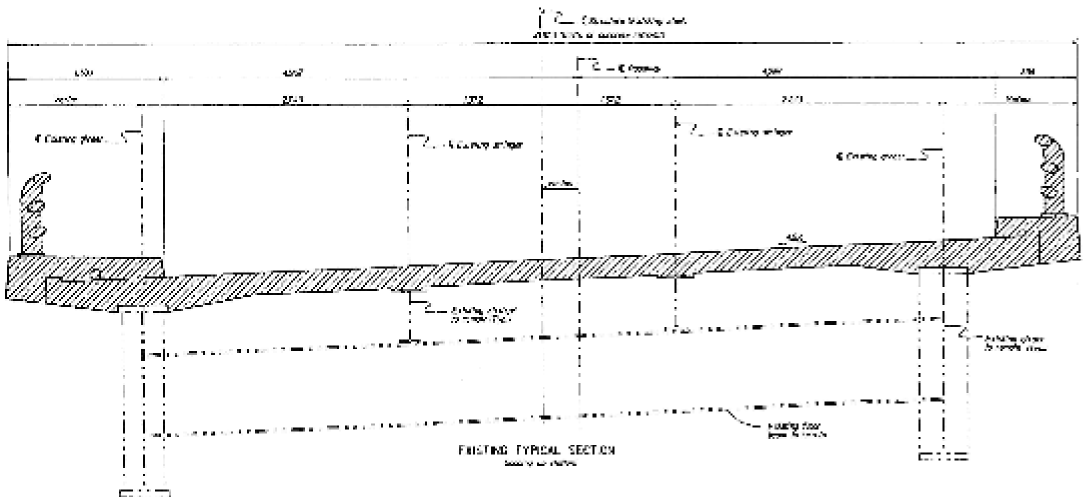


Figure A.2.4.3-3 Cross Section of the bridge with the old deck

Full width, 7.9 inch (200 mm) thick, prismatic precast concrete panels were used to replace the old deck. The precast panels were transversely pretensioned in the precast yard and longitudinally pot-tensioned after being installed on the bridge. Figure A.2.4.3-4 shows the cross section of the bridge with the new precast deck. A 1.2 in. (30 mm) latex modified concrete overlay was used to protect the precast panels and provide the texture required for the riding surface.

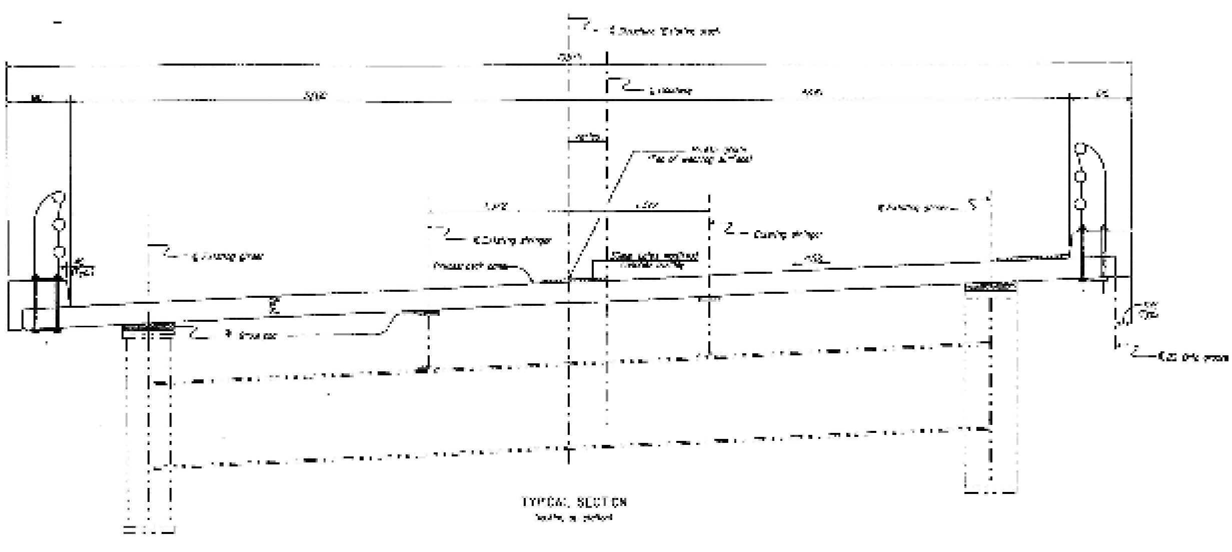


Figure A.2.4.3-4 Cross section of the bridge with the new deck

Each panel was reinforced with 30– ½ in. (12.7 mm) diameter, 270 ksi (1.86 GPa) strands on two layers, as shown in Figure A.2.4.3-5 and A.2.4.3-6. Conventional reinforcement was provided in the transverse and longitudinal directions for shrinkage and temperature effects. Specified concrete strength was 4.0 and 5.5 ksi (28 MPa and 38 MPa) at transfer and prior to stressing longitudinal post-tensioning, respectively.

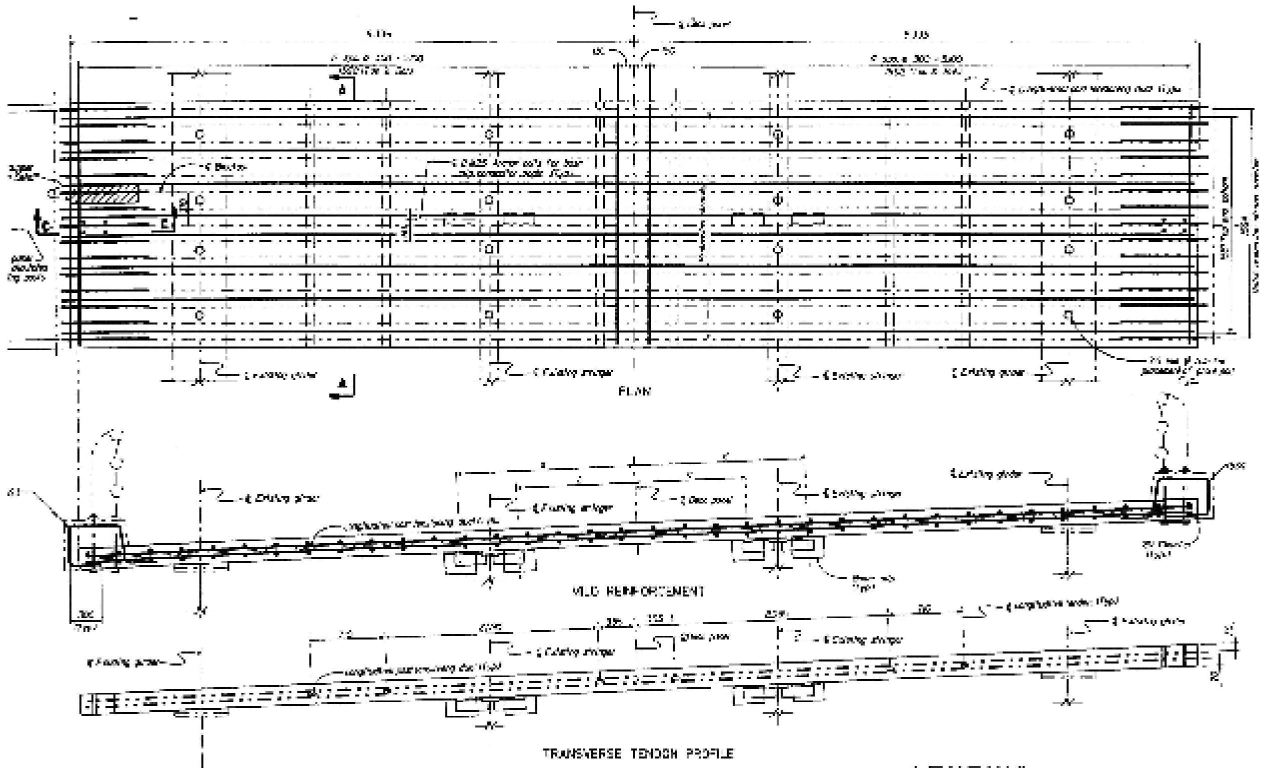


Figure A.2.4.3-5 Details of a typical precast panel

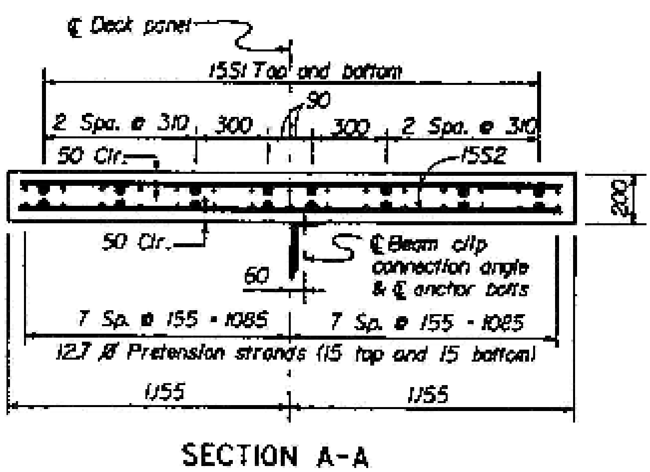


Figure A.2.4.3-6 Typical cross section of the precast panels

Four, 3-in. (75 mm) diameter, holes were provided in each panel over each girder line for placement of the grout pad. In order to protect the panels from moving upward during post-tensioning, each panel was clipped to the interior stringers using two 8x4x½ in. (203x102x12.7 mm) angles bolted to the bottom surface of the panel and two 1x2 in. (25x50 mm) clip plates welded to each steel stringer, as shown in Figure A.2.4.3-7. The designer gave the contractor the option of bolting or welding the angles and the clip plates. The elevation of each panel was adjusted using three leveling eyebolts, two bolts on the north side exterior girder and one bolt on the south side exterior girder. Four, 1 in. (26 mm) diameter, coil inserts were provided in the precast panels for that reason.

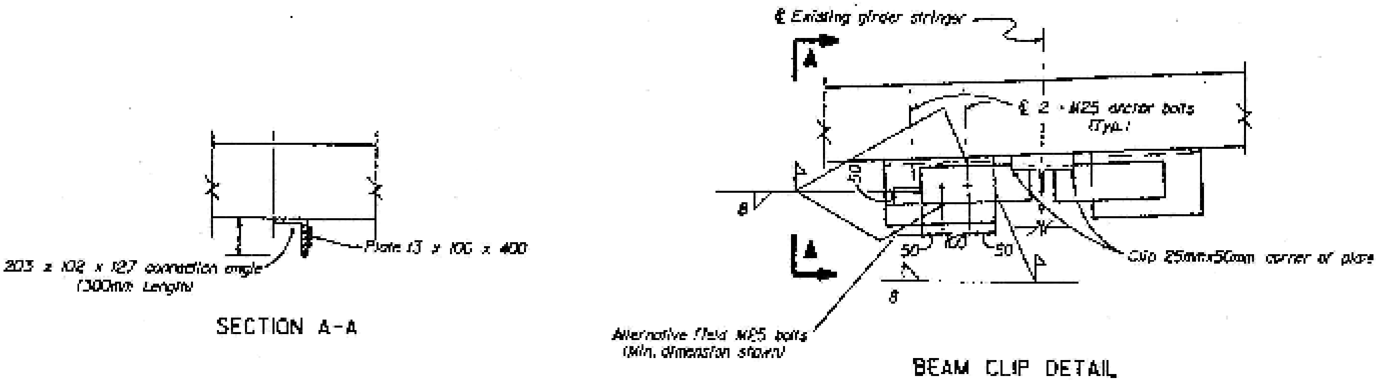


Figure A.2.4.3-7 Typical clip detail

In order to simplify the production of the panels, the following issues were considered by the designer:

1. Although the bridge had a mild curvature in plan, rectangular precast panels 35 ft x 7 ft-7 in. (10670 x 2310 mm) were used. This resulted in a trapezoidal transverse joint between adjacent panels, where the gap between panels was 2.6 in. (66 mm) and 1.2 in. (30 mm) on the outside and inside edges of the curvature respectively, as shown in Figure A.2.4.3-8.

2. In order to use the same type of side railing that was used on the old deck, the length of the panels in the longitudinal direction of the bridge was adjusted to 7 ft-7 in. (2310 mm). This resulted in one connection per panel at mid length of the panel. Four, 1.5 in. (38 mm) diameter, holes were provided in the precast panel to connect the railing posts, as shown in Figure A.2.4.3-9.

3. Because prismatic precast panels were used, a 2.4 to 3.9 in. (60 to 100 mm) and 1.2 in. (30 mm) thick grout pads were used on the exterior girders and interior stringers respectively.

4. Cast-in-place concrete was used to form the curb rather than casting it in the precast yard. The shear connectors between the precast panel and the cast-in-place concrete curb were provided by using #5 pins anchored in the panel and extended outside the top surface of the precast panel.

A female type shear key was created along the transverse edges of each panel, as shown in Figure A.2.4.3-10. Wood forming was used to support the grout at the transverse panel-to-panel joints. The design specifications called for the use of a non-shrink grout (with zero volume change) with a minimum concrete strength of 2,000 psi (14 MPa) at 24 hours.

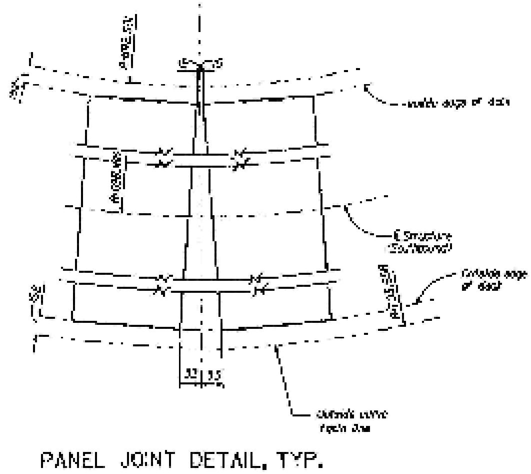


Figure A.2.4.3-8 Panel-panel joint detail

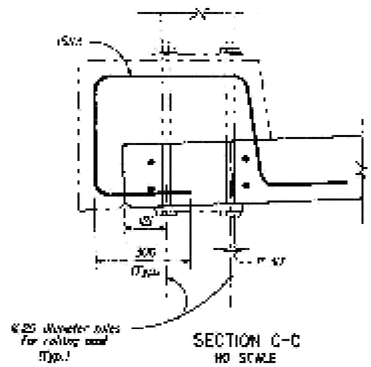


Figure A.2.4.3-9 Railing connection detail

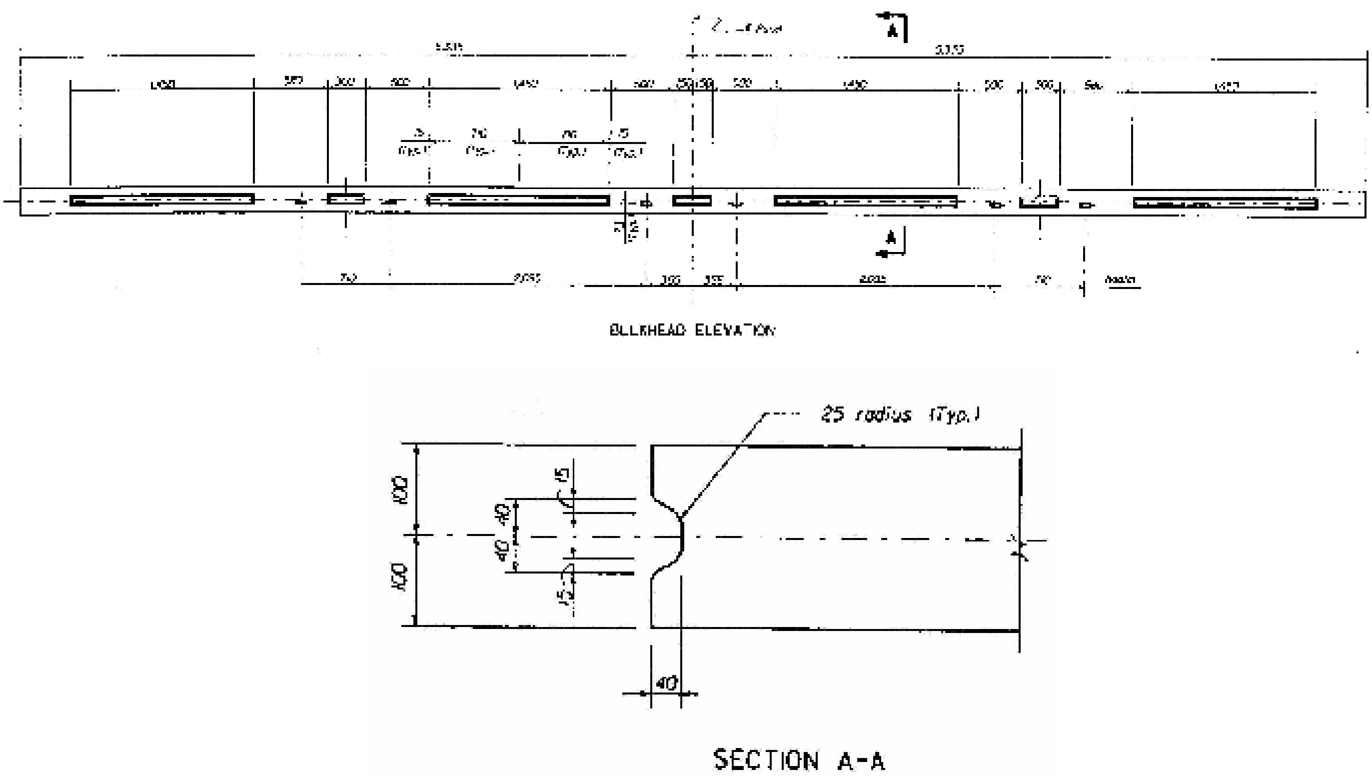


Figure A.2.4.3-10 Shear key detail

Longitudinal post-tensioning was provided by six tendons spread along the full width of the bridge. Two tendons spaced at 28 in. (710 mm) were provided in the center of each span between the four supporting girders, as shown in Figure A.2.4.3-5. Each tendon had 4- ½ in. (12.7 mm), 270 ksi (1.86 GPa) strands. Flat ducts, 1x3 in. (25x75 mm), were provided in the precast panels to house the tendons. The ducts were grouted after the tendons were post-tensioned.

Due to the high traffic volume, the contractor was allowed to close the bridge for traffic only during weekends (from Friday evening to Monday morning). Staged construction scenario was called for the deck replacement project, where the contractor had to remove the old deck and construct the new precast deck of one full span in one weekend. Figure A.2.4.3-11 shows the construction stages. The precast deck installation started on the west side abutment and proceeded towards the east side abutment.

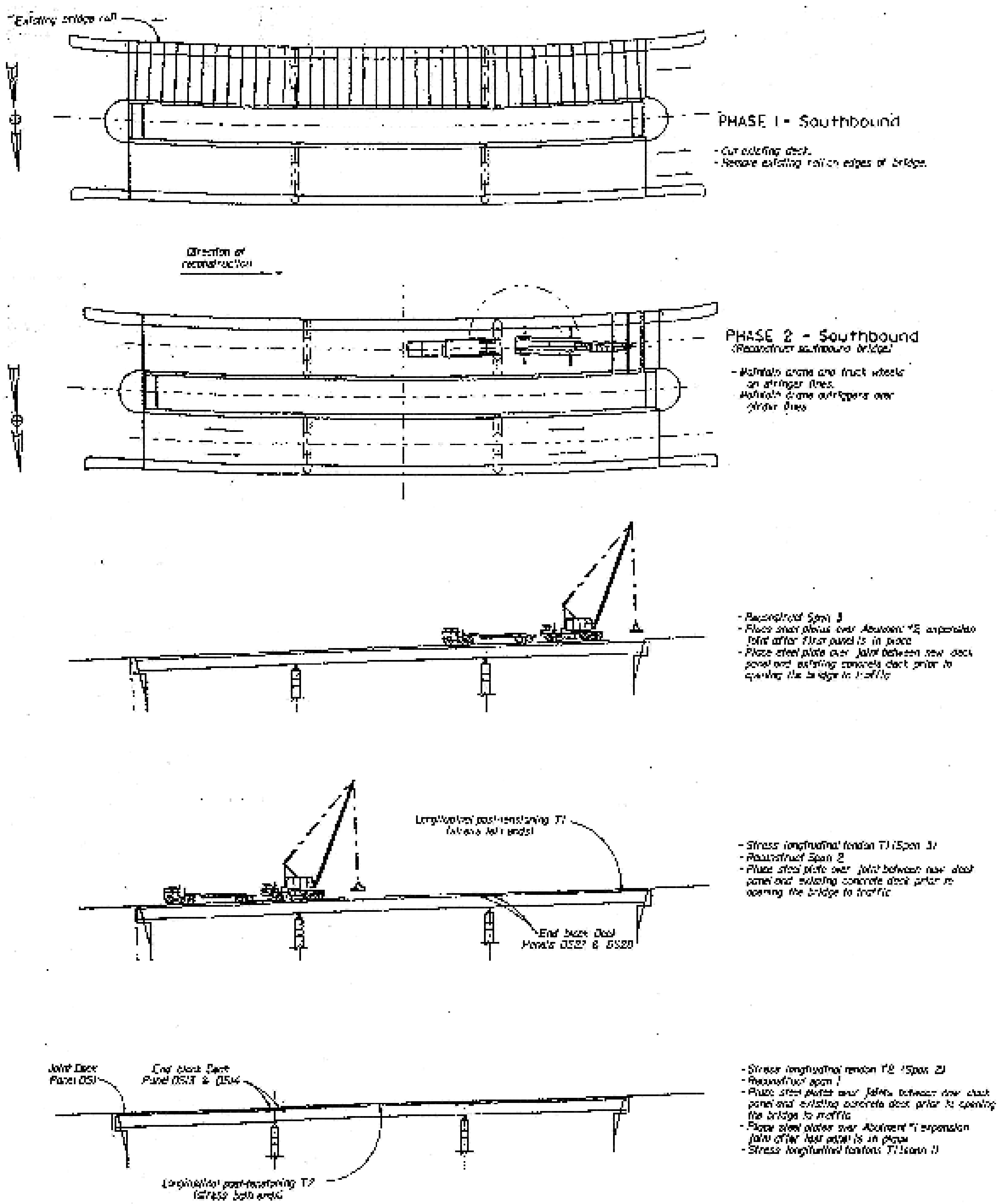


Figure A.2.4.3-11 Construction stages

As a result of the staged construction scenario, three different types of precast panels were produced: (1) typical panel as shown in Figure A.2.4.3-5 (34 panels), (2) end block panel for staged post-tensioning at piers (4 panels), and (3) end deck panels by the abutments (2 panels). Figure A.2.4.3-12 shows the arrangement of various panels.

Figure A.2.4.3-13 and A.2.4.3-14 show the details of the end block panel as provided by the designer. In order to accommodate the anchorage device, the post-tensioning tendons were diverted downward and accommodated in a haunch, where the thickness of the precast panel was raised from 7.9 in. (200 mm) to 19 in. (480 mm) as shown in section A-A in Figure A.2.4.3-14.

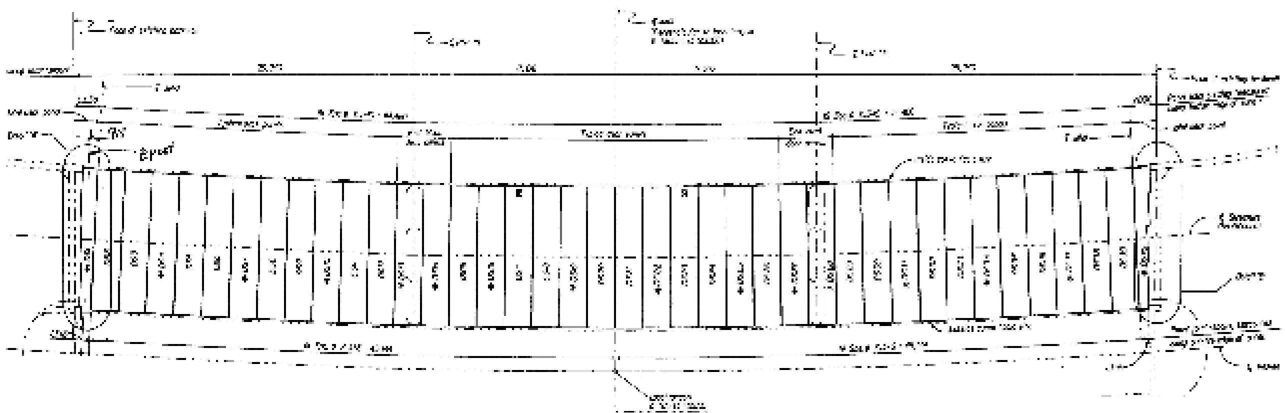


Figure A.2.4.3-12 Arrangements of the panels

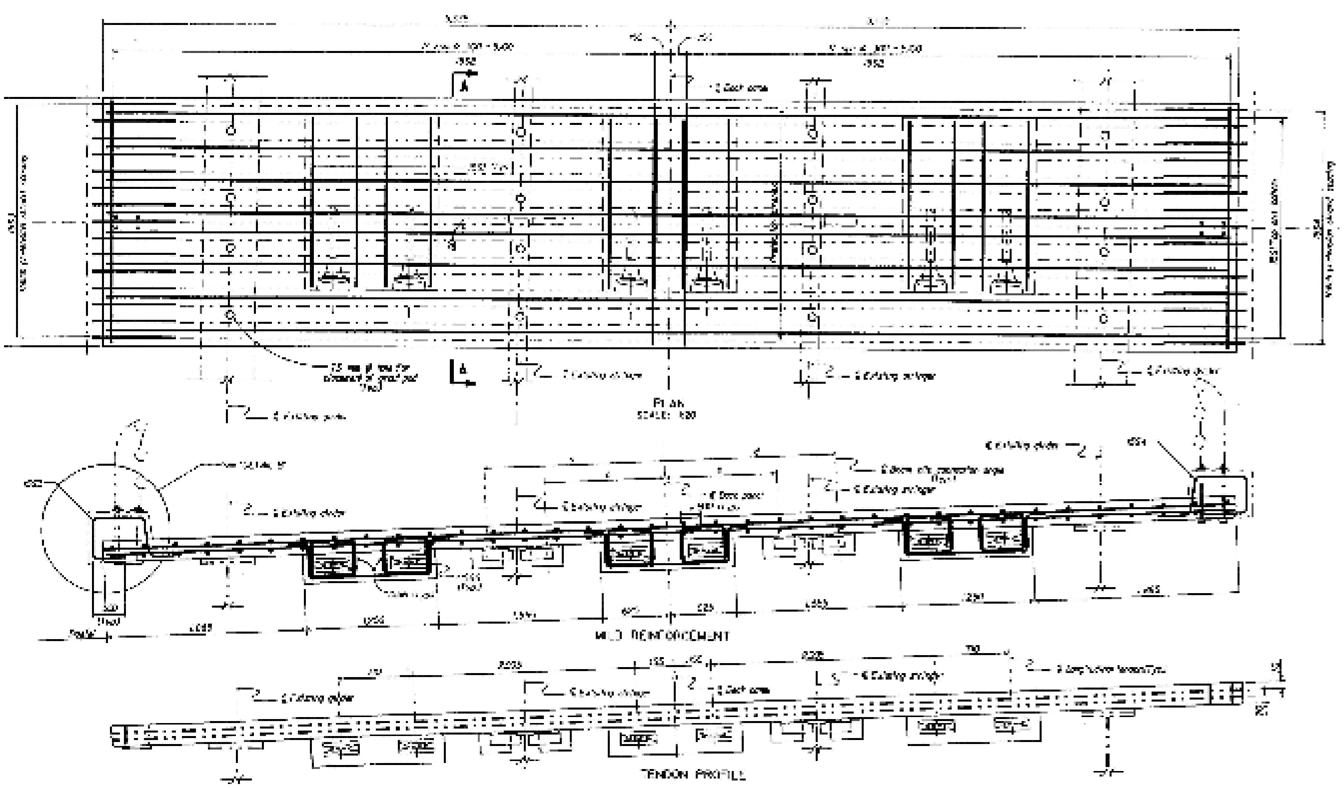


Figure A.2.4.3-13 Details of the end block panel as provided by the designer

After wining the bid, the contractor consulted with the precast concrete producer and then called for changing the details of the end block panels. Instead of diverting the post-tensioning tendons downward and raising the panel thickness, a prismatic panel was used and the post-tensioning tendons were staggered in plan as shown in Figure A.2.4.3-15. This modification significantly simplified the production of the panels.

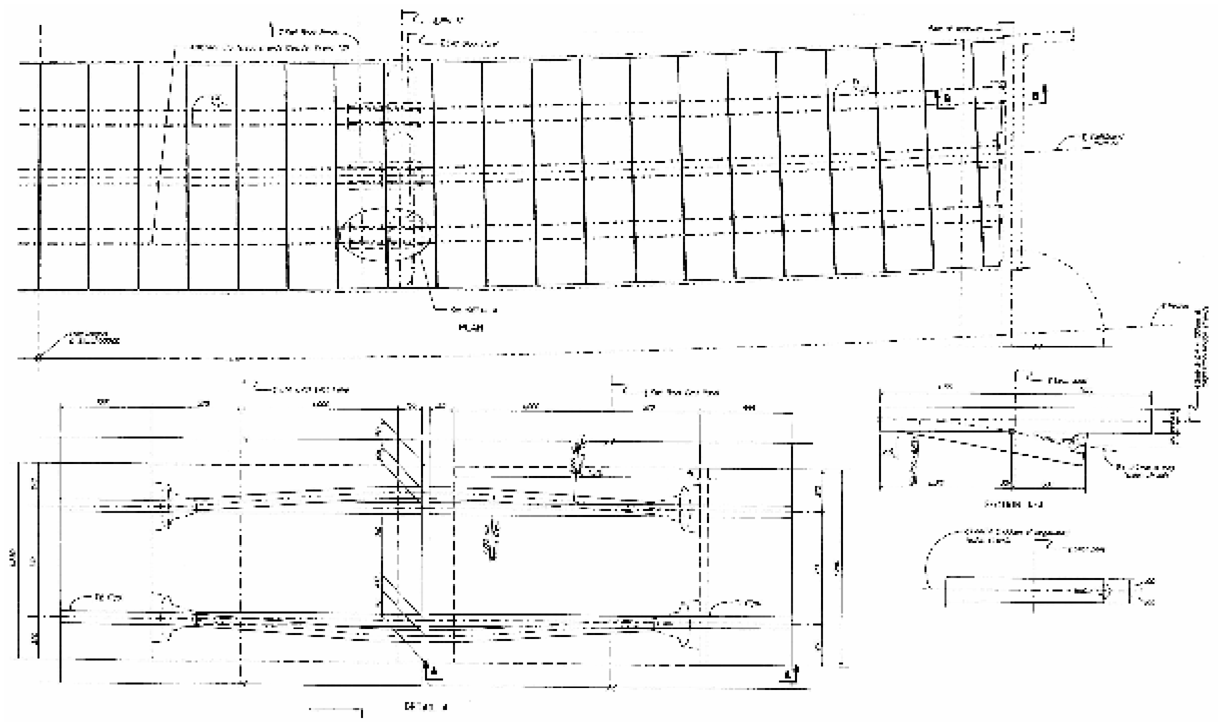


Figure A.2.4.3-14 Details of the staged post-tensioning scenario as provided by the designer

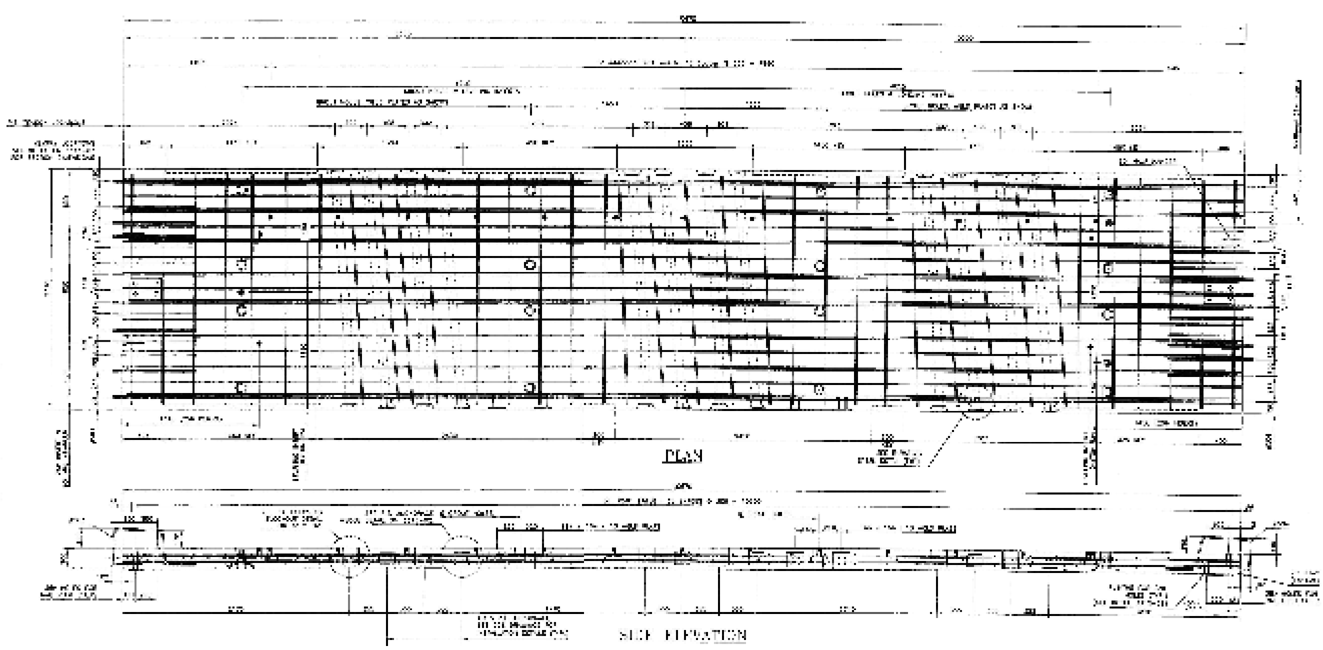


Figure A.2.4.3-15 Details of the end block panel as modified by the contractor

Figure A.2.4.3-16 shows the details of the end deck panels. They were transversely pretensioned with nine top and nine bottom ½ in. (12.7 mm) strands. The end panels were prismatic and accommodate the anchorage devices of the post-tensioning tendons.

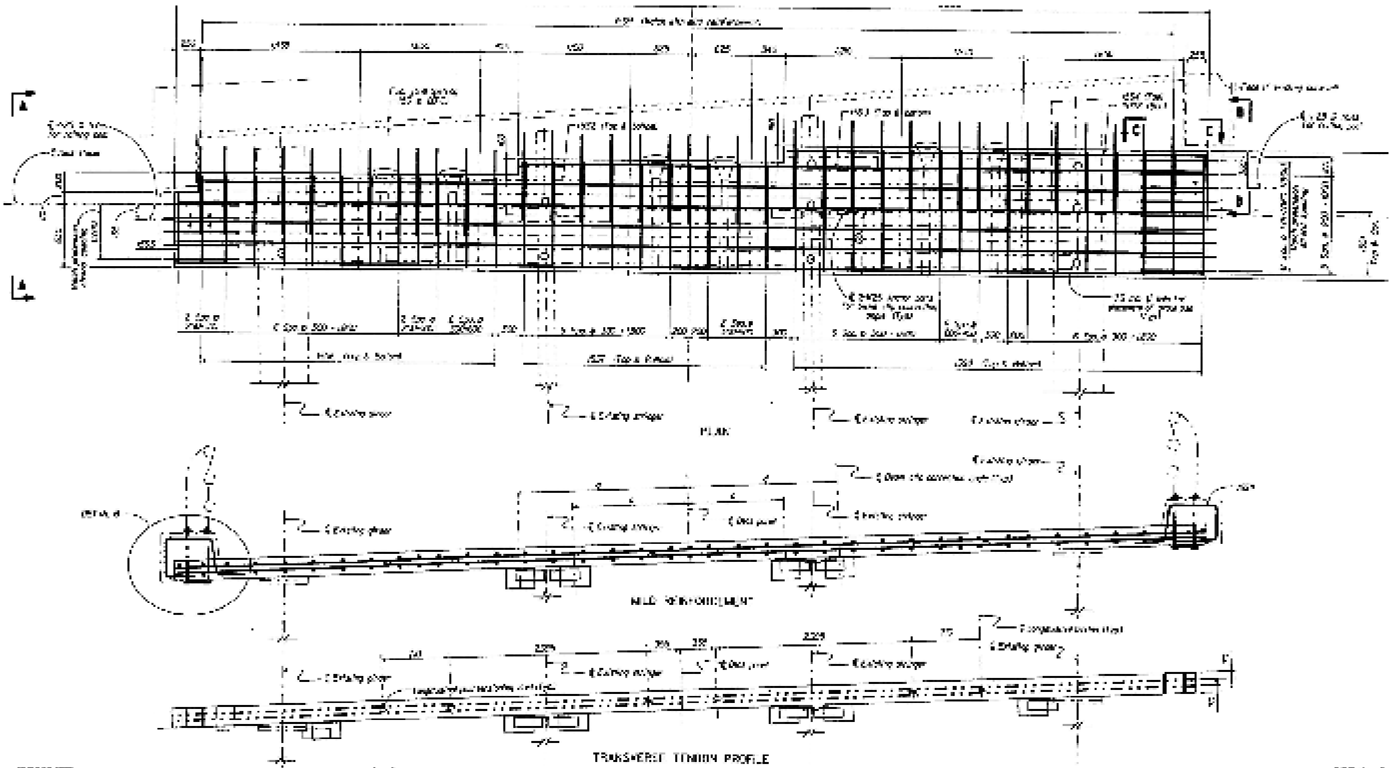


Figure A.2.4.3-16 Details of the end panels

Temporary steel plates were installed at the gap between the old and new deck, as shown in Figure A.2.4.3-17. On the bridge, the steel plates were clipped directly against the old and the new deck. At the abutments and before installing the cast-in-place part of the deck over the abutment wall, the plates were clipped against the top flange of the steel girders and stringers. After the cast-in-place part of the deck was cast over the abutment wall, the steel plates were clipped against the abutment wall.

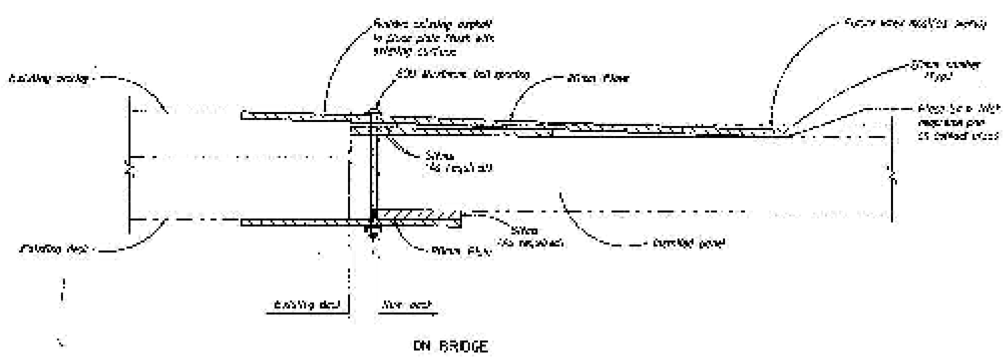


Figure A.2.4.3-17a Details of the temporary joint between the old and new deck on bridge

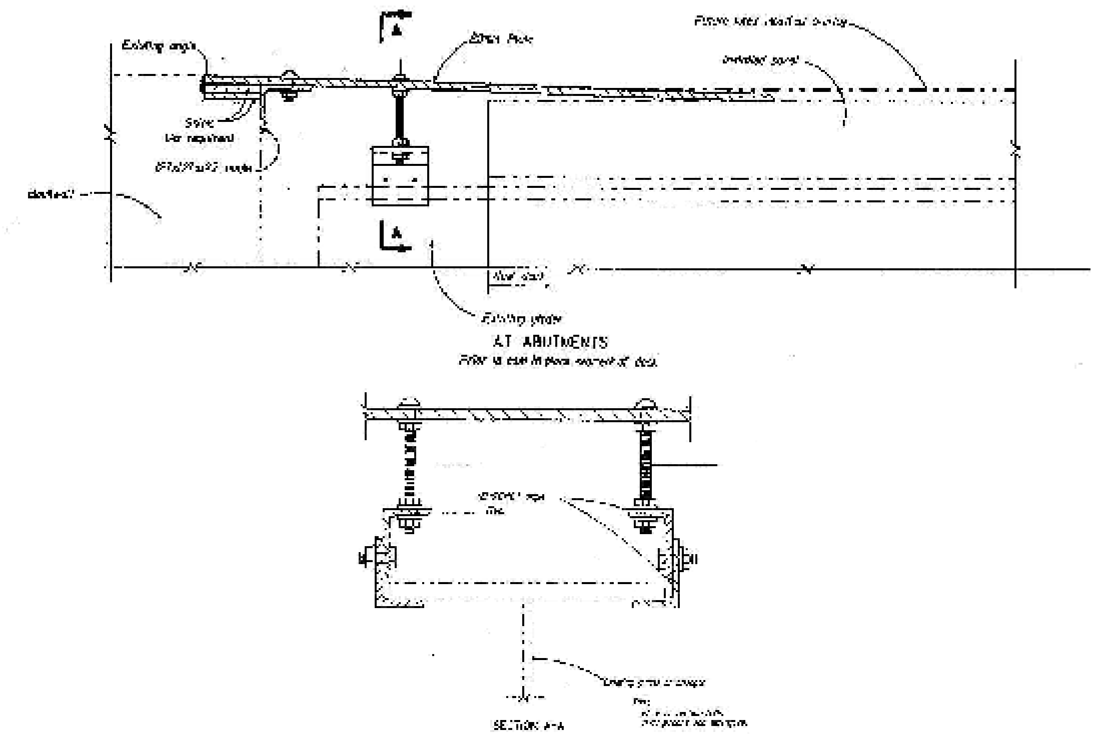


Figure A.2.4.3-17b Details of the temporary joint between the old and new deck at abutment prior to cast-in-place segment of the deck

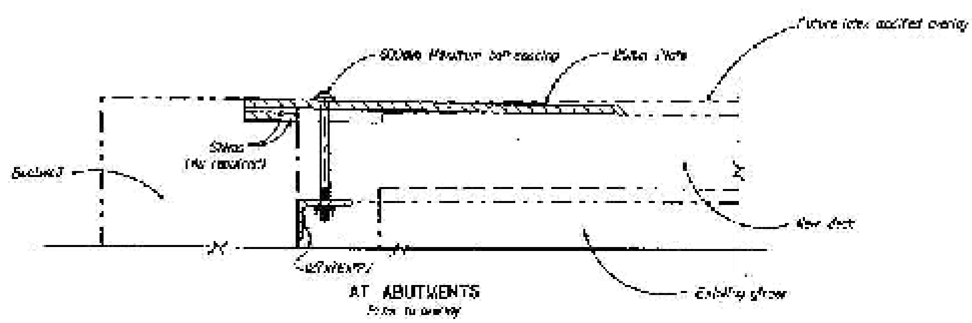


Figure A.2.4.3-17c Details of the temporary joint between the old and new deck at abutment prior to overlay

The Turkey Run Structure

The same type of full-depth, non-composite precast panel system, used on the southbound bridge of the Dead Run structure, was used on the Turkey Run structure. The following section discusses only the differences where they exist.

The structure had two separate bridges, the northbound and the southbound bridges. Deck replacement was called for both bridges. Both bridges had four spans, 92’-8”, 108’-4”, 108’-4” and 92’-8” (28.25, 33.02, 33.02 and 28.25 m) respectively and a total width of 36 ft (10.97 m). Both bridges had zero skew and were located at the bottom of a vertical curve alignment. The deck had a crown in the middle with a 2% cross slope on both sides of the crown. Figure A.2.4.3-18 shows the cross section of the bridges with the old and new deck.

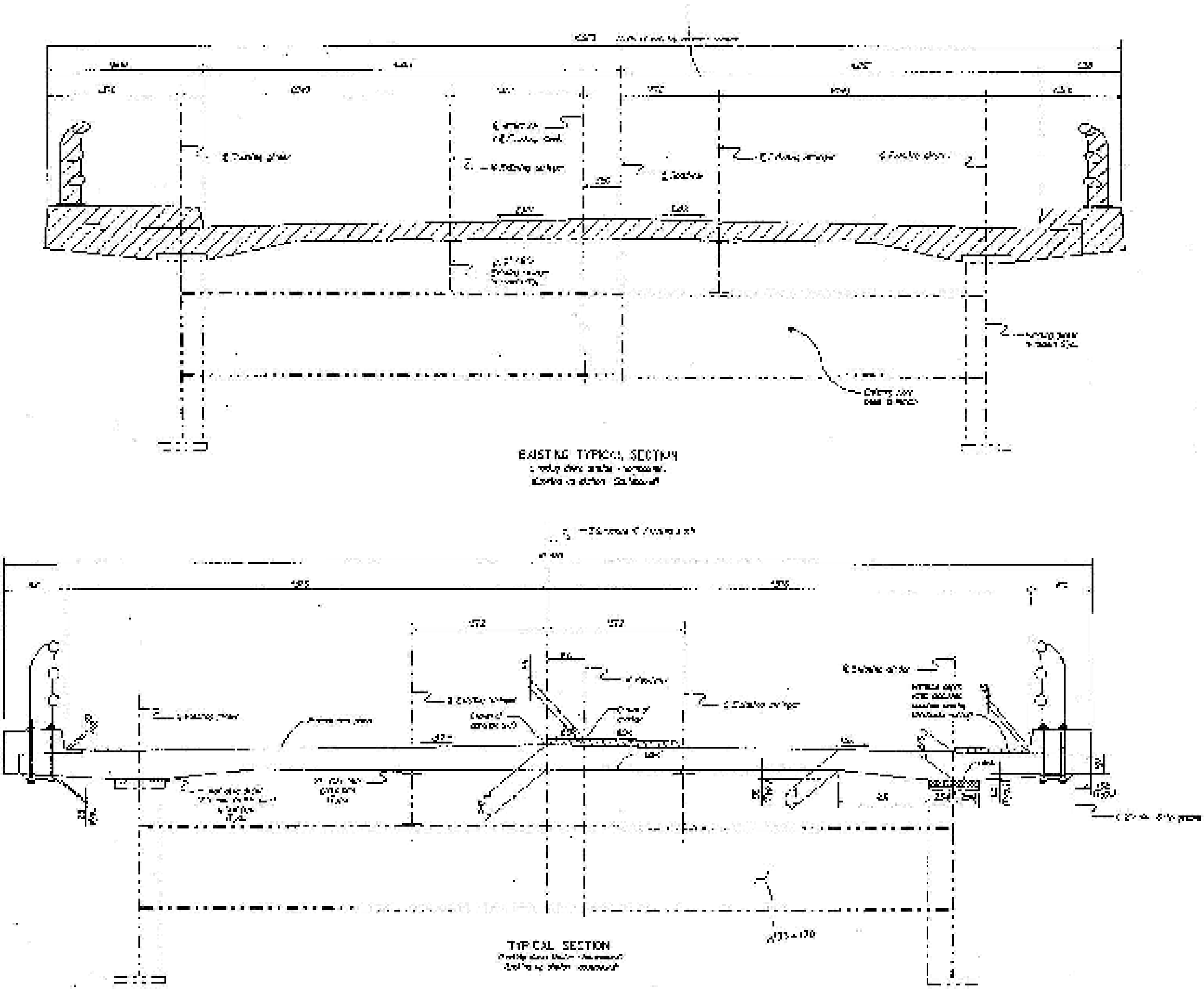


Figure A.2.4.3-18 Cross section of the bridge with the old and new deck

Variable thickness precast panels were used. Haunches were created at the exterior girder lines, as shown in Figure A.2.4.3-19. This decision was taken by the design engineer to avoid modifying the existing system of girders and stringers supporting the deck, which would cause significant delay to the construction schedule. Straight pretensioned strands were used to simplify the production of the panels. The tensile stresses resulted from the eccentricity of the prestressing force at the haunches of the panel were resisted by conventional reinforcement.

A phased construction scenario, similar to that used in Dead Run southbound Bridge, was used in Turkey Run bridges, where construction started at the west side abutment and moved towards the east side abutment, as shown in Figure A.2.4.3-20.

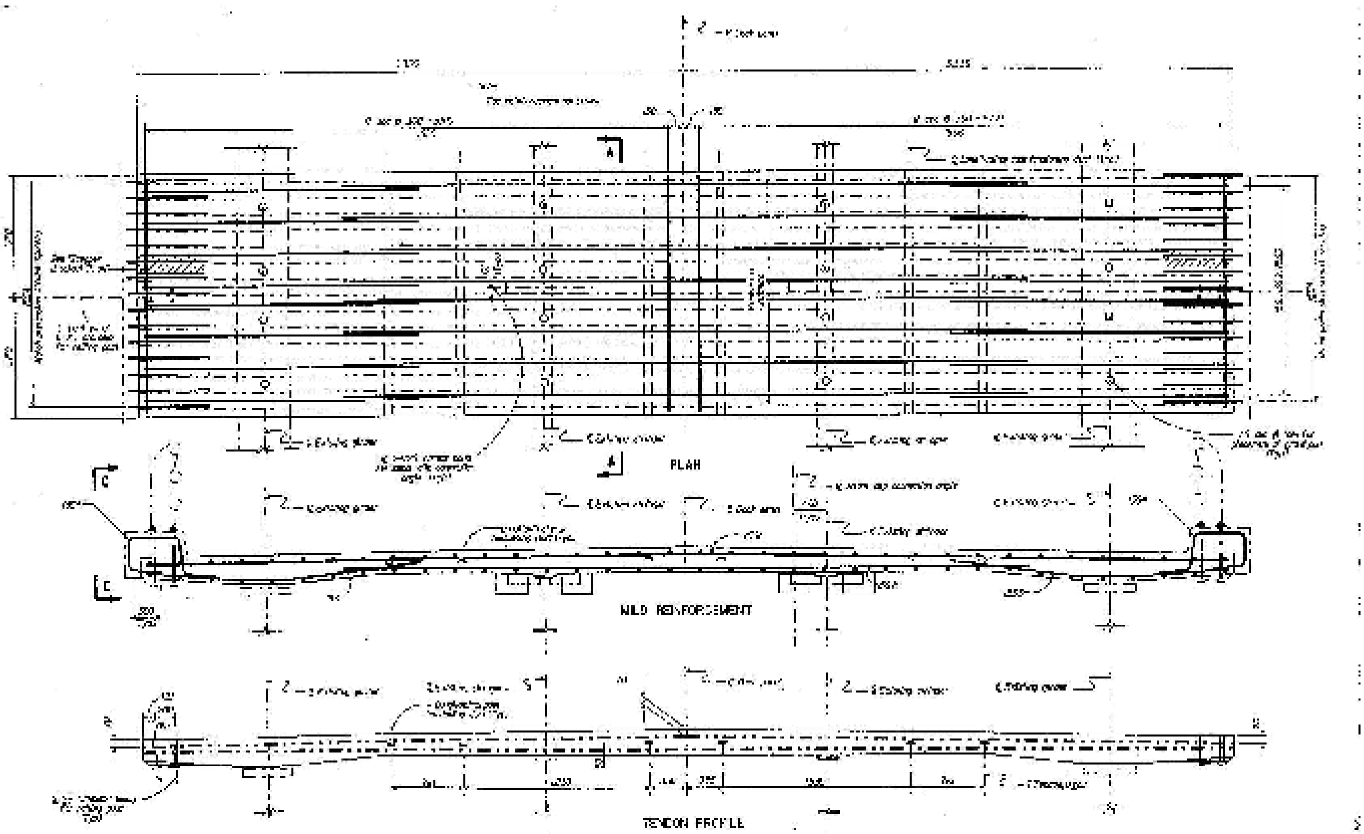


Figure A.2.4.3-19 Details of a typical precast panel

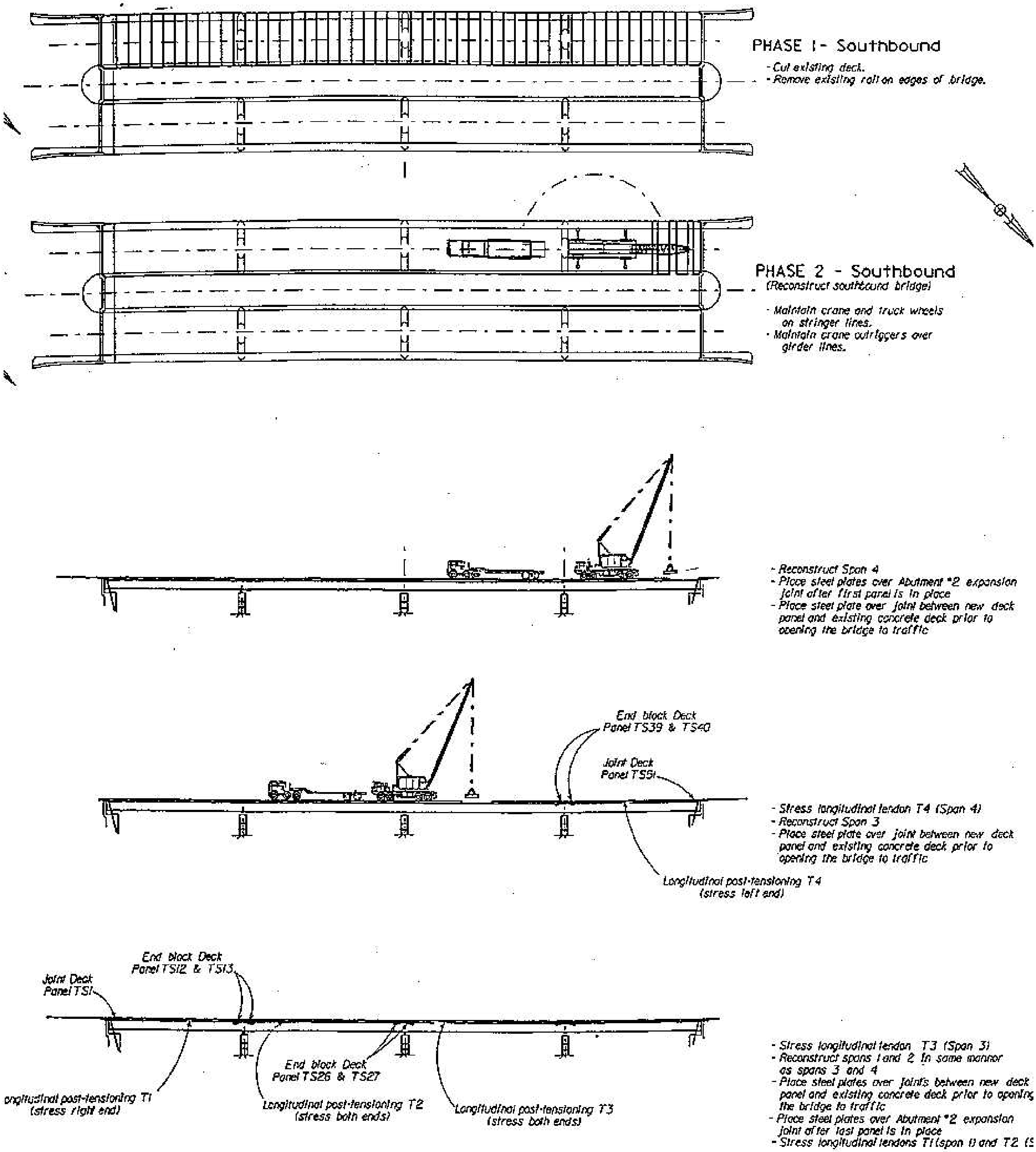


Figure A.2.4.3-20 Phased construction scenario

A.2.4.4 Illinois Department of Transportation

In 1999, Illinois Department of Transportation used a precast concrete panel deck on Bridge-4 constructed on Route 75, Sangamon County. The bridge had four spans, 56 ft-5 in, 67 ft, 67 ft and 56 ft–5 in (17221, 20485, 20485 and 17221 mm). The superstructure was made of six steel girders spaced at 6 ft–4 in. (1956 mm) made composite with concrete deck slab. The bridge had a total width of 37 ft. (11300 mm) and a super elevation with a cross slope of 1.5 percent.

Longitudinal post-tensioning was provided by twenty-two 1.0 in. (25.4 mm) high strength threaded bars spread along the full width of the bridge at 18.2 in. (462 mm) in each span between the six supporting girders, as shown in Figure A.2.4.4-1. The bars were post-tensioned in the order shown in Figure A.2.4.4-1, i.e. from the north edge to south edge of the bridge cross section.

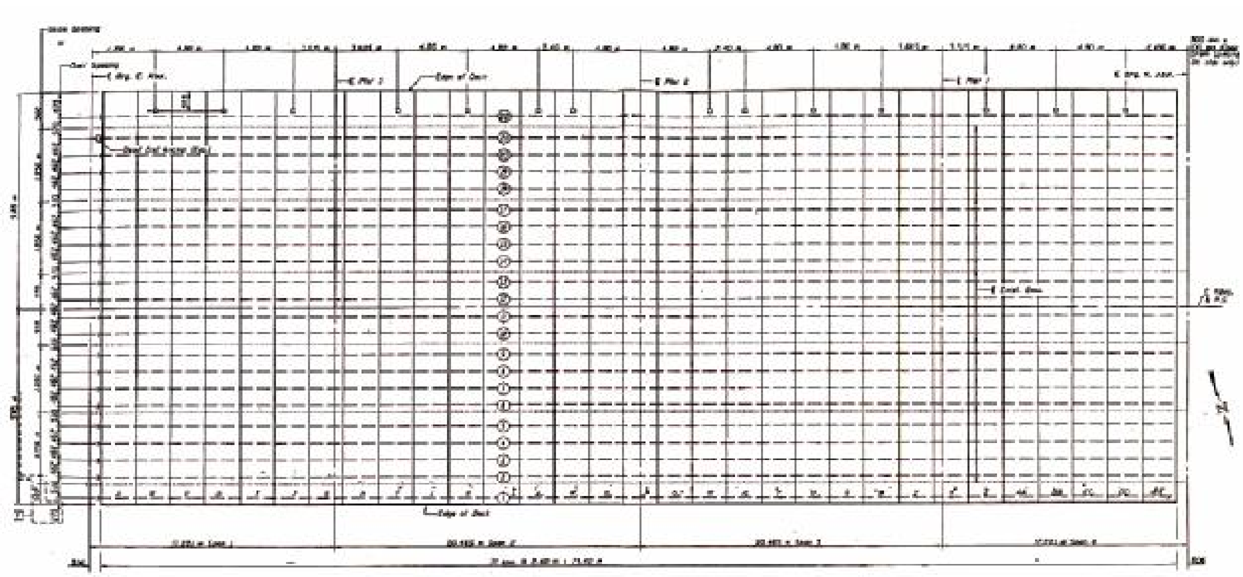


Figure A.2.4.4-1 Arrangements of the panels

The bars were coupled using a standard coupler, as shown in Figure A.2.4.4-2a. A cast-in-place closure pour was provided at each abutments to accommodate the anchorage device, as shown in Figure A.2.4.4-2b.The ducts were grouted after the tendons were post-tensioned to protect the bars against corrosion.

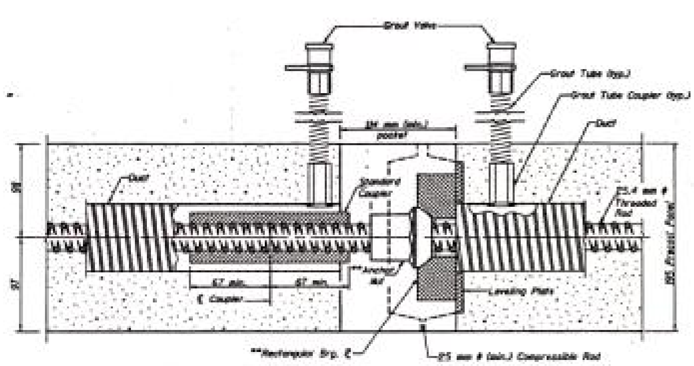


Figure A.2.4.4-2a Coupler detail of Post-tensioned tendon

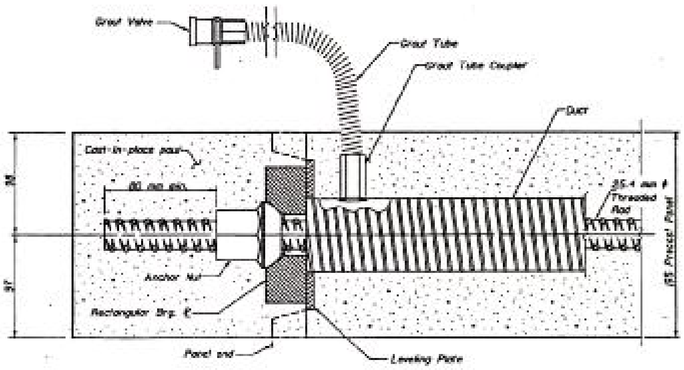
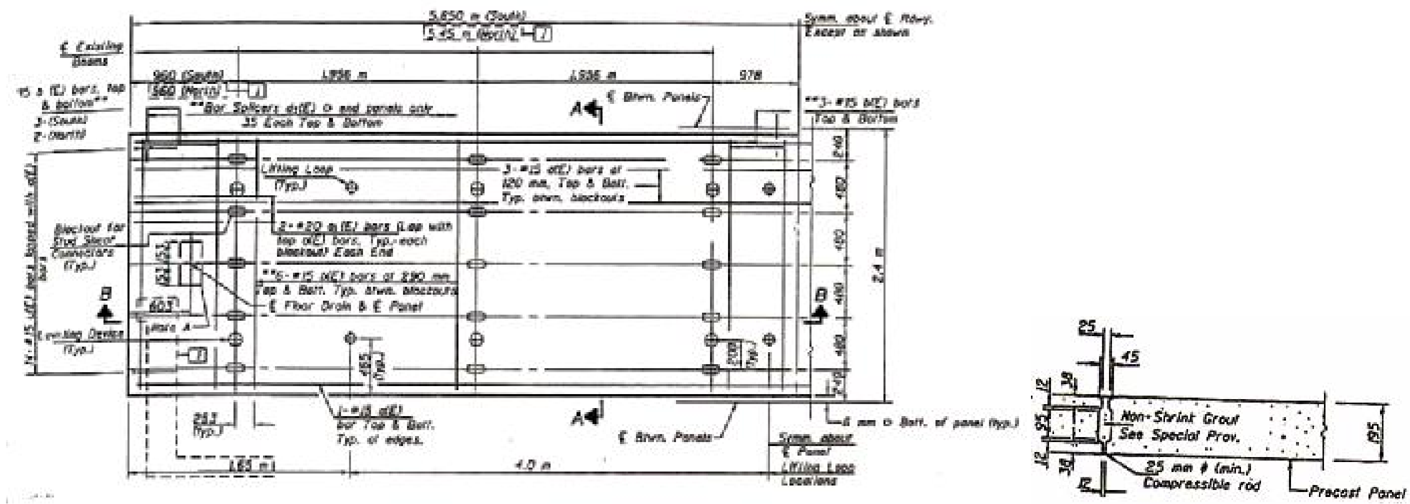
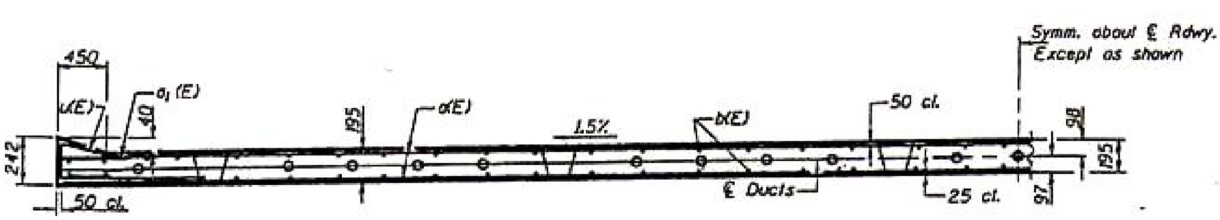


Figure A.2.4.4-2b End anchorage detail of the post-tensioned tendon details

Figure A.2.4.4-3 shows a plan view of the precast concrete panel. The panel had a constant depth of 7.7 in. (195 mm). The depth of the panel was raised to 9.5 in. (242 mm) at both ends over a distance of 17.7 in. (450 mm) as shown in Figure A.2.4.4-4. The width of panel was 7 ft–10 in. (2388 mm). The panel was transversely and longitudinally conventionally reinforced with two layers of epoxy coated reinforcing bars in each direction. The transverse edges of the panel were provided with a female shear key as shown in Section A-A in Figure A.2.4.4-3. A ½ in (12 mm) gap was left between adjacent panels and a compressible rod was used to block this gap.



Section A-A



Section B-B

Figure A.2.4.4-3 Detail of a typical precast panel

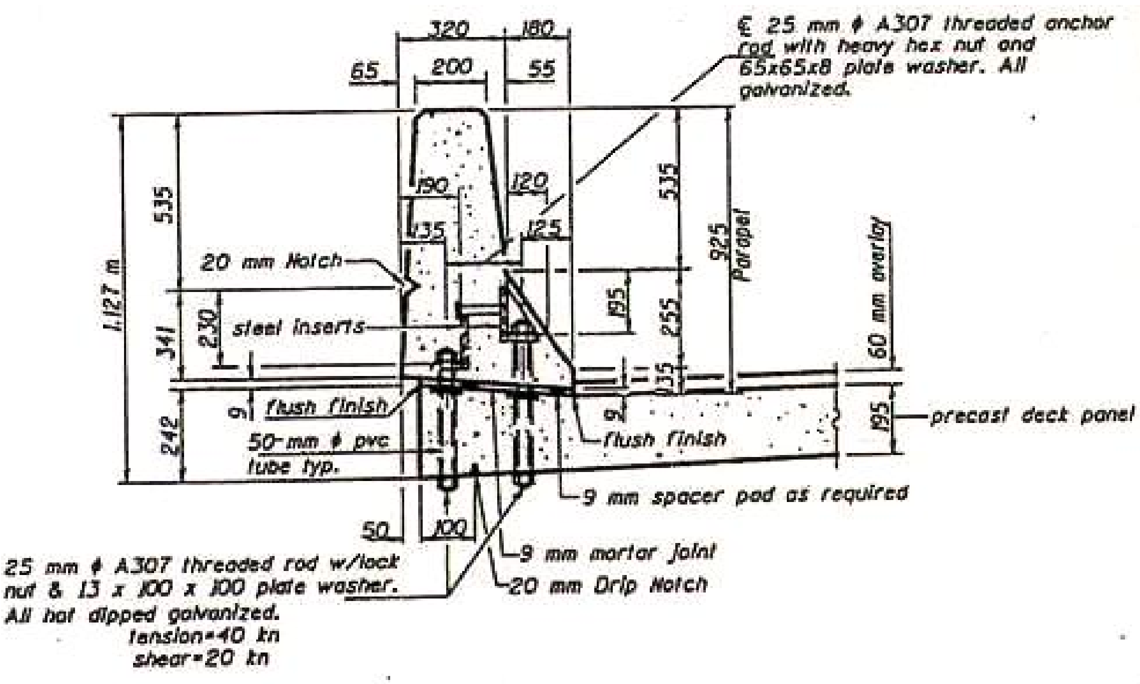


Figure A.2.4.4-4 Detail of a precast panel and a barrier connection

Twelve 7/8 in. (22 mm) diameter, leveling screws per panel were used to adjust for grade as shown in Figure A.2.4.4-3. After a panel was leveled, the leveling screws were cut and recessed inside the panel. High density Styrofoam, glued to the top flange of the steel girders and the bottom surface of the precast panel, was used as a grout barrier. A cement based non-shrink grout was used for grouting the transverse panel-to-panel joints as shown in Figure A.2.4.4-5.

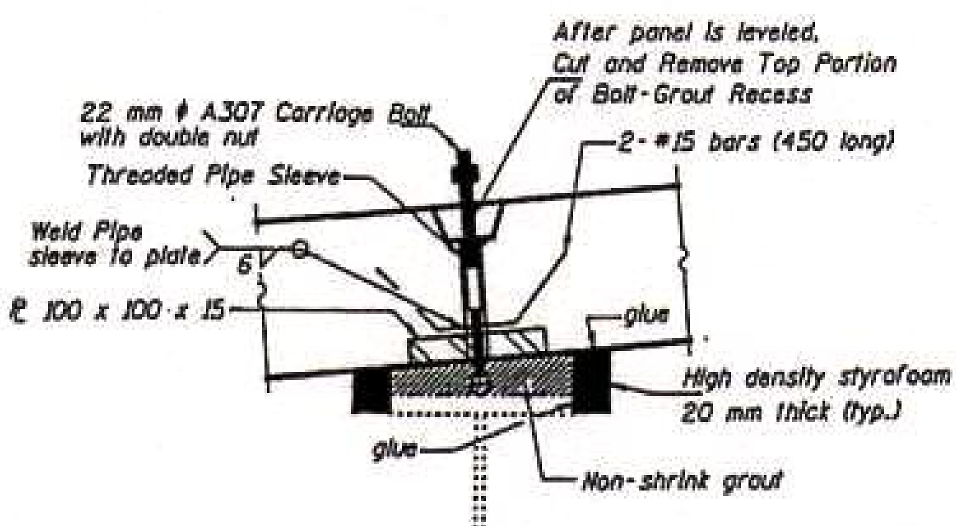


Figure A.2.4.4-5 Detail of a leveling screw

The precast panels were made composite with the steel girders by using tapered grouted shear pockets of 13x7 in. (340x180 mm) as shown in Figure A.2.4.4-6. Each pocket accommodated three 7/8 in. (22 mm) shear connectors, as shown in Figure A.2.4.4-7. The shear connectors were welded to the top flange of the steel girders after the panels were installed and their elevation was adjusted. The shear pockets were provided in the longitudinal direction at a uniform spacing of 18-7/8 in. (480 mm), as shown in Figure A.2.4.4-3.

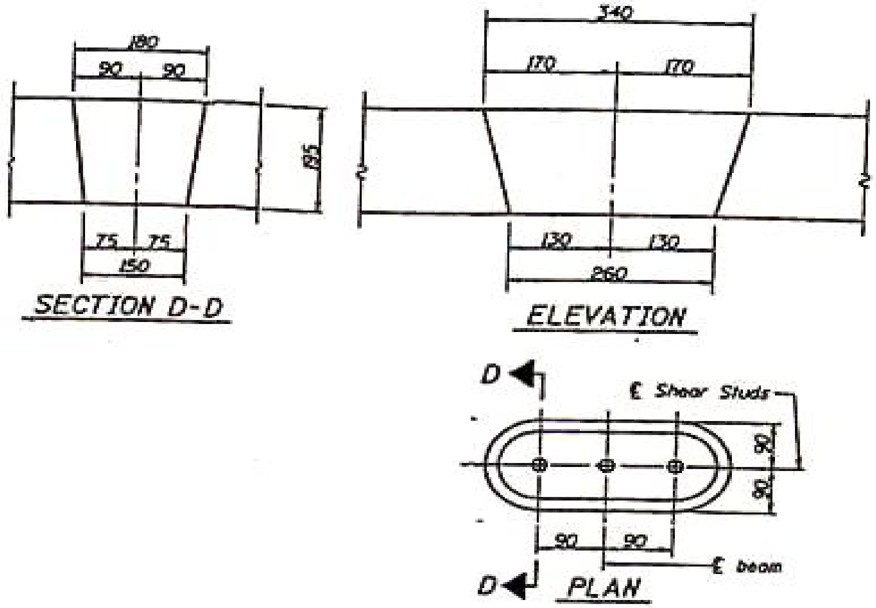


Figure A.2.4.4-6 Stud shear blockout details

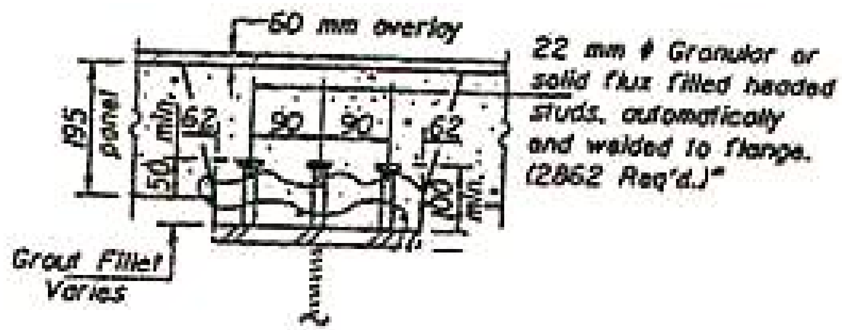


Figure A.2.4.4-7 Connection detail of a precast panel and a girder

Precast New Jersey barriers were connected to the precast panels using galvanized 1.0 in. (25 mm) diameter bolts as shown in Figure A.2.4.4-4. A ½ in. (12 mm) mortar bed was provided between the precast panel and the precast barrier. A 2.3 in. (60 mm) thick, 5,000-psi (35 MPa) microsilica concrete overlay was used to provide for the riding surface.

A.2.4.5 Kentucky Department of Highways

The U.S. 231 Bridge was built over Ohio River and Indiana 66. The bridge was made of: (1) two approach structures constructed with CIP panels and (2) the main structure crossing the Ohio River constructed with full-depth precast deck panels.

The main structure was a cable-stay bridge made of two pylons and three spans, 450, 900 and 450 ft (137000, 274000 and 137000 mm), shown as Spans 8, 9 and 10 in Figure A.2.4.5-1. A cantilever balancing system was used to construct the cable stayed main span of the bridge. Total width of the bridge was 75 ft (22917 mm) and had a crown at the center of the roadway with a 2 percent slope both ways. The superstructure was made of six interior longitudinal steel plate girders spaced at 12 ft-3 in (3748 mm) and two exterior longitudinal steel plate girders. The longitudinal steel girders were supported by transverse floor beams spaced at 15 ft (4571 mm). A typical cross section of the bridge is shown in Figure A.2.4.5-2. An overlay of latex modified concrete of thickness 1½" was used over the precast panels to provide for the riding surface.

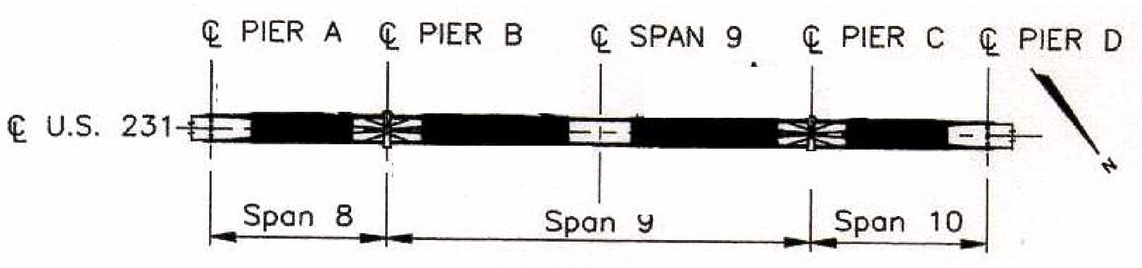


Figure A.2.4.5-1 Schematic plan of the main structure

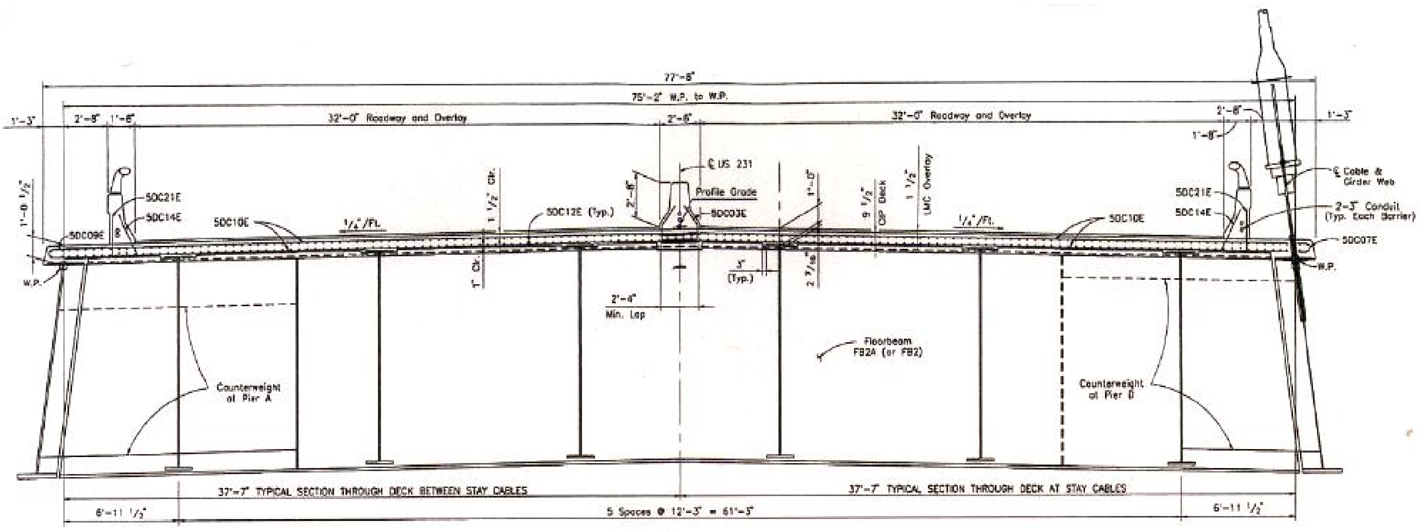


Figure A.2.4.5-2 Typical cross section

Two solid precast panels connected at the crown-point were used across the width of the bridge. A cast-in-place (CIP) concrete pour, 28 in. (711 mm) wide, was used at the crown to connect the two panels. Also, two CIP concrete pours were cast along the edges of the bridge outside the road barrier. The precast panels were 9½ in. (241 mm) thick and conventionally reinforced with two layers of epoxy coated steel bars in each direction. The transverse reinforcement of the panel was extended outside the panel into the CIP concrete pours as shown in Figure A.2.4.5-3. Shear connectors between the deck and the precast deck was provided only on the exterior steel girders. This arrangement resulted in using solid panels with no shear pockets, which simplified the fabrication of the panels. The longitudinal edges of the precast panels were provided with shear keys that provided horizontal mechanical interlocking with the CIP longitudinal pours, as shown in Figure A.2.4.5-4.

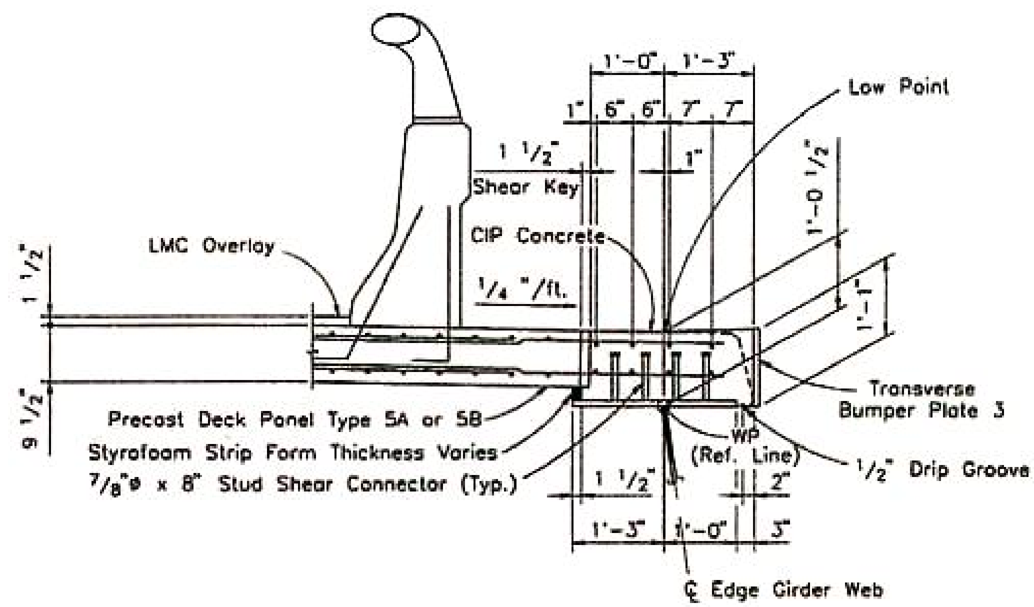


Figure A.2.4.5-3 Connection detail of precast panel with the barrier and the exterior girder

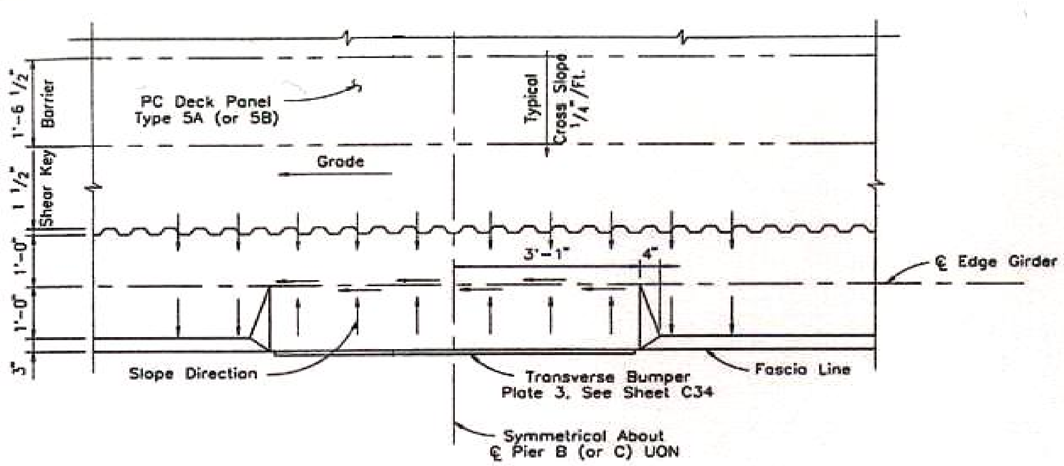


Figure A.2.4.5-4 Plan view showing detail of the shear key system

Figure A.2.4.5-5 shows a plan view of a bridge at pier B. Each panel was supported on two adjacent transverse floor beams. No longitudinal post-tensioning was used to connect the panels in the longitudinal direction. The longitudinal reinforcement of a panel was extended outside and spliced with similar reinforcement of the adjacent panel, as shown in Figure A.2.4.5-6. A 6-in. (152 mm) gap was maintained between panels over the floor beams and the bottom layer of the panel reinforcement extending into this gap was bend to form hook. Two transverse bars were installed inside the hook. To splice the top layer of reinforcement of the panels, the depth of the panel was lowered to 4 in. (102 mm) and CIP concrete was used to fill this area.

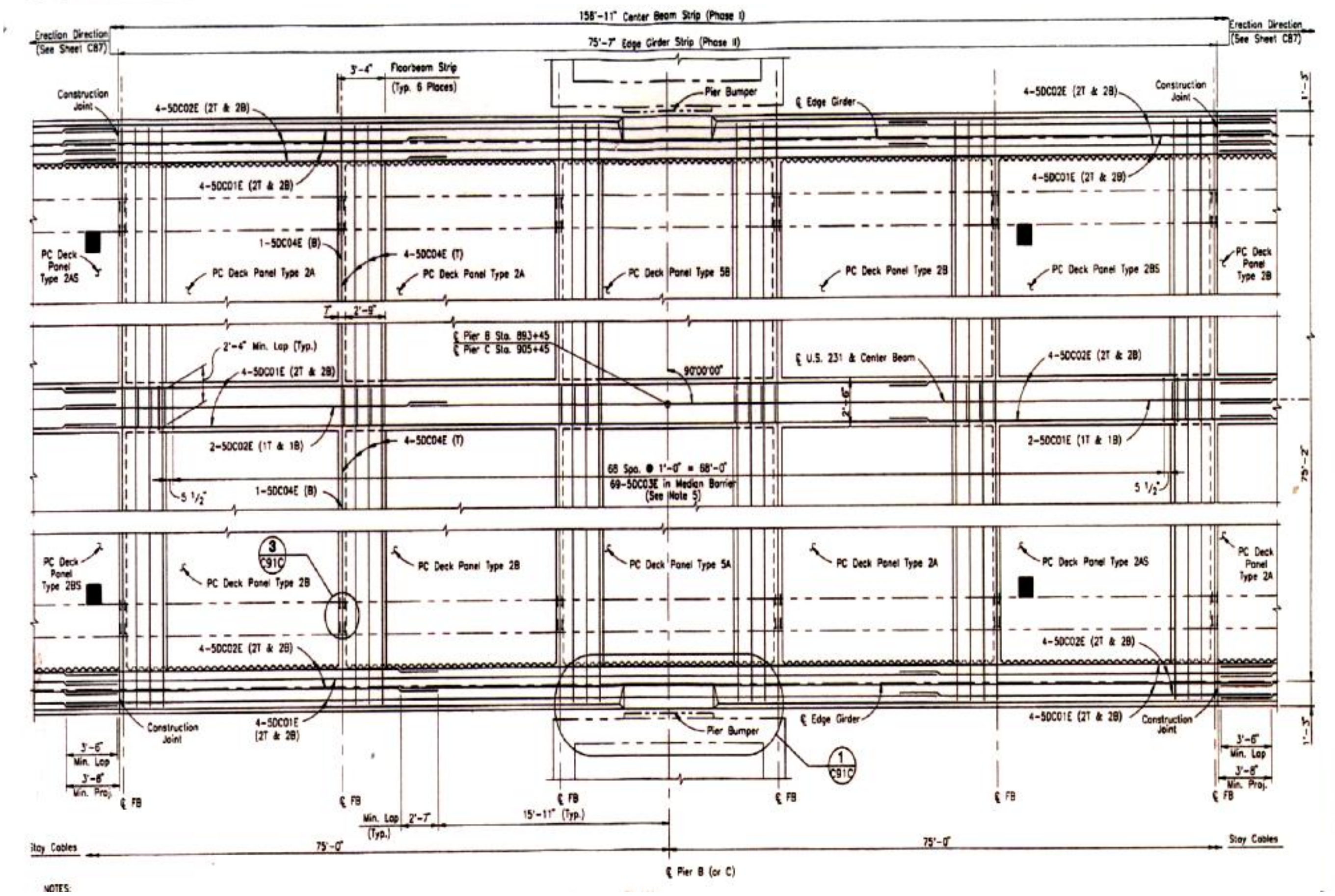


Figure A.2.4.5-5 Plan view of a bridge at pier B

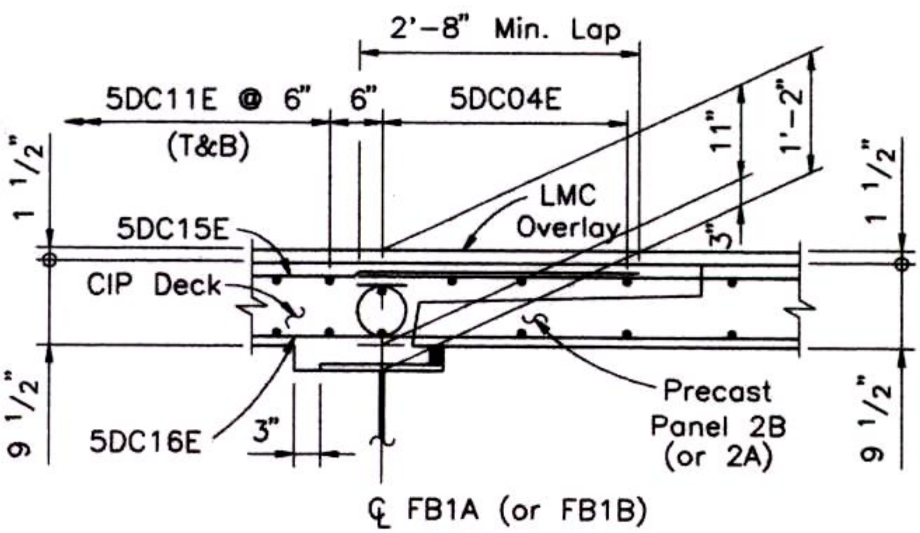


Figure A.2.4.5-6 Transverse connection between adjacent panels

A.2.4.6 Missouri Department of Transportation

Missouri Department of Transportation (MoDOT) has decided to use a full depth precast deck panel system for the new Bill Emerson Memorial Bridge, spanning the Mississippi River between Southeast Missouri and Southern Illinois. This bridge will replace the current structure that was built in 1927. It will link Cape Girardeau, Mo., and East Cape Girardeau, Ill., and span the Mississippi River. The estimated cost of the bridge is $100 million. Eighty percent of the funding comes from the federal government. Missouri and Illinois each contribute 10 percent. The Bill Emerson Memorial Bridge is anticipated to open December 2003.

Although MoDOT has provided the research team with details of the precast panel system, MoDOT has asked the research team to disseminate only the information that is made available to the public due to security reasons. The following sections give an overall idea of the bridge and general description of the precast deck panel system.

The main span of the structure over the Missouri river is 4,000-foot (1219 m) long cable-stay bridge. Total width of the bridge is about 100 ft (30480 mm). The superstructure is made of three longitudinal girders spaced at about 50 ft (15240 mm) and transverse floor beams spaced at 18 ft (5486 mm). The roadway is crowned at its centerline directly over the center girder. Figure A.2.4.6-1 shows a general view of the old and new bridges, and Figure A.2.4.6-2 shows a top view of the precast deck with the superstructure system.

The deck slab is made of two precast panels across the width of the bridge meeting at the center girder, which has resulted in using flat panels. The length of the panel is made so that each panel spans between two adjacent floor beams. This arrangement has resulted in a rectangular panels supported on four side, as shown in Figure A.2.4.6-2.

Figure A.2.4.6-1 General view of the old and new Bill Emerson Memorial Bridges

The precast deck panels are conventionally reinforced with top and bottom meshes of epoxy coated bars. The thickness of the precast panels is about 10 in. (254 mm). Cast-in-place concrete is used in constructing the side and median barriers. Shear connectors between the precast panels and the barriers are provided with precast panels.

Cast-in-place concrete joints are used to connect the panels in the transverse and longitudinal directions. The connection consists of reinforcing bars extended outside the panel and overlapped in the gap between panels with similar bars extending from the neighboring panel. A horizontal shear key detail is provided on all edges of the precast panel that provides shear interlocking in the horizontal direction. A short cast-in-place concrete cantilever is cast over the edge girders, as shown in Figure A.2.4.6-2.

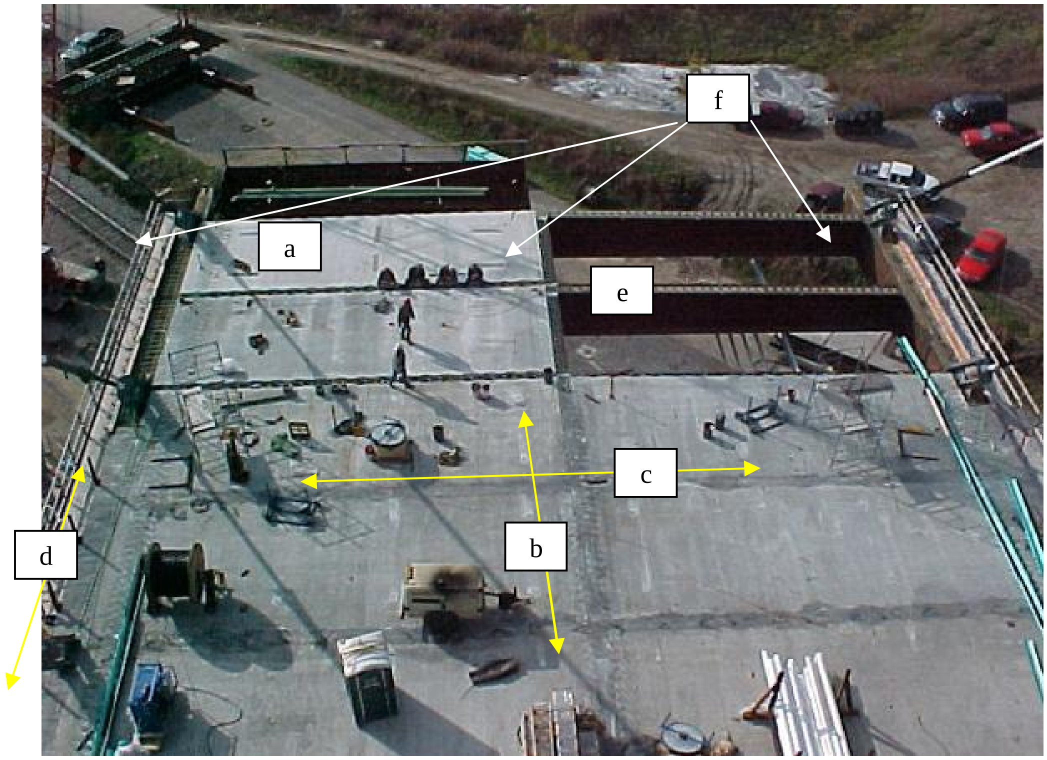


Figure A.2.4.6-2 Top view of the new Bill Emerson Memorial Bridge

(a) Typical Precast Panel, (b) CIP Longitudinal Joint, (c) Transverse CIP Joint, (d) Short CIP Cantilever, (e) Typical Floor Beam, and (f) Longitudinal Girders

The panels are installed in groups of two lines of panels in the longitudinal direction on each side of the pylon. After the joints between panels are cast and cured, longitudinal post-tensioning is applied. Then this part of the deck can support construction loads and is used as a platform for installing next group of precast panels. High strength post-tensioning bars spaced at about 12 in. (304.8 mm) across the width of the panel, as shown in Figure A.2.4.6-3. A 3-in. (76 mm) silica fume overlay is cast on the precast panels to provide for the riding surface.

The precast deck is made composite with the superstructure using steel studs welded to the top flange of the steel girders and cross-floor beams, as shown in Figure A.2.4.6-4. Due to the unique arrangement of the super structure girders and floor beams, no shear pockets are created in the precast panels, which has significantly simplified and speeded up the production process of the panels.

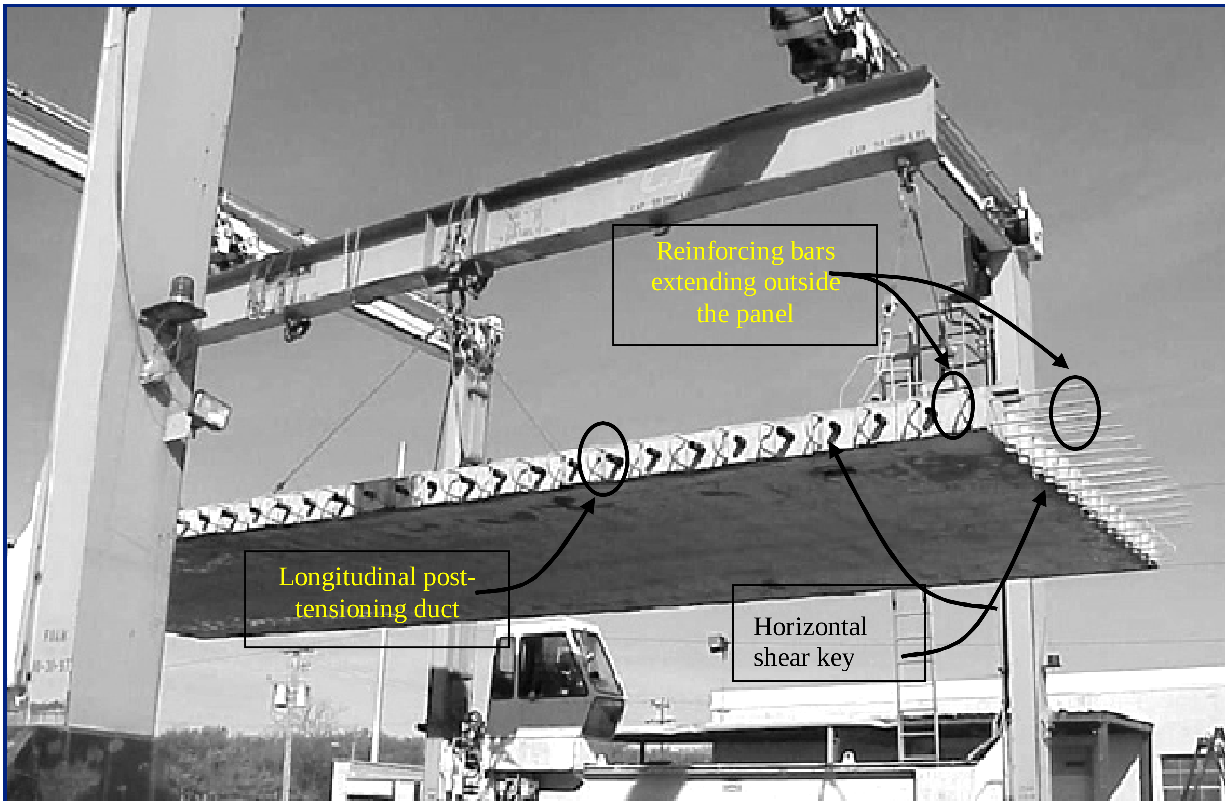


Figure A.2.4.6-3 General view of the Precast Panel showing the longitudinal Post-Tensioning Ducts, the #5 (M16) bars and the shear key

Figure A.2.4.6-4 Steel studs welded to the top flange of the girders and floor beams

A.2.4.7 Montana Department of Transportation

A rehabilitation project was conducted to replace the 33 ft 6 in. (10.2 m) wide concrete deck of the ***Lake Koocanusa Bridge, Lincoln County,*** with a 36 ft 2 in. (11.0 m) precast concrete deck. The old deck slab was a 6 in. (153 mm) CIP lightweight concrete cast on stay-in-place metal forms supported non-compositely on steel girders.

The bridge had six continuous spans, 285, 400, 500, 500, 400, and 352 ft (86868, 121920, 152400, 152400, 12190, and 107290 mm). Total width of the bridge was 36 ft – 2 in. (11024 mm). To maintain traffic on the bridge during construction, a staged construction sequence was planned, which included the removal of half width of the old deck while keeping the other half open for traffic. The new deck consisted of two prestressed precast panels across the width of the bridge supported compositely on the steel girders. The south panels are 15 ft 7 in. (4750 mm) wide and the north panels are 20 ft 7 in. (6275 mm) wide. All panels are 8 ft (2440 mm) long, as shown in Figure A.2.4.7-1.

The composite action between the precast deck and the superstructure was provided by using $^{7}/_{8}$ in. (22 mm) diameter studs welded on the first interior steel girders and C4x7.25x10 in. structural steel sections welded on the exterior steel girders. No composite action was provided at the center steel girder to avoid conflict with the longitudinal connection between panels. To avoid making any changes of the superstructure supporting girders, the thickness of the panel was raised from 7 in. (178 mm) to (254 mm) at the exterior girders. The precast panels were transversely pretensioned with 16-½ in. (12.7 mm), 270 ksi (1.86 GPa), as shown in Figure A.2.4.7-2.

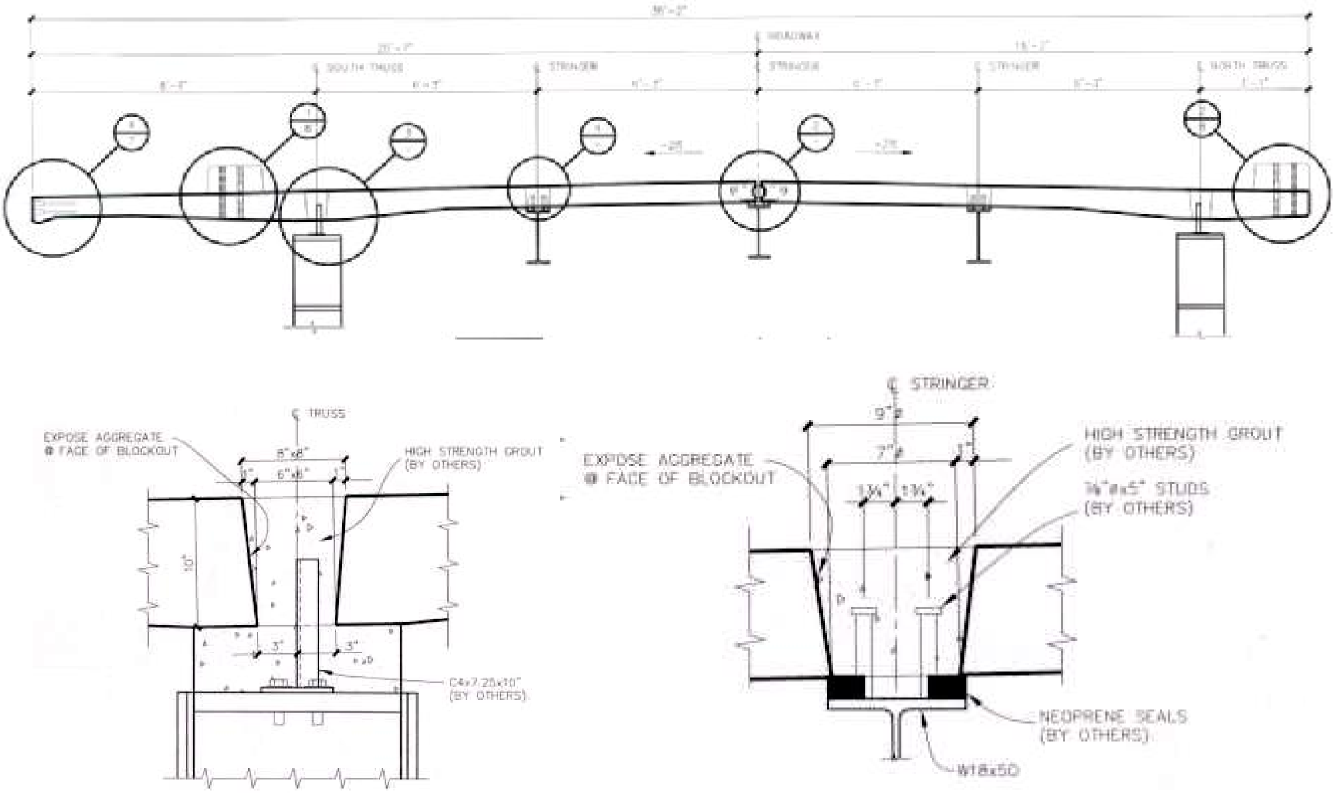


Figure A.2.4.7-1 Cross section of the new deck slab and composite connection details

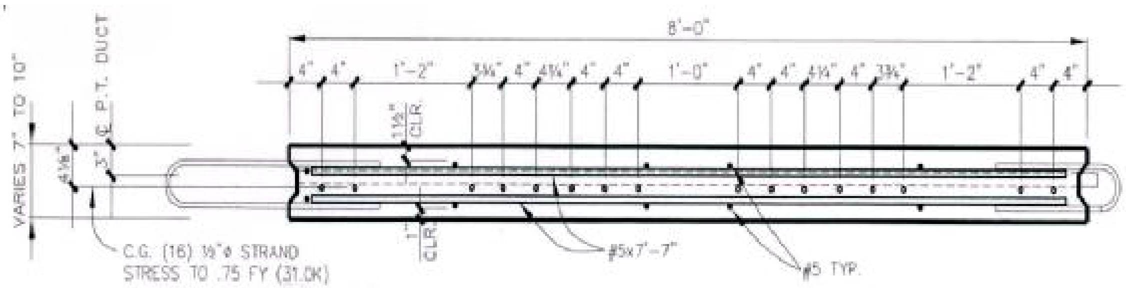


Figure A.2.4.7-2 Typical cross section of the precast panel

The precast panels were post-tensioned longitudinally. The post-tensioned tendons were anchored at 50 ft (15,240 mm) bays. Each bay consisted of six panels, two edge panels that accommodated the anchorage devices and 4 interior panels, as shown in Figures A.2.4.7-3 and A.2.4.7-4. The transverse edge of all of the panels was provided with a vertical shear key and #4 (M13) U-bar pins extended outside the panels in a 4 in. (100 mm) gap created at the transverse joint between adjacent panels as shown in Figure A.2.4.7-2 and A.2.4.7-4.

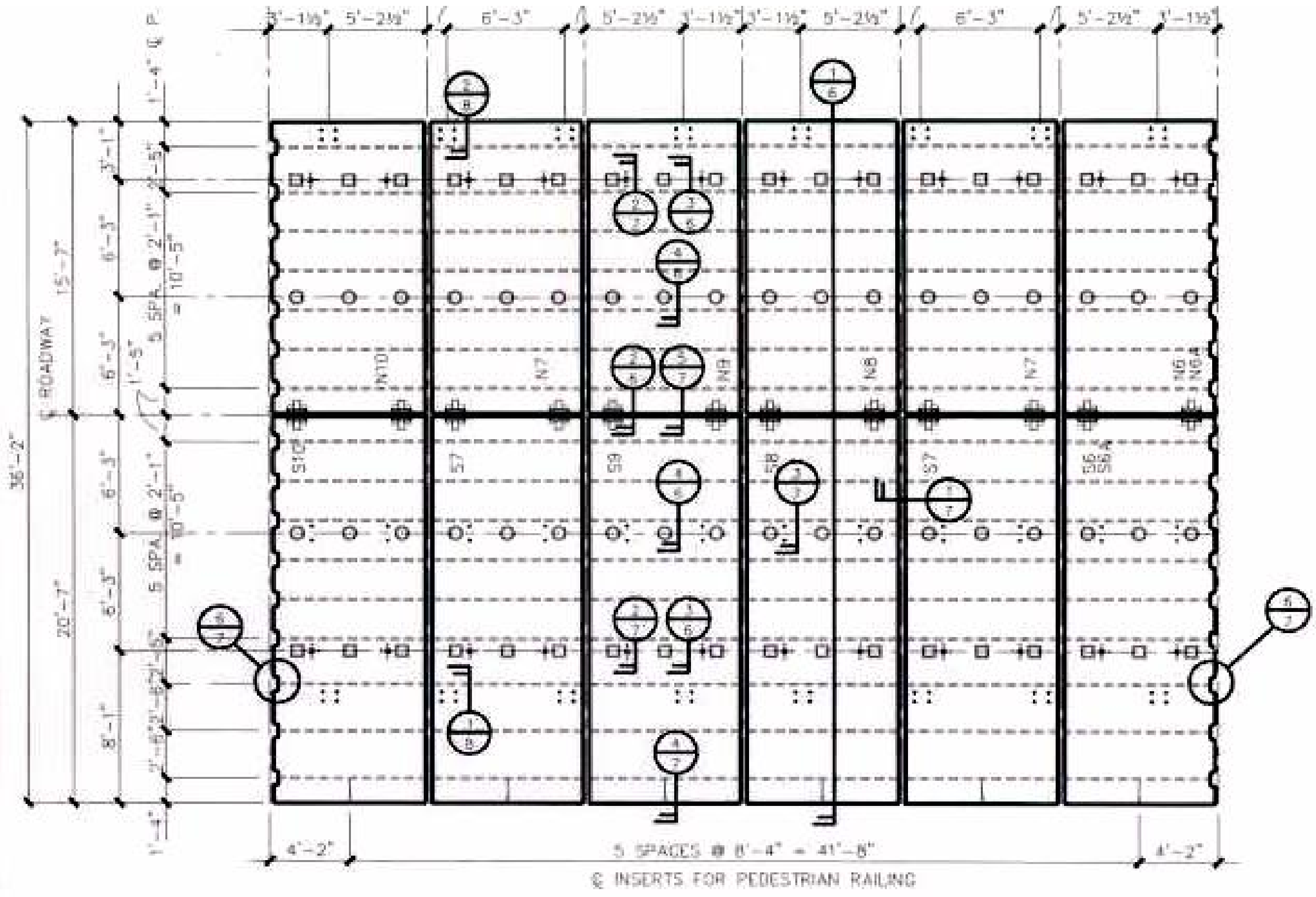


Figure A.2.4.7-3 Typical 50-ft bay with longitudinal post-tensioning

In order to connect the south and north panels together across the width of the bridge, a galvanized steel assembly, as shown in Figure A.2.4.7-5, was provided at 5 ft 6 in. (1,676 mm) spacing. The connection was made over the center steel girder where no composite action was provided.

The elevation of the precast panels was adjusted using a leveling screw device. The details of the device are given in Figure A.2.4.7-6.

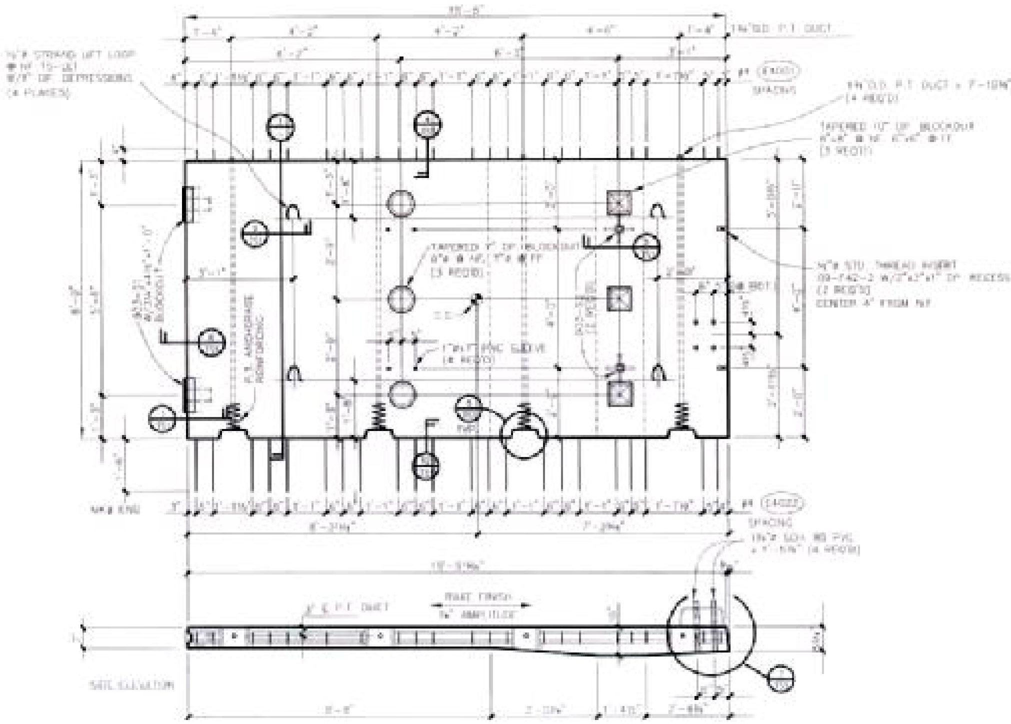


Figure A.2.4.7-4a Details of edge panels of a typical 50-ft bay

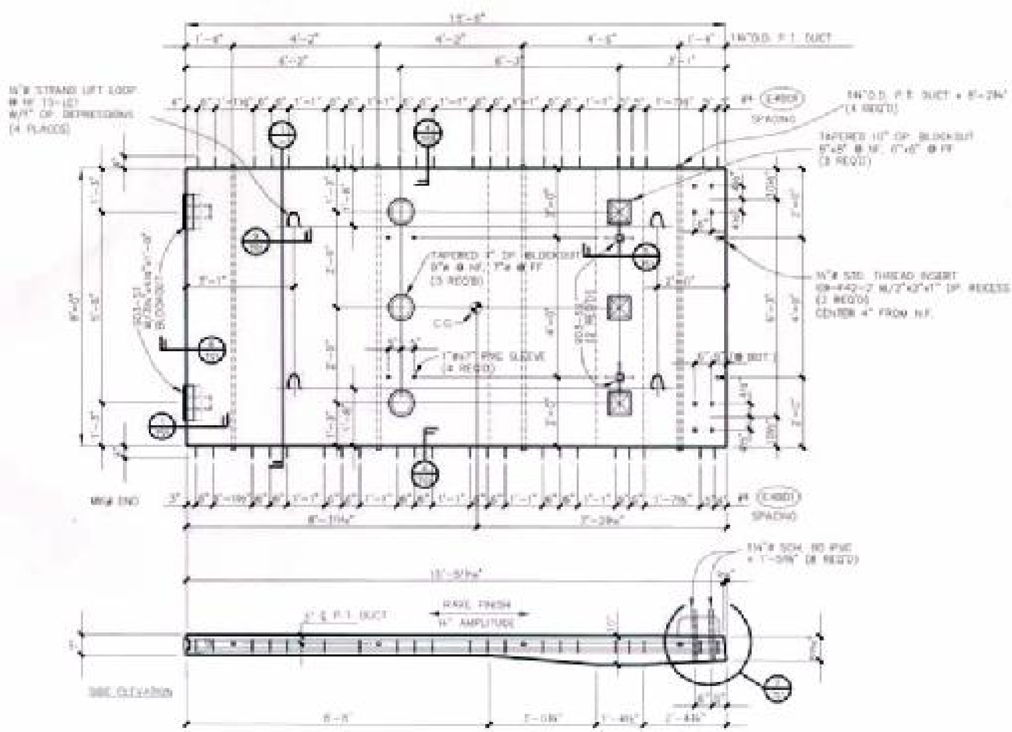


Figure A.2.4.7-4b Details of interior panels of a typical 50-ft bay

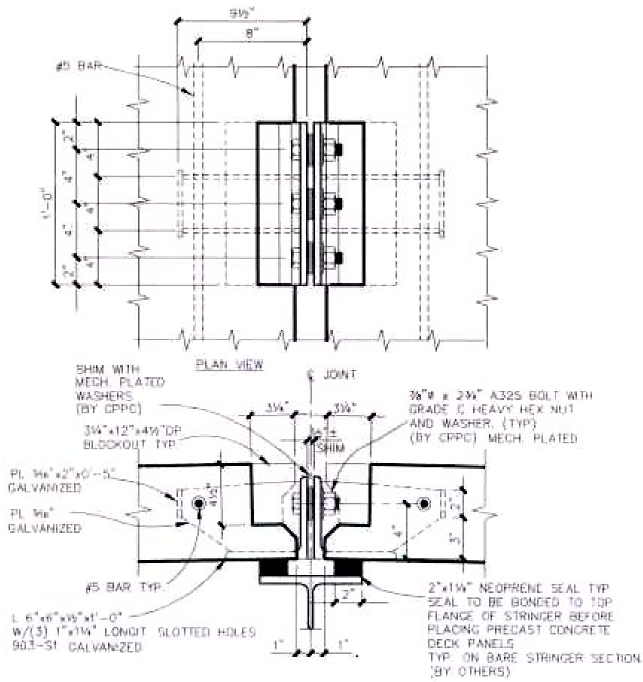


Figure A.2.4.7-5 Longitudinal connection between the south and the north panels

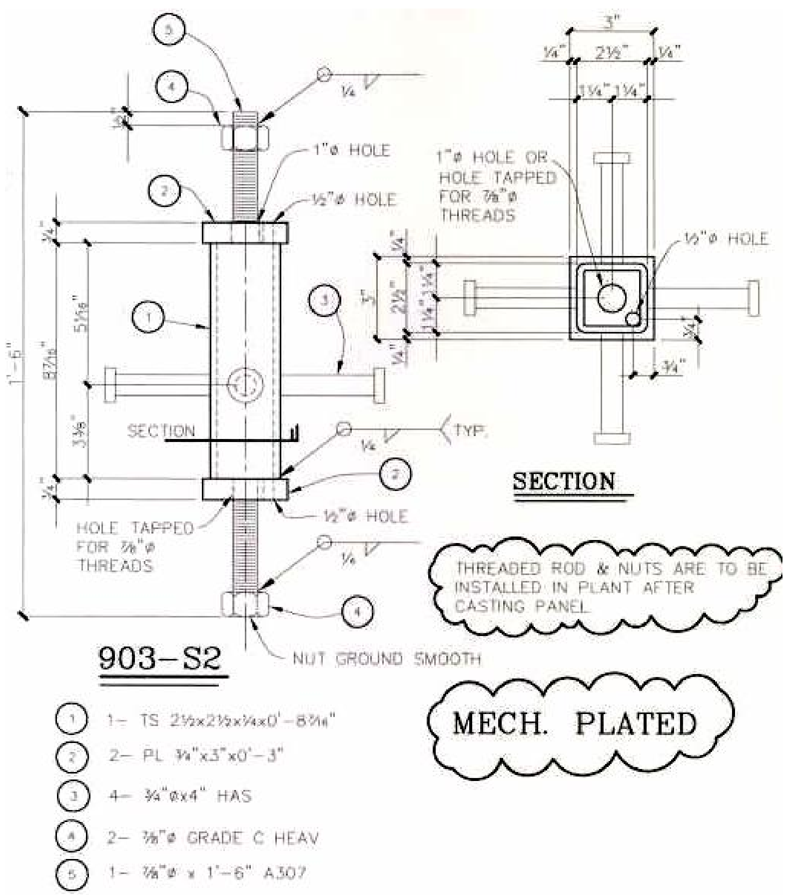


Figure A.2.4.7-6 Details of the leveling screw device

A.2.4.8 Nebraska Department of Roads (13)

After the NUDECK system was developed as a stay-in-place precast panel system with a composite cast-in-place topping *(14,15,16,17)*, Nebraska Department of Roads (NDOR) decided to use this system after modifying it to a full depth precast panel system. NDOR chose the Skyline Drive Bridge over West Dodge Street, located in Douglas County, Omaha, to implement the system.

The bridge had two spans, 89'-0" and 124'-6" (27150 and 37950 mm) respectively, 25-degree skew angle, and a 1.4 percent slope in the direction of traffic, as shown in Figure A.2.4.8-1. The super structure was made of five steel plate girders spaced at 10'-10" (3300 mm) measured in the normal direction to the girders. The deck had two cantilevers, 4'-1" (1250 mm) each, as shown in Figure A.2.4.8-2. The bridge had two 12 ft (3600 mm) traffic lanes with 7 ft (2130 mm) shoulder on each side, and a 9'-10" pedestrian lane on one side.

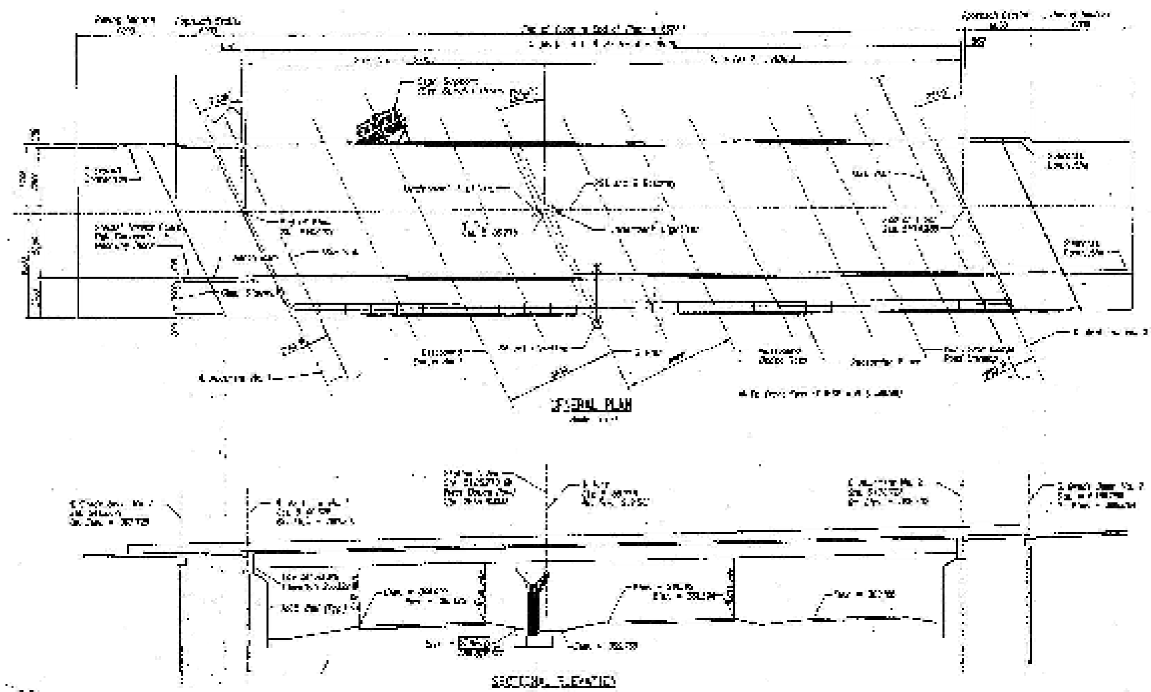


Figure A.2.4.8-1 Plan and elevation views of the bridge

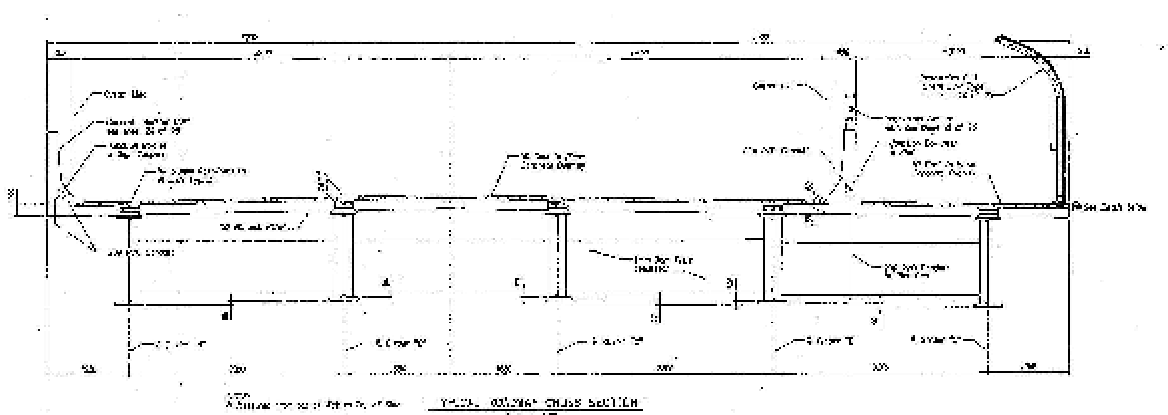


Figure A.2.4.8-2 Cross section of the bridge

The deck slab was made of 5.9 in. (150 mm) full depth precast panel with a 2 in. (50 mm) concrete overlay. The precast panels covered the full width of the bridge and they were transversely pretensioned and longitudinally post-tensioned. Two types of panels were used, 26 typical panels on the bridge between abutments and two end panels at the abutments where the anchorage devices for the longitudinal post-tensioning were housed. Figure A.2.4.8-3 shows a plan view of the panel arrangement.

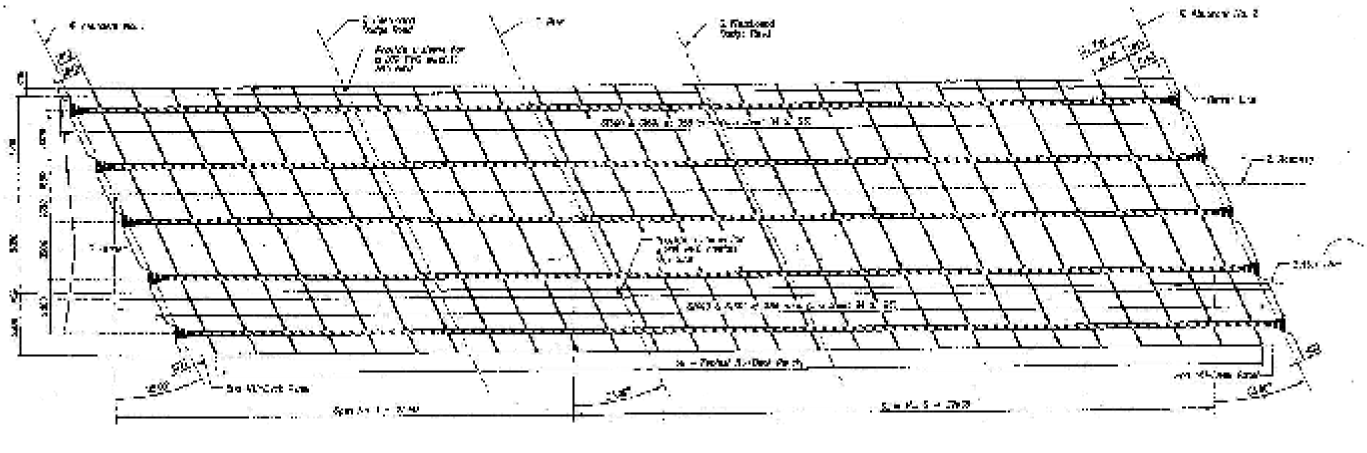


Figure A.2.4.8-3 Plan view of the panel arrangement

Figure A.2.4.8-4 shows a plan view of a typical panel. The panel length was 7'-0" (2145 mm) and was reinforced over its full width with 8- ½ in. (12.7 mm) strands arranged on two layers. Over every girder line, a blockout (i.e. a gap with no concrete) was created to give maximum flexibility for installing and arranging the shear connectors. In order to transfer the prestressing force from one block of concrete to the adjacent block over the gap, four #7 (#22) bars arranged on two layers were installed by each group of two strands. The size of these bars was designed to protect them from buckling under the compression force and to maintain the prestressing force over the gap. The top bars were made of short pieces 5 ft (1551 mm) long at interior girder lines and 6'-10" (2100 mm) long at exterior girders, while the bottom bars covered the full width of the panel. The release and 28-day strength of the concrete mix used were 4,300 and 6,0 00ksi (30 and 42 MPa) respectively.

In the overhang part of the panel, each group of two strands was confined with a 4½ in. (114 mm) OD, 4 in. (100 mm) ID, 1 in. (25 mm) pitch, 145 ksi (1000 MPa) spiral. The spirals were used to reduce the required development length of the ½ in. (12.7 mm) strands from about 7 ft (2133 mm) to 2 ft (610 mm). For more information about this technique see (*14,15*).

The top surface of the panels was roughened to ¼ in. (6 mm) amplitude using a finishing broom. A v-shape shear key was created at the transverse edges of the panel and a backer rod was used to close the gap between adjacent panels and protect grout from leaking. Non-shrink early-strength, 6,000 psi (42 MPa) specified strength grout, was chosen for the project. The design plans did not specify any treatment of the shear keys prior to grouting. However, the plans specify that the top surface of the grouted joint should be roughened to ¼ in. (6 mm) amplitude.

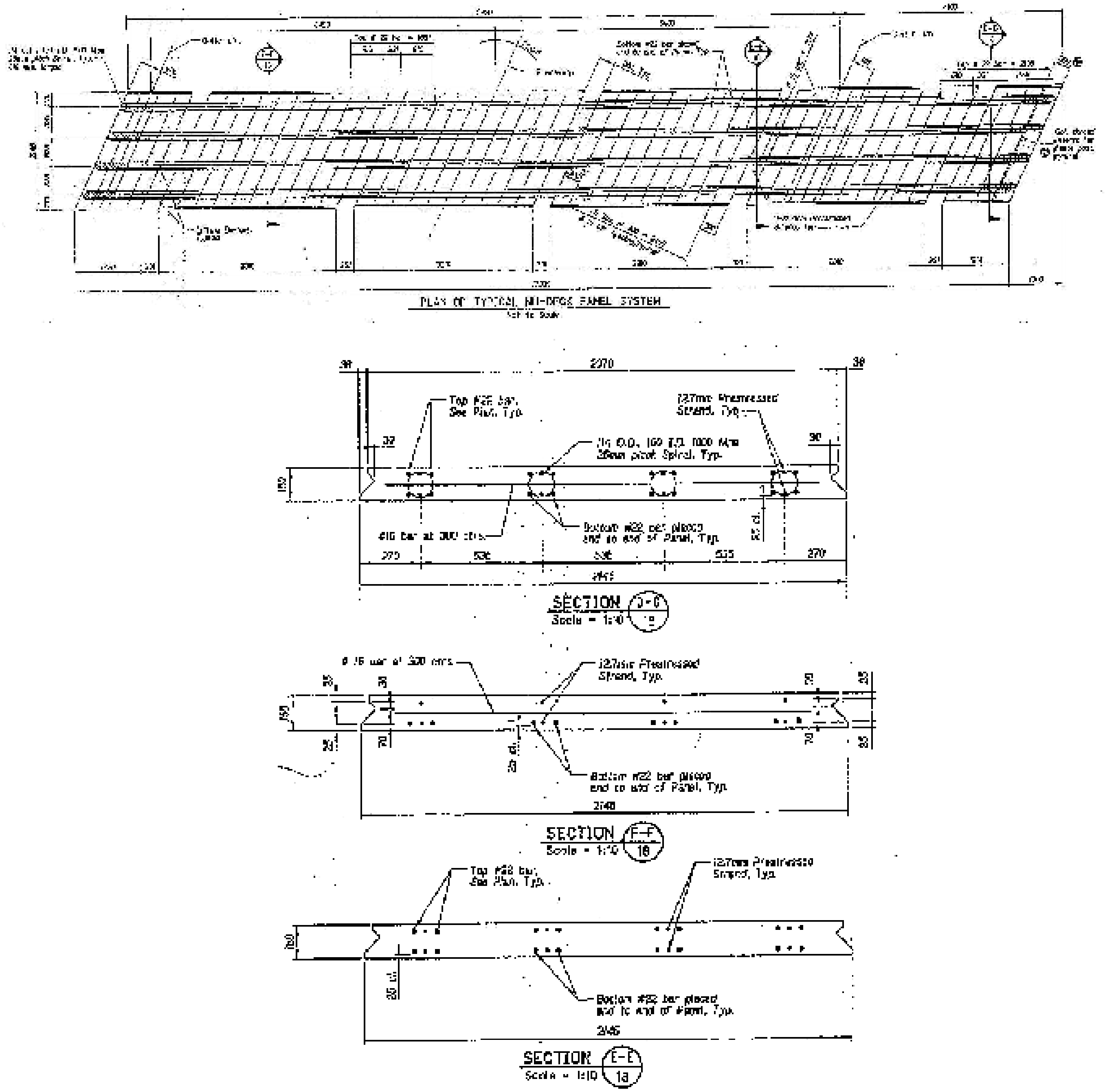


Figure A.2.4.8-4 Plan View and details of a Typical Panel

The edges of the panels were skewed to 25 degrees to match the skewed layout of the bridge. The precast panels were fabricated with a crown to match the road profile. The following technique was used to crown the panels in the precast yard: (1) the forms and prestressing strands were set horizontally, (2) the strands were pretensioned and a pocket was created at the crown point at every strand line (3) concrete was cast and after it reached a strength 4,300 psi (30 MPa), the strands were cut and the forms were removed, (4) the panel was lifted and set on a saddle that had the same cross slope of the bridge and temporary shims were used to support the straight panel, (5) the top strands at the crown points were cut through the pockets and the temporary shims were removed so that the panel rests on the saddle and took the required cross slope, (6) a horizontal steel plate was welded at each pocket to lock the joint. Figure A.2.4.8-5 to A.2.4.8-7 show the panels at the precast yard.

Figure A.2.4.8-5 The NUDECK panel being lifted from the Prestressing Bed

Figure A.2.4.8-6 The NUDECK panel during Shipping

Figure A.2.4.8-7 Storage of the NUDECK Panels in the Precast Yard

The post-tensioning reinforcement consisted of tendons located at every girder line. Each tendon was made of 16- ½ in. (12.7 mm) strands threaded in the gap between the two layers of pretensioned strands, as shown in Figure A.2.4.8-8.

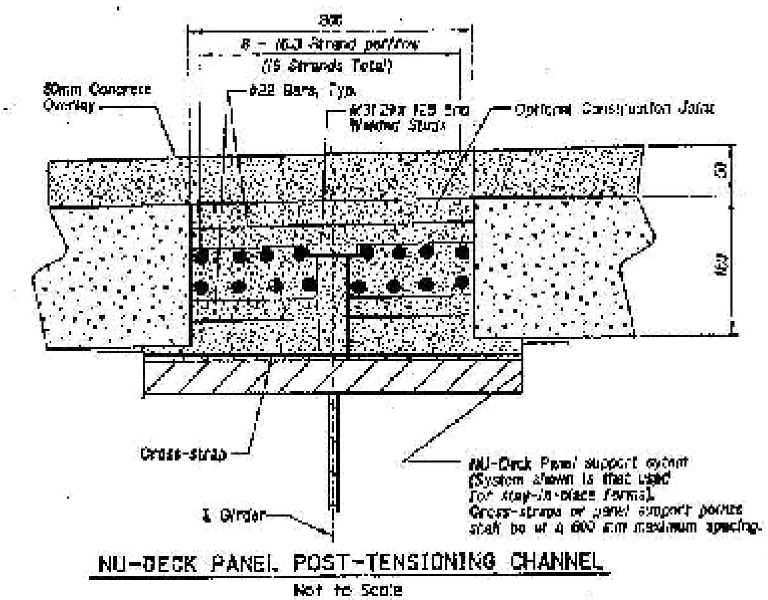


Figure A.2.4.8-8 Cross section of the panel at a girder line

Figure A.2.4.8-9 shows the details of the anchorage devices at the end panel. The anchorage device was made of two vertical side plates and four vertical cross plates. The front cross plate was made curved in order to maximize the space needed to anchor each strand with an individual chuck. As a result of the anchorage device layout, the end panel was pretensioned with an eccentric group of four strands. Therefore, after cutting the strands, the panel bent about an axis normal to its plane resulting in high tensile stresses on one side of the panel. Conventional reinforcement was used to resist these tensile stresses.

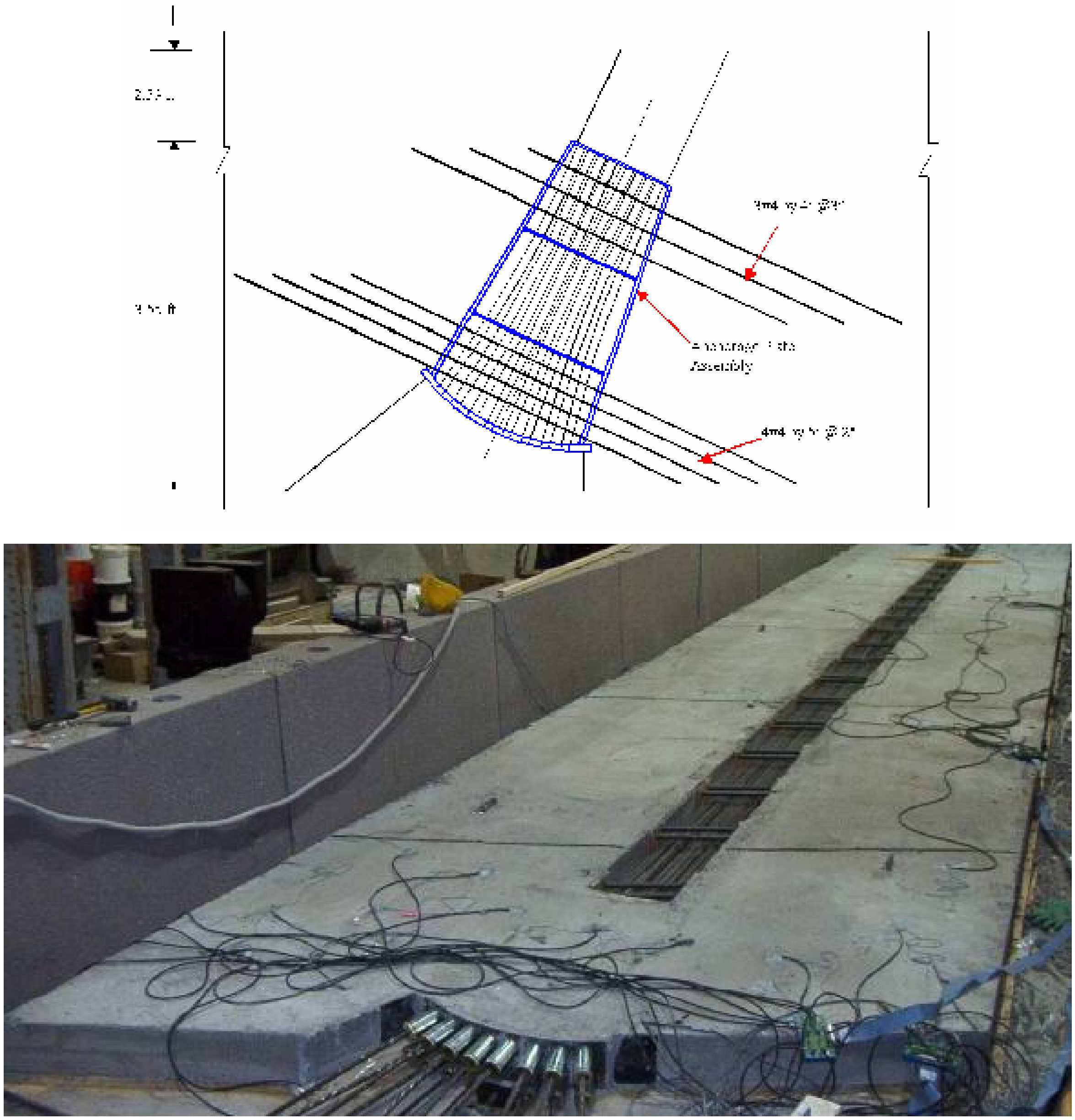


Figure A.2.4.8-9 Details of the end panel with the anchorage device

A.2.4.9 New Hampshire Department of Transportation

New Hampshire Department of Transportation used a full depth conventionally reinforced concrete deck panel system on a simple span through truss bridge. Figure A.2.4.9-1 shows a plan view and cross section elevation of the panel. The super structure is made of eight W18x35 structural steel longitudinal members spaced at 2 ft 9 5/8 in. (854 mm), which were supported by cross-floor beams. The bridge had a total width of 20 ft – 10 in. (6350 mm) and a crown at the center of the roadway.

The precast panel was 8 ft (2438 mm) long and had a variable depth, 3¾ in. (95 mm) at both ends and 5¾ in. (146 mm) at the crown. The panel was conventionally reinforced with one layer of epoxy-coated bars in each direction. The reinforcement was set horizontally parallel to the bottom surface of the panel and at mid height at both ends, as shown in Figure A.2.4.9-2.

The precast deck was made composite with the superstructure by using grouted shear pockets, as shown in Figure A.2.4.9-1. Figure A.2.4.9-3 shows the details of the shear pockets. Three shear pockets were provided over each girder line along the length of the panel and four 7/8 in. (22.2 mm) steel studs were provided in each pocket. The transverse edges of the panel were provided with a female shear key as shown in Figure A.2.4.9-4. Compressed foam rods were used to block the gap between adjacent panels and non-shrink grout was used to fill the shear pockets and the transverse joints. Neither longitudinal post-tensioning nor conventional reinforcement was provided across the transverse joints between panels.

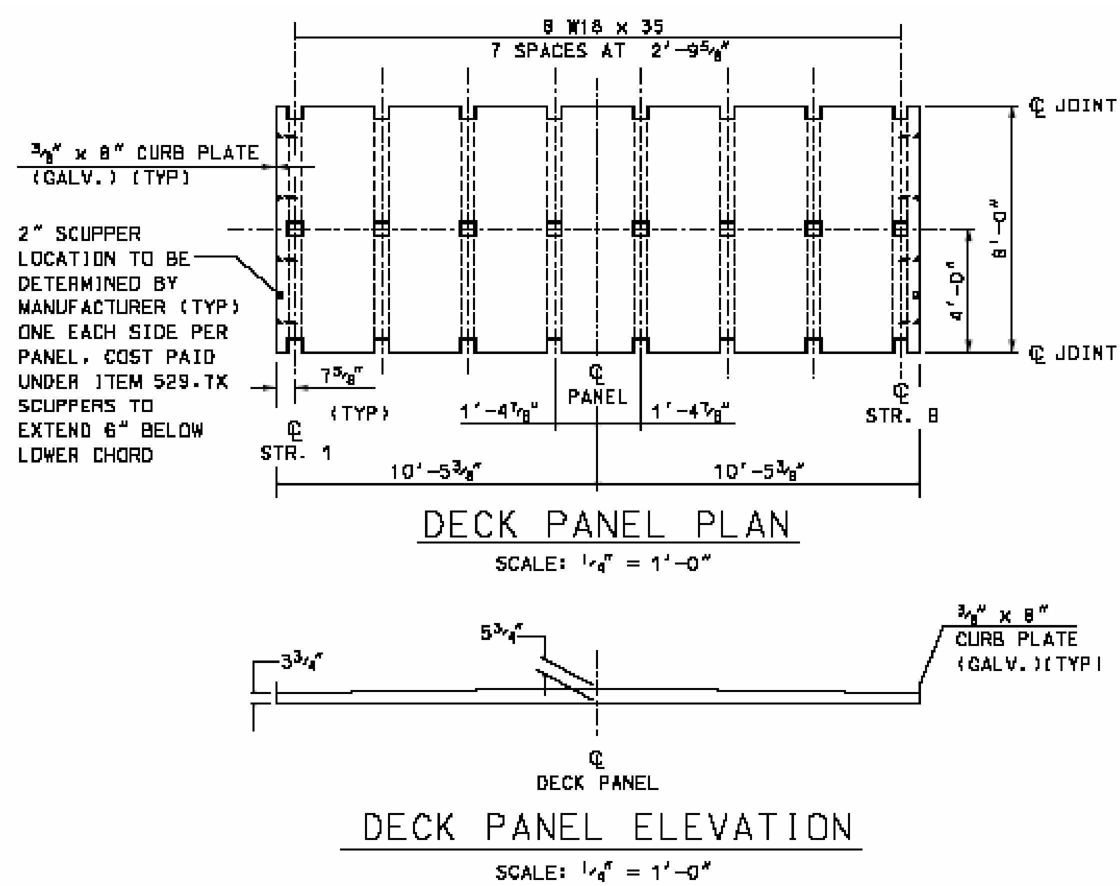


Figure A.2.4.9-1 Plan view and cross-section elevation of the panel

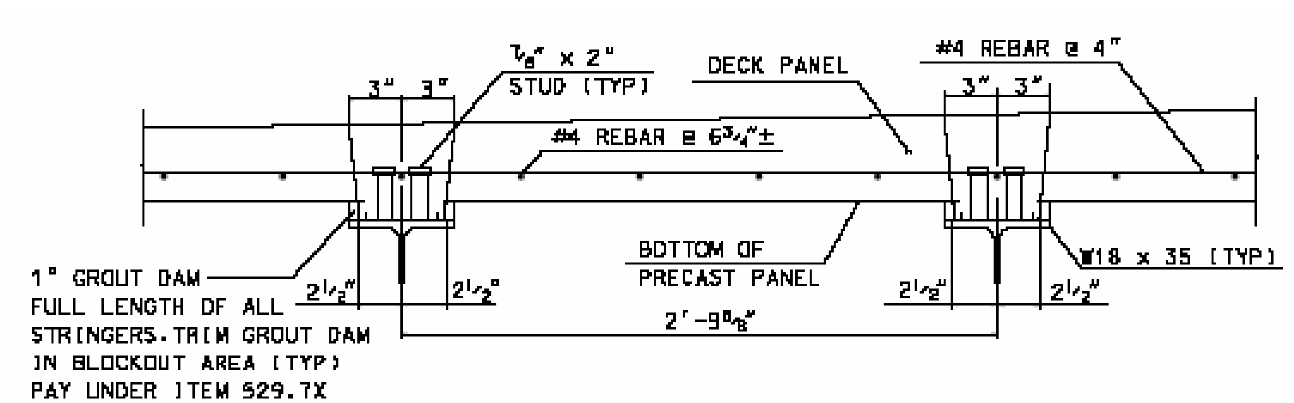


Figure A.2.4.9-2 Cross-section elevation showing details of reinforcement

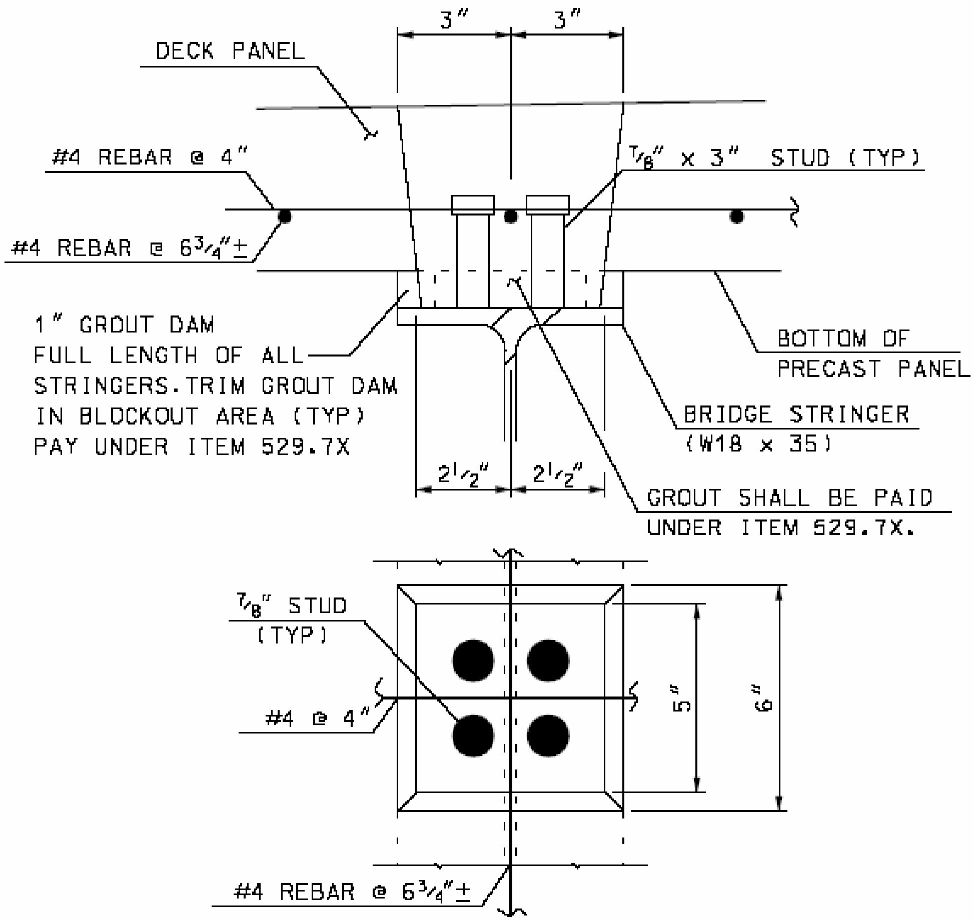


Figure A.2.4.9-3 Details of the grouted shear pockets

Figure A.2.4.9-4 General view of the panels showing the transverse shear keys

In order to attach the steel side rail to the precast panels, a 3/8 x 8 in. (10 x 203 mm) galvanized curb plate was attached vertically to the sides of the panel through four 5/8 in. (16 mm) diameter studs welded to the curb plate. Figure A.2.4.9-5 shows the details of the curb plate.

During shipping of the first group of panels to the bridge site, the panels experienced heavy cracking on the top surface, as shown in Figure A.2.4.9-6. After investigating this issue, it was found that these panels were lifted from two points along the first interior girder lines, which created negative bending moment at these locations (i.e. tensile stresses at top surface). Because the panels had only one layer of reinforcement located close to the bottom surface, the effective depth of the tensile reinforcement was small and the cracks penetrated the top surface of the panel all the way to the tensile reinforcement, as shown in Figure A.2.4.9-7. These panels were rejected and the contractor was asked to lift the panels as a simple span member to avoid applying any negative moments on the panels. No cracks were reported on the successive groups of panels. Figure A.2.4.9-8 shows the bridge after it was open to traffic. A difference in color between the precast concrete and the non-shrink grout could be seen because no overlay was used on top of the precast deck.

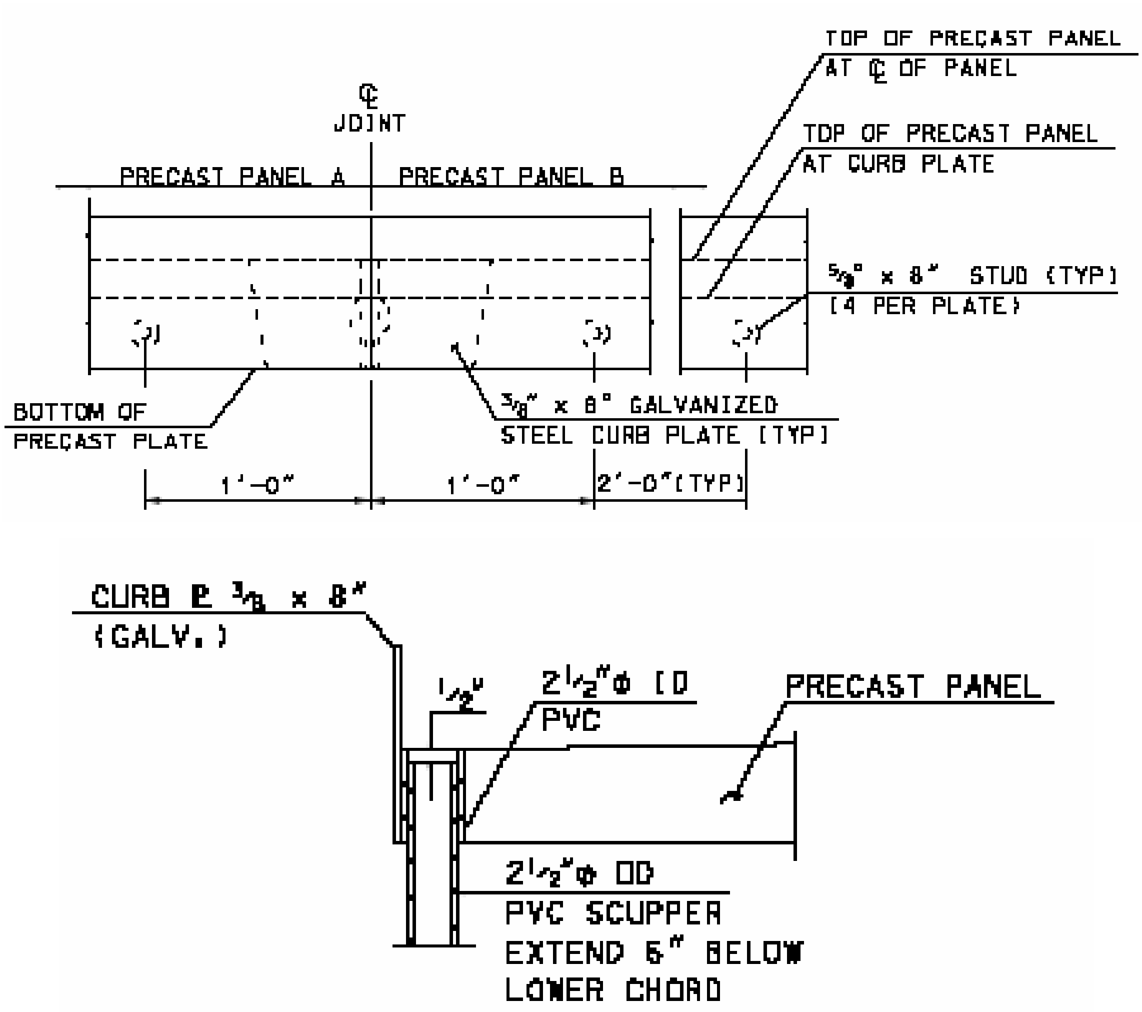


Figure A.2.4.9-5 Details of the curb plate

Figure A.2.4.9-6 Top view of the panel showing the top surface cracks

Figure A.2.4.9-7 Side view of the panel showing the top surface crack penetrating the top surface to the reinforcement level

Figure A.2.4.9-8 General view of the bridge shortly after it was open to traffic

A.2.4.10 New York Department of Transportation

New York department of transportation used full-depth precast panels on a major bridge over the Westchester expressway (I-287). The bridge was a curved with a skew of about 32 degrees. The bridge had 9 spans with total length of 1280 ft (390 m). The bridge had a total width of 125 ft - 7 in (38323 mm). The cross section had two crowns and one downward crown at the centerline of the center stage. The superstructure was made of six longitudinal steel open box girders spaced at 12 ft-8 in. (3860 mm). The cross section of the bridge is shown in Figure A.2.4.10-1. In order to maintain traffic on the bridge during construction, the precast deck was made of three stages across the bridge.

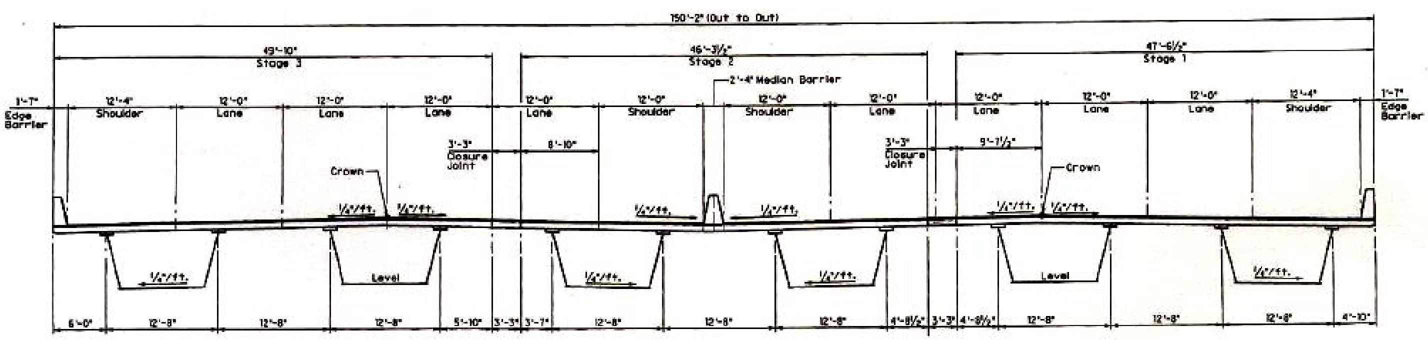


Figure A.2.4.10-1 Cross section of the bridge

The precast panels were 9 in. (229 mm) thick, and conventionally reinforced with two layers of reinforcement in each direction. The conventional reinforcement was designed to resist handling and erection stresses. Each stage of the deck was individually post-tensioned in the transverse direction. The precast panels were made composite with the superstructure using clustered groups of 7/8 in. (22.2 mm) steel studs spaced at 30 in. (762 mm). The steel studs were welded to the top flange of the open steel boxes after a panel was installed. The transverse and longitudinal edges of the panels were provided with a shear key detail. Cast-in-place concrete pours, 3 ft- 3 in. (991 mm) wide, were cast between stages 1 and 2 and stages 2 and 3 to connect various stages. These CIP joints were conventional reinforced by extending the transverse reinforcement of the panels into these joints. Figure A.2.4.10-2 shows details of the precast panel system used for each construction stage. Rectangular precast panels installed normal to the longitudinal girders were used in order to simplify fabrication of the panels. Cast-in-place concrete was used at the abutments to handle the skew profile of the bridge, as shown in Figure A.2.4.10-3.

Zigzag patterned longitudinal post-tensioning system was used because the bridge had a mild curved plan, as shown in Figure A.2.4.10-4. This system enabled the contractor to post-tension each span individually, while maintaining continuity of the deck over the intermediate piers to resist the negative moment resulted from the super imposed live and dead loads. The transverse and longitudinal post-tensioning tendons were made of 4- 6/10 in. (15.2 mm) low relaxation strands placed in flat ducts. The transverse tendons were installed at mid height of the panel and the longitudinal tendons were staggered up and down the transverse tendons as shown in Figure A.2.4.10-5. This arrangement resulted in only axial compression stresses due to the post-tensioning tendons.

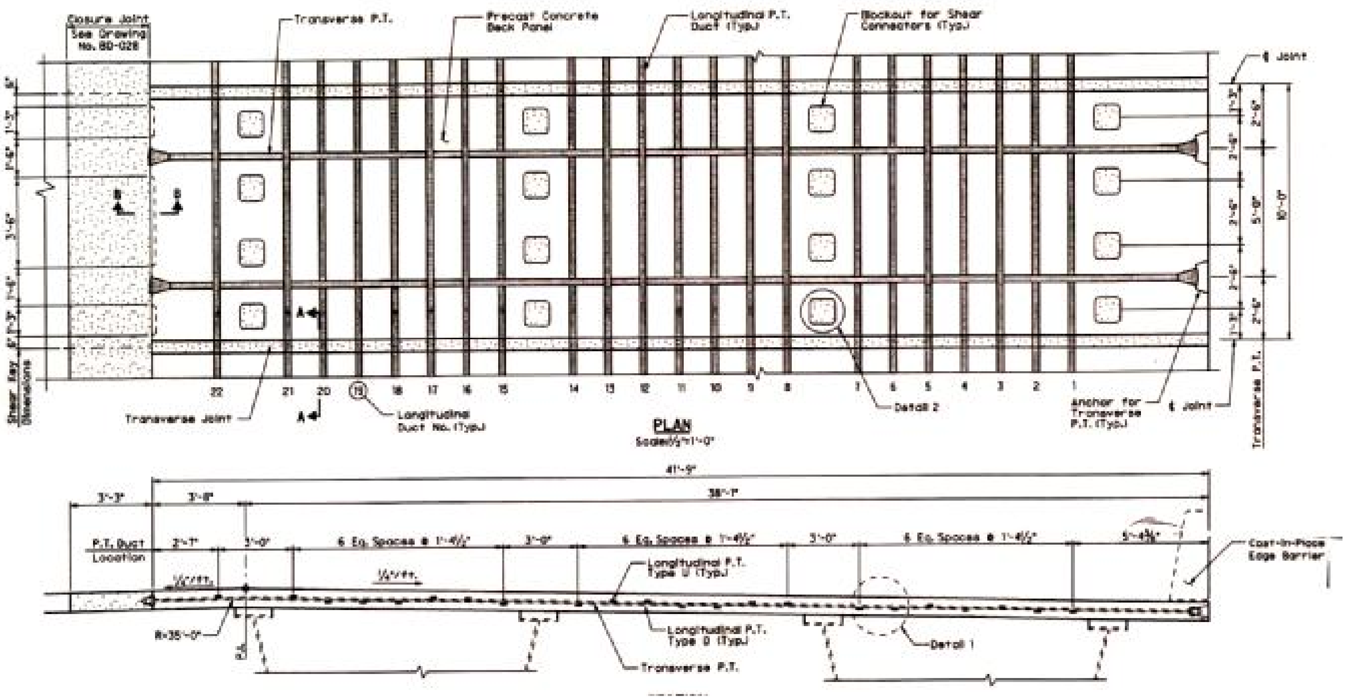


Figure A.2.4.10-2a Details of Stage 1 of the precast panels

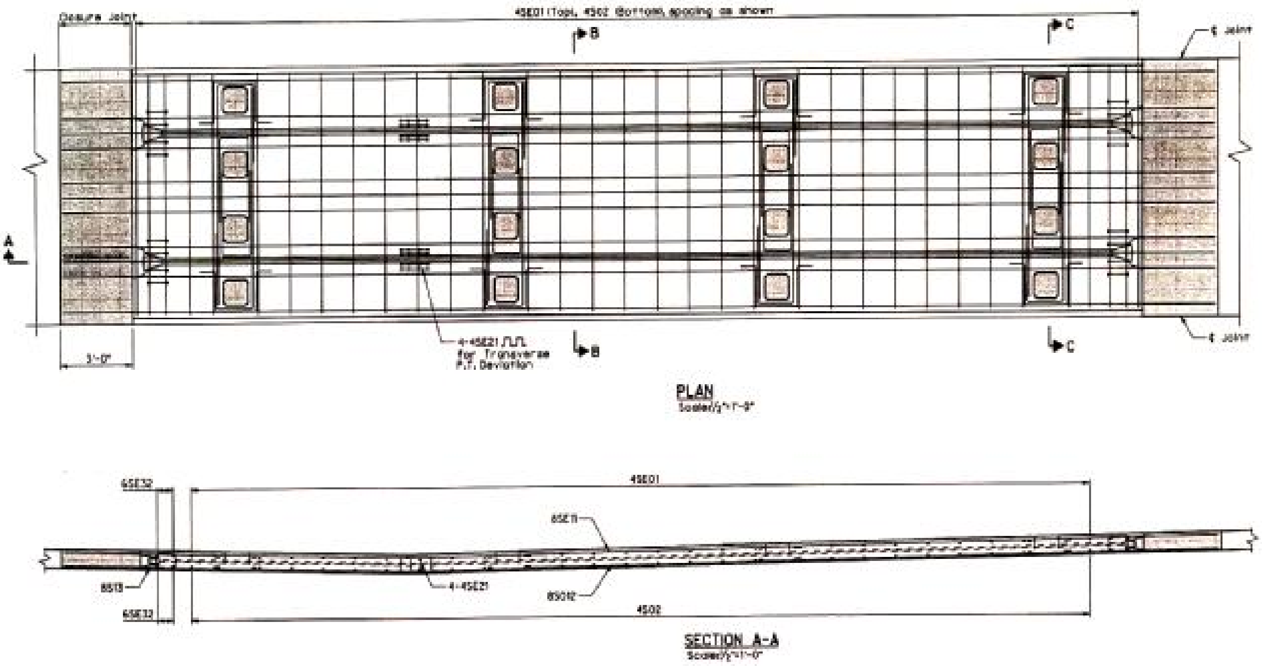


Figure A.2.4.10-2b Details of Stage 2 of the precast panels

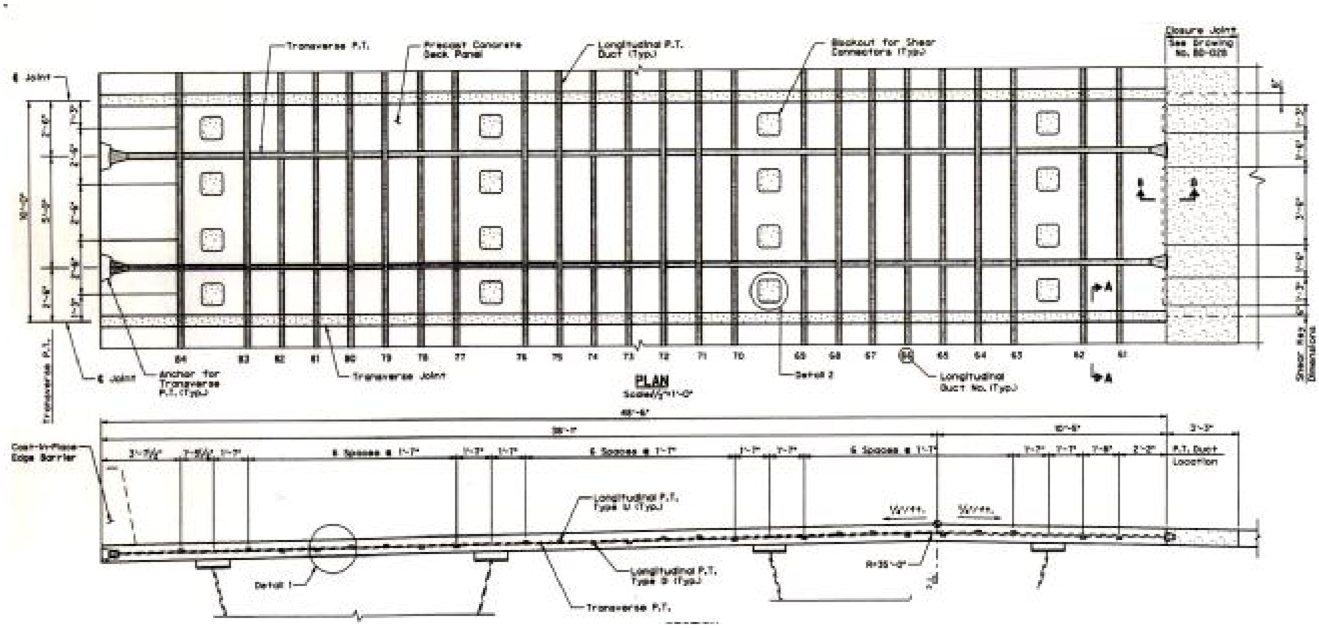


Figure A.2.4.10-2c Details of Stage 3 of the precast panels

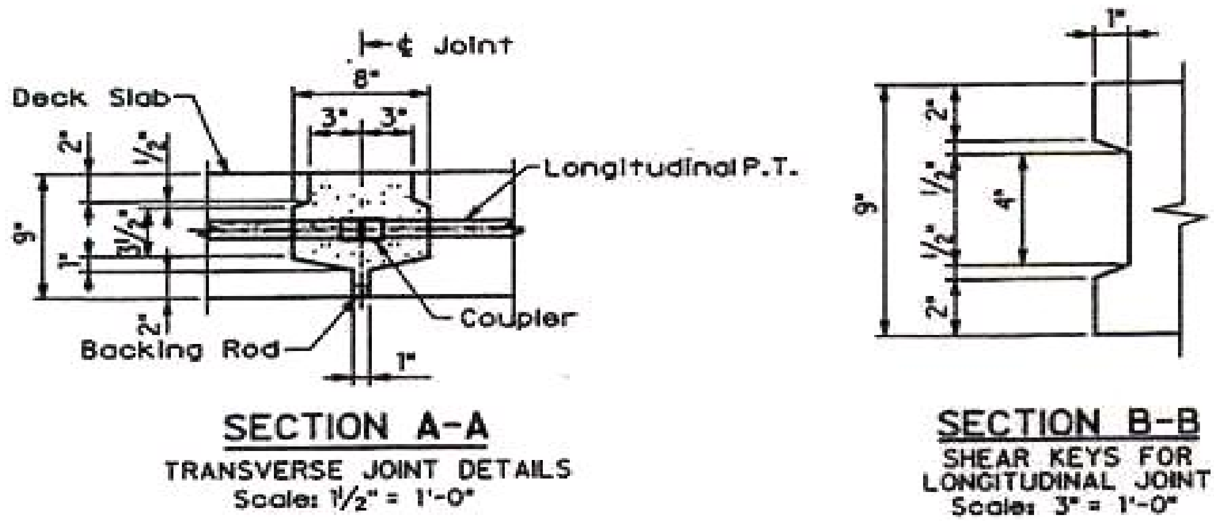


Figure A.2.4.10-2d Sections A-a and B-B of the precast panels

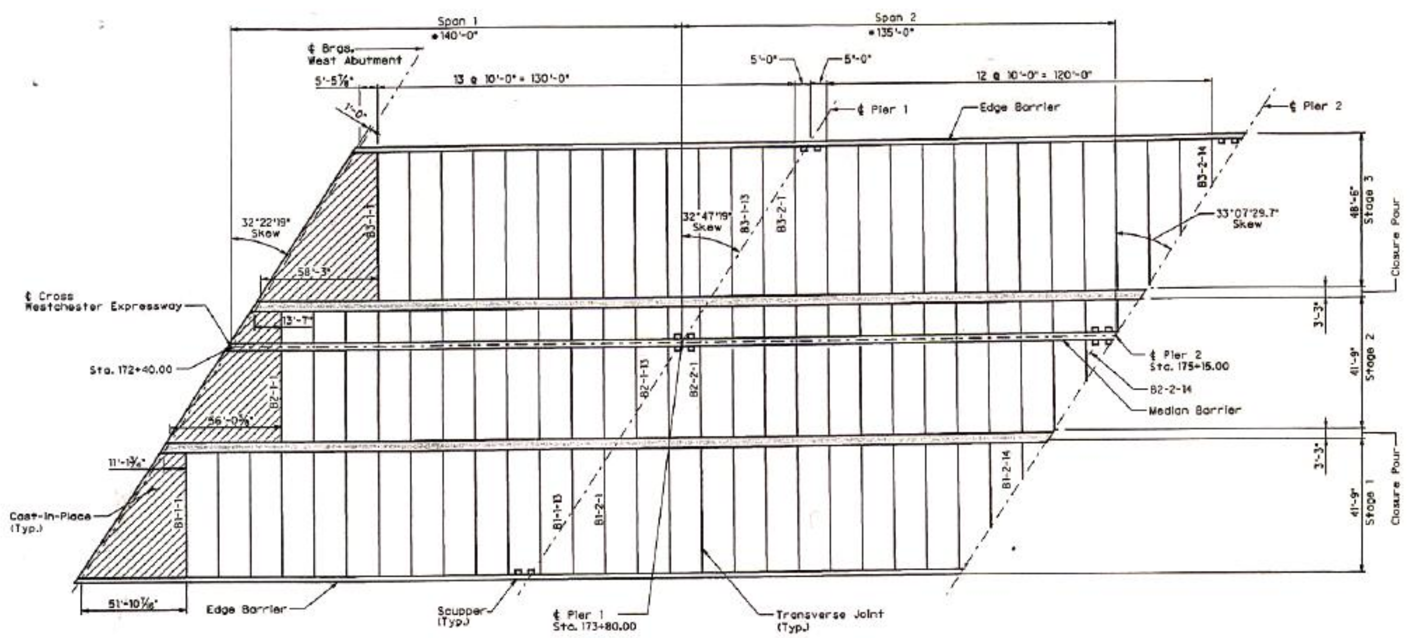


Figure A.2.4.10-3 Cast-in-place approach slab

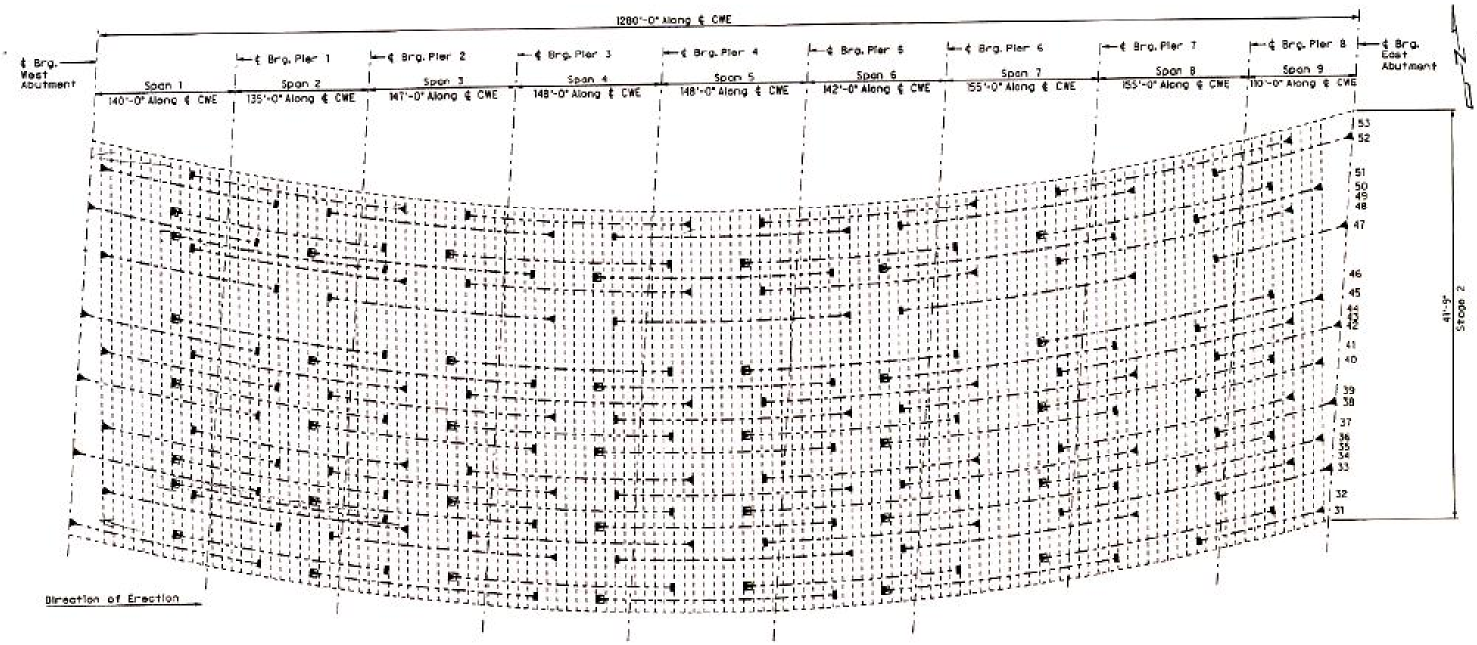


Figure A.2.4.10-4 Typical arrangement of precast panels

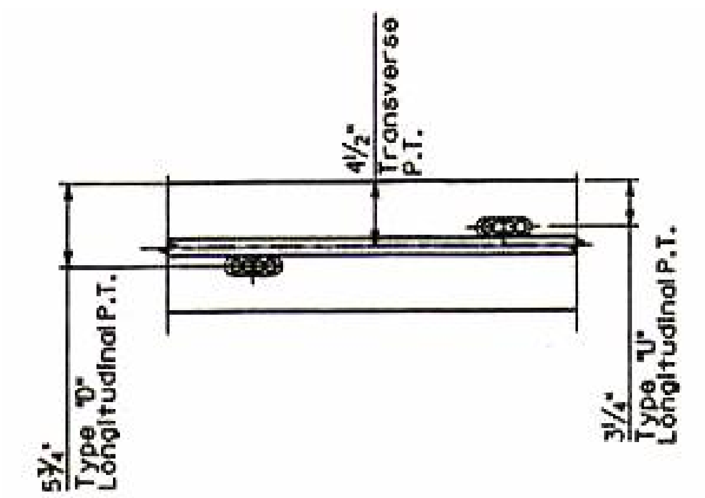


Figure A.2.4.10-5 Arrangement of intersecting transverse and longitudinal post tensioning

The elevation of the precast panels was adjusted by using plastic shim packs as shown in Figure A.2.4.10-6. Lightweight steel angles attached to the top flange of the steel girders.

Figure A.2.4.10-7 shows the details of the hold-down device used to tie the precast panels to the steel girders before applying the post-tensioning force. A hard wood block is installed over a one of the shear pockets and secured by attaching it to a threaded bolt welded to the top flange of the steel girder. Two hold-down devices were used per panel.

Figure A.2.4.10-8 shows the details of the connection between the CIP barriers and the precast panel. Steel couplers were imbedded in the precast panel to couple the barrier reinforcement to the precast panel.

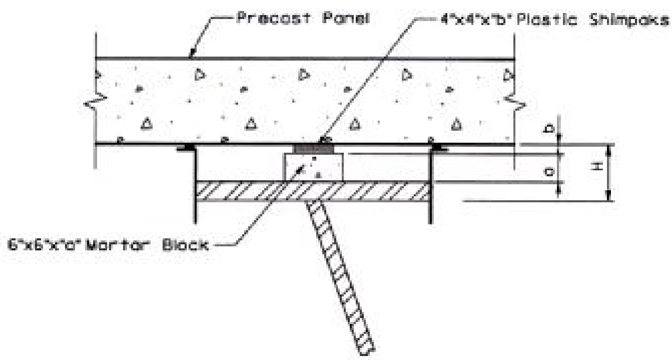


Figure A.2.4.10-6 Details of elevation adjustment and built-up haunch

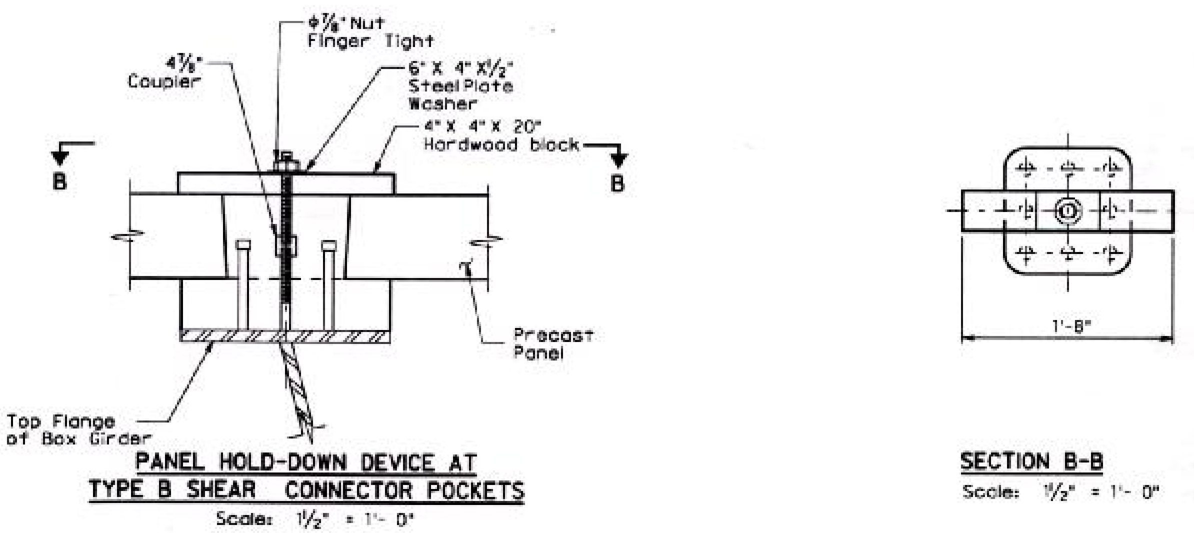


Figure A.2.4.10-7 Details of the hold-down device

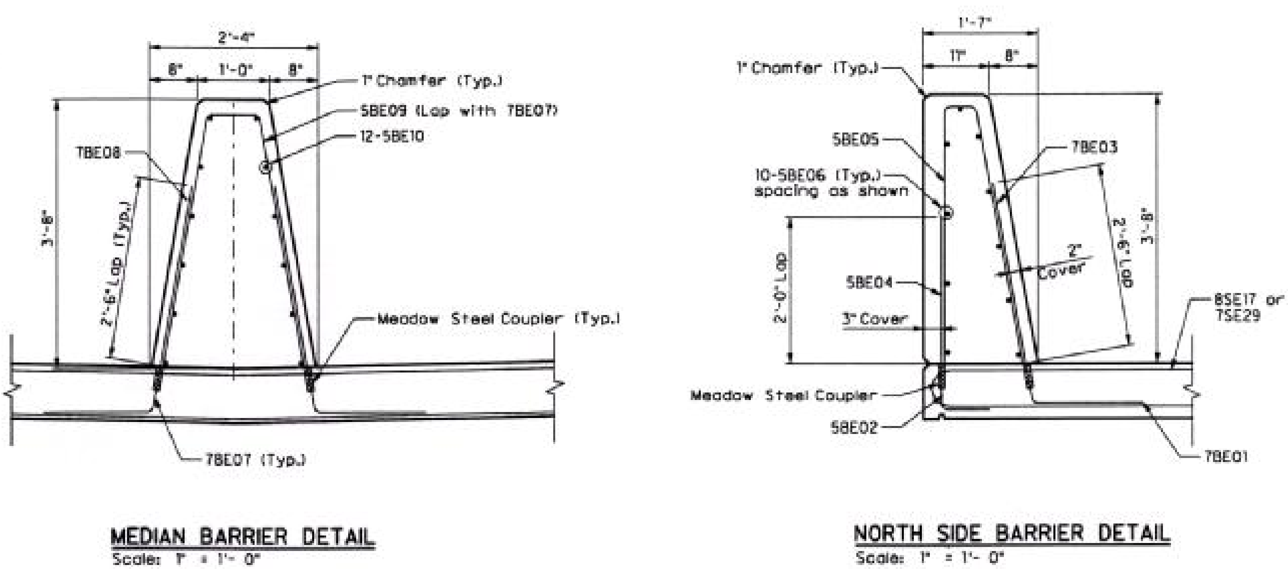


Figure A.2.4.10-8 Details of the connection between the CIP barriers and the precast panel

A.2.4.11 Texas Department of Transportation

Texas Department of Transportation (TxDOT) used full depth precast panels in two projects. These were: (1) the SPUR 326 Bridge at AT&SF Railway and (2) the Hazard, Woodhead, Dunlavy and Mandell Street Tied Arch Bridges.

The SPUR 326 Bridge at AT&SF Railway:

The project had two separate structures. Each structure was divided into three units: (a) a 50-ft simple span, (b) a four span 290 ft (88392 mm) continuous unit and (c) a three-span 205 ft (62484 mm) continuous unit. The original structure included a 33 ft (10058 mm) wide and 6.5 in (165 mm) thick cast-in-place slab supported on four longitudinal steel girders spaced at 8 ft (2439 mm).

In 1989, due to signs of early deck deterioration and the necessity to widen the roadway width, the deck slab was replaced and the roadway width was increased by adding two steel girders spaced at 7 ft (2133 mm), as shown in Figure A.2.4.11-1 and A.2.4.11-2.

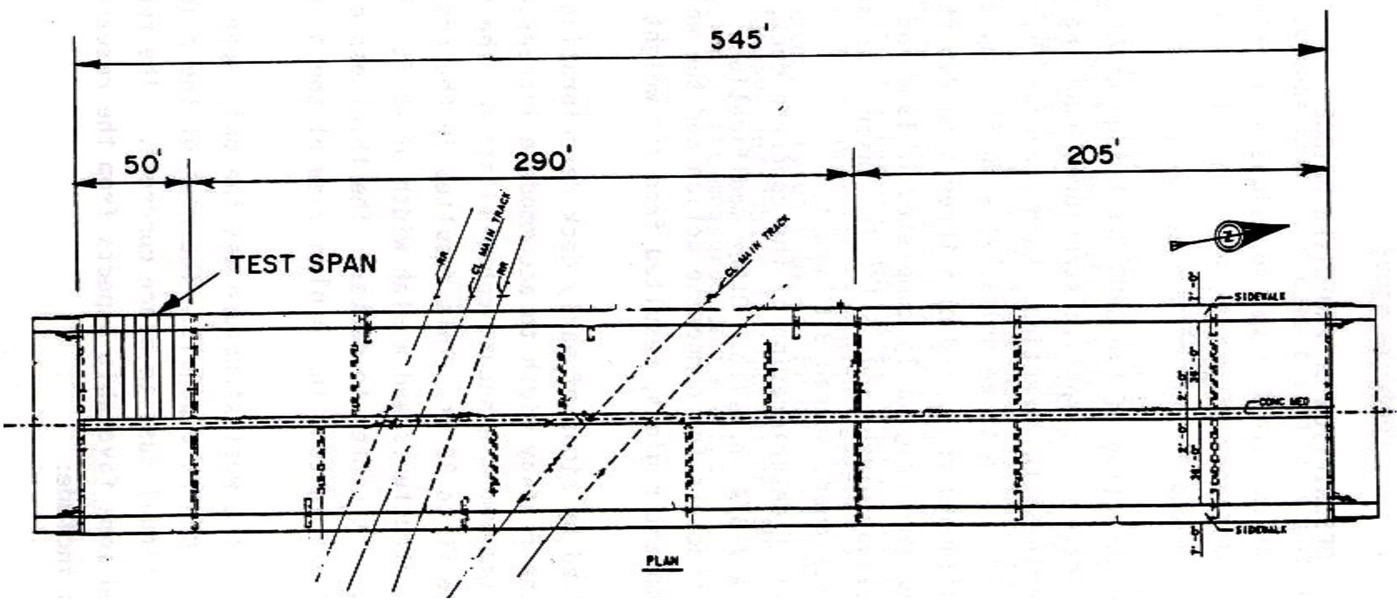


Figure A.2.4.11-1 Plan view of the structure

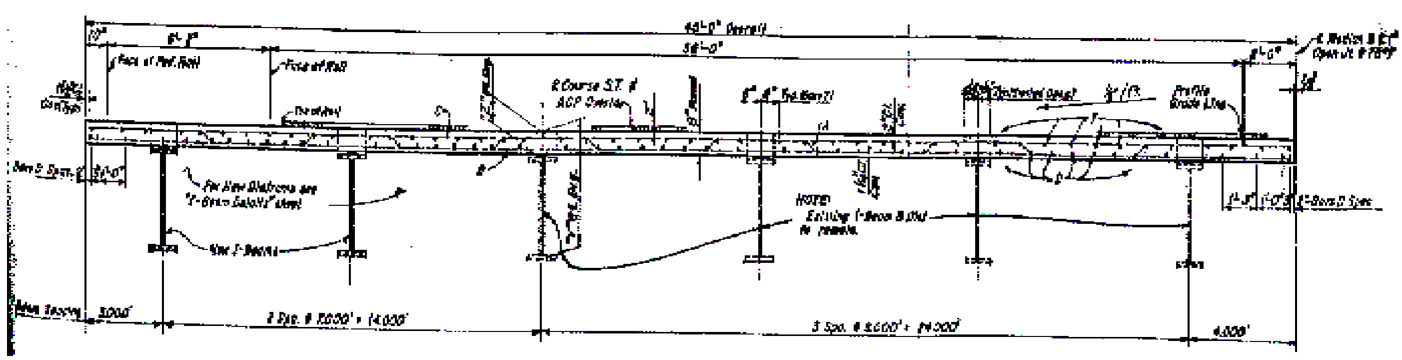


Figure A.2.4.11-2 Cross section of the bridge

The 50-ft span of the west structure was the only span that was redecked using precast concrete panels made composite with the supporting girders. Prismatic 8 in (202 mm) thick conventionally reinforced precast panels were used. Figure A.2.4.11-3 shows a plan view of the panel. Three blockout holes were provided per panel per girder to accommodate the shear connectors. A female type shear keys were created at the transverse edges of the panels, as shown in Figure A.2.4.11-4. The shear connector blockouts and the transverse joints were filled with epoxy mortar from the top surface of the panels. The epoxy grout had a minimum sand-epoxy ratio of three and a minimum compressive strength of 5,000 psi (34.5 MPa) and 7,000 psi (48.3 MPa) at 24 hours and 7 days, respectively. The bottom face of the shear key was sealed using ½ in. (12.7 mm) thick by 2 in. (50 mm) wide strips of wood. The strips were set in place from the bottom of the bridge and tied from the top. The panels were fabricated and cast next to the bridge span.

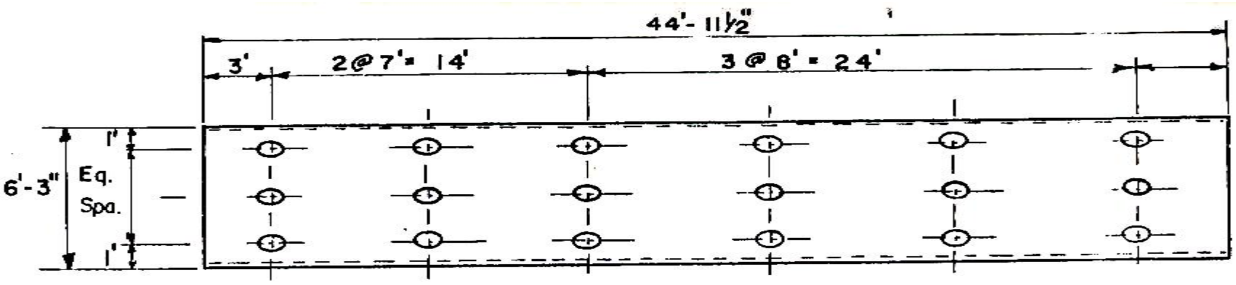


Figure A.2.4.11-3 Plan view of the precast panel

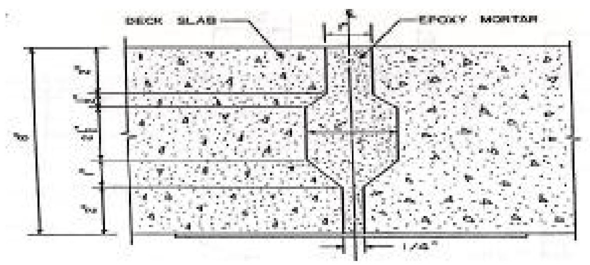


Figure A.2.4.11-4 Shear key details

To adjust the elevation of the panels on the steel girders, shims and neoprene bearing pads were placed on top of the steel girders, as shown in Figure A.2.4.11-5. Galvanized angles, fastened to the precast panels, were used as grout barriers. The gap between the galvanized angles and the steel beams were sealed using heavy-duty tape. This detail did not perform well and many points experienced severe leakage during grouting.

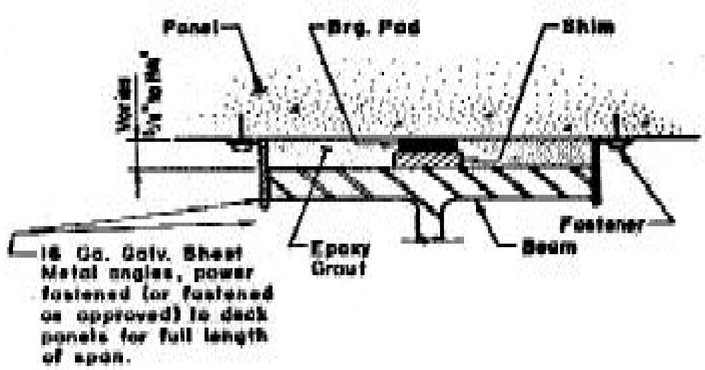


Figure A.2.4.11-5 Cross section of the deck at a girder line showing the details of shimming and grout barriers

US 59: Hazard, Woodhead, Dunlavy and Mandell Street Tied Arch Bridges

Four bridges were replaced as a part of the widening project on US 59 route. A steel tied arch system with clear span of 224 ft (68275 mm) was used for the replacement project. The arch system consisted of two arches, set 45 ft (13716 mm) apart, fabricated from steel plates and braced with rectangular structural tubing. Total width of the bridge was 60 ft (18288 mm).

Full depth precast concrete deck panel system, transversely pretensioned and longitudinally post-tensioned, was used. The precast panels were suspended from the tie of the arch using bolts. Figure A.2.4.11-6 shows an elevation profile of the precast panel. The panel dimensions were 60 ft (18288 mm) wide, and 7 ft (2134 mm) long. Because the roadway had a crown at its centerline, the precast panels were crowned at the centerline of the roadway by increasing the thickness of the panel. The variable thickness helps to optimize the self-weight of the panel and provide for the depth needed for the positive moment section at centerline of the roadway.

Figure A.2.4.11-7 shows the cross section of the panel. The panel was reinforced with two layers of pretensioned strands. The bottom layer of strands was split into three groups (one center group and two edge groups) to avoid interference with the bolts used to the panel with the ties of the arch. The top layer of strands was provided to resist the negative moment of the overhangs.

No shear key was provided at the transverse edges of the panels. However, the vertical surface of the transverse edges was roughened during fabrication of the panels. This was achieved by painting the side forms with a retarding agent and washing the panels edges with high-pressure water, which resulted in aggregate exposed uniformly roughened surface. A 3-in. (76 mm) wide gap was provided between adjacent panels and wood forming from under the deck was used to bridge the gap during placement of the grout. See section D.6 of this report for more details.

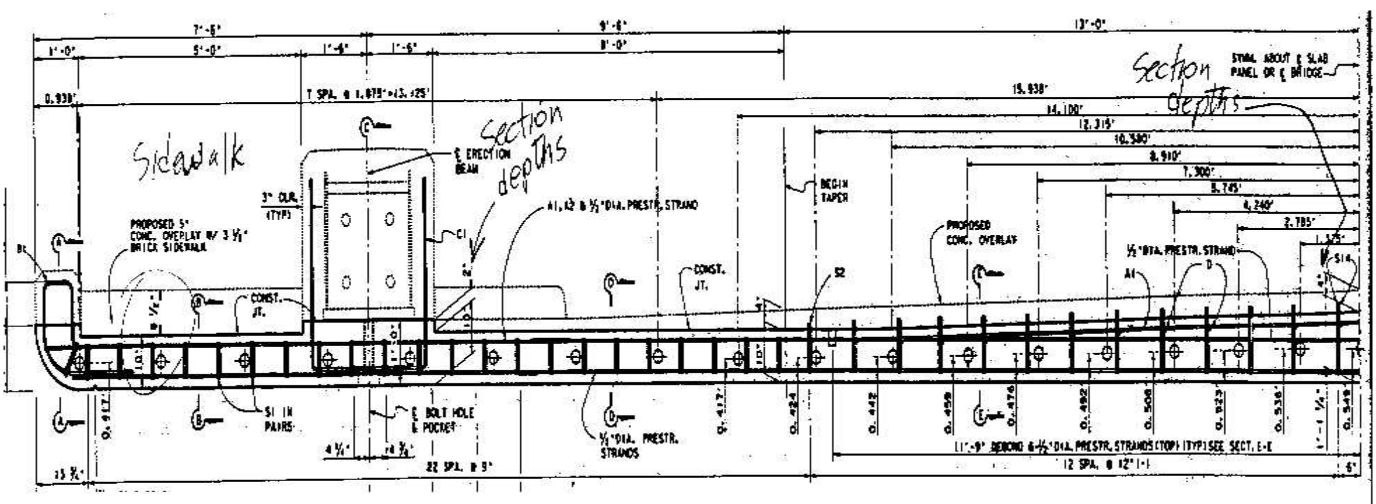


Figure A.2.4.11-6 Half cross section of the arch tied bridge

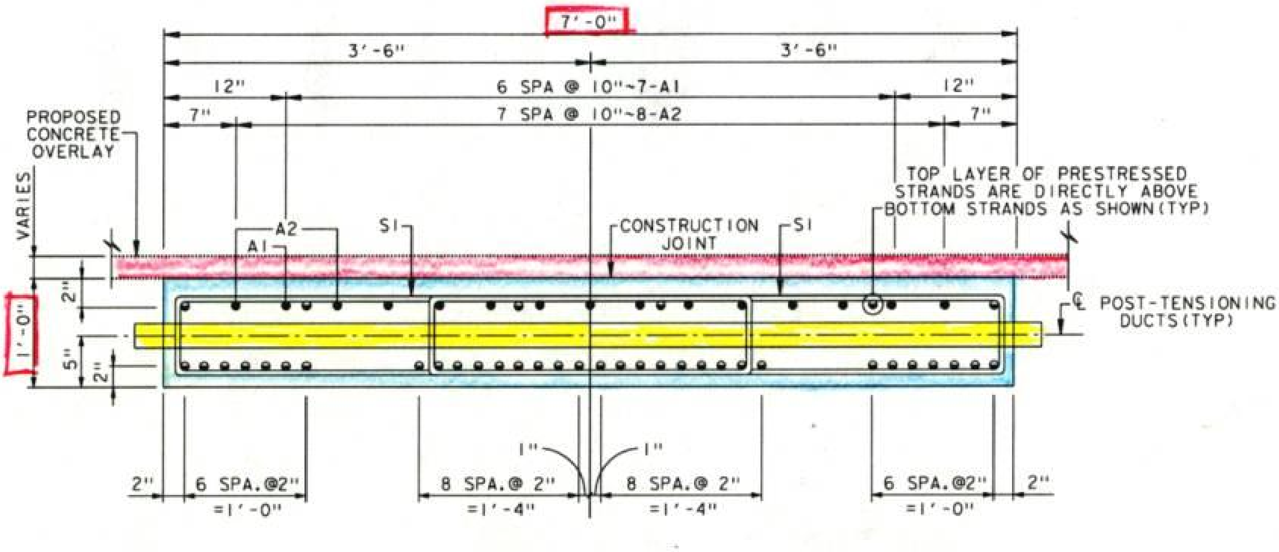


Figure A.2.4.11-7 Cross section of the precast deck of the arch tied bridge

A 4-inch (102 mm) thick composite concrete overlay was used over the precast panels to solve the problem of differential camber in the prestressed panels set side by side. One layer of conventional reinforcement was provided in the overlay, as shown in Figure D.4.11-8.

Figure A.2.4.11-8 Reinforcement of the cast-in-place topping of the arch tied bridge

A.2.4.12 Utah Department of Transportation

Utah Department of Transportation has recently decided to use a full depth precast concrete deck panel system for the rehabilitation project of the deck of the C-437 of the County Road over I-80 to Wanship. The bridge has four continuous spans 42.5, 81.5, 81.5 and 42.5 ft (12954, 24841, 24841 and 12954 mm). The existing superstructure is made of four steel plate girders spaced at 9 ft – 10 in. (2997 mm). The total width of the bridge is 35 ft – 7½ in. (10859 mm). The bridge has a crown at the centerline of the road with a 2 percent cross slope both ways. The bridge has a 45-degree skew angle. Figure A.2.4.12-1 shows a plan view and a cross section elevation of the bridge.

Two straight precast panels are used across the width of the bridge. The precast panels have a ¼ in. (6 mm) concrete grinding allowance for correcting uneven roadway surface at transverse and longitudinal joints between panels and between panels and the end of the bridge deck. A polymer overlay is provided after grinding is complete. A 3 ft – 4 in. wide gap filled with cast-in-place concrete is created at the crown to connect the panels.

A typical rectangular 8¼ in. (210 mm) thick panel is used across the bridge except at the abutments where a skewed panel is used to accommodate the skew angle of the bridge. Figure A.2.4.12-2 shows a plan view of a typical precast panel. The dimensions of the typical panels are chosen to minimize the number of the panels shipped and installed on the bridge and to avoid splicing two adjacent panels over the piers, as shown in Figure A.2.4.12-1. This arrangement has resulted in using 12 typical panels and four end panels.

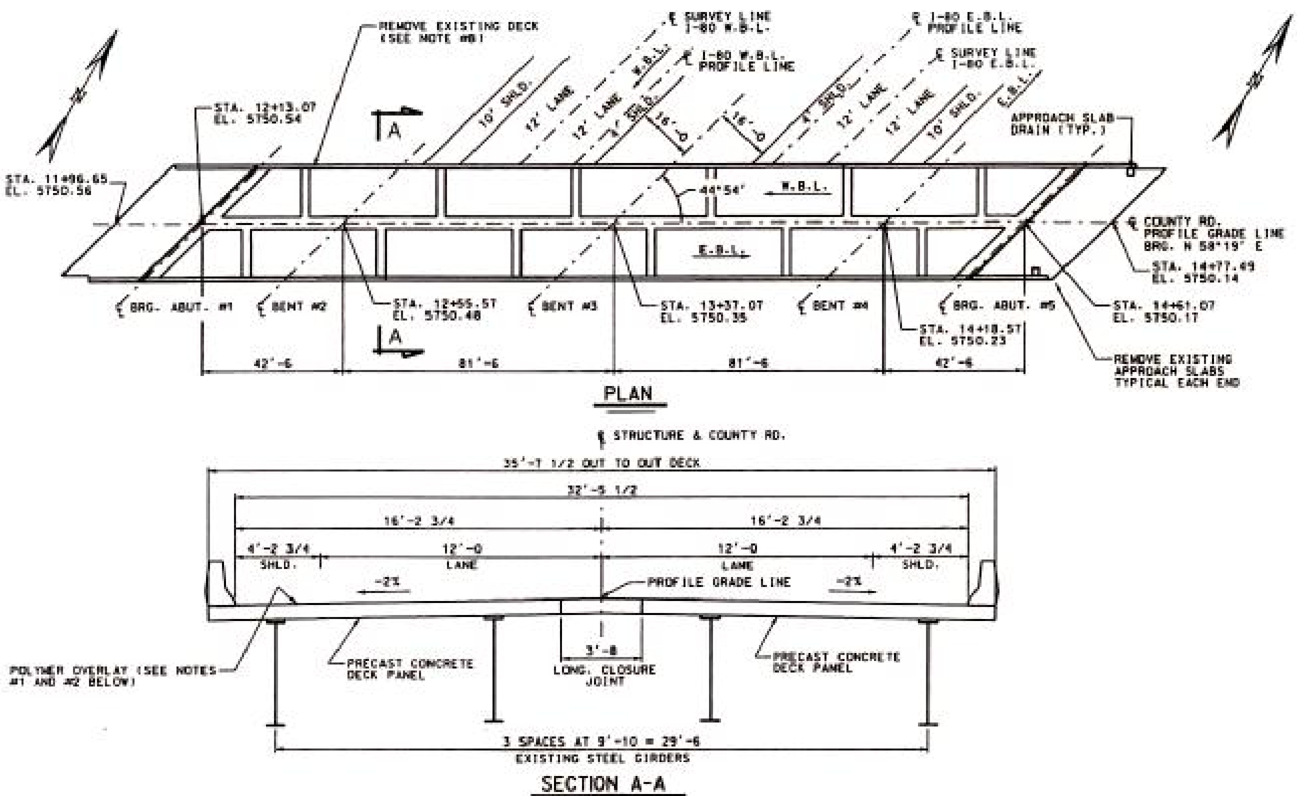


Figure A.2.4.12-1 Plan view and cross section elevation of the C-437 structure over I-80 to Wanship

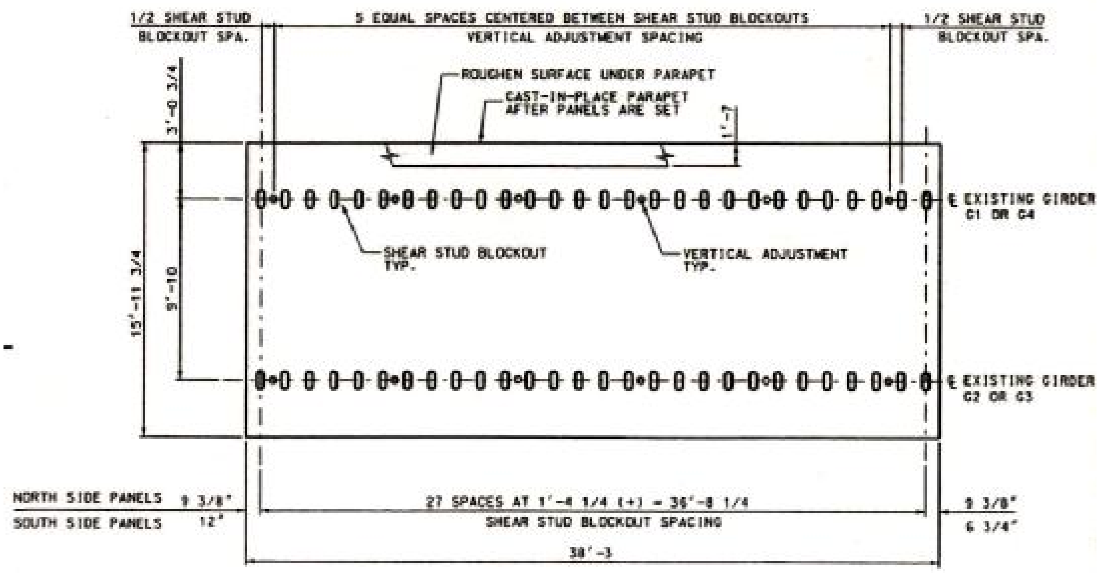


Figure A.2.4.12-2 Plan view of a typical precast panel

The precast panels are conventionally reinforced with two layers of epoxy coated steel bars in each direction, as shown in Figure A.2.4.12-3. The longitudinal reinforcement is designed to resist the negative moment over the piers resulted from the superimposed dead and live loads applied after the deck is made composite with the superstructure.

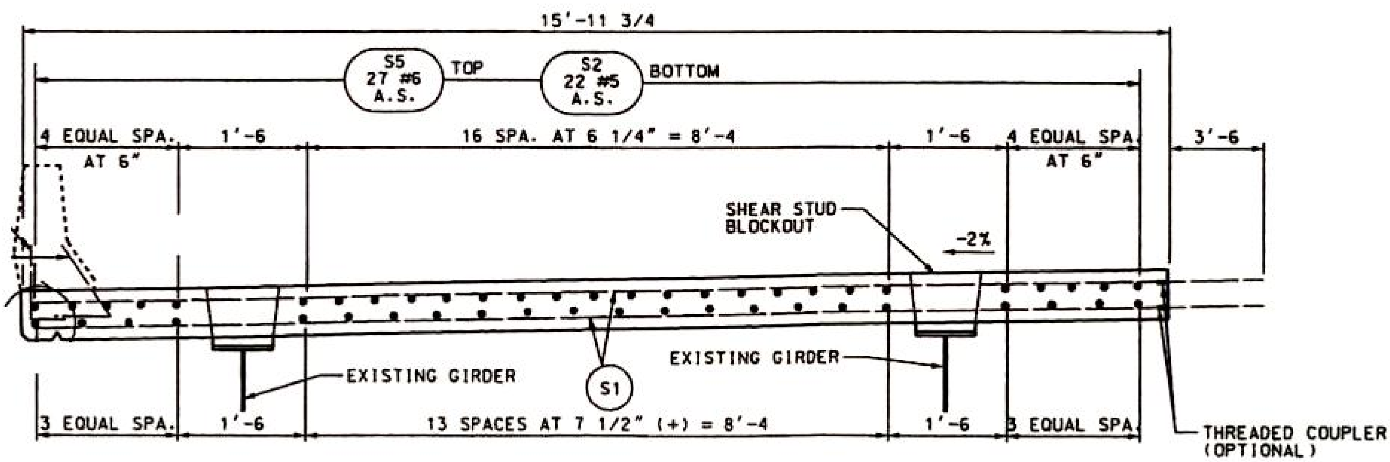


Figure A.2.4.12-3 Cross-section elevation of a typical precast panel

The precast deck is made composite with the superstructure by using grouted shear pockets spaced at 16¼ in. (413 mm) along the length of the panel. Each shear pocket accommodates three 7/8 in. (22.2 mm) steel studs, which are welded to the top flange of the steel girders after the panels are installed. Figure A.2.4.12-4 shows the details of the shear pockets.

Twelve vertical adjustment devices are used per each panel. The vertical adjustment device is made of a coil insert and a 1 in. (25.4 mm) diameter bolt as shown in Figure A.2.4.12-5. After the level of a panel is adjusted, each bolts is cut in a 2-in. (50 mm) deep recess created on the top surface of the panel. Then the shear pockets and recess is filled with non-shrink grout.

To connect the panels at the crown, the transverse reinforcement of the panel extended outside the panel for a distance of 42 in. (1067 mm), as shown in Figure A.2.4.12-6. Also, the plans of the bridge show that the designer allows the use of threaded coupler as an alternative. Figure A.2.4.12-6 shows also various details of transverse and longitudinal joints between panels. It is worthy to note that no shear keys are provided at the transverse or longitudinal edges of the panel and no roughening requirement is specified for these edges.

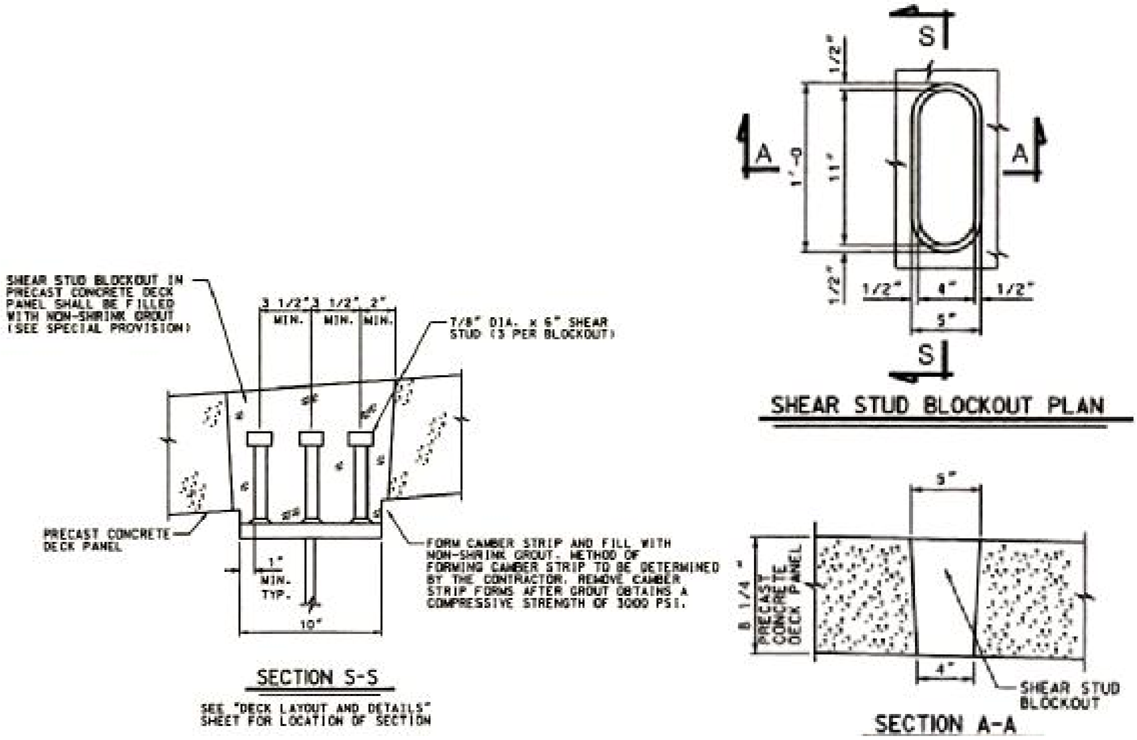


Figure A.2.4.12-4 Details of the shear pockets

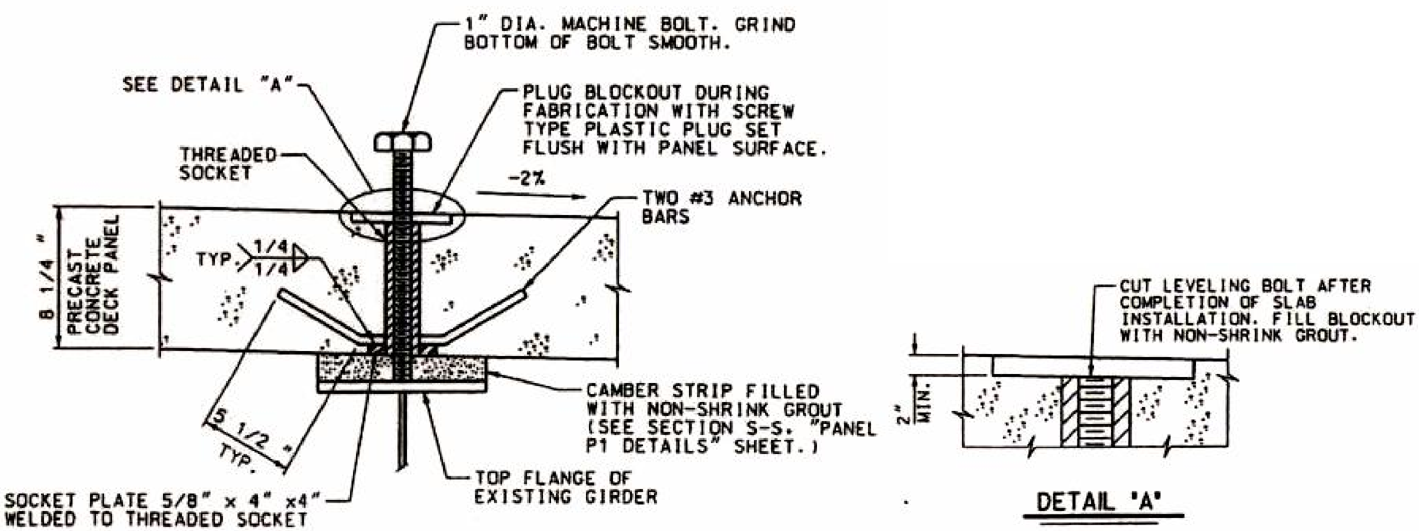


Figure A.2.4.12-5 Details of the vertical adjustment screw

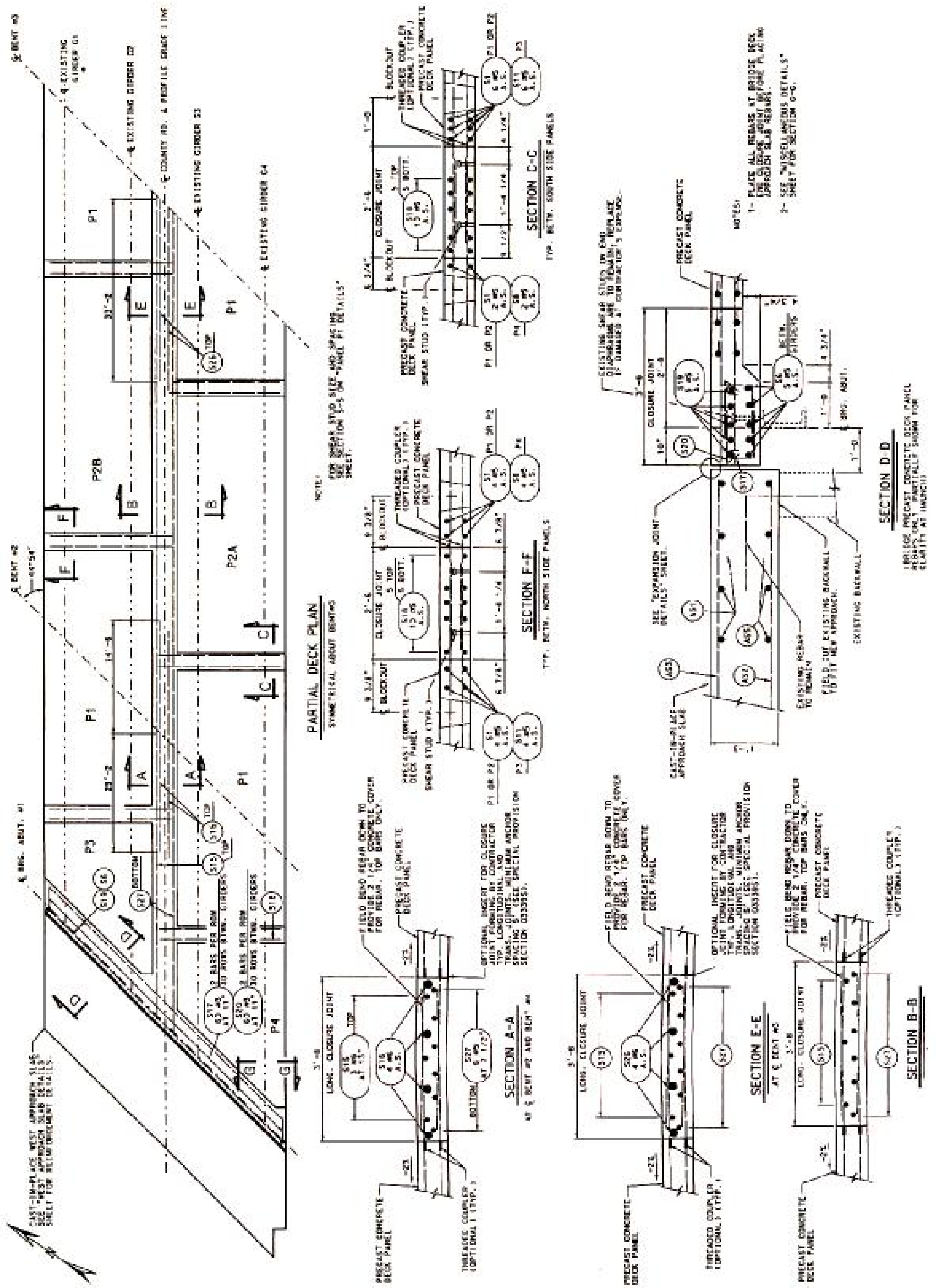


Figure A.2.4.12-6 Details of the transverse and longitudinal joints between the precast panels

A.2.4.13 Virginia Department of Transportation

Route 7 over Route 50, Fairfax County, Virginia (***18***) consisted of two structures, Route 7 Eastbound & Route 7 Westbound. These two structures were part of four structures at the Route 7 Route 50 Interchange built by the Virginia Department of Transportation in 1950's (Figure A.2.4.13-1). The structures were single span, steel plate girder bridges with composite concrete decks. The eastbound bridge was 138' long and 50’ wide. The westbound bridge was 110' long and 49’ wide. After about 40 years of service, the concrete decks had deteriorated extensively and needed frequent repair. Repair and maintenance of the bridge decks was a major problem due to the congested traffic at the interchange. The interchange is a heavily trafficked area in the Metropolitan Washington. The Eastbound carried four traffic lanes towards Washington, D.C. and the Westbound carries three traffic lanes towards Tyson’s Corner, Virginia. Any traffic disturbance on these bridges would create a chain reaction in the whole interchange causing significant traffic delays. Therefore, the Virginia Department of Transportation planned to replace the bridge decks.

Although a nighttime construction of the precast deck alternate would cost about $250,000 more than using cast-in-place (CIP) concrete deck with full closure of the bridge, it would save about $2,000,000 for the community in terms of user cost. Construction began in September of 1999 and was successfully completed in about 1 1/2 months.

The construction requirements mandated that construction operations should be conducted in such a manner that all lanes on the bridges are open to traffic from 5:00 a.m. to 9:00 p.m. and during construction (i.e., 9:00 p.m. to 5:00 a.m.) the bridges should be partially open to traffic at all times. Therefore, the existing composite concrete deck was replaced in three stages. In each stage, a portion of the transverse section was removed and replaced the full length of the bridge, while two traffic lanes were allowed on the bridge during the replacement, as shown in Figure A.2.4.13-1.

A total of 18 different panel shapes were designed to fit the three construction stages and the skewed ends of the deck slab, while conforming to the weight limit for transportability and constructibility (10 tons). A typical panel was 10 ft (3048 mm) in the longitudinal direction of the bridge (direction of traffic) and its width varied depending on the stage of construction. Lightweight concrete was used in fabrication of the panels to compensate for the additional weight of the overlay. The concrete compressive strength was 5,000 psi (34.5 MPa). Panel reinforcement was designed based on the AASHTO method of concrete slab design with the main reinforcement perpendicular to the direction of traffic. Typically, epoxy-coated, #5 (M16) bars were used in both transverse and longitudinal directions. The reinforcement density in the transverse direction was approximately #5 (M16) bar at 6 in. (152 mm) at the top and bottom layers. The reinforcement density in the longitudinal direction was approximately #5 (M16) bar at 14 in. (356 mm) at the top layer and #5 (M16) bar at 7 in. (178 mm) at the bottom layer.

The panel elevation was adjusted by a leveling bolt system**.** Each panel had 4 bolts threaded through cast-in-place sockets. These bolts temporarily bear on the existing girders and were adjusted by a wrench. After positioning the panel, the haunch between the panel and girder was built with a high-early-strength concrete. The high-early-strength concrete was a latex-modified concrete that gains 3,000 psi (20.7 MPa) compressive strength in 3 hours sufficient to allow traffic on the bridge. The strength in 24 hours is 6,000 psi (41.4 MPa).

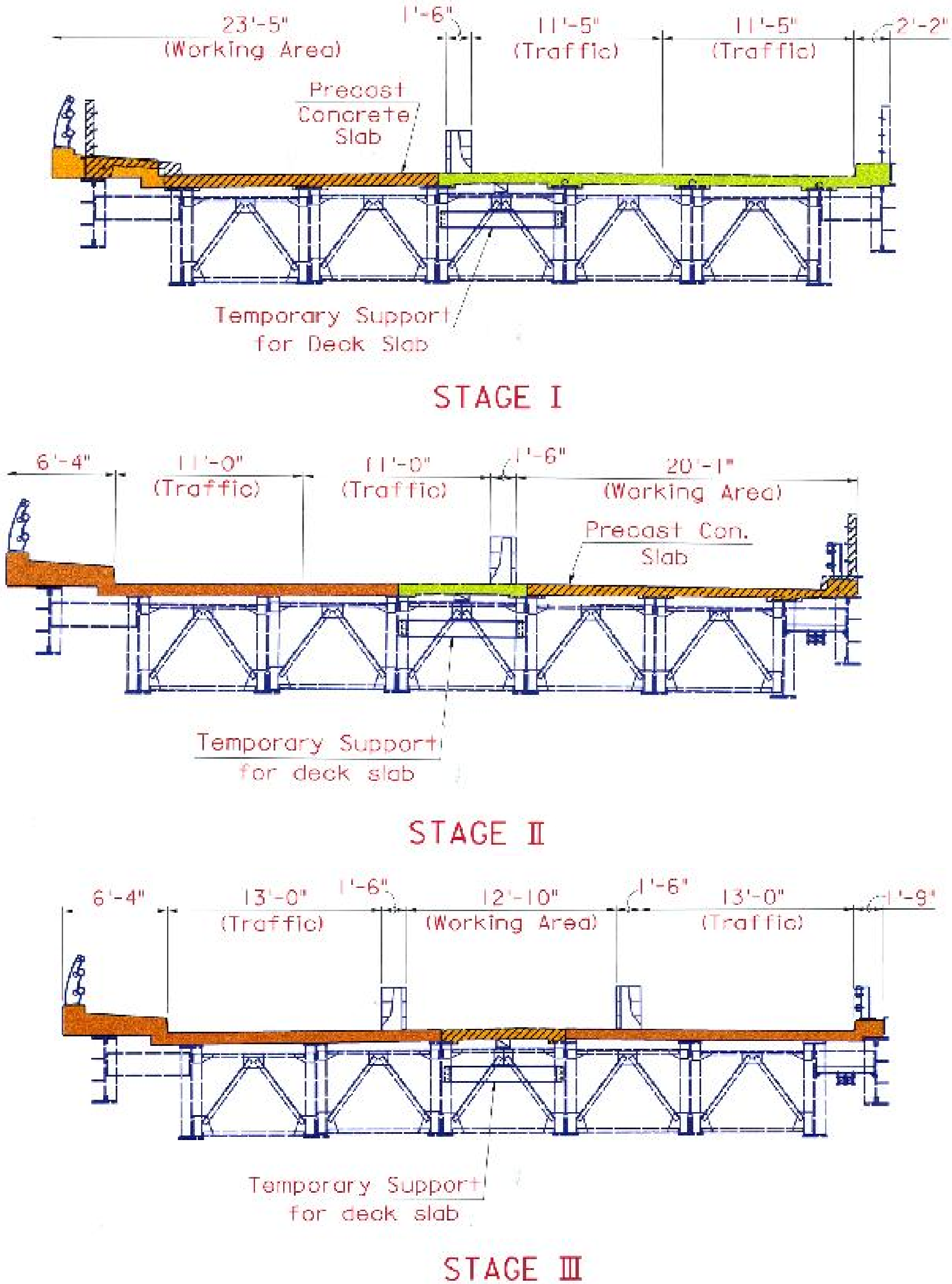


Figure A.2.4.13-1 Construction stages

Continuity across the transverse joints was accomplished by providing a post-tensioned, grouted shear key (Figure A.2.4.13-2). High-early-strength grout was used in the shear key. The gap at the bottom of the shear key compensated for the dimensional tolerance of the panels. Post-tensioning provided 200 psi (1.4 MPa) compression at the transverse joint. This amount of compression was sufficient for simple span bridges because the deck was under compression from superimposed dead loads and live loads. The post-tensioning strands ran along oblong ducts placed at mid-depth of the panels. The ducts were spliced at each transverse joint in small blockouts. Each post-tensioning duct used three 6/10 in. (15.2 mm) diameter, seven-wire, low relaxation strands with an ultimate tensile strength of 270 ksi (1.86 GPa). After post-tensioning, the ducts were pressure grouted and the blockouts were filled with high-early-strength concrete. To improve the shear transfer, welded sliding shear plates were installed across each transverse joint (Figure A.2.4.13-3).

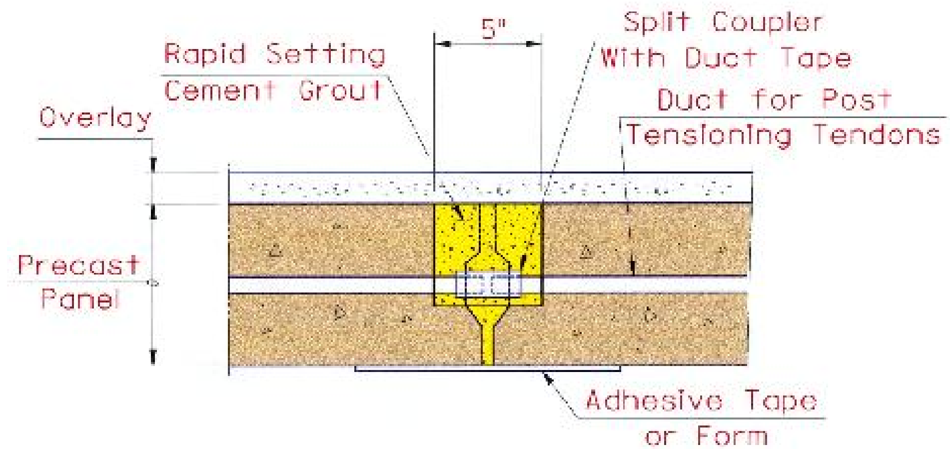


Figure A.2.4.13-2 Post-Tensioning duct splice blockout

The two longitudinal joints between the three construction stages were oriented over girders. The negative moment transfer at the longitudinal joints was provided by spliced top transverse bars embedded in a 3 ft (910 mm) strip of partial depth, high-early-strength, cast-in-place concrete, as shown in Figure A.2.4.13-4.

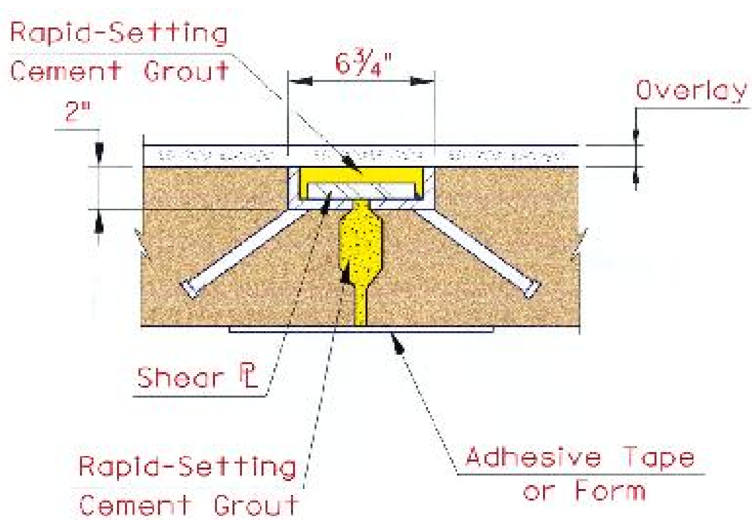


Figure A.2.4.13-3 Shear plates

The panels had shear stud blockouts located over the girders. Composite action was provided by studs welded to the girders in these blockouts. The blockouts had a tapered wall to prevent uplift of the panel and they were filled with the high early strength concrete. It should be noted that the shear stud blockouts were filled after post-tensioning, to prevent exerting positive moments onto superstructure from post-tensioning.

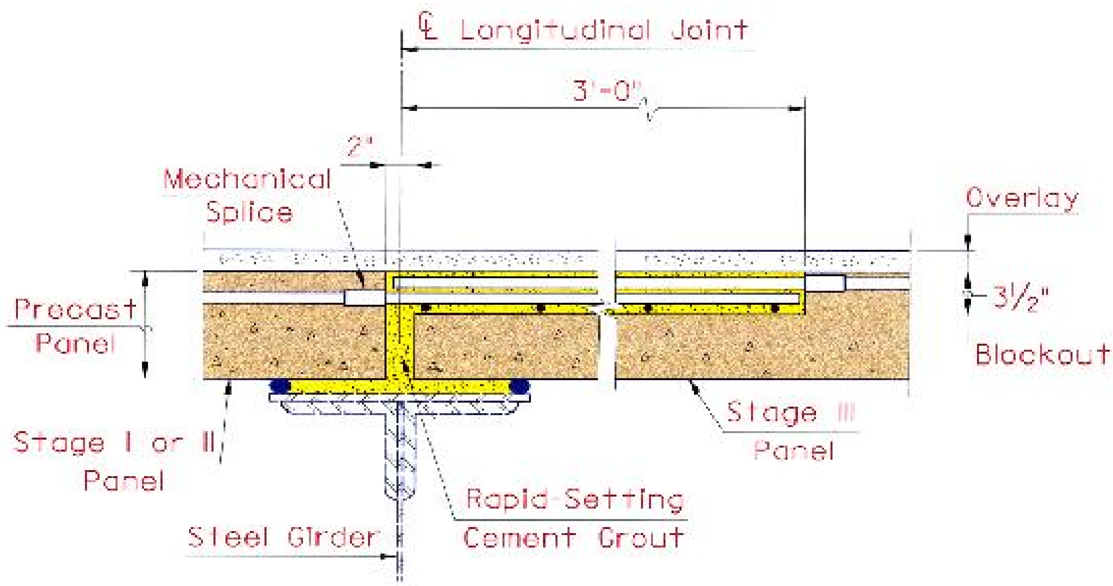


Figure A.2.4.13-4 Longitudinal connection between Stage 3 and Stage 1 & 2

In order to temporarily restrain the panels against movements caused by the daytime traffic, two bolts were welded to the girder in the blockout in place of the two exterior shear studs, as shown in Figure A.2.4.13-5. These bolts secured a temporarily hold-down plate in the blockout and restrained vertical and horizontal panel movements. The blockout was temporarily filled with sand and topped with asphalt concrete in preparation for daytime traffic. After post-tensioning, the asphalt concrete, filler sand, and hold-down plate were removed. Subsequently, the hold-down bolts were cut to the size and the blockout was filled with the high early strength concrete.

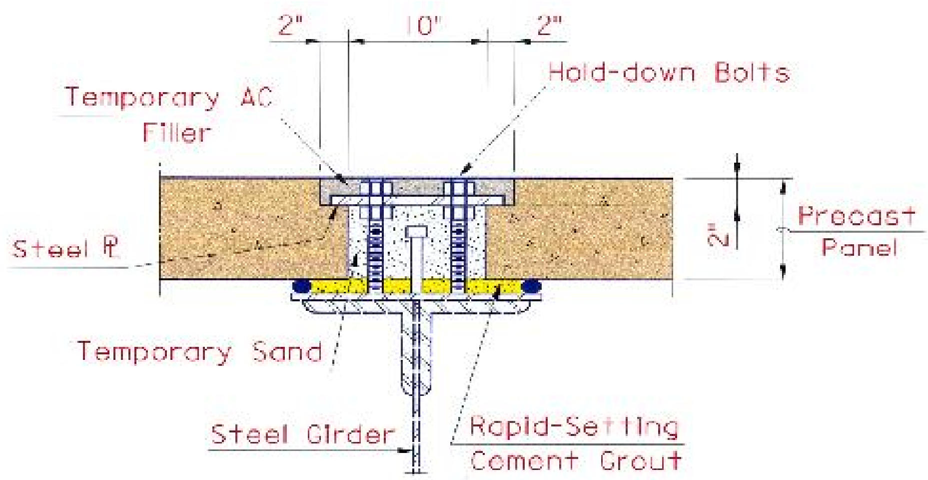


Figure A.2.4.13-5 Shear stud blockout with down hold bolts

The panels were fabricated based on the as-built dimensions and shipped/stored nearby the job site prior to construction**.** Every night, after saw cutting and removing the existing slab, the panels were lowered into position and placed on the existing steel framing. The final elevation of the top of the deck was achieved by adjusting the leveling bolts. Every night, the haunch between the beam flange and the panel soffit was built with a flowable, high-early-strength concrete fed through the blockouts located over the beam.

Composite construction was initiated by welding studs to the beam flange in the same blockouts prior to building the haunch. The blockouts were temporarily filled with sand and topped with asphalt in preparation for the daytime traffic. Also, the temporary hold-down bolts and plates were installed in selected blockouts to prevent panel movements under the daytime traffic. Every night, the panel shear keys along the transverse side were filled with the high-early-strength concrete and the welded sliding plates were installed across the shear key. The high-early-strength concrete was also used every night during the construction of Stage 3 to embed the spliced negative moment bars across the longitudinal joints**.**

After all panels were installed, they were post-tensioned in the longitudinal direction to assure tight transverse joints between the panels. The anchorage assembly blockouts (at the ends of the bridge) were then filled with the high-early-strength concret**e.** Subsequently, the shear stud blockouts were cleaned from sand and asphalt and filled with the high-early-strength concrete to achieve composite construction. Finally, an overlay was applied on the entire deck to provide a smooth ride over the panel joints and to waterproof the joints. The overlay consisted of asphalt concrete with a waterproofing membrane. The membrane was a preformed sheet membrane that was unrolled on the panels and torch welded at the overlaps prior to placement of the asphalt concrete. In addition to waterproofing, the membrane has the ability to bridge the panel joints and prevent reflection of the joints in the asphalt concrete. The panels on the sidewalk were sealed and covered with a thin layer of polymer with sand broadcast over the polymer for texture**.**

A.2.4.14 Wisconsin Department of Transportation (19)

WisDOT chose the US Interstate 39/90, Door Greek Project, as a demonstration bridge of the use of precast deck panel systems. The original project was a twin bridge carrying two lanes. Each bridge was an 83 ft (25.30 m) long, 40 ft – 2 in. (12.24 m) wide, single span structure with a 30 degree skew, supported on five 60 in. (1524 mm) deep steel plate girders spaced at 8 ft- 10 in. (2.69 m) on center. The deck replacement project included widening both bridges to 64 ft – 6 in. (19.66 m) by adding three steel plate girders at 7 ft-6 in. (2.29 m) on center, as shown in Figure A.2.4.14-1. The proposed precast system consists of full-depth precast concrete deck panels, which are constructed off-site and brought to the site ready for placement. The panels are then post-tensioned together in place in both the longitudinal and transverse directions. For the prototype bridge, stage construction was used, which required that a longitudinal construction joint be present. Figure A.2.4.14-1 shows a plan view of the proposed deck panel layout, where skew panels are used.

Because the panels were post-tensioned in place for both the longitudinal and transverse directions, post-tensioning ducts had to be placed in both directions. As seen in Figure A.2.4.14-2, the longitudinal post-tensioning duct is located in the center of the slab, while the transverse post tensioning ducts are placed above and below the longitudinal ducts. The panel thickness for the Door Creek Bridge is 8¾ in.

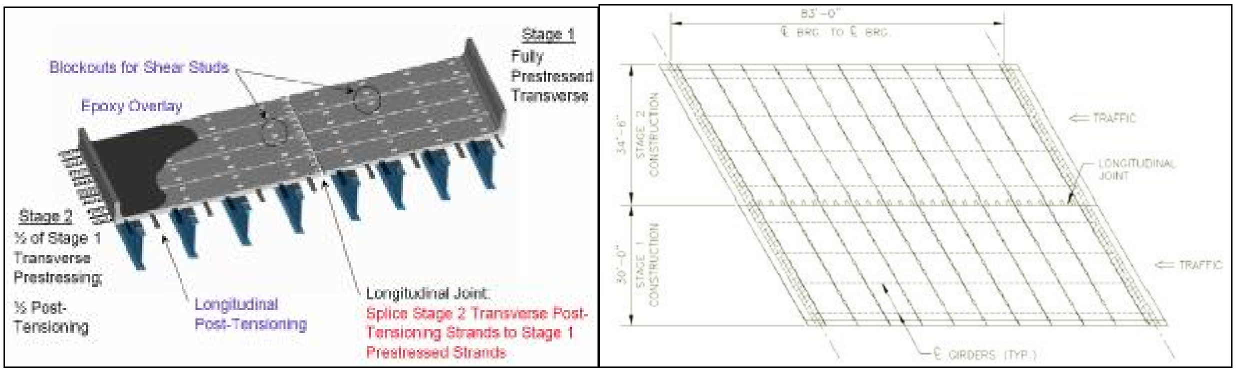


Figure A.2.4.14-1 General layout of the precast deck system

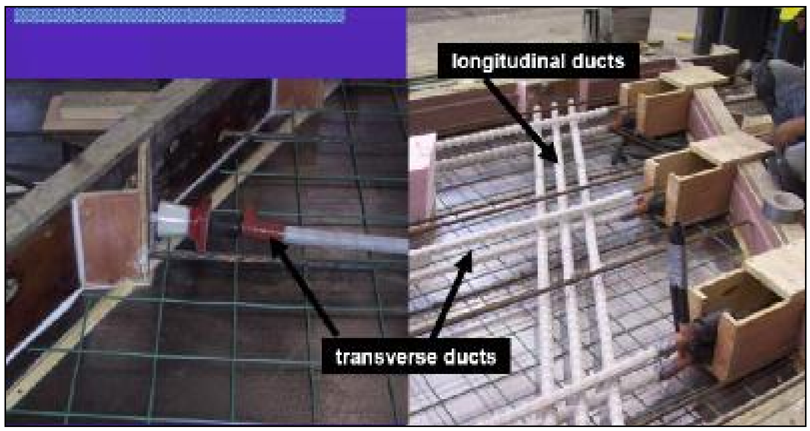


Figure A.2.4.14-2 Longitudinal and transverse post-tensioning ducts

The longitudinal joint between stage 1 and 2 consists of a post-tensioned female-female joint. A system was developed in which the stage 1 panels were fully pretensioned transversely to resist panel stresses due to handling, transportation, and placement, as well as vehicle induced bending due to traffic during the stage 2 construction. Only longitudinal post-tensioning was needed before traffic could be applied on stage 1 panels. Roadway crown or cross slope was achieved using flat panels and a “kink” at the longitudinal joint. Half of the transverse pretensioning strands, spaced at 28.5 in. (42 cm), were left protruding from the stage-1 panels. The transverse post-tensioning ducts in the second stage panels were placed to match the locations of the top and bottom protruding pre-tensioned strands of the stage 1 panels at the longitudinal joint. Post-tensioning strand is placed in these stage 2 panel ducts and then coupled to the protruding strand from the first stage construction. These post-tensioning strands, along with an equal amount of pre-stressing strand already cast into the panel for handling, transportation, and placement, resist vehicle induced bending in stage 2 panels. Part of the design was to prevent any cracking at service load levels. The post-tensioned joint should create a much more durable bridge deck and is ultimately the reason it was incorporated. All ducts were grouted with Sika 300PT prepackaged grout. The longitudinal joint for the bridge is not located over a girder; instead the joint occurs between girders. When traffic is on the stage 1 deck, during stage 2 construction, the cantilevered portion of the deck to this joint had to resist moments from a temporary barrier wall.

The moments induced by traffic over the completed joint, however, controlled the design. If the deck cracks at the joint when under traffic load, it will occur at the bottom of the deck rather than the top because the joint is placed in the positive moment region. This should reduce ingress of salt solutions and leakage along the joint, making the deck more durable.

To achieve full composite action between the precast deck panels and girders, whether steel or concrete, shear connector block-outs are provided within precast deck panels. Headed shear studs (or stirrups in precast concrete beam girder construction) that are attached to the girder extend into these block-outs to achieve the desired composite action. The number of studs or stirrups in each pocket is usually based on the American Association of State Highway and Transportation Officials3 design requirements. Based on previous projects with decks on steel girders, the maximum shear stud spacing or distance between shear stud block-outs is 2 ft (610 mm), which conforms to AASHTO, both LRFD3 and the Standard Specifications design limits. The writers believe this limit is a safe "rule-of-thumb" limit imposed by the AASHTO to assure complete composite action and avoid fatigue conditions. In most circumstances this spacing is based on the fatigue capacity of the studs, and not the ultimate capacity. With precast panels it is beneficial, however, to place the shear connector block outs at the largest spacing possible. This allows for fewer block outs in the panels, which in turn increases panel strength for shipping and decreases manufacturing time and cost. An alternate spacing of 4 ft (1,220 mm) is used on the Door Creek ridge. The original shear connectors on the existing steel girders are removed and the new studs are placed in the pockets after all of the panels are positioned and post-tensioned longitudinally. The pockets are subsequently grouted along with the haunches between the girders and panels. Achieving a shear connection with an existing prestressed concrete girder would be more difficult, but would likely entail attaching steel plates to the top of the girders, to which studs would be welded later. Finally, the deck is milled to provide a smooth driving surface and then receives a two layer epoxy overlay. Epoxy was selected to allow observation of joint behavior over time. Figures A.2.4.14-3 to A.2.4.14-6 show some of the construction steps.

Figure A.2.4.14-3 Construction Stage #1

Figure A.2.4.14-4 Forming, grouting & longitudinal post-tensioning of Construction Stage #1

Figure A.2.4.14-5 Milling and overlaying of Stage #1

Figure A.2.4.14-6 Longitudinal post-tensioned joint between Stage 1 and 2

A.2.4.15 Ontario Ministry of Transportation, OMOT

OMOT implemented the first field application of fabricated bridge technology for a bridge replacement project in northern Ontario, Moose Creek Bridge, in late 2003. The structure is a single span bridge 101 ft – 8 in. (22000 mm), as shown in Figure A.2.4.15-1. Total width of the bridge is 48 ft (14640 mm) and has a crown at the centerline of the road.

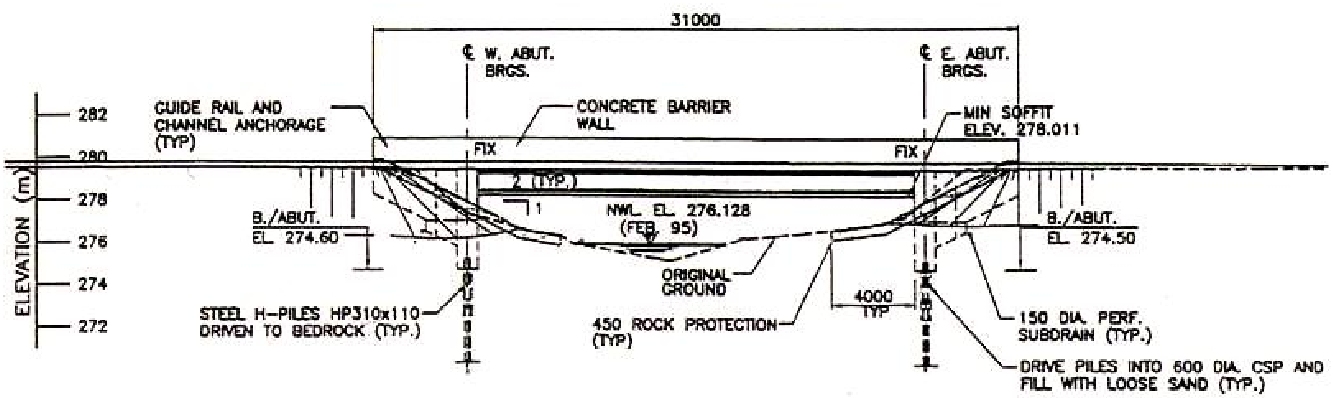


Figure A.2.4.15-1 Elevation view of the Moose Creek Bridge

The superstructure is made of six, 47 in. (1200 mm) deep AASHTO precast, prestressed concrete beams spaced at 8 ft (2450 mm). In order to minimize shipping and construction cost, the designer has decided to cast the deck slab monolithically with the precast beam, which resulted in six T-beams, as shown in Figure A.2.4.15-2. Compressive concrete strength at release and at 28 days are 4,600 and 7,250 psi (32 and 50 MPa) respectively.

The T-beams are connected in the longitudinal direction by extending the transverse reinforcement of the deck into a 13.7 in. (350 mm) joint created between the beams. This gap is filled with cast-in-place concrete. To avoid field forming for the joints, one edge of the T-beam is provided with a 2.75 in. (70 mm) thick concrete tongue that could support the weight of the concrete filling the gap. Figure A.2.4.15-3 and A.2.4.15-4 give the details of the T-beam. The T-beams are hold in place using temporary bracing, which is removed after the longitudinal joints are cast. The traffic barriers are cast in the field.

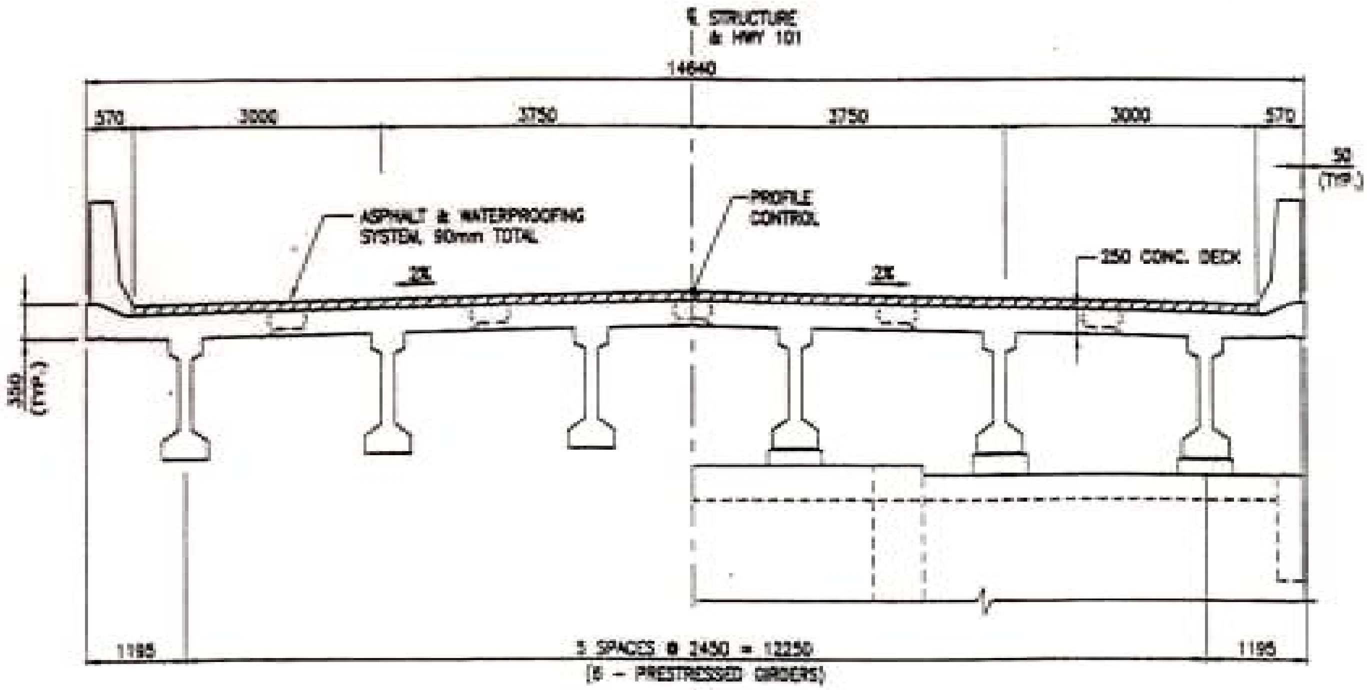


Figure A.2.4.15-2 Cross section of the Moose Creek Bridge

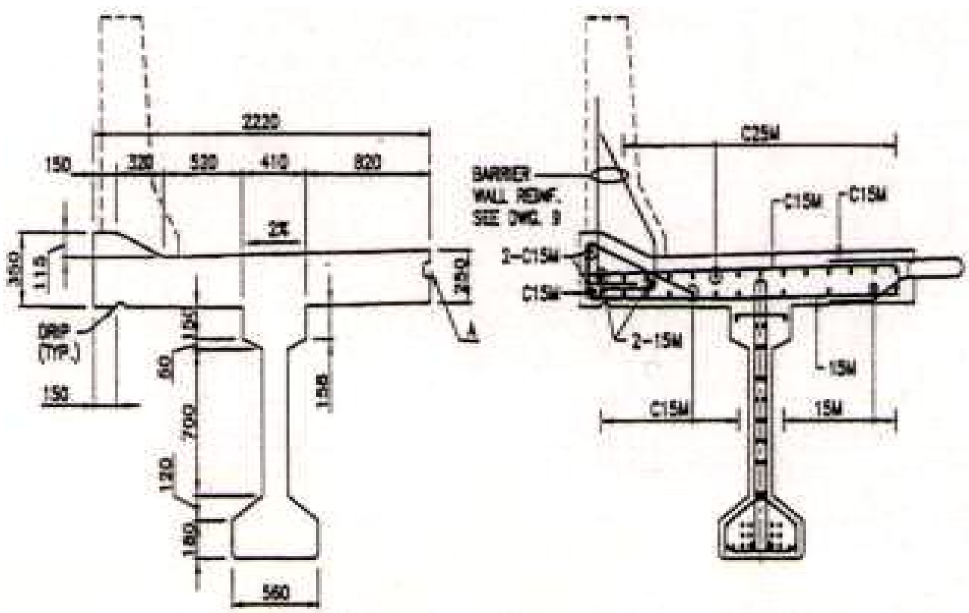


Figure A.2.4.15-3a Details of the exterior T-beams

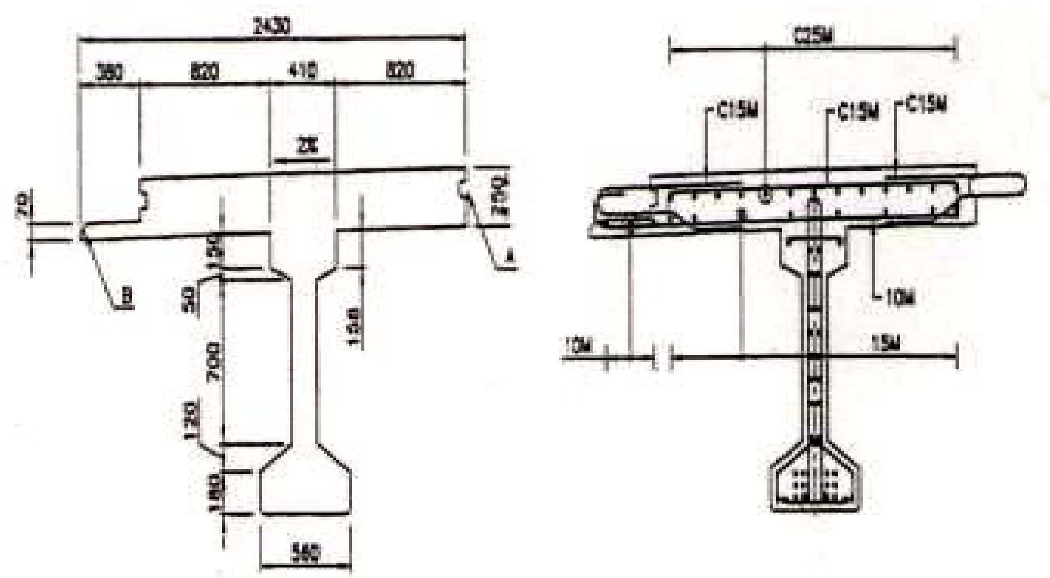


Figure A.2.4.15-3b Details of the interior T-beams

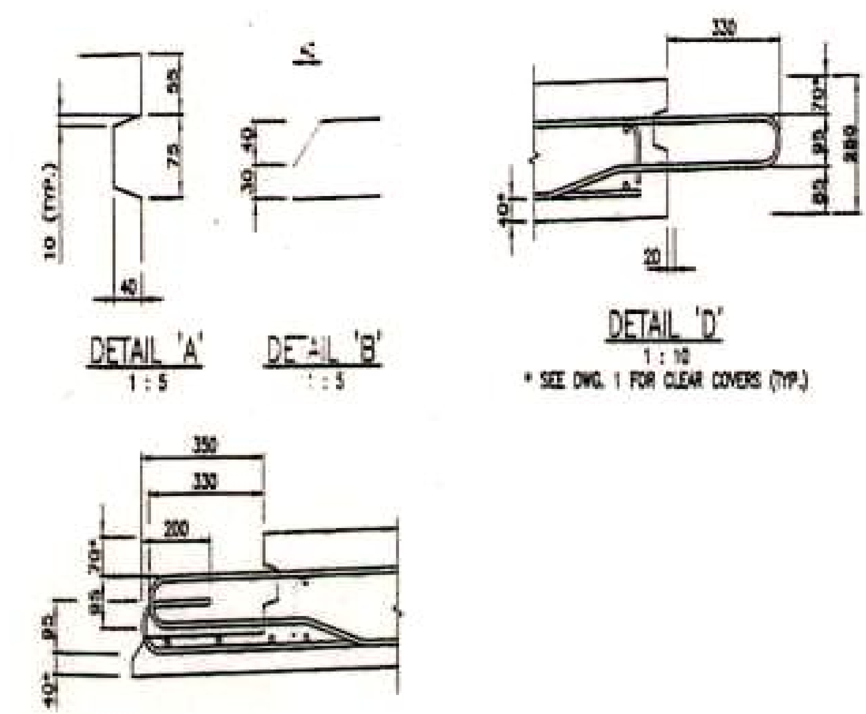


Figure A.2.4.15-4 Details of the shear key and tongue details

A.2.5 Miscellaneous Full-Depth Concrete Precast Deck Systems

A.2.5.1 Full-Depth Precast Prestressed Concrete Bridge Deck System Developed by University of Nebraska

This system was developed under the NCHRP 12-41, titled "Rapid Replacement of Bridge Deck," at the University of Nebraska *(20)*. An overview of the new system is shown in Figure A.2.5.1-1. This system is made of precast concrete panels made composite with the supporting girders. The length of the panel is 7'-10" (2390 mm) and it covers the full width of a bridge. The precast panels are transversely pretensioned and longitudinally post-tensioned. The panels are clamped to the supporting girders with threaded studs attached to the girder top surface.

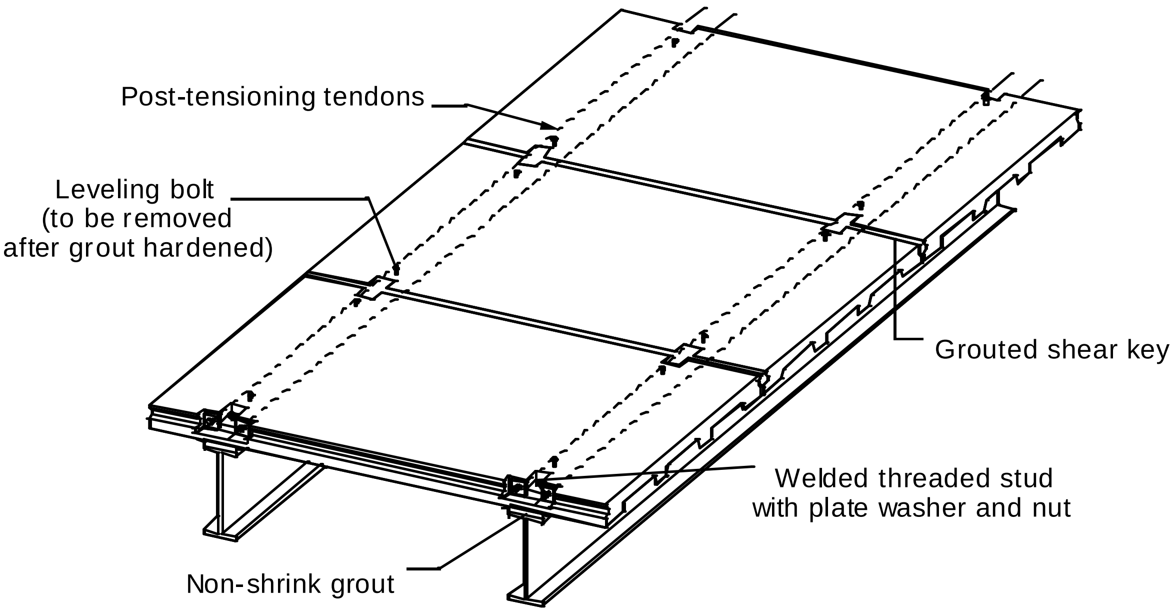


Figure A.2.5.1-1 General View of the University of Nebraska Deck System

A typical transverse cross-section of the precast panel has a $4^1/_2$ in. (115 mm) thick solid slab, 11.6 in. (295 mm) wide external stems and 5.9 in. (150 mm) wide internal stems, as shown in Figure A.2.5.1-2 and A.2.5.1-3. Using multi-stemmed section reduces self-weight of the deck and the amount of longitudinal post-tensioning required. The AASHTO Specifications (*21,22*) require $2^1/_2$ in. (63 mm) clear cover at the top of the slab when deicing compounds are used and 1 in. (25.4 mm) clear at the bottom of the slab. The solid slab thickness was determined to accommodate top strands and welded wire fabric with the required clear cover. Two-way shear (punching shear) strength was checked to support the decision. The width of external stems was set to accommodate two bottom strands and provide an adequate blockout for a post-tensioning anchorage device and threaded studs. The width of the interior stems was set to accommodate two bottom strands.

A specified strength of concrete for the precast panels of 5,000 psi (34.5 MPa) at transfer of prestress and a 28-day strength of 7,500 psi (51.71 MPa) have been used in developing the system. Twenty ½ in. (12.7 mm) diameter, 270 ksi (1.86 GPa) indented strands are used as the main flexural reinforcement in the transverse direction. Indented strands are used to reduce the required transfer and development length of the strands in the overhang. Also, confinement reinforcement bars are added at both edges of the panel for the same reason. Welded wire reinforcement (WWR) is used for temperature and shrinkage effects.

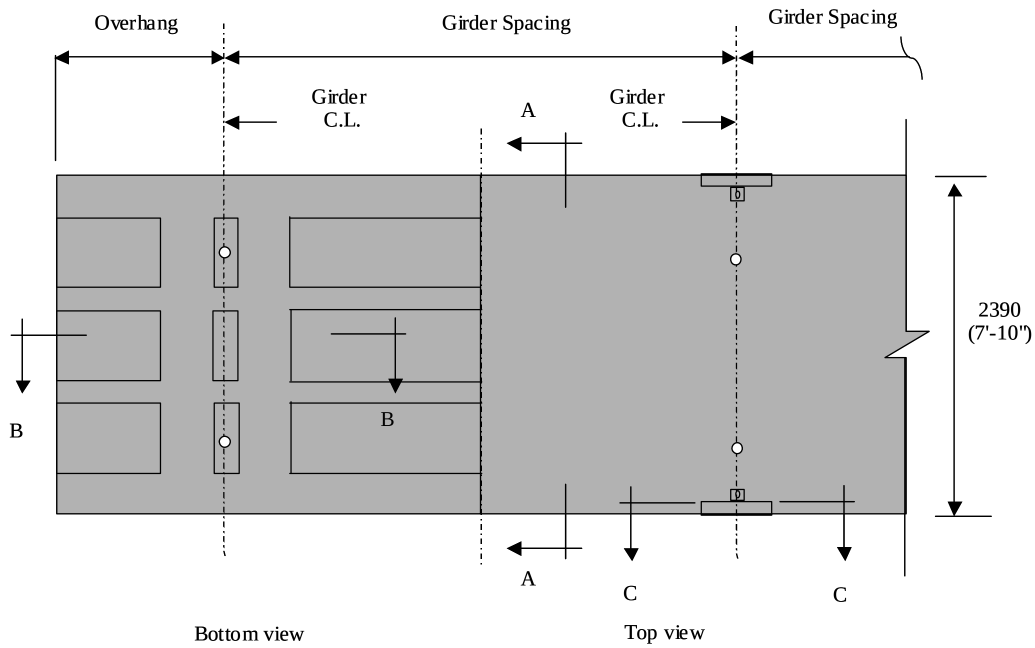


Figure A.2.5.1-2 Plan view of the panel

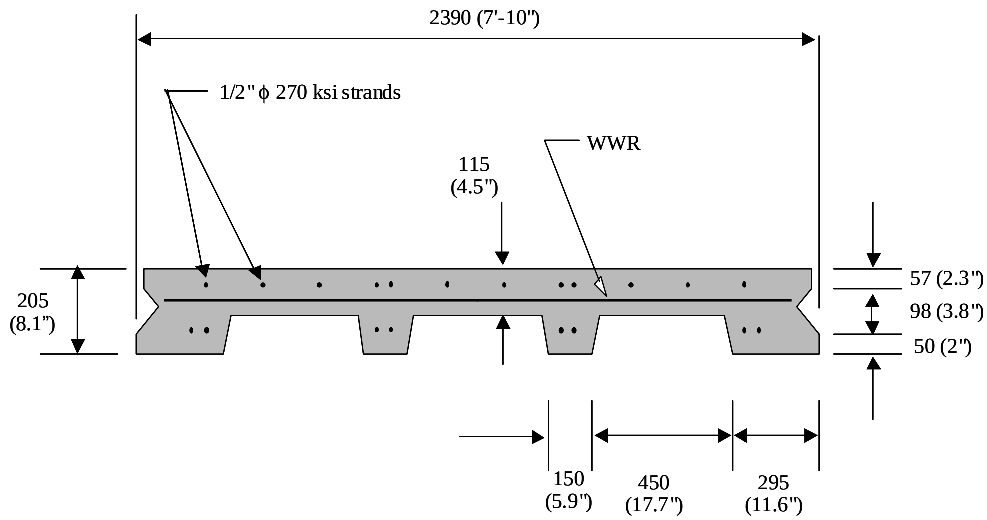


Figure A.2.5.1-3 Section A-A, Typical cross section of the precast panel

The longitudinal cross section consists of 8.1 in. (205 mm) thick solid sections at each girder location and 4.5 in. (115 mm) thick sections between them, as shown in Figure A.2.5.1-4. The thick portion at the girder location is used to accommodate post-tensioning steel and to eliminate eccentricity of post-tensioning forces. Two 1 in. (25 mm) diameter, 150 ksi (1.03 GPa) post-tensioning galvanized bars are used at each girder location, which provide 200 psi (1.38 MPa) of longitudinal compressive stress in the panels.

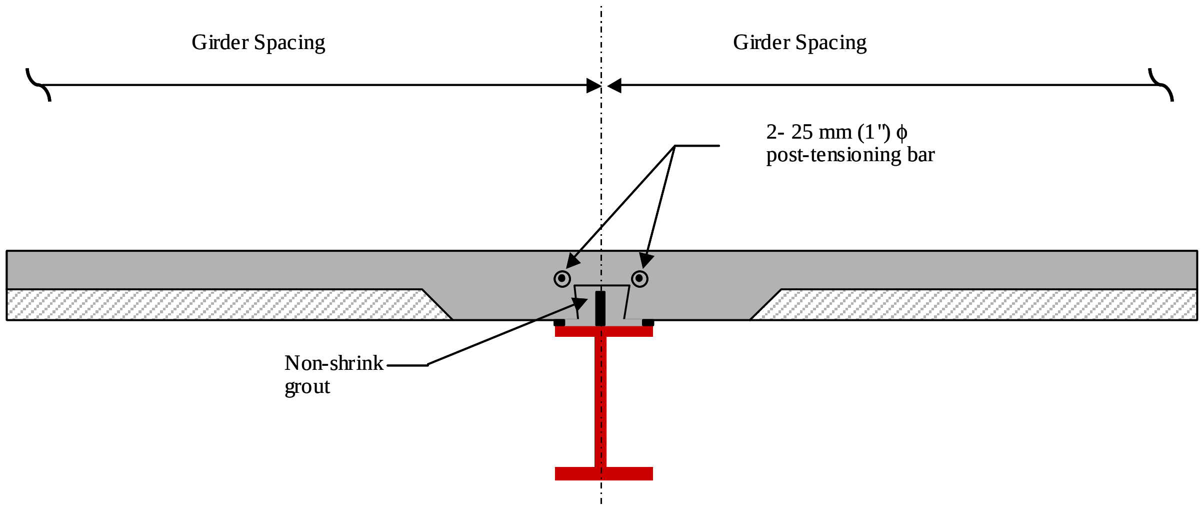


Figure A.2.5.1-4 Section B-B over a girder line

Blockouts are provided for anchorage and couplers of the post-tensioned bars at both transverse edges of panels as shown in Figure A.2.5.1-5. The blockouts are used also, to house the threaded studs that are needed to clamp the panel to the girder prior to applying the post tensioning force.

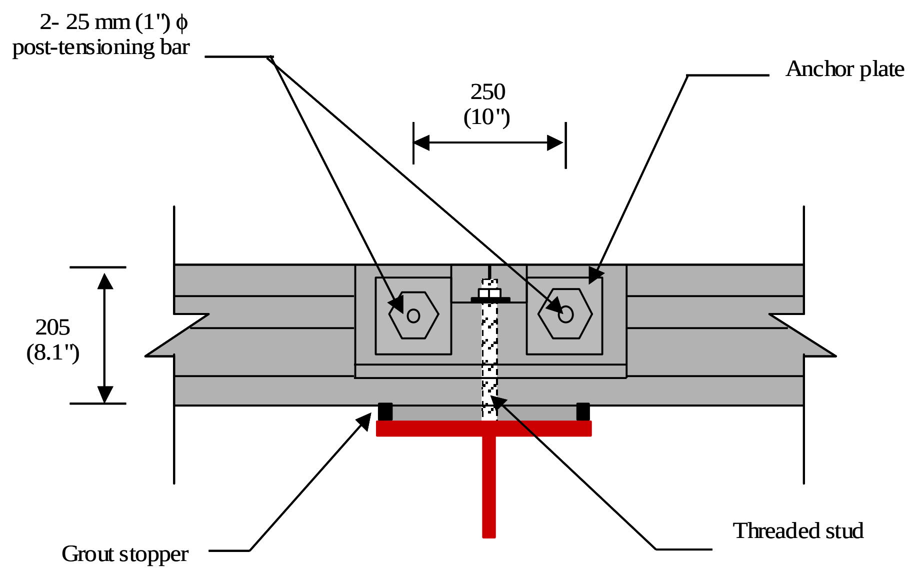


Figure A.2.5.1-5 Section C-C

The transverse edges of the panels are formed to create a shear key as shown in Figure A.2.5.1-3. When the panels are installed next to each other, a clear spacing of 0.4 inch (10 mm) is provided for production and construction tolerance. A flowable rapid-set, non-shrink grout (such as Set 45 by Master Builders) is used to fill the transverse joints between panels and packer rods are used to close this gap to protect the grout from leaking during the grout process.

To provide for the composite action between the precast panels and the girders, welded headless studs are used as shown in Figure A.2.5.1-6. The studs are clustered in three groups per panel per girder line. The clustered studs are housed in three pockets created in the precast panel. The headless studs are used, instead of the headed 3/8 in. (9.5 mm) or 7/8 in. (22.2 mm) headed studs, in order to facilitate deck removal in the future. In addition to the headless studs, threaded studs are used to clamp the panels with the girder.

Figure A.2.5.1-6 Horizontal shear connection between the panel and the girder

This system had gone under comprehensive testing investigation through full-scale testing of a bridge mockup *(Yamane et al 1998)*. Based on the results of this investigation, the following conclusions were drawn:

1. The proposed system was demonstrated to be cost competitive with other concrete panel system yet 10 to 30 percent lighter.
2. Panels can be rapidly produced, constructed, and removed.
3. Indented transverse pretensioning strands performed well. Their use is recommended.
4. Grouted post-tensioned transverse joints between precast panels showed excellent performance under service load and fatigue loading. The performance met all the requirements for a precast panel bridge deck system.
5. Deflections of precast panels under service load are fairly small at most locations.
6. The AASHTO punching shear requirements appears too conservative. The new precast panel system carried approximately 190 percent of the required factored load. The panel failed due to punching shear at an ultimate stage.
7. Headless studs and relatively little longitudinal reinforcement facilitate panel removal.

A.2.5.2 The Effideck System

Effideck is a lightweight composite precast bridge deck system. It consists of a 5-in. (127 mm) precast concrete deck slab supported on closely spaced structural steel tubes. Figure A.2.5.2-1 gives a plan view of the panel and Figure A.2.5.2-2 gives the cross section of the panel.

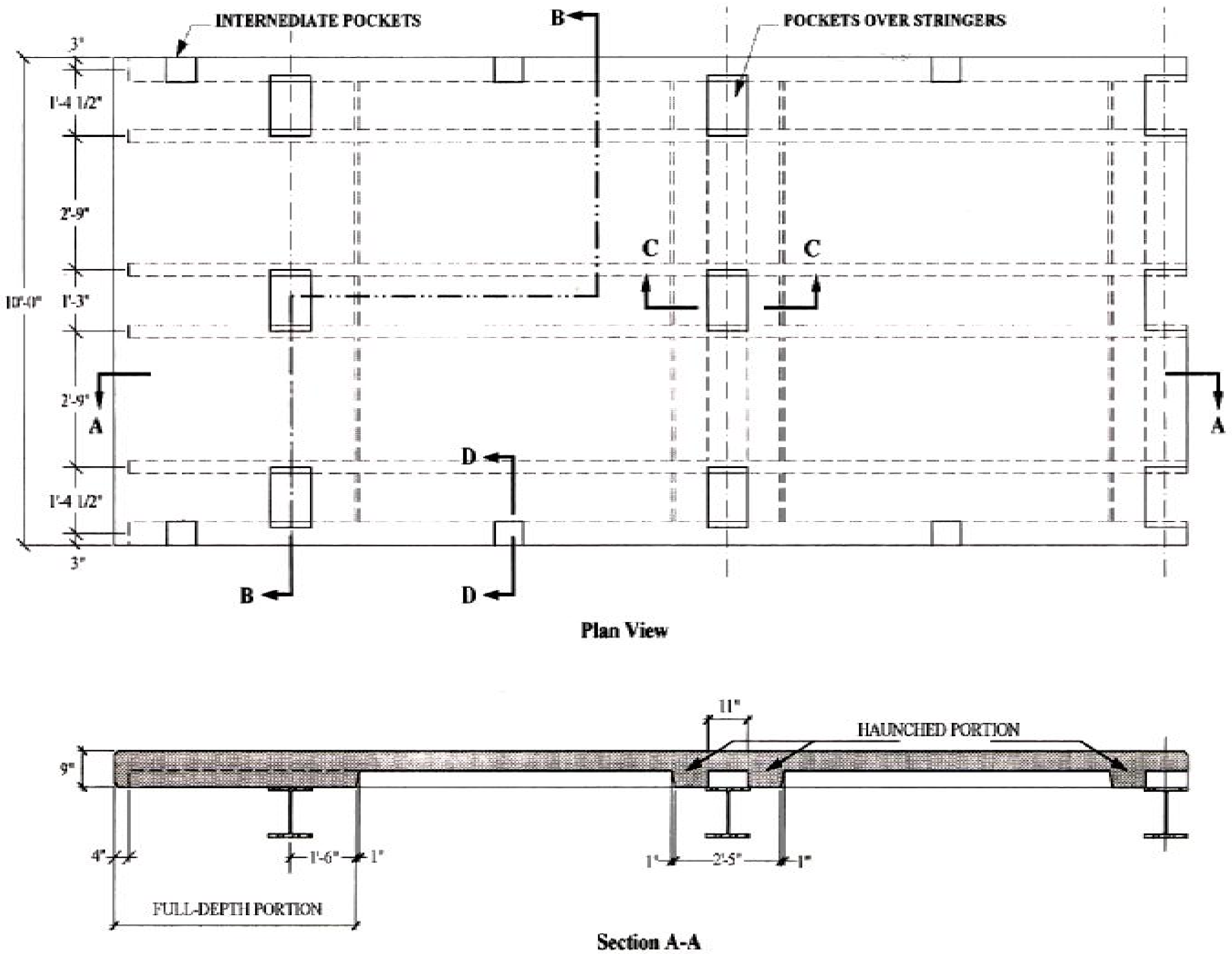


Figure A.2.5.2-1 Plan view and cross section elevation of the Effideck system

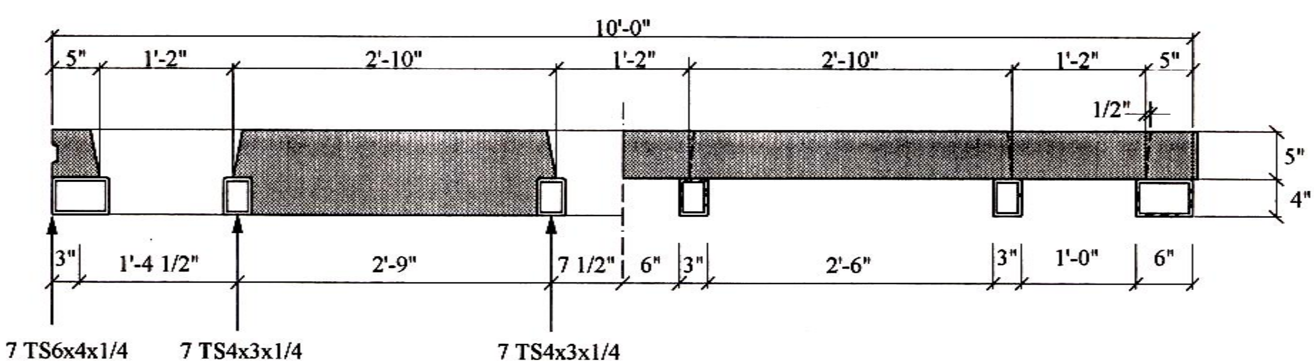


Figure A.2.5.2-2 Cross section view of the Effideck system (Section B-B)

The structural steel tubes are arranged in the transverse direction at 2 ft –9 in. (838 mm) and rest directly on the longitudinal girders (stringers) of the bridge superstructure. They are provided with headed steel studs in order to act compositely with the concrete slab. Therefore, the composite precast deck behaves like a multi-stem system supported by the bridge girders, as shown in Figure A.2.5.2-2. The concrete slab is conventionally reinforced with one layer of Grade 60, epoxy-coated steel bars and made from 5,000 to 6,000 psi (34.5 to 41.4 KPa) lightweight concrete.

At the negative moment areas across the panel, which are at the girder lines and at the overhang part of the deck, the deck is cast full-depth with concrete to provide haunches, as shown in Figure A.2.5.2-1. These haunches provide the compression block needed to resist the negative moment, and they work as barriers for the grout in the shear pockets. Figure A.2.5.2-3 gives a bottom view of the precast panel showing the transverse steel tubes and the concrete haunches at the girder line location.

Figure A.2.5.2-3 Bottom view of the Effideck system

The EFFIDECK system can be made composite with the superstructure girders. The panel is provided with shear pockets located at the girder lines and spaced at 4 ft (1220 mm) to accommodate steel studs clustered in groups, as shown in Figure A.2.5.2-4 and A.2.5.2-5. The panel is installed temporarily on shims and its elevation is adjusted using leveling screws, as shown in Figure A.2.5.2-6. Then the shear pockets and the space between the panels and the girders are filled with non-shrink, high early strength grout, which provides permanent support for the panels.

Figure A.2.5.2-4 Shear pockets accommodating clustered groups of steel studs

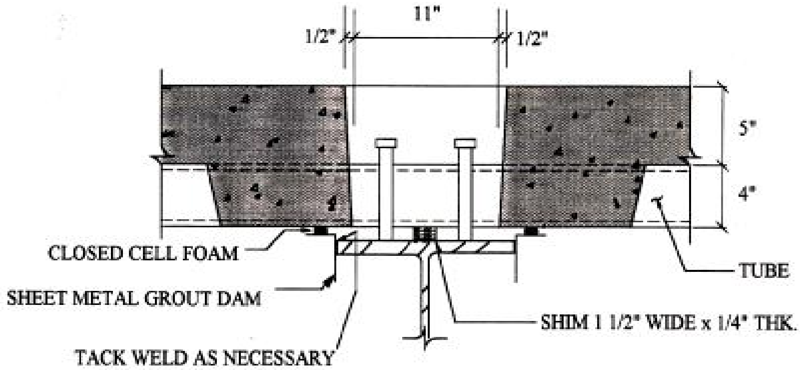


Figure A.2.5.2-5 Section C-C of the Effideck system showing girder-to-panel connection

Figure A.2.5.2-6 Adjustment of the panel elevation using leveling screws

In order to connect the panels in the longitudinal direction, each panel is provided with a female shape shear key along its transverse edges. A 3/4 in. (19 mm) gap is maintained during installation between adjacent panels and a backer rod is used to block the gap and work as barrier for the cast-in-place grout, as shown in Figure D.5.2-7.

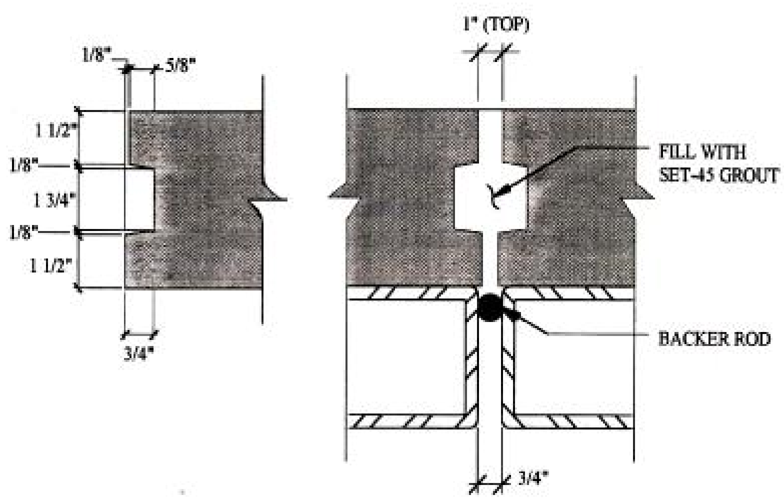


Figure A.2.5.2-7 Shear key details of the Effideck system

In addition to the shear key, the panels are positively connected using structural steel channel sections located at intermediate pockets along the transverse edges of the panel, as shown in Figure A.2.5.2-8 and A.2.5.2-9. The steel channel sections are bolted with the structural steel tubes of the panel.

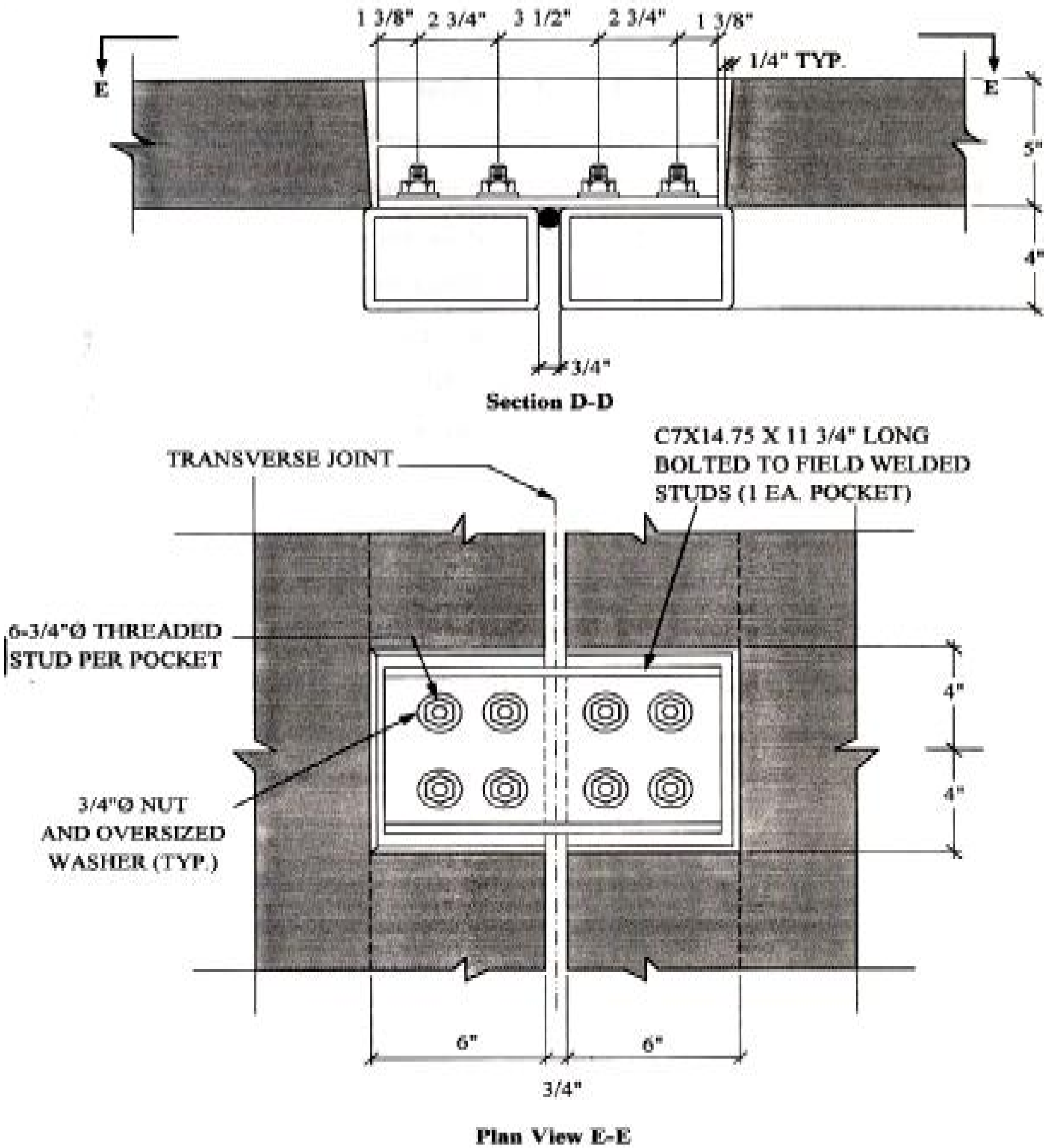


Figure A.2.5.2-8 Transverse panel-to-panel connection of the Effideck system

Figure A.2.5.2-9 Transverse panel-to-panel connection using a structural steel channel

The EFFIDECK panel can cover the full width of a bridge, as shown in Figure D.5.2-10. However, in case of very wide bridges where shipping of long precast panels is not possible, multiple panels can be used across the width of the bridge and connected with cast-in-place longitudinal joints as shown in Figure A.2.5.2-11.

Figure A.2.5.2-10 Full-width EFFIDECK panel

Figure A.2.5.2-11 Partial-width EFFIDECK panel

A.3 SHEAR KEY GEOMETRY, JOINT FORMING AND GROUT MATERIAL

A.3.1 Introduction

Typically, the shear key that extends along the transverse edges of a precast panel plays an important role in the service performance of the finished deck. The shear key has to be designed to protect adjacent panels form relative vertical movement and transfer the traffic load from one panel to the next panel without failure at the panel-to-panel joint.

Under traffic load, a panel-to-panel joint experiences two types of straining actions: (1) a vertical shear force that tries to break the pond between the panel and the grout filling the joint, and (2) a bending moment that puts the top half of the joint in compression and the bottom half of the joint in tension.

Accordingly, two modes of failure can be expected at the joint. These are: (1) bond failure between the grout and the panel, and (2) crushing of the grout close to the top surface of the panel. Searching the literature has shown that most of the problems at panel-to-panel joints are attributed to the first failure mode. This is due to the following facts:

1. The grout as a cementitious mix has tendency to shrink, which puts the interface between the panel and the grout in tension that may exceed the bond strength.
2. If the elevation of the top surface of adjacent panels is not carefully lined up, the impact effects of the traffic load at the joint is significantly magnified and eventually breaks the bond between the grout and the panel. This is true especially if no overlay is used and the top surface of the precast panels is used as the riding surface without filing.
3. Avoiding the second failure mode can be easily achieved by specifying a grout mix with a compressive strength that matches the panel concrete strength.

A.3.2 Shear Key Shape

Various shear key shapes have been used with full-depth precast concrete panels. The following section gives a short summary of the most common shapes that have been used.

A.3.2.1 Non-grouted Match-cast Joints

Figure A.3.2.1-1 shows the details of the joint. This detail was used by Indiana Department of Transportation. Although match casting can be achieved in a controlled fabrication environment, i.e. in a precast concrete plant, it has been found that it is very difficult to achieve a perfect match in the field after installing the panels due to construction tolerances and elevation adjustment of the panels. This detail was used in conjunction with longitudinal post-tensioning. Also, thin neoprene sheets were installed between adjacent panels to avoid high stress concentrations. Cracking and spalling of concrete at the panel joints were observed after five years of service (5), which eventually lead to a leakage problem at the joints.

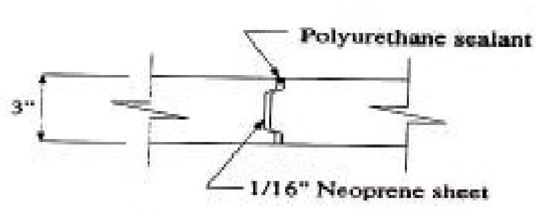
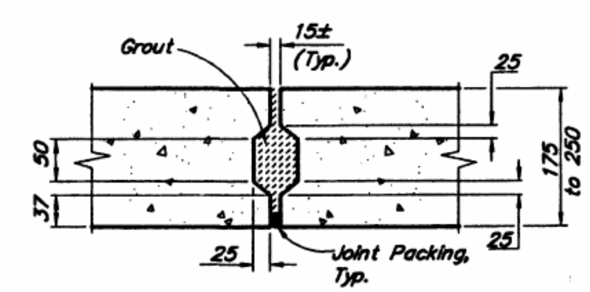


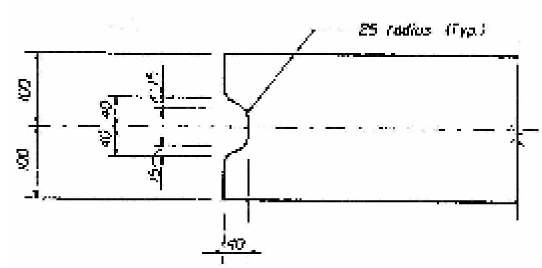
Figure A.3.2.1-1 Non-grouted Match-cast Joint

A.3.2.2 Grouted Female-to-Female Joints

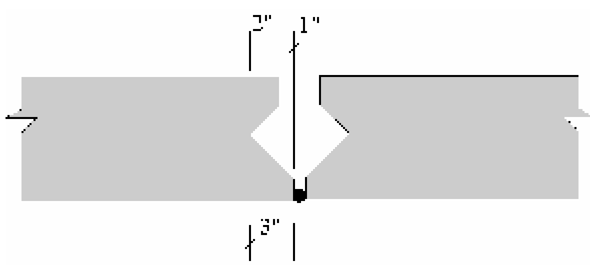
In this group of joints, grout is used fill the joint between adjacent panels. Inclined surfaces are provided in the shear key detail to enhance the vertical shear strength capacity of the joint. Therefore, vertical shear forces applied at the joint are resisted by bearing and by bond between the grout and the panel. The shear key is recessed at the top to create a relatively wide gap that allows casting the grout in the joint. Figure A.3.2.2-1 gives some of these details that have been used in bridges.



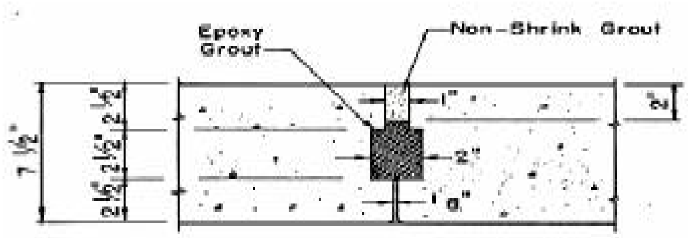
(a) Trapezoidal-shape shear key detail used in the Pedro Creek Bridge, Alaska



(b) Semi-circle shear key detail used in the George Washington Memorial Parkway Bridges, Washington DC



(c) V-Shape shear key detail used in the Skyline Drive Bridge, Omaha, Nebraska



(d) Rectangular shear key detail used in the Delaware River Bridge, New York

Figure A.3.2.2-1 Various grouted female-to-female joint details

With grouted joints, a form has to be provided at the bottom surface of the panels to protect the grout from leaking during casting. Two methods of forming have been used:

(a) Polyethylene backer rods in the tight space between panels at the bottom of joint (see Figure A.3.2.2-1 (a) and (b)): This detail has been used for a very long time by many highway authorities. Although, this detail does not require any construction work to be done form under a bridge, it has been reported (*Nottingham 1996, Gulyas 1996, and Issa et al 2003*) that due to fabrication and construction tolerances the joint may end up partially full, i.e. the grout does not fill the full height of the joint, as shown in Figure A.3.2.2-2. Partially-filled grouted joints cause high stress concentrations at the panel edges, especially if longitudinal post-tensioning is applied, and initiate cracking close to the bottom surface of the panels.

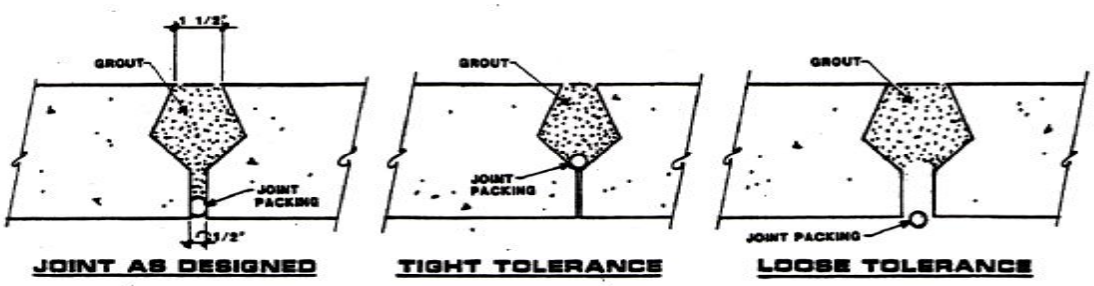


Figure A.3.2.2-2 Effect of tight and loose tolerances on panel-to-panel joints

(b) Wood forming from under the panel (as shown in Figure A.3.2.2-2): In this detail, a gap of 1 to 3 in. (25 to 76 mm) is maintained between adjacent panels and wood forms are installed from under the panel. The forms are hanged from the top surface of the precast panels using threaded rods and nuts. Using this detail usually results in a full-height grouted joint with excellent service performance (*23,24*).

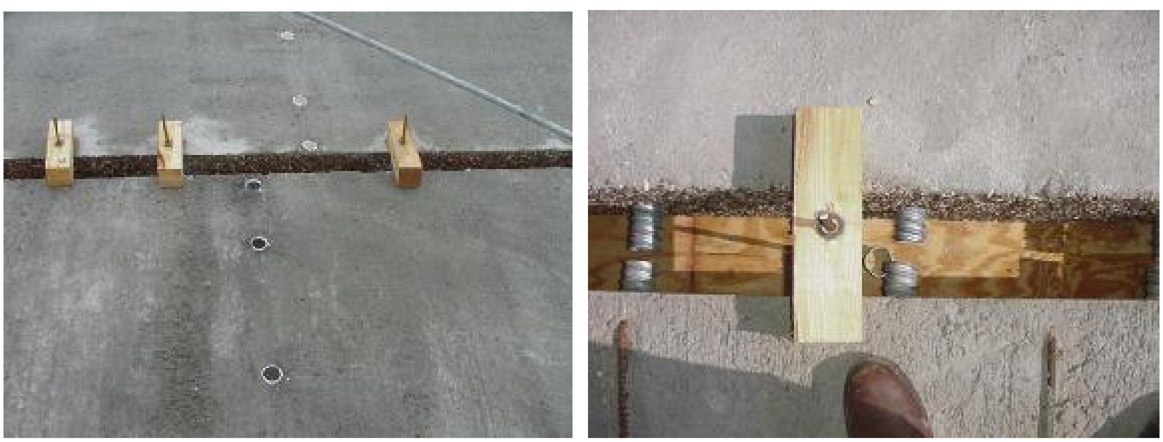


Figure A.3.2.2-2 Wood forming of the panel-to-panel joint used in the Arch Tied Bridges, Texas

A.3.3 Shear Key Texture

The bond between the grout and the shear key surface can be significantly enhanced by roughening the shear key surface (*25*). This has been found extremely important in connecting precast panels when no longitudinal post-tensioning is used and the joint is not pre-compressed.

Roughening can be achieved by sand blasting the shear key surface that is followed by a thoroughly washing procedure. This operation can be done in the precast plant before shipping the panels or on the bridge site before installing the panels on the bridge.

Also, roughening can be achieved during fabrication of the panels by painting the side forms with a retarding agent. After removing the side forms, the shear key is washed with water under high pressure, so that the aggregate of the concrete will be exposed and a uniformly roughened surface is created. This concept was used by Texas Department of Transportation in the precast concrete panels used for the Arch Tied Bridges, as shown Figure A.3.3-1.

Figure A.3.3-1 Exposed aggregate roughened surface used in the Arch Tied Bridges, Texas (see Section D.4.9 of this report)

A.3.4 Grout Material

Several grout material have been used in filling the shear pockets and the transverse joints between adjacent panels. Some of these grout material are commercial products and some are developed by state highway agencies. The common properties that exist among all types of grout are: (1) relatively high strength (2,000 to 4,000 psi) at young age (1 to 24 hours), (2) very small shrinkage deformation, (4) superior bonding with hardened concrete surfaces, and (3) low permeability. Through the literature review that has been conducted in this project, the researchers have noticed that the majority of state highway agencies specify the properties required for the grout material rather than specifying a certain type of grout material. Therefore, the contractor has to take the responsibility of choosing the type of grout material and then seeks the approval from the highway agency.

The following sections provide a summary of the most common types of grout that have used with full depth precast panels. Also, the following sections provide information about some of the recent research that has been done to compare the performance of various types of grout.

A.3.4.1 Commercial Products

Through the literature review conducted in this project, the researcher has found that the following commercial products have been used with full depth precast concrete deck.

SET 45: Chemical-action repair mortar: It is a one-component concrete repair and anchoring material, which sets in approximately 15 minutes, for use in ambient temperatures below 85°F (29°C).

SET 45 Hot Weather (HW): It is a one-component concrete repair and anchoring material, which sets in approximately 15 minutes, for use in ambient temperatures below 85-100°F (29-38°C).

SET GROUT: General construction, natural aggregate non-shrink grout: It is a Portland cement-based product, non-catalyzed, multi-purpose construction grout containing mineral aggregate.

EMACO 2020: Polymer concrete system: It is a methyl methacrylate (MMA), polymer concrete system designed for the protection and rehabilitation of horizontal, formed vertical or overhead concrete surfaces. It consists of three parts denominated A, B, and C, for binder, aggregate and initiator, respectively.

EMACO 2041: Bonding agent for EMACO 2020: It is a one-component, moisture-tolerant acrylic bonding agent applied to concrete or steel prior to the placement of EMACO 2020 polymer concrete system.

Recently, these types of grout material have gone under experimental investigation to measure their performance in full depth concrete deck panels. The findings of the experimental investigation are given in Section A.3.4.3 of this appendix.

A.3.4.2 Non-commercial Grout Material

The non-commercial grout materials presented in this section were used for regular construction schedule, where the bridge was closed for extended period of time, and the grout needs extended period of time of continuous curing (at least 7 days).

Hydraulic Cement Concrete (HCC):

HCC mixes were used on some the bridges built before 1972. The specifications for these mixes contained a minimum concrete strength of 4,000 psi (27.6 MPa), relatively high slump (about 6 in., 153 mm), and a maximum aggregate size of ½ in. (12.7 mm).

Latex Modified Concrete (LMC):

LMC mixes are different from HCC mixes in the essence that a latex emulsion is added to the mix. The latex forms a thin film on the aggregate surface, which enhances the bond between the past and the aggregate and results in high compressive strength and less permeable concrete mix.

Many state highway agencies have developed their own LMC mix. The following are the specifications of the LMC mix that has been developed and used by Virginia Department of Transportation (*26*,*27*).

Portland cement III (minimum)	7 bags, 658 lb/yd^3 (388 kg/m^3)
Water (maximum)	2.5 gal/bag of cement
W/C	0.35 to 0.40
Styrene butadiene latex emulsion	3.5 gal/bag of cement

Air content	3 to 7%
Slump (measured 4.5 minutes after discharge)	4-6 in. (100-200 mm)
Cement/Sand/Aggregate by weight	1.0/2.5/2.0

Type K-cement Concrete Mix:

A type K-cement concrete mix was used on the Skyline Bridge in Omaha, NE to fill the longitudinal open channels that house the post-tensioned cables. The concrete mix has a specified concrete strength of 4,000 psi (27.6 MPa) and only cement Type K is used in the mix. The concrete mix has no fly ash and the maximum aggregate size is 3/8 in. (9.5 mm).

Type K cement is an expansive cement that contains anhydrous calcium aluminateupon, which being mixed with water, forms a paste, that during the early hydrating period occurring after setting, increases in volume significantly more than does portland cement paste.

A.3.4.3 Recent Research related to Grouting Material

Grout Material filling the Transverse Joint

In a recent study conducted by *Issa et al* (*25*), the researchers studied the behavior of a female-to-female joint detail using SET 45, SET 45 HW, SET GROUT, and EMACO 2020. The joint was tested for direct vertical shear, direct tension, and flexure as shown in Figure A.3.4.3-1. A total of 36 specimens were tested. The compressive strength of the elements that resented the precast panels was about 6,250 to 6,500 psi (43 to 45 MPa). Figure A.3.4.3-2 gives the mix proportions and the strength development of various types of grouting material used in the study. For all the specimens, the joint surfaces were sandblasted and thoroughly cleaned. Also, no reinforcement crossing the interface between the joint and the precast panel was present. In addition to the full scale testing of the joint, the permeability and shrinkage properties of the grouting material was conducted in accordance with ASTM C 1202-97 and ASTM C157 respectively.

Findings of the experimental program are given in Figures A.3.4.3-3 and A.3.4.3-4 and can be summarized as follow:

(1) Failure of specimens made with EMACO 2020 occurred away from the joint in the precast panels, while failure of the specimens made with SET GROUT occurred simultaneously through the joint and in the precast panels. For specimens made with SET 45 and SET 45 HW, failure occurred through the joint.

(2) The shear, tensile and flexural strength of joints made with EMACO 2020 were the highest among all types of grouting material.

(3) The shear, tensile and flexural strength of joints made with SET GROUT were higher than those of SET 45 and SET 45 HW.

(4) Moisture and carbonation at the joint surface adversely affected the bond and strength of joints made with SET 45.

(5) EMACO 2020 and SET 45 set very fast, which require fast mixing and installation process.

(6) EMACO 2020 was significantly less permeable and showed much lower shrinkage deformation compared to other grout material.

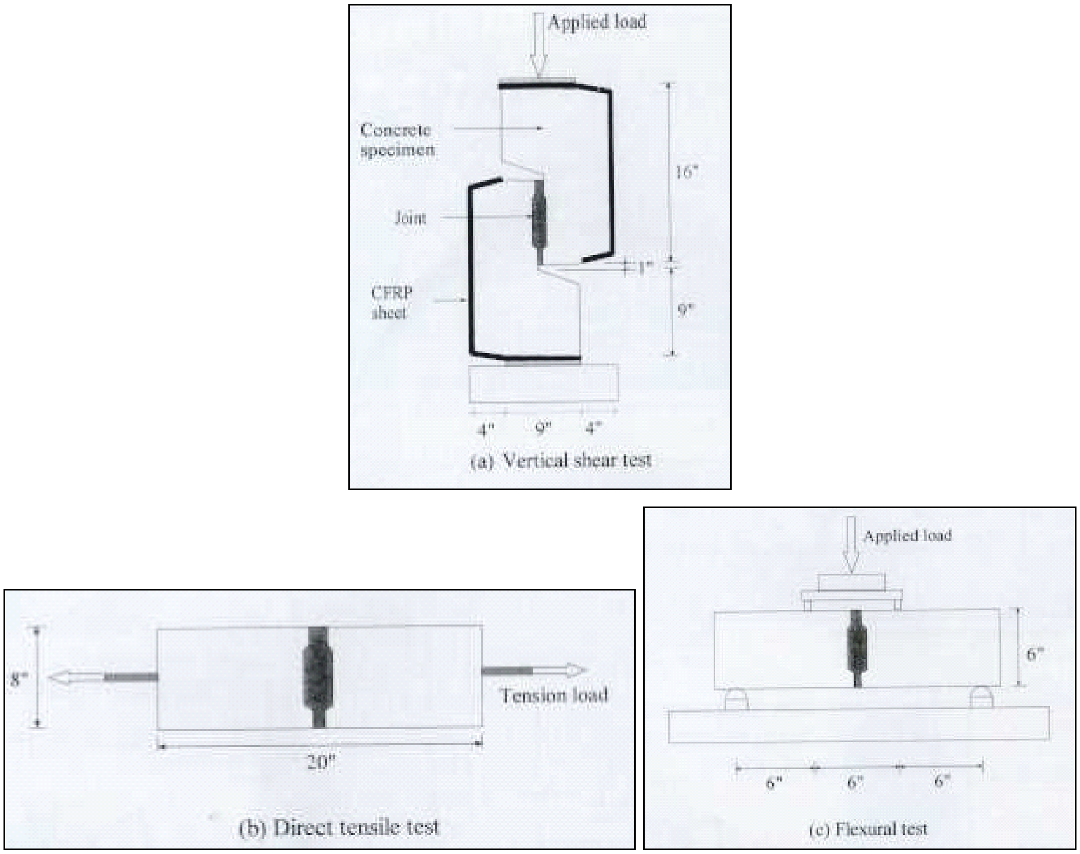


Figure A.3.4.3-1 Configuration and test setups of the test specimens *(Issa et al 2003)*

Table 3. Mix proportions and compressive strength of grouting material.

Mix design and properties	Grouting material			
	Set 45	Set 45 HW	Set Grout	Polymer (Emaco 2020)
Grouting material, lb	50	50	50	Part A : B : C =
Mix water, lb	3.97	3.97	7.05	1.0 : 10.9 : 0.1
Compressive strength at the age of testing of specimens, psi	5820	5658	7700	10,810

Note: 1 lb = 0.454 kg; 1 psi = 0.0069 MPa.

Table 4. Strength development of grouting material.

Age at testing	Type of test	Strength of grouting material, psi		
		Set 45	Set Grout	Polymer concrete
3 hours	Compressive	—	—	9752
6 hours	Compressive	3718	—	10169
1 day	Compressive	3775	2841	10357
3 days	Compressive	4294	5109	10460
	Tensile	574	548	988
7 days	Compressive	5516	6312	10550
	Tensile	587	598	1130
28 days	Compressive	6122	10031	10756
	Tensile	605	703	1153

Note: 1 psi = 0.0069 MPa.

Figure A.3.4.3-2 Mix proportions and the strength development of various types of grout material *(Issa et al 2003)*

Table 5. Direct shear test results.

Material type	Specimen number	Shear stress (psi)	Average shear stress (psi)	Concrete f'_c (psi)	Grouting material f'_c (psi)	Mode of failure
Set 45	S1	301.1	325.2	6500	5820	Fracture through joint
	S2	320.4				
	S3	354.1				
Set 45 HW	S1	285.3	298.7	6250	5658	Fracture through joint
	S2	305.9				Fracture through joint
	S3	305.0				Fracture through joint and concrete
Set Grout	S1	401.5	358.3	6500	7700	Fracture through joint and concrete
	S2	343.3				
	S3	330.1				
Polymer concrete	S1	748.4	704.3	6500	10810	Fracture of concrete away from joint
	S2	667.1				
	S3	697.4				

Note: 1 psi = 0.0069 MPa.

Table 6. Direct tensile test results.

Material type	Specimen number	Tensile stress (psi)	Average tensile stress (psi)	Concrete f'_c (psi)	Grouting material f'_c (psi)	Mode of failure
Set 45	T1	207.8	200.9	6250	5820	Fracture through joint
	T2	175.9				
	T3	219.0				
Set 45 HW	T1	198.4	205.6	6250	5658	Bond (interface)
	T2	214.6				Fracture through joint
	T3	203.8				Fracture through joint
Set Grout	T1	197.0	223.7	6250	7700	Fracture through joint and concrete
	T2	246.3				
	T3	227.9				
Polymer concrete	T1	330.1	291.6	6250	10810	Fracture of concrete away from joint
	T2	288.8				
	T3	256.0				

Note: 1 psi = 0.0069 MPa.

Table 7. Flexural test results.

Material type	Specimen number	Flexural stress (psi)	Average flexural stress (psi)	Concrete f'_c (psi)	Grouting material f'_c (psi)	Mode of failure
Set 45	F1	266.6	272.7	6250	5820	Fracture through joint
	F2	284.3				
	F3	267.6				
Set 45 HW	F1	516.5	498.1	6500	5658	Bond (interface)
	F2	531.4				
	F3	446.5				
Set Grout	F1	633.9	620.3	6250	7700	Fracture through joint and concrete
	F2	601.4				
	F3	625.6				
Polymer concrete	F1	783.9	773.1	6250	10810	Fracture of concrete away from joint
	F2	685.6				
	F3	849.7				

Note: 1 psi = 0.0069 MPa.

Figure A.3.4.3-3 Test results of the shear, tensile and flexure specimen *(Issa et al 2003)*

Table 8. Coulomb permeability test results.

Material type	Average Coulomb value	Chloride ion permeability
Set 45	606	Very low
Set Grout	2544	Moderate
Polymer concrete	22	Negligible

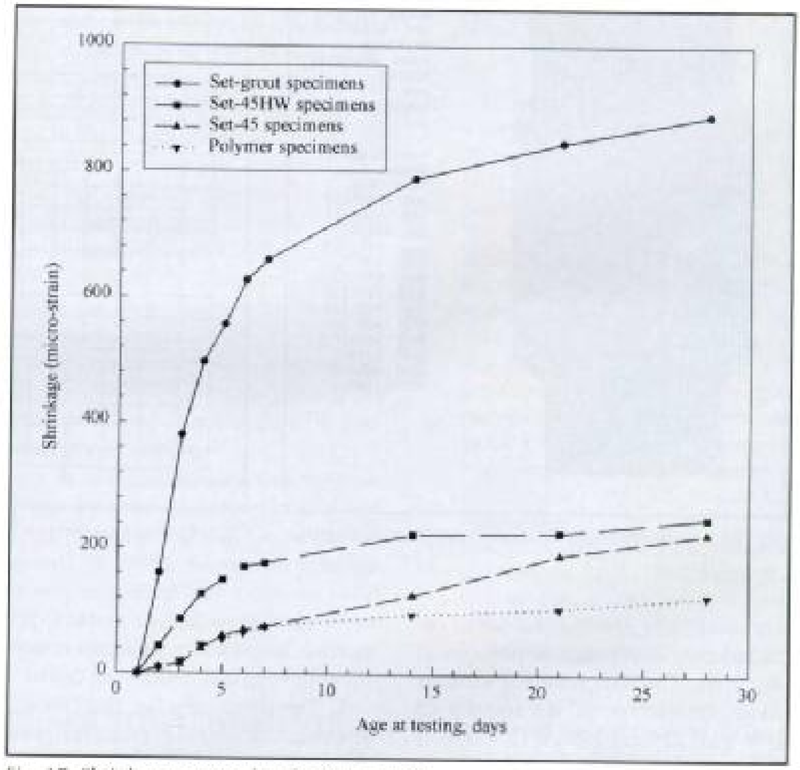


Fig. 17. Shrinkage test results of joint materials.

Figure A.3.4.3-4 Test results of the permeability and shrinkage *(Issa et al 2003)*

__Grout Material filling the Haunch between Girders and Panels:__

Menkulasi and Roberts-Wollman (*28*) have recently conducted an experimental investigation using two types of grout material. These are LMC and SET 45 HW, where angular pea gravel filler was added for both types. The test included only direct shear specimens that simulated precast concrete panels supported on prestressed concrete girders, as shown in Figure A.3.4.3-5. Three specimens with different amount of reinforcement crossing the interface were used, no reinforcement, 1#4 (1#M13) bar and 1#5 (1#16) bar. The height of the haunch used in all specimens was 1.0 in. (25.4 mm).

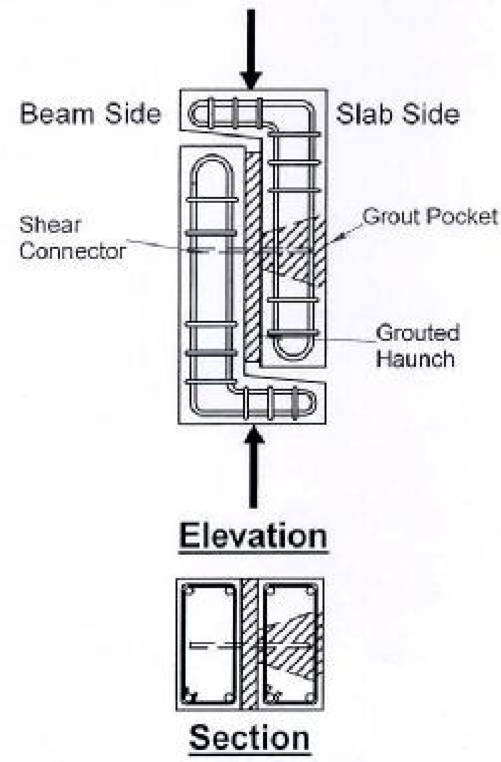


Figure A.3.4.3-5 Push-off test specimen (*Menkulasi and Roberts-Wollman 2005*)

The experimental investigation revealed that specimens made SET 45 HW and LMC had almost the same shear capacity when no or small amount of shear reinforcement was presented. However, at high amount of shear reinforcement, the specimens made with SET 45 HW showed higher strength than those made with LMC. The researchers were in favor of using SET 45 HW over LMC as the recommended grout material.

The experimental investigation also showed that changing the height of the haunch from 1.0 to 3.0 in. (25.4 to 76 mm) had almost no effect on the shear capacity of the specimens made with SET 45 HW grout.

A.4 ACKNOWLEDGEMENT

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APPENDIX B

DESIGN CALCULATIONS OF THE PROPOSED SYSTEM CD-1

APPENDIX B

DESIGN CALCULATIONS OF PROPOSED SYSTEM CD-1

(References that are used in this appendix are listed at the end of the appendix)

B.1 DESIGN CRITERIA

The following general criteria have been set in advance to pave the way for the development of this system. Please, note that these criteria have been set after careful study of the bridges covered in the literature review and discussing these criteria with a panel of national experts on this type of construction.

1. Type of superstructure: The slab/I girder bridge type has been used. This decision has been made based on the fact that approximately 50 to 60 percent of the bridges in USA are made of this type, according to National Bridge Inventory (*1*).
2. Construction material: The deck slab is made from conventionally or prestressed reinforced concrete. The supporting I-girder can be made of concrete or steel.
3. Composite versus non-composite superstructure: It was an evident from the literature review that the superstructure of the majority of bridges built with this system is made composite with the deck. Typically, composite systems have many advantages over non-composite system. These advantages include: (1) shallower depth of the superstructure, (2) longer spans, (3) smaller deflection and less vibration due to moving traffic and (4) larger clearance.
4. New construction projects versus deck replacement projects: The details of the precast deck system presented in this chapter have been developed to fit new construction projects as well as deck replacement projects. This decision has been made because there is almost a 50/50 percent split between new construction and deck replacement project nation wide.
5. Design Specifications: The 3rd Edition of the AASHTO LRFD Bridge Design Specifications (2004) with the 2005 & 2006 Interim Revisions (*2*) are used.

B.2 EVOLUTION OF A FULL-DEPTH PRECAST CONCRETE DECK PANEL SYSTEM

In order to develop a full depth precast concrete deck panel system, there is a need to determine the straining action, such as bending, and the amount of reinforcement required to resist these actions. Therefore, the research team has developed a model bridge and used it through out the design calculations of the system. The model bridge has the following criteria:

Total width	44 ft (two-lane, undivided two-way bridge)
Superstructure	Four steel girders spaced at 12 ft with top flange width of the steel girders = 12 in. to 14 in.
	OR
	Four BT-72 or NU1800 prestressed precast concrete girders space at 12 ft.

- Please, note that steel girders and 12-ft girder spacing is chosen to provide extreme straining actions in the deck, and consequently, the highest amount of reinforcement

- The 12-ft girder spacing has been chosen because it is the maximum girder spacing currently used in the states.

Deck slab Structural slab thickness = 8 in.

- Section 9.7.5.2 of the LRFD Specifications (*2*) states that the depth of a precast concrete slab excluding any provisions for grinding, grooving, and sacrificial surface, should not be less than 7.0 in.
- Minimum cover shall be in accordance with the provisions of section 5.12.3 of LRFD Specifications, which is 2.0 in.

The panel dimensions are considered as follow:

44-ft in length to cover the full width with one panel and 8-ft in width. The 8-ft dimension has been chosen because it allows shipping of the panels on trailers without the need of special permit.

Concrete properties:

- Unit weight = 150 pcf
- Concrete compressive strength at 28-day, $f_c^{'}$ = 6.0 ksi

  Typically, $f_c^{'}$ = 6.0 ksi can be easily achieved with precast concrete elements because the production is made in controlled environment and express curing methods.

- Concrete strength at release, $f_{ci}^{'}$ = 5.0 ksi

  Typically, with steam curing, $f_{ci}^{'}$ = 0.8 to 0.85 $f_c^{'}$ can be achieved in 18 to 24 hours.

Design specifications: AASHTO LRFD Specifications (*2*):

- Vehicular live loading on the roadway of bridges or incidental structures, designated HL-93, consist of a combination of the:
  1. Design truck or design tandem, and
  2. Design lane load with impact effect.
- Each design lane under consideration shall be occupied by either the design truck or tandem, coincident with the lane load, where applicable. The loads shall be assumed to occupy 10.0 ft transversely within a design lane.

Future wearing surface:

- 2-in. of concrete wearing surface, 150 pcf.

  Although, one of the main goals of this project is to develop deck systems with no overlays, the design calculations developed in this section have considered the use of a future overlay for the sake of completeness of the design calculations. In case of no overlay is used, this load should not be considered in the design.

The research team is aware that there are other types of overlays that can be used (such as a thin layer of epoxy overlay, asphalt concrete overlay, and latex concrete overlay). The 2-in., 150 pcf concrete overlay is considered in this study because it is the heaviest overlay among all types of overlays that can be used.

Side barriers: NJ Barriers, 420 plf, the barrier is 16 in. wide at bottom and 42 in. high. The center of gravity of the barrier is at 5.1 in. from the exterior face.

Reinforcement type: The precast panel is transversely pretensioned and longitudinally conventionally reinforced.

Pretensioned strands:

½ in. diameter, 270 ksi, Low Relaxation, 7 wire strands

Conventional reinforcement:

Grade 60 ASTM steel

Composite system: The precast panel is made fully composite with supporting girders. Two cases are considered as follows:

1. Steel girders; where composite action is created by welding 1 ¼" steel studs on top surface of the girders. The studs are embedded in the panel in a prefabricated shear pockets.
   - Shear studs used in composite steel bridge construction are typically ¾ in. or 7/8 in. in diameter. However, in this report the 1¼ in. diameter steel stud is used (*3*). The 1¼-in. stud has about twice the strength and a higher fatigue capacity than the 7/8-in. studs. The research team has decided to use 1¼-in. stud in this project for the following reasons: (1) fewer studs are required along the length of the steel girders, (2) higher construction speed, (3) and reduced possibility of damage to the studs and girder top flange during deck removal.
2. Concrete girders; there are two issues involved in the design.
   - Shear connectors extending outside the top flange are used.

B.3 DESIGN OF VARIOUS ELEMENTS OF THE PRECAST DECK PANEL SYSTEM

In order to develop the precast deck panel system, the following elements need to be designed in the following order:

- Design of the positive moment areas between girderlines (see section B.3.1)
- Design of the panel–to–girder connection for full composite action (see section B.3.2)
- Design of the negative moment areas over interior girderlines (see section B.3.3)
- Design of overhang part of the panel (see section B.3.4)
- Design of the longitudinal reinforcement (see section B.3.5)

- Design of panel–to–panel transverse connection (see section B.3.6)

Final details of the precast deck panel system are given in Figures B-1 to B-7.

B.3.1 Design of the positive moment areas between girderlines

Section 4.6.2.1.1 of the LRFD Specifications (*2*) states that the deck slab can be analyzed by subdividing it into strips normal to the supporting girders. This method is called the "Strip Method". Also, section 4.6.2.1.1 states that wherever the strip method is used, the extreme positive moment in any deck panel between girders shall be taken to apply to all positive moment regions. Similarly, the extreme negative moment over any beam or girder shall be taken to apply to all negative moment regions.

The deck slab is then analyzed as a continuous beam supported by the supporting girders. The girders are considered as non-settled supports and their width is taken equal to zero. A 12-in. wide strip is considered in the following calculations.

Loads applied on the structural model are as follow:

DC: Dead loads due to

Slab self weight = (8 /12) x 0.150 = 0.100 k/ft^2 (uniformly distributed load)

Barrier self weight = 0.420 k/ft/side (concentrated load)

DW: Dead load due to

2 in. concrete wearing surface = (2/12) x 0.150 = 0.025 k/ft^2

LL: Live load HL-93 due to truck load and lane load with dynamic allowance

Design Limit States and Load Factors (*2*):

1. Strength I:

Strength I limit state shall be taken to ensure that strength and stability, both local and global, are provided to resist the specified statically significant load combination relating to the normal vehicular use of the bridge without wind.

DC: Minimum = 0.90, Maximum = 1.25

DW: Minimum = 0.65, Maximum = 1.50

LL: 1.75

2. Service I:

Service I limit state shall be used for checking deflection and to control crack width in reinforced concrete structures:

DC: 1.00

DW: 1.00

LL: 1.00

3. Service III:

Service III limit state shall be used for checking tension in prestressed concrete structures with the objective of crack control:

DC: 1.00

DW: 1.00

LL: 0.80

Figure B-1 shows the service load moment due to DC and DW.

B.3.1.1 Estimate required prestress force

Investigation of the bending moment (Figure B-1) shows that the midspan section of the center span controls the design, where:

Slab wt.	M_{slab}	= 0.520 ft-k/ft
Barrier wt.	$M_{barrier}$	= 0.300 ft-k/ft
Wearing surface	M_{ws}	= 0.130 ft-k/ft

Moment due to live load can be determined using the equivalent strip on which the wheels of the 32-kip axle of the design truck will be distributed. In this case, various combinations of one, two or three trucks with the proper multi-presence factor should be considered to get the maximum moment effects. However, Table A4.1-1 of the LRFD Specifications gives the maximum moment effect based on girder spacing. Please, refer to sections 3.6.1.3.3, 4.6.2.1.2 and Appendix A4 of the LRFD Specifications.

Live load M_{LL+IM} = 8.01 ft-k/ft

In order to estimate the number of strands, assume that the tensile stresses at the extreme tension fibers, f_b, of the cross section controls the design, where

$$f_b = \frac{P_{pe}}{A} - \frac{(M_{slab} + M_{ws} + M_{barrier} + 0.8M_{LL+IM})}{S_b}$$

Therefore, at Limit State III:

$M_{service\ III}$ $= 1.0\ M_{DC} + 1.0\ M_{DW} + 0.8\ M_{LL+IM}$

$= 1.0\ (0.520 + 0.3) + 1.0\ (0.110) + 0.8\ (8.01)$

= 7.338 ft-k/ft

= 58.704 ft-k/panel

Assume ½" diameter, with f_{pu} = 270 ksi are used to estimate the required number of strands. Assume the initial prestress just before cutting the strands

$= 0.75f_{pu} = 0.75 \times 270 = 202.5$ ksi

Assume the total prestressed losses (i.e. elastic shortening, creep, shrinkage and prestress loss) at service = 15%. Therefore, the effective prestress in the strands at service,

$f_{pe} = (0.75 \times 270\text{ ksi})\ (1-0.15) = 172.125$ ksi

In order to avoid having the panel deflected upward or downward after releasing the prestress force, two layers of strands will be used with their centroid concentric with the centroid of the panel cross section. Therefore, the strand eccentricity, e_p = zero.

Effective prestress force P_{pe}= (A_P x 172.125) kips

Allowable tensile stress in pretensioned members (LRFD Sec. 5.9.4.2)

$$= 0.19\sqrt{f_c'} = 0.19\sqrt{6.0} = 0.465\ ksi$$

$$-0.465 = \frac{P_{Pe}}{8x12x8} - \frac{(0.520 + 0.11 + 0.300 + 0.8x8.01)(8)(12)}{(8x12)(8^2)/6}$$

P_{pe} = 171.41 kips

n x 0.153 x 172.125 = 171.41

n = 6.51 strands

Use eight ½ in. diameter, 270 ksi strands per panel, placed on two layers, four strands per layer. For each layer, provide 2-in. clear concrete cover.

Because, this estimate is based on only satisfying the service tensile stresses in concrete, it is needed to finalize the design of maximum positive moment section by:

1. Determining the prestress losses.
2. Checking the service compressive stresses in concrete.
3. Check the design moment capacity of this section due to Strength I Limit State.

Typically, this procedure is an iterative procedure because any change in the design parameters (such as the total prestress losses at service, amount of prestressing reinforcing, and/or adding conventional reinforcement) will affect all other aspects of design.

B.3.1.2 Prestress losses

LRFD Specifications provide two methods for computation of prestress losses, which are the detailed method and the lump-sum method. The detailed method is used here as it provides more accurate measure of prestress losses.

Total prestress losses is:

$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR2}$ (LRFD Eq. 5.9.5.1-1)

Where,

Δf_{pES} = loss due to elastic shortening

Δf_{pSR} = loss due to shrinkage

Δf_{pCR} = loss due to creep

Δf_{pR2} = loss due to relaxation of steel after transfer

<u>Elastic shortening:</u>

$\Delta f_{pES} = (E_p / E_{ci})\ f_{cgp}$ (LRFD, Sec. 5.9.5.2.3a)

Where

E_p = modulus of elasticity of prestressing strands = 28,500 ksi

f'_{ci} = concrete strength at release = 5.0 ksi

E_{ci} = modulus of elasticity of slab at release

$= 33,000(150)^{1.5}\sqrt{5.0}$ = 4,287 ksi

f_{cgp} = sum of concrete stresses at center of gravity of prestressing strands due to prestressing force at transfer and the self-weight of the member at sections of maximum moment.

The *LRFD Specifications*, Art.5.9.5.2.3a, states that f_{cgp} can be calculated on the basis of prestressing steel stress assumed to be $0.7f_{pu}$ for low-relaxation strands. However, common practice assumes the initial losses as a percentage of initial prestressing stress before release, fpi. In both procedures, assumed initial losses should be checked and if different from assumed value, a second iteration should be carried out. In this document, 1% f_{pi} initial loss is used.

Force per strand at transfer = 0.75 x 270 x 0.153 x (1-0.01) = 30.673 kips

The strand group is concentric with the panel cross section, therefore $f_{cgp} = \frac{P_i}{A}$

P_i = total prestressing force at release = 8 strands x 30.673 = 245.384 kips

f_{cgp} = 245.384/768 = 0.320 ksi

Therefore, loss due to elastic shortening:

$\Delta f_{pES} = \frac{28,500}{4,287}(0.320)$ = 2.127 ksi

Percent actual loss due to elastic shortening = (2.12/202.5) x 100 = 1.04%, which is very close to the assumed value, so second iteration is not necessary.

Shrinkage:

Δf_{pSR} = (17-0.15H) *(LRFD Eq. 5.9.5.4.2-1)*

Where H = relative humidity (assume 70%),

Relative humidity varies significantly from one area of the country to another, see Table 5.4.2.3.3-1 in the *LRFD Specifications*.

Δf_{pSR} = 17-0.15(70) = 6.5 ksi

Creep of concrete:

$\Delta f_{pCR} = 12\ f_{cgp} - 7\ \Delta f_{cdp}$ (LRFD Eq. 5.9.5.4.3-1)

Where Δf_{cdp} = change of stresses at center of gravity of prestressing due to permanent loads, except dead load acting at time the prestress force is applied calculated at the same section as f_{cgp}

The strand group is concentric with the panel cross section, therefore Δf_{cdp} = zero

Δf_{pCR} = 12 x 0.320 = 3.84 ksi

Relaxation of strands:

Initial loss due to relaxation of prestressing steel is accounted for in the slab fabrication process. Therefore, loss due to relaxation of the prestressing steel prior to transfer is not be computed, $\Delta f_{pR1} = 0$. Recognizing this for pretensioned members, the LRFD Specifications (*2*), Sec. 5.9.5.1, allows the portion of the relaxation loss that occurs prior to transfer to be neglected in computing the final loss.

For low-relaxation strands, loss due to relaxation after transfer (LRFD, Sec. 5.9.5.4.4c):

Δf_{pR2} = 30 %$\{20 - 0.4\ \Delta f_{pES} - 0.2(\Delta f_{pSR} + \Delta f_{pCR})\}$

= 0.3{20 – 0.4(2.127) – 0.2(6.500 + 3.840)}= 5.125 ksi

- *Total losses at transfer,* $\Delta f_i = \Delta f_{pES}$ = 2.127 ksi

  Stress in tendons after transfer,

  $f_{pT} = f_{pi} - \Delta f_i$ = 202.5 – 2.127 = 200.373 ksi

  Force per strand = f_{pT} x area of strand = 200.373 x 0.153 = 30.660 ksi

- *Total losses at service loads:*

  $\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR2}$

  = 2.127 + 6.500 + 3.840 + 5.125 = 17.592 ksi

  Stress in tendons after all losses, $f_{pe} = f_{pi} - \Delta f_{pT}$ = 202.5 – 17.585 = 184.908 ksi

  Check prestressing stress limit at service limit state: Table 5.9.3-1 of the LRFD Specifications states that $f_{pe} \leq 0.8\ f_{py}$, therefore:

  $f_{py} = 0.9 \times f_{pu}$ = 0.9 x 270 = 243 ksi

  f_{pe} = 184.908 ksi ≤ 0.8 f_{py} = 0.8(243) = 194.4 ksi O.K.

  Final prestress losses, % = ($\Delta f_{pT} / f_{pi}$) = (17.592 x 100)/ 202.5 = 8.68 %

B.3.1.3 Check of concrete stresses at service loads at the positive moment area

The total prestressing force after all losses,

P_{pe} = 202.5 x 0.153 x 8 x (1 – 0.0868) = 226.350 kips

Stress limits for concrete: (LRFD Sec. 5.9.4.2)

- Compression:

  Due to permanent and transient loads (i.e. all dead loads and live loads), limit state Service I = 0.6 $f_c^{'}$ = 0.6 x 6.0 = + 3.6 ksi

- Tension:

  For components with bonded prestressing tendons, limit state Service III

  $$= -0.19\ \sqrt{f_c^{'}} = -0.19\sqrt{6.0} = -0.465\ ksi$$

- Concrete stress at the top fiber of the deck:

  Under permanent and transient loads, Service I:

$$f_t = +\frac{P_{pe}}{A} + \frac{(M_{slab} + M_{barrier} + M_{ws} + M_{LL+IM})}{S_t}$$

$$= \frac{226.350}{768} + \frac{(0.520 + 0.110 + 0.300 + 8.010)(8)(12)}{(8x12)(8^2)/6 = 1024} = +\ 1.133 \text{ ksi}$$

  Compressive stress limit: + 3.6 ksi O.K.

- Service tensile stress in bottom of deck, Service III:

$$f_t = +\frac{P_{pe}}{A} - \frac{(M_{slab} + M_{barrier} + M_{ws} + 0.8M_{LL+IM})}{S_b}$$

$$= \frac{226.350}{768} - \frac{(0.520 + 0.110 + 0.300 + 0.8x8.010)(8)(12)}{(8x12)(8^2)/6 = 1024} = -\ 0.393 \text{ ksi}$$

  Tensile stress limit: -0.465 ksi O.K.

B.3.1.4 Check of flexural strength

Total moment due to Strength I Limit State (LRFD, Section 3.4.1),

$M_{strength\ I}$ = 1.25 M_{DC} + 1.5 M_{DW} + 1.75 M_{LL+IM}

= 1.25 (0.520 + 0.300) + 1.5 (0.110) + 1.75 (8.010)

= 15.208 ft-k/ft

= 121.66 ft-k/panel

The design procedure given by the LRFD Specifications (Section 5.7.3.1.1) can not be used with this case because the LRFD procedure deals with the strands as they are lumped at their gravity, while in this case we have two layers of strands that are far away from each other, one layer is close to the top fiber and the second layer is close to the bottom layer. Therefore, a more detailed analysis should be carried out using the strain compatibility approach and the power stress-strain formula of the prestressing and conventional reinforcement.

Assume that depth of rectangular stress block, a, less than < 2.25 in. and that all layers of strands are on the tension side of the N.A. and that the stress in the strands is f_{py}, except the top layer which is very close to the N.A. assume the stress equal f_{pe} = 184.908 ksi.

From equilibrium of forces, T = C

Where: T = Tension force in strands = A_{ps} x f_{ps}

C = Compression force in concrete = 0.85 x f_c' x b x a

b = Width of section, b = 8x12 = 96 in.

(4 x 0.153 x 184.908) + (4 x 0.153 x 243) = (0.85 x 6 x 8 x 12 x a)

Depth of rectangular stress block, a = 0.535 in.

Distance from top of section to neutral axis, $c = a/\beta_1$

$\beta_1 = 0.85 - 0.05(f'_c - 4) = 0.85 - 0.05(6 - 4) = 0.75$

$c = 0.535/0.75 = 0.713$ in.

Check the assumed stresses in each layer of strands:

- *Bottom layer:*

Depth of bottom layer = 5.75 in., and decompression stress, f_{pe} = 184.908 ksi

$\varepsilon_p = 0.003((5.75\text{-}c)/c) + (184.908/28500)$

$= 0.003\ ((5.75\text{-}0.713)/0.713) + 0.00649 = 0.0277$

Based on the power stress-strain formula developed by Devalapura and Tadros, (*4*)

$$f_{ps} = \varepsilon_p \left[887 + \frac{27{,}613}{\left\{1 + \left(112.4\varepsilon_p\right)^{7.36}\right\}^{1/7.36}} \right] \le 270\ ksi$$

$$f_{ps} = 0.0277 \left[887 + \frac{27{,}613}{\left\{1 + \left(112.4x0.0277\right)^{7.36}\right\}^{1/7.36}} \right] = 270.2\text{ ksi} > 270\text{ ksi}$$

Therefore, f_{ps} = 270 ksi

- *Top layer:*

Depth of bottom layer = 2.25 in., and decompression stress, f_{pe} = 184.908 ksi

$\varepsilon_p = 0.003((2.25\text{-}c)/c) + (184.908/28500)$

$= 0.003\ ((2.25\text{-}0.713)/0.713) + 0.00649 = 0.0130$

$$f_{ps} = 0.0130 \left[887 + \frac{27{,}613}{\left\{1 + \left(112.4x0.0130\right)^{7.36}\right\}^{1/7.36}} \right] = 255.140\text{ ksi} < 270\text{ ksi}$$

Therefore, f_{ps} = 255.140 ksi

A second round of calculations is required because the final stresses in the strands do not match the assumed values. In this round, assume that the final stress at the top layer of strands = 256.038 ksi and at the bottom layer of strands = 270.000 ksi. This iterative process should be carried out until the assumed values match the calculated values. Results of the final round of calculations are as follow:

Depth of rectangular stress block, a = 0.646 in.

Depth of the neutral axis, c = 0.862 in

Strain in top layer of strands = 0.01132 (tension)

Stress in top layer of strands = 250.49 ksi (tension)

Strain in bottom layer of strands = 0.02350 (tension)

Stress in bottom layer of strands = 266.44 ksi (tension)

The flexural capacity of the section:

ϕM_n = ϕ (A_{ps} x f_{ps})(d_p-a/2)

$$= 1.0\left\{(4x0.153x250.49)(2.25 - \frac{0.646}{2}) + (4x0.153x266.44)(5.75 - \frac{0.646}{2})\right\}$$

= 1,180.286 in.-k/panel

= 98.35 ft-k/panel < $M_{strength\ I}$ = 121.66 ft-k/panel NG

- Because the flexural capacity is not safe, add 4-#5 bars in each layer of reinforcement with a 2-in. clear concrete cover over the #5 bars. Assume that the decompression stress in the top and bottom #5 bars = -25 ksi.

  Results of the final round of calculations are as follow:

  Depth of rectangular stress block, a = 0.908 in.

  Depth of the neutral axis, c = 1.21 in

  Strain in top layer of strands = 0.00906 (tension)

  Stress in top layer of strands = 233.64 ksi (tension)

  Strain in top layer of #5 bars = 0.0187 (tension)

  Stress in top layer of #5 bars = 54.17 ksi (tension)

  Strain in bottom layer of strands = 0.0177 (tension)

  Stress in bottom layer of strands = 261.16 ksi (tension)

  Strain in top layer of #5 bars = 0.01023 (tension)

  Stress in top layer of #5 bars = 60.00 ksi (tension)

The flexural capacity of the section:

ϕM_n = ϕ [(A_{ps} x f_{ps})(d_p-a/2) + (A_s x f_s)(d_s-a/2)]

$$= 1.0\left\{(4x0.153x233.64)(2.25 - \frac{0.908}{2}) + (4x0.153x261.16)(5.75 - \frac{0.908}{2})\right\}$$

$$+1.0\left\{(4x0.31x54.17)(2.3125 - \frac{0.908}{2}) + (4x0.31x60)(5.6875 - \frac{0.908}{2})\right\}$$

= 1617.485 in-k/panel

= 134.79 ft-k/panel < $M_{strength\ I}$ = 121.66 ft-k/panel OK

B.3.1.5 Check of maximum reinforcement limit

The maximum amount of reinforcement must be such that $\frac{c}{d_e} \leq 0.42$

(AASHTO LRFD Eq. 5.7.3.3.1-1)

However, this equation is developed based on the condition that all the tensile reinforcement is lumped at one point close to the extreme tensile fiber of the section, which is satisfied in this case. A better approach to check the maximum reinforcement limit in the present case is to make sure that the extreme tensile layer of reinforcement (the bottom layer) will yield before the concrete reaches its maximum compressive failure strain (i.e. 0.003). By checking the stresses and strains in the bottom layer of reinforcement, it is found that both types of reinforcement have passed the yield point before concrete reaches its ultimate strain. This gives an indication that the section will show wide cracks and large amount of deflection before failure occurs (i.e. ductile failure).

B.3.2 Design of panel–to–girder connection for full composite action

As discussed in the literature review submitted in the interim report of this project, shear connectors that are fully anchored with the girders are extended into the deck panel to create the full composite action between the deck panels and the girders. Typically, this is achieved by creating shear pockets in the panel over every girderline that accommodate the shear connectors. Also, the shear connectors have to be clustered in groups to match the locations of these shear pockets. As a rule of thump for precast concrete production, the fewer the number of shear pockets that a precast panel has, the less expensive the panel will be due to savings on time and labor that are associated with forming for these pockets.

As discussed in the Interim Report, QPR3 and QPR4 of this project, the research team has adopted the idea of extending the maximum spacing of clustered group of shear connectors to 48 in. This spacing is used in the design calculations presented in the section.

In order to determine the size of the shear pockets of the precast panel, either for use with steel or precast concrete girders, it is required to determine the amount of shear connector reinforcement. To do that, the research team has used the following resources:

Bridges built with steel girders:

As stated in the design criteria, 1¼ in. diameter studs (*3*) are used as the shear connectors. The use of this size of studs for this system is advantageous because one 1¼ in. diameter stud replaces two 7/8 in. studs, which will minimize the size of the shear pockets.

In a study conducted in NCHRP 12-41 (*5*), the researchers ran a parametric study for the horizontal shear requirement for a wide range of simply supported bridges, where the span length ranged from 40 to 130 ft and the girder spacing ranged from 6 to 12 ft. The researchers found that using 1¼ in. studs uniformly spaced at 6 in. throughout the span of the bridge would sufficiently satisfy the horizontal shear requirements.

Therefore, for the precast panel system under development, if the clusters of studs will be spaced at 48 in., this means that each cluster will have 8-1¼ in. studs. Using two studs per rows, and the stud rows spaced at 3 in., the dimensions of each shear pockets will be 12 in. (in the transverse direction) and 15 in. (in the longitudinal direction).

Bridges built with precast concrete girders:

For precast concrete girders, typically, the vertical shear reinforcement is extended outside the top flange to provide the required horizontal shear reinforcement. The vertical shear reinforcement usually takes an L-shape or an inverted U-shape to provide anchorage and fully develop the yield strength of the reinforcement. Although this detail provides an inexpensive way to provide for the composite action, it does not fit precast deck panels where the shear connectors have to be extended outside the top flange of the girder only at the locations of the shear pockets. For this reason, it is recommended to separate the vertical shear reinforcement of the precast girder and the horizontal shear reinforcement required for full composite action.

To determine the amount of reinforcement required per shear pocket for both types of shear connectors, the research team studied the design examples provided in the PCI-Bridge Design Manual (*6*). Six design examples of slab/I-girder bridge systems are given in this reference, where the bridge structures range from simply supported spans to three continuous span structures, with a span length up to 120 ft and girder spacing from 9 to 12 ft. Studying these examples reveals that the maximum horizontal factored shear force at the interface between the deck slab and the precast concrete girders is about 3.7 kip/in. of the longitudinal direction of the girder.

Therefore, the required horizontal nominal shear strength =

V_n = (3.71 kip/in.)(8x12 in.)/(ϕ = 0.9) = 396 kips/panel

Two types of shear connectors can be used:

(1) Individual inverted #5 U-bars that are embedded in the girder top flange and extended into the panel shear pockets. The inverted U-bars are clustered at 48 in. This detail has been used successfully in bridges in Nebraska (*7*).

Using 6- #5 U-bars per pocket spaced at 7 in., two U-bars per row, the pocket dimension will be 12 in. wide (in the transverse direction) and 24 in. long (in the longitudinal direction).

The nominal shear resistance of the interface plane is:

$V_n = c\, A_{cv} + \mu\, A_{vf}\, f_y$ (LRFD Eq. 5.8.4.1-1)

Where:

c = cohesion factor = 0.1 ksi for concrete placed against clean, hardened concrete with surface intentionally roughened (LRFD Art. 5.8.4.2)

μ = friction factor = 1.0 for concrete placed against clean, hardened concrete with surface intentionally roughened (LRFD Art. 5.8.4.2)

A_{cv} = area of concrete engaged in shear transfer

= (12 in. x 24 in.)(2 pockets) = 576 in^2

f_y = yield strength of the shear reinforcement

A_{vf} = area of shear reinforcement crossing the shear plane

= (6 U-bars x 2 legs x 0.30 in^2/leg)(2 pockets) = 7.20 in^2/panel

V_n = (0.1 ksi)(576 in^2) + 1.0(7.20)(60 ksi)

=489.6 kip/panel > 396 kips/panel OK

(2) Use of clustered 3-1¼ in. diameter double head studs per pocket, spaced at 4 in., one stud per row. The studs are made from the same material used to fabricate the 1¼ in. studs used with steel girders, SAE 1018 with ultimate tensile strength 64 ksi. The pocket dimensions are 12 in. wide (in the transverse direction) and 15 in. long (in the longitudinal direction).

This detail was experimental investigation by the research team, where it shows that this detail was able to develop the yield strength of the 1¼ in. double-headed studs. However, additional web reinforcement is required. Please, see Chapter 3 of this report for more information on the experimental investigation.

A_{cv} = area of concrete engaged in shear transfer

= (12 in. x 16 in.)(2 pockets) = 384 in^2

A_{vf} = area of shear reinforcement crossing the shear plane

$= (\pi \frac{1.25^2}{4})$(3 studs per pocket)(2 pockets) = 7.38 in^2/panel

V_n = (0.1 ksi)(384 in^2) + 1.0(7.38)(54 ksi)

= 436.9 kip/panel > 396 kips/panel OK

This detail is going under experimental investigation by the research team.

B.3.3 Design of the negative moment areas over interior girderlines

Section 4.6.1.2.6 of the LRFD Specifications states that the critical section for flexural design at the negative moment area should be at a distance "χ" from the centerline of the support, where "χ" for slabs supported on steel girders

= Least of 15" and 1/4 (the width of the flange of the steel girder)

while "χ" for slabs supported on steel girders = ¼ the width of the flange of the steel girder

Steel girders are considered here as it provides higher straining actions. Assuming that the minimum width of steel girder top flange can be as small as 12 in., therefore, $\chi = 12/4 = 3$ in. Bending moment at 3 in. from the centerline of interior support is as follow:

Slab wt. M_{slab} = 0.520 ft-k/ft

Barrier wt. $M_{barrier}$ = 0.300 ft-k/ft

Wearing surface M_{ws} = 0.130 ft-k/ft

Live load M_{LL+IM} = 9.400 ft-k/ft (Table A4.1-1, LRFD Specifications)

$M_{strength\ I}$ = 1.25 (DC) + 1.5 (DW) + 1.75 (LL+IM)

= 1.25(1.133) + 1.5(0.305) + 1.75(9.40)

= 18.32 ft-k/ft

= 146.59 ft-k/panel

Try two layers of reinforcement, top layer has 4–½ in. strands and 6#5 bars and the bottom layer has 4–½ in. strands and 4#5 bars. Provide a 2.0 in. clear concrete cover over each layer. The decompression stress in the strands, f_{pe} = 184.908 ksi, and in the #5 bars = -25 ksi. Using the method of Strain Compatibility as shown before yields:

The flexural capacity of the section:

ϕM_n = 1,795.901 in-k/panel = 149.7 ft-k/panel < $M_{strength\ I}$ == 146.59 ft-k/panel OK

Although, the maximum negative moment at the interior supports dies very quickly on both sides of the girderline, the design engineer has opted to provide the 6 bars on both layers for the full width of the panel. This will simplify the fabrication process.

B.3.4 Design of the overhang (negative moment section at exterior girderline)

According to Section A13.4.1 of the LRFD Specifications (*2*), the overhang should be designed for the following cases separately:

B.3.4.1 Case I: Due to transverse vehicular collision loads using Extreme Event Limit State II

Because the New Jersey Barrier adopted in this document is crash tested and the LRFD Specifications state that the deck should be stronger than the railing system used, the collision moment and the horizontal collision force will be determined based on the reinforcement and geometry of the New Jersey Barrier as follows:

The base of the NJ Barrier is 16 in. wide and reinforced with a two layers of #5 bars @ 12 in. One layer is close to the inner face of the barrier and the second layer is close to the exterior face of the barrier. Using a 2-in. clear concrete cover over each layer and using the strain compatibility analysis procedure, the nominal flexural capacity of the section,

$M_{n\ base}$ = 280.6 in-k/ft

Section 1.3.2.1 of the LRFD Specifications (*2*) states that the strength reduction factor for the Extreme Event Limit State, ϕ = 1.0. Therefore, the flexural capacity of the base section is,

$\phi M_{n\ base}$ = 1.0 x 280.6 in-k/ft = 23.38 ft-k/ft

In order to complete the design of the overhang, it is required to determine the collision force applied at the top level of the barrier, R_W, and the distance over which this force is distributed, L_C. Typically, these parameters depend on the barrier dimensions and failure mechanisms. The research team consulted with the following publication, "FHWA HI-95-017," of the National Highway Institute (*8*) to determine these parameters. This publication uses a NJ Barrier identical to the NJ Barrier used in this document. From this publication, the values of R_W and L_C are as follow:

R_W = 147.03 kips, L_C = 13.589 ft

To determine the collision force at the base of the barrier, T_{base}, assume that R_w is distributed over a distance of (L_C + 2H) at the barrier base, where H is the height of the barrier, H = 42 in.

$T_{base} = R_W / (L_C + 2H)$ = 147.03 / [13.589 + (2x42/12)] = 7.14 k/ft

Therefore, due to the collision force, the following straining actions are transferred to precast panel at the inner face of the barrier:

$M_{collision}$ = 23.38 ft-k/ft and T_{base} = 7.14 k/ft (tension force)

Two sections of the overhang need to be checked. The first section is at the inner face of the barrier (Section 1-1, Figure B-8) and the second section is at the 3 in. from the centerline of the exterior girder (Section 2-2, Figure B-8).

<u>Check capacity of Section 1-1:</u>

$M_{collision}$ = 23.38 ft-k/ft

T_{base} = 7.14 k/ft

$M_{barrier.\ wt.}$ = (0.42)(16-5.2)/12 = 0.378 ft-k/ft

M_{slab} = $0.1 \times (16/12)^2/2$ = 0.09 ft-k/ft

$M_{Service\ I}$ = 23.3+0.378+0.09 = 23.8 k-ft

$M_{Extreme\ event\ II}$ = 1.25DC + 1.5DW + 1.0CT

= 1.25(0.378+0.09) + 1.0(23.38)

= 23.965 ft-k/ft

$T_{Extreme\ event\ II}$ = 1.0 x 7.14 = 7.14 kips/ft

Because Section (1-1) is 16 in. away from the edge of the panel, it is required to check the maximum strength that can be provided by the reinforcement based on the available development length. The reinforcement provided at this section is made of ½ in. strands and #5 Grade 60 bars.

For the ½ in. strands, the development length is,

$L_d = (f_{ps} - 2f_{pe}/3)\, d_b$ (LRFD, Sec. 5.11.4.2)

= (184.908/3)(0.5) + (259.98-184.908)(0.5)

= 68.4 in. = 5.7 ft

f_{pe} and f_{ps} are determined from the calculations of the negative moment section over interior girderlines.

$$\text{Available strength} = 259.98 \text{ ksi } \left(\frac{16-2}{68.4}\right) = 53.0 \text{ ksi}$$

It is assumed that the strands are recessed 2 in. from the edge of the panel to satisfy the corrosion protection requirements.

Based on eth available strength of the strands, it is clear that Section 1-1 can not be designed as a fully pretensioned section.

For the #5 Grade 60 bars, the development length is,

$$L_d = greater \left| \begin{matrix} \dfrac{1.25 A_b f_y}{\sqrt{f_c^{'}}} \\ 0.4 d_b f_y \end{matrix} \right\rangle \quad \text{(LRFD, Sec. 5.11.2)}$$

$$= greater \left| \begin{array}{l} \frac{1.25x0.31x60}{\sqrt{6.0}} = 9\ in. \\ 0.4x\frac{5}{8}x60 = 15\ in. \end{array} \right\rangle = 15\ in.$$

Assume that the #5 bars are epoxy coated, therefore, L_d = 1.2 x 15 = 18 in.

Available strength, f_s = 60 ksi ($\frac{16-2}{18}$) = 47.0 ksi

It is assumed that a 2 in. clear concrete cover is provided from the edge of the panel to the #5 bars to satisfy the corrosion protection requirements.

Based on eth available strength of the strands, it is clear that Section 1-1 should be designed as a partially pretensioned section. However, to simplify the calculations, the research team decided to design the section as a conventionally reinforced section. Try #5 at 6 in.

$T_{Extreme\ event\ II}$ = T – C

= (A_sxf_s) – (0.85 x f'_c x b x a)

7.14 = (2x0.31x47) – (0.85x6.0x12)(a)

a = 0.36 in.

d = 8 – 2 – $0.5x\frac{5}{8}$ = 5.6875 in.

ϕM_n = $\phi\{A_s$ x f_s (d-a/2) – T_u (d/2 – a/2)}

= 1.0 {2 x 0.31 x 47 (5.6875 - 0.36/2) – 7.14 (5.6875/2 - 0.36/2)}

= 141.5 in-k/ft

= 11.8 ft-k/ft < $M_{Extreme\ event\ II}$ = 23.965 ft-k/ft N.G.

Try #6 standard hook @ 5 in.:

A_s = 0.44 x $\frac{12}{5}$ = 1.056in^2

L_d = 1.2 x $\frac{38.0d_b}{\sqrt{f'_c}}$ = 1.2 x $\frac{38.0x\frac{3}{4}}{\sqrt{6.0}}$ = 13.96 in. (LRFD, Sec. 5.11.2.4)

f_s = 60 ksi ($\frac{16-2}{13.96}$) = 60.0 ksi

7.14 = (1.056 x 60.0) – (0.85 x 6.0 x 12)(a)

a = 0.919 in.

d = 8 – 2 – $0.5x\frac{6}{8}$ = 5.625 in.

ϕM_n $= \phi\{A_s \times f_s (d-a/2) - T_u (d/2 - a/2)\}$

$= 1.0 \{1.056 \times 60 (5.625 - 0.919/2) - 7.14 (5.625/2 - 0.919/2)\}$

$= 310.5$ in-k/ft

$= 25.9$ ft-k/ft $> M_{\text{Extreme event II}} = 23.965$ ft-k/ft OK

<u>Check capacity of Section 2-2:</u>

At the inside face of the barrier (Section 1-1), the collision effects are distributed over a distance L_c for moment $M_{collision}$ and $(L_c + 2H)$ for the axial tension force T_{base}. Assume that the effects will spread between sections 1-1 and 2-2 on a 30° angle. Therefore the collision effects at section 2-2 are:

$$M_{\text{collision 2-2}} = (M_{\text{collision 1-1}} \times L_c) / [L_c + (2 \times \frac{29}{12} \tan 30)]$$

$$= (23.38 \times 13.589) / (13.589 + 2 \times \frac{29}{12} \times \tan 30)$$

$$= 19.40 \text{ ft-k/ft}$$

Please, note that the distance between Section 1-1 and 2-2 = 29 in.

$$T_{\text{collision 2-2}} = R_W / (L_C + 2H + 2 \times 29 \tan 30)$$

$$= 147.03 / [\ 13.589 + (2 \times \frac{42}{12}) + (2 \times \frac{29}{12}) \tan 30\ ]$$

$$= 6.29 \text{ k/ft}$$

M_{slab} $= (8/12 \times 0.150)(3.75^2/2) = 0.70$ ft-k/ft

$M_{barrier}$ $= 0.42\,((45-5.2)/12) = 1.40$ ft-k/ft

M_{ws} $= ((2/12) \times 0.150)(2.417^2/2) = 0.073$ ft-k/ft

$M_{\text{Exreme event II}}$ $= 1.25DC + 1.5DW + 1.0CT$

$= 1.25(0.70 + 1.40) + 1.25(0.073) + 1.0(19.40)$

$= 22.17$ ft-k/ft

$T_{\text{Exreme event II}}$ $= 1.0(T_{col.}) = 1.0(6.26) = 6.29$ k/ft

At section 2-2, the strands are still not fully developed. To simplify the calculations, this section is designed as a conventionally reinforced section. If #6 standard hook @ 5 in. are used, the flexural design capacity of the section is,

$\phi M_n = 25.9$ ft-k/ft $> M_{\text{Extreme event II}} = 22.17$ ft-k/ft OK

B.3.4.2 Case 2: Due dead and live loads

Due to combined dead and live load, the flexural capacity of Section 2-2 should be checked. Load effects at Section 2-2 are as follow:

M_{slab} $= (8/12 \times 0.150)(3.75^2/2) = 0.70$ ft-k/ft

$M_{barrier}$ $= 0.42\,((45-5.2)/12) = 1.40$ ft-k/ft

M_{ws} = ((2/12) x 0.150) (2.417^2/2) = 0.073 ft-k/ft

Live load effects:

Section 3.6.1.3 of the LRFD Specifications (2) state that where primary strips are transverse and their span does not exceed 15.0 ft, the transverse strips should be designed for the wheels of the 32.0 kip axle. Also, the center of the outside 16.0-kip wheel is positioned 1 ft from the curb face for the design of the deck overhang.

Section 3.6.2.1.3 of the LRFD Specifications (2) state that the live load effects should be distributed over a distance, L (in.) = 45.0 + 10.0 X, where X (in.) = distance form the wheel load to the section under consideration = 17 in.

Live load moment, M_{LL+IM} = IM x m (16 X) / L

Where, m = multiple presence factor

= 1.20 (LRFD Specifications, Table 3.6.1.1.2-1)

IM = dynamic load allowance

= 1.33 (LRFD Section 3.6.2.1, Table 3.6.2.1-1)

$$M_{LL+IM} = 1.33 \text{ x } 1.2 \frac{16x\frac{17}{12}}{\left(45+10x\frac{17}{12}\right)\frac{1}{12}} = 7.34 \text{ ft-k/ft}$$

$M_{Strength\ I}$ = 1.25DC + 1.5DW + 1.75(LL+IM)

= 1.25(0.70 + 1.40) + 1.5(0.073) + 1.75(7.34)

= 15.58 ft-k/ft

At section 2-2, the strands are still not fully developed. To simplify the calculations, this section is designed as a conventionally reinforced section. If #6 standard hooks @ 5 in. are used, the flexural design capacity of the section is,

ϕM_n = 25.9 ft-k/ft > $M_{Strength\ I}$ = 15.58 ft-k/ft OK

B.3.4.3 Details of overhang reinforcement

Required overhang reinforcement = #6 standard hook, Grade 60 bars @ 5 in.

Required area of reinforcement per panel = 0.44 x $\frac{96\ in.}{5\ in.}$ = 8.5 in^2

At the top layer of reinforcement in the overhang, 4–½ in. strands and 6#5, Grade 60 bars are extended from the exterior span. Please, note that the ½ in. strands are not fully developed at the inner face of the barrier (available strength that can be developed by the strands at this section = 53.0 ksi).

Therefore, additional Grade 60 reinforcement = 8.5 – 4x0.153x$\frac{53}{60}$ - 5x0.31

$= 6.4 \text{ in}^2 = 15.6$ #6 standard hook, Grade 60 bars

Additional 15#6 standard hook, Grade 60 bars should be provided in the top layer of reinforcement in the overhang. These bars should extend 3 ft from the centerline of the exterior girderline into the exterior span. Please, note that the additional reinforcement of the overhang is not shown on the figures attached to this appendix. This is because the design of the overhang depends on the barrier types and its flexural capacity, which vary from one state to another. The calculations that are shown in this section are just for the completeness of the subject matter.

B.3.5 Design of longitudinal reinforcement

The longitudinal reinforcement in bridge deck slabs, where the main reinforcement is provided in the transverse direction, is typically provided for the following reasons:

1. To distribute the concentrated live load in the longitudinal direction.
2. To protect the deck slab from drying shrinkage cracking that may occur due to loss of hydration water.
3. In case of continuous span bridges, the deck slab houses the negative moment reinforcement that is needed at the vicinity of the intermediate supports (i.e. piers).

Because, the amount of the longitudinal reinforcement is affected by the structural system of the superstructure, whether it is a simply supported span or a continuous span, two cases are discussed in this section:

B.3.5.1 Longitudinal reinforcement for simply supported span bridges

The LRFD Specifications do not provide guidelines for determining the longitudinal reinforcement for precast panel systems. However, Section 9.7.3.2 of the LRFD Specifications, gives guidelines for deck slabs, which have four layers of conventional reinforcement in two directions. In this case, the amount of the longitudinal reinforcement is determined as percentage of the transverse reinforcement as follow:

$$\text{Longitudinal reinforcement} = (\text{Transverse reinforcement, } A_s) \text{ x } least\left[\begin{array}{l}\dfrac{220}{\sqrt{S}} \\ 67\%\end{array}\right)$$

Where S = the clear spacing between girders measured between the inner faces of two adjacent girders = 12 ft

$$\text{Longitudinal reinforcement} = (\text{Transverse reinforcement, } A_s) \text{ x } least\left[\begin{array}{l}\dfrac{220}{\sqrt{12}} = 63.5\% \\ 67\%\end{array}\right)$$

$$= (\text{Transverse reinforcement, } A_s)\ (63.5\%)$$

The distribution of longitudinal reinforcement should be provided close to the bottom of the slab. Because this empirical rule is developed for conventionally reinforced slabs, there is a need to determine A_s at the positive moment section as a conventionally reinforced section.

Slab wt. $M_{slab} = 0.520$ ft-k/ft

Barrier wt. $M_{barrier} = 0.300$ ft-k/ft

Wearing surface M_{ws} = 0.110 ft-k/ft

Live load M_{LL+IM} = 8.01 ft-k/ft

$M_{Strength\ I}$ = 1.25 DC + 1.5 DW + 1.75 (LL+IM)

= 1.25 (0.520 + 0.300) + 1.5 (0.110) + 1.75 (8.01)

= 15.2 ft-k/ft

Required conventional A_s can be determined as follow:

Try #4 @ 3.87 in.

$$A_s = 0.20\text{x}\frac{12}{3.87} = 0.62 \text{ in}^2/\text{ft}$$

$$a = \frac{0.62x60}{0.85x6x12} = 0.60 \text{ in.}$$

$$c = 0.60 / 0.75 = 0.81 \text{ in.}$$

$$d = 8 - 2 - 0.5\text{x}\frac{4}{8} = 5.75 \text{ in.}$$

$$\varepsilon_t = 0.003 \text{ x } \frac{5.75 - 0.81}{0.81} = 0.018 > 0.002\text{, therefore } \phi = 0.9$$

$$\phi M_n = 0.9\ (0.60\text{x}60)(5.75 - 0.5 \text{ x } 0.60)$$

$$= 176.7 \text{ in-k/ft} = 15.2 \text{ ft-k/ft} = M_{Strength\ I} \qquad \text{OK}$$

$A_{s\ longitudinal\ reinforcement}$ = 63.5% x 0.62 = 0.39 in^2/ft

Use #6 bars @ 13.3 in.

$$A_s = 0.44 \text{ x } \frac{12}{13.3} = 0.397 \text{ in}^2/\text{ft} \qquad \text{OK}$$

Section 5.10.8.2 of the LRFD Specifications states that the spacing between the longitudinal reinforcement bars < 18 in. and three times the slab thickness (3x8 = 24 in.). These conditions are satisfied using #6 @ 13.3 in.

The #6 bars are spliced at the panel-to-panel joint using two connection details that have been presented in Chapter 3 of this report. Figures of the system in this appendix show only one connection detail.

B.3.5.2 Longitudinal reinforcement for continuous span bridges

Many factors affect the amount of the negative moment reinforcement that is required over intermediate supports (i.e. piers) of continuous span bridges. These factors include: (1) number of spans, (2) relative span lengths, (3) relative stiffness of various spans, (4) depth of the superstructure, (5) concrete strength of the diaphragms provided over piers, and (6) type of reinforcement used (i.e. mild steel bars versus high strength threaded rods).

These factors have to be addressed in the design of longitudinal girders, which is beyond the scope of this document. Therefore, the researcher explored resources that contain information about the average required area of longitudinal reinforcement over piers. These resources include the PCI-BDM (6), the Nebraska Department of Roads POP Manual (9) and the Washington State DOT Design Manual. Also, the researcher contacted bridge designers from various DOTs, such as Tennessee, Nebraska, Florida, and Washington State, to get an average value of the longitudinal reinforcement used in bridges designed according to the LRFD Specifications. The researcher has found that for a typical three-span bridge in the range of 100-130 ft per span with concrete girders spaced at 9 to 12 ft, the average amount of longitudinal reinforcement is about 15 in^2 per girderline, using mild steel with 60 ksi yield strength.

B.3.6 Design of the panel-to-panel transverse connection

Among various shapes of the shear keys presented in chapter 2, the researcher decided to use, female-to-female grouted shear key. Figure B-9 gives the proposed dimensions of the shear key. A 1-in. wide gap is maintained between adjacent panels.

The modified shear friction theory (7) is used to determine the vertical shear strength of the shear key joint. The theory depends on depicting possible modes of failure of the joint. These modes are (see Figure B-9):

(a) Bearing failure at side "bc" of the shear key:

$$P_u \le \phi\left(0.85 f_c^{'}\right)\left(12 * L_{bc}\right) \text{ kip/ft}$$

Where: ϕ = strength reduction factor for bearing = 0.7 (Section 5.5.4.2.1, LRFD)

$f_c^{'}$ = specified concrete strength of the precast panel or the grout material, whichever is smaller = 6.0 ksi

L_{bc} = length of the side "bc" of the shear key = 1.06 in.

P_u = factored wheel load with dynamic allowance, calculated in kips per linear ft in the transverse direction.

$P_u \le$ (0.7x0.85x6.0x12x1.06) = 45.4 kips/ft

(b) Shear failure along "be" inside the shear key:

$$P_u \le \phi\left(c * 12 * L_{be} + \mu A_v f_y\right) \text{ kip/ft}$$

Where: ϕ = strength reduction factor for shear = 0.9 (Section 5.5.4.2.1, LRFD)

$f_c^{'}$ = specified concrete strength of the grout material = 6.0 ksi

L_{be} = length of the distance from "b" to "e" = 5.0 in.

c = cohesion strength of the grout material

μ = friction coefficient of the grout material

For concrete cast monolithically, c = 0.15 ksi and μ = 1.4 (Section 5.8.4.2, LRFD)

A_v = longitudinal reinforcement crossing the shear interface per foot

= 0.44x12/13.3 = 0.397 in^2/ft

f_y = yield strength of the longitudinal reinforcement = 60 ksi

P_u = factored wheel load with dynamic allowance, calculated in kips per linear ft in the transverse direction.

$P_u \leq$ 0.9(0.15x12x5 + 1.4x0.397x60) = 38.1 kips/ft

Therefore, P_u = 38.1 kips/ft

According to Section C3.6.1.2.5 of the LRFD Specifications, which gives guidelines to determine the tire contact area of the design truck of the HL-93 live load, the width of the contact area in inches = (P/0.8), where P = design wheel load in kips = 16 kips.

Therefore, the width of the contact area = (16/0.8) = 20 in.

The applied factored wheel load = P (LL load factor) (dynamic allowance IM)

= 16x1.75x1.33 = 37.24 kips/20 in.

= 22.3 kips/ft < 38.1 kips/ft

B.3.7 Miscellaneous design issues

The previous sections presented detailed design calculations of the proposed system under service conditions, i.e. after the deck panel system is installed, connected with the supporting girders and opened for traffic. However, during the life span of the precast panels from the time of fabrication to the time of opening the bridge to traffic, there are some other stages where the stresses of the deck panel have to be checked. These stages typically resulted from the fact that the panel is a pretensioned concrete member. For this type of members, the pretension force is usually released between 18 and 24 hours after casting the concrete. At that age, the concrete does not have its maximum strength yet and the prestressing force is at its maximum value. This stage is typically called “At Transfer” or “At Release”. At this stage the critical section is at the girder lines where there are ungrouted shear pockets that reduce the size of the concrete cross section that will resist the applied prestressing force.

After the prestressing force is released, the panel will be lifted and moved to a temporary storage place. The location of the lifting points on the panel have to be pre-determined by the design engineer in order to make sure that the stresses due to the weight of the panel combined with the prestressing force exists will not cause any damage to the panel.

The design engineer also has to check the stresses in the panel at time of installing the panel on the supporting girders. At this stage the concrete has reached its maximum compressive strength and got rid of all the creep and shrinkage deformation. Also, the prestressing strands have had almost all the relaxation deformation. The loads that should be used at this stage are the panel weight, the prestressing force after all losses are counted, and any construction load. The construction loads can be concentrated load due to a fork lift that will be used to carry the panels and install them in place or a uniform live load that represents the crew and equipment that are used during installation of the panels. Typically, the construction loads varies in magnitude and nature from a project to another depending on the way the precast panels are installed. Therefore, it is the contractor’s responsibility to provide the design engineer with the construction plan, and it is the design engineer’s responsibility to check the stresses in the panel due to this plan. The design engineer has to state clearly on the plans that the construction plan of the panels has to be checked and approved by him/her prior to construction. Check of stresses in the panel for the above discussed stages is given in the next sections.

B.3.7.1 Check of concrete stresses at time of transferring the prestressing force

f'_{ci} = 5.0 ksi

f_{pi} = strand stress after elastic shortening losses = 200.373 ksi

P_i = 200.373 x 8 x 0.153 = 245.3 kips

Stress limits for concrete: (LRFD, Sec. 5.9.4.1)

- Compression:

  $0.6\ f'_{ci} = 0.6 \times 5.0 = +3.0$ ksi

- Tension:

  In areas with bonded reinforcement sufficient to resist 120% of the tension force in the cracked concrete computed on the basis of an uncracked section:

  $0.22\sqrt{f'_{ci}} = 0.22\sqrt{5.0} = -0.492$ ksi

Stresses at top or bottom fibers of the slab:

f_t or f_b = P_i / A = 245.3 / 768 = + 0.319 ksi

Compressive stress limit for concrete: +3.0 ksi OK

B.3.7.2 Check of concrete stresses during lifting the panel from the presterssing bed

The following assumptions are used to check the stresses during lifting the panel from the prestressing bed:

1. The time elapsed between releasing the strands and lifting up the panel is very small. Therefore the concrete strength f'_{ci} and strand stress f_{pi} used to check stresses at release will be used at this stage.
2. The panel will be lifted at every girderline.

<u>Check of stresses at mid span section of the exterior span:</u>

f_t = (P_i / A) + (M_{slab} / S_t)

= (245.3 / 768) +(0.520 x 12 / 1024)

= + 0.326 ksi

Compressive stress limit for concrete: +3.0 ksi OK

f_b = (P_i / A) - (M_{slab} / S_b)

= (245.3 / 768) – (0.520 x 12 / 1024)

= + 0.313 ksi

Compressive stress limit for concrete: +3.0 ksi OK

<u>Check of stresses at first interior girderline:</u>

f_t = (P_i / A) - (M_{slab} / S_t)

$= (245.3 / 768) – (1.28 \times 12 / 1024)$

$= + 0.304$ ksi

Compressive stress limit for concrete: +3.0 ksi OK

$f_b = (P_i / A) - (M_{slab} / S_b)$

$= (245.3 / 768) + (1.28 \times 12 / 1024)$

$= + 0.334$ ksi

Compressive stress limit for concrete: +3.0 ksi OK

B.4 REFERENCES OF APPENDIX B

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3. Badie, S. S.; Tadros, M. K.; Kakish, H. F.; Splittgerber, D. L.; and Baishya, M. C, “Large Studs for Composite Action in Steel Bridge Girders.” *Bridge Journal, American Society of Civil Engineering (ASCE),* Vol. 7, No. 3 (May-June 2002) pp. 195-203.
4. Devalapura, R. K.; and Tadros, M. K., “Stress-Strain Modeling of 270-ksi Low-Relaxation Prestressing Strands.” *Precast/Prestressed Concrete Institute (PCI) Journal,* May/April 1992, pp. 100-106.
5. Tadros, M. K.; and Baishya, M. C., “Rapid Replacement of Bridge Decks.” *NCHRP Report 407,* Transportation Research Board, Washington, DC (1998).
6. PCI-Bridge Design Manual (PCI-BDM), Precast/Prestressed Concrete Institute, Chicago, IL, 2nd edition, (2003).
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8. National Highway Institute (NHI), FHWA NI-95-017, “Load and Resistance Factor Design for Highway Bridges.” NHI Course No. 13061 (1995).
9. Nebraska Department of Roads Design Manual and Policies (NDOR POP) (2006).

B.5 FIGURES OF APPENDIX B

Figures B-1 to B-9 provide details of the system.

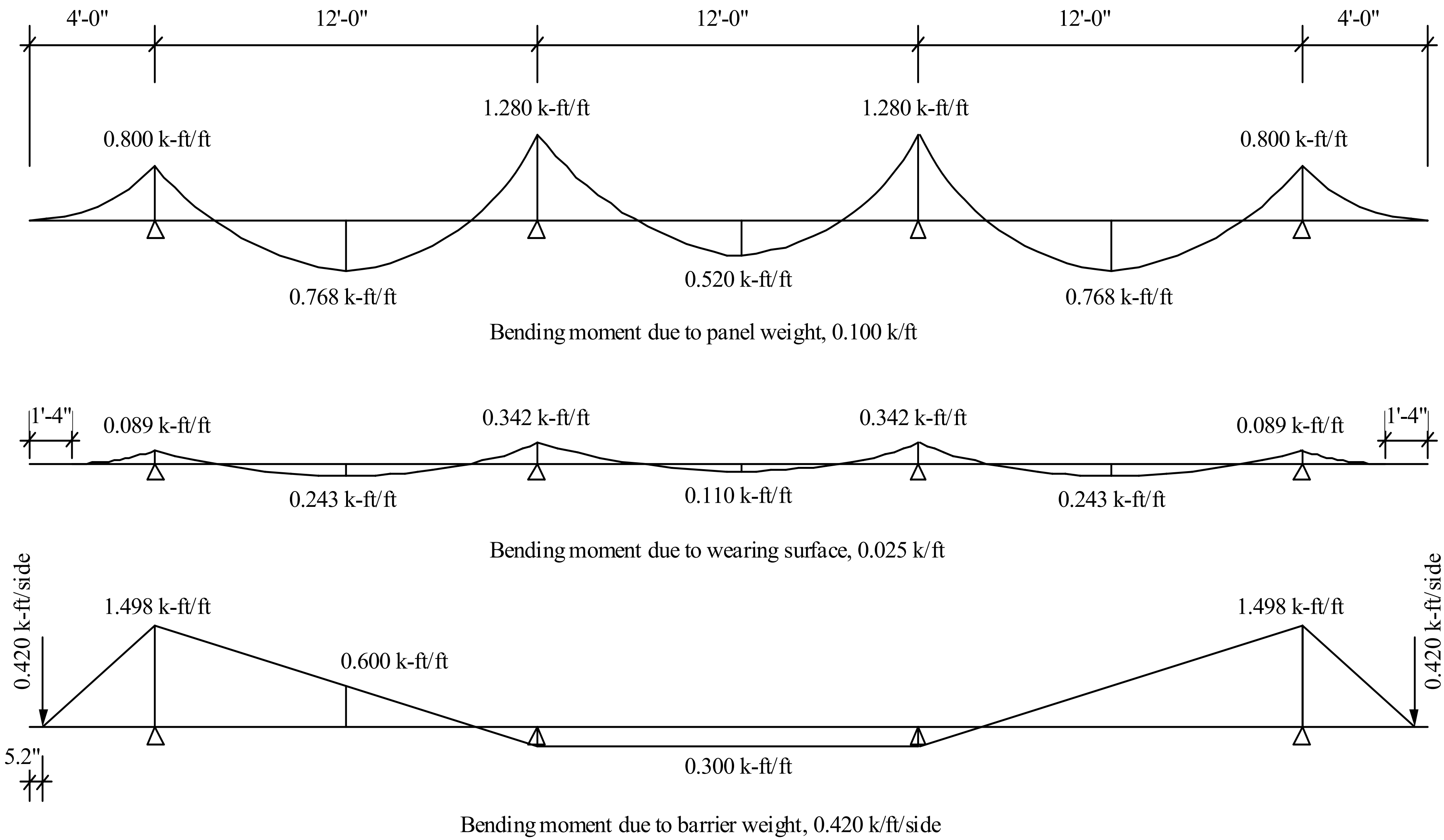


Figure B-1. Bending Moment due to Service loads

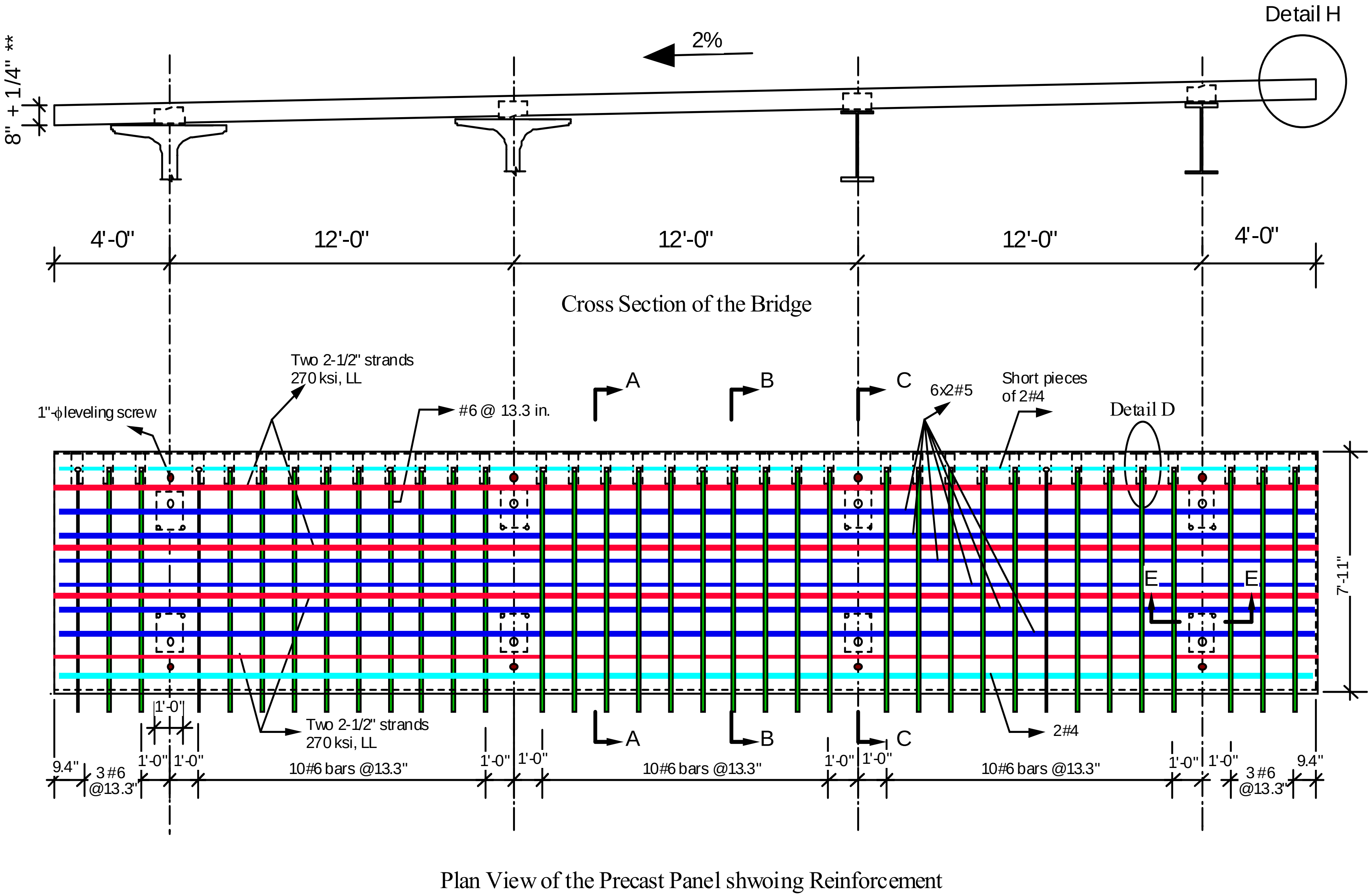


Figure B-2. Cross Section and Plan View of CD-1
(** 1/4 in. is used as a sacrificial layer for texturing)

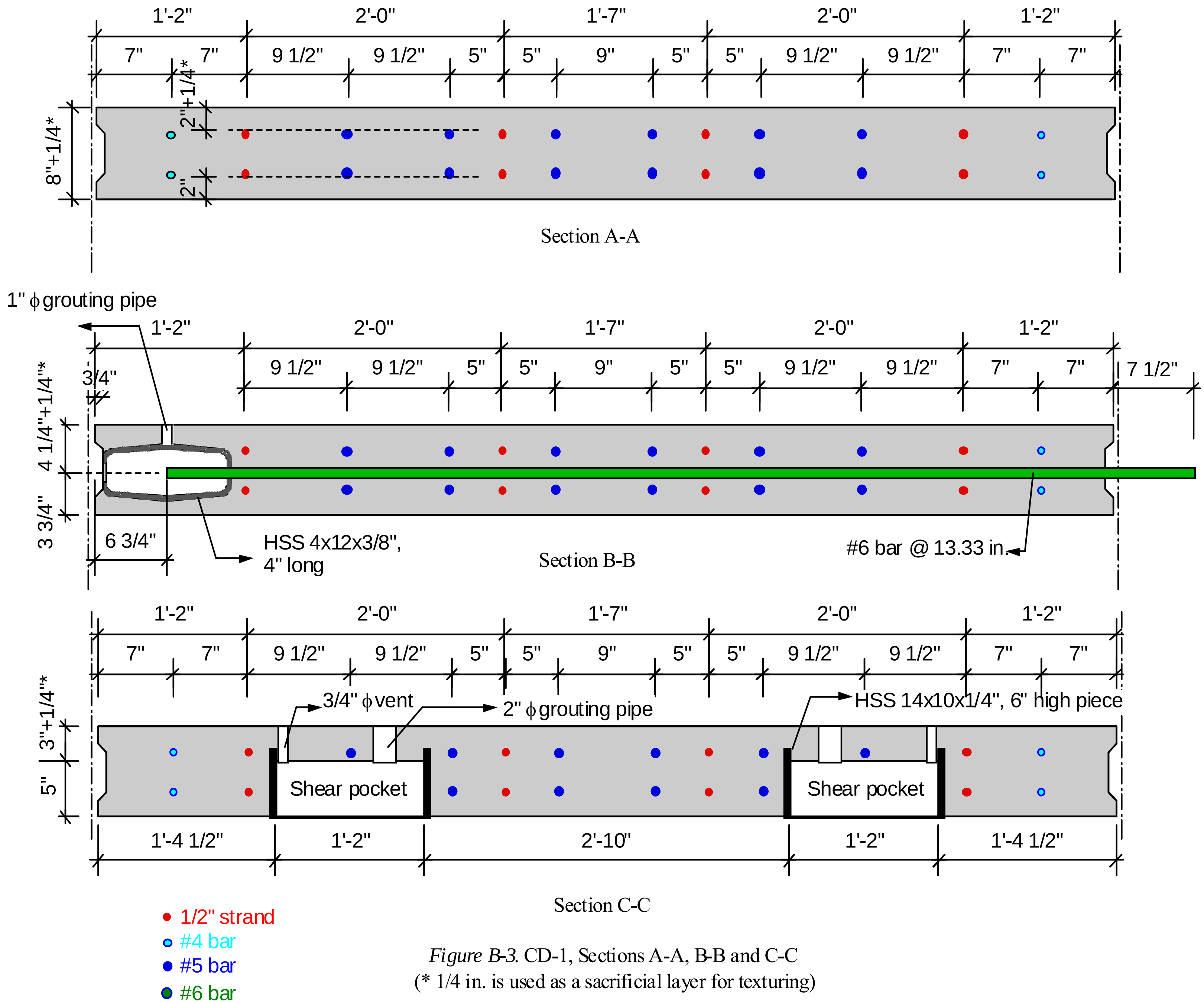


Figure B-3. CD-1, Sections A-A, B-B and C-C
(* 1/4 in. is used as a sacrificial layer for texturing)

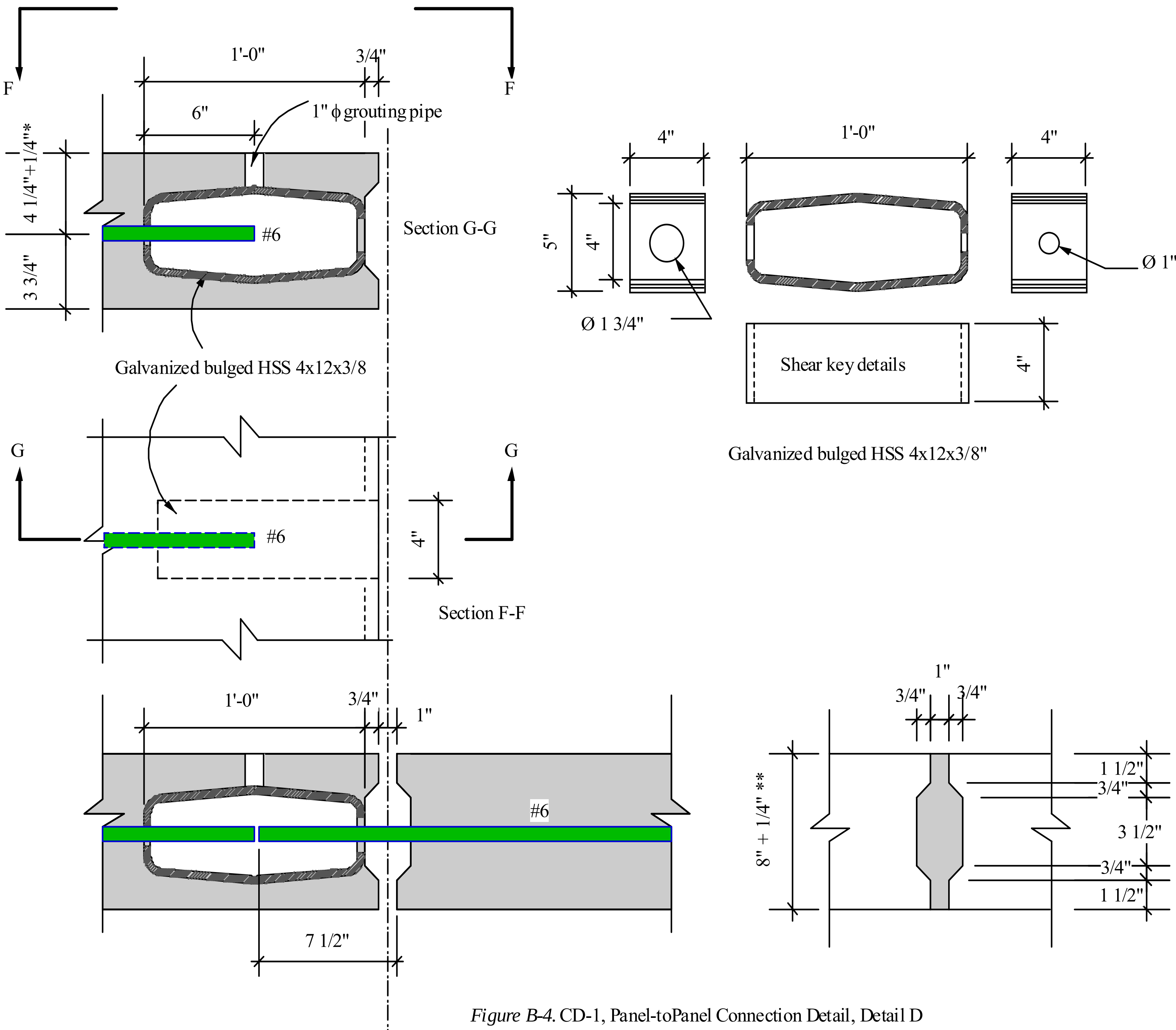
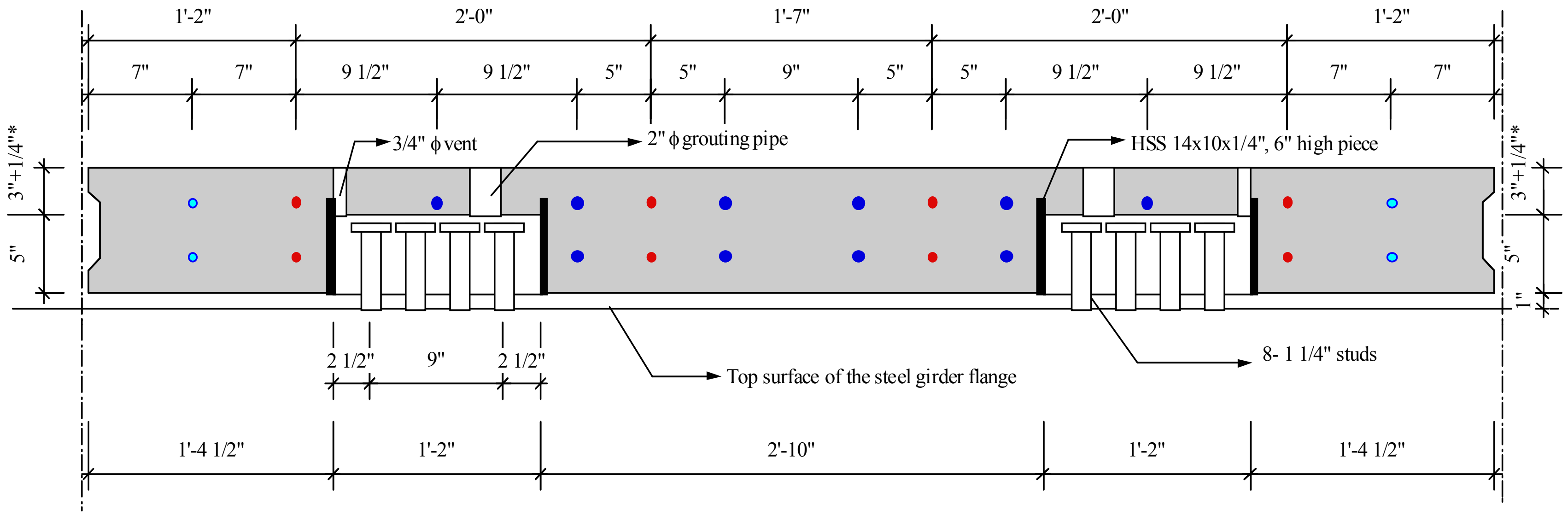
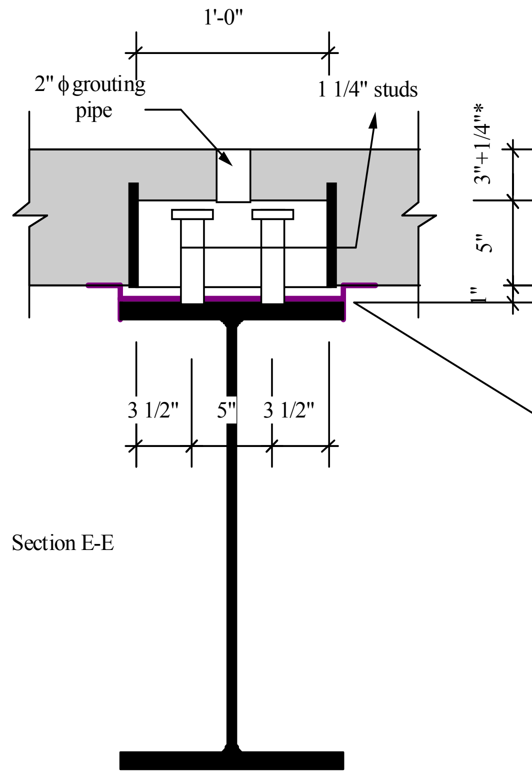


Figure B-4. CD-1, Panel-toPanel Connection Detail, Detail D





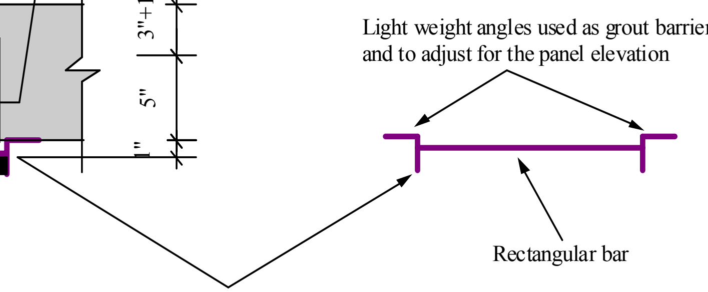


Figure B-5. CD-1, Sections C-C & E-E for Steel Girders
(* 1/4 in. is used as a sacrificial layer for texturing)

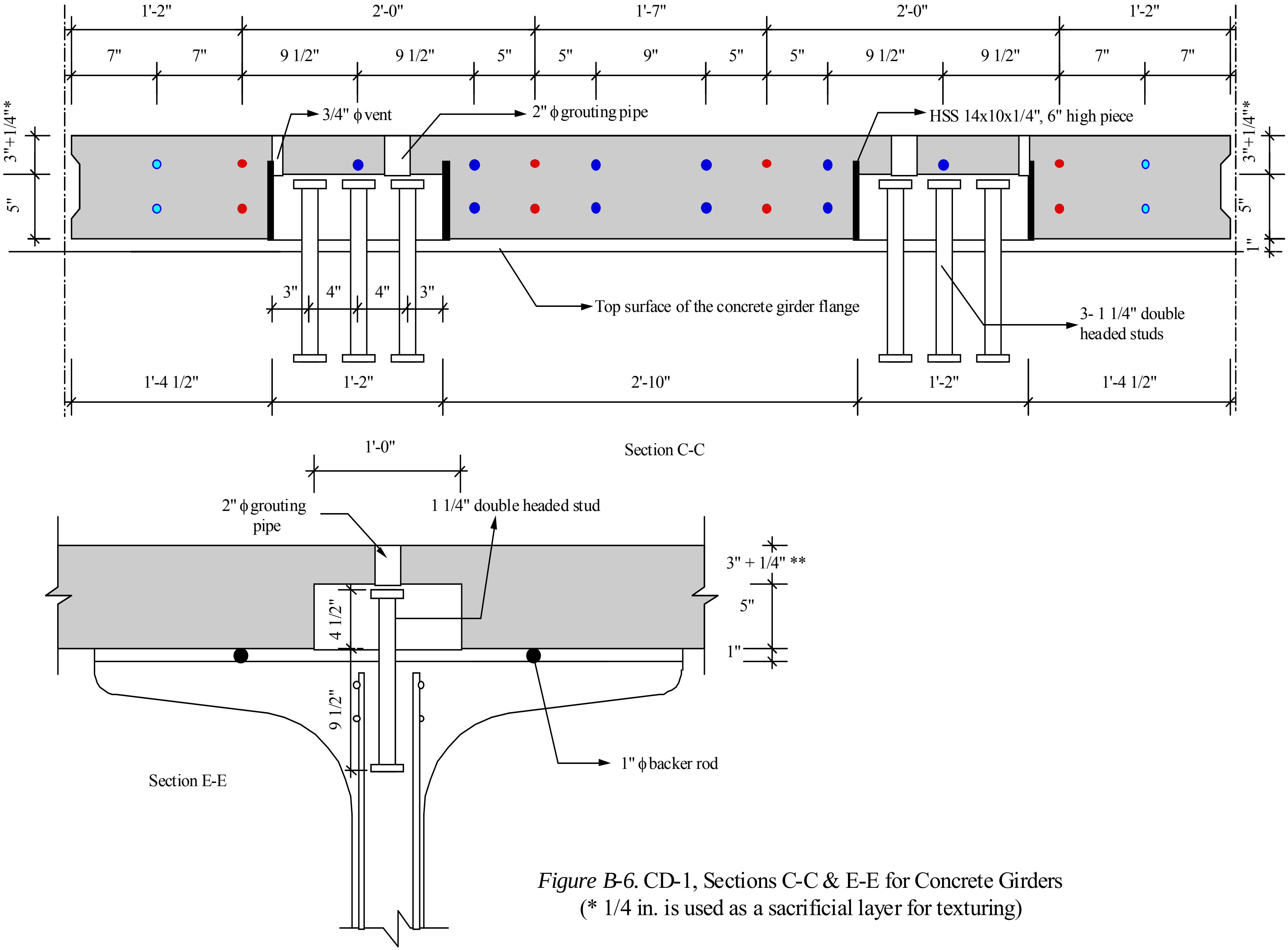


Figure B-6. CD-1, Sections C-C & E-E for Concrete Girders
(* 1/4 in. is used as a sacrificial layer for texturing)

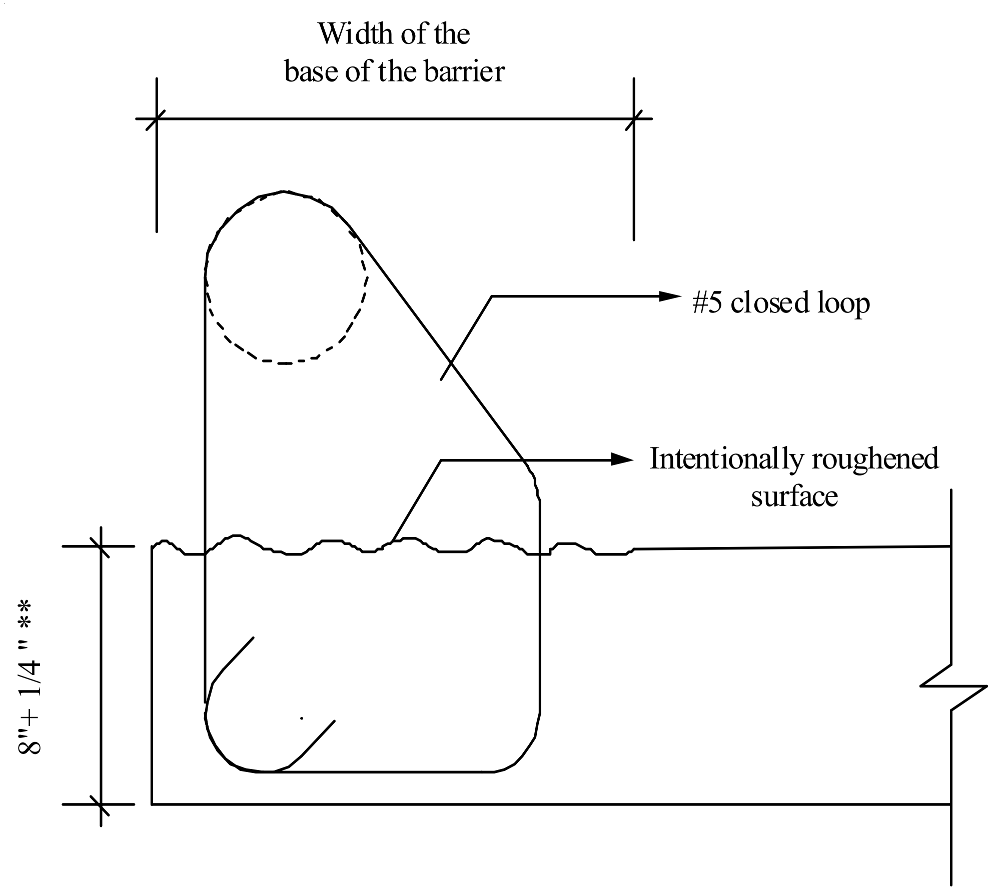


Figure B-7. CD-1, Detail H for the Panel-to-Barrier Connection
(* 1/4 in. is used as a sacrificial layer for texturing)

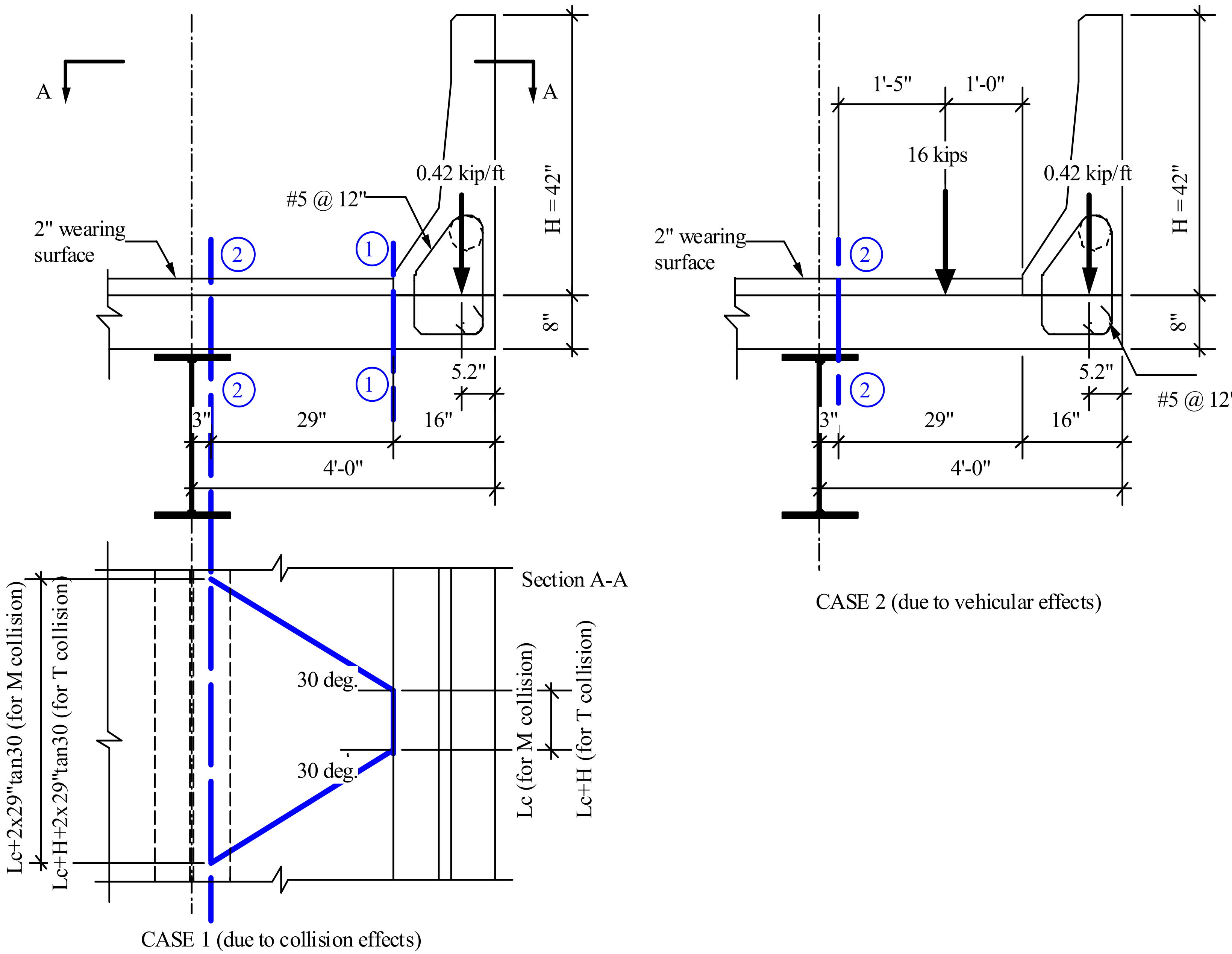


Figure B-8. CD-1, Design parameters of the overhang

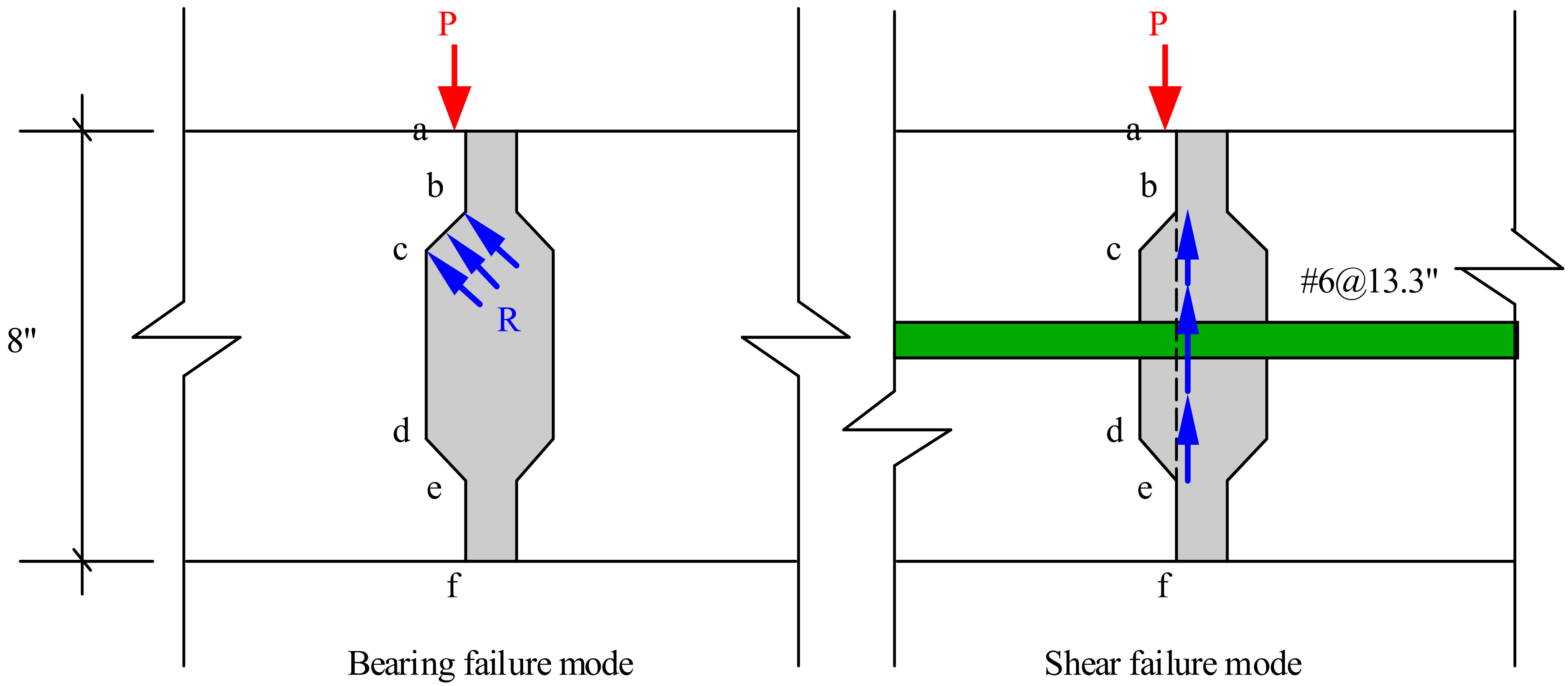


Figure B-9. Design Parameters of the Shear Key

APPENDIX C

DESIGN, DETAILING, FABRICATION & INSTALLATION GUIDE

The intent of this document is to provide design, detailing, fabrication, and installation guidelines for full-depth, precast-concrete bridge deck panel systems. This guide does not cover any proprietary full-depth, precast-concrete bridge deck panel systems. Typically, each deck construction project has its unique features and restraints that may affect the design, fabrication and construction process. Therefore, the reader should evaluate the provisions relevance of this guide in connection with his/her needs. The structure of the guide is as follow:

C.1 DESIGN GUIDE

C.1.1 Slab thickness

1. Section 9.7.5 of AASHTO LRFD Specifications (*1*) recommends a minimum thickness of 7 inches (178 mm) excluding any provision for grinding, grooving and sacrificial surface.
2. In any case, the minimum slab thickness is controlled by the minimum concrete cover requirements. AASHTO Standard and LRFD Specifications require a 2 in. (50 mm) minimum concrete cover on top layer of reinforcement and a 1 in. (25 mm) minimum cover on bottom layer of reinforcement. Based on information established from the literature review, it has been found that to satisfy the minimum concrete cover limits and to insure adequate reinforcement clearances, especially if longitudinal post-tensioning is provided, the minimum slab thickness should be 7½ in (190 mm).
3. Structurally, 7½ in. (190 mm) thick slab can be used satisfactorily for deck slabs with girder spacing up to 10 to 11 ft (3.05 to 3.35 m). Typically, it is more economical to use the least possible number of girder lines rather than reducing the slab thickness based on satisfying the flexural capacity. Also, it has been found that a girder spacing between 10 and 12 ft (3.05 and 3.66 m) usually produces the most economical superstructure system of a slab/girder bridge.

C.1.2 Composite action between the precast deck and the superstructure

1. Typically, full depth precast decks use shear connectors on the girders that project into grouted pockets in the deck sections.
2. Full composite action between the precast deck and the girders of the superstructure is created after the grout in the shear pocket gains adequate strength.
3. The designer should consider the performance of the supporting girders prior to having the full composite action. During this stage the girder alone should support the following loads: (1) its self weight, (2) the form weight, (3) the precast deck self weight, and (4) construction load, which may include crew members, and equipment used in handling and installing the panels. Loads added after the deck is made composite with the girder are resisted by the composite slab/girder section.
4. If the bridge is allowed to open to traffic before the composite action is achieved, the designer should consider two groups of loading: (a) girder and deck weight plus construction loads, and (b) girder and deck weight plus traffic loads. The designer might have to enforce a low speed limit on the bridge, under these conditions, to minimize the impact effects of the moving traffic on the girders.
5. Prestressed concrete girders would be required to be designed for serviceability and strength. Conventionally reinforced concrete and steel girders would be required to be designed for strength. For steel members the designer would also be required to check the top flange against local buckling and may use permanent

or temporary bracing to support the top flange during this stage if buckling is a concern.

6. Because the majority of the supporting girders are cambered when they installed on the bridge, the designer should consider the minimum height of the haunch between the deck slab and the girder to determine the geometrical properties of the composite section. This applies both to prestressed and steel girders.
7. To determine the geometrical properties of the composite slab/girder section, the designer should use effective flange width of the slab as given in Section 4.6.2.6.1 of the LRFD Specifications (*1*). Also, the designer should consider the difference in material between the slab and the girder by incorporating the modular ratio in the calculations.
8. When checking service stresses in the slab, the designer should superimpose the longitudinal post-tensioning stresses in the slab prior to becoming a composite section, with dead and live load stresses on the composite section.
9. If the longitudinal post-tensioning reinforcement on the slab is not uniformly distribute across the slab width (i.e. it is concentrated over or between girder lines), the designer should conduct rigorous analysis, such as the finite element analysis, to determine exact distribution of the longitudinal post-tensioning stresses in the deck slab.
10. If the slab is not made composite with the girder, then all the straining actions resulted from loads added after installing the precast slab will be distributed between the slab and the girder based on their relative stiffness. However, it is conservative to consider all the loads supported only by the girder.

C.1.3 Staged construction

The term "construction period" used in this section refers to all the activities that will be conducted in one construction period, which may be an "over-night" or a "weekend" period.

1. If no longitudinal post-tensioning is used in the precast deck, it is recommended that each segment of the precast deck, constructed during one "construction period", is made composite with the superstructure before the bridge is open to traffic, otherwise the provisions of Section C.1.2.3 must be satisfied. In this regard, the designer should specify the minimum strength of the grout that should be attained at time of opening the bridge for traffic. Also, the method of determining the grout strength should be specified.
2. For deck replacement projects, the designer should specify the start and end locations on the deck of each "construction period." Also, he/she should consider the effect of the overall construction scheme on the superstructure. For example, if the contractor will not be able to construct one full span during one "construction period", the composite girder-deck member will not have uniform properties along its length due to the difference in thickness and concrete strength between the existing and new deck. Another example, if the old bridge had been made continuous over the piers and the contractor could not restore this continuity by the end of one "construction period," the designer should check the girder

capacity as a simply supported beam. This check should be done not only for the spans adjacent to the pier where continuity is lost, but also for all affected spans of the bridge.

3. If longitudinal post-tensioning is provided with staged construction, the end panel of every "construction period" should be provided with special devices that allow anchoring of the longitudinal post-tensioning of the current "construction period" and splicing it with the longitudinal post-tensioning of the next "construction period". It is possible that the stresses in an end panel due to the longitudinal post-tensioning force will be doubled if two separate anchorages are used for the current and next "construction period".

4. It is a common practice that temporarily barriers are installed close to the edge of a newly installed stage, to open it for traffic while the second stage is under construction. The designer should consider this load as part of the loads applied to the precast panel during this stage. The location of the barriers used in the calculations should be chosen to produce the highest straining actions on the deck.

   Also, the weight of the temporarily barriers may cause the precast panels to deform in a way that alters the panel's elevation especially at the closure pour location. It is recommended that the temporary barriers be removed and the panel's elevation at the edge is checked before installing the closure pour.

C.1.4 Panel-to-girder joint detail

1. Shear connectors clustered in groups can be used and should be made to match the shear pocket locations of the precast panels.

2. Experimental investigation (NCHRP 12-65) has shown that the stud cluster spacing can be extended up to 4 ft (1220 mm), without affecting the full-composite behavior of the system, if confinement to the stud group and the grout surrounding them is provided. Confinement helps in distributing the shear force among the studs in each cluster, and protecting the grout at the base of each stud against crushing. The 4 ft (1220 mm) spacing helps in reducing the fabrication and grouting cost of the panels.

3. Confinement can be provided by using hollow structural steel (HSS) tubes or closed ties set horizontally across the panel thickness. If closed ties are used, the lowest tie should be placed as close to the top surface of the girder as possible.

4. Equation 5.8.4.1-1 should be used to estimate the ultimate capacity of stud clusters spaced up to 4 ft (1220 mm) for steel and concrete composite beams. Limits on the nominal horizontal shear capacity given by Equation 5.8.4.1-2 and 5.8.4.1-3 will not apply if confinement is provided.

5. For steel composite beams:

   - Equation 6.10.10.2-1 of the LRFD Specifications, that is currently used to estimate the fatigue capacity for single studs, does not require modification for design of stud clusters spaced up to 4 ft (1220 mm).

- Equation 6.10.10.4.3-1 for single studs should not be used for stud clusters at 2 to 4 ft (610 to 1220 mm) spacing. This recommendation is expected to result in about 30 percent increase in the required number of studs.
- The recommendation just given above is based on the results of the of push-off testing of stud groups at 4 ft (1220 mm) spacing, which gave about 30 percent lower capacity than the current LRFD Equation 6.10.10.4.3-1 for single studs. This conclusion may be unnecessarily conservative as the push-off testing is not as realistic in modeling beam behavior as actual beam test, which showed no reduction in capacity due to use of stud clusters. However, this is a conservative approach and it does not significantly impact the overall economy of bridges.

C.1.5 Transverse panel-to-panel joint detail

1. A grout-filled joint should be provided between adjacent panels. Direct butting between adjacent panels must be avoided.
2. Grout should fill the full height of the joint. Wood forming installed under the panel or galvanized metal plates attached to the panel should be used to achieve this goal.
3. Shear friction theory as given by the AASHTO LRFD Specifications can be used to check the shear capacity of the joint.
4. For bridge decks on a horizontal curvature, it is recommended to use trapezoidal panels to keep the width of the joint constant and to a minimum. For bridges with large horizontal curvature, rectangular panels may be used to simplify the production process of the panels. A 1 in. (25 mm) minimum width and 3 in. (76 mm) maximum width of the joint should be used whether trapezoidal or rectangular shape panels are used.
5. Flowable, self-leveling, freeze thaw durable, non-shrink grout mix should be used. The grout material, if stored on the construction site, should be kept protected from the environmental factors such as humidity and rain.
6. If no overlay is used over the precast panels, finishing of the top surface of the grout filling of the joint should match the finishing of the precast panel.

C.1.6 Longitudinal panel-to-panel joint detail

1. This joint will result only if multiple panels are used across the width of a bridge. This may occur due to staged construction or to avoid crowning of the panels.
2. If the joint is not directly over a girder flange, positive moment reinforcement should be adequately coupled. The width of the joint should be adequate to provide the necessary coupling device.
3. If the joint is over a girder flange, investigation of negative moment capacity should be made and reinforcement provided accordingly. Top reinforcement continuity would be needed to, as a minimum, control longitudinal cracking over the girder line.
4. Precast crowned panels may be used if satisfactorily produced and installed.

C.1.7 Old-to-new deck joint detail

1. Temporary connection between the old and new deck should be developed. The connection should satisfy the following conditions:
   (a) It does not cause high stress concentration in the existing or new deck.
   (b) It does not require drilling holes or modifying the standard new precast panels.
   (c) It can be installed and removed with minimum disruption to traffic on the bridge, or below the bridge in overpass situations.
2. An edge support should be provided for the last precast panel, installed in a construction period, to protect it from excessive stress.

C.1.8 Longitudinal post-tensioning (PT)

1. PT force should be applied only the precast deck. Therefore, the panel-to-girder connection should not be constructed until the PT reinforcement is tensioned and anchored.
2. The minimum average effective stress on concrete due to PT, after considering all types of losses and longitudinal flexural stresses in continuous span bridges, should not be less than 0.250 ksi. This recommendation is consistent with Section 9.7.5.3 of the AASHTO LRFD Specifications (*1*).
3. It is recommended that PT tendons are uniformly distributed across the slab width and are not be farther apart, center-to-center, than 4 times the total composite minimum thickness of the slab. In this case, the average stress due to PT reinforcement can be determined using flexural elastic stress distribution, i.e. $\left(P/A \pm MC/I \right)$. The composite thickness refers to slabs with bonded overlays. This recommendation is consistent with Section 5.10.3.4 of the AASHTO LRFD Specifications (LRFD 2004).

   However, if the longitudinal post-tensioning reinforcement on the slab is not uniformly distributed across the slab width (i.e. it is concentrated at or between girder lines), the designer should conduct rigorous analysis to determine exact distribution of the longitudinal post-tensioning stresses in the deck slab. Rigorous analysis includes finite element analysis, plate analysis, or any analysis that can detect accurately the stress distribution across the deck width.
4. The minimum concrete cover as stated by Section 5.12.3 of the LRFD Specifications (*1*) should be maintained on all anchorage and splicing devices of the PT reinforcement.
5. The maximum jacking stress in PT reinforcement should not exceed 80 percent of the specified minimum ultimate tensile strength of the post-tensioning steel.
6. When longitudinal post-tensioning is provided with staged construction, the end panel of every "construction period" should be provided with special devices that allow anchoring of the longitudinal post-tensioning of the current "construction period" and splicing it with the longitudinal post-tensioning of the next

"construction period". It is possible that the stresses in an end panel due to the longitudinal post-tensioning force will be doubled if two separate anchorages are used for the current and next "construction period".

C.1.9 Overhang design

1. The overhanging slab should not be made shouldower than the interior slab.
2. At least two sections inside the overhang should be designed, which are at the inner face of the side barrier and at critical section over the exterior girder line.
3. The overhang should not be tapered. This is because: (1) to simplify the fabrication process and (2) satisfy the flexural demand of the overhang at the interior face of the barrier.
4. The load effects of the overhang can be determined in accordance to Section A13.4.1 of the AASHTO LRFD Specifications.
5. Special attention should be given to the design of the slab overhanging with regard to the development of prestressing and conventional reinforcement. Effective reinforcement stress should be prorated based on the available embedment length at the section under consideration.
6. If the prestressing reinforcement cannot develop the required strength within the slab overhang, mild reinforcement or/and post tensioning can be used.
7. The conventional reinforcement should not be larger than number 6 bar.
8. Special arrangements can be used to reduce the development length of prestressing and conventional reinforcement, such as using mechanical anchorage devices and confining the concrete surrounding the reinforcement by high strength spirals.

C.2 DETAILING GUIDE

C.2.1 Shear key (at panel-to-panel transverse joints)

1. It is recommended to use grouted shear key transverse joints between adjacent panels. Although, grouted joints require more time and labor to be done in the field, compared to the match-cast non-grouted joints, they provide higher construction tolerance and guarantee full contact between adjacent panels. Figure F-1 show some details that have been successfully used.
2. In order to have a successful shear key detail, the following issues should be considered:
    - The top edge of the shear key should be recessed about ½ to 1 in. (25 mm) relative to the bottom edge. This will give the contractor flexibility in filling the joint with grout while minimizing the risk of having poorly consolidated grout.

- A 1 (25 mm) wide gap should be provided between adjacent panels at the bottom edge of the shear key. This gap will provide enough tolerance for fabrication and construction process.
- It is recommended to provide the shear key with sharp edge corners rather than curved corners in order to optimize the mechanical interlocking effect of the shear key.
- It is recommended to build the form for the grout filling the gap from the bottom surface of the panels. The forms can be hung from the top surface of the panel, or can be attached to the bottom surface of the panels suing special inserts provided during fabrication of the panels. Wood, metal, plastic or foam can be used to build the form.
- In case of using longitudinal post-tensioning in the precast deck system, the shear key may be replaced with straight vertical edge joint. The vertical edges should be intentionally roughened to optimize the shear capacity of the interface between the grout and the panel.
- Roughening can be achieved by sand blasting the panel edges, which is followed by a thorough washing procedure. This operation can be done in the precast yard after curing of the panels, or on the bridge site before installing the panels on the bridge.
- Roughening can be also provided during fabrication of the panels by painting the side forms with a retarding agent. After removing the side forms, the panel edges are washed with water under high pressure, so that the aggregate of the concrete will be exposed and a roughened surface is created.
- In case of using longitudinal post-tensioning in the precast deck system, blockouts or pockets should be provided at the panel's edge in order to provide enough space for splicing the longitudinal post-tensioning ducts and secure their joints against possible leakage. See also Section C.2.3 of the guidelines for more information.

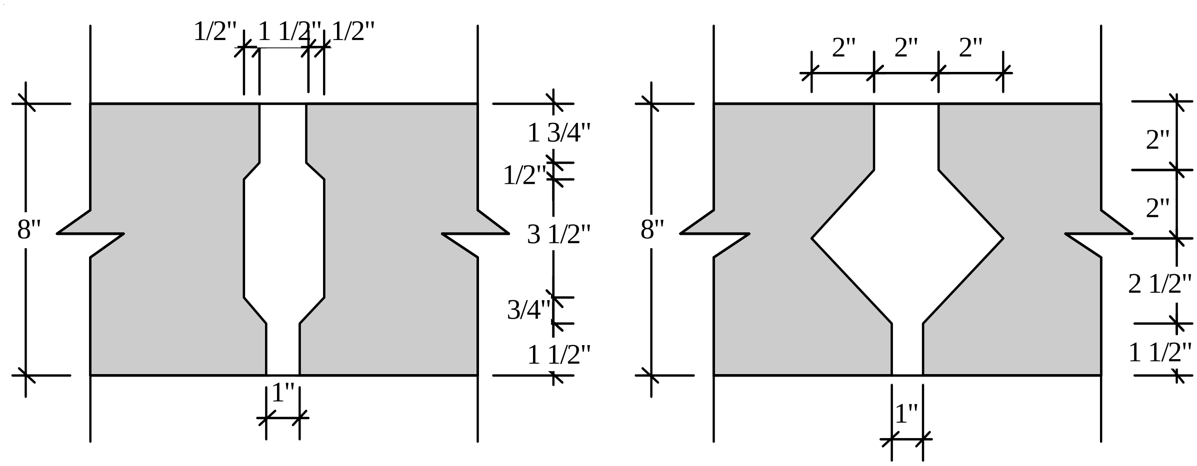


Figure F-1. Successful Shear Key Details

C.2.2 Shear pockets at panel-to-girder joints

1. The shear pocket can be full or partial height of the panel thickness, as shown in Figure F-2.

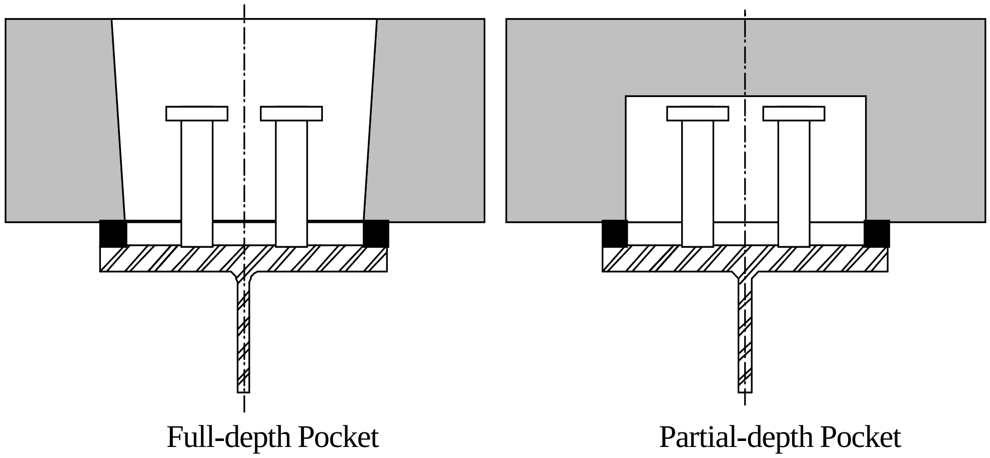


Figure F-2. Beveled-Shape Shear Pocket

2. If no overlay is provided on the precast deck system, it is recommended to use partial-height shear pocket, for the following reasons:
   (a) To protect the deck from water leakage at the interface between the sides of the shear pocket and the grout filling it.
   (b) To avoid non-uniformity of color of the top surface of the deck
3. The depth of partial-height pockets should satisfy the following conditions:
   (a) The shear connector should be embedded in the shear pockets so that its highest point is at least 2 in. (50 mm) above the bottom layer of transverse reinforcement of the panel.
   (b) A ½ in. (13 mm) minimum clearance between the highest point of the shear connector and the pocket
   (c) A 2 in. (50 mm) minimum distance between the highest point of the shear connector and the top surface of the panel
4. Partial-height pockets should be provided with injection ports and vents. A 1½ to 2 in. (38 to 50 mm) diameter injection ports are recommended to provide enough flexibility during injection. Two or more vents should be provided if the width of the pocket is greater than 10 in. (254 mm). The vents should be provided on the opposite side of the injection port to guarantee complete filling of the pocket.
5. If the spacing between pockets is greater than 24 in. (610 mm), an intermediate injection port should be provided in the panel

C.2.3 Longitudinal post-tensioning (PT)

1. It is recommended to provide the PT reinforcement as near mid height of the panel as possible to avoid residual deflection or camber of the slab.

2. The type and size of the PT reinforcement should be chosen so that the minimum concrete cover requirement on the main (transverse) reinforcement is satisfied.
3. The transverse edges of the precast panel should be provided with blockouts at the location of PT ducts to allow coupling the PT ducts. A relatively larger blockouts should be used at joints where the PT reinforcement is spliced.
4. The end panels should be designed to accommodate the anchorage device. Exact dimensions of the anchorage device should be obtained from the PT reinforcement supplier.

C.2.4 Haunch

1. A minimum 1-inch thick haunch should be provided between the precast panels and the steel stringers to allow for any misalignment or irregularity and guarantee complete filling of the haunch with grout.
2. If it is desired not to totally fill the haunch with grout for the entire width of the top flange, the grout should extend horizontally at least 2 in. beyond the pocket width. The grouted area should be provided with adequate ventilation to assure absence of trapped air.

C.2.5 leveling of precast slabs on supporting system

Elevation of the precast panel can be adjusted using leveling bolts and shim packs. The shim packs are recommended to be placed next to a stud pocket or any accessible areas to be easily adjusted during installation. The following equation should be used to determine the grade of the deck panels after being installed on the bridge:

$$DG = FG - WS + DCL$$

where: DG = Deck grade after installing the panels on the bridge

FG = Final grade of deck (known)

WS = Thickness of wearing surface (known)

DCL = Deflection due to composite loads. DCL is determined from the structural analysis of the composite deck/girder system. Composite loads refer to any loads added to the deck after the composite action between the deck and the girders is provided, such as wearing surface, barriers, and utilities loads.

The following issues should be considered if leveling bolts are used:

- All leveling bolts and their inserts should be corrosion resistant.
- Each bolt should be torque to insure that there is approximately equal bearing on each leveling bolt providing proper dead load distribution to each girder.
- The leveling bolts can be removed after the grout between the deck and the girders gain enough strength to support the panel weight and the construction loads.
- If the designer opted to permanently keep the leveling bolts after the grouting process is complete, the bolts should be torch cut and recessed at least 2 in. (50

mm) below the top surface of the panel. Concrete blockouts should be provided during fabrication of the panel for that purpose. After torch cutting the bolts, these pockets should be filled with non-shrink cementitious grout of a concrete strength that at least matches that of the precast panel.

- There should be the same number of leveling bolts over each girder. A minimum of 2 leveling bolts per panel should be provided over each girder line.

Recently, galvanized stay-in-place (SIP) angle grout dams have been used on bridges in Nebraska and Texas. The SIP angles, similar to those used for SIP forms, are welded to the top flange or to straps across the top flange to secure it in place. They are installed at the required pre-calculated haunch height minus a fraction of an inch to allow room for a closed cell foam grout seal bonded to the top surface of the angle.

C.3 FABRICATION, STORAGE, HANDLING, TRANSPORTATION & INSTALLATION GUIDE

C.3.1 Precast panel

1. It is recommended to reduce the number of types of panels needed for a project as much as possible to increase fabrication speed and reduce fabrication cost.
2. Clean steel beds and steel side forms should be used to provide panels with exact dimensions.
3. Shear and splicing blockouts can be formed of wood, steel or polystyrene foam or any material that can be cut to the exact dimensions with a maximum tolerance of ± 1/8 in. (3 mm), and do not react with concrete or steel reinforcement. Also, they have to be well secured to the bed and/or the side forms so that they do not travel during casting of concrete.

C.3.2 Longitudinal post-tensioning (PT)

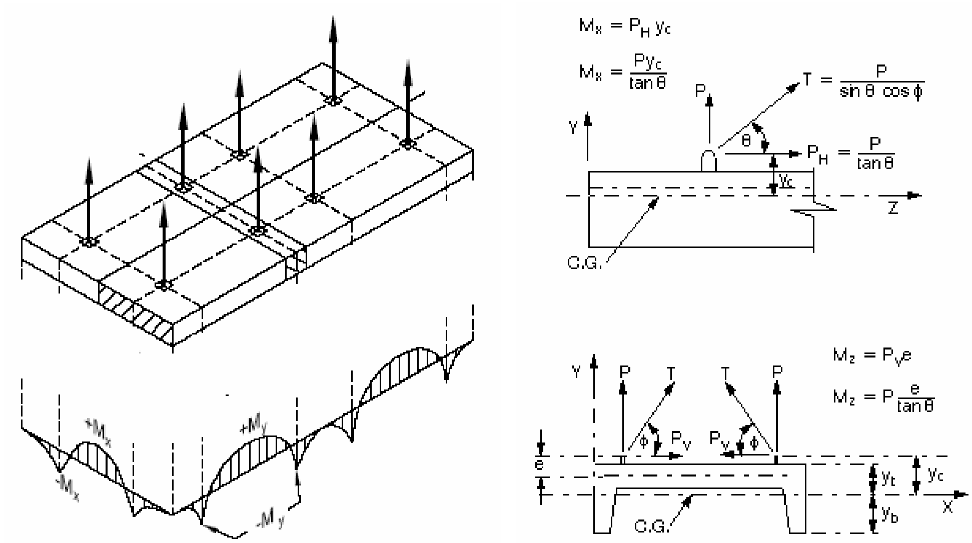
Post-tensioning is to be completed in accordance with the provisions of Section 5 of these specifications. Special effort should be made to provide accurate alignment of post-tensioning ducts from panel to panel. Grouting should be done using thixotropic grout material and should be performed in accordance to the following publication:

"Guide Specification for Grouting of Post-Tensioned Structures," Post-Tensioning Institute, First Edition, February, 2001, Post-Tensioning Institute, Phoenix, AZ.

C.3.3 Storage and Handling

1. If the panels are stored horizontally, they should be supported on temporary supports located as near as possible to the permanent support locations.
2. During storage, the panels should be protected from environmental effects, such as rain and snow to protect any exposed steel components from rusting.
3. Handling, storage and shipping of slender precast deck panels required special care to avoid damage due to these temporary activities.

4. The precast concrete producer should be responsible for design of the lifting inserts and their location in coordination with the rigging scheme plans of the erector. The location of lifting inserts and their specifications along with the rigging scheme should be clearly shown on the shop drawings prepared by the precast producer. The shop drawings should be reviewed and approved by the design engineer.
5. During lifting and handling, panels should be subjected to vertical lifting forces only, through use of spreader beams and strong backs as needed.
6. The precast panel should be designed to resist the following straining actions during handling:
   - Longitudinal and transverse flexural effects due to the weight of the panel, as shown in Figure F-3(a)
   - Axial forces in the plan of the panel due to the slope of the sling, as shown in Figure F-3(b)
   - Concentrated moment at the lifting points due to the eccentricity of the rigging hooks from the panel centroid, as shown in Figure F-3(b). When the sling angle is small, the component of force parallel to the longitudinal axis of the member will generate a significant moment. Therefore, it is recommended to have the sling angle between 60 to 70 degrees. While this effect can and should be accounted for, it is not recommended that it would be allowed to dominate design moments. Rather, consideration should be given to using spreader beams, two cranes or other mechanisms to increase the sling angle.



(a) Due to panel weight (b) due to lifting slings

Figure F-3. Straining Actions during Handling

7. A 1.33 dynamic load factor should be used in addition to the load factors associated with the considered Limit State (i.e. Service, Ultimate, etc.) because handling will induce dynamic loads on the panel.

8. In order to ensure that an embedded insert acts primarily in tension, a swivel plate as shown in Figure F-4 can be used. Sufficient threads must be engaged to develop the strength of the bolt. Some manufacturers use a long bolt with a nut between the bolt head and the swivel plate. After the bolt has "bottomed out", the nut is turned against the swivel plate. Also, inverted U-shape bars can be used. The U-bars should be provided with horizontal L-shape ends and tied with the bottom layer of reinforcement of the panel.

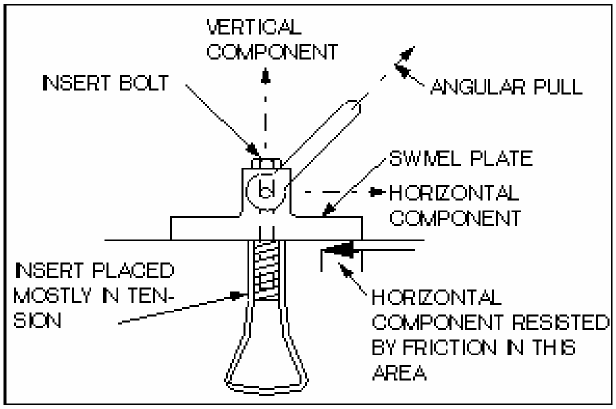


Figure F-4. Swivel Plate

C.3.4 Transportation

1. Transportation restrictions add to the cost of precast deck panels. Size and weight limitations vary from one state to another. The designer and/or the precast concrete producer should check these limits with the local authorities in early stages of design to determine the most economical dimensions of the panel. The following bullets provide some guidelines of these restrictions:
   - The common payload for standard trailers without special permits is 20 tons (178 kN) with width restricted to 8-ft (2.44 m).
   - Maximum width with permit varies among states from 10 to 14 ft (3.05 to 4.27 m).
   - Some states allow lengths over 70 ft (21.34 m) with only a simple permit, while others require for any load over 55 ft (15.24 m) a special permit, front and rear escorts. Also, travel may be limited to certain times of the day.
   - In some states, weights of up to 100 tons (890 kN) are allowed with permit, while in other states there are very severe restrictions on loads over 25 tons (222 kN).
2. Panel up to 8 to 10 ft (2.44 to 3.05 m) wide can be shipped horizontally on flatbed trailers. Wider panels can be shipped vertically on flatbed or low-boy trailers using supporting frames.
3. If more than one panel shipped horizontally, 4x4 in. (100x100 mm) temporary support lumber should be provided between individual panels. Also, the temporary supports have to be vertically lined up.

4. The locations of the temporary supports have to be carefully studied to avoid over stressing and/or excessive deflection of the panels. The designer should be aware that long flatbed and lowboy trailers deform during hauling. Support points between the panel and the flatbed must assure statically determinate section, by using spreader beams or strong backs.
5. A 1.33 dynamic load factor should be added to the weight of the panel in addition to the appropriate load factors used with the limit state used in checking the stresses to count for the vibration effects that the panel may encounter during shipping.

C.3.5 Installation

1. Backer rods, polystyrene strips, cementitious strips, or light gage angles, can be used as grout dams.
2. The grout dams have to be well secured to the top surface of the supporting girders to prevent leakage during grouting.
3. Camber of the supporting girders should be taken into consideration during installation of the grout dam. At all points of the length of the dam, a tight contact between the dam and the bottom surface of the precast panel should be provided to avoid leakage during grouting.
4. If longitudinal post-tensioning is provided in the system, grouting of the panel-to-panel transverse connections should be completed and the grout should reach the design strength before longitudinal post-tensioning commences.
5. If longitudinal post-tensioning is used, the construction steps should follow the following sequence:
    - Install the panels on temporarily shims
    - Adjust the panel elevation using the leveling bolts
    - Carefully, splice the post-tensioning ducts
    - Inspect the PT ducts and make sure that they are not blocked
    - Fill the panel-to-panel transverse joints with grout and cure it
    - Re-inspect the PT ducts and make sure that they are not blocked and free of water
    - Insert the post-tensioning tendons tension it
    - Fill the panel-to-girder shear pockets with grout and cure it
    - Use cast-in-place concrete at the closure pours and cure it

C.3.6 Grinding of top surface of the panels

1. Equipment utilizing diamond mounted on a self propelled machine designed for grinding and texturing pavement should be used
2. Equipment that causes ravel, aggregate fracture, spalls or disturbance to the transverse or longitudinal panel-to-panel joints should not be used

3. The deck top surface should be carefully cleaned from residue and excess water
4. Grinding should be performed in lines normal to the pavement centerline
5. The resultant surface should be in a parallel corduroy type texture consisting of grooves with depth between 1/16 and 1/8 in. (1.5 and 3 mm).
6. The drainage cross slope should be maintained with no depressions or misalignment of slope greater than ¼ in. (6 mm) in 10 ft (3.05 m) when tested with a 10 ft (3.05 m) straight edge

C.3.7 Deck removal for deck replacement projects

1. Equipments capable of removing the existing deck without damaging the superstructure should be used
2. If old deck is made composite with the superstructure, special care should be given to avoid damaging the shear connectors as well as the top surface of the supporting girders.
3. Remove shear connectors to the limit that is necessary to avoid interference between the shear connectors and deck panels. In some cases, total removal of the existing shear connector system will be required. The decision of partial or total removal of the shear connectors should be taken by the designer after careful investigation of the existing system and its matching degree with the precast panel system. In some cases, the physical condition of the shear connectors after removing the old deck system (rusty or damaged connectors) may mandate total removal.
4. After deck removal is complete, the top surface of the girder should be thoroughly cleaned from any foreign material such as paint, scale, slag, rust, moisture, grease, concrete debris, etc.
5. For steel girders, the unwanted shear studs should be removed by torch cutting them close to their base. Parts of the studs remained after torch cutting should be removed by grinding.
6. For concrete girders, the shear connectors that are in conflict with the new deck system should be torch cut as close to the top of the girder as possible. Additional pot-installed anchors should be placed directly over the centerline of the girder in accordance with manufacturers recommendations. The top surface of the girder should be cleaned to remove all remains of the old deck by means of sand or water-jet blasting and thorough cleaning.
7. The contractor should provide the following information to the designer for approval:
    - Proposed tools and methods for deck removal
    - Proposed tools and methods for stud removal, if needed
    - Proposed tools and methods for preparation of the top surface of the supporting girders
    - Proposed tools and methods for reattaching the shear connectors

C.4 REFERENCES OF APPENDIX C

1. AASHTO LRFD Bridge Design Specifications, American Association of State Highway and Transportation Officials, Washington, D.C., 3rd Edition (2004) with the 2005 & 2006 Interim Revisions.
2. Badie, S. S., Tadros, M. K. and Girgis, A. F., "Full-Depth, Precast Concrete Bridge Deck Panel Systems," National Cooperative Highway Research Program (NCHRP), Project 12-65, Final Report, July 2006.

APPENDIX D

PROPOSED AASHTO LRFD SPECIFICATIONS REVISIONS

This appendix presents a proposal of revisions to Section 9 of the AASHTO LRFD Specifications. The objective of the proposed revisions is to promote the use the full-depth precast deck panel systems on highway bridges by providing the designers with related requirements and guidance.

AASHTO LRFD SPECIFICATIONS LANGUAGE AND COMMENTARY

The following definition should be added to Article 9.3 of the current AASHTO LRFD Specifications(AASHTO LRFD Bridge Design Specifications, 3rd edition (2004) with the 2005 & 2006 Interim Revisions)..

Construction period- Refers to all the activities that will be conducted in one construction period, which may be an "over-night" or a "weekend" period.

Haunch-The built-up section between the precast deck panels and the supporting girders

The following article shall replace Article 9.7.5 of the current AASHTO LRFD Specifications (AASHTO LRFD Bridge Design Specifications, 3rd edition (2004) with the 2005 & 2006 Interim Revisions).

9.7.5 Precast Deck Slabs on Girders

- 9.7.5.1 General
- 9.7.5.2 Transversely Joined Precast Decks
- 9.7.5.3 Longitudinally Post-Tensioned Precast Decks
- 9.7.5.4 Composite Action between the Slab and the Girders
- 9.7.5.5 Staged Construction
- 9.7.5.6 Panel-to-Girder Joint
- 9.7.5.7 Transverse Panel-to-Panel Joint
- 9.7.5.8 Longitudinal Panel-to-Panel Joint
- 9.7.5.9 Old-to-New Deck Joint
- 9.7.5.10 Detailing Requirements
- 9.7.5.11 Fabrication, Storage, Handling, Transportation and Installation Requirements
- 9.7.5.12 Deck Removal for Deck Replacement Projects

The following references should be added to the REFERENCES of Section 9 of the current AASHTO LRFD Specifications (AASHTO LRFD Bridge Design Specifications, 3rd edition (2004) with the 2005 & 2006 Interim Revisions).

- "Guide Specification for Grouting of Post-Tensioned Structures," February, 2001, Post-Tensioning Institute, First Edition, Post-Tensioning Institute, Phoenix, AZ.
- Badie, S. S., Tadros, M. K. and Girgis, A. F., "Full-Depth, Precast Concrete Bridge Deck Panel Systems," National Cooperative Highway Research Program (NCHRP), Project 12-65, Final Report, November 2006.

9.7.5 Precast Deck Slabs on Girders

9.7.5.1 General

Both reinforced and prestressed precast concrete slab panels may be used. The depth of the slab, excluding any provision for grinding, grooving, and sacrificial surface, shall not be less than 7.0 in.

C9.7.5.1

9.7.5.2 Transversely Joined Precast Decks

Flexurally discontinuous decks made from precast panels and joined together by shear keys may be used. The design of the shear key and the grout used in the key shall be approved by the owner. The provisions of Article 9.7.4.3.4 may be applicable for the design of bedding.

The longitudinal reinforcement of the deck can be made of conventional reinforcement that is spliced at the transverse joints or by utilizing longitudinal post-tensioning.

C9.7.5.2

9.7.5.3 Longitudinally Post-Tensioned Precast Decks

The precast components may be placed on beams and joined together by longitudinal post-tensioning. The minimum average effective prestress shall not be less than 0.25 ksi.

The transverse joint between the components and the block-outs at the coupling of post-tensioning ducts shall be specified to be filled with a nonshrink grout having a minimum compressive strength of ~~5.0~~ 6.0 ksi at ~~24 hours~~ at time of opening bridge to traffic.

Block-outs shall be provided in the slab around the shear connectors and shall be filled with the same grout upon completion of post-tensioning.

The panel-to-girder connection shall not be grouted until post-tensioning of the deck is completed.

When longitudinal post-tensioning is provided with staged construction, the end panel of every *construction period* shall be provided with special devices that allow anchoring of the longitudinal post-tensioning of the current *construction period* and splicing it with the longitudinal post-tensioning of the next "construction period".

C9.7.5.3

Decks made flexurally continuous by longitudinal post-tensioning are the more preferred solution because they behave monolithically and are expected to require less maintenance on the long-term basis.

The post-tensioning ducts should be located at the center of the slab cross section. Block-outs should be provided in the joints to permit the splicing of post-tensioning ducts.

Panels should be placed on the girders without mortar or adhesives to permit their movement relative to the girders during prestressing. Panels can be placed directly on the girders or located with the help of shims of inorganic material or other leveling devices. If the panels are not laid directly on the beams, the space therein should be grouted at the same time as the shear connector block-outs.

A variety of shear key deformations has been used in the past. Recent prototype tests indicate that a "V" joint may be the easiest to form and to fill.

Post-tensioning force should be applied only to the precast deck before it is made composite with the girders. Panels shall be placed on the girders without mortar or adhesives to permit their movement relative to the girders during post-tensioning.

It is possible that the stresses in an end panel due to the longitudinal post-tensioning force will be doubled if two separate anchorages are used for two subsequent construction periods.

9.7.5.4 Considerations before Composite Action between the Slab and the Girders

The deck shall be fully capable of resisting the effects during construction as well as after the bridge is opened to traffic.

Top flange of simply supported girders shall be checked against local/torsional buckling due to loads applied prior to achieving full composite action. Temporary bracing shall be used to support the top flange if local/trosional buckling is a concern.

C9.7.5.4

Construction loads may include weight of crew members, and equipment used in handling and installing the panels. The construction load shall be clearly stated on the project plans.

9.7.5.5 Staged Construction

If no longitudinal post-tensioning is used in the precast deck, all segments of the precast deck constructed during any given *construction period* shall be made composite with the superstructure before the end of that period. The effect of the overall construction scheme on the superstructure shall be carefully investigated.

The start and end locations on the slab of each *construction period* shall be specified on the project documents.

C9.7.5.5

9.7.5.6 Panel-to-Girder Joint

Spacing between stud clusters can be extended up to 4 ft (1220 mm) if confinement to the stud clusters and the grout surrounding them is provided.

Confinement can be provided by using hollow structural steel (HSS) tubes or closed ties set horizontally across the panel thickness. If closed ties are used, the lowest tie should be placed as close to the top surface of the girder as possible.

Equation 5.8.4.1-1 shall be used to estimate the ultimate capacity of stud clusters spaced up to 4 ft (1220 mm) for steel and concrete composite beams. Limits on V_n given by Equation 5.8.4.1-2 and 5.8.4.1-3 shall not apply if confinement is provided.

For steel composite beams:

1. Equation 6.10.10.2-1, that is currently used to estimate the fatigue capacity for single studs, does not require modification for design of stud clusters spaced up to 4 ft (1220 mm).
2. Equation 6.10.10.4.3-1 shall not be used for stud clusters at 2 to 4 ft (610 to 1220 mm) spacing. This recommendation is expected to result in about 30 percent increase in the required number of studs.

C9.7.5.6

Experimental investigation (NCHRP 12-65) has shown that spacing between stud clusters can be extended up to 4 ft (1220 mm), without affecting the full-composite behavior of the system, if confinement to the stud group and the grout surrounding them is provided.

Confinement helps in distributing the shear force among the studs in each cluster, and protecting the grout at the base of each stud against crushing.

This recommendation is based on the results of the of push-off testing of stud groups at 4 ft (1220 mm) spacing, which gave about 30 % lower capacity than the current LRFD equation for single studs. The conclusion may be unnecessarily conservative as the push-off testing is not as realistic in modeling beam behavior as actual beam test, which showed no reduction in capacity due to use of stud clusters. However, this is a conservative approach and it does not significantly impact the overall economy of bridges.

9.7.5.7 Transverse Panel-to-Panel Joint

A grout-filled shear key joint shall be provided between adjacent panels. Direct bearing between adjacent panels shall be avoided.

For bridge decks on a horizontal curvature, trapezoidal panels may be used to keep the width of the joint constant and small. If rectangular panels are used to simplify production of the panels, the narrowest joint width allowed must not be less than 1 in.

Flowable, self-leveling, non-shrink grout mix with satisfactorily freeze-and-thaw resistance shall be used. Grout must fill the full height of the joint. Wood, metal, or foam forms installed under the panel shall be used to achieve this goal.

The shear key shall be sand blasted and thoroughly cleaned before the grout is placed.

If no overlay is used over the precast panels, finishing of the top surface of the grout filling of the joint shall match the finishing of the precast panel.

C9.7.5.7

Previous experience with direct panel-to-panel bearing has shown that high stress concentration exists at the edges of the panel, which leads to concrete spalling and possible moisture leakage through the joint.

Figure C9.7.5.7-1 shows a shear key shape that has been shown to give good performance:

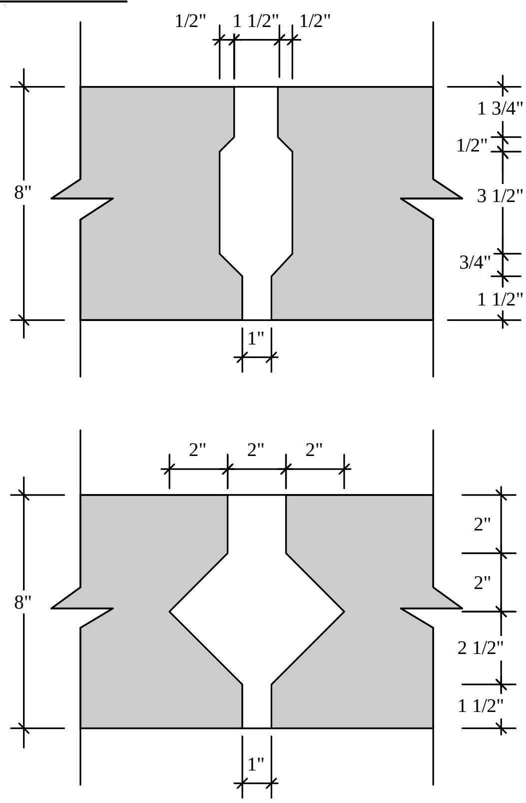


Figure C9.7.5.7-1 -1 Shear Key Details

9.7.5.8 Longitudinal Panel-to-Panel Joint

If the joint is not directly over a girder flange, transverse positive moment reinforcement of the slab shall be adequately coupled. The width of the joint shall be adequate to provide the necessary coupling device.

It is recommended that the temporary barriers be removed before installing closure pours. The weight of the temporary barriers, if left on during closure joint construction, may cause the precast panels to have unacceptable differential deformation.

C9.7.5.8

This may occur due to staged construction or to avoid crowning of the panels.

9.7.5.9 Old-to-New Deck Joint

A temporary edge support shall be provided for the last precast panel, installed in a construction period, unless the panel edge is designed for this temporary condition.

C9.7.5.9

9.7.5.10 Detailing Requirements

If no overlay is provided on the precast deck system, partial-depth shear pockets shall be used, as shown in Figure C9.7.5.10-1.

The depth of the pocket shall satisfy the following conditions:

(a) The top of the shear connector is at least 2 in. (50 mm) above the bottom layer of transverse reinforcement of the panel.

(b) A ½ in. (13 mm) minimum clearance between the top of the shear connector and top of the pocket shall be provided

(c) A 2 in. (50 mm) minimum distance between the top of the shear connector and the top surface of the panel shall be provided

Partial-height pockets shall be provided with injection ports and vents. 1½ to 2 in. diameter injection ports are recommended to provide enough flexibility during injection. Two or more vents shall be provided if the width of the pocket is greater than 12 in. (300 mm). The vents shall be provided on the opposite side of the injection port to guarantee complete filling of the pocket during grouting.

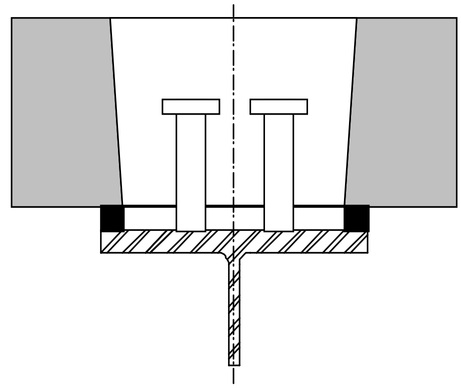
If the spacing between pockets is greater than 24 in. (610 in.), an intermediate injection port should be provided in the panel to guarantee complete grouting of the bedding layer.

Longitudinal post tensioning reinforcement, if used, shall be provided as near mid height of the panel as possible to avoid residual deflection or camber of the slab.

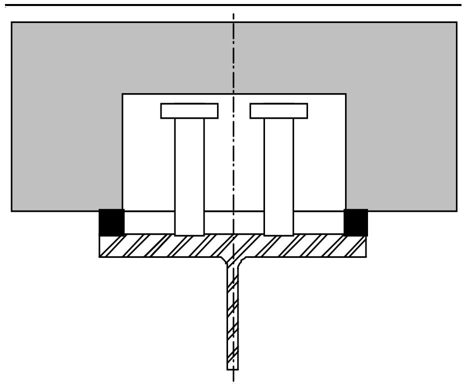
A minimum 1-in. thick haunch shall be provided between the precast panels and the stringers to allow for any misalignment or irregularity.

C9.7.5.10

With no overlay provided on the precast deck panels, partial-depth shear pockets protect the deck from water leakage at the interface between the grout and the surrounding concrete, and minimize non-uniformity of color of the top surface of the deck



Full-depth Pocket



Partial-depth Pocket

Figure C9.7.5.10-1 Full- and partial depth pockets

If it is desired not to totally fill the haunch with grout for the entire width of the top flange, the grout shall extend horizontally at least 2 in. beyond the pocket width. The grouted area must be provided with adequate ventilation to assure absence of trapped air.

Leveling bolts or stay-in-place light gage angle grout dams shall be used to adjust the grade of the precast panels during placement.

All leveling devices, bolts, inserts or light gage angles shall be corrosion resistant.

The leveling bolts can be removed after the grout between the deck and the girders gain the design strength. If the designer opted to permanently keep the leveling bolts after the grouting process is complete, the bolts shall be torch cut and recessed at least 2 in. (50 mm) below the top surface of the panel. Concrete blockouts shall be provided during fabrication of the panel for that purpose. After torch cutting the bolts, these pockets shall be filled with non-shrink cementitious grout of a concrete strength that at least matches that of the precast panel.

Galvanized stay-in-place (SIP) light gage angle grout dams have been recently used on bridges in Nebraska and Texas. The SIP angles, similar to those used for SIP forms, are welded to the top flange or to straps across the top flange to secure them in place. They are installed at the required pre-calculated haunch height minus a fraction of an inch to allow room for a closed cell foam grout seal bonded to the top surface of the angle. Also, allowance is made for pre-calculated deflection due to deck weight.

9.7.5.11 Fabrication, Storage, Handling, Transportation and Installation Requirements

C9.7.5.11

9.7.5.11.1 Fabrication

It is recommended to standardize dimensions and details of panels needed for a project as much as possible.

The precast concrete producer shall be responsible for design of the lifting inserts and their location in coordination with the rigging scheme plans of the erector. The location of lifting inserts and their specifications along with the rigging scheme shall be clearly shown on the shop drawings prepared by the precast producer. The shop drawings shall be reviewed and approved by the design engineer.

9.7.5.11.2 Storage and Handling

If the panels are stored horizontally, they shall be supported on temporary supports located as near as possible to the permanent support locations.

During storage, the panels shall be protected from environmental effects, such as rain and snow to protect any exposed steel components from rusting.

During lifting and handling, panels shall be subjected to vertical lifting forces only, through use of spreader beams and strong backs as needed.

Non-planar slabs may require special support provisions to prevent unwanted stress in the slabs during storage and shipping. Shipping supports may also need to be specially designed to prevent unwanted stresses in the slabs as a result of truck bed twisting during shipping.

9.7.5.11.3 Transportation

Since handling will induce dynamic loads on the panel, a 1.33 dynamic load factor shall be used in addition to the load factors associated with the considered Limit State (i.e. Service, Ultimate, etc.).

Transportation of panels, especially if they are transported in a flat position, must be done with extreme care. Support points between the panel and the flatbed must assure statically determinate section, by using spreader beams or strong backs.

9.7.5.11.4 Installation

Post-tensioning is to be completed in accordance with the provisions of Section 5 of these specifications. Special effort shall be made to provide accurate alignment of post-tensioning ducts from panel to panel. Grouting of post tensioning ducts shall be done using thixotropic grout material and shall be performed in accordance to the following publication: "Guide Specification for Grouting of Post-Tensioned Structures."

Backer rods, polystyrene strips, cementitious strips, or light gage angles, can be used as grout dams.

If longitudinal post-tensioning is provided in the system, grouting of the panel-to-panel transverse connections shall be completed and the grout shall reach the design strength before longitudinal post-tensioning commences.

Equipment utilizing diamond mounted on a self propelled machine designed for grinding and texturing pavement shall be used

Care shall be exercised as not to cause ravel, aggregate fracture, spalling or disturbance to the transverse or longitudinal panel-to-panel joints.

Grinding shall be performed in lines normal to the pavement centerline.

The resultant surface shall be in a parallel corduroy-type texture consisting of grooves with depth between 1/16 and 1/8 in.

The drainage cross slope shall be maintained with no depressions or misalignment of slope greater than ¼ in. in 10 ft when tested with a 10-ft long straight edge.

9.7.5.12 Deck Removal for Deck Replacement Projects

Special care shall be given to avoid damaging the shear connectors as well as the top surface of the supporting girders.

The contractor shall provide the following information to the design engineer for approval:

- Proposed tools and method used for deck removal
- Proposed tools and method used for stud removal, if needed
- Proposed tools and method used for preparation of the top surface of the supporting girders
- Proposed tools and method used for reattaching the shear connectors.

For steel girders, the shear studs interfering with the new panels shall be removed by torch cutting as close to the stud base as possible.

For concrete girders, the shear connectors that are in conflict with the new deck system shall be torch cut as close to the top of the girder as possible. Additional pot-installed anchors shall be placed directly over the centerline of the girder in accordance with manufacturers recommendations. The top surface of the girder shall be cleaned to remove all remains of the old deck by means of sand or water-jet blasting and thorough cleaning.

C9.7.5.12

APPENDIX E

SPECIFICATIONS OF SELECTED COMMERCIAL GROUT MATERIAL

BASF
The Chemical Company

PRODUCT DATA

3 03 01 20 **Concrete Rehabilitation**

SET® 45 AND SET® 45 HW

Chemical-action repair mortar

Description

Set® 45 is a one-component magnesium phosphate-based patching and repair mortar. This concrete repair and anchoring material sets in approximately 15 minutes and takes rubber-tire traffic in 45 minutes. It comes in two formulations: Set® 45 Regular for ambient temperatures below 85° F (29° C) and Set® 45 Hot Weather for ambient temperatures ranging from 85 to 100° F (29 to 38° C).

Yield

A 50 lb (22.7 kg) bag of mixed with the required amount of water produces a volume of approximately 0.39 ft³ (0.011 m³); 60% extension using 1/2" (13 mm) rounded, sound aggregate produces approximately 0.58 ft³ (0.016 m³).

Packaging

50 lb (22.7 kg) multi-wall bags

Color

Dries to a natural gray color

Shelf Life

1 year when properly stored

Storage

Store in unopened containers in a clean, dry area between 45 and 90° F (7 and 32° C).

Features	Benefits
• Single component	Just add water and mix
• Reaches 2,000 psi compressive strength in 1 hour	Rapidly returns repairs to service
• Wide temperature use range	From below freezing to hot weather exposures
• Superior bonding	Bonds to concrete and masonry without a bonding agent
• Very low drying shrinkage	Improved bond to surrounding concrete
• Resistant to freeze/thaw cycles and deicing chemicals	Usable in most environments
• Only air curing required	Fast, simple curing process
• Thermal expansion and contraction similar to Portland cement concrete	More permanent repairs
• Sulfate resistant	Stable where conventional mortars degrade

Where to Use

APPLICATION

- Heavy industrial repairs
- Dowel bar replacement
- Concrete pavement joint repairs
- Full-depth structural repairs
- Setting of expansion device nosings
- Bridge deck and highway overlays
- Anchoring iron or steel bridge and balcony railings
- Commercial freezer rooms
- Truck docks
- Parking decks and ramps
- Airport runway-light installations

LOCATION

- Horizontal and formed vertical or overhead surfaces
- Indoor and outdoor applications

How to Apply

Surface Preparation

1. A sound substrate is essential for good repairs. Flush the area with clean water to remove all dust.

2. Any surface carbonation in the repair area will inhibit chemical bonding. Apply a pH indicator to the prepared surface to test for carbonation.

3. Air blast with oil-free compressed air to remove all water before placing Set® 45.

Protection and Repair

Technical Data

Composition

Set® 45 is a magnesium-phosphate patching and repair mortar.

Test Data

PROPERTY	RESULTS				TEST METHODS
Typical Compressive Strengths*, psi (MPa)					ASTM C 109, modified
	Plain Concrete 72° F (22° C)	**Set® 45 Regular 72° F (22° C)**	**Set® 45 Regular 36° F (2° C)**	**Set® 45 HW 95° F (35° C)**	
1 hour	—	2,000 (13.8)	—	—	
3 hour	—	5,000 (34.5)	—	3,000 (20.7)	
6 hour	—	5,000 (34.5)	1,200 (8.3)	5,000 (34.5)	
1 day	500 (3.5)	6,000 (41.4)	5,000 (34.5)	6,000 (41.4)	
3 day	1,900 (13.1)	7,000 (48.3)	7,000 (48.3)	7,000 (48.3)	
28 day	4,000 (27.6)	8,500 (58.6)	8,500 (58.6)	8,500 (55.2)	

NOTE: Only Set® 45 Regular formula, tested at 72° F (22° C), obtains 2,000 psi (13.8 MPa) compressive strength in 1 hour.

PROPERTY	RESULTS		TEST METHODS
Modulus of Elasticity, psi (MPa)	**7 days**	**28 days**	ASTM C 469
Set® 45 Regular	4.18×10^6 (2.88×10^4)	4.55×10^6 (3.14×10^4)	
Set® 45 Hot Weather	4.90×10^6 (3.38×10^4)	5.25×10^6 (3.62×10^4)	
Freeze/thaw durability test, % RDM, 300 cycles, for Set® 45 and Set 45® HW		80	ASTM C 666, Procedure A (modified**)
Scaling resistance to deicing chemicals, Set® 45 and Set 45® HW			ASTM C 672
5 cycles		0	
25 cycles		0	
50 cycles		1.5 (slight scaling)	
Sulfate resistance			ASTM C 1012
Set® 45 length change after 52 weeks, %		0.09	
Type V cement mortar after 52 weeks, %		0.20	
Typical setting times, min, for Set® 45 at 72° F (22° C), and Set® 45 Hot Weather at 95° F (35° C)			Gilmore ASTM C 266, modified
Initial set		9 – 15	
Final set		10 – 20	
Coefficient of thermal expansion,*** both Set® 45 Regular and Set® 45 Hot Weather coefficients		7.15×10^{-6}/° F (12.8×10^{-6}/° C)	CRD-C 39
Flexural Strength, psi (MPa), 3 by 4 by 16" (75 by 100 by 406 mm) prisms, 1 day strength,			ASTM C 78, modified
Set® 45 mortar		550 (3.8)	
Set® 45 mortar with 3/8" (9 mm) pea gravel		600 (4.2)	
Set® 45 mortar with 3/8" (9 mm) crushed angular noncalcareous hard aggregate		650 (4.5)	

* All tests were performed with neat material (no aggregate)

**Method discontinues test when 300 cycles or an RDM of 60% is reached.

***Determined using 1 by 1 by 11" (25 mm by 25 mm by 279 mm) bars. Test was run with neat mixes (no aggregate). Extended mixes (with aggregate) produce lower coefficients of thermal expansion.

Test results are averages obtained under laboratory conditions. Expect reasonable variations.

Mixing

1. Set® 45 must be mixed, placed, and finished within 10 minutes in normal temperatures (72° F [22° C]). Only mix quantities that can be placed in 10 minutes or less.

2. Do not deviate from the following sequence; it is important for reducing mixing time and producing a consistent mix. Use a minimum 1/2" slow-speed drill and mixing paddle or an appropriately sized mortar mixer. Do not mix by hand.

3. Pour clean (potable) water into mixer. Water content is critical. Use a maximum of 4 pts (1.9 L) of water per 50 lb (22.7 kg) bag of Set® 45. Do not deviate from the recommended water content.

4. Add the powder to the water and mix for approximately 1 – 1-1/2 minutes.

5. Use neat material for patches from 1/2 – 2" (6 – 51 mm) in depth or width. For deeper patches, extend a 50 lb (22.7 kg) bag of Set® 45 HW by adding up to 30 lbs (13.6 kg) of properly graded, dust-free, hard, rounded aggregate or noncalcareous crushed angular aggregate, not exceeding 1/2" (6 mm) in accordance with ASTM C 33, #8. If aggregate is damp, reduce water content accordingly. Special procedures must be followed when angular aggregate is used. Contact your local BASF representative for more information. (Do not use calcareous aggregate made from soft limestone.
Test aggregate for fizzing with 10% HCL).

Application

1. Immediately place the mixture onto the properly prepared substrate. Work the material firmly into the bottom and sides of the patch to ensure good bond.

2. Level the Set® 45 and screed to the elevation of the existing concrete. Minimal finishing is required. Match the existing concrete texture.

Curing

No curing is required, but protect from rain immediately after placing. Liquid-membrane curing compounds or plastic sheeting may be used to protect the early surface from precipitation, but never wet cure Set® 45.

For Best Performance

- Color variations are not indicators of abnormal product performance.
- Regular Set® 45 will not freeze at temperatures above -20° F (-29° C) when appropriate precautions are taken.
- Do not add sand, fine aggregate, or Portland cement to Set® 45.
- Do not use Set® 45 for patches less than 1/2" (13 mm) deep. For deep patches, use Set® 45 Hot Weather formula extended with aggregate, regardless of the temperature. Consult your BASF representative for further instructions.
- Do not use limestone aggregate.
- Water content is critical. Do not deviate from the recommended water content printed on the bag.
- Precondition these materials to approximately 70° F (21° C) for 24 hours before using.
- Protect repairs from direct sunlight, wind, and other conditions that could cause rapid drying of material.
- When mixing or placing Set® 45 in a closed area, provide adequate ventilation.
- Do not use Set® 45 as a precision nonshrink grout.
- Never featheredge Set® 45; for best results, always sawcut the edges of a patch.
- Prevent any moisture loss during the first 3 hours after placement. Protect Set® 45 with plastic sheeting or a curing compound in rapid-evaporation conditions.
- Do not wet cure.
- Do not place Set® 45 on a hot (90° F [32° C]), dry substrate.
- When using Set® 45 in contact with galvanized steel or aluminum, consult your local BASF sales representative.
- Make certain the most current versions of product data sheet and MSDS are being used; call Customer Service (1-800-433-9517) to verify the most current versions.
- Proper application is the responsibility of the user. Field visits by BASF personnel are for the purpose of making technical recommendations only and not for supervising or providing quality control on the jobsite.

Health and Safety

SET® 45

Caution

Risks

Eye irritant. Skin irritant. Lung irritant. May cause delayed lung injury.

Precautions

KEEP OUT OF THE REACH OF CHILDREN. Avoid contact with eyes. Wear suitable protective eyewear. Avoid prolonged or repeated contact with skin. Wear suitable gloves. Wear suitable protective clothing. Do not breathe dust. In case of insufficient ventilation, wear suitable respiratory equipment. Wash soiled clothing before reuse.

First Aid

Wash exposed skin with soap and water. Flush eyes with large quantities of water. If breathing is difficult, move person to fresh air.

Waste Disposal Method

This product when discarded or disposed of is not listed as a hazardous waste in federal regulations. Dispose of in a landfill in accordance with local regulations.

For additional information on personal protective equipment, first aid, and emergency procedures, refer to the product Material Safety Data Sheet (MSDS) on the job site or contact the company at the address or phone numbers given below.

Proposition 65

This product contains materials listed by the state of California as known to cause cancer, birth defects, or reproductive harm.

VOC Content

0 lbs/gal or 0 g/L.

For medical emergencies only, call ChemTrec (1-800-424-9300).

BASF Building Systems
889 Valley Park Drive
Shakopee, MN, 55379
www.BASFBuildingSystems.com
Customer Service 800-433-9517
Technical Service 800-243-6739

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The Chemical Company

PRODUCT DATA

3 03 36 00 **Grouts**

CONSTRUCTION GROUT

General construction, mineral-aggregate nonshrink grout

Description

Construction Grout is a noncatalyzed, multi-purpose construction grout containing mineral aggregate.

Yield

One 50 lb (22.7 kg) bag of Construction Grout mixed with 1.15 gallons (4.35 L) of water (flowable mix) provides approximately 0.45 ft^3 (0.013 m^3) of mixed grout.

Packaging

50 lb (22.7 kg) multi-wall paper bags

Color

Concrete gray when cured

Shelf Life

1 year when properly stored

Storage

Store in unopened bags under clean, dry conditions.

Features	Benefits
• Concrete gray color (after curing)	Blends in with surrounding concrete
• No organic accelerators, including chlorides or other salts	Will not corrode reinforcing steel
• Can be extended with clean, well-graded coarse aggregate	Fills large voids without additional mix water
• Hardens free of bleeding when properly placed	Provides high effective bearing area for proper support and load transfer

Where to Use

APPLICATION

- Normal loads for columns and baseplates
- Bedding grout for precast panels
- Repairing of cavities resulting from ineffective concrete consolidation
- Caulking concrete pipe
- Backfilling, underpinning foundations, and pressure grouting of slabs needing alignment
- General construction applications
- Damp pack applications

LOCATION

- Interior or exterior

How to Apply

Application

Consult the Construction Grout product bag for details on the installation of Construction Grout. For aggregate extension guidelines refer to Appendix MB-10: Guide to Cementitious Grouting.

Curing

Cure all exposed grout shoulders by wet curing for 24 hours and by applying a recommended curing compound compliant with ASTM C 309 or preferably ASTM C 1315.

For Best Performance

- Contact your local representative for a pre-job conference to plan the installation.
- Construction Grout is designed for the 50 to 90° F (10 to 32° C) application temperature range. Consult your BASF representative when applying outside this range. Use cold and hot weather concreting practices (ACI 305 and ACI 306) when grouting within 10° F (6° C) of these minimum and maximum temperature ranges.
- To ensure optimum performance of Construction Grout, place at a plastic or flowable consistency and at ambient temperatures of 50° F (10° C) and above.
- For best results, allow a minimum of 1" (25 mm) vertical clearance under baseplates when placing Construction Grout.
- Do not use Construction Grout where it will come in contact with steel designed for stresses above 80,000 psi (552 MPa). Use Masterflow® 816, Masterflow® 1205, or Masterflow® 1341 post-tensioning cable grouts.
- Do not add plasticizers, accelerators, retarders, or other additives unless advised in writing by BASF Technical Services.
- The surface to be grouted should be clean, strong, and roughened to CSP 5 – 9 according to ICRI Guideline 03732 to permit proper bond. For freshly placed concrete, consider using Liquid Surface Etchant (see Form No. 1020198).
- Do not place Construction Grout in lifts greater than 6" (152 mm) unless the product is extended with aggregate to dissipate hydration heat.

Protection and Repair

Technical Data

Composition

Construction Grout is a noncatalyzed hydraulic cement-based grout containing mineral aggregate.

Compliances

- CRD C 621 and ASTM C 1107, Grade C, at flowable or plastic consistency
- City of Los Angeles Research Report Number RR 23137

Typical Properties

Mixed Grout Data* (Flowable Mix)

PROPERTY	VALUE
Approximate Water, gal (L)	1.15 (4.35)
Initial set, hrs, at 70° F (21° C)	6
Final set, hrs, at 70° F (21° C)	8

*At a constant percent of water, consistency will vary with temperature. Final set takes place in approximately 8 hours at a flowable consistency and 70° F (21° C).

Test Data

PROPERTY	RESULTS	TEST METHODS
Flow, %, 5 drops	126 – 145	ASTM C230
Volume change, %, flowable consistency, after 28 days	0.08	ASTM C 1090
Compressive strength, psi (MPa)		ASTM C 942, according to ASTM C 1107

	Consistency		
	Flowable[1]	**Plastic**[2]	**Stiff**[3] (damp pack)
1 day	1,500 (10)	—	—
3 days	5,000 (34.5)	6,000 (41.4)	8,000 (55.2)
7 days	6,000 (41.3)	7,000 (48.3)	9,500 (65.5)
28 days	7,000 (48.0)	8,500 (58.6)	10,000 (69.0)

[1] 140% flow on flow table, ASTM C 230, 5 drops in 3 seconds
[2] 100% flow on flow table, ASTM C 230, 5 drops in 3 seconds
[3] 40% flow on flow table, ASTM C 230, 5 drops in 3 seconds
Test results are averages obtained under laboratory conditions. Reasonable variations can be expected.

- Where precision alignment and severe service, such as heavy loading, rolling, or impact resistance are required, use metallic-reinforced, noncatalyzed Embeco® 885 grout. If the amount of impact resistance needed is not great enough to require metallic reinforcement, use natural-aggregate, Masterflow® 928.
- The water requirement may vary with mixing efficiency, temperature, and other variables.
- The concrete surfaces should be saturated (ponded) with clean water for 24 hours before grouting. Remove water immediately before application.
- Make certain the most current versions of product data sheet and MSDS are being used; call Customer Service (1-800-433-9517) to verify the most current versions.
- Proper application is the responsibility of the user. Field visits by BASF personnel are for the purpose of making technical recommendations only and not for supervising or providing quality control on the jobsite.

Health and Safety

CONSTRUCTION GROUT

Caution

Risks

Eye irritant. Skin irritant. Causes burns. Lung irritant. May cause delayed lung injury.

Precautions

KEEP OUT OF THE REACH OF CHILDREN. Avoid contact with eyes. Wear suitable protective eyewear. Avoid prolonged or repeated contact with skin. Wear suitable gloves. Wear suitable protective clothing. Do not breathe dust. In case of insufficient ventilation, wear suitable respiratory equipment. Wash soiled clothing before reuse.

First Aid

Wash exposed skin with soap and water. Flush eyes with large quantities of water. If breathing is difficult, move person to fresh air.

Waste Disposal Method

This product when discarded or disposed of is not listed as a hazardous waste in federal regulations. Dispose of in a landfill in accordance with local regulations. For additional information on personal protective equipment, first aid, and emergency procedures, refer to the product Material Safety Data Sheet (MSDS) on the job site or contact the company at the address or phone numbers given below.

Proposition 65

This product contains materials listed by the state of California as known to cause cancer, birth defects, or reproductive harm.

VOC Content

0 lbs/gal or 0 g/L.

For medical emergencies only, call ChemTrec (1-800-424-9300).

BASF Building Systems

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BASF
The Chemical Company

PRODUCT DATA

3 03 36 00 **Grouts**

SS MORTAR®

Splice-sleeve grout

Description

SS Mortar® is a cement-based metallic-aggregate mortar for grouting of NMB Splice Sleeves. This nonshrink, high-strength, ready-to-use grout is used for NMB Class-1 and Class-2 splice sleeves.

Yield

One 55 lb (25 kg) bag of SS Mortar® mixed with 8.25 lbs (3.75 kg) of water, produces approximately 0.42 ft³ (0.012 m³) of material.

Packaging

55 lb (25 kg) multi-wall paper bags

Shelf Life

1 year when properly stored

Storage

Store in a cool, clean, dry environment.

Features	Benefits
• Extended working time	Allows for ease of placement
• Can be installed from 40 to 90° F (4 to 32° C)	Apply over a wide range of temperatures and conditions
• Can be pumped or poured	Facilitates rapid placement
• Hardens free of bleeding, settlement, or drying shrinkage	Lessens weather dependency

Where to Use

APPLICATION

- NMB splice-sleeve mechanical connectors for Class-1 and Class-2 sleeves; the purchaser should specify the type of coupler being grouted

How to Apply

Surface Preparation

1. For pre-grout applications, ensure that the splice sleeves are clean and free of all debris, water, and any other foreign matter before starting grouting operations.

2. For post-grout applications, remove the hole seals in the grout inlet and outlet tubes and inspect for blockage. Inspect the sleeves and grout tubes with a bright light to ensure there are no foreign materials or obstructions. Blow air through the sleeves using the inlet or outlet tube to verify there are no obstructions.

Temperature

1. Store and mix mortar to produce the required mortar temperature under jobsite conditions. Use warm water in cold weather and cold water in hot weather. Ideally, the splice sleeve or substrate should be in the 50 to 90° F (10 to 32° C) range. Mixed mortar temperatures should also fall between 50 and 90° F (10 and 32° C).

Recommended Temperature Guidelines

PREFERRED RANGE 50 – 80° F (10 – 27° C)

	MINIMUM ° F (° C)	MAXIMUM ° F (° C)
Splice sleeves	40 (4)	90 (32)
Mixing water	40 (4)	90 (32)
SS Mortar® as mixed and placed	50 (10)	90 (32)

CAUTION: When grouting at minimum temperatures, see that the splice-sleeve and grout temperatures do not fall below 40° F (4° C) until after final set. Protect the grout from freezing (32° F [0° C]) until it has reached 1,500 psi (10.3 MPa) compressive strength. Excessive grout temperatures may result in difficult pumping and premature stiffening.

Technical Data

Composition

SS Mortar® is a hydraulic cement-based metallic-aggregate mortar.

Compliances

- ICBO requirements for Type II couplers

Test Data

PROPERTY	RESULTS	TEST METHODS
Compressive strengths, psi (MPa), at 70° F (21° C)		ASTM C 942
1 day	4,000 (28)	
3 days	5,400 (38)	
7 days	7,000 (49)	
28 days	11,000 (76)	
Typical flexural strengths, psi (MPa)		ASTM C 348
7 days	1,000 (6.9)	
28 days	1,100 (7.6)	

The data shown are based on controlled laboratory tests. Expect reasonable variations from the results shown because of varying temperatures and atmospheric conditions at the jobsite. Control field and laboratory tests on the basis of the desired placing consistency rather than strictly on water content.

Consistency

Description of flow test

Use the BASF Flow Guide that consists of a 2 by 4" (51 by 102 mm) cylinder placed in the center of a level, smooth, nonabsorbent surface. The cylinder is filled with SS Mortar® level with the surface and immediately but slowly lifted until the SS Mortar® is discharged. Measure the diameter of the spread in two locations perpendicular to one another and take the average of the two readings. The BASF Flow Guide is available from BASF but may also be assembled on the jobsite using rigid, non-absorbent materials.

Flow tests were run using the BASF Flow Guide at a spread of 5 – 6" (127 – 152 mm).

Mixing

1. Use potable water only. For mixing grout, use an electric drill with a mixing blade or a horizontal-shaft mortar mixer. Do not mix by hand. Do not add cement, sand, aggregate, admixtures, or other additives unless specifically advised in writing by BASF Technical Service.

2. The amount of water needed to produce the desired consistency will depend upon mixing time, the type of mixer, the temperature of the grout following mixing, and the size of the batch. A batch should contain increments of full bags. The recommended field consistency is 5.0 – 6.0" (127 – 152 mm) as determined by the use of the BASF Flow Guide.

3. The suggested amount of mixing water for the initial trial mix to produce this flow is 12 – 15% by weight of SS Mortar® or 1 gallon (8.34 lbs) [3.8 L or kg] per 55 lb (25 kg) bag.

4. If additional water is required to meet the consistency specification, it should not result in a BASF Flow Guide diameter of greater than 6.5" (165 mm), and the total water content of the mix should not exceed a maximum of 17% by weight of SS Mortar or 1.1 gallon (9.36 lbs) [4.2 L or kg)] per 55 lb (25 kg) bag.

5. As the first step in mixing, place all water into the mixing pail, then pour all of the SS Mortar® into the pail while stirring it with a high-speed mixer. The water requirement should be established in a test batch. After all materials are put into the pail, mix grout for a minimum of 5 minutes or longer, if required, for a uniform mixture. After the grout has been mixed, use it within 30 minutes. Do not retemper grout by adding water and remixing after it stiffens.

6. The BASF Flow Guide for SS Mortar® is designed for placement at a spread of 5 – 6" (127 – 152 mm) (maximum of 6.5" (165 mm) when a cylinder of nominal dimensions—2" (51 mm) in diameter by 4" (102 mm) in length—is filled and lifted off a nonabsorbent flat surface (such as a BASF Flow Guide). The exact amount of water needed will depend upon the temperature of the grout following mixing and the size of the batch mixed, but it should not be greater than 17% of the total mix by weight. Warm mixing water, not exceeding 90° F (32° C), may be used either with cold grout or with placements at the lower temperature limit of 50° F (10° C).

Application

1. In pre-grout applications, pour the grout continuously into the sleeve and rod with a small diameter rod (such as a welding rod) to remove any entrapped air.

2. In post-grout applications, pump the grout into the inlet tube until it flows freely without air bubbles from the outlet tube. Seal the outlet tube with a rubber stopper of the proper size. Remove the pump nozzle from the inlet tube and immediately seal the inlet tube with a rubber stopper to avoid any loss of grout.

3. Following grouting operations, remove the filling connections and verify the absence of any voids and trapped debris.

Curing

Cure all exposed grout by wet curing for 24 hours and then applying a recommended curing compound compliant with ASTM C 309 or preferably ASTM C 1315.

For Best Performance

- The water requirement may vary with mixing efficiency, temperature, and other variables.
- Do not add plasticizers, accelerators, retarders, or other additives unless advised in writing by BASF Technical Service.
- Contact your local representative for a pre-job conference to plan the installation.
- Store SS Mortar® in a dry area at a controlled temperature (50 to 90° F [10 to 32° C]). Mix SS Mortar® to produce the desired mixed-grout temperatures under jobsite conditions. Material may be stored warmer for cold-weather applications or cooler for hot-weather applications.
- Ideally, the splice sleeve or substrate should be in the 50 to 90° F (10 to 32° C) range. The temperature of the mixed grout should fall between 50 and 90° F (10 to 32° C). Consider using heated water in cold weather or chilled water in hot weather to help adjust the mixed grout temperature.
- When grouting at minimum temperatures, see that the splice sleeve and grout temperatures do not fall below 40° F (4° C) until final set and that the grout is protected from freezing (32° F [0° C]). Maintain heat until the grout in the sleeves has reached a minimum of 1,500 psi (10.3 MPa) compressive strength as determined by 2" by 2" by 2" (51 mm) restrained cubes (ASTM C 942 or ASTM C 1107) in metal cube molds. Grout temperatures outside the recommended range may result in difficulty pumping and premature stiffening.
- An authorized laboratory should perform testing of SS Mortar®; conduct sampling at the jobsite. Follow ASTM C 942 or ASTM C 1107 for sampling, fabrication, storage, and curing of specimens. Samples must be moist cured or cured under water at the jobsite for 24 hours before transfer to the laboratory for wet or moist-room curing. Samples may be tested to determine early strength gain to determine when the sleeves have developed sufficient strength to remove temporary supports or to determine compliance with project requirements. Consult your local BASF representative for additional information.
- Do not use if bag is damaged.
- Make certain the most current versions of product data sheet and MSDS are being used; call Customer Service (1-800-433-9517) to verify the most current versions.
- Proper application is the responsibility of the user. Field visits by BASF personnel are for the purpose of making technical recommendations only and not for supervising or providing quality control on the jobsite.

Health and Safety

SS MORTAR®

Caution

Risks

Eye irritant. Skin irritant. Causes burns. Lung irritant. May cause delayed lung injury.

Precautions

KEEP OUT OF THE REACH OF CHILDREN. Avoid contact with eyes. Wear suitable protective eyewear. Avoid prolonged or repeated contact with skin. Wear suitable gloves. Wear suitable protective clothing. Do not breathe dust. In case of insufficient ventilation, wear suitable respiratory equipment. Wash soiled clothing before reuse.

First Aid

Wash exposed skin with soap and water. Flush eyes with large quantities of water. If breathing is difficult, move person to fresh air.

Waste Disposal Method

This product when discarded or disposed of is not listed as a hazardous waste in federal regulations. Dispose of in a landfill in accordance with local regulations.

For additional information on personal protective equipment, first aid, and emergency procedures, refer to the product Material Safety Data Sheet (MSDS) on the job site or contact the company at the address or phone numbers given below.

Proposition 65

This product contains materials listed by the state of California as known to cause cancer, birth defects, or reproductive harm.

VOC Content

0 lbs/gal or 0 g/L.

For medical emergencies only, call ChemTrec (1-800-424-9300).

BASF
The Chemical Company

PRODUCT DATA

3 03 36 00 **Grouts**

MASTERFLOW® 928

High-precision mineral-aggregate grout with extended working time

Description

Masterflow® 928 grout is a hydraulic cement-based mineral-aggregate grout with an extended working time. It is ideally suited for grouting machines or plates requiring precision load-bearing support. It can be placed from fluid to damp pack over a temperature range of 45 to 90° F (7 to 32° C). Masterflow® 928 grout meets the requirements of ASTM C 1107, Grades B and C, and the Army Corp of Engineers' CRD C 621, Grades B and C, at a fluid consistency over a 30-minute working time.

Yield

One 55 lb (25 kg) bag of Masterflow® 928 grout mixed with approximately 10.5 lbs (4.8 kg) or 1.26 gallons (4.8 L) of water, yields approximately 0.50 ft³ (0.014 m³) of grout.

The water requirement may vary due to mixing efficiency, temperature, and other variables.

Packaging

55 lb (25 kg) multi-wall paper bags

3,300 lb (1,500 kg) bulk bags

Shelf Life

1 year when properly stored

Storage

Store in unopened bags in clean, dry conditions.

Features	Benefits
Extended working time	Ensures sufficient time for placement
Can be mixed at a wide range of consistencies	Ensures proper placement under a variety of conditions
Freeze/thaw resistant	Suitable for exterior applications
Hardens free of bleeding, segregation, or settlement shrinkage	Provides a maximum effective bearing area for optimum load transfer
Contains high-quality, well-graded quartz aggregate	Provides optimum strength and workability
Sulfate resistant	For marine, wastewater, and other sulfate-containing environments

Where to Use

APPLICATION

- Where a nonshrink grout is required for maximum effective bearing area for optimum load transfer
- Where high one-day and later-age compressive strengths are required
- Nonshrink grouting of machinery and equipment, baseplates, soleplates; precast wall panels, beams, columns; curtain walls, concrete systems, other structural and nonstructural building members; anchor bolts, reinforcing bars, and dowel rods
- Applications requiring a pumpable grout
- Repairing concrete, including grouting voids and rock pockets
- Marine applications
- Freeze/thaw environments

LOCATION

- Interior or exterior

How to Apply

Surface Preparation

1. Steel surfaces must be free of dirt, oil, grease, or other contaminants.

2. The surface to be grouted must be clean, SSD, strong, and roughened to a CSP of 5 – 9 following ICRI Guideline 03732 to permit proper bond. For freshly placed concrete, consider using Liquid Surface Etchant (see Form No. 1020198) to achieve the required surface profile.

3. When dynamic, shear or tensile forces are anticipated, concrete surfaces should be chipped with a "chisel-point" hammer, to a roughness of (plus or minus) 3/8" (10 mm). Verify the absence of bruising following ICRI Guideline 03732.

4. Concrete surfaces should be saturated (ponded) with clean water for 24 hours just before grouting.

5. All freestanding water must be removed from the foundation and bolt holes immediately before grouting.

6. Anchor bolt holes must be grouted and sufficiently set before the major portion of the grout is placed.

Protection and Repair

Technical Data

Composition

Masterflow® 928 is a hydraulic cement-based mineral-aggregate grout.

Compliances

- ASTM C 1107, Grades B and C, and CRD 621, Grades B and C, requirements at a fluid consistency over a temperature range of 40 to 90° F (4 to 32° C)
- City of Los Angeles Research Report Number RR 23137

Test Data

PROPERTY	RESULTS			TEST METHODS
Compressive strengths, psi (MPa)				ASTM C 942, according to ASTM C 1107
		Consistency		
	Plastic[1]	**Flowable**[2]	**Fluid**[3]	
1 day	4,500 (31)	4,000 (28)	3,500 (24)	
3 days	6,000 (41)	5,000 (34)	4,500 (31)	
14 days	7,500 (52)	6,700 (46)	6,500 (45)	
28 days	9,000 (62)	8,000 (55)	7,500 (52)	
Volume change*				ASTM C 1090
	% Change	**% Requirement of ASTM C 1107**		
1 day	> 0	0.0 – 0.30		
3 days	0.04	0.0 – 0.30		
14 days	0.05	0.0 – 0.30		
28 days	0.06	0.0 – 0.30		
Setting time, hr:min				ASTM C 191
		Consistency		
	Plastic[1]	**Flowable**[2]	**Fluid**[3]	
Initial set	2:30	3:00	4:30	
Final set	4:00	5:00	6:00	
Flexural strength,* psi (MPa)				ASTM C 78
3 days	1,000 (6.9)			
7 days	1,050 (7.2)			
28 days	1,150 (7.9)			
Modulus of elasticity,* psi (MPa)				ASTM C 469, modified
3 days	2.82×10^{6} (1.94×10^{4})			
7 days	3.02×10^{6} (2.08×10^{4})			
28 days	3.24×10^{6} (2.23×10^{4})			
Coefficient of thermal expansion,* in/in/° F (mm/mm/° C)	6.5×10^{-6} (11.7×10^{-6})			ASTM C 531
Split tensile and tensile strength,* psi (MPa)				ASTM C 496 (splitting tensile) ASTM C 190 (tensile)
	Splitting Tensile	**Tensile**		
3 days	575 (4.0)	490 (3.4)		
7 days	630 (4.3)	500 (3.4)		
28 days	675 (4.7)	500 (3.4)		
Punching shear strength,* psi (MPa), 3 by 3 by 11" (76 by 76 by 279 mm) beam				BASF Method
3 days	2,200 (15.2)			
7 days	2,260 (15.6)			
28 days	2,650 (18.3)			
Resistance to rapid freezing and thawing	300 Cycles RDF 99%			ASTM C 666, Procedure A

[1]100 – 125% flow on flow table per ASTM C 230

[2]125 – 145% flow on flow table per ASTM C 230

[3]25 to 30 seconds through flow cone per ASTM C 939

*Test conducted at a fluid consistency

Test results are averages obtained under laboratory conditions. Expect reasonable variations.

Test Data, continued

PROPERTY		RESULTS		TEST METHODS
Ultimate tensile strength and bond stress				ASTM E 488, tests*
Diameter in (mm)	**Depth in (mm)**	**Tensile strength lbs (kg)**	**Bond stress psi (MPa)**	
5/8 (15.9)	4 (101.6)	23,500 (10,575)	2,991 (20.3)	
3/4 (19.1)	5 (127.0)	30,900 (13,905)	2,623 (18.1)	
1 (25.4)	6.75 (171.5)	65,500 (29,475)	3,090 (21.3)	

*Average of 5 tests in ≥ 4,000 psi (27.6 MPa) concrete using 125 ksi threaded rod in 2" (51mm) diameter, damp, core-drilled holes.

Notes:

1. Grout was mixed to a fluid consistency.
2. Recommended design stress: 2,275 psi (15.7 MPa).
3. Refer to the "Adhesive and Grouted Fastener Capacity Design Guidelines" for more detailed information.
4. Tensile tests with headed fasteners were governed by concrete failure.

Jobsite Testing

If strength tests must be made at the jobsite, use 2" (51 mm) metal cube molds as specified by ASTM C 942 and ASTM C 1107. DO NOT use cylinder molds. Control field and laboratory tests on the basis of desired placement consistency rather than strictly on water content.

7. Shade the foundation from sunlight 24 hours before and 24 hours after grouting.

Forming

1. Forms should be liquid tight and nonabsorbent. Seal forms with putty, sealant, caulk, polyurethane foam.
2. Moderately sized equipment should utilize a head form sloped at 45 degrees to enhance the grout placement. A moveable head box may provide additional head at minimum cost.
3. Side and end forms should be a minimum 1" (25 mm) distant horizontally from the object grouted to permit expulsion of air and any remaining saturation water as the grout is placed.
4. Leave a minimum of 2" between the bearing plate and the form to allow for ease of placement.
5. Use sufficient bracing to prevent the grout from leaking or moving.
6. Eliminate large, nonsupported grout areas wherever possible.
7. Extend forms a minimum of 1" (25 mm) higher than the bottom of the equipment being grouted.
8. Expansion joints may be necessary for both indoor and outdoor installation. Consult your local BASF field representative for suggestions and recommendations.

Temperature

1. For precision grouting, store and mix grout to produce the desired mixed-grout temperature. If bagged material is hot, use cold water, and if bagged material is cold, use warm water to achieve a mixed-product temperature as close to 70° F (21°C) as possible.

Recommended Temperature Guidelines for Precision Grouting

	MINIMUM °F (°C)	PREFERRED °F (°C)	MAXIMUM °F (°C)
Foundation and plates	45 (7)	50 – 80 (10 – 27)	90 (32)
Mixing water	45 (7)	50 – 80 (10 – 27)	90 (32)
Grout at mixed and placed temp	45 (7)	50 – 80 (10 – 27)	90 (32)

2. If temperature extremes are anticipated or special placement procedures are planned, contact your local BASF representative for assistance.
3. When grouting at minimum temperatures, see that the foundation, plate, and grout temperatures do not fall below 40° F (7° C) until after final set. Protect the grout from freezing (32° F or 0° C) until it has attained a compressive strength of 3,000 psi (21 MPa).

Mixing

1. Place estimated water (use potable water only) into the mixer, then slowly add the grout. For a fluid consistency, start with 9 lbs (4 kg) (1.1 gallon [4.2L]) per 55 lb bag.
2. The water demand will depend on mixing efficiency, material, and ambient-temperature conditions. Adjust the water to achieve the desired flow. Recommended flow is 25 – 30 seconds using the ASTM C 939 Flow-Cone Method. Use the minimum amount of water required to achieve the necessary placement consistency.
3. Moderately sized batches of grout are best mixed in one or more clean mortar mixers. For large batches, use ready-mix trucks and 3,300 lb (1,500 kg) bags for maximum efficiency and economy.
4. Mix grout a minimum of 5 minutes after all material and water is in the mixer. Use mechanical mixer only.
5. Do not mix more grout than can be placed in approximately 30 minutes.
6. Transport by wheelbarrow or buckets or pump to the equipment being grouted. Minimize the transporting distance.
7. Do not retemper grout by adding water and remixing after it stiffens.
8. DO NOT VIBRATE GROUT TO FACILITATE PLACEMENT.

9. For aggregate extension guidelines, refer to Appendix MB-10: Guide to Cementitious Grouting.

Application

1. Always place grout from only one side of the equipment to prevent air or water entrapment beneath the equipment. Place Masterflow® 928 in a continuous pour. Discard grout that becomes unworkable. Make sure that the material fills the entire space being grouted and that it remains in contact with plate throughout the grouting process.

2. Immediately after placement, trim the surfaces with a trowel and cover the exposed grout with clean wet rags (not burlap). Keep rags moist until grout surface is ready for finishing or until final set.

3. The grout should offer stiff resistance to penetration with a pointed mason's trowel before the grout forms are removed or excessive grout is cut back. After removing the damp rags, immediately coat with a recommended curing compound com-pliant with ASTM C 309 or preferably ASTM C 1315.

4. Do not vibrate grout. Use steel straps inserted under the plate to help move the grout.

5. Consult your BASF representative before placing lifts more than 6" (152 mm) in depth.

Curing

Cure all exposed grout with an approved membrane curing compound compliant with ASTM C 309 or preferably ASTM C 1315. Apply curing compound immediately after the wet rags are removed to minimize potential moisture loss.

For Best Performance

- For guidelines on specific anchor-bolt applications, contact BASF Technical Service.
- Do not add plasticizers, accelerators, retarders, or other additives unless advised in writing by BASF Technical Service.
- The water requirement may vary with mixing efficiency, temperature, and other variables.
- Hold a pre-job conference with your local representative to plan the installation. Hold conferences as early as possible before the installation of equipment, sole plates, or rail mounts. Conferences are important for applying the recommendations in this product data sheet to a given project, and they help ensure a placement of highest quality and lowest cost.
- The ambient and initial temperature of the grout should be in the range of 45 to 90° F (7 to 32° C) for both mixing and placing. Ideally the amount of mixing water used should be that which is necessary to achieve a 25 – 30 second flow according to ASTM C 939 (CRD C 611). For placement outside of the 45 to 90° F (7 to 32° C) range, contact your local BASF representative.
- For pours greater than 6" (152 mm) deep, consult your local BASF representative for special precautions and installation procedures.
- Use Embeco® 885 grout for dynamic load-bearing support and similar application conditions as Masterflow® 928.
- Use Masterflow® 816, Masterflow® 1205, or Masterflow® 1341 post-tensioning cable grouts when the grout will be in contact with steel stressed over 80,000 psi (552 MPa).
- Masterflow® 928 is not intended for use as a floor topping or in large areas with exposed shoulders around baseplates. Where grout has exposed shoulders, occasional hairline cracks may occur. Cracks may also occur near sharp corners of the baseplate and at anchor bolts. These superficial cracks are usually caused by temperature and moisture changes that affect the grout at exposed shoulders at a faster rate than the grout beneath the baseplate. They do not affect the structural, nonshrink, or vertical support provided by the grout if the foundation-preparation, placing, and curing procedures are properly carried out.
- The minimum placement depth is 1" (25 mm).
- Make certain the most current versions of product data sheet and MSDS are being used; call Customer Service (1-800-433-9517) to verify the most current version.
- Proper application is the responsibility of the user. Field visits by BASF personnel are for the purpose of making technical recommendations only and not for supervising or providing quality control on the jobsite.

Health and Safety

MASTERFLOW® 928

Caution

Risks

Eye irritant. Skin irritant. Causes burns. Lung irritant. May cause delayed lung injury.

Precautions

KEEP OUT OF THE REACH OF CHILDREN. Avoid contact with eyes. Wear suitable protective eye-wear. Avoid prolonged or repeated contact with skin. Wear suitable gloves. Wear suitable protective clothing. Do not breathe dust. In case of insufficient ventilation, wear suitable respiratory equipment. Wash soiled clothing before reuse.

First Aid

Wash exposed skin with soap and water. Flush eyes with large quantities of water. If breathing is difficult, move person to fresh air.

Waste Disposal Method

This product when discarded or disposed of, is not listed as a hazardous waste in federal regulations. Dispose of in a landfill in accordance with local regulations.

For additional information on personal protective equipment, first aid, and emergency procedures, refer to the product Material Safety Data Sheet (MSDS) on the job site or contact the company at the address or phone numbers given below.

Proposition 65

This product contains materials listed by the state of California as known to cause cancer, birth defects, or reproductive harm.

VOC Content

0 lbs/gal or 0 g/L.

For medical emergencies only, call ChemTrec (1-800-424-9300).

BASF Building Systems

889 Valley Park Drive
Shakopee, MN, 55379

www.BASFBuildingSystems.com

Customer Service 800-433-9517
Technical Service 800-243-6739

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Construction Chemicals

747 RAPID-SETTING GROUT

Rapid-setting, nonshrink cement-based grout

PRODUCT DATA

3 03600 **Grouts**

Description

747 Rapid-Setting Grout is a nonmetallic cement-based grout. It can be used in a wide range of grouting applications—wherever a rapid-setting material is needed to secure heavy equipment and machinery bases, bearing plates, posts, and bolts. 747 Rapid-Setting Grout is NSF / ANSI approved for use with potable water.

Features	Benefits
• High early strength	Minimized downtime
• Nonshrink	Reduces stress at the bond line; maximizes effective bearing area (EBA)
• Chloride free	High early strength without the risk of corrosion
• Nonrusting	Suitable for outdoor use
• NSF / ANSI approved	For use with potable water

Yield

Approximately 0.50 ft³ (0.015 m³) per 55 lb (25 kg) bag

Yield may vary, depending upon mixed consistency

Packaging

55 lb (25 kg) bag

Shelf Life

1 year when properly stored

Storage

Store and transport in clean, dry conditions from 40 to 85° F (4 to 29° C) in unopened containers. High temperatures or high humidity will reduce the shelf life.

Where to Use

APPLICATION

- Structural columns
- Pump bases
- Heavy equipment and machinery bases
- Bearing plates
- Precast tee joints
- Rail posts, seating bolts

LOCATION

- Interior or exterior

SUBSTRATE

- Concrete

How to Apply

Surface Preparation

1. Concrete must be fully cured (28 days). Surfaces must be free from oil, grease, or any loose material.
2. If the concrete surface is defective or has laitance, it must be cut back to a sound base.
3. Blow boltholes or mortar pockets clear of any dirt or debris.

BASE-PLATE PREPARATION

1. The area being grouted must be clean and free from oil, grease, or scale.
2. Provide air-pressure relief holes to allow venting of any isolated high spots.

LEVELING SHIMS

Treat any shims that will be removed after the grout has hardened with a thin layer of grease, wax, or form-release agent.

FORMWORK

1. Use design methods that will enable the grout to flow by gravity.
2. Use forms of sufficient strength to provide for rapid, continuous, and complete grout placement.
3. On the pouring side, slant the form 45 degrees. Allow a minimum entrance clearance of 3" (76 mm). On the opposite side, provide for a minimum 1" (25 mm) horizontal distance between the form and bed-plate and a minimum 1" (25 mm) height for the rising head grout. Aim to minimize exposed area of grout on the other two sides of the form.

ThoRoc

www.DegussaBuildingSystems.com

Technical Data

Composition

747 Rapid-Setting Grout is a nonmetallic cement-based grout.

Compliances

- CRD C 621
- ASTM C 1107, Class C (modified for rapid-setting grout)

Test Data

PROPERTY	RESULTS			TEST METHODS
	Plastic psi (MPa)	**Flowable psi (MPa)**	**Fluid psi (MPa)**	
Compressive strength				ASTM C 109 at 77° F
1 Day	8,000 (55)	7,560 (52)	4,625 (32)	
3 Days	9,050 (62)	8,235 (57)	6,950 (48)	
7 Days	11,650 (80)	10,560 (73)	8,280 (57)	
14 Days	12,250 (84)	11,510 (79)	8,755 (60)	
28 Days	12,950 (89)	12,640 (87)	9,230 (64)	
Water by weight, %	11.4	12.3	15.8	
Flow table analysis, % 5 drops in 3 sec	123	140	N/A	ASTM C 230
Flow cone analysis, sec	N/A	N/A	23 – 28	CRD C 611
Setting time				ASTM C 191
Initial set, min	18 – 22	22 – 26	N/A	
Final set, min	70 – 80	95 – 105	N/A	

All application and performance values are typical for the material, but may vary with the test method, conditions, and configurations.

PRE-SOAKING

Several hours before grouting, presoak the area to achieve a saturated surface-dry (SSD) condition. Immediately before grouting, remove any standing water, taking particular care to blow out all boltholes and pockets.

Mixing

1. Use a mechanical mixer to ensure complete wetting of the material. Do not use free-fall mixers. For small quantities of 1 – 2 bags, use a slow-speed electric drill with suitable paddle.

2. Use an accurate measuring method to obtain the desirable consistency. The amount of water added must be precise. The consistencies described below conform to CRD C 621.

Water required for each 55 lb (25 kg) unit:

Fluid consistency: 4-1/4 qts (4.0 L)

Flowable consistency: 3 – 3-1/4 qts (2.8 – 3.0 L)

Plastic consistency: 2-3/4 – 3 qts (2.6 – 2.8 L)

In cold conditions, warm water (95 to 110° F [35 to 43° C]) may be used to accelerate the strength development.

AGGREGATE EXTENSION

- For areas 2 – 4" (51 – 102 mm) in depth, add a minimum of 15 – 25 lbs (6.8 – 11.4 kg) of 3/8" (10 mm) clean, washed, nonreactive, well-graded pea gravel per 50 lb (22.7 kg) bag.
- For areas greater than 4" (102 mm) in depth, add a minimum of 25 – 50 lbs (11.4 – 22.7 kg) of 3/8" (10 mm) clean, washed, nonreactive, well-graded pea gravel per bag.
- The maximum aggregate extension is 50 lbs (22.7 kg) of pea gravel per bag.

The type and condition of the aggregate and the amount added can affect the performance of 747 Rapid-Setting Grout. Use trials, testing, and previous experience to determine aggregate amounts and suitability.

Application

1. The machine mixing capacity and labor must be adequate to ensure the grouting operation proceeds continuously. This may require the use of a holding tank that can be gently agitated to maintain the mixture's fluidity.

2. Accurately measure the selected water content into the mixer. Slowly add the total contents of the 747 Rapid-Setting Grout bag. Mix continuously for 1 – 2 minutes, making sure that a smooth, even consistency is obtained.

3. Place the grout within 5 minutes of mixing. Minimum application thickness is 1/2" (13 mm).

4. Where large volumes will be placed, 747 Rapid-Setting Grout may be pumped. For best results, use a heavy-duty diaphragm pump.

Placing

1. A continuous grout flow is essential when placing 747 Rapid-Setting Grout. Have sufficient grout available before starting, and regulate the time taken to pour a batch with the time taken to prepare the next.

2. Pour and place the grout from one side only to eliminate voids in the cured grout caused by air entrapment or surplus pre-soaking water. Maintain a grout head at all times to achieve a continuous grout flow.

Curing

After the grouting operation, thoroughly cure any exposed areas that will not be cut back; use concrete-curing methods and practices in accordance with ACI 308.

Clean Up

Before curing, 747 Rapid-Setting Grout may be cleaned up using water. Cured material must be mechanically removed.

For Best Performance

- Not recommended for placing below 35° F (2° C).
- Do not exceed limitations set by the American Concrete Institute.
- Pretreat wood forms with form oil.
- Do not retemper the grout after initial mixing.
- Make certain the most current versions of product data sheet and MSDS are being used; call Customer Service (1-800-433-9517) to verify the most current version.
- Proper application is the responsibility of the user. Field visits by Degussa personnel are for the purpose of making technical recommendations only and not for supervising or providing quality control on the jobsite.

Health and Safety

747 RAPID-SETTING GROUT

Caution

747 Rapid-Setting Grout contains silicon dioxide, Portland cement, amorphous silica, calcium sulfate, and fly ash.

Risks

Product is alkaline on contact with water and may cause injury to skin or eyes. Ingestion or inhalation of dust may cause irritation. Contains free respirable quartz, which has been listed as a suspected human carcinogen by NTP and IARC. Repeated or prolonged overexposure to free respirable quartz may cause silicosis or other serious and delayed lung injury.

Precautions

KEEP OUT OF THE REACH OF CHILDREN. Prevent contact with skin and eyes. Prevent inhalation of dust. DO NOT take internally. Use only with adequate ventilation. Use impervious gloves, eye protection and if the TLV is exceeded or used in a poorly ventilated area, use NIOSH/MSHA approved respiratory protection in accordance with applicable federal, state and local regulations.

First Aid

In case of eye contact, flush thoroughly with water for at least 15 minutes. SEEK IMMEDIATE MEDICAL ATTENTION. In case of skin contact, wash affected areas with soap and water. If irritation persists, SEEK MEDICAL ATTENTION. Remove and wash contaminated clothing. If inhalation causes physical discomfort, remove to fresh air. If discomfort persists or any breathing difficulty occurs or if swallowed, SEEK IMMEDIATE MEDICAL ATTENTION.

Refer to Material Safety Data Sheet (MSDS) for further information.

Proposition 65

This product contains material listed by the state of California as known to cause cancer, birth defects, or other reproductive harm.

VOC Content

0 lbs/gal or 0 g/L

For medical emergencies only, call ChemTrec (1-800-424-9300).

degussa.
Construction Chemicals

SONOGROUT® 10K

Cementitious, shrinkage-compensated, nonmetallic high-strength grout

PRODUCT DATA

3 03600 **Grouts**

Description

Sonogrout® 10K is Portland-cement-based, shrinkage-compensated high-strength grout that conforms to Corps of Engineers CRD-C-621-83 and ASTM C 1107. Its nonmetallic formula does not rust, bleed, or harm metals on contact. Its cost-effectiveness makes it ideal for large jobs.

Features	Benefits
• One component	Mixes easily with water
• Dimensionally stable	Needs no special treatment at edges, opened ends, cappings, or cut backs
• Dry pack to pourable consistency	Easy to place; versatile
• No added chlorides, iron, or gypsum	Will not rust, bleed, or harm metals on contact
• Freeze/thaw cycling stable	Excellent durability
• Nonefflorescing	Maintains a clean appearance; may be painted or coated

Yield

0.40 ft³/50 lb bag
(0.011 m³/22.7 kg bag)

Packaging

50 lb (22.7 kg) bags

Color

Concrete gray

Shelf life

9 months when properly stored

Storage

Store in unopened containers in a clean, dry area.

Where to Use

APPLICATION

- Structural columns
- Machinery bases
- Bearing plates
- Pump and equipment bases
- Rail posts
- Seating bolts
- Power-line stanchions
- Food and chemical plants
- General new construction
- Restoration

LOCATION

- Interior or exterior

SUBSTRATE

- Concrete

How to Apply

Surface Preparation

1. All areas to be grouted must be clean and free of oil, grease, dirt, and contaminants. Remove all loose materials.

2. Concrete must be fully cured a minimum of 28 days.

3. Where required, provide air-relief openings to avoid entrapment of air.

4. All metal components to be in contact with Sonogrout® 10K must be free of rust, paint, or oils. For additional protection, coat reinforcing steel with Sonoprep™ Plus (See Form No. 1017986).

5. All concrete to come into contact with the grout must be thoroughly saturated with clean water for a minimum of 12 hours before placement of grout. Remove excess water from holes and voids just before grout placement.

Sonneborn®

www.DegussaBuildingSystems.com

Technical Data

Composition

Sonogrout® 10K contains Portland cement and silicon dioxide.

Compliances

- Corps of Engineers CRD-C-621
- ASTM C 1107, Grade A
- City of Los Angeles Research Report # RR25187T

Test Data

PROPERTY	RESULTS	TEST METHODS
Compressive strength, psi (MPa)		ASTM C 109
Dry pack		
28 days	11,500 (79.2)	
Very stiff paste		
28 days	10,000 (68.9)	
Minimum flow (soft to plastic)		
1 day	2,900 (20.0)	
3 days	6,500 (44.8)	
7 days	7,200 (49.6)	
28 days	9,200 (63.4)	
Moderate flow (plastic to pourable)		
1 day	1,600 (11.0)	
3 days	3,800 (26.2)	
7 days	5,100 (35.2)	
28 days	6,200 (42.7)	

Dry pack mixed at 2 qts water/50 lb bag (1.9 L /22.7 kg bag); very stiff paste mixed at 2-1/4 qts water/50 lb bag (2.14 L/22.7 kg bag); minimum flow tested at 2-1/3 qts water/50 lb bag (2.21 L/22.7 kg bag); moderate flow tested at 3-1/8 qts water/50 lb bag (3.0 L/22.7 kg bag).

PROPERTY	RESULTS	TEST METHODS
Volume change, %		ASTM C 1090
1 day	+0.032	
3 days	+0.032	
7 days	+0.034	
14 days	+0.034	
28 days	+0.035	

Test results are averages obtained under laboratory conditions. Reasonable variations can be expected.

Mixing

1. Precondition Sonogrout® 10K to 70° F ±5° (21° C ±3°) before mixing.

2. Sonogrout® 10K is ready to use and requires only the addition of water. Use the minimum water required to achieve the desired placement consistency, approximately the following amounts.

   a) Dry pack: 2 qts/50 lb bag (1.9 L/ 22.7 kg bag)
   b) Very stiff paste: 2-1/4 qts/50 lb bag (2.14 L/22.7 kg bag)
   c) Minimum flow: 2-1/3 qts/50 lb bag (2.21 L/22.7 kg bag)
   d) Moderate flow: 3-1/8 qts/50 lb bag (3.0 L/22.7 kg bag)

3. Mix mechanically with a slow speed drill and mixing paddle

4. Mix no longer than 5 minutes and place immediately.

Application

1. For column base plates and machinery bases, follow forming procedures that allow for rapid and continuous placement of the grout and complete filling of the space to be grouted.

2. Support elements should be anchored so that no movement is possible. Remove supports only after Sonogrout® 10K has hardened sufficiently.

3. Sonogrout® 10K should be placed in accordance with standard grouting procedures and ACI recommendations for placing and curing of concrete.

4. Use chains, rods, or tamping devices to compact grout tightly, completely removing all air voids. Place grout quickly and continuously, striking off exposed areas. The minimum placement depth is 1/2" (13 mm).

5. For grouting beyond 2" (51 mm) in depth, extend Sonogrout® 10K with up to 50 lbs (22.7 kg) of properly sized, washed, saturated surface-dry (SSD), graded, low-absorption, high-density aggregate. Use ACI concrete guidelines for proper practices regarding aggregate extension.

6. Water addition and temperatures will determine set time. From dry pack to flowable, set time will vary from approximately 3 – 8 hours. Lower temperatures will lengthen and higher temperatures will shorten the working and setting times.

Curing

Curing of the installed Sonogrout® 10K is mandatory. Either moist cure for at least 3 days or use any ASTM C 309-compliant Sonneborn® curing compound on the exposed surfaces. Take extra care to retain moisture on installations in direct sunlight, high temperatures, and wind.

Clean Up

Clean all tools and equipment with water immediately. Cured material can be removed by mechanical means only.

For Best Performance

- Place Sonogrout® 10K at a temperature as close to 70° F (21° C) as possible.
- Pre-treat wood surfaces that can absorb moisture with forming oils.
- Edges of concrete to be grouted that are less than 1" (25 mm) thick must be cut back to form a square-cut edge.
- For improved bond to existing concrete, use either EpoGrip™ or Sonoprep™ Plus (see Form No. 1017922 or 1017986).
- Exceeding published water requirements will decrease performance.
- Do not add any cement or any other additives to Sonogrout® 10K.
- Do not retemper grout after initial mixing.
- Exposed finished grout must be cured; use sheeting, water, or any Sonneborn® curing compound.
- Handle like concrete regarding protection against temperatures and weather; do not exceed limitations set by the American Concrete Institute on placement of concrete.
- When grouting particularly heavy equipment or when anticipating excessive vibration, consider the use of Sonneborn® 14K Hy Flow (see Form No. 1017942).
- Additional information on grouts and trade practices can be found in "Cementitious Grouts and Grouting," published by the Portland Cement Association.
- Make certain the most current versions of product data sheet and MSDS are being used; call Customer Service (1-800-433-9517) to verify the most current version.
- Proper application is the responsibility of the user. Field visits by Degussa personnel are for the purpose of making technical recommendations only and not for supervising or providing quality control on the jobsite.

Health and Safety

SONOGROUT® 10K

Caution

Sonogrout® 10K contains Portland cement, and silicon dioxide (quartz).

Risks

Product is alkaline on contact with water and may cause injury to skin or eyes. Ingestion or inhalation of dust may cause irritation. Contains small amount of free respirable quartz which has been listed as a suspected human carcinogen by NTP and IARC. Repeated or prolonged overexposure to free respirable quartz may cause silicosis or other serious and delayed lung injury.

Precautions

KEEP OUT OF THE REACH OF CHILDREN. Prevent contact with skin and eyes. Prevent inhalation of dust. DO NOT take internally. Use only with adequate ventilation. Use impervious gloves, eye protection and if the TLV is exceeded or is used in a poorly ventilated area, use NIOSH/MSHA approved respiratory protection in accordance with applicable federal, state and local regulations.

First Aid

In case of eye contact, flush thoroughly with water for at least 15 minutes. SEEK IMMEDIATE MEDICAL ATTENTION. In case of skin contact, wash affected areas with soap and water. If irritation persists, SEEK MEDICAL ATTENTION. Remove and wash contaminated clothing. If inhalation causes physical discomfort, remove to fresh air. If discomfort persists or any breathing difficulty occurs, SEEK IMMEDIATE MEDICAL ATTENTION. If swallowed, SEEK IMMEDIATE MEDICAL ATTENTION.

Refer to Material Safety Data Sheet (MSDS) for further information.

Proposition 65

This product contains materials listed by the state of California as known to cause cancer, birth defects, or other reproductive harm.

VOC Content

0 lbs/gal or 0 g/L, less water and exempt solvents.

For medical emergencies only, call ChemTrec (1-800-424-9300).

APPENDIX F

FINITE ELEMENT ANALYSIS

F.1 PANEL-TO-CONCRETE GIRDER CONNECTION

The finite element analysis was used to investigate the behavior of the slab/concrete girder pullout specimens. A commercial finite element package "NASTRAN" was used in the analysis. The concrete specimen was modeled using the 8-node cubic 3-D element. Each node has three displacement degrees of freedom, in the x, y and z directions. The "x" direction is transverse to the girder longitudinal axis, the "y" direction is parallel to the girder longitudinal axis, and the "z" direction is parallel to the girder height. The following mechanical properties were assigned to the concrete specimen: compressive concrete strength = 8 ksi (55 MPa), unit weight = 150 pcf (23.6 kN/m^3), and Poisson ratio = 0.15.

The 1¼ in. (31.8 mm) studs and web reinforcement bars were also modeled using the 20-node cubic element. The circular cross sectional area of the stud's stem and head and the web reinforcement bars were replaced with the equivalent square cross sectional area, as shown in Table F-1. This simplification helped to refine the mesh in the vicinity of the 1¼ in. (31.8 mm) studs. The following mechanical properties were assigned to the stud: tensile strength = 64 ksi (441 MPa), yield strength = 54 ksi (372 MPa), unit weight = 490 pcf (76.9 kN/m^3), and Poisson ratio = 0.30.

Table F-1. Dimensions of the Equivalent Square Area used for the Finite Element Analysis

	Actual diameter (in.)	Cross sectional area (in^2)	Equivalent square area (in. x in.)
Stud stem	1.25	1.227	1.108 x 1.108
Stud head	2.5	4.909	2.216 x 2.216
#4 bar	0.5	0.200	0.447 x 0.477

The following mechanical properties were assigned to the vertical web reinforcement: yield strength = 60 ksi (414 MPa), unit weight = 490 pcf (76.9 kN/m^3), and Poisson ratio = 0.3. Figure F-1 shows the details of the finite element model. Each stud was loaded with a tensile axial force equivalent to the stud yield capacity, 66.4 kips (295 kN). This load was applied as a surface load uniformly distributed on the stud cross sectional area, 54 ksi (372 MPa).

Figure F-2 shows the principal stress distribution on the top and side surfaces of the specimens. Please, note that for Group #2 specimen, the additional web reinforcement helped in widening the base area of the inverted pyramid, which resulted in lower stress concentration at the junction between the top flange and the web. This observation is consistent with the experimental program results, where the size of the side crack at failure of the Group #1 specimens was wider than that of Group #2 specimens.

In order to study the internal stress concentration around the studs, three sections were chosen as shown in Figure F-3. Section 1-1 is at the free side of the external stud, section 2-2 is at the mid distance between two adjacent studs, and section 3-3 is at the centerline of the center stud. The z-direction and principal stress distributions for these three sections are given in Figures F-4 to F-6. Studying these figures indicates that:

1. The additional web reinforcement helped to distribute the tension force provided by the studs on a wider and deeper volume resulting in reducing the stress concentration around the studs. This can be seen from the following observations:

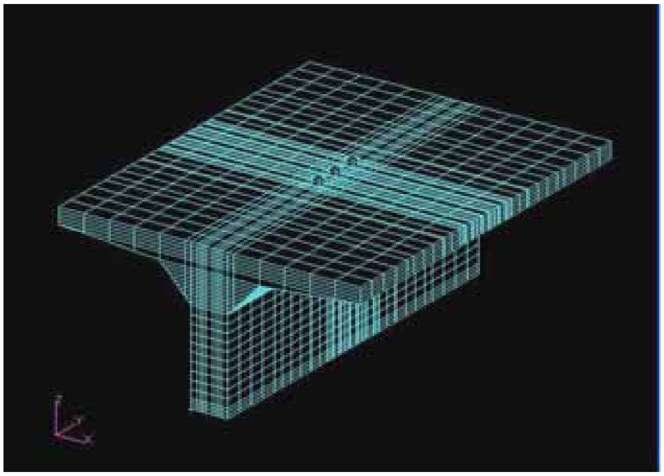
   1.a) Stress concentration at the flange-to-web junction in Group #1 specimen is higher than that of Group #2 specimen

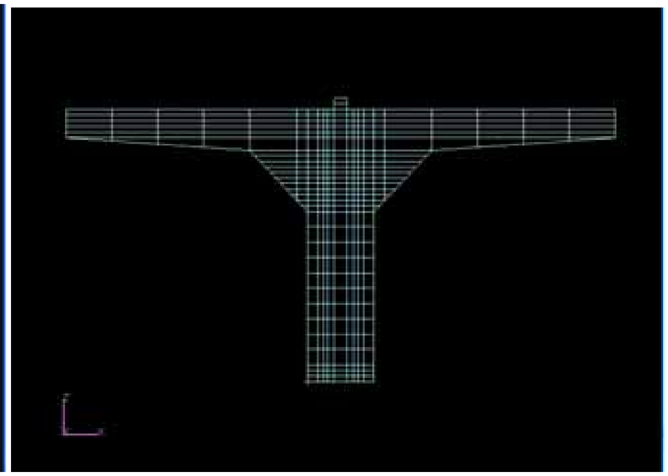
   1.b) The concrete stress in the vicinity of the stud's stem in Group #1 specimen is higher and extends for deeper distance than that of Group #2 specimen

2. The stress distribution at section 3-3 (in the z-direction or principal stress) shows that the proposed 18 in. (457 mm) embedment of the additional web reinforcement is quite enough to develop its yield strength. The high tensile stresses generated in concrete between adjacent rows of additional web reinforcement do not extend to the bottom surface of the concrete specimen. This finding is consistent with the experimental test results where no signs of slippage or vertical side-surface cracking parallel to the additional web reinforcement were observed.

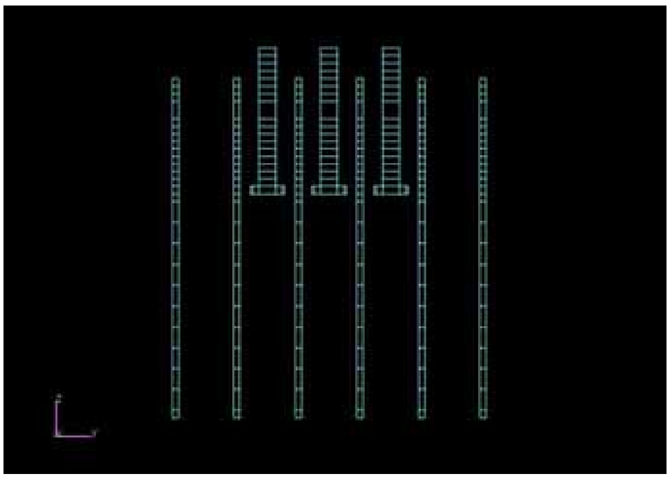
3. The principal stresses at all sections are higher than the z-direction stresses due to the specimen test up that puts the top flange of the specimen in tension.

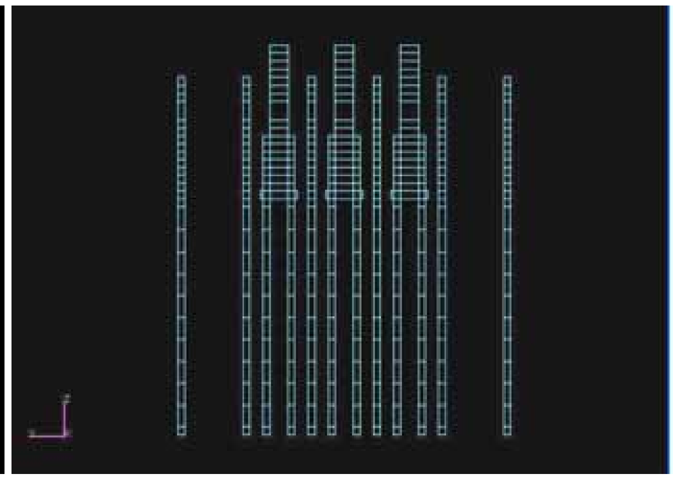
4. The compressive stress at the flange-web junction is about 2.0 ksi (14 MPa), which is less than the concrete bearing strength, 0.85x 8 ksi = 6.8 ksi (47 MPa). This observation is consistent with the test result as no concrete crushing in this location was observed at failure. It is believed that the web reinforcement helped to confine the concrete and consequently protected it from premature cracking.



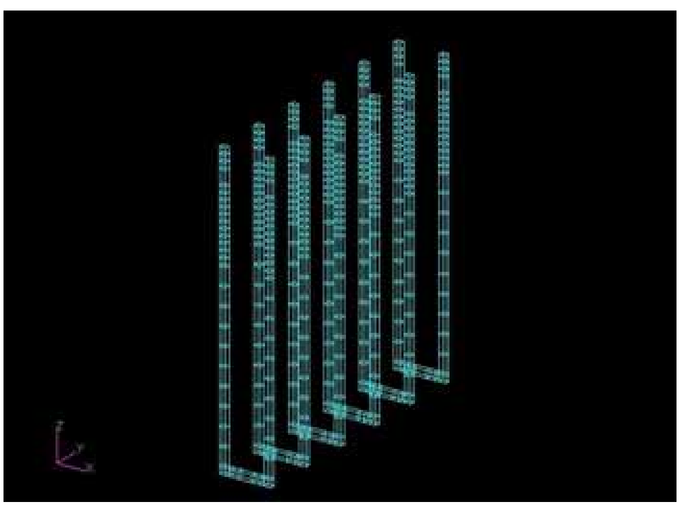


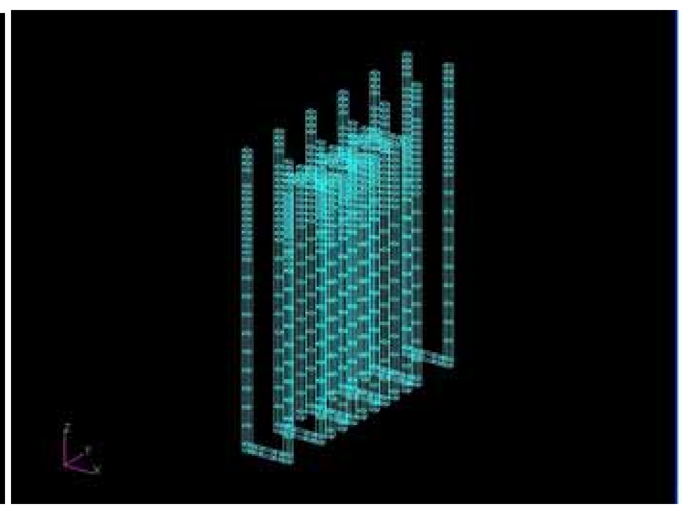
(a) Full model (b) Typical cross section of the model





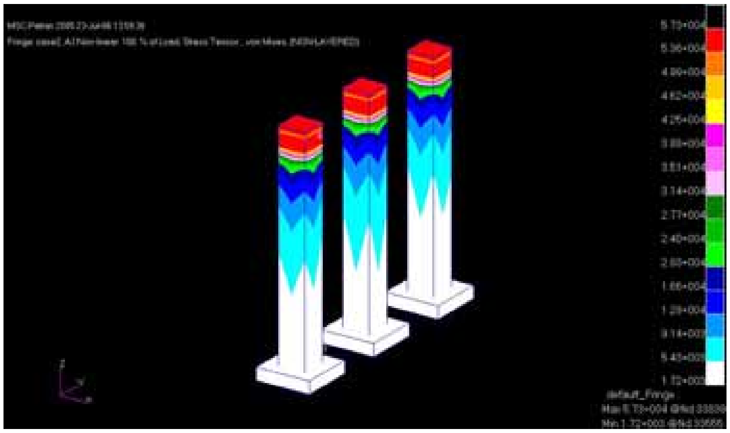
(c) Web reinforcement and stud model (Group 1) (d) Web reinforcement and stud model (Group 2)

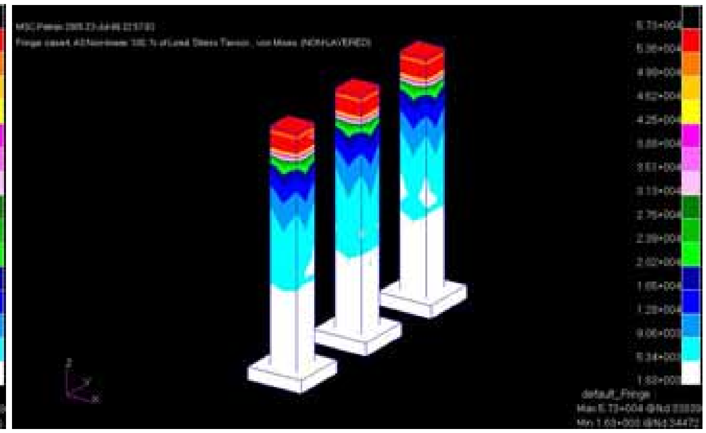




(e) 3-D view of web reinforcement model (Group 1) (f) 3-D view of web reinforcement model (Group 2)

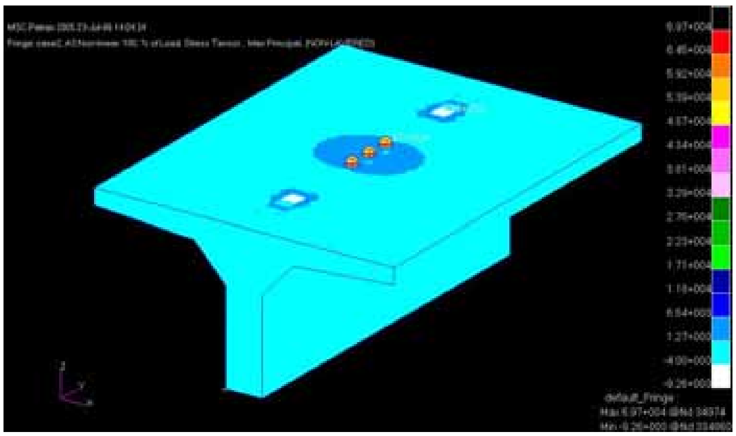
Figure F-1. Finite element model of the slab/concrete girder pullout specimen

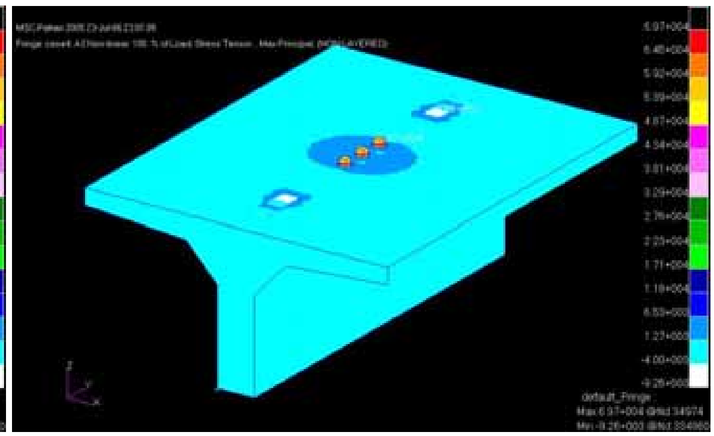


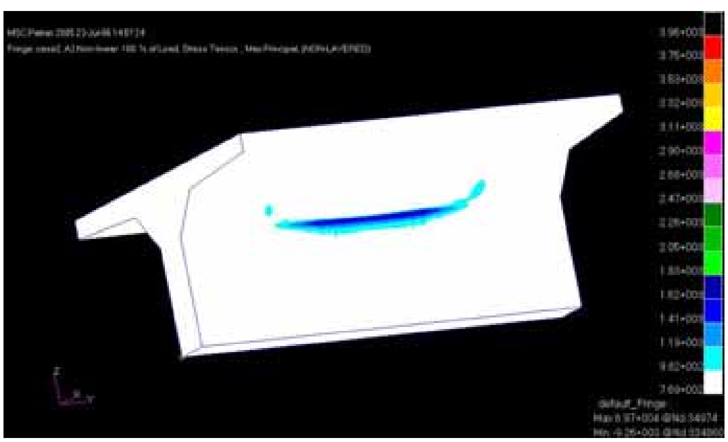


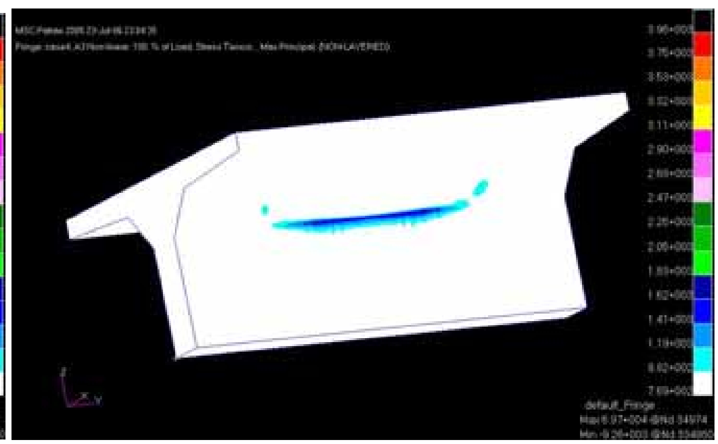
Z-stress of Group 1 Specimen

Z-stress of Group 2 Specimen









Principal stress of Group 1 Specimen

Principal stress of Group 2 Specimen

Figure F-2. Stress Distribution in the Studs and the Concrete Specimen

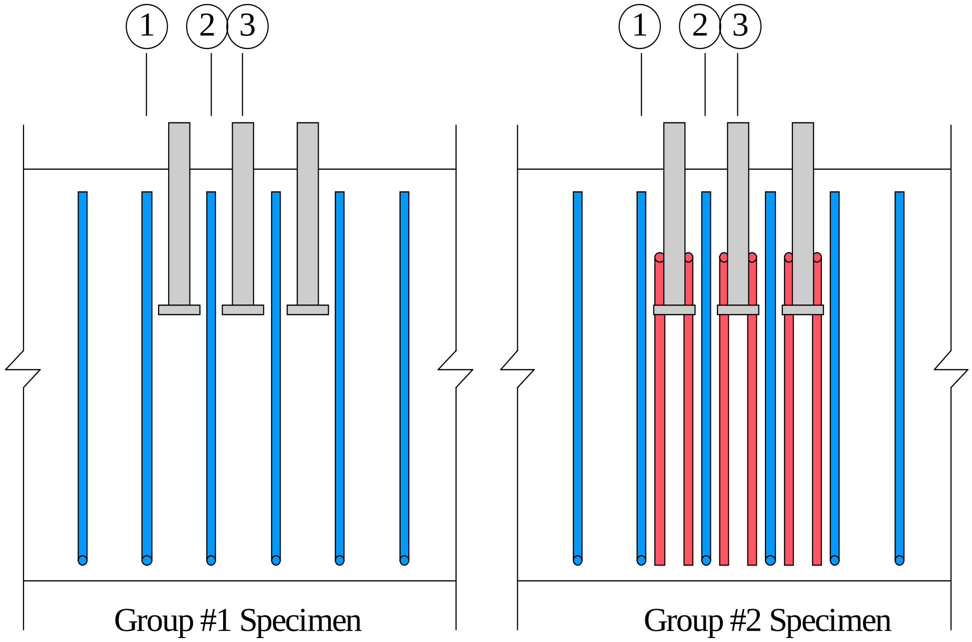
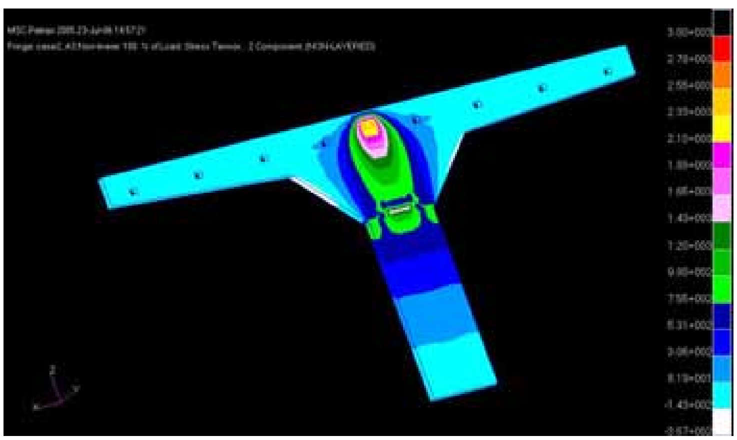
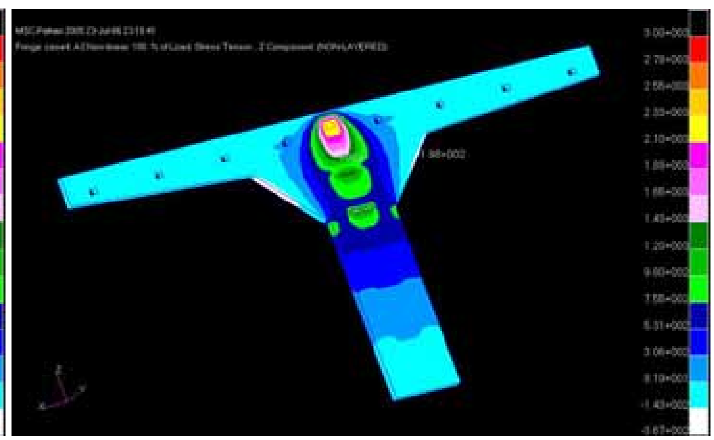


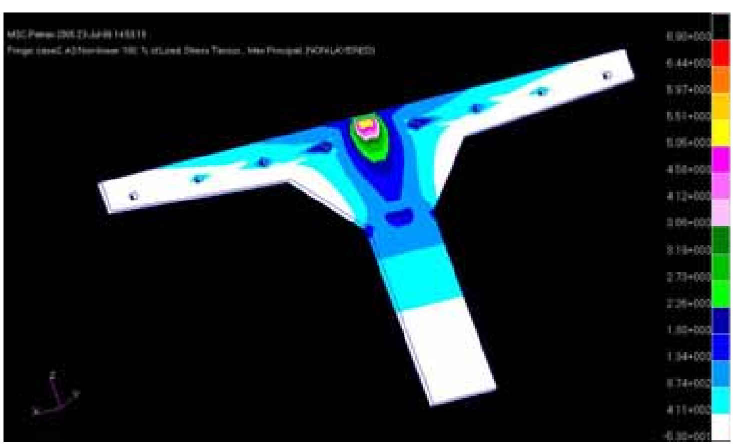
Figure F-3. Location of Sections 1, 2 and 3



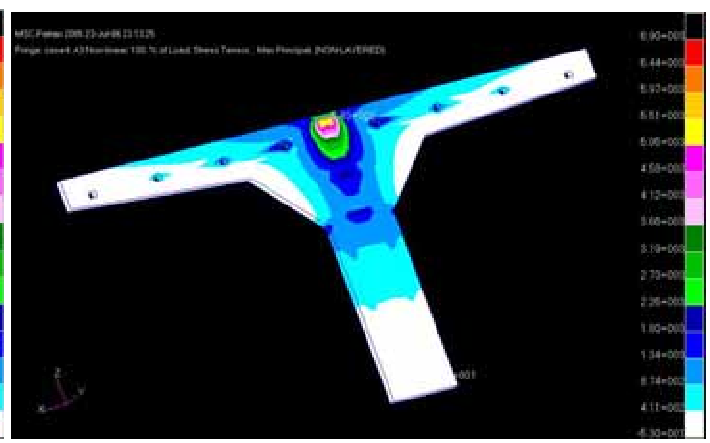
Z-stress of Group 1 Specimen



Z-stress of Group 2 Specimen

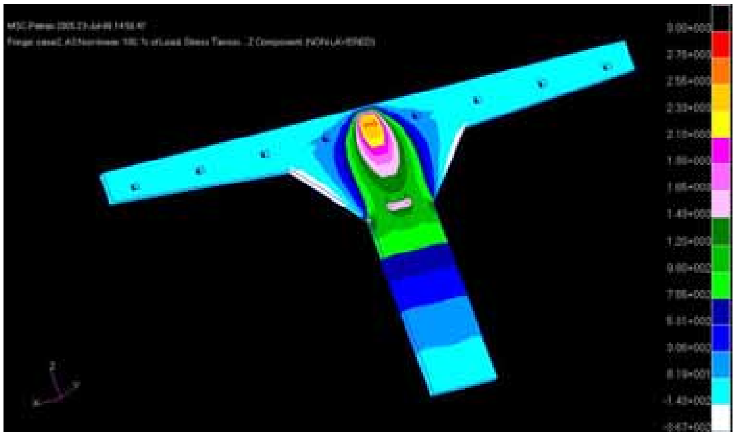


Principal stress of Group 1 Specimen

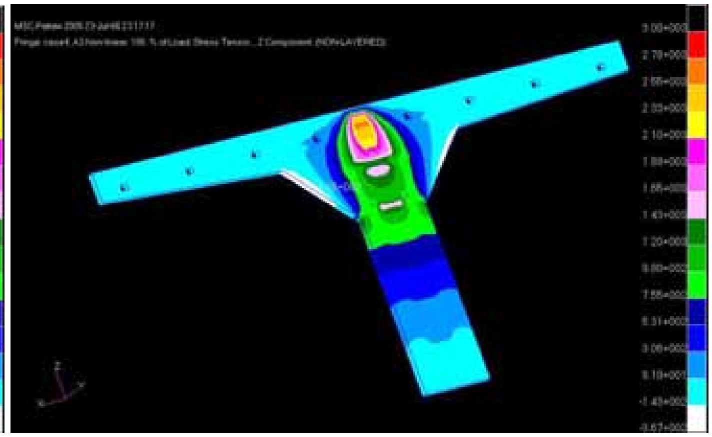


Principal stress of Group 2 Specimen

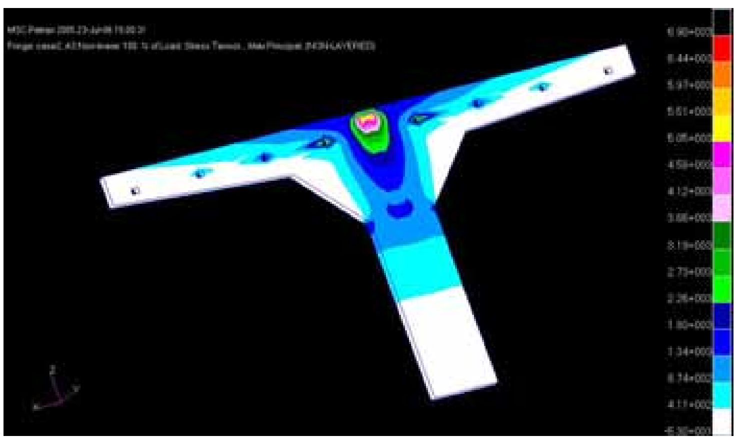
Figure F-4. Z-direction and Principal Stresses at Section 1



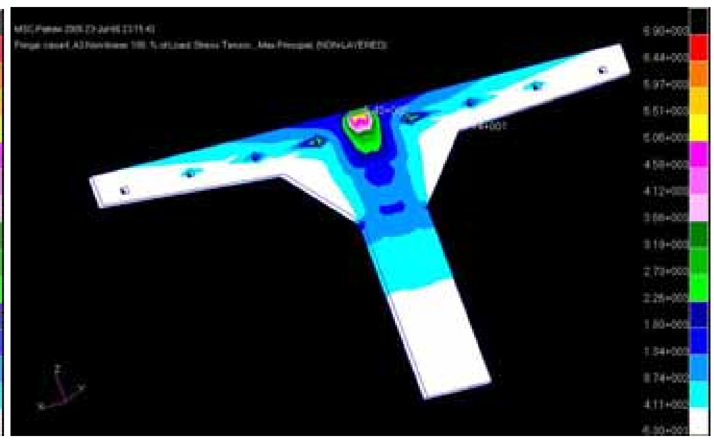
Z-Stress of Group 1 Specimen



Z-Stress of Group 2 Specimen

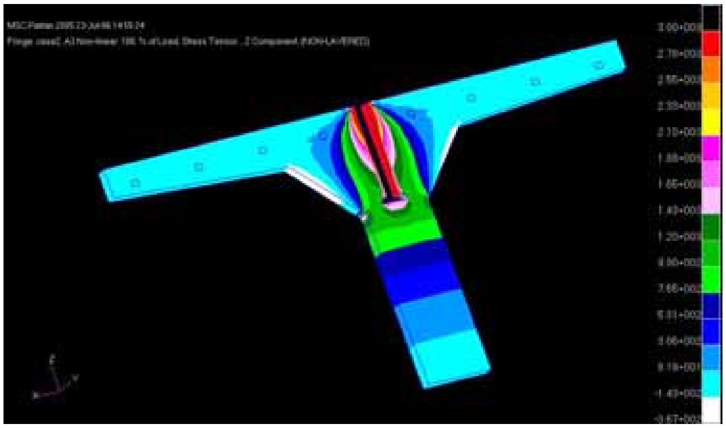


Principal stress of Group 1 Specimen

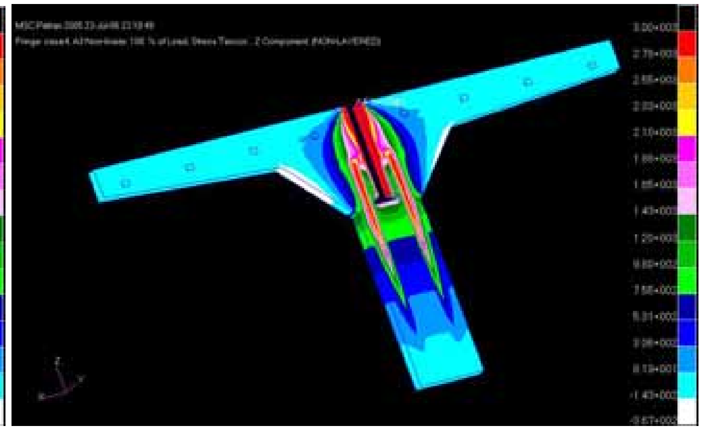


Principal stress of Group 2 Specimen

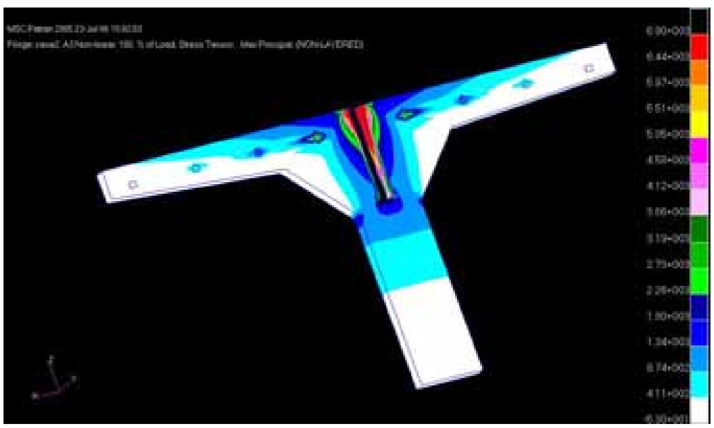
Figure F-5. Z-direction and Principal Stresses at Section 2



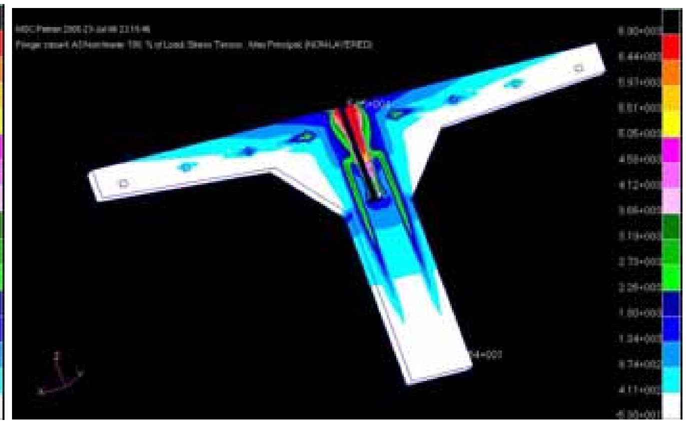
Z-stress of Group 1 Specimen



Z-stress of Group 2 Specimen



Principal stress of Group 1 Specimen



Principal stress of Group 2 Specimen

Figure F-6. Z-direction and Principal Stresses at Section 3

F.2 PANEL-TO-STEEL GIRDER CONNECTION

Finite Element Investigation of the Push-off Specimens

The finite element method was used to investigate the behavior of the push-off specimens. A commercial finite element package "NASTRAN" was used in the analysis. The push-off concrete specimen and the grout filling the shear pocket were modeled using a 8-node cube element. Each node has three translational degrees of freedom (x, y and z direction). The confining tube and the individual closed ties were modeled using the thin shell element. The circular cross section of the studs was replaced with a square cross section with equivalent area. The studs were modeled using the 20-node cube element.

The following mechanical properties were assigned to the concrete mix of the specimen: compressive strength = 6.2 ksi (42.7 MPa), unit weight = 150 pcf (23.6 kN/m^3), and Poisson ratio = 0.15. The following mechanical properties were assigned to the grout mix: compressive strength = 9.6 ksi (66.2 MPa), unit weight = 145 pcf (22.8 kN/m^3), and Poisson ratio = 0.15. The following mechanical properties were assigned to the stud: tensile strength = 64 ksi (441 MPa), yield strength = 54 ksi (372 MPa), unit weight = 490 pcf (76.9 kN/m^3), and Poisson ratio = 0.30.

Figure F-7 shows the details of the finite element model. In order to check the validity of Equation 6.10.10.4.3-1 of the AASHTO LRFD Specifications (*7*) for studs clustered in groups, each specimens was loaded with a horizontal equal to the ultimate horizontal shear capacity determined by this equation. The load was loaded as a surface loaded on a 10x10 in. (254x254 mm) area on the bearing block of the specimen to simulate the test setup. The resultant of the surface load was at mid height of the 8-in. (203 mm) thick slab. Figures F-8 to F-11 give the results of the FE analysis of various specimens, and Table F-2 gives a summary of the maximum stresses in the stud, grout and the confinement tool, where the following conclusions can be drawn:

1. The 4- and 8-stud specimens are not able to deliver the horizontal ultimate shear capacity as given by Equation 6.10.10.4.3-1 of the AASHTO LRFD Specifications (*7*). This can be seen from the average axial tensile stress at the stud base, which is higher than the ultimate tensile strength of the stud material, SAE 1018, 64 ksi (441.3 MPa).
2. The upper limit of Equation 6.10.10.4.3-1 of the AASHTO LRFD Specifications (*7*), $A_{sc}F_u$, does not recognize the fact that the stud close to failure is subjected to a combination of axial tensile and normal flexural stresses. This can be seen by checking the average principal tensile stress at the stud base, which is about 155 percent of the axial tensile stress, as shown in Table F-2. This means that the upper limit of Equation 6.10.10.4.3-1 overestimates the studs shear capacity. This finding was confirmed by the push-off test, where almost none of the specimens was able to reach the capacity determined by Equation 6.10.10.4.3-1 of the AASHTO LRFD Specifications (*7*).
3. Maximum bearing stress in the grout is located in front of each stud close to the stud base. It extends vertically for a distance approximately equal to the stud diameter.

The maximum bearing stress is about 30 ksi (206.9 MPa), which is about 310 percent of the compressive strength of unconfined grout mix, 9.6 ksi (66.2 MPa). However, if confinement is provided around the shear pocket, the compressive strength of the grout can be significantly increased, as follow:

Effective lateral confining pressure, f_l, = $\dfrac{\sum A_s f_{yh}}{sb_c}$ (2)

$$= \frac{\left(2\, sides\, x 1\, in.\, x \frac{5}{16}\, in.\right)\left(36\, ksi\right)}{\left(1\, in.\right)\left(12\, in.\right)} = 1.875 \text{ ksi} \quad \text{(for steel tube confinement)}$$

$$= \frac{\left(2\, legs\, x\, 0.44\, in^2\, per\, leg\, x\, 3\, bars\right)\left(60\, ksi\right)}{\left(1.75\, in.\right)\left(15\, in.\right)} = 6.034 \quad \text{(for closed ties confinement)}$$

Confined grout strength, $f_{c0} = f_0 + 4.1 k f_l$ (1)

= 9.6 + 4.1x1x1.875 = 17.3 ksi (119.3 MPa) (for steel tube confinement)

= 9.6 + 4.1x1x6.034 = 34.3 ksi (236.8 MPa) (for closed ties confinement)

4. The confinement around the stud group helps to distribute the bearing stresses of the grout volume on the concrete slab in front of the grout volume. The highest bearing stress is about 2.30 ksi (15.9 MPa) and the average bearing stress over the slab height is about 2.0 ksi (13.8 MPa).
5. The confinement provided by the steel tube helps to distribute the bearing stresses on a wider part of the slab resulting in reducing the compressive in the slab compared to the case where the closed ties are used.

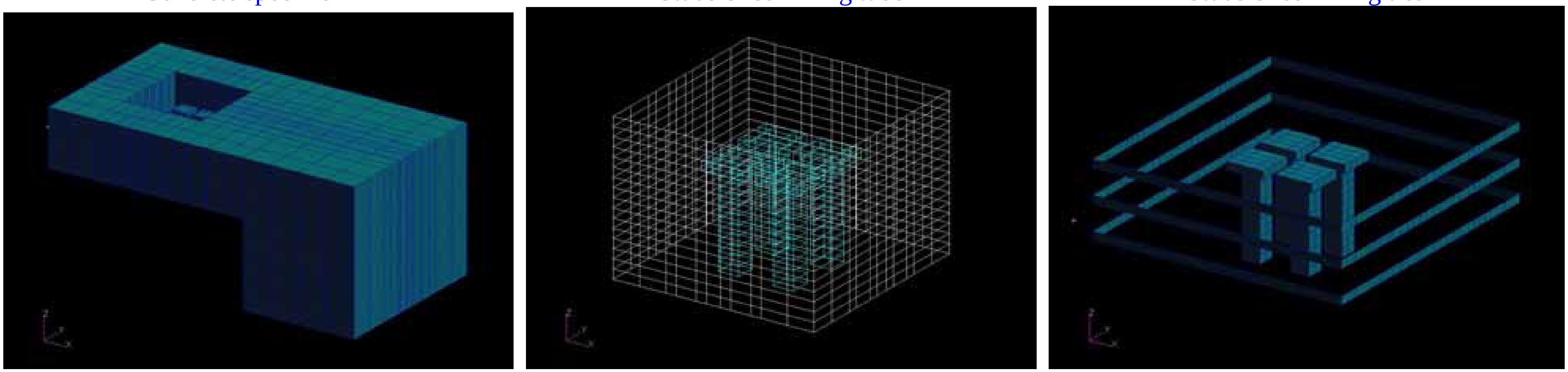
The truncated shape of the shear pocket and grout volume helps in distributing the bearing stresses more uniformly across the slab height.

Table F-2. Summary of the Finite Element Analysis Results for the Push-off Specimens

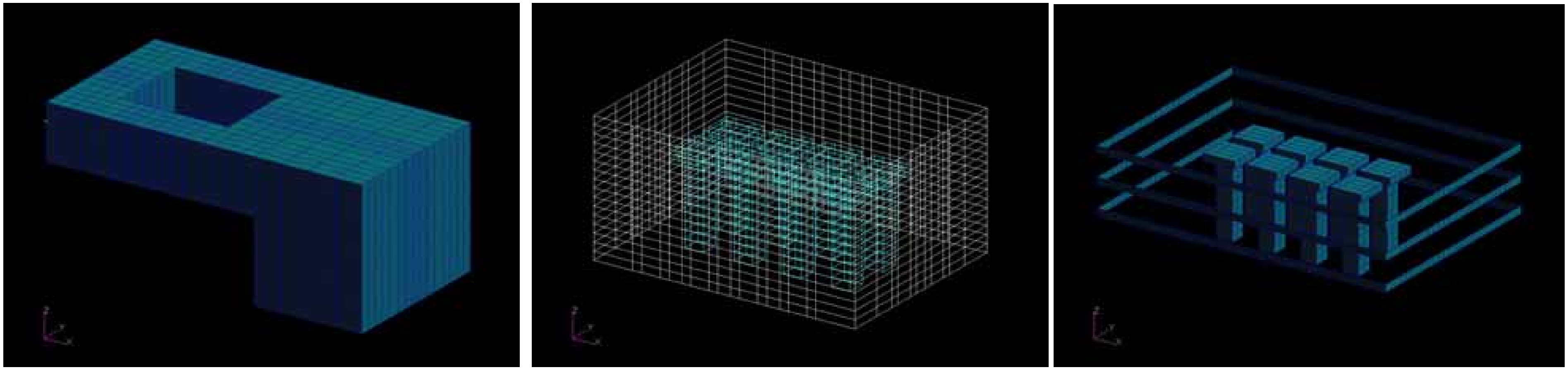
	4-stud specimens			8-stud specimens		
	P-4-ST-U (Steel tube)	P-4-CT-U (Closed ties)	Average	P-8-ST-U (Steel tube)	P-8-CT-U (Closed ties)	Average
Applied horizontal load (kips)*	314.8			629.6		
Maximum axial tensile stress at base of the stud (ksi)	58.4	99.1	78.8	99.9	74.9	87.4
Maximum tensile principal stress at base of the stud (ksi)	92.5	157.0	124.8	162.0	117.0	139.5
Maximum longitudinal movement of the stud head (in.)	0.0075	0.0103	0.0089	0.0109	0.00954	0.01022
Maximum axial tensile stress in confinement material in the transverse direction of the specimen (ksi)	21.0	3.7	NA	30.7	5.3	NA
Maximum bearing stress in grout in front of the stud (ksi)	29.1	31.8	30.5	27.1	31.6	29.4
Maximum bearing stress in the concrete in front of the grout volume (ksi)	2.31	2.31	2.31	2.30	2.30	2.30

* Determined using Equation 6.10.10.4.3-1 of the AASHTO REFD Specifications (*7*)

Concrete specimen | Studs & confining tube | Studs & confining ties

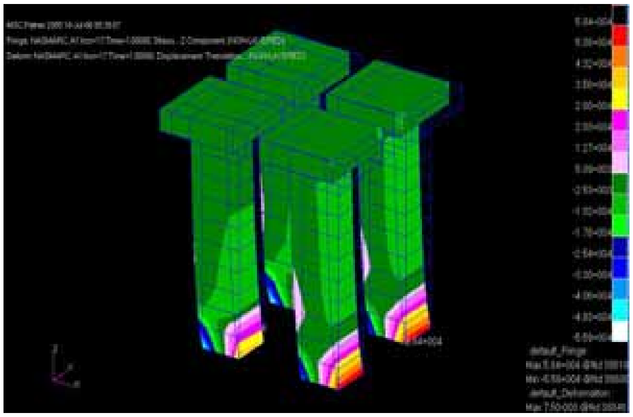


(a) Finite element modeling of P-4-ST-U and P-4-CT-U specimens

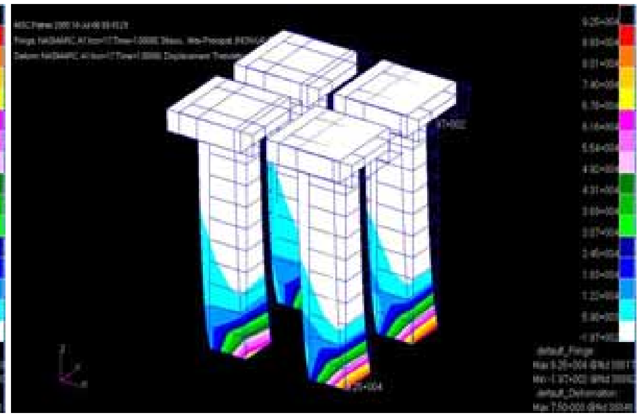


(b) Finite element modeling of P-8-ST-U and P-8-CT-U specimens

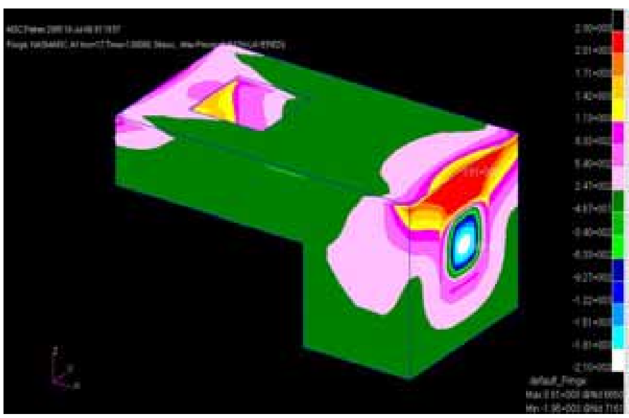
Figure F-7. Finite Element Model of the Push-off Specimens



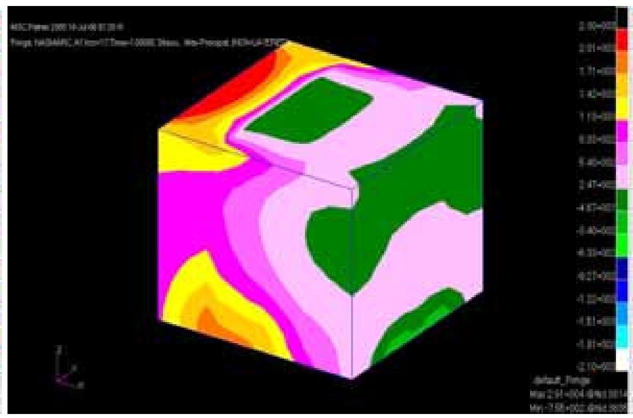
Axial Tensile Stresses in the Studs



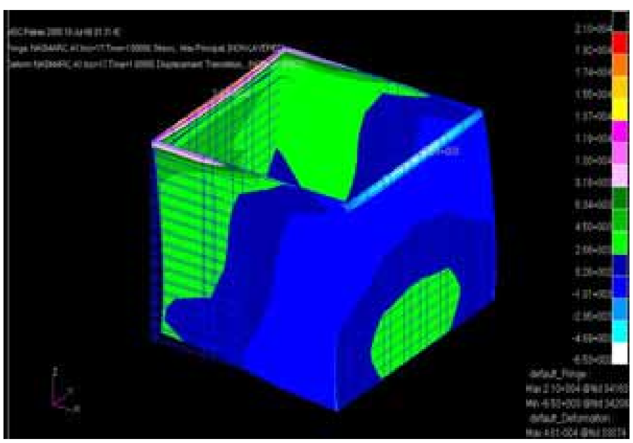
Principal Tensile Stresses in the Studs



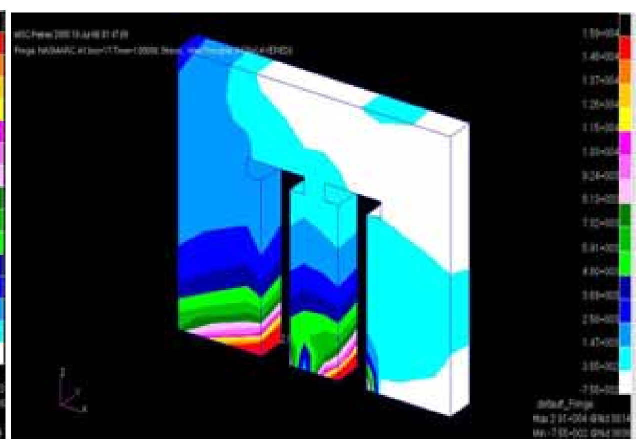
Longitudinal Compressive Stresses in Concrete



Longitudinal Compressive Stresses in Grout

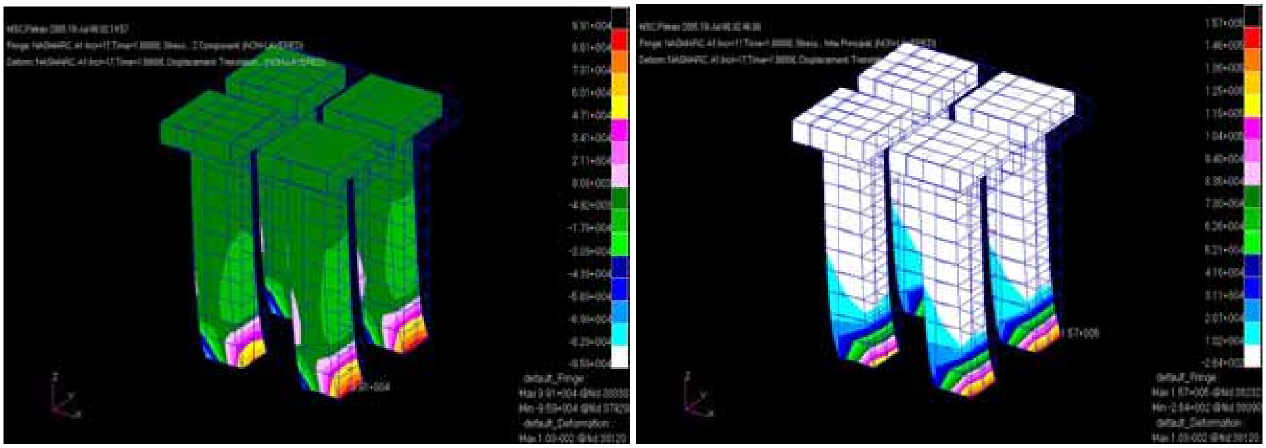


Transverse Axial Tensile Stress in Steel Tube

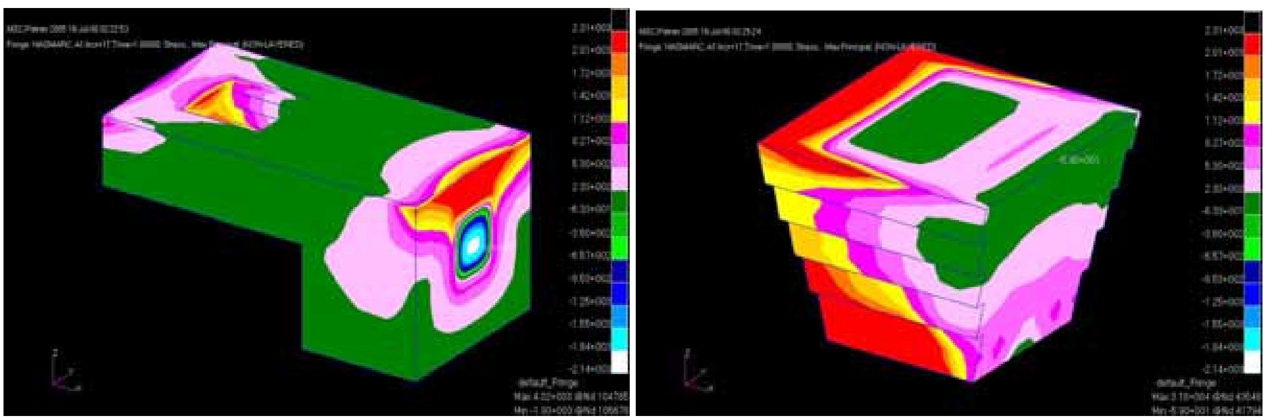


Principal Stresses in Grout in front the Studs

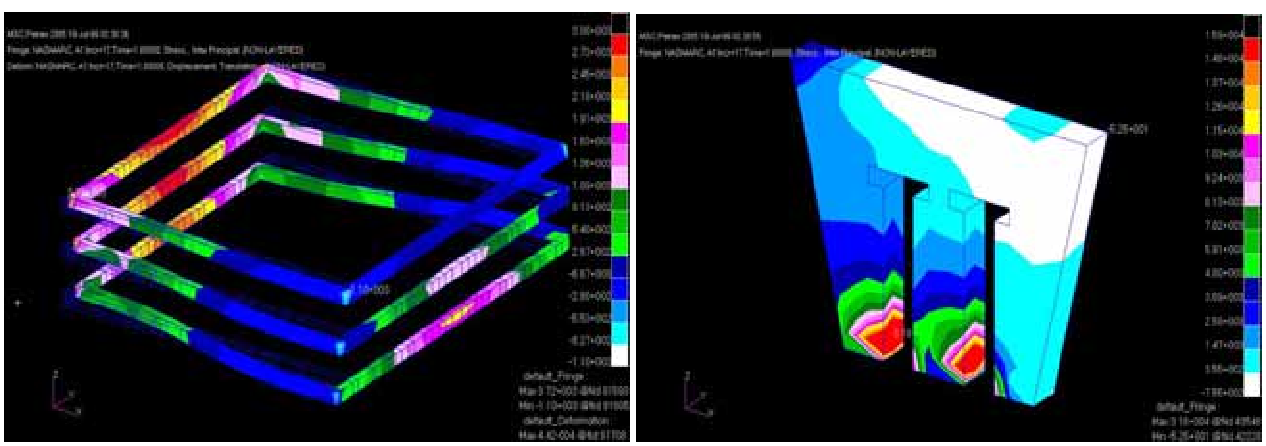
Figure F-8. Stresses in Specimen P-4-ST-U due to LRFD Load (314.8 kips)



Axial Tensile Stresses in the Studs | Principal Tensile Stresses in the Studs

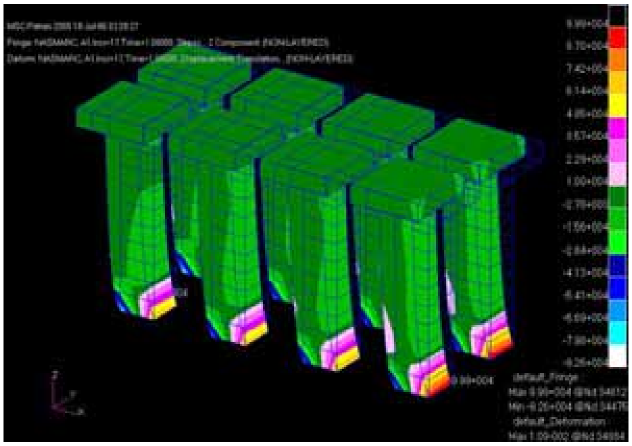


Longitudinal Compressive Stresses in Concrete | Longitudinal Compressive Stresses in Grout

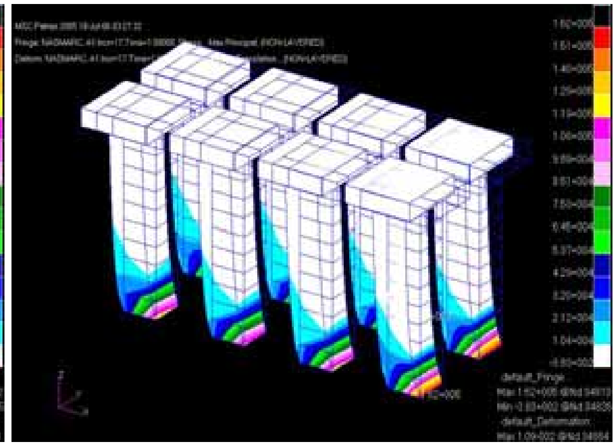


Transverse Axial Tensile Stress in Steel Tube | Principal Stresses in Grout in front the Studs

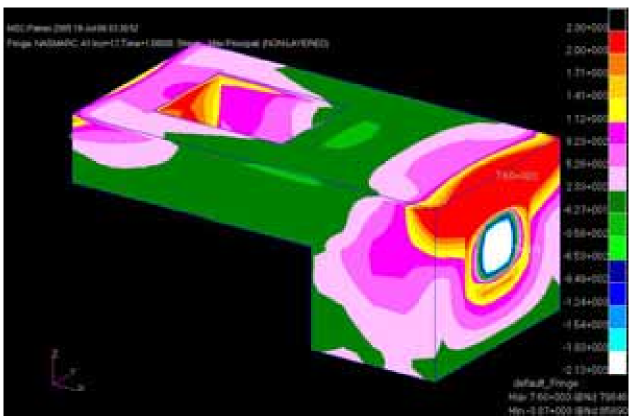
Figure F-9. Stresses in Specimen P-4-CT-U due to LRFD Load (314.8 kips)



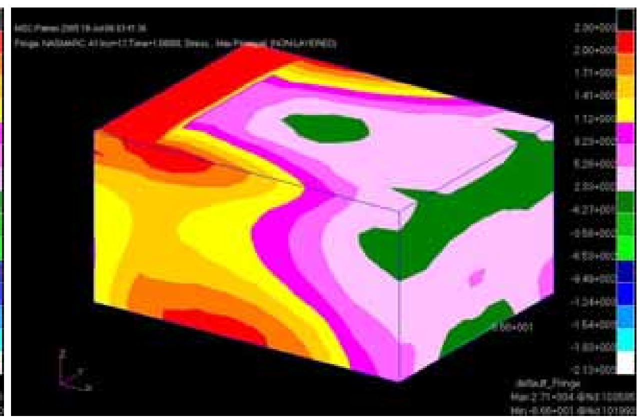
Axial Tensile Stresses in the Studs



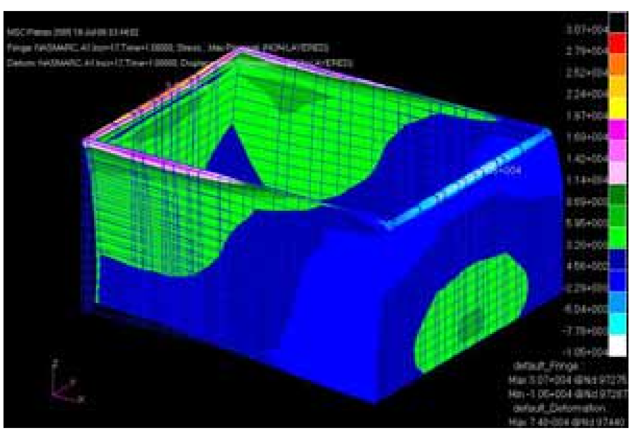
Principal Tensile Stresses in the Studs



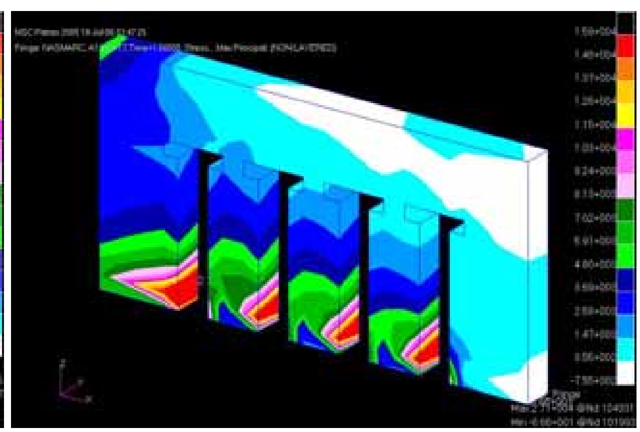
Longitudinal Compressive Stresses in Concrete



Longitudinal Compressive Stresses in Grout

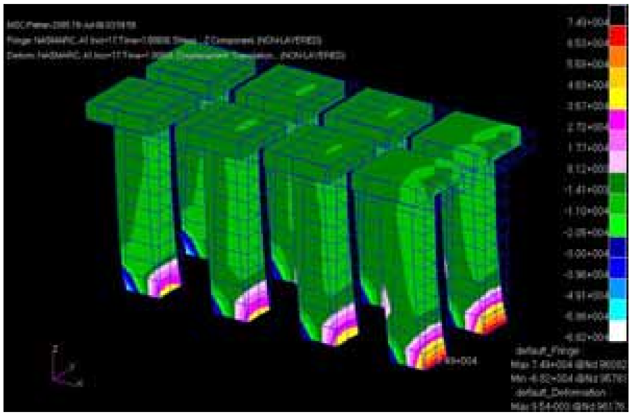


Transverse Axial Tensile Stress in Steel Tube

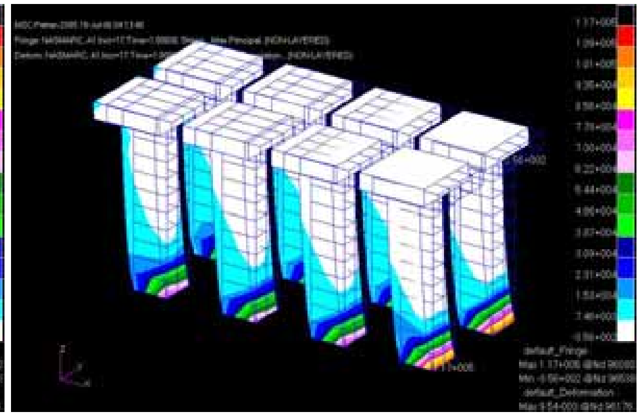


Principal Stresses in Grout in front the Studs

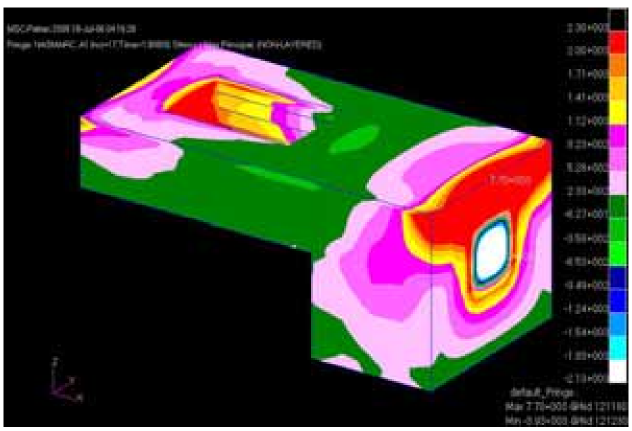
Figure F-10. Stresses in Specimen P-8-ST-U due to LRFD Load (629.6 kips)



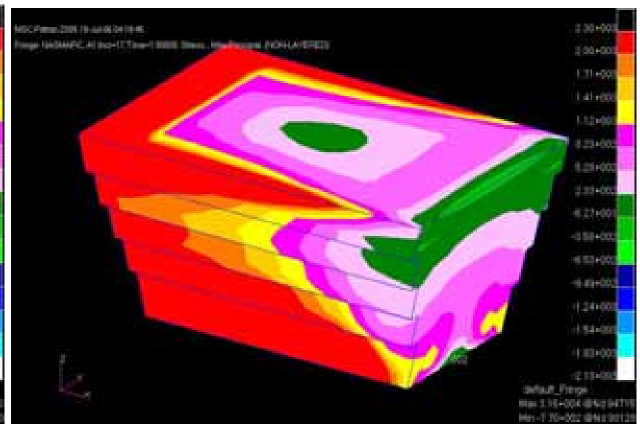
Axial Tensile Stresses in the Studs



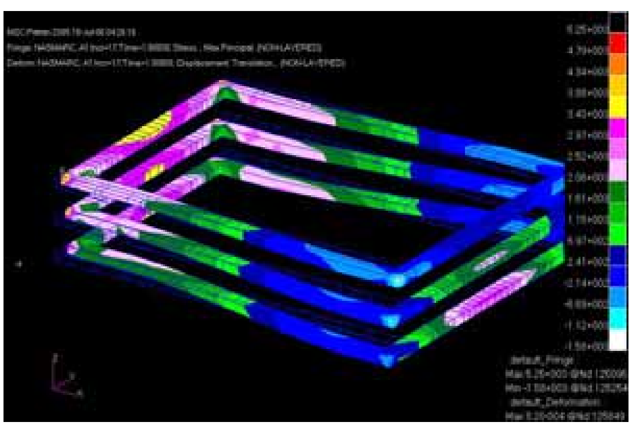
Principal Tensile Stresses in the Studs



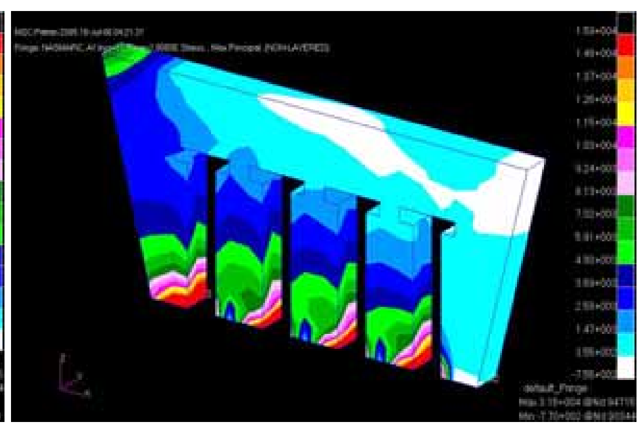
Longitudinal Compressive Stresses in Concrete



Longitudinal Compressive Stresses in Grout



Transverse Axial Tensile Stress in Steel Tube



Principal Stresses in Grout in front the Studs

Figure F-11. Stresses in Specimen P-8-CT-U due to LRFD Load (629.6 kips)