Precast Element Evaluation for the US 90 Bridges over Little River and Hurricane Creek

Contract Number BDV30-307-01
FSU Project ID 032819

Submitted to:

Florida Department of Transportation

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FAMU-FSU Engineering

July 2019
FINAL REPORT

DISCLAIMER

SI* (MODERN METRIC) CONVERSION FACTORS

*APPROXIMATE CONVERSIONS **TO** SI UNITS*

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
LENGTH				
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
AREA				
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
VOLUME				
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
NOTE: volumes greater than 1000 L shall be shown in m^3				
MASS				
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams	Mg (or "t")
TEMPERATURE (exact degrees)				
°F	Fahrenheit	5(F-32)/9 or (F-32)/1.8	Celsius	°C
ILLUMINATION				
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
FORCE and PRESSURE or STRESS				
kip	1000 pound force	4.45	kilonewtons	kN
lbf	pound force	4.45	newtons	N
lbf/in^2	pound force per square	6.89	kilopascals	kPa

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.

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*APPROXIMATE CONVERSIONS **FROM** SI UNITS*

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
°C	Celsius	1.8C+32	Fahrenheit	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
kN	kilonewtons	0.225	1000 pound force	kip
N	newtons	0.225	pound force	lbf
kPa	kilopascals	0.145	pound force per square inch	lbf/in^2

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16. Abstract

The Florida Department of Transportation (FDOT) designed a new bridge system that includes these precast concrete elements: intermediate bent caps, prestressed girders (45" Florida-I beams), non-prestressed deck panels, and prestressed deck panels. A bridge replacement project on US Highway 90 (SR 10) was chosen to implement the new system. Four (4) bridges were replaced with 110-ft typical spans.

The research involved monitoring and documenting the precast activities, precast member installation, construction of joints and connections, and construction schedule. During construction, the main tasks were to document construction activities and schedule, the contractors' fabrication and construction methods, and the grouting demonstration test – as well as inspect haunches and take core samples of the pockets and joints. There was a significant learning curve during construction. Grouting the precast panel connections to the girders was challenging, particularly with forming and creating a good seal. Fabrication and construction of the precast bent caps went relatively smoothly, except for misalignment of some dowel bars that required modification to the reinforcing steel in the cap. After construction, the bridge deck panels were mapped for cracks every three months for two years. The non-prestressed panels exhibited significantly more cracking than the prestressed panels. The cracks formed almost exclusively in the longitudinal direction, parallel to traffic – i.e., transverse to the panel; many of them were along the girder line where the shear pockets were located in the panels.

The bridges were load tested before they were placed in service, and again after being in service for 19-25 months. The girder and panel response remained linear with increasing load. Deflections and strains were mostly consistent and repeatable from one bridge to another. The non-prestressed panel bridges responded similarly to the prestressed panel bridges. Calculated distribution factors showed that AASHTO equations are conservative. Composite action was achieved between the girders, deck panels, and joints. The transverse strain data for the panels was a little erratic. The bent caps seemed to perform well, although the strains were unequal on the two faces. The bent caps likely behaved as a cracked section. Overall, the bent caps proved to be a viable option for use in Florida bridges.

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Thanks also to FDOT for setting up maintenance of traffic and keeping an eye on us while we mapped cracks. Bill Duncan, thank you for always checking on us and making sure everything went well. Matt Goines and Huie Brown, thank you for your company and bringing the snooper so we could inspect the haunches and also instrument for the nine load tests.

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The authors would also like to thank the many students who helped with crack mapping. Whether it was hot, cold, sunny, windy, foggy, humid, or rainy, they came to the bridge site with a great attitude and had fun.

EXECUTIVE SUMMARY

This research was motivated by the Federal Highway Administration's (FHWA) Innovative Bridge Research and Deployment (IBRD) program, which was established to promote innovative accelerated bridge design and construction technologies. The Florida Department of Transportation (FDOT) designed a new bridge system that includes the following precast concrete elements: intermediate bent caps, prestressed girders (45" Florida-I beams), non-prestressed deck panels, and prestressed deck panels. Precast girders have been widely used for bridge projects in the state; however, precast bent caps and deck panels have not. A bridge replacement project on US Highway 90 (SR 10) was chosen to implement the new system. Four (4) bridges were replaced with 110-ft typical spans.

Because the precast components were newly developed, the research involved monitoring and documenting the precast activities, precast member installation, construction of joints and connections, and construction schedule.

During construction, the main tasks were to document: construction activities and schedule; the contractors' fabrication and construction methods; and the grouting demonstration test. This included inspecting haunches and taking core samples of the pockets and joints. There was a significant learning curve during construction. Grouting the precast panel connections to the girders was challenging, particularly with forming and creating a good seal to prevent leakage during grouting operations. Fabrication and construction of the precast bent caps went relatively smoothly, except for misalignment of some dowel bars that required modification to the reinforcing steel in the cap.

After construction, the bridge deck panels were mapped for cracks every 3 months for 2 years. The non-prestressed panels exhibited significantly more cracking than the prestressed panels. The cracks formed almost exclusively in the longitudinal direction, parallel to traffic – i.e., transverse to the panel; many of them were along the girder line where the shear pockets were located in the panels.

The bridges were load tested before they were placed in service, and again after being in service for 19-25 months. The girder and panel response remained linear with increasing load. Deflections and strains were mostly consistent and repeatable from one bridge to another. The response of the bridges with non-prestressed panels was similar to those with prestressed panels. Calculated distribution factors from strain and deflection data showed that AASHTO equations are conservative. Composite action was achieved between the girders, deck panels, and joints. The transverse strain data for the panels was a little erratic. The bent caps seemed to perform well, although the strains were unequal on the two faces, and one of the bent caps had higher strains than the other. The bent caps likely behaved as a cracked section. Overall, the bent caps proved to be a viable option for use in Florida bridges.

Table of Contents

** Reports 1-5 listed above contains a Table of Contents, List of Figures, List of Tables, and Appendices as applicable. Chapter and page numbers begin anew with each report.*

List of Figures *

List of Tables *

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BACKGROUND

The Federal Highway Administration's (FHWA) Innovative Bridge Research and Deployment (IBRD) program was established to promote innovative accelerated bridge design and construction technologies. Activities that are supported by IBRD include demonstration, evaluation, and documentation of the application of innovative designs, materials, and construction methods – as applied to the construction, repair, and rehabilitation of bridges and other highway structures.

FHWA's Every Day Counts (EDC) program identifies and deploys innovation that shortens project delivery, enhances the safety of our roadways, and protects the environment. EDC includes Accelerated Bridge Construction (ABC), one facet of which is the use of Prefabricated Bridge Elements and Systems (PBES). The goals of PBES are to improve safety on the construction site, reduce onsite construction time, improve constructability, decrease mobility impact times, and improve quality. For example, when precast elements are used instead of cast-in-place elements, time-consuming tasks and concrete curing are done away from the work zone – with the benefit of improving construction zone safety and minimizing the amount and duration of traffic impacts. In addition, the elements are made under controlled conditions, which can improve quality. However, building a bridge with precast elements usually requires special joint connections that are made on site and in situ; these joints can leak or require extra maintenance. In addition, if connections between elements are compromised, then distribution and sharing of loads among the structural members can be diminished. Another drawback is unfamiliarity with design and construction aspects on the part of contractors, precasters, and engineers.

The IBRD and ABC programs motivated the Florida Department of Transportation (FDOT) to design a new bridge system that could be built, and possibly standardized, using PBES. During the design phase, a mock-up and testing program were conducted at the FDOT Marcus H. Ansley Structures Research Center (SRC) in Tallahassee, Florida, to improve constructability and refine the project specifications.

The US 90 (SR 10) bridge replacement project, located east of Quincy in Gadsden County, Florida, was a good opportunity to implement this system. (See **Figure 1** for a state map showing the project location.) It was thus made a pilot project for evaluation by FDOT – bid by contractors under the normal letting process for a bridge construction project. FDOT's prototype was designed to replace all four (4) existing bridges: Westbound over Little River, Eastbound over Little River, Westbound over Hurricane Creek, and Eastbound over Hurricane Creek (**Figures 2 and 3** are photos of the existing bridges). Each existing and replacement bridge carries two unidirectional lanes of moderate traffic volume in a rural area. See **Figure 4** for a plan view of the replacement bridges and connecting roadway and **Figure 5** for a rendering.

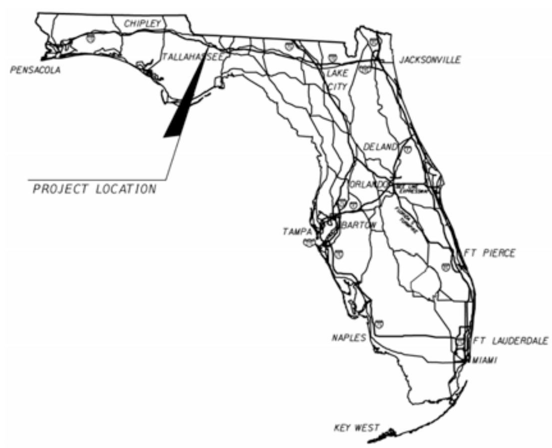

Fig. 1. Location of US 90 Bridge Replacement Project in Florida

Fig. 2. Existing Little River Bridge Before Construction: Northwest Corner, Looking East

Fig. 3. Existing Hurricane Creek Westbound Box Culvert, with Eastbound Box Culvert Demolished

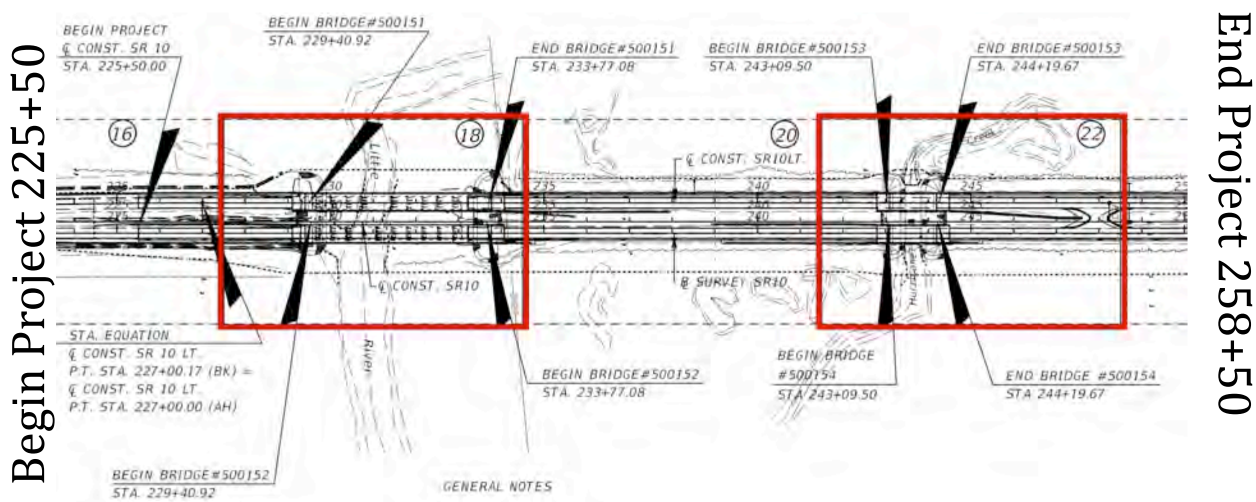

Fig. 4. Plan View of Four Replacement Bridges and Connecting Roadway (from Design Plans)

The design includes these precast concrete elements: intermediate bent caps, prestressed girders (45" Florida-I beams), non-prestressed deck panels, and prestressed deck panels. Precast girders have been widely used for bridge projects in the state; however, precast bent caps and deck panels have not. Early in the design process, FDOT searched literature on precast deck systems that have been investigated or used by others. This included articles by Markowski et al. (2005), Menkulasi and Roberts-Wollmann (2005), Kassner et al. (2007), FHWA (2011), and Precast/Prestressed Concrete Institute Northeast (PCINE 2011). Additionally, the ABC standard plans from the Strategic Highway Research Program 2 (SHRP2 2014) were adapted for the precast bent cap design, except that the drilled shaft-to-cap connections were based on the grouted corrugated metal ducts described in NCHRP Report 681 (NCHRP 2010).

FDOT designed special details for the elements, joint connections, and forming. Also, because FDOT may support or promote the use of these new bridge elements in future projects, they developed Specification Section 404 entitled, "Precast Concrete Elements for Bridge Construction", and included them in the US 90 project specifications package.

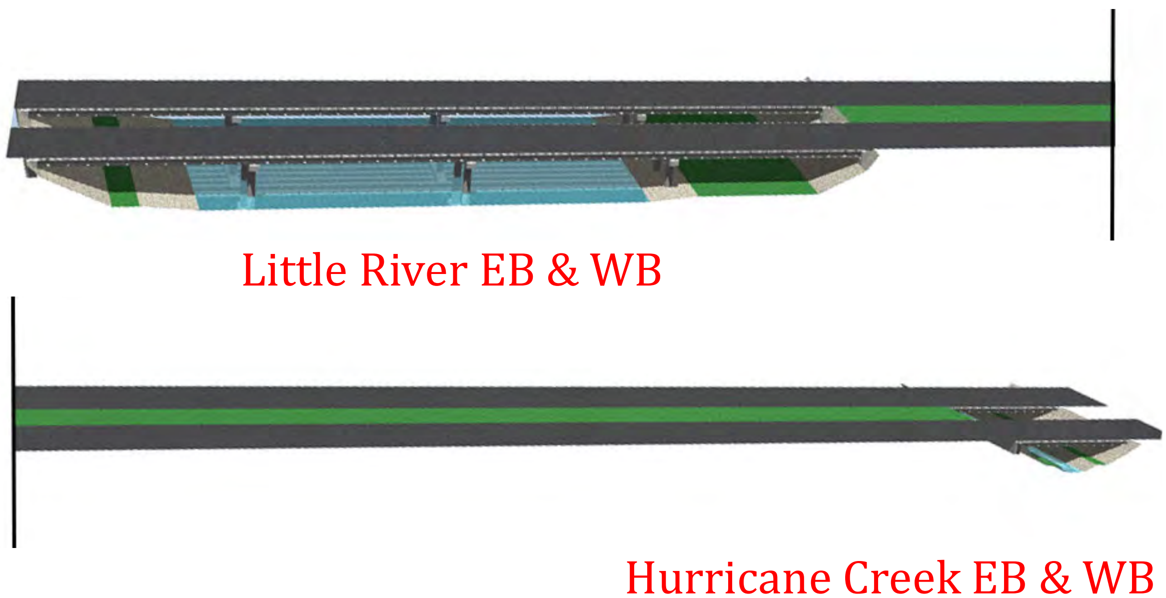

Fig. 5. Rendering of Four Replacement Bridges and Connecting Roadway

Table 1 shows which bridges implement each precast element.

Table 1. Precast Elements Used in the Four Replacement Bridges

Precast Element	Little River WB	Little River EB	Hurricane Creek WB	Hurricane Creek EB
Precast Bent Caps (Intermediate)	✓	✓		
Precast/Prestressed Florida-I Beams, 45"	✓	✓	✓	✓
Non-prestressed Precast Deck Panels, 8.5"		✓		✓
Prestressed Precast Deck Panels, 8.5" & 10"	✓		✓	

RESEARCH PROJECT

Objectives

This research project commenced when the bridge construction project began in early 2013, and it occurred simultaneously with and parallel to the construction. Because the precast components were newly developed, it was prudent to monitor and document the precast activities, especially the fabrication of precast elements, construction of joints and connections, and construction schedule.

The objectives of the research were as follows: to show evidence of the benefits of using the new precast components; to verify that FDOT's developmental specifications will ensure quality control for future projects; and to help improve the specifications for future use. In addition, the deliverables from the research were prepared for technology transfer purposes, for FHWA and for states that wish to consider implementing the precast components.

Tasks

The main tasks *during* construction were as follows: to document the construction activities and schedule; to document the methods used by the bridge contractor to fabricate and construct the precast elements; to document the quality of the constructed bridge; to document the contractor's grouting demonstration test; to inspect haunches; to take core samples of the pockets and joints; and to measure the performance of the in-service bridge by inspection monitoring and load testing. The main tasks *after* construction were to inspect the bridge deck in intervals after being placed in service to show whether cracks are occurring or growing. In addition, the bridges were load tested before they were placed in service, and again after being in service for 19-25 months, to look for signs of unexpected performance or degradation.

Purpose

The results of the research – with regard to construction, quality of the end product, and behavior of the in-service bridges – will help FDOT to make an informed decision about implementing and promoting the use of these new bridge elements in future projects. It will also help FDOT to build upon their Specification Section 404. Having drawings and specifications in place would help make the technology market-ready for future use. The ultimate goals are to improve the state's transportation infrastructure, shorten project delivery, reduce bridge construction cost, and ensure good quality and durability of the constructed bridge.

LIST OF RESEARCH TEST REPORTS

This research focused on the precast elements listed in Table 1. The research results are presented in the following reports:

Report 1: INTROUCTION TO CONSTRUCTION PROJECT

Report 2: FABRICATION OF PRECAST ELEMENTS

Report 3: CONSTRUCTION OF PRECAST ELEMENTS

Report 4: MONITORING OF CRACKS IN PRECAST PANELS

Report 5: LOAD TEST REPORT

Each report was submitted, as the work was completed, for FDOT review. Throughout the work, the researchers tried to be objective and to observe from many vantage points – considering the designer, precaster, contractor, inspector/maintainer, and end user.

REFERENCES

(1) Markowski, S. M, Ehmke, F. G., Oliva, M. G., Carter III, J. W., Bank, L. C., Russell, J. S., Woods, S. and Becker, R. (2005). "Full-depth, Precast, Prestressed Bridge Deck Panel System for Bridge Construction in Wisconsin." *Proceedings of the PCI National Bridge Conference*, Palm Springs, CA, October 16-19.

(2) Menkulasi, F. and Roberts-Wollman, C. L. (2005). "Behavior of Horizontal Shear Connections for Full-Depth Precast Concrete Bridge Decks on Prestressed I-Girders." *PCI Journal*, 50(3), 60-73.

(3) Kassner, B. L., Brown, M. C., and Schokker, A. J. (2007). "Material Investigation of the Full-Depth, Precast Concrete Deck Panels of the Old Woodrow Wilson Bridge." *FHWA/VTRC Report 08-R2*, Virginia Transportation Research Council, Charlottesville, VA.

(4) FHWA (2011). "Accelerated Bridge Construction – Experience in Design, Fabrication and Erection of Prefabricated Bridge Elements and Systems." *Publication No. HIF-12-013*, Highways for Life, Federal Highway Administration, Washington, D.C.

(5) PCINE (2011). "Full Depth Deck Panels Guidelines For Accelerated Bridge Deck Replacement Or Construction - Second Edition." *Report No. PCINER-11*, Precast/ Prestressed Concrete Institute (PCI) Northeast, Belmont, MA.

(6) SHRP 2 (2014). "ABC Standard Plans." *Report S2-R04-RR-1*, Transportation Research Board, Washington, D.C., 375-377.

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March 2016

REPORT 1: INTRODUCTION TO CONSTRUCTION PROJECT

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REPORT 1: INTRODUCTION TO CONSTRUCTION PROJECT

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CHAPTER 1

BID INFORMATION AND CONTRACTORS FOR BRIDGE CONSTRUCTION PROJECT

The US 90 bridge construction project was let on 5/22/2013 under FDOT Contract No. T3417. Bids were received from six (6) contractors, as shown in **Table 1-1**; the bids included roadway and structures items. With the lowest bid, Anderson Columbia Co., Inc., was awarded the contract. With the exception of the highest bid, all of the bids were within 8.8% of the lowest.

Table 1-1. US 90 Bridge Construction Bids from 6 Contractors

Contractor	Bid Total
Anderson Columbia Co., Inc.	$9,576,951.91
GLF Construction Corp.	$9,670,927.11
Sema Construction, Inc.	$10,161,111.00
Superior Construction Co. Southeast, Inc.	$10,294,502.23
Scott Bridge Co., Inc.	$10,420,342.13
Orion Marine Construction, Inc.	$12,622,164.98

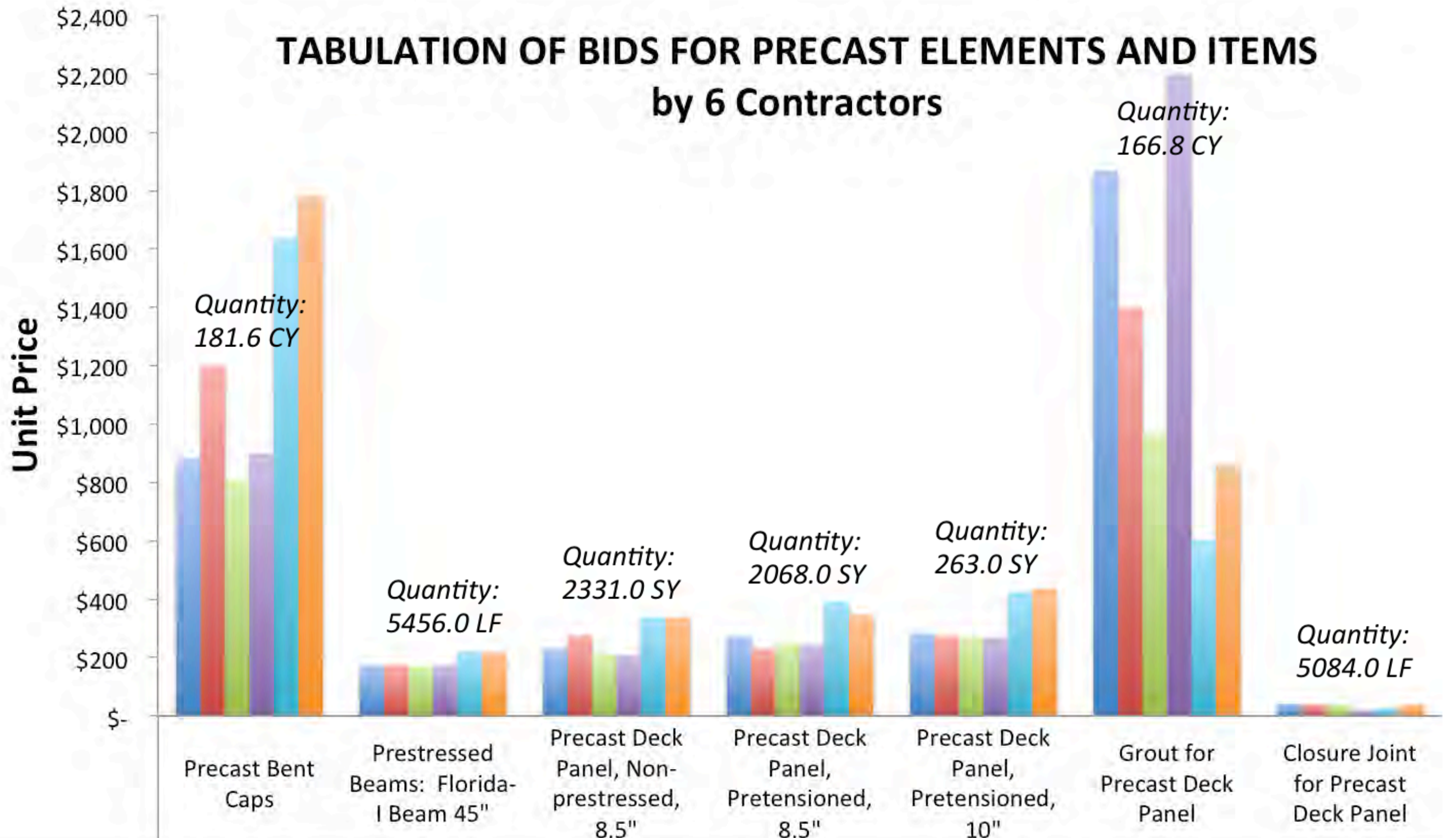
Each precast element was a separate item in the construction bid documents. Other bid items related to the Precast Deck Panels included Grout (to fill the shear pockets, lifting loops, and leveling bolts) and Closure Joints. **Figure 1-1** shows the six (6) contractors' bids – for the total project and also for the precast elements and items portion. The contractors' unit-price bids for the precast elements and items, as well as the quantities for each, are provided in **Figure 1-2**.

Although Anderson Columbia's bid for most of the items was among the lowest, it is interesting that their bid for Grout was one of the highest. Anderson Columbia's bid amounts for each precast element and item are shown in **Figure 1-3**, along with each item's cost as a percentage of all precast element costs.

The selected contractor chose CDS Manufacturing, Inc. (CDS) to fabricate all precast elements. CDS is located in Gretna, Florida, about 10 miles west of the bridge site. They specialize in precast and prestressed concrete members such as piles, beams, and hollow-core slabs for residential, commercial, and bridge applications. CDS already had forms for the standard Florida-I Beams, but they designed and constructed forms for the other members.

RS&H provided Construction Engineering and Inspection (CEI) services to FDOT, and Jacobs Engineering provided contract support.

Fig, 1-1. US 90 Bridge Construction Project Bids from 6 Contractors, Total Project and Precast Elements and Items



	Precast Bent Caps	Prestressed Beams: Florida-I Beam 45"	Precast Deck Panel, Non-prestressed, 8.5"	Precast Deck Panel, Pretensioned, 8.5"	Precast Deck Panel, Pretensioned, 10"	Grout for Precast Deck Panel	Closure Joint for Precast Deck Panel
Anderson Columbia Co., Inc.	$881.65	$173.81	$231.42	$271.35	$283.96	$1,868.26	$42.32
GLF Construction Corp.	$1,200.00	$175.00	$275.00	$230.00	$275.00	$1,400.00	$41.00
Sema Construction, Inc.	$810.00	$170.00	$210.00	$250.00	$270.00	$970.00	$38.00
Superior Construction Co. Southeast, Inc.	$900.00	$175.00	$208.00	$244.00	$270.00	$2,200.00	$20.00
Scott Bridge Co., Inc.	$1,638.73	$220.49	$339.91	$392.16	$425.42	$603.71	$27.11
Orion Marine Construction, Inc.	$1,781.00	$218.00	$339.00	$347.00	$437.00	$859.00	$39.00

Fig. 1-2. Bid Unit Prices and Quantities of Precast Elements and Items

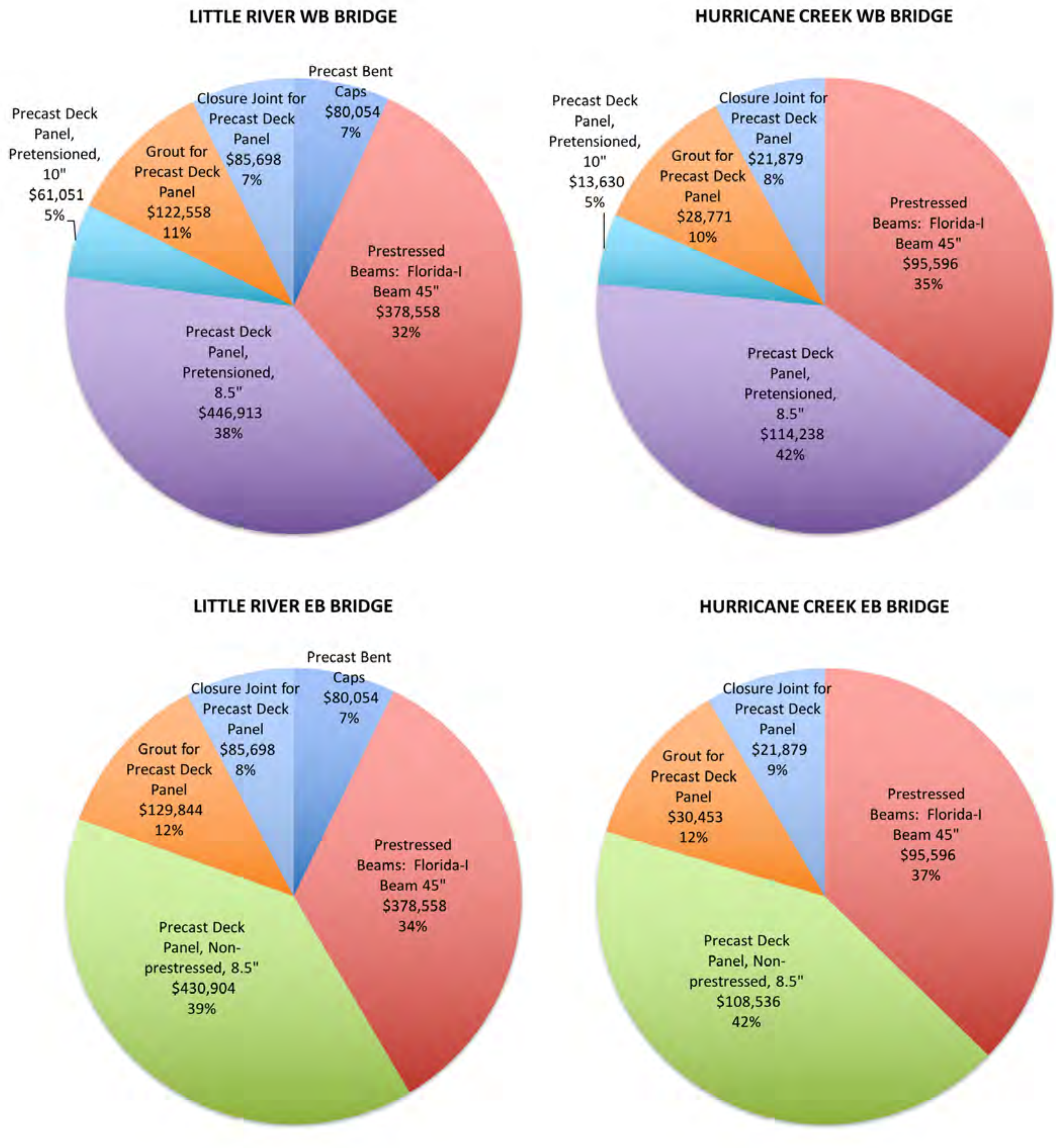


Fig. 1-3. Anderson Columbia's Bids for Precast Elements and Items (for Each Bridge)

CHAPTER 2
US 90 BRIDGE CONSTRUCTION PROJECT

2.1 Construction Contract

The construction project entails the building of four (4) new bridges and approaches on SR 10 (US 90) to replace the two existing bridges over Little River and the existing box culvert at Hurricane Creek. The water bodies are prone to flooding, so the hydraulic performance of the site will be improved by the increased bridge length and vertical clearance and by raising the approach roadways. The new bridges also have wider shoulders than the existing bridges.

The contract also includes drainage improvements, guardrail, signing and pavement markings, and, at the eastern end of the project, friction course milling and resurfacing and shoulder pavement. Other items include excavation, removal of the existing structures, and maintenance of traffic.

2.2 Project Specifications and Contract Requirements

The construction project specifications and contract requirements contained several items that were essential to address or facilitate the unique design aspects and the involvement of researchers.

A **pre-bid meeting** was held in April 2013. Meeting attendance was mandatory for any contractor who wished to submit a construction bid. Prospective bidders had access to plans and specifications prior to the pre-bid meeting, but were issued bidding documents only after attending the pre-bid meeting.

A **pre-construction conference** was held in July 2013, after the contractor was awarded the project. This meeting was for FDOT personnel, the contractor's construction superintendent, equal employment officer, safety officer, and anticipated subcontractors. The contractor was required to submit several items at the meeting, including, but not limited to: a Work Progress Schedule Chart; written work plan; maintenance of traffic plan;

erosion control plan and schedule; and safety program check list. Other items of discussion included concrete (with regard to supply, placement, equipment, protection, and curing), shop drawing submittal procedure, materials testing plan/procedures, and quality control plan. Special project requirements were discussed. The meeting also allowed an opportunity to discuss any plan errors, omissions, or ambiguities that were known to exist. The contractor was to submit a Quality Control Manual no later than 21 days after award of the contract or at the preconstruction meeting, whichever was earlier.

The contractor was required to submit to the Engineer for acceptance a **Critical Path Method (CPM) Contract Schedule** within 30 days after execution of the contract or at the pre-construction conference, whichever was earlier. The schedule was to be updated monthly by the contractor. The specifications required that the schedule include the following items: a CPM Network Diagram; a report that includes each construction activity with other information such as original and remaining durations; and a report that describes the current project schedule status, potential delays, impacts to or shifts in the critical path, and schedule duration changes. In addition, the Engineer held weekly meetings with the contractor to discuss progress, upcoming activities, problems, and proposed solutions.

Developmental Specification Section 404, "Precast Concrete Elements for Bridge Construction", were developed by FDOT. The specifications address the unique aspects of the precast concrete bent caps and deck panels with regard to fabrication, handling, storage, shipping, and erection. They specify requirements for shop drawings and dimensional tolerances, as well as forms and materials. Minimum age or percentage of 28-day compressive strength is specified for moving, shipping, and placing precast members and for opening the bridge to traffic (**Table 2-1**). Also included are requirements for grout material, trial batching, placement, and testing. Basis of payment for related pay items (bent caps, deck panels, grout for panels, and closure joints between panels) is also specified. The specifications require the contractor to submit for approval the following: Shop Drawings; Precast Placement Plan; Precast Stabilization Plan; Grouting Plan; and

Grout Demonstration/Mock-Up Details. Details for these items are discussed in the paragraphs that follow.

The contractor was required to submit **Shop Drawings** to the Engineer for approval. Drawings were produced for the Florida-I beams, precast bent caps, and precast panels (prestressed and non-prestressed). Developmental Specification Section 404 called for the inclusion of all reinforcing details including supplemental steel that remains in place as part of the finished product, the size and type of ducts or other types of grout or block out forms for all precast element connections, duct supports, tremie tubes, air vents, drains, and erection marks to indicate the precast element location and orientation. The precast producer also included in the shop drawings the following: fall protection details; contractor-specific elements that were not on the design drawings; and lifting eyes/apparatuses.

Table 2-1. Minimum Age and Compressive Strength for Moving, Shipping, and Placing Precast Members and for Opening to Traffic

Element	Activity	Minimum Age	Minimum % of 28-day Compressive Strength
Precast Cap	Move from casting bed to storage	7 days	75
	Ship and/or Erect	14 days	85
	Grouting Operation	14 days	85
	Set beams on cap	28 days	100
Grout for Precast Cap	Remove temporary bracing	1 day	40
	Set beams on cap	3 days	60
Precast Deck Panel (non-prestressed)	Move from casting bed to storage	9 days	80
	Ship and/or Erect	28 days	100
	Grouting Operation	28 days	100
Precast Deck Panel (pretensioned)	Move from casting bed to storage	9 days	80
	Ship and/or Erect	28 days	100
	Grouting Operation	60 days	100
Grout for Precast Deck Panel	Remove temporary connections	1 day	40
	Remove leveling bolts	3 days	60
Transverse Closure Joints	Place traffic railing	3 days	60
	Open to traffic	28 days	100

Per the project specifications, minor contractor-proposed modifications would be considered by the Engineer via the shop drawing process. For major construction modifications or contractor redesigns, the contractor was required to submit a Cost Savings Initiative Proposal to FDOT. However, the **specifications did not allow the precast components to be modified to cast-in-place**. Furthermore, eliminating the prestressed or nonprestressed deck panels was not allowed.

Developmental Specification Section 404 required the contractor to submit a **Precast Placement Plan** to describe the proposed construction sequence and show details that would enable construction of all precast elements. The plan was required to include, at a minimum, items such as the following:

- Proposed construction sequence and steps;
- Erection plan (such as crane location, specimen delivery, lifting details, and temporary bracing);
- Precast stabilization plan;
- Geometry control measures, including shimming and leveling methods;
- Hardware used to hold precast elements in position prior to connection grouting;
- Grouting plan (material, mixing, and placing methods – including vent tubes, equipment, and hardware);
- Manufacturer's information on grouts (including performance characteristics, mixing requirements, working time, and curing requirements);
- Void repair plan;
- Weather protection system; and
- Grouting Demonstration Test/Mock-Up details.

The contract required the contractor to perform a **Grouting Demonstration Test** to demonstrate grout properties, adequacy of equipment, and forming material, and to familiarize job site personnel with grouting procedures. Details for full-scale mock-ups of the precast panels and precast bent cap were included in the contract drawings.

1. *Precast Panels:* To demonstrate grouting of the precast panels, an 8-in.-thick, 4-ft by 8-ft top slab (representative of a precast deck panel) was placed over a bottom slab (representative of the Florida-I beam top flange surface), with a gap in between the slabs that varied from 1/2 inch to 4 inches at the four corners. This gap mimics the haunch. Grout flowability and strength were to be measured, and the top slab was to be removed after grout setup so that grout consolidation could be inspected. Payment for the Grout Demonstration Test was made under the pay item for the precast panel grout.
2. *Drilled Shaft – Bent Cap Connection*: To demonstrate grouting of the connection between the drilled shaft and precast bent cap, a full-scale mock-up was constructed. It included a short drilled shaft section with all corrugated steel ducts and dowel reinforcing bars. A gap was left between the top of shaft and bottom of bent cap to emulate the connection. After the grout set up, the bent cap form was removed and the gap was inspected for voids. Payment for the Grout Demonstration Test was made under the pay item for precast bent caps.

More details and the results of the Grouting Demonstration Tests are provided in a separate report, Report 3, Construction of Precast Elements.

The construction contract documents alerted the contractor about the researchers' involvement – that they would be conducting observations, measurements, and testing on the precast concrete elements during the construction process. The **specifications required the contractor to coordinate with research personnel** for the duration of all bridge construction phases and provide them with safe access to the precast elements both in the precast plant and on the project site. This was so that the researchers could observe and document precasting and construction activities. Furthermore, the contractor was required to provide a 30-day notice to the Engineer of when each bridge would be completed and ready for load testing by the researchers. For each bridge test, seven consecutive days of access were given to the researchers after the bridge was completed and before opened to traffic. During that time, no construction personnel, equipment, materials, or activities were permitted on the bridge. All work and costs for this effort was to be included in the maintenance of traffic item and no separate payment was to be made.

2.3 Bridge Dimensions and Arrangements

The new bridges and their span arrangements are shown in **Table 2-2**. The span lengths were kept consistent as much as feasible, while considering the location of the Little River channel and avoiding existing pile foundations. The Hurricane Creek bridges were lengthened a small amount, beyond what was needed for the site, to match the Little River bridge spans.

Table 2-2. US 90 Replacement Bridges and Their Span Arrangements

Bridge No.	Bridge Name	Span Arrangement
500151	US 90 over Little River Westbound	106’-1”—110’-0”—110’-0”—110’-1”
500152	US 90 over Little River Eastbound	106’-1”—110’-0”—110’-0”—110’-1”
500153	US 90 over Hurricane Creek Westbound	110’-2”
500154	US 90 over Hurricane Creek Eastbound	110’-2”

Figures 2-1 and 2-2, from FDOT’s design drawings, show the plan and elevation of Little River and Hurricane Creek bridges. Cross sections of the westbound (WB) and eastbound (EB) bridges are shown in **Figure 2-3**. Each bridge is 43’-1” wide, including barriers, and will carry two (2) 12’-0” lanes and two (2) shoulders with widths of 10’-0” and 6’-0”. The bridges are non-skewed and on a straight horizontal alignment. The Little River bridges lie in a sag vertical curve, with a change in elevation of 2.9 ft over its length of 436.17 ft. The Hurricane Creek bridges are on a flat vertical alignment. Each bridge has a 2% cross-slope for stormwater drainage.

2.4 Overview of Construction Methods

Figures 2-4 and 2-5 show construction renderings of Hurricane Creek and Little River bridges. An important feature of the precast components/system is that overlays, match-casting, and post-tensioning are not required – which if required would complicate construction or necessitate specially-trained workers. Also, the precast components have many ingredients that are similar to the familiar, conventional slab-on-girder bridge.

Conventional methods were used to construct some of the bridge elements. All four (4) bridges have conventional cast-in-place end bents (i.e., abutments) (illustrated in **Figure 2-6**) and 48-inch diameter drilled shafts. The drilled shaft reinforcing steel for the west end bent of Hurricane Creek EB Bridge is shown in **Figure 2-7**; see also **Figure 2-8** for a photo of a tied reinforcing steel cage ready to be installed in a drilled shaft. Photos of the end bent formwork are shown in **Figures 2-9 to 2-11**, and **Figures 2-12 to 2-14** are photos of the cast end bents. With four (4) spans each, the Little River bridges also have intermediate bents, which are each supported by two (2) 48-inch diameter drilled shafts and cast-in-place columns that extend from the drilled shafts. (Reinforcing steel and formwork for the drilled shafts/columns are shown in **Figures 2-15 to 2-17**). **Figure 2-18** shows two completed columns; the dowel bars extending from the top are for connecting a precast bent cap that will support the beams.

Precast construction methods were used to construct the remaining elements.

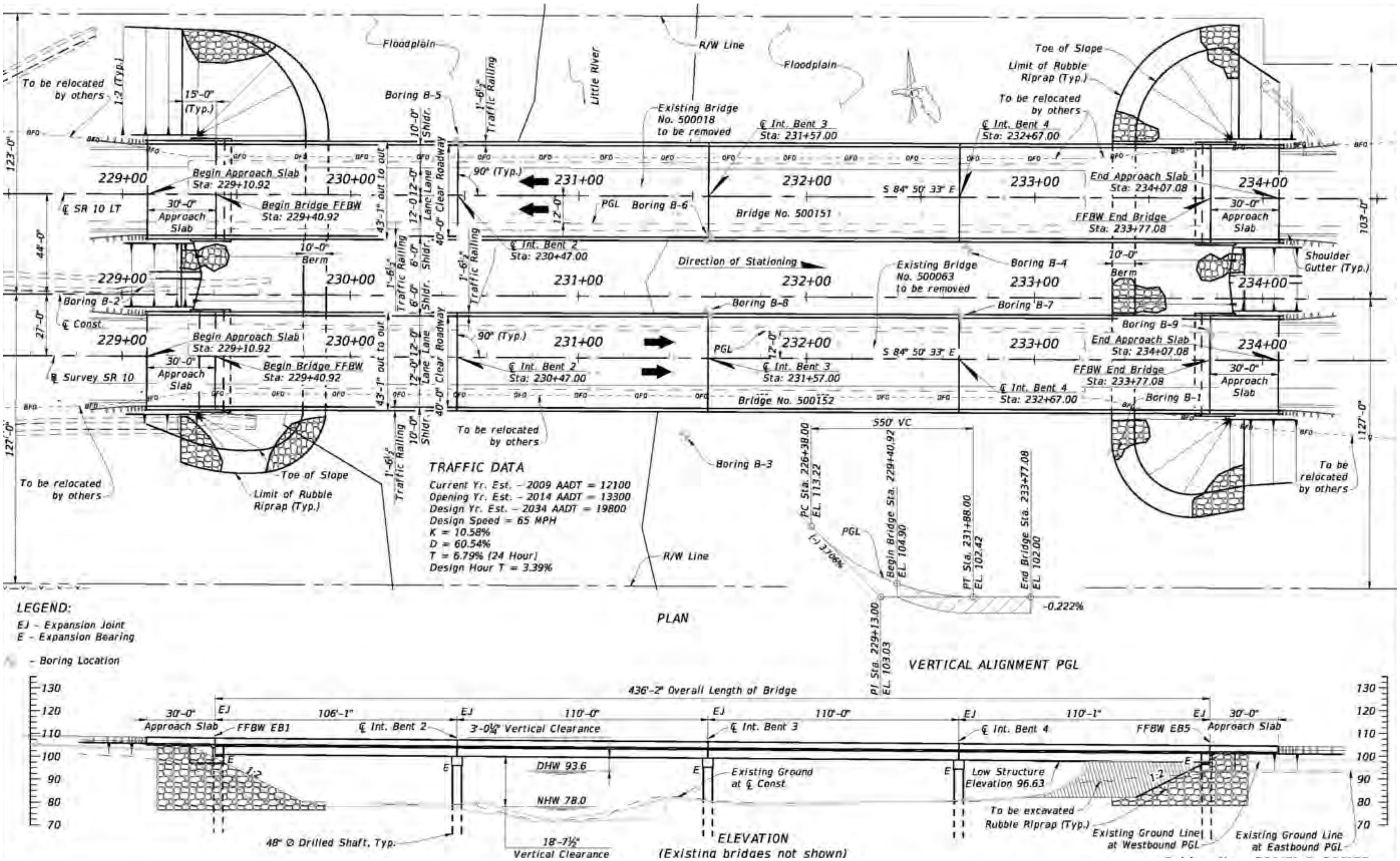


Fig. 2-1. Little River WB and EB Bridges, Plan and Elevation (from Design Plans)

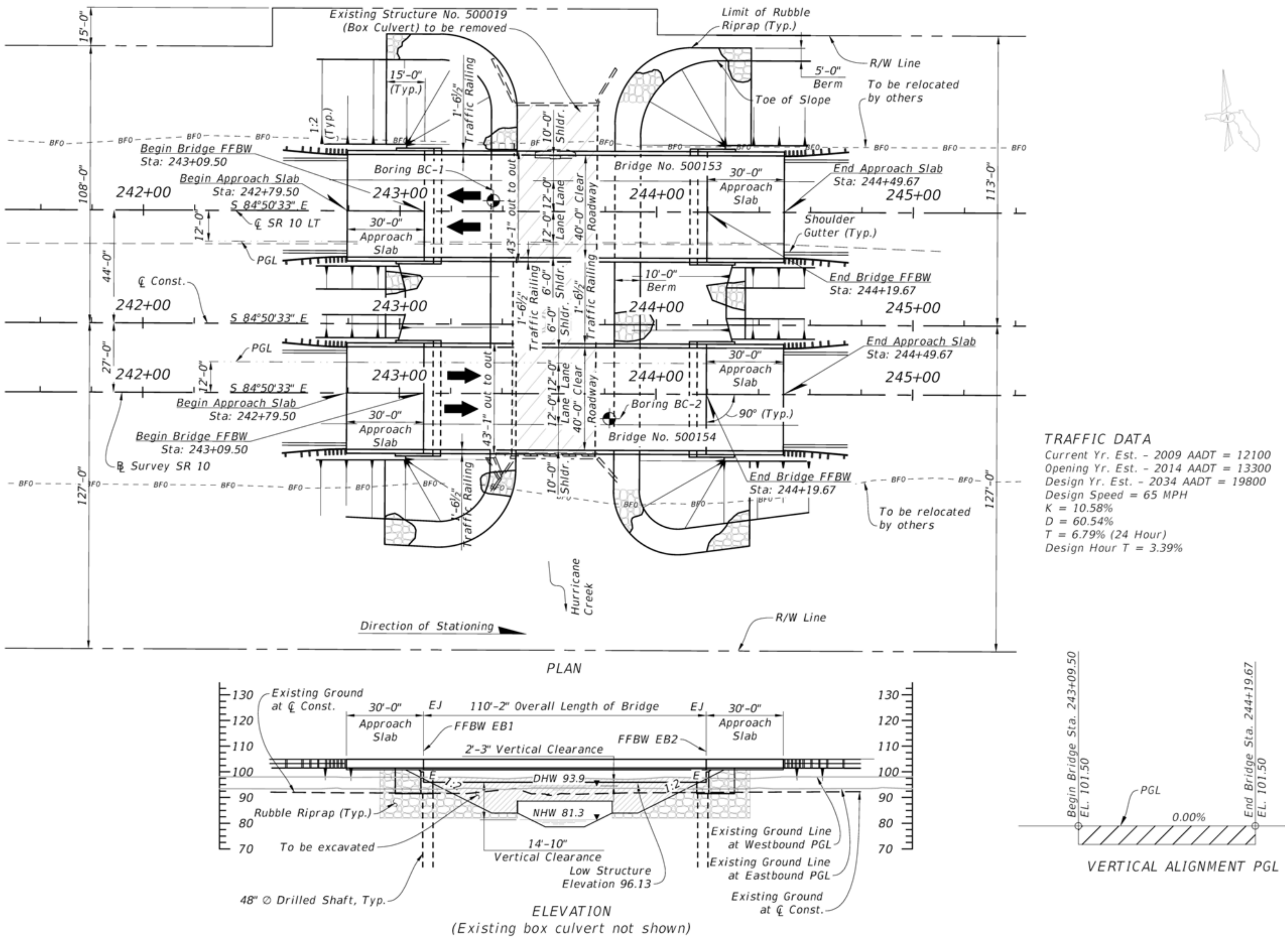
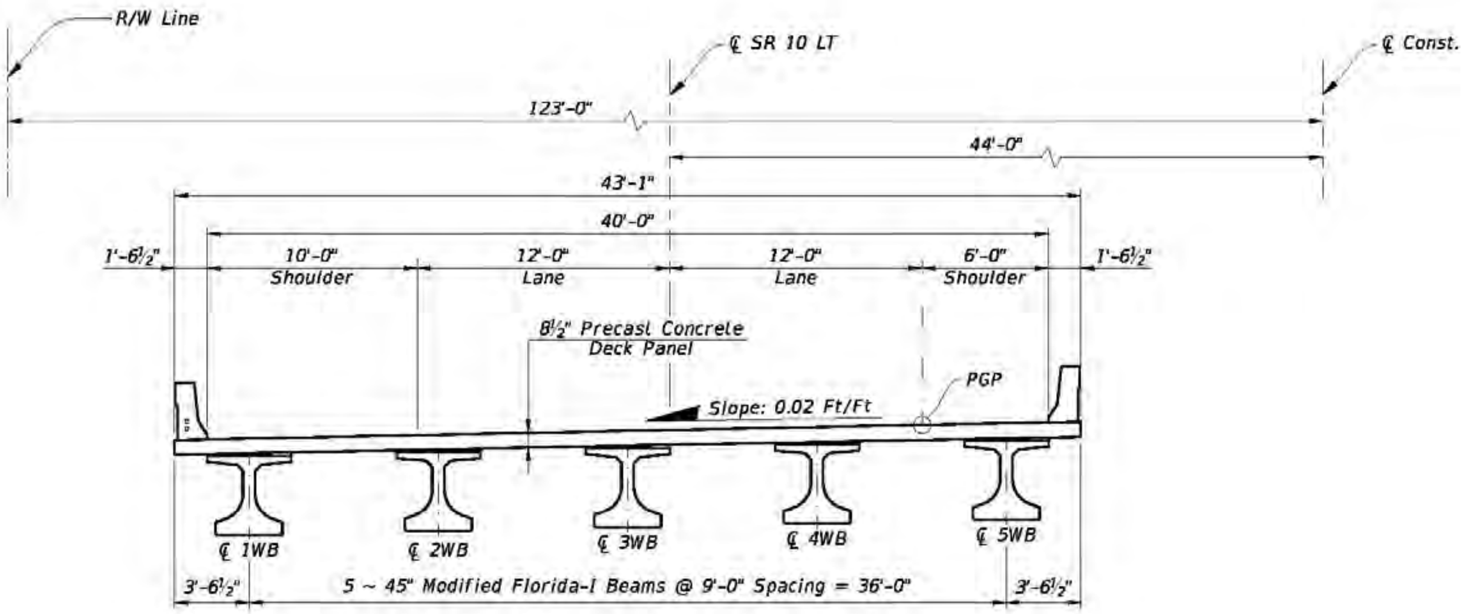
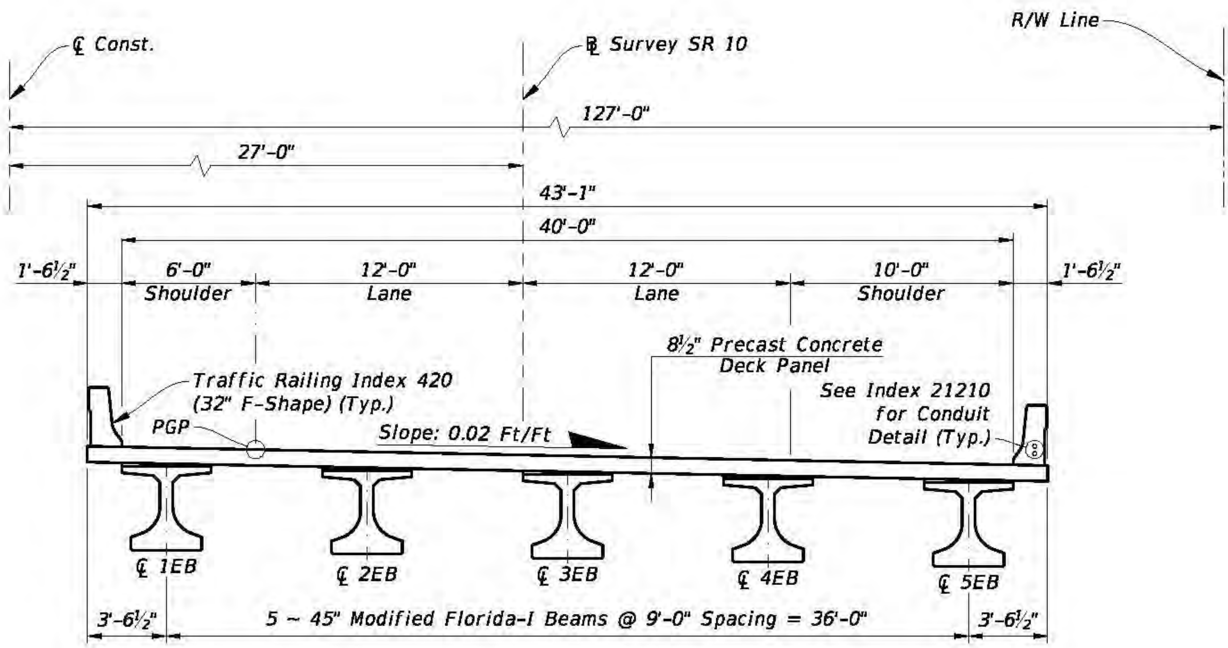


Fig. 2-2. Hurricane Creek WB and EB Bridges, Plan and Elevation (from Design Plans)

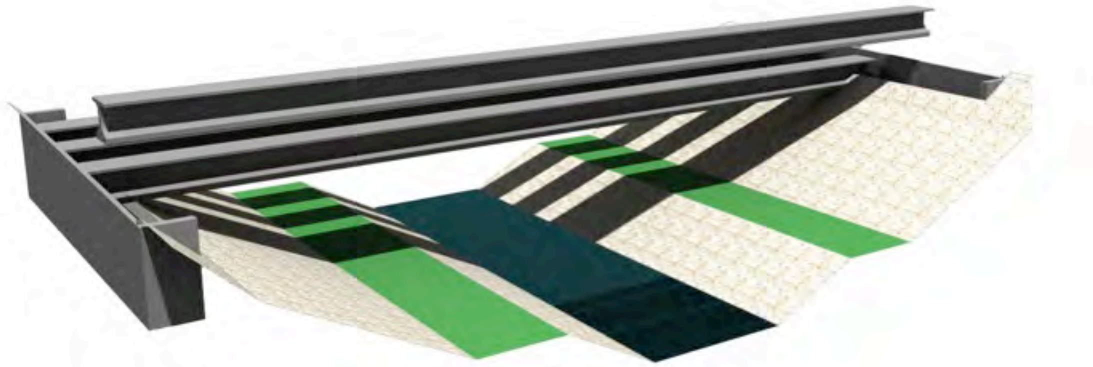


(a) Westbound

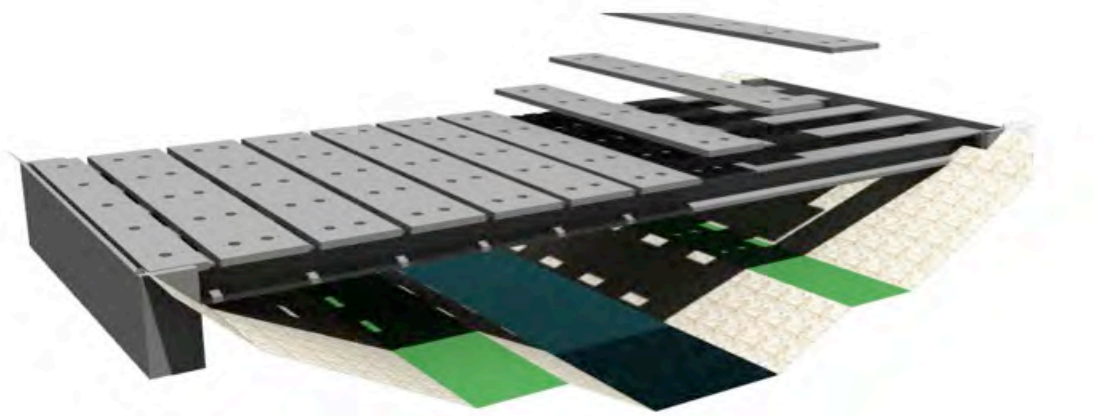


(b) Eastbound

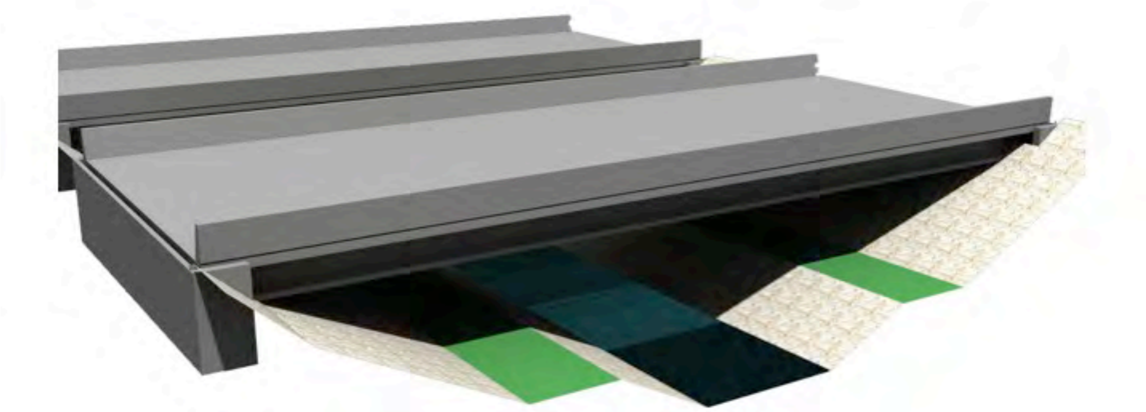
Fig. 2-3. Cross Section of Bridges, Looking East (from Design Plans)



(a) Erection of Florida-I Beams

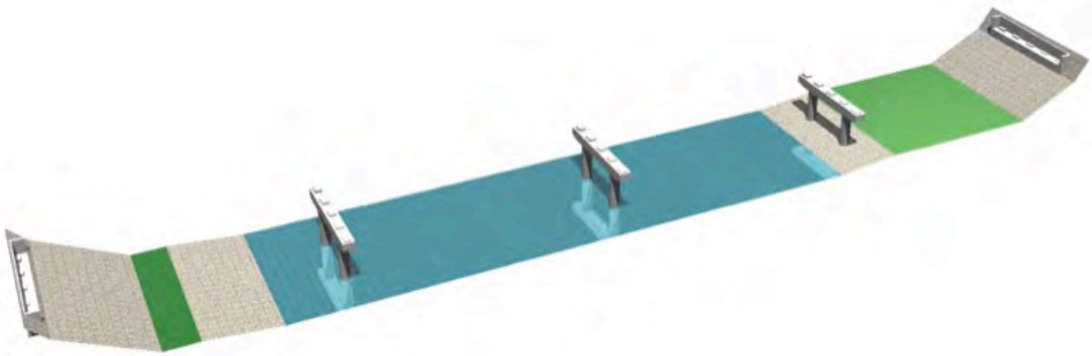


(b) Erection of Precast Deck Panels

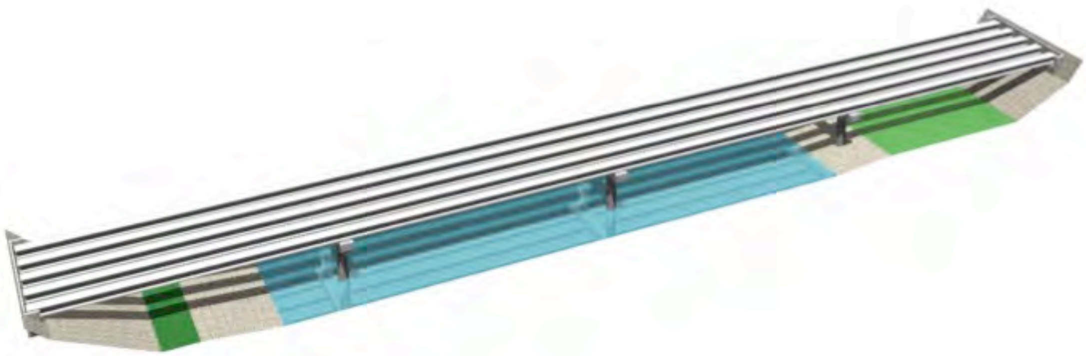


(c) Completed Parallel Bridges with Barriers

Fig. 2-4. Rendering of Hurricane Creek Replacement Bridge Construction



(a) Constructed End Bents and 3 Intermediate Bents



(b) Erected Florida-I Beams

(c) Erected Precast Deck Panels. Construction of Parallel Bridge to Follow.

Fig. 2-5. Rendering of Little River Replacement Bridge Construction

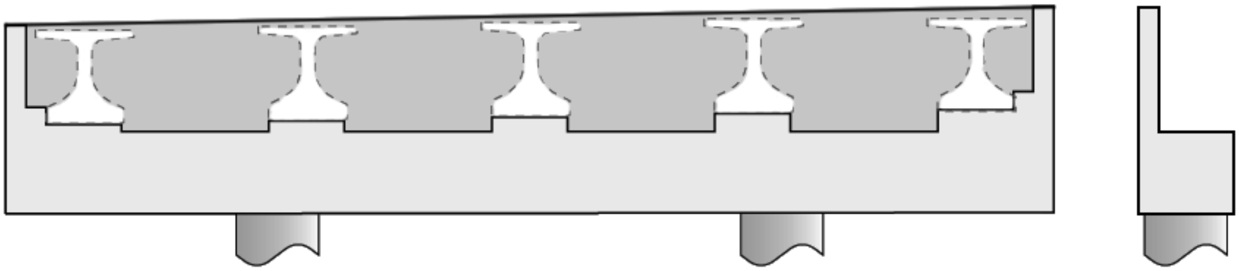


Fig. 2-6. Conventional Cast-in-Place End Bent

Fig. 2-7. Hurricane Creek, Looking East from West End Bent

Fig. 2-8. Drilled Shaft Reinforcing Steel Cage for Little River EB Bridge, View from Southeast Corner, Looking West

Fig. 2-9. Hurricane Creek Bridge, Looking East at End Bent Formwork

Fig. 2-10. Little River Bridge, East End Bent Formwork and High Water Level

Fig. 2-11. Little River Bridge, Finishing Front Face of East End Bent


Fig. 2-12. Hurricane Creek Bridge, West End Bent


Fig. 2-13. Hurricane Creek Bridge, Completed West End Bent, with Pedestals and Bearing Pads in Position

Fig. 2-14. End Bents on Both Ends of Hurricane Creek EB Bridge

Fig. 2-15. Reinforcement Extending from Drilled Shafts for Connection to Columns for Little River Bridge Intermediate Bent

Fig. 2-16. Column Extending from Drilled Shaft for Little River Bridge Intermediate Bent

Fig. 2-17. Both Columns for a Little River Bridge Intermediate Bent, Columns for Two Other End Bents In Background

Fig. 2-18. Columns Extending from Drilled Shafts, Showing Dowel Bars to Connect to Bent Cap, for Little River Bridge Intermediate Bent

Precast Element Evaluation for the US 90 Bridges over Little River and Hurricane Creek

Contract Number BDV30-307-01
FSU Project ID 032819

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Florida Department of Transportation

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March 2016
REPORT 2: FABRICATION OF PRECAST ELEMENTS

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REPORT 2: FABRICATION OF PRECAST ELEMENTS

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CHAPTER 1
INTRODUCTION

This report provides details on the precast element fabrication. The precast yard was visited several times to observe and document the precast activities, and the bridge site was visited often to monitor construction. The role of the researchers was *not* for quality control or quality assurance, as that was left to other parties of the construction contracts.

The research involved observing the methods used to construct the precast elements and the bridges, for example, with regard to formwork, materials used, and the contractor's means and methods. The researchers looked for field issues or construction methods that could affect the constructability, construction schedule, or bridge quality. Although the separate report, Report 3, is about construction, some details that overlapped the precast and construction activities are presented in this report.

Design drawings were examined, shop drawings were studied, Quality Control and Quality Assurance reports were gathered and summarized, and available correspondence (such as construction meeting minutes, contractor's project schedule, and Requests for Information) was examined. Photographs and videos were taken and recorded.

An outline of this report is as follows:

1. PRECAST PRESTRESSED FLORIDA-I BEAMS
2. PRECAST DECK PANELS (GENERAL)
3. NON-PRESTRESSED PRECAST DECK PANELS
4. PRESTRESSED PRECAST DECK PANELS
5. PRECAST BENT CAPS

For each type of precast element, information is included such as:

- Design and shop drawing details
- Photos of formwork, reinforcing steel, embedded items, cast elements

- Casting/stripping/release dates
- Erection dates
- Materials and strengths
- Observations
- Related Requests for Information (RFIs)

CHAPTER 2

PRECAST PRESTRESSED FLORIDA-I BEAMS

2.1 Description

Little River WB and EB Bridges have four (4) spans each, and Hurricane Creek WB and EB Bridges have one (1) span each. All four (4) bridges have 110-ft typical spans, except for the 106-ft Span 1 in the Little River EB and WB Bridges. Each span has five (5) precast, prestressed 45" Florida-I Beams (FIBs) spaced at 9'-0", as shown in Figure 2-3 in Report 1, for a total of 50 FIBs. **Figures 2-1 to 2-3** are excerpts from the design plans that show the FIB dimensions and reinforcement details. FDOT's FIB standards were modified for this project to accommodate the precast deck panels and cast-in-place transverse closure pours.

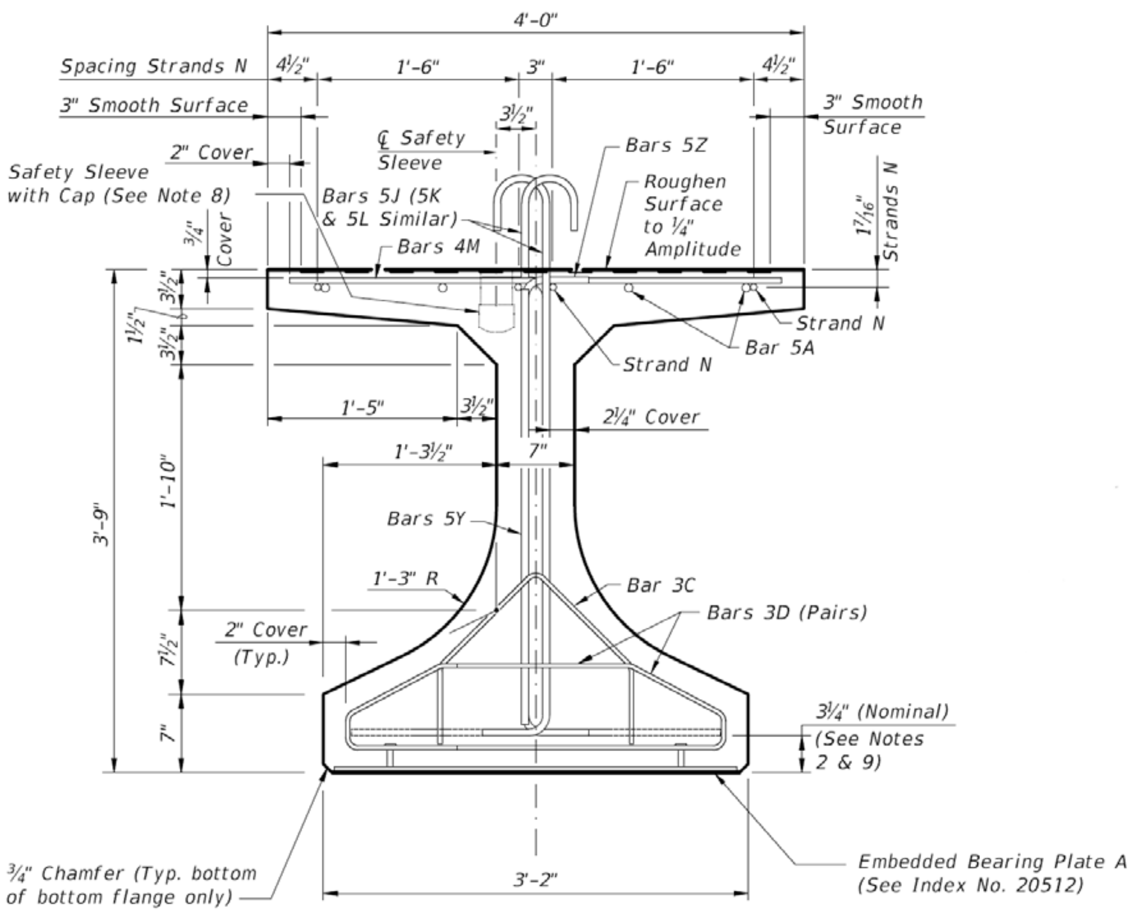


Fig. 2-1. Florida-I Beam Cross Section, Dimensions and Reinforcement Details (from Design Plans)

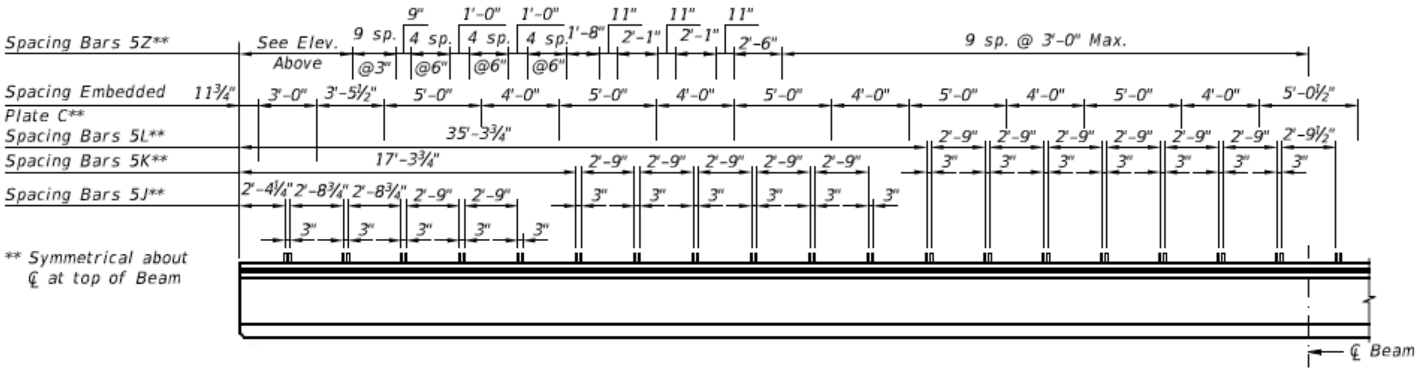


Fig. 2-2. Florida-I Beam Elevation, Spacing of Embedded Plates and Shear Connector Bars (from Design Plans)

The FIBs were fabricated by CDS. **Figures 2-4 to 2-6** are photos taken in their casting yard, showing the reinforcement and strands being positioned before the concrete was cast.

Figure 2-7 illustrates the prestressing strand layout and debonding pattern. Strand pattern Type 1, with 47 strands, applies to 40 of the 50 FIBs; Type 2, with 44 strands, applies to the other 10 FIBs, located on the 106-ft Span 1 Little River EB and WB Bridges. **Figure 2-8** is a photo of a FIB in the casting yard, before the beam end and strands were coated with epoxy as required for protection.

As illustrated in **Figure 2-9**, shear connectors with 180-degree hooks protrude above the FIB top flange to make the FIBs and deck act as a composite section. The shear connectors protrude into the shear pockets of the precast deck panels (**Figure 2-10**) when they are erected at the bridge site. The shear pockets are then to be grouted, along with the haunches. Shear connectors were also placed in the FIBs at the locations of the deck panel closure joints (**Figure 2-11**), which are to be filled with concrete on site. The provided shear connectors (two (2) per panel and one (1) per closure joint) are longitudinally spaced at a maximum of 36 in. **Figure 2-12** illustrates the reinforcing bar bend details for the FIBs; bars 5J, 5K, and 5L are the shear connectors.

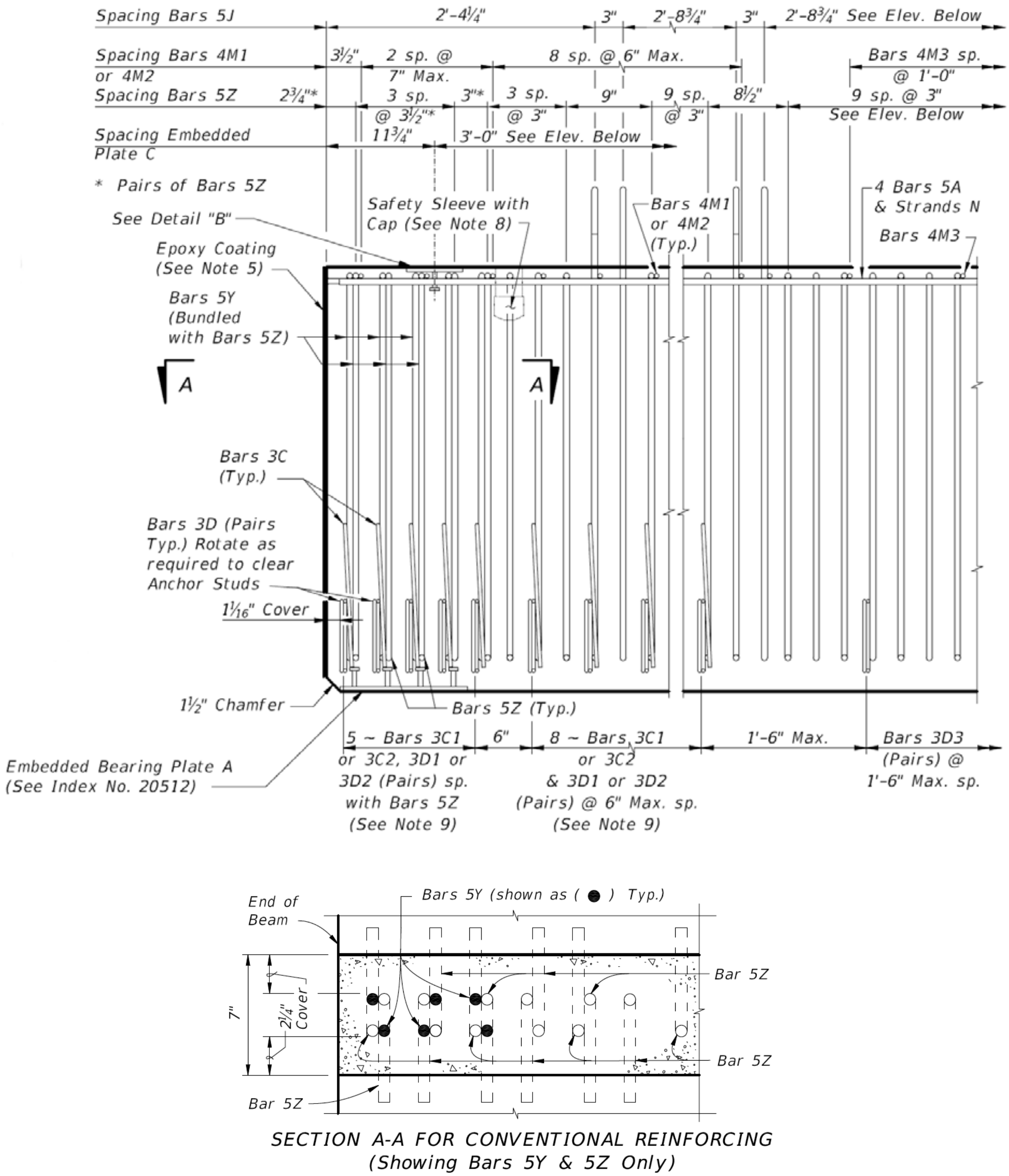
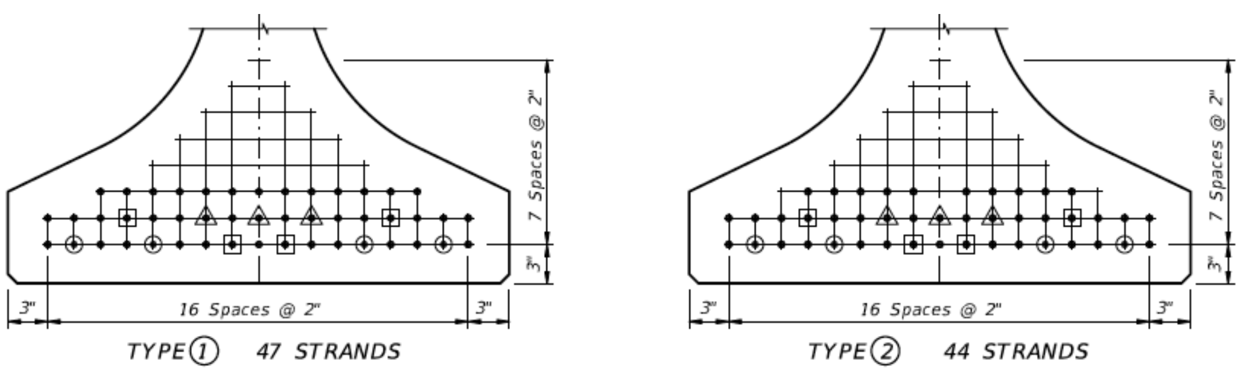


Fig. 2-3. Florida-I Beam End Elevation, Reinforcement Details (from Design Plans)

Fig. 2-4. Precasting Florida-I Beams: Precast Bed, Reinforcing Steel, and Strands

Fig. 2-5. Precasting Florida-I Beams: Close-up of Bars 4M, 5Z and 5J, Strands N

Fig. 2-6. Precasting Florida-I Beams: Getting Bars 5Z in Position



STRAND DESCRIPTION: Use 0.6" Diameter, Grade 270, low relaxation Strands stressed at 43.94 kips each. Area per strand equals 0.217 sq. in.

● – fully bonded strands.

⊙ – strands debonded 25'-0" from end of beam.

▣ – strands debonded 15'-0" from end of beam.

△ – strands debonded 10'-0" from end of beam.

Fig. 2-7. Florida-I Beam Prestressing Strand Layout and Debonding Pattern (from Design Plans)

Fig. 2-8. Photo of Florida-I Beam in the Casting Yard

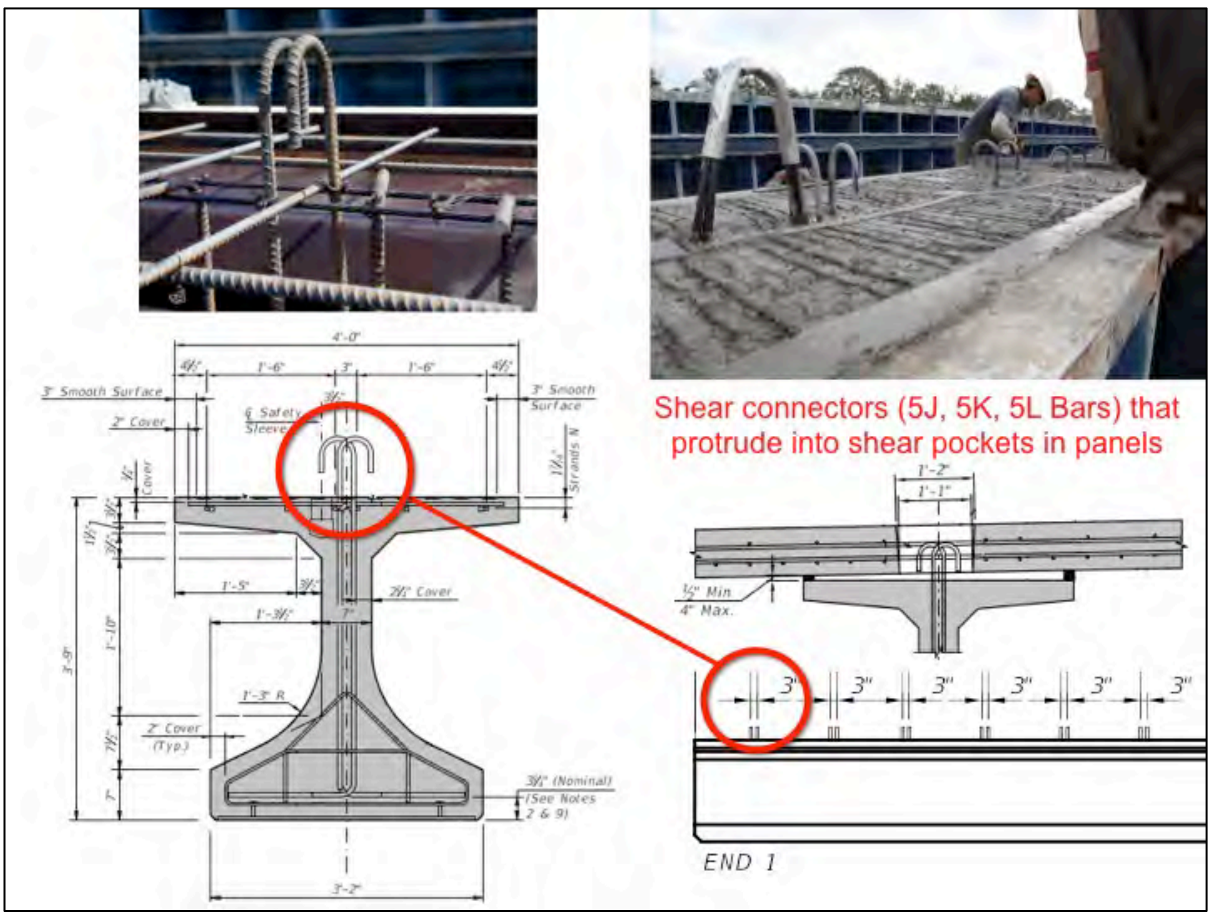


Fig. 2-9. Reinforcing Steel Shear Connectors in FIB Top Flange Will Connect to Precast Deck Panel Shear Pockets

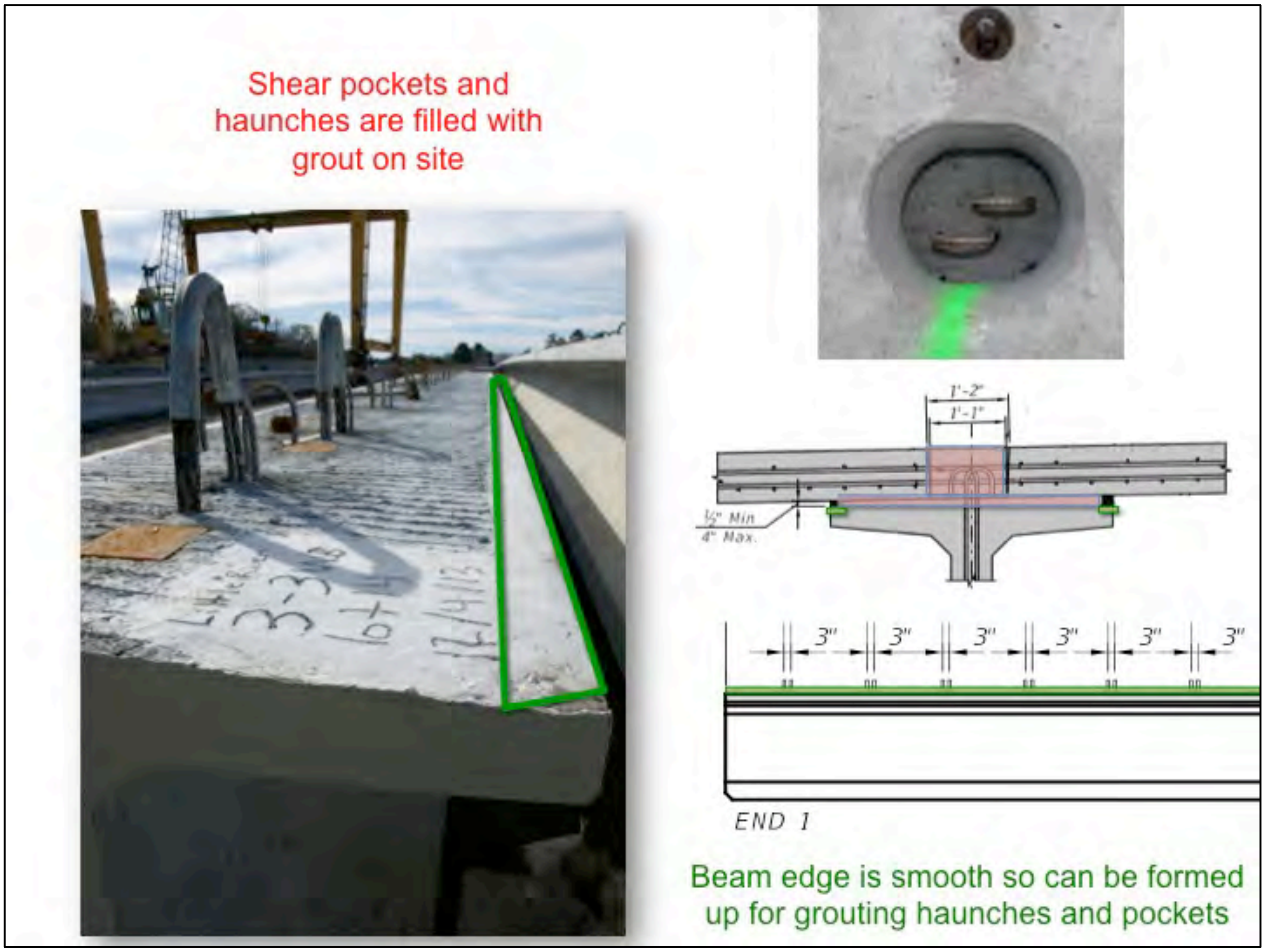


Fig. 2-10. FIB-to-Shear Pocket Connection. Left: Precast FIB (Green Denotes Where Polyethylene Forms Will Be Placed to Contain Grout). Top Right: View from Top of Precast Panel, Showing FIB Shear Connector Bars.

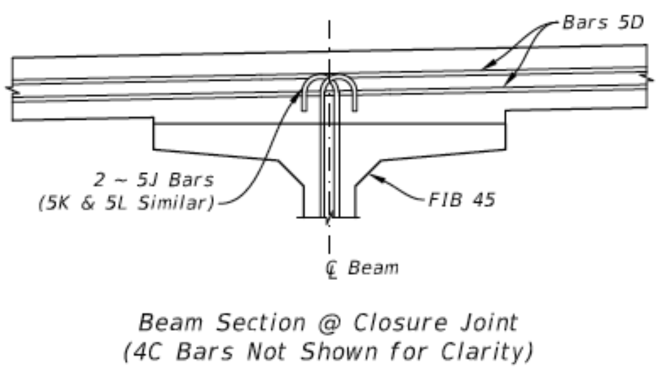


Fig. 2-11. Shear Connectors in FIBs Extend into Precast Deck Panel Closure Joints (from Design Plans)

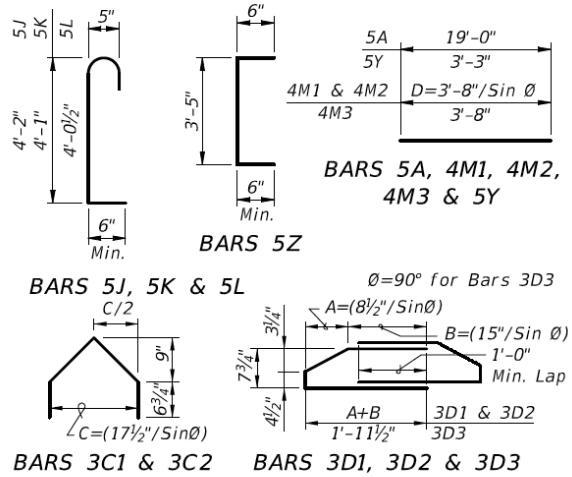


Fig. 2-12. Florida-I Beam Reinforcement Bar Bend Details (from Design Plans)

Figure 2-13 is a photo of the top mat of reinforcing steel, as viewed from above and looking down one side of the forms. After the concrete was cast, embedded plate assemblies were pressed into the top flange and the flange was roughened with a rake, as shown in **Figure 2-14**. (The longitudinal edges of the top flange were smoothed, as shown in Figure 2-10, so that the edges could be formed up with polyethylene, for grouting the haunches and shear pockets.) The embedded plate assemblies were placed at the leveling bolt locations in the precast deck panels (see Figures 2-2 and 2-3 for their spacings). (See also **Figures 2-15 and 2-16** for a photo and diagram of the plate assembly.)

Figure 2-17 is a photo of the FIBs erected on Little River EB Bridge. To erect the precast deck panels, safety lines for fall protection were mounted into the FIB top flanges (**Figure 2-18**). For this purpose, sleeves were cast in the top flange of the FIBs when they were manufactured in the precast yard.

Fig. 2-13. Precasting Florida-I Beams: Showing Top Mat of Reinforcing Steel, Lifting Loop, and Formwork Sides & Bracing

Fig. 2-14. Precasting Florida-I Beams: Close-up of Embedded Plate Assembly and a Lifting Loop; Roughened Surface on Top of FIB

Fig. 2-15. Precasting Florida-I Beams: Close-up of Embedded Plate C Assembly Before Being Placed in Top of FIB

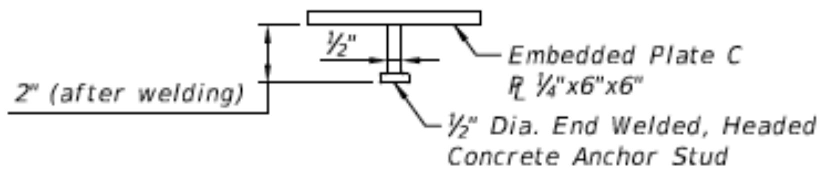


Fig. 2-16. Florida-I Beam Embedded Plate C Assembly Details (from Design Plans)

Fig. 2-17. Florida-I Beams Erected on Little River EB Bridge

Fig. 2-18. Safety Lines Mounted to FIBs for Erecting Precast Deck Panels

2.2 Casting Data

Table 2-1 provides casting information about the Florida-I Beams, including beam # designations and casting, prestress release, and shipping dates/ages.

Table 2-2 provides concrete strengths at prestress release and at 28 days, as well as FDOT inspection notes. A synopsis of the FIB data in Tables 2-1 and 2-2 is as follows.

Casting:

The 50 Florida-I Beams were cast in pairs in the casting bed, using one (1) set of prestressing strands. Casting occurred on:

- 10/9/2013 – 12/11/2013
- Over a period of 63 days (46 weekdays)

Prestress release occurred shortly after casting (most within one (1) day), as summarized in **Table 2-3**.

Table 2-1. Florida-I Beam Designations and Casting/Shipping Dates

BRIDGE # / NAME	SPAN # / BEAM LENGTH	BEAM #	CAST TOGETHER	CAST DATE	RELEASE DATE	AGE AT RELEASE (days)	SHIPPING DATE	AGE AT SHIPPING (days)
500151 LITTLE RIVER WB	1 105' 11"	LR-45-1-1WB		11/13/13	11/14/13	1	8/13/15	638
		LR-45-1-2WB		11/15/13	11/18/13	3	8/13/15	636
		LR-45-1-3WB		11/15/13	11/18/13	3	8/13/15	636
		LR-45-1-4WB		11/19/13	11/20/13	1	8/13/15	632
		LR-45-1-5WB		11/19/13	11/20/13	1	8/13/15	632
	2 109' 10.5"	LR-45-2-1WB		11/27/13	11/28/13	1	8/13/15	624
		LR-45-2-2WB		11/27/13	11/28/13	1	8/13/15	624
		LR-45-2-3WB		11/28/13	12/2/13	4	8/13/15	623
		LR-45-2-4WB		11/28/13	12/2/13	4	8/13/15	623
		LR-45-2-5WB		12/3/13	12/4/13	1	8/13/15	618
	3 109' 10.5"	LR-45-3-1WB		12/3/13	12/4/13	1	8/13/15	618
		LR-45-3-2WB		12/4/13	12/5/13	1	8/13/15	617
		LR-45-3-3WB		12/4/13	12/5/13	1	8/13/15	617
		LR-45-3-4WB		12/5/13	12/6/13	1	8/13/15	616
		LR-45-3-5WB		12/5/13	12/6/13	1	8/13/15	616
	4 109' 11"	LR-45-4-1WB		12/6/13	12/7/13	1	8/14/15	616
		LR-45-4-2WB		12/6/13	12/7/13	1	8/14/15	616
		LR-45-4-3WB		12/7/13	12/9/13	2	8/14/15	615
		LR-45-4-4WB		12/7/13	12/9/13	2	8/14/15	615
		LR-45-4-5WB		12/9/13	12/10/13	1	8/14/15	613
500152 LITTLE RIVER EB	1 105' 11"	LR-45-1-1EB		11/9/13	11/10/13 *	1	9/18/14	313
		LR-45-1-2EB		11/9/13	11/10/13 *	1	9/18/14	313
		LR-45-1-3EB		11/12/13	11/13/13	1	9/18/14	310
		LR-45-1-4EB		11/12/13	11/13/13	1	9/18/14	310
		LR-45-1-5EB		11/13/13	11/14/13	1	9/18/14	309
	2 109' 10.5"	LR-45-2-1EB		11/20/13	11/21/13	1	9/18/14	302
		LR-45-2-2EB		11/20/13	11/21/13	1	9/18/14	302
		LR-45-2-3EB		11/21/13	11/22/13	1	9/18/14	301
		LR-45-2-4EB		11/21/13	11/22/13	1	9/18/14	301
		LR-45-2-5EB		11/22/13	11/23/13	1	9/18/14	300
	3 109' 10.5"	LR-45-3-1EB		11/22/13	11/23/13	1	9/19/14	301
		LR-45-3-2EB		11/23/13	11/25/13	2	9/19/14	300
		LR-45-3-3EB		11/23/13	11/25/13	2	9/19/14	300
		LR-45-3-4EB		11/25/13	11/27/13	2	9/19/14	298
		LR-45-3-5EB		11/25/13	11/27/13	2	9/19/14	298
	4 109' 11"	LR-45-4-1EB		12/9/13	12/10/13	1	9/20/14	285
		LR-45-4-2EB		12/10/13	12/11/13	1	9/20/14	284
		LR-45-4-3EB		12/10/13	12/11/13	1	9/20/14	284
		LR-45-4-4EB		12/11/13	12/12/13	1	9/20/14	283
		LR-45-4-5EB		12/11/13	12/12/13	1	9/20/14	283
500153 HURRICANE CREEK WB	1 109' 11.5"	HC-45-1-1WB		10/9/13	10/11/13	2	6/15/15	614
		HC-45-1-2WB		10/10/13	10/11/13	1	6/15/15	613
		HC-45-1-3WB		10/15/13	10/16/13	1	6/15/15	608
		HC-45-1-4WB		10/15/13	10/16/13	1	6/15/15	608
		HC-45-1-5WB		10/17/13	10/18/13	1	6/15/15	606
500154 HURRICANE CREEK EB	1 109' 11.5"	HC-45-1-1EB		10/17/13	10/18/13	1	3/24/14	158
		HC-45-1-2EB		10/22/13	10/23/13	1	3/24/14	153
		HC-45-1-3EB		10/22/13	10/23/13	1	3/24/14	153
		HC-45-1-4EB		10/24/13	10/25/13	1	3/24/14	151
		HC-45-1-5EB		10/24/13	10/25/13	1	3/24/14	151

* Release Date was assumed for LR-45-1-1EB and LR-45-1-2EB.

Beams were in line in casting bed and used same set of prestressing strands.

Table 2-2. Florida-I Beam Concrete Strengths and Inspection Notes

BEAM #	CAST TOGETHER	CDS CONCRETE LOT #	CDS CONCRETE STRENGTH AT RELEASE (psi)	CDS CONCRETE STRENGTH AT 28 DAYS (psi)	FDOT CONCRETE SAMPLE # TO VERIFY CDS	FDOT CONCRETE STRENGTH AT 28 DAYS (psi)	FDOT SPECIAL NOTES ("post-pour" observations)
LR-45-1-1WB							0 cracks
LR-45-1-2WB		G345010Q	9 900	12 440			0 cracks
LR-45-1-3WB							0 cracks
LR-45-1-4WB		G345011Q	7 410	12 750	G345011V	11 490	4 cracks
LR-45-1-5WB							1 crack
LR-45-2-1WB		G345017Q	6 170	13 020			0 cracks
LR-45-2-2WB							0 cracks
LR-45-2-3WB		G345018Q	10 040	12 920			0 cracks
LR-45-2-4WB							0 cracks
LR-45-2-5WB		G345019Q	6 300	11 180			0 cracks
LR-45-3-1WB							0 cracks
LR-45-3-2WB		G345020Q	7 370	12 460	G345020V	11 920	0 cracks
LR-45-3-3WB							1 crack
LR-45-3-4WB		G345021Q	6 590	10 890			0 cracks/minor spall end
LR-45-3-5WB							0 cracks
LR-45-4-1WB		G345022Q	7 270	12 060	G345022V	11 520	1 crack
LR-45-4-2WB							0 cracks
LR-45-4-3WB		G345023Q	7 990	11 240			0 cracks
LR-45-4-4WB							0 cracks
LR-45-4-5WB		G345024Q	6 790	11 740			1 crack
LR-45-1-1EB		G345007Q	7 670	11 750			1 crack
LR-45-1-2EB							1 crack
LR-45-1-3EB		G345008Q	7 310	12 780			5 cracks
LR-45-1-4EB							9 cracks
LR-45-1-5EB		G345009Q	6 360	13 570			0 cracks
LR-45-2-1EB		G345012Q	6 970	12 630			--
LR-45-2-2EB							--
LR-45-2-3EB		G345013Q	7 410	12 910			3 cracks
LR-45-2-4EB							1 crack
LR-45-2-5EB		G345014Q	6 620	12 440			0 cracks
LR-45-3-1EB							0 cracks
LR-45-3-2EB		G345015Q	7 710	11 330	G345015V	9 870	1 crack
LR-45-3-3EB							1 crack
LR-45-3-4EB		G345016Q	8 690	11 420			0 cracks
LR-45-3-5EB							0 cracks
LR-45-4-1EB							0 cracks
LR-45-4-2EB		G345025Q	7 440	11 460			1 crack
LR-45-4-3EB							0 cracks
LR-45-4-4EB		G345026Q	7 850	12 210	G345026V	11 920	3 cracks
LR-45-4-5EB							1 crack
HC-45-1-1WB		G345001Q	8 055	11 940			0 cracks
HC-45-1-2WB		G345002Q	6 540	9 790			0 cracks
HC-45-1-3WB		G345003Q	7 980	12 290	G345003V	11 600	2 small cracks
HC-45-1-4WB							0 cracks
HC-45-1-5WB		G345004Q	7 410	11 060			3 cracks
HC-45-1-1EB							2 cracks
HC-45-1-2EB		G345005Q	7 840	11 840	G345005V	11 640	0 cracks
HC-45-1-3EB			7 840				0 cracks
HC-45-1-4EB		G345006Q	7 240	12 480			0 cracks
HC-45-1-5EB			7 240				0 cracks

Note: All beams used MIX 03-1722, which is FDOT Class VI (8500 psi at 28 days). 6000 psi required at release.

Beams were in line in casting bed and used same set of prestressing strands.

Table 2-3. FIB Prestress Release and Number of Days After Casting

# of Days Prestress Release Occurred after Casting	# of Beams
1	39
2	7
3	2
4	2

Materials:

The design plans specified Class VI (8500 psi) concrete for prestressed concrete beams, with a required strength of 6000 psi at the time of prestress release. All beams were cast with MIX 03-1722, which is FDOT Class VI (8500 psi). **Table 2-4** summarizes the minimum, maximum, and average concrete strengths measured at prestress release and at 28 days.

Table 2-4. FIB Measured Concrete Strengths (Minimum, Maximum, and Average)

FIB Concrete Strength	At Prestress Release (psi)	At 28 days (psi)
Minimum	6 170	9 790
Maximum	10 040	13 570
Average	7 500	12 023

The design plans specified 0.6-inch diameter, Grade 270, low-relaxation strands stressed at 43.94 kips each. Material certifications for the strands used in the beams consistently showed a modulus of elasticity of 28,800 ksi.

Erection:

From Table 2-1, for Hurricane Creek EB Bridge, the 5 beams were erected on 3/24/2014. The age on erection day ranged from 151 to 158 days, with an average of 153 days. For Little River EB Bridge, the 20 beams were erected on 9/18/2014 – 9/20/2014. The age on erection day ranged from 283 to 313 days, with an average of 299 days.

For Hurricane Creek WB Bridge, the 5 beams were erected on 6/15/2015. The age on erection day ranged from 606 to 614 days, with an average of 610 days. For Little River WB Bridge, the 20 beams were erected on 8/13/2015 – 8/14/2015. The age on erection day ranged from 613 to 638 days, with an average of 622 days.

2.3 Camber and Sweep of FIBs

Data was gathered on the FIB camber measurements made in the precast yard (**Table 2-5**). The precaster took measurements on the release date for each beam and also monthly between 1/6/2014 and 8/23/2014; thereafter, the contractor took the measurements. The FIBs' ages on the dates of measurement are provided in **Table 2-6**. The data from Tables 2-5 and 2-6 are plotted for each bridge in **Figures 2-19 to 2-22**. (In the plots, "Day 0" is the release date, and the dates of the monthly readings were converted to "Days after Release" for each beam). Initial cambers ranged between 1 1/8" and 2 1/2", with an average of 1.90". As of 8/23/2014, some beams over 250 days old had cambers in excess of 5".

In Figures 2-19 to 2-22, the "+" symbol indicates the theoretical (design) cambers at 120 days, based on the concrete design strength of 8500 psi and assuming release at 1 day after casting. These theoretical cambers are 4" and 4 7/16" for the 106-ft and 110-ft beams, respectively. Five (5) beams had measured 120-day cambers greater than theoretical, within about a half inch, but the measured cambers were *typically less than* theoretical, within about an inch. Figures 2-19 to 2-22 also show the theoretical (predicted) cambers through 240 days, based on the average measured concrete strength of 12,000 psi. These values are shown by the dashed and dotted lines, assuming release at 1 day and 3 days, respectively, after casting. Measured cambers were *typically more than* these predictions.

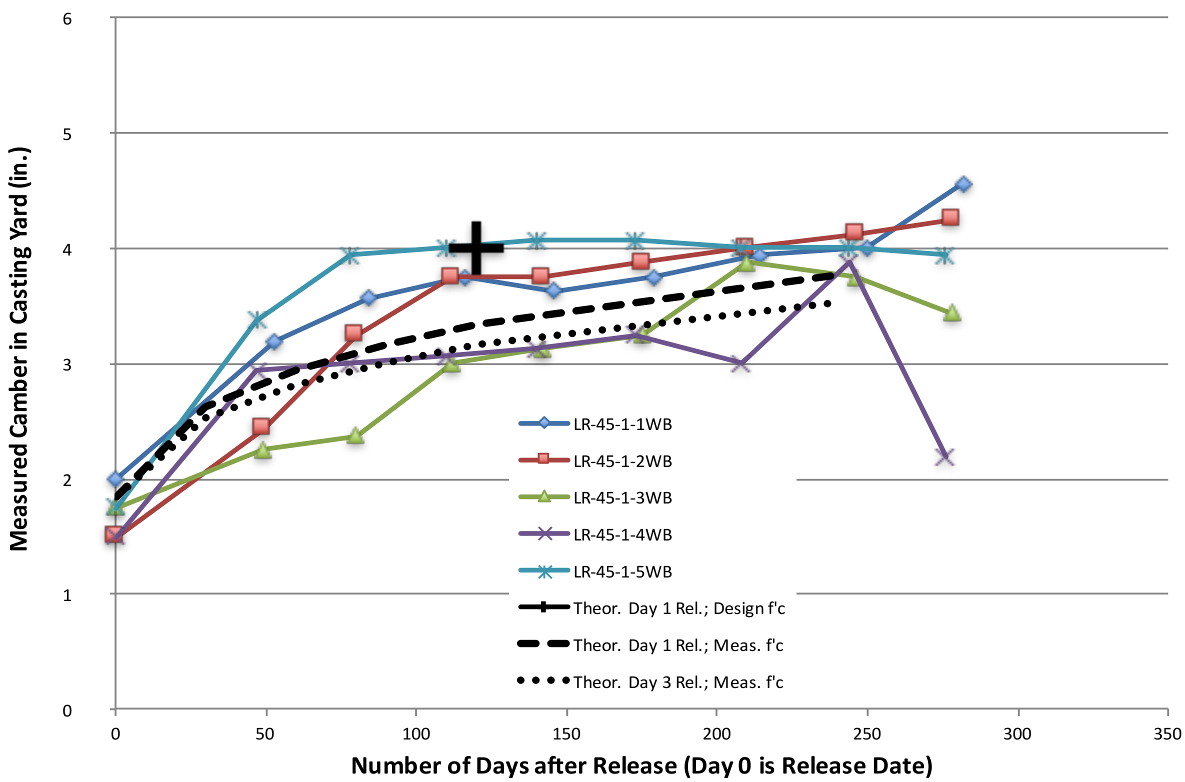
Sweep measurements were also made in the precast yard (**Table 2-7**). Sweeps ranged between 1/16" and 3/4". The monthly readings, averaged for all beams, were between 0.19" and 0.27" over the initial eight-month period. Excessive sweep could influence how well the beams fit over the precast deck panel shear pockets, especially after including casting tolerances.

Table 2-5. Florida-I Beam Camber Measurements in Precast Yard

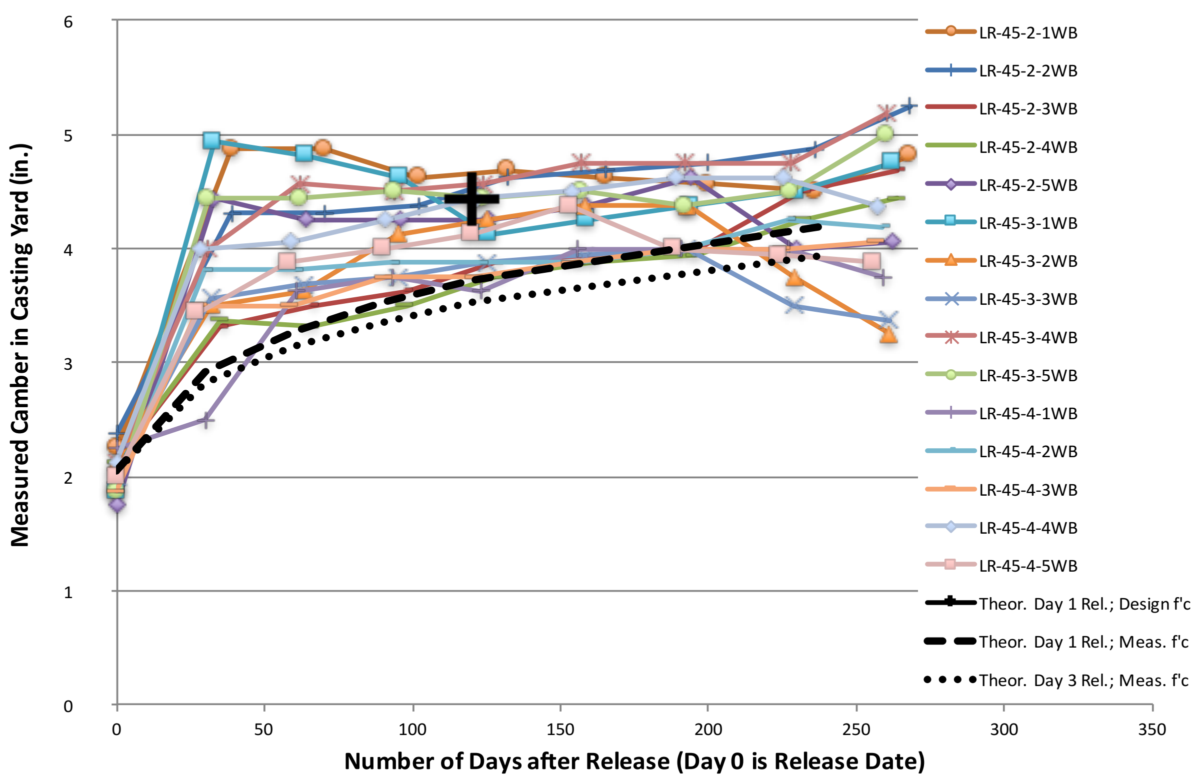
BEAM #	CAMBER AT RELEASE (in.)	CAMBER ON 1/6/2014 (in.)	CAMBER ON 2/6/2014 (in.)	CAMBER ON 3/10/2014 (in.)	CAMBER ON 4/9/2014 (in.)	CAMBER ON 5/12/2014 (in.)	CAMBER ON 6/16/2014 (in.)	CAMBER ON 7/22/2014 (in.)	CAMBER ON 8/23/2014 (in.)
LR-45-1-1WB	2	3 3/16	3 9/16	3 3/4	3 5/8	3 3/4	3 15/16	4	4 9/16
LR-45-1-2WB	1 1/2	2 7/16	3 1/4	3 3/4	3 3/4	3 7/8	4	4 1/8	4 1/4
LR-45-1-3WB	1 3/4	2 1/4	2 3/8	3	3 1/8	3 1/4	3 7/8	3 3/4	3 7/16
LR-45-1-4WB	1 1/2	2 15/16	3	3 1/16	3 1/8	3 1/4	3	3 7/8	2 3/16
LR-45-1-5WB	1 3/4	3 3/8	3 15/16	4	4 1/16	4 1/16	4	4	3 15/16
LR-45-2-1WB	2 1/4	4 7/8	4 7/8	4 5/8	4 11/16	4 5/8	4 9/16	4 1/2	4 13/16
LR-45-2-2WB	2 3/8	4 5/16	4 5/16	4 3/8	4 5/8	4 11/16	4 3/4	4 7/8	5 1/4
LR-45-2-3WB	2 1/8	3 5/16	3 1/2	3 5/8	3 7/8	3 15/16	4	4 1/2	4 11/16
LR-45-2-4WB	2 1/8	3 3/8	3 5/16	3 1/2	3 3/4	3 7/8	3 15/16	4 1/4	4 7/16
LR-45-2-5WB	1 3/4	4 7/16	4 1/4	4 1/4	4 1/4	4 3/8	4 5/8	4	4 1/16
LR-45-3-1WB	1 7/8	4 15/16	4 13/16	4 5/8	4 1/8	4 1/4	4 3/8	4 1/2	4 3/4
LR-45-3-2WB	2 1/8	3 1/2	3 5/8	4 1/8	4 1/4	4 3/8	4 3/8	3 3/4	3 1/4
LR-45-3-3WB	2	3 9/16	3 11/16	3 3/4	3 7/8	3 15/16	4	3 1/2	3 3/8
LR-45-3-4WB	2 1/8	4	4 9/16	4 1/2	4 9/16	4 3/4	4 3/4	4 3/4	5 3/16
LR-45-3-5WB	1 7/8	4 7/16	4 7/16	4 1/2	4 7/16	4 1/2	4 3/8	4 1/2	5
LR-45-4-1WB	2 1/4	2 1/2	3 5/8	3 3/4	3 5/8	4	4	4	3 3/4
LR-45-4-2WB	1 7/8	3 13/16	3 13/16	3 7/8	3 7/8	3 7/8	4	4 1/4	4 3/16
LR-45-4-3WB	1 7/8	3 1/2	3 1/2	3 3/4	3 3/4	3 7/8	4	4	4 1/16
LR-45-4-4WB	2 1/8	4	4 1/16	4 1/4	4 7/16	4 1/2	4 5/8	4 5/8	4 3/8
LR-45-4-5WB	2	3 7/16	3 7/8	4	4 1/8	4 3/8	4	3 15/16	3 7/8
LR-45-1-1EB	1 7/8	2 15/16	2 3/4	3	2 3/4	2 7/8	2 15/16	2 15/16	2 3/4
LR-45-1-2EB	2 1/4	2 15/16	3 1/8	3 1/2	2 3/4	2 3/4	2 15/16	2 3/4	2 5/8
LR-45-1-3EB	1 1/8	3 3/4	3 13/16	3 7/8	3 3/4	3 3/4	3 7/8	4	4 7/16
LR-45-1-4EB	1 1/4	3 3/8	2 13/16	3 1/8	3 3/8	3 5/8	3 5/8	3 7/8	3 15/16
LR-45-1-5EB	1 3/4	3 1/2	4	3 15/16	3 15/16	3 7/8	3 7/8	4 1/8	5
LR-45-2-1EB	1 7/8	3 11/16	4 5/16	4 1/4	4	4 1/16	4 1/8	4 1/8	3 7/8
LR-45-2-2EB	1 7/8	4 1/8	4 1/16	4 1/4	4 1/4	4 1/2	3 7/8	3 7/8	3 11/16
LR-45-2-3EB	1 7/8	4	3 7/8	4 1/16	4 1/16	4 1/4	3	3 1/16	2 15/16
LR-45-2-4EB	2 1/8	3 11/16	3 1/2	3 3/4	3 7/8	3 15/16	3 3/4	3 1/2	3 1/4
LR-45-2-5EB	1 3/4	3 15/16	3 7/8	4	4	4 1/16	4 1/16	4 1/16	4
LR-45-3-1EB	2 1/8	2 15/16	4 9/16	4 5/8	4 5/8	4 3/4	4 1/2	4 1/2	4 1/8
LR-45-3-2EB	1 7/8	4 1/4	4 1/2	4 5/8	4 5/8	4 3/4	4 3/4	4 3/4	4 7/8
LR-45-3-3EB	2 1/8	3 13/16	3 5/8	3 7/8	3 7/8	3 15/16	3 15/16	4	3 3/4
LR-45-3-4EB	2 1/2	3 1/2	3 3/4	3 7/8	3 3/4	3 7/8	3 3/4	4 1/16	3 1/2
LR-45-3-5EB	2 1/4	3 11/16	4 1/16	4 1/8	4 1/8	4 1/8	4 1/4	4 3/8	4 1/2
LR-45-4-1EB	2 1/8	3	3 3/16	3 3/4	3 1/2	3 3/4	3 3/4	3 3/4	3 7/16
LR-45-4-2EB	1 7/8	4 1/8	4 1/16	4 1/8	4 1/8	4 1/8	4 5/8	4 5/8	4 3/4
LR-45-4-3EB	1 7/8	4 7/16	4 3/4	4 1/2	4 1/2	4 5/8	4 5/8	4 5/8	4 15/16
LR-45-4-4EB	2	3 1/2	3 13/16	4 1/8	4 1/4	4 3/8	4 3/4	4 7/8	5 1/8
LR-45-4-5EB	2 1/4	2 7/8	3 7/16	3 3/4	3 7/8	4	4 1/4	4 1/4	4 1/2
HC-45-1-1WB	2 1/4	3 3/4	2 3/4	3 3/4	4	4 1/16	4 3/16	4 1/4	4 3/8
HC-45-1-2WB	1 3/4	3 5/16	3 3/4	3 7/8	3 15/16	3 15/16	4 1/16	4 1/8	3 5/8
HC-45-1-3WB	1 1/2	3 1/4	3 1/4	3 3/4	3 7/8	4	4	4	4
HC-45-1-4WB	1 1/2	3	3 11/16	3 7/8	3 7/8	4	4	3 7/8	3 1/2
HC-45-1-5WB	1 3/8	4	3 15/16	3 3/4	3 5/8	3 3/4	4 7/16	4 7/16	4 1/4
HC-45-1-1EB	1 1/2	3 5/16	3 1/4	3 3/4	ship 3/24	ship 3/24	ship 3/24	ship 3/24	ship 3/24
HC-45-1-2EB	1 5/8	4	3 3/4	4 1/16	ship 3/24	ship 3/24	ship 3/24	ship 3/24	ship 3/24
HC-45-1-3EB	1 3/4	3 7/16	3 1/4	4	ship 3/24	ship 3/24	ship 3/24	ship 3/24	ship 3/24
HC-45-1-4EB	1 3/4	3 3/4	3 11/16	4 1/8	ship 3/24	ship 3/24	ship 3/24	ship 3/24	ship 3/24
HC-45-1-5EB	1 3/4	3 5/8	3 1/4	4	ship 3/24	ship 3/24	ship 3/24	ship 3/24	ship 3/24

Table 2-6. Florida-I Beam Ages on Camber Measurement Dates

BEAM #	AGE AT RELEASE	AGE ON 1/6/2014	AGE ON 2/6/2014	AGE ON 3/10/2014	AGE ON 4/9/2014	AGE ON 5/12/2014	AGE ON 6/16/2014	AGE ON 7/22/2014	AGE ON 8/23/2014
LR-45-1-1WB	0	53	84	116	146	179	214	250	282
LR-45-1-2WB	0	49	80	112	142	175	210	246	278
LR-45-1-3WB	0	49	80	112	142	175	210	246	278
LR-45-1-4WB	0	47	78	110	140	173	208	244	276
LR-45-1-5WB	0	47	78	110	140	173	208	244	276
LR-45-2-1WB	0	39	70	102	132	165	200	236	268
LR-45-2-2WB	0	39	70	102	132	165	200	236	268
LR-45-2-3WB	0	35	66	98	128	161	196	232	264
LR-45-2-4WB	0	35	66	98	128	161	196	232	264
LR-45-2-5WB	0	33	64	96	126	159	194	230	262
LR-45-3-1WB	0	33	64	96	126	159	194	230	262
LR-45-3-2WB	0	32	63	95	125	158	193	229	261
LR-45-3-3WB	0	32	63	95	125	158	193	229	261
LR-45-3-4WB	0	31	62	94	124	157	192	228	260
LR-45-3-5WB	0	31	62	94	124	157	192	228	260
LR-45-4-1WB	0	30	61	93	123	156	191	227	259
LR-45-4-2WB	0	30	61	93	123	156	191	227	259
LR-45-4-3WB	0	28	59	91	121	154	189	225	257
LR-45-4-4WB	0	28	59	91	121	154	189	225	257
LR-45-4-5WB	0	27	58	90	120	153	188	224	256
LR-45-1-1EB	0	57	88	120	150	183	218	254	286
LR-45-1-2EB	0	57	88	120	150	183	218	254	286
LR-45-1-3EB	0	54	85	117	147	180	215	251	283
LR-45-1-4EB	0	54	85	117	147	180	215	251	283
LR-45-1-5EB	0	53	84	116	146	179	214	250	282
LR-45-2-1EB	0	46	77	109	139	172	207	243	275
LR-45-2-2EB	0	46	77	109	139	172	207	243	275
LR-45-2-3EB	0	45	76	108	138	171	206	242	274
LR-45-2-4EB	0	45	76	108	138	171	206	242	274
LR-45-2-5EB	0	44	75	107	137	170	205	241	273
LR-45-3-1EB	0	44	75	107	137	170	205	241	273
LR-45-3-2EB	0	42	73	105	135	168	203	239	271
LR-45-3-3EB	0	42	73	105	135	168	203	239	271
LR-45-3-4EB	0	40	71	103	133	166	201	237	269
LR-45-3-5EB	0	40	71	103	133	166	201	237	269
LR-45-4-1EB	0	27	58	90	120	153	188	224	256
LR-45-4-2EB	0	26	57	89	119	152	187	223	255
LR-45-4-3EB	0	26	57	89	119	152	187	223	255
LR-45-4-4EB	0	25	56	88	118	151	186	222	254
LR-45-4-5EB	0	25	56	88	118	151	186	222	254
HC-45-1-1WB	0	87	118	150	180	213	248	284	316
HC-45-1-2WB	0	87	118	150	180	213	248	284	316
HC-45-1-3WB	0	82	113	145	175	208	243	279	311
HC-45-1-4WB	0	82	113	145	175	208	243	279	311
HC-45-1-5WB	0	80	111	143	173	206	241	277	309
HC-45-1-1EB	0	80	111	143	ship 3/24	ship 3/24	ship 3/24	ship 3/24	ship 3/24
HC-45-1-2EB	0	75	106	138	ship 3/24	ship 3/24	ship 3/24	ship 3/24	ship 3/24
HC-45-1-3EB	0	75	106	138	ship 3/24	ship 3/24	ship 3/24	ship 3/24	ship 3/24
HC-45-1-4EB	0	73	104	136	ship 3/24	ship 3/24	ship 3/24	ship 3/24	ship 3/24
HC-45-1-5EB	0	73	104	136	ship 3/24	ship 3/24	ship 3/24	ship 3/24	ship 3/24

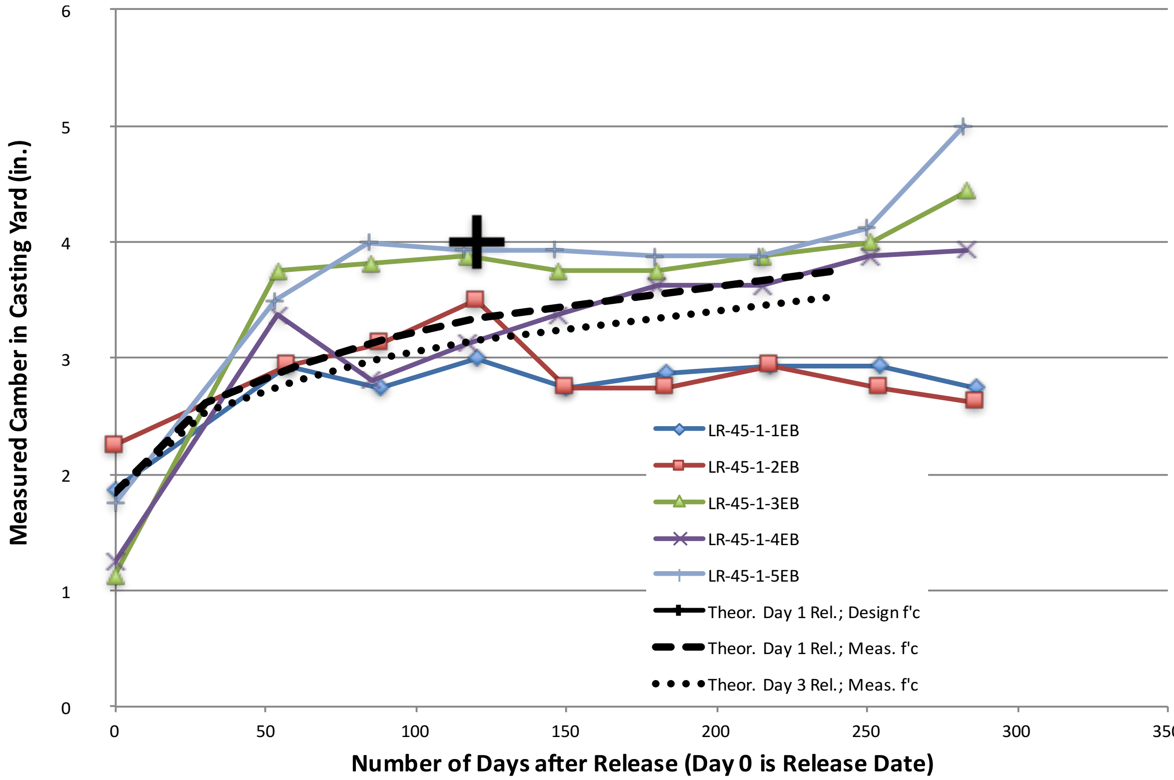


(a) 106-ft Beams

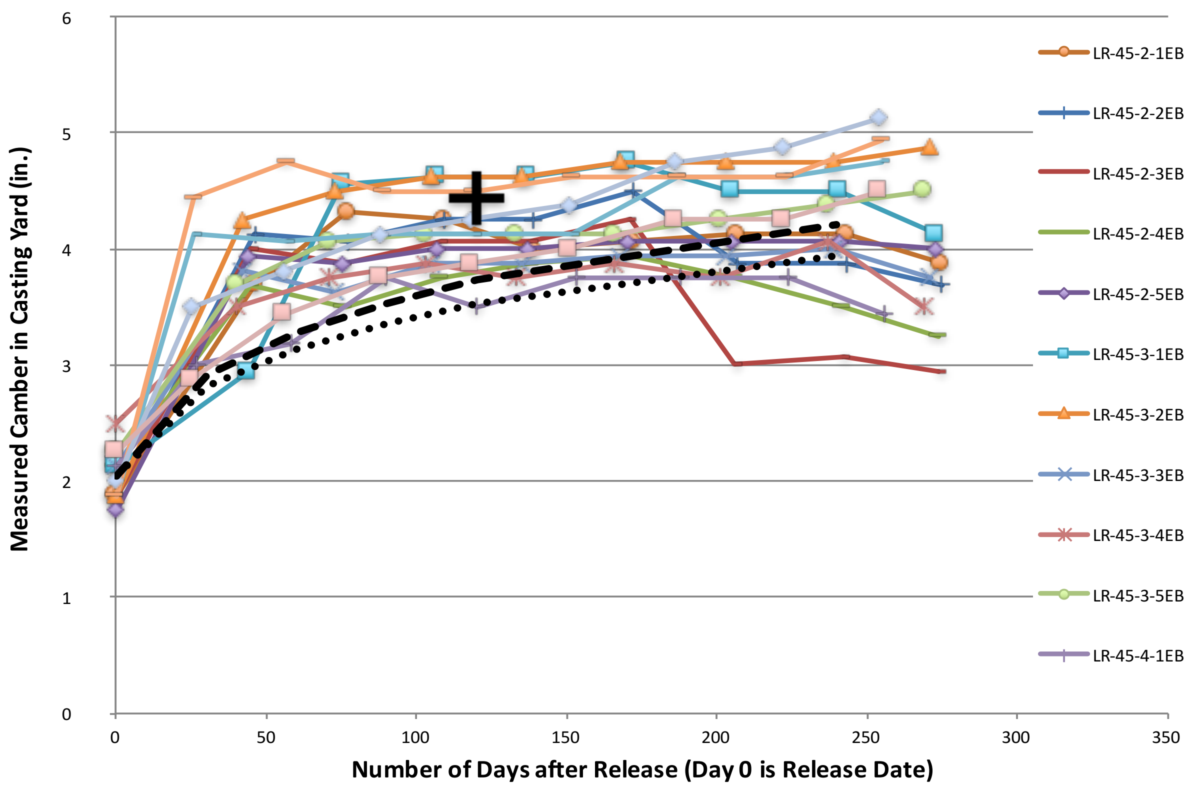


(b) 110-ft Beams

Fig. 2-19. FIB Camber Measured in Casting Yard, Little River WB



(a) 106-ft Beams



(b) 110-ft Beams

Fig. 2-20. FIB Camber Measured in Casting Yard, Little River EB

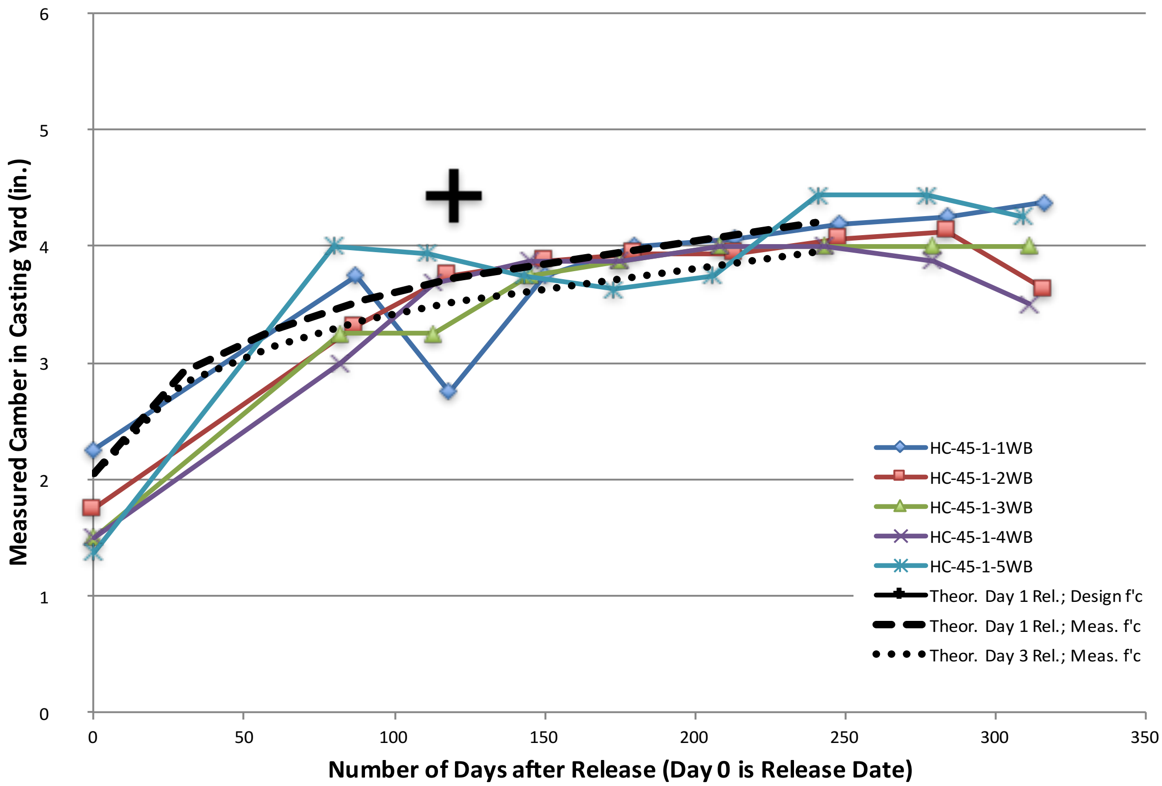


Fig. 2-21. FIB Camber Measured in Casting Yard, Hurricane Creek WB

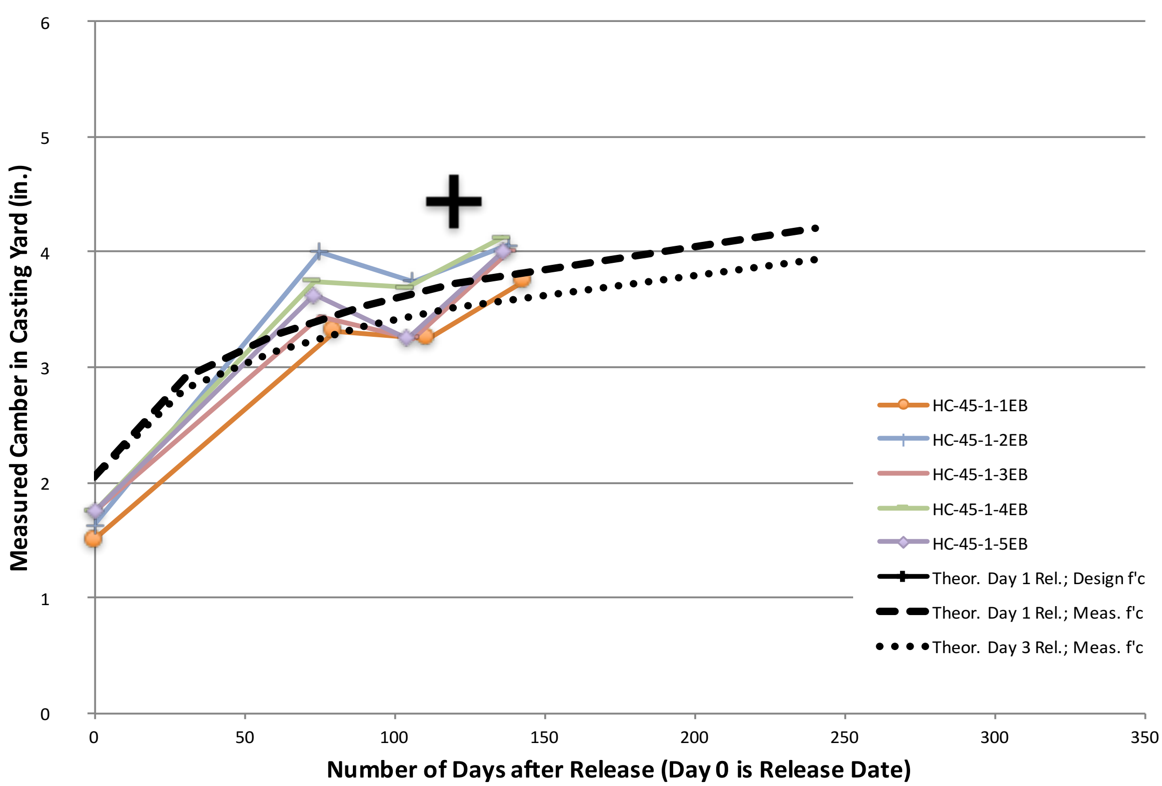


Fig. 2-22. FIB Camber Measured in Casting Yard, Hurricane Creek EB

Table 2-7. Florida-I Beam Sweep Measurements in Precast Yard

BEAM #	SWEEP AT RELEASE (in.)	SWEEP ON 1/6/2014 (in.)	SWEEP ON 2/6/2014 (in.)	SWEEP ON 3/10/2014 (in.)	SWEEP ON 4/9/2014 (in.)	SWEEP ON 5/12/2014 (in.)	SWEEP ON 6/16/2014 (in.)	SWEEP ON 7/22/2014 (in.)	SWEEP ON 8/23/2014 (in.)
LR-45-1-1WB	1/4	1/4	1/4	1/4	1/4	1/4	1/8	1/8	3/8
LR-45-1-2WB	1/8	1/4	1/4	1/8	3/8	3/8	1/8	1/8	3/8
LR-45-1-3WB	1/4	1/8	1/8	1/8	1/8	1/8	1/8	1/8	1/4
LR-45-1-4WB	3/8	1/8	1/8	1/4	3/8	3/8	3/8	3/8	1/2
LR-45-1-5WB	3/8	1/8	1/8	1/8	1/8	1/8	1/8	1/8	3/16
LR-45-2-1WB	1/8	1/8	1/8	1/4	5/8	5/8	1/2	1/2	3/8
LR-45-2-2WB	1/8	1/8	1/8	1/8	1/8	1/8	1/8	1/8	1/4
LR-45-2-3WB	1/4	1/4	1/4	1/8	1/8	1/8	1/4	1/4	1/8
LR-45-2-4WB	1/8	1/4	1/4	1/4	1/4	1/4	1/8	1/8	1/16
LR-45-2-5WB	1/8	1/2	1/2	3/4	5/8	5/8	1/2	1/2	1/4
LR-45-3-1WB	1/4	1/4	1/4	1/4	1/4	1/4	1/4	1/4	1/8
LR-45-3-2WB	1/4	3/4	3/4	3/4	3/4	3/4	3/4	3/4	3/16
LR-45-3-3WB	1/4	1/8	1/8	1/8	1/8	1/8	1/8	1/8	1/8
LR-45-3-4WB	1/4	1/8	1/8	1/8	1/8	1/8	1/8	1/8	1/16
LR-45-3-5WB	3/8	1/8	1/8	1/4	1/4	1/4	1/4	1/4	1/8
LR-45-4-1WB	1/4	1/8	1/8	1/4	1/8	1/8	1/8	1/8	1/8
LR-45-4-2WB	1/8	1/8	1/8	1/4	1/4	1/4	1/4	1/4	3/16
LR-45-4-3WB	1/8	1/8	1/8	1/4	1/4	1/4	1/4	1/4	3/16
LR-45-4-4WB	1/8	1/8	1/8	1/8	3/8	3/8	3/8	3/8	1/4
LR-45-4-5WB	1/8	1/8	1/8	1/8	3/8	3/8	3/8	3/8	1/4
LR-45-1-1EB	1/8	1/16	1/16	1/8	1/4	1/4	3/8	3/8	1/16
LR-45-1-2EB	1/8	1/8	1/8	1/4	1/4	1/4	1/4	1/4	1/16
LR-45-1-3EB	1/8	1/8	1/8	1/8	1/8	1/8	1/4	1/4	1/8
LR-45-1-4EB	3/16	1/4	1/4	1/4	1/4	1/4	1/8	1/8	1/4
LR-45-1-5EB	1/4	1/8	1/8	1/8	1/4	1/4	1/4	1/4	3/8
LR-45-2-1EB	1/4	1/8	1/8	1/4	1/4	1/4	1/4	1/4	1/8
LR-45-2-2EB	1/8	1/8	1/8	1/4	1/4	1/4	1/4	1/4	1/8
LR-45-2-3EB	1/8	1/8	1/8	1/4	1/4	1/4	1/4	1/4	3/16
LR-45-2-4EB	1/4	1/4	1/4	1/8	1/8	1/8	1/8	1/8	1/8
LR-45-2-5EB	1/8	3/16	3/16	1/8	1/8	1/8	1/8	1/8	1/4
LR-45-3-1EB	1/8	1/4	1/4	3/8	1/2	1/2	3/8	3/8	1/16
LR-45-3-2EB	1/8	3/8	3/8	1/2	1/2	1/2	5/8	5/8	1/4
LR-45-3-3EB	1/8	1/2	1/2	1/2	5/8	5/8	3/4	3/4	1/4
LR-45-3-4EB	1/8	1/8	1/8	1/8	1/8	1/8	1/8	1/8	1/8
LR-45-3-5EB	1/8	1/8	1/8	1/4	1/4	1/4	1/4	1/4	3/16
LR-45-4-1EB	3/8	1/8	1/8	1/8	1/8	1/8	1/8	1/8	1/4
LR-45-4-2EB	3/8	1/8	1/8	1/8	3/8	3/8	1/2	1/2	1/8
LR-45-4-3EB	1/4	1/8	1/8	1/8	1/8	1/8	1/8	1/8	3/16
LR-45-4-4EB	1/4	1/4	1/4	1/4	1/4	1/4	1/4	1/4	3/16
LR-45-4-5EB	5/8	3/8	3/8	1/4	3/8	3/8	3/8	3/8	1/4
HC-45-1-1WB	1/4	1/4	1/4	1/8	1/8	1/8	1/8	1/8	3/8
HC-45-1-2WB	1/4	1/4	1/4	1/4	1/4	1/4	1/4	1/4	1/4
HC-45-1-3WB	1/8	1/8	1/8	1/4	1/4	1/4	1/4	1/4	3/8
HC-45-1-4WB	1/4	1/8	1/8	1/4	1/8	1/8	1/4	1/4	1/4
HC-45-1-5WB	1/4	1/8	1/8	1/8	1/8	1/8	1/8	1/8	1/8
HC-45-1-1EB	1/4	1/8	1/8	1/8	ship 3/24	ship 3/24	ship 3/24	ship 3/24	ship 3/24
HC-45-1-2EB	1/4	1/4	1/4	1/4	ship 3/24	ship 3/24	ship 3/24	ship 3/24	ship 3/24
HC-45-1-3EB	1/4	1/8	1/8	1/4	ship 3/24	ship 3/24	ship 3/24	ship 3/24	ship 3/24
HC-45-1-4EB	1/8	1/8	1/8	1/8	ship 3/24	ship 3/24	ship 3/24	ship 3/24	ship 3/24
HC-45-1-5EB	1/4	1/8	1/8	1/8	ship 3/24	ship 3/24	ship 3/24	ship 3/24	ship 3/24

2.4 Observations

Stabilization of vertical reinforcing bars:

An FDOT inspector at the casting yard recommended (for future designs) adding a dormant strand down the center of the web, near the beam's mid height, to provide another location to tie off and stabilize vertical bars 5J, 5K, 5L, and 5Z. (Alternatively, this could be presented as an option by the precaster.) Figures 2-5 and 2-6 show how the vertical bars are tied off at the top and bottom.

Safety lines for fall protection:

To erect the precast deck panels on the FIBs, safety lines were installed (**Figure 2-23**). The vertical steel pipes that hold the lines were inserted into safety sleeves that were precast into the FIB top flange, as shown in Figure 2-1. The sleeve spacing was designed by the contractor/precaster.

Fig. 2-23. Safety Lines for Fall Protection (2 FIBs Are Temporarily Shifted Out of Position)

For conventional construction of a cast-in-place slab, the sleeves would be automatically filled with concrete when the deck is cast. However, for this project, after the FIBs and precast deck panels were erected, the sleeves were located under shear pockets (**Figure 2-24(a)**), in closure joints (**Figure 2-24(b)**), or underneath a panel. After the panels were erected, the safety lines were no longer needed and were therefore removed. For the sleeves located under the shear pockets, the inspector then directed the contractor to fill the sleeves with grout, since they would not be reliably filled during haunch-and-pocket grouting operations. For the sleeves located in closure joints, it was assumed that the sleeves would be filled with concrete when the closure joints were cast. The sleeves located underneath the panels were not visible or accessible and, therefore, were left alone.

(a) (b)

Fig. 2-24. Top View of Circular Sleeves Embedded in FIBs for Fall Protection. (a) Grouted Sleeve Below Precast Deck Panel Shear Pocket, (b) Ungrouted Sleeve Below Panel Closure Joint.

The contractor kept the safety lines in place as long as possible, and he only removed them little by little, as each panel was erected. For future projects, it may be helpful to coordinate the sleeve spacing with the deck panel dimensions and spacings. Perhaps the sleeves could be placed only at closure joint locations. The sleeve spacing, however, is normally considered part of the contractor's means and methods, and the engineer should not interfere in this.

2.5 Related RFIs

RFI#2 from QA/Verification Inspectors at CDS Precast Yard, dated 10/2/2013:

1) Need a hook distance on Bars 5J, 5K and 5L. Do we need to change the length in the bar bending details?

Response: A standard 180 degree hook per Standard Index 21300 is required, and that is how the bar length was calculated.

2) For Bar 5L, can the step down be changed to 4'-0" instead of 4'-0 1/2" which is hard to make work in the field?

Response: Will discuss at progress meeting on 10/3/2013.

3) Does embedded plate C need to be galvanized? It gets encased in grout.

Response: At the Contractor's option.

4) 3" smooth surface, is it needed due to the fact that it will be covered by grout?

Response: The smooth surface is required for gluing the polyethylene form to the beams in order to contain the grout. See sheet B-13 for detail and Developmental Specification 404-5.

RFI #3, dated 10/8/2013:

There seems to be a conflict in dimensions on the FIB45 drawings between Plan Sheets B1-36, B1-37, B2-20, and the 2013 Design Standards Index 20045. Please reference the bending diagram for Bars 3D1, 3D2, and 3D3 in the index and the plan sheets. Note that the 4.5" dimension is on the top in the Design Standards but in the bottom in the Plan Sheets. Should the 4.5" dimension be in the top or the bottom.

Response: The design standard is correct. The 4.5" dimension should be the top dimension.

CHAPTER 3

PRECAST DECK PANELS (GENERAL)

3.1 Description

The Precast Deck Panels (non-prestressed and prestressed) are similar in function, appearance, and plan dimensions. They are full-depth, have widths of 43'-1" to match the bridge width, and are limited to an 8-ft length for shipping constraints and shear pocket spacing. The 8-ft-long Panel Type A (or D) is typical, and 7.5-ft- and 5-ft long Panel Types B (or E) and C (or F) are used on the ends of the spans (**Figure 3-1**). Types A, B, and C are non-prestressed, and D, E, and F are prestressed.

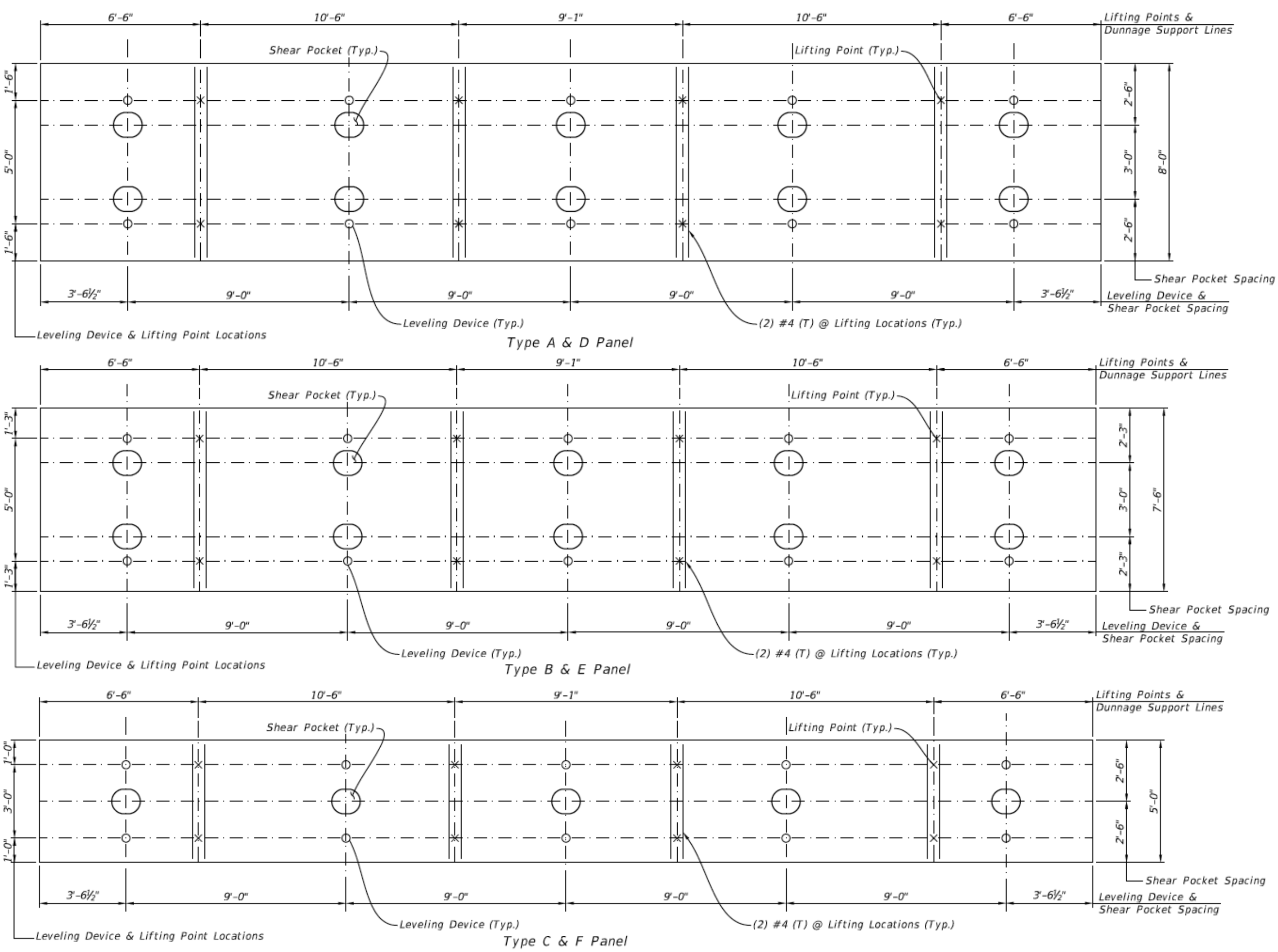


Fig. 3-1. Panel Types A – F, Plan View (from Design Plans)

The panels were to be erected transversely to the Florida-I Beams, with twelve-inch-wide transverse joints between each panel, except at the end bents and intermediate end bents. The joints were to be filled with concrete (**Figure 3-2**) after all panels were erected on the bridge and set to the desired elevation and after haunches and shear pockets were grouted. A keyway along the panel's short edges (**Figure 3-3**) and Bars 4C (**Figure 3-4**) that extend through the keyway both help to secure the cast-in-place closure joint concrete to the panel. The concrete was specified to contain a shrinkage-reducing admixture.

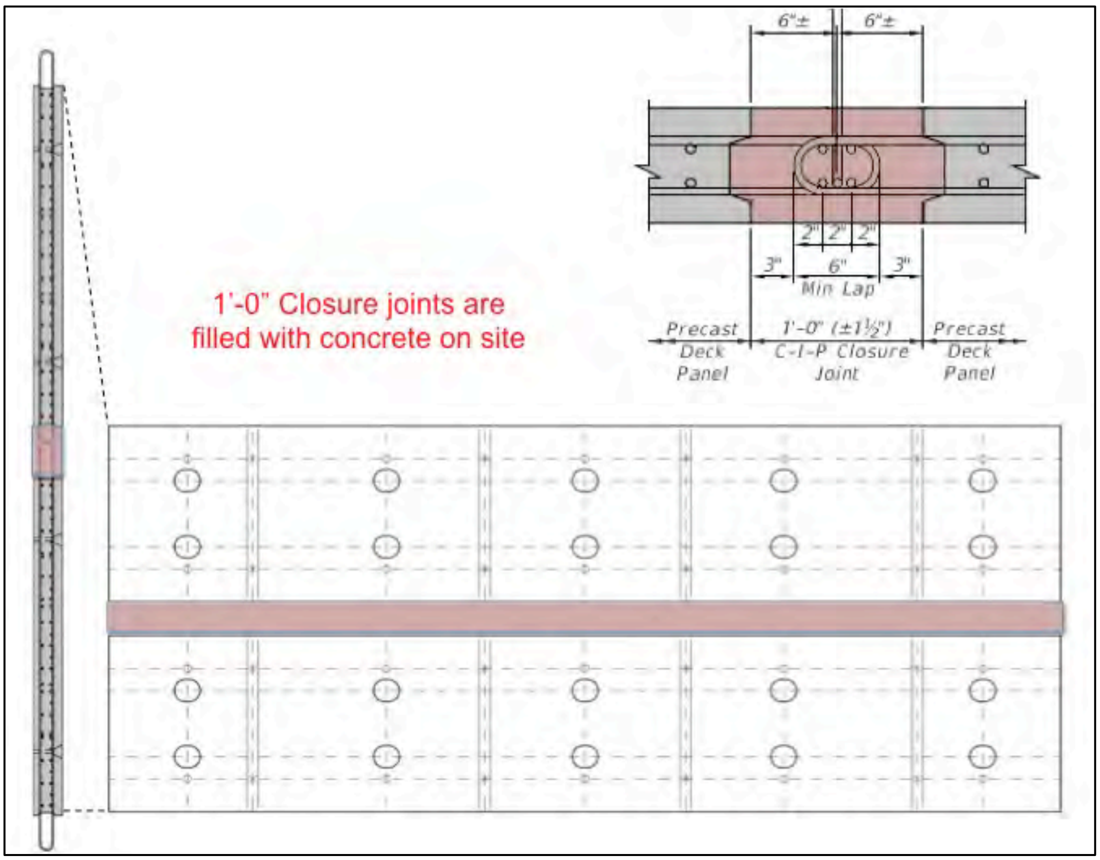


Fig. 3-2. Cast-in-Place Closure Joints (pink shaded areas) Between Precast Deck Panels

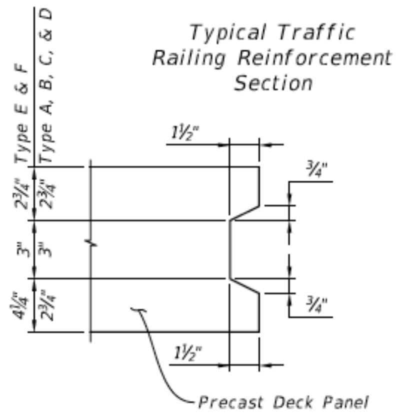


Fig. 3-3. Keyway Detail Along Panel's Edge for Closure Joint (from Design Plans)

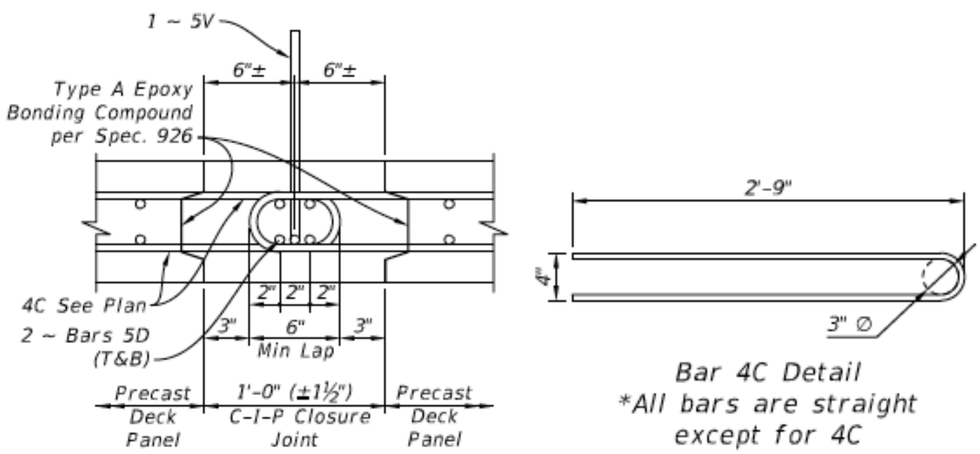
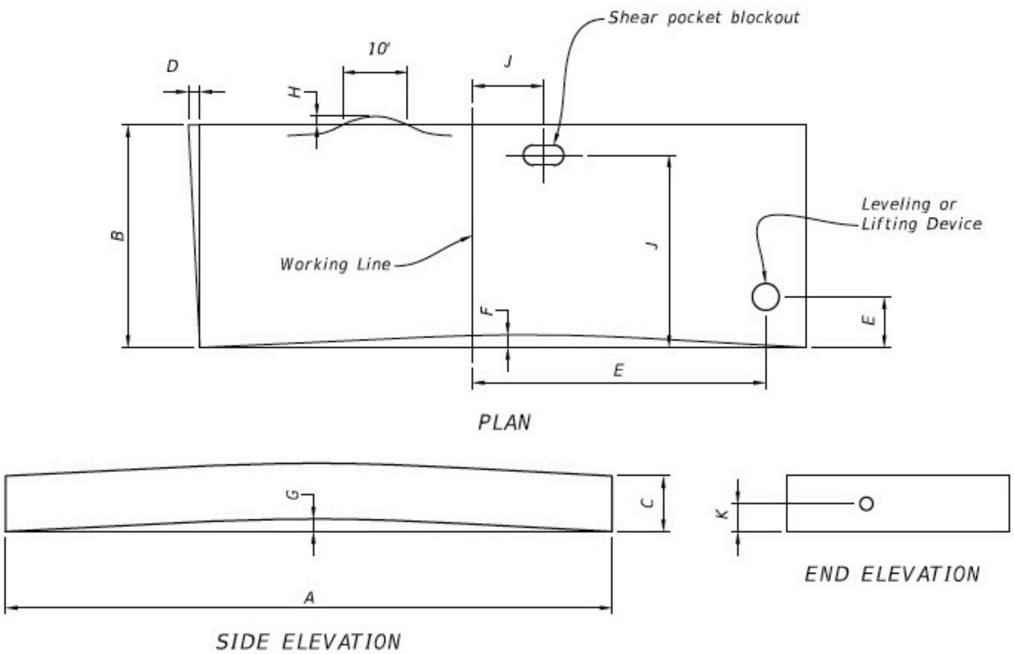


Fig. 3-4. Reinforcement that Extends through Keyway to Connect Panel with Closure Joint (from Design Plans)

Figure 3-5 shows tolerances for the Precast Deck Panels.

The panels were fabricated by CDS. **Figure 3-6** shows the panel raised from the casting bed, with the side forms being removed. The side forms have slots in them for Bars 4C to go through; each hole was covered with duct tape before concrete casting. In **Figure 3-7**, a casting bed for a non-prestressed panel is shown, along with some embedded items. Red arrows with dashed lines indicate the direction of the FIBs in relation to the panel. The top surface was roughened with a broom finish, except for the area under where the barrier would be placed which was raked. Along the FIB lines, the bottom of the panel was roughened, as shown in **Figure 3-8**. The panels have shear pockets, leveling devices, and lifting devices embedded in them (**Figure 3-9**); each of these items is discussed below.



ITEM	DESCRIPTION	TOLERANCE
A	LENGTH	± ¼"
B	WIDTH	± ¼"
C	DEPTH	± ⅛"
D	VARIATION FROM SPECIFIED PLAN SKEW	± ¼"
E	LOCATION OF LEVELING OR LIFTING DEVICE	±1"
F	SWEEP	± ⅜"
G	CAMBER VARIATION FROM DESIGN CAMBER	±1/16"
H	LOCAL SMOOTHNESS OF ANY SURFACE	± ⅛" IN 10 FEET
J	LOCATION OF SHEAR POCKET	± ½"
K	LOCATION OF PRESTRESSING STRAND (Where Applicable)	± ⅛"

Fig. 3-5. Precast Deck Panel Tolerances (from Design Plans)

The shear pockets, cast as voids in the panels, provide a space to connect (with grout, on site) the precast panel to the 5J and 5K reinforcing bars that protrude from the FIB top flange. These shear connectors are spaced every 3 ft, by way of the two (2) shear pockets in each panel plus the closure joints. **Figure 3-10** shows photos of the shear pocket forms and a cast panel, as well as schematics of the ten (10) shear pocket locations. Reinforcing details around a typical shear pocket are shown in **Figure 3-11**, and **Figure 3-12** shows the shear pocket dimensions, including a vertical taper so that the formers could be more easily removed from the precast product. The connection between a FIB and panel at a shear pocket is illustrated in **Figure 3-13**.

Fig. 3-6 Precast Deck Panel Being Removed from Forms. Keyway and Closure Joint Bars 4C Shown

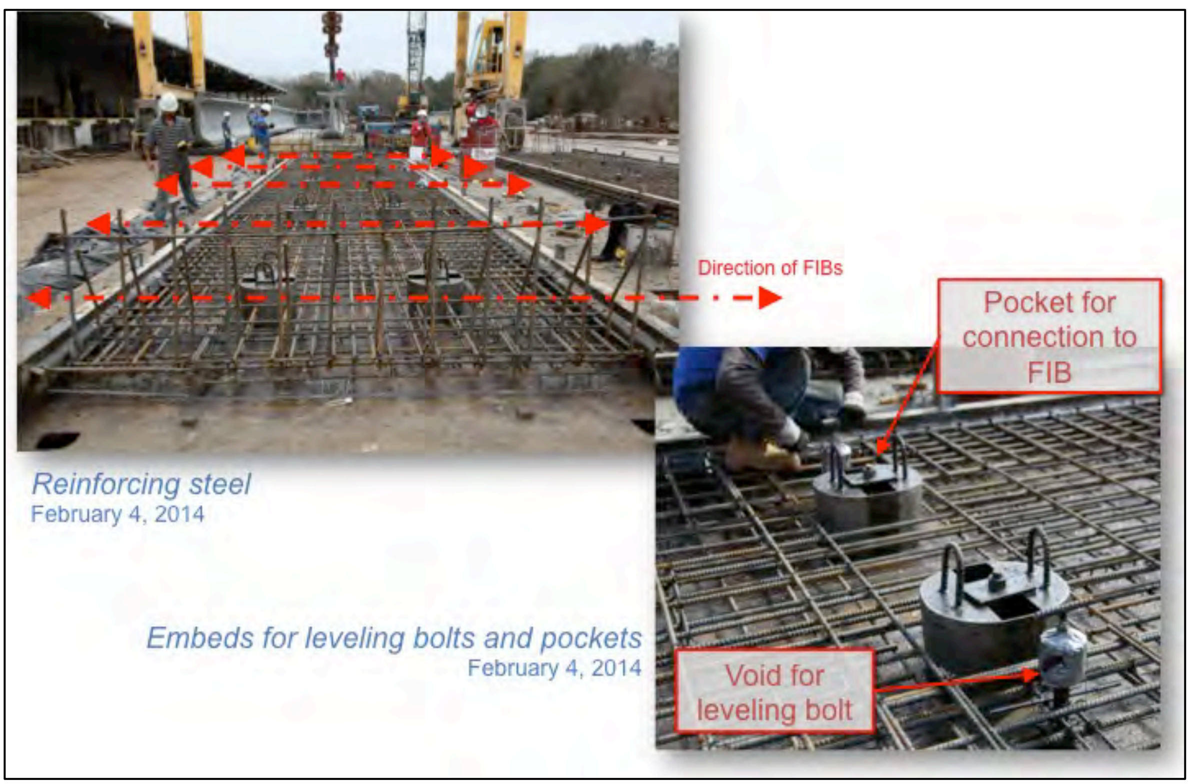


Fig. 3-7. Non-prestressed, Precast Deck Panel Casting Bed and Embedded Items

(a) (b)

Fig. 3-8. Roughening on Bottom of Precast Deck Panel, Along FIB Lines in Regions of Shear Pockets. (a) Strips on Forms, (b) View of Panel from Underneath

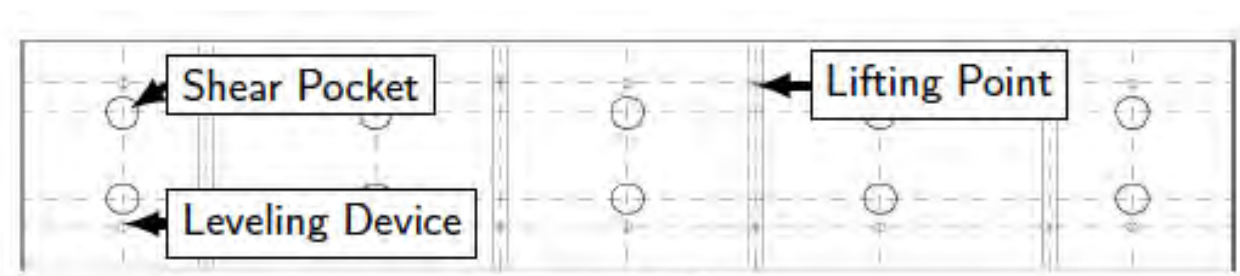


Fig. 3-9. Precast Deck Panel, Top View, Showing Shear Pockets, Leveling Devices, and Lifting Point Locations

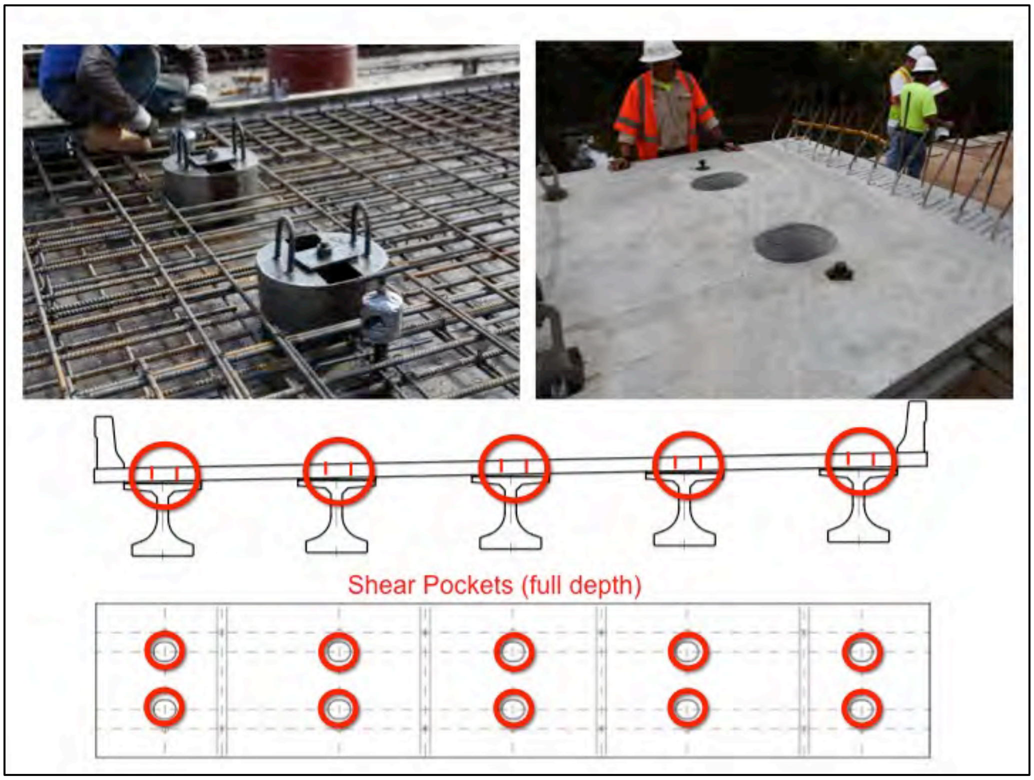


Fig. 3-10. Precast Deck Panel, Steel Forms for Shear Pockets, Cast Panel, and Shear Pocket Locations

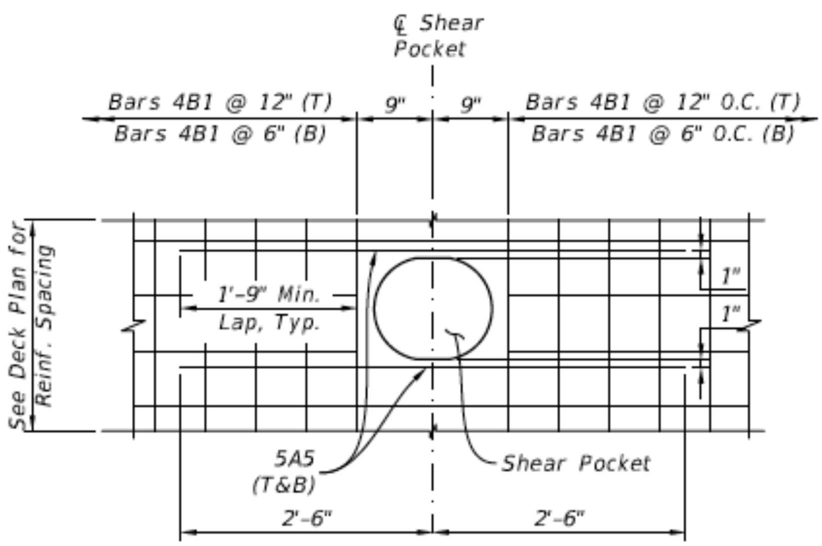


Fig. 3-11. Reinforcing Around Typical Shear Pocket (from Design Plans)

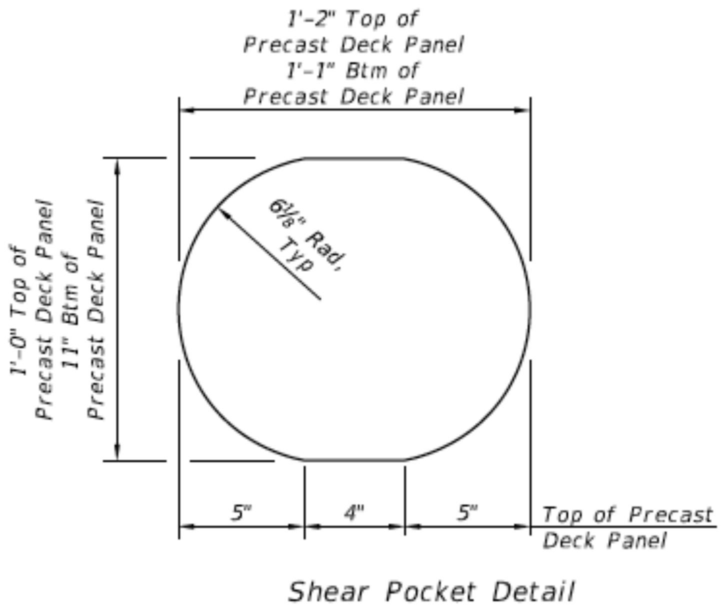


Fig. 3-12. Shear Pocket Dimensions (from Design Plans)

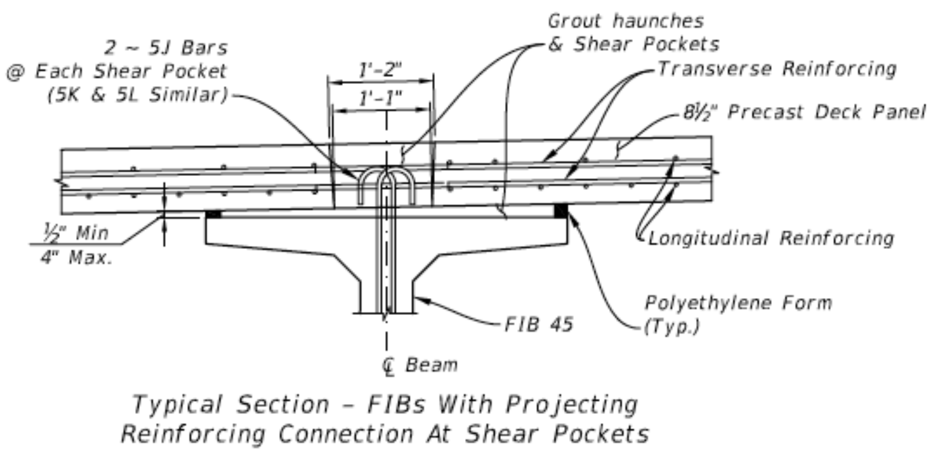


Fig. 3-13. Connection Between FIB and Precast Deck Panel at a Shear Pocket (from Design Plans)

Two (2) leveling devices per beam line, for a total of ten (10) per panel, were for temporarily supporting the panels on the FIBs during panel erection. The leveling device locations are illustrated in **Figure 3-14**, and details from the design plans are shown in **Figures 3-15 and 3-16**. The conical void at the top (Figure 3-16a) was changed by the precaster to a 4-in.-diameter void that was formed with corrugated plastic pipe; the reason for the change was so that the grout would be more secure in the "roughened" void when the grout was placed on site. The corrugated plastic pipe was removed after curing. Initially, the precaster unwittingly placed duct tape around the pipe, which reduced the desired roughening effect; this practice was eliminated for later panels, and the corrugation then worked as intended. The device ("nut") that the precaster embedded in the panel had to be compatible with the leveling bolts that the contractor would use. The contractor made each bolt by welding all-thread rod to a nut. The nut served as the bolt head, to be twisted to lower or raise the slab to the desired elevation when the panels were placed on the FIBs. They make contact with the Embedded Plate Assembly in the FIB top flange shown in Figures 2-14 to 2-16. The bolts were to be removed after grouting and closure joint pours were complete.

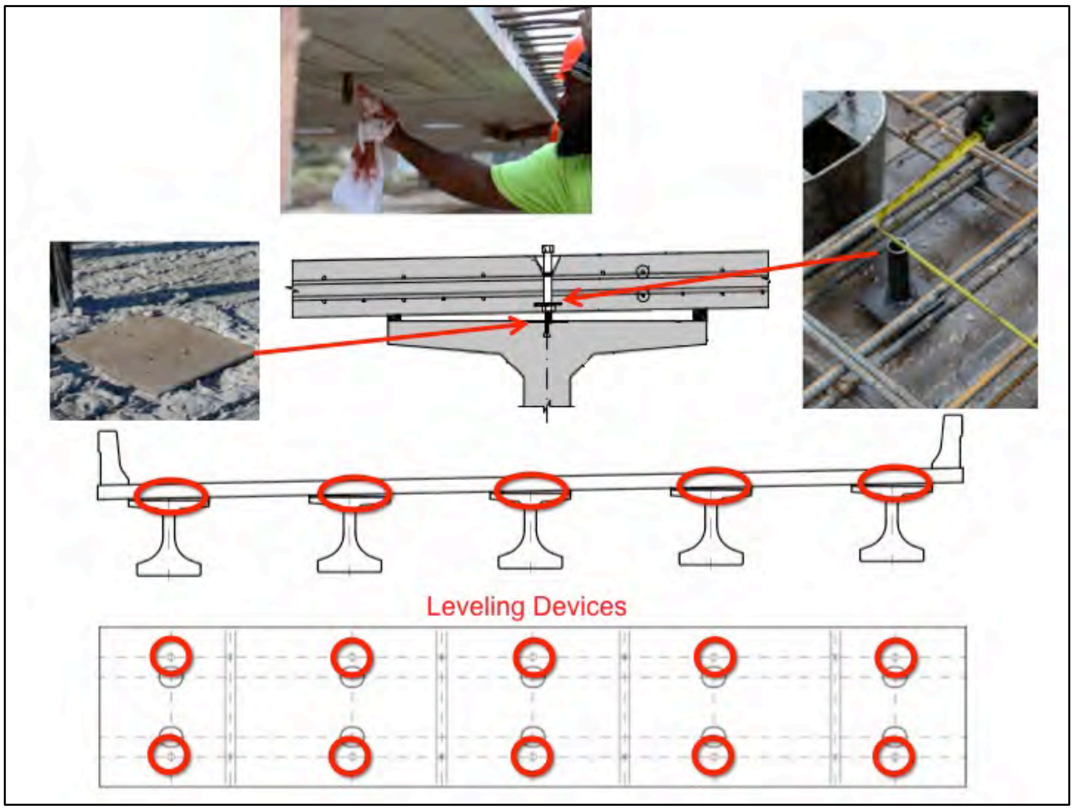


Fig. 3-14. Precast Deck Panel Leveling Device Locations and Photos

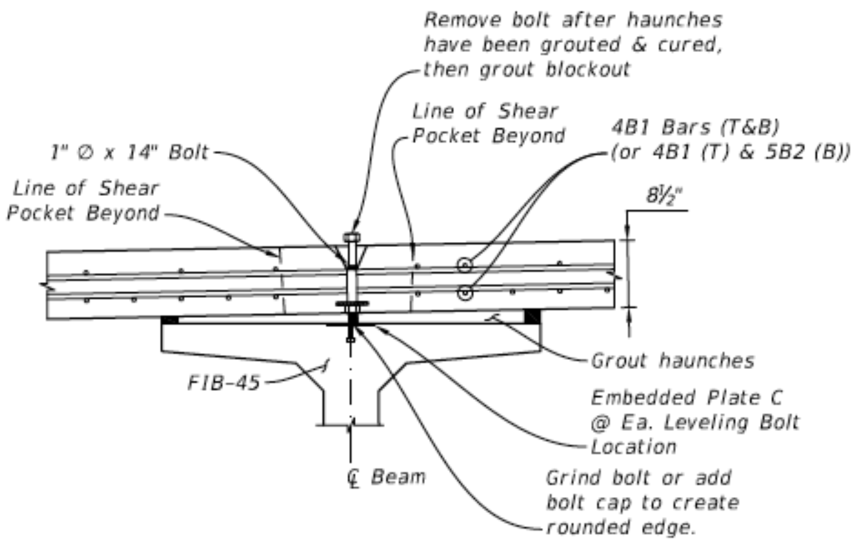
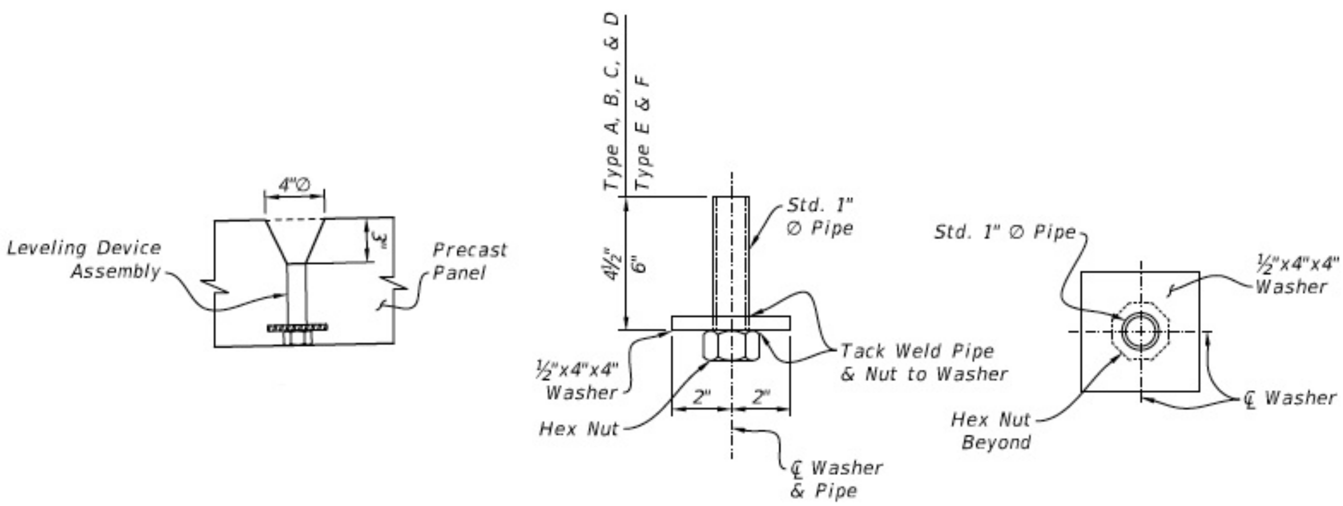


Fig. 3-15. Leveling Bolt Detail (from Design Plans)



(a) (b) (c)

Fig. 3-16. Leveling Device (a) Assembly, (b) Elevation View of 1" Pipe, (c) Plan View of Washer (from Design Plans)

Photos of the lifting devices and a schematic of their eight (8) locations are provided in **Figure 3-17**. MeadowBurke RL-26 T-bar anchors were embedded and recessed in the panels. The 5.25-in. size was used so that a 2-in. minimum cover could be maintained between the device and the bottom of panel. A MeadowBurke RL-27 plastic void former was used at the top of the anchor to provide space for a reusable ring clutch assembly, to which the lifting cables could be hooked.

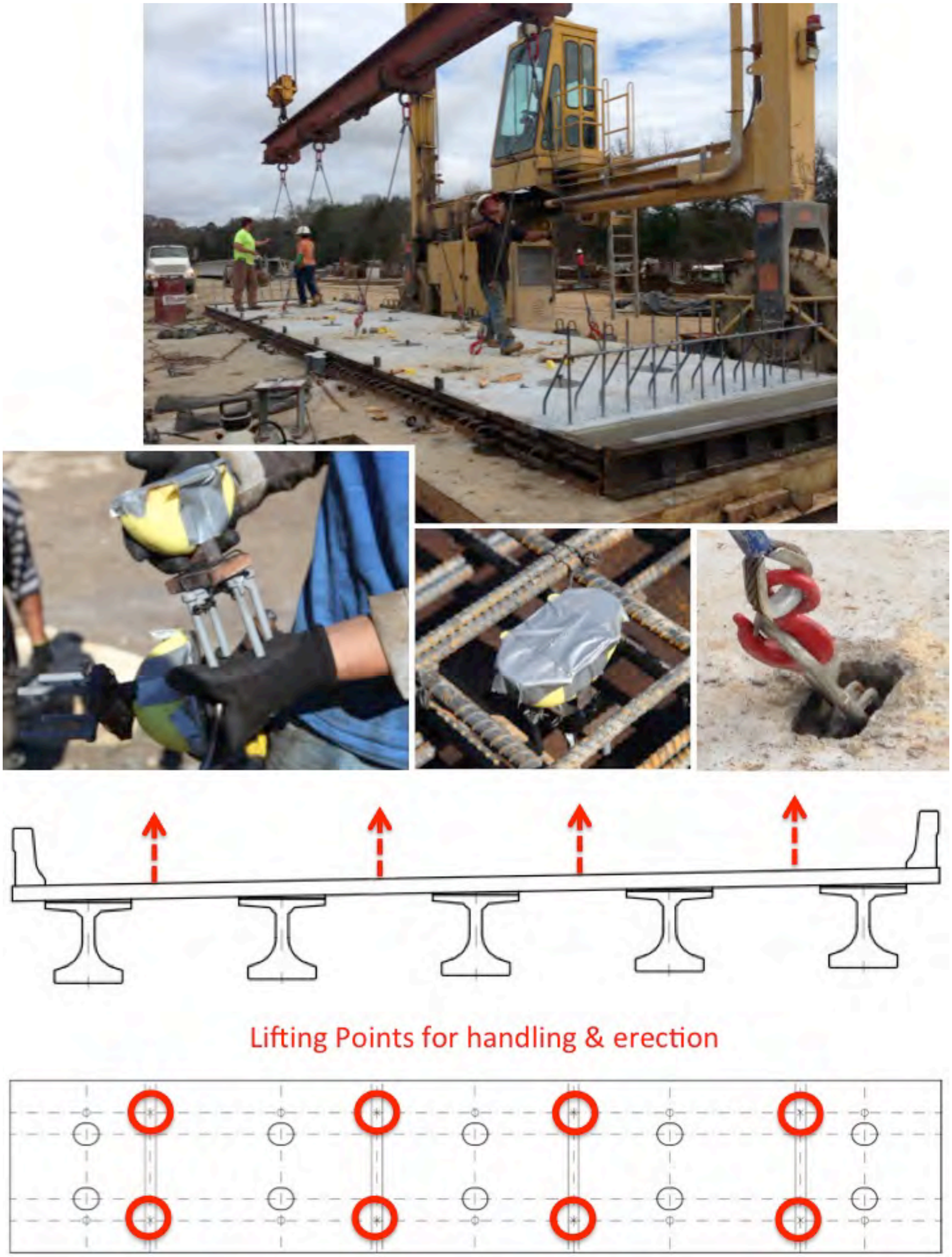


Fig. 3-17. Precast Deck Panel Lifting Devices

At the request of the contractor, the precaster placed coil inserts (Dayton Superior Product # 122709) in the bottom of the panels. An insert was placed on each end of the panel, at 10 in. from the panel's short edge and centered between the long edges (at 3 ft 9 in. from the long edge).

Per Developmental Specification Section 404, the panels could be shipped to the site at a minimum age of 28 days or when the concrete had reached 100% of the 28-day compressive strength, whichever occurred first, but not prior to the completion of a 72-hour curing period. Temporary stresses during handling, including impact loading, must also be within the acceptable limits for crack control. The Quality Control Manager at the precast plant had to certify the member before it was shipped. One cause for concern was that hauling trailer beds are usually cambered, which is ill-matched with the flat precast panel. Any cracks were marked with paint before the precast panel left the precast yard; after the panel arrived at the site, it was checked for additional cracks that may have occurred during transport. The access road at the bridge site was sloped, so the ground around the delivery truck axles was leveled so that the panel would be horizontal when lifted. This would help to ensure equal lifting among the multiple lifting devices.

To provide a mechanical connection between the cast-in-place barriers and precast deck panels, reinforcing steel was embedded in the panel along its short edges, as shown in **Figure 3-18**. Also shown is the rake-roughened panel top surface between the vertical legs of the barrier reinforcement, details for which are in **Figure 3-19**. A longitudinal drip groove (**Figure 3-20**) was precast along the underside of the panel's short edges, under the barrier.

3.2 Observations

<u>Spalls around lifting eye blockouts</u>:

Some areas around the lifting eye blockouts had small spalls. For Hurricane Creek EB Bridge, after the panels were erected, the inspector directed the contractor to saw around the blockouts to remove the spalled areas, and to fill the area with grout. For Little River

EB Bridge, repairs were made to the lifting eye blockouts in the casting yard, before the panels were shipped to the site.

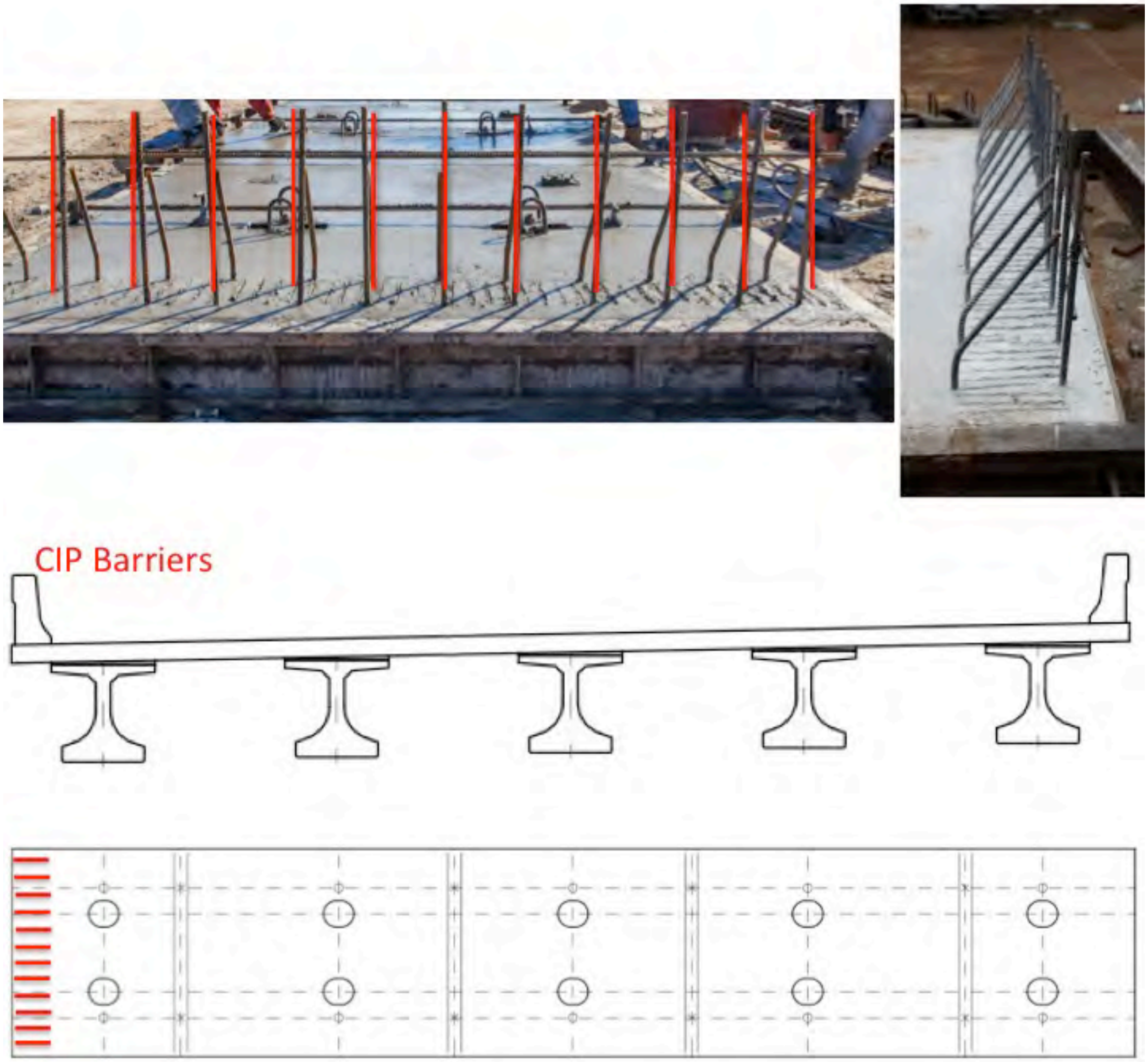


Fig. 3-18. Precast Deck Panel, Reinforcement for Cast-in-Place Barrier

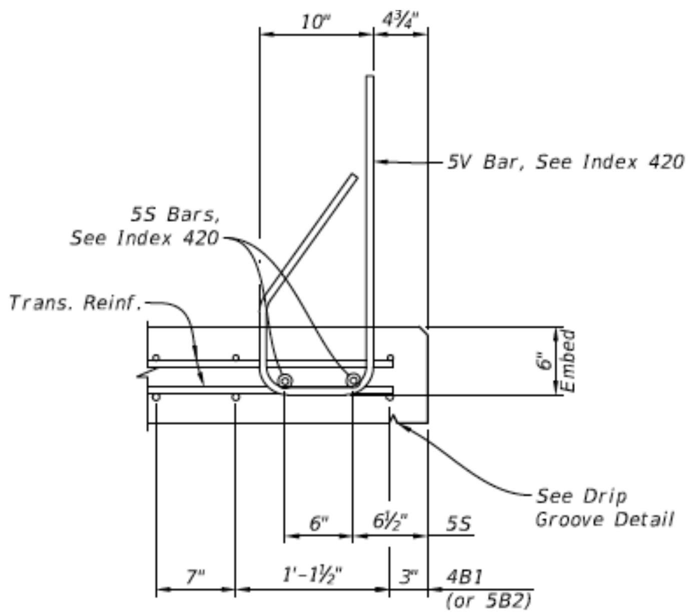


Fig. 3-19. Barrier Reinforcement Details (from Design Plans)

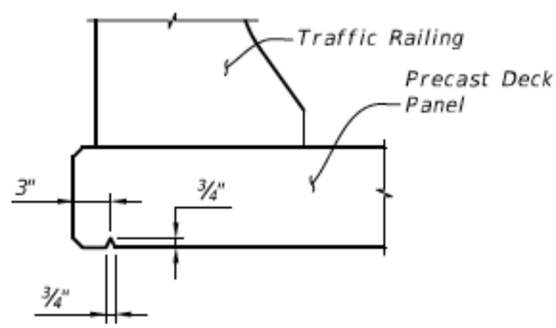


Fig. 3-20. Drip Groove Detail on Bottom of Panel, Under the Barrier (from Design Plans)

<u>Leveling bolt tolerance</u>:

When the precast deck panels were erected on the FIBs, some of the bolts did not make contact with the 6" x 6" Embedded Plate Assembly in the FIB top flange. Tolerances, the sweep of the FIB, and beam shortening caused by creep and shrinkage could contribute to a condition in which the bolts do not contact the plates. To resolve the immediate issue, the panel was shifted so that the bolt would make contact; this changed the adjacent closure joints dimensions slightly. For future projects, perhaps specify the amount of lean that is allowed for the leveling bolts that are embedded in the panels. Or specify that after the panels are erected in their proper position, the leveling bolts must come in contact with the steel plate, or a field correction must be submitted for approval.

3.3 Related RFIs

RFI #9, dated 1/13/2014:

Which way does the 1/8" amplitude run, transversely or longitudinally with the slab? Plan sheet B-12 shows a 1" diameter pipe for the leveling device assembly but shouldn't that diameter be 1-1/8" to accommodate the 1" diameter bolt shown on plan sheet B-13?

Response: 1/8" amplitude is transverse to bridge stationing. It should be parallel to the textured surface on the top of the beam when the panel is placed. The Pipe 1 Std. has an inside diameter of 1.05", slightly larger than the 1" bolt, but a Pipe 1-1/4" Std. can be used if preferred.

CHAPTER 4
NON-PRESTRESSED PRECAST DECK PANELS

4.1 Description

Eastbound bridges over Little River and Hurricane Creek have Non-prestressed Precast Deck Panels that contain mild reinforcement but no prestressing strands. (Plan Views showing the precast deck panel layout are shown in **Figures 4-1 and 4-2**.) Each span consists mostly of 8-ft-long Type A panels, with either 7.5-ft-long Type B or 5-ft-long Type C panels at the ends of the span. Either 12 or 13 precast panels comprise a span, depending on the span length. Panel Types A, B, and C are 8.5 in. thick, which includes a 0.5-in. thickness for sacrificial wear. The 0.5-in. sacrificial thickness was included in the dead load of the deck slab but omitted from the section properties used for design.

Photos of the twin, parallel casting beds are provided in **Figures 4-3 to 4-5**. **Figures 4-6 and 4-7** show the panels being transported around and stored in the casting yard. Soon after being cast, the panels experienced cracking, mostly along the short direction and particularly around the shear pockets, leveling bolts, and lifting devices (examples of cracks are shown in **Figures 4-8 and 4-9**). Each panel was marked with its designation (**Figure 4-10**). When the researchers documented panel cracks with photos, they started on the marked end and worked left to right and towards the unmarked edge.

4.2 Casting Data

Table 4-1 provides casting information about the non-prestressed precast deck panels, including panel # designations and casting, stripping, and shipping dates/ages.

Table 4-2 provides concrete strengths at stripping and at 28 days, as well as FDOT inspection notes. A synopsis of the Non-prestressed Precast Deck Panel data in Tables 4-1 and 4-2 is as follows.

Fig. 4-1. Little River EB Bridge, Non-prestressed Precast Deck Panel Layout (from Design Plans)

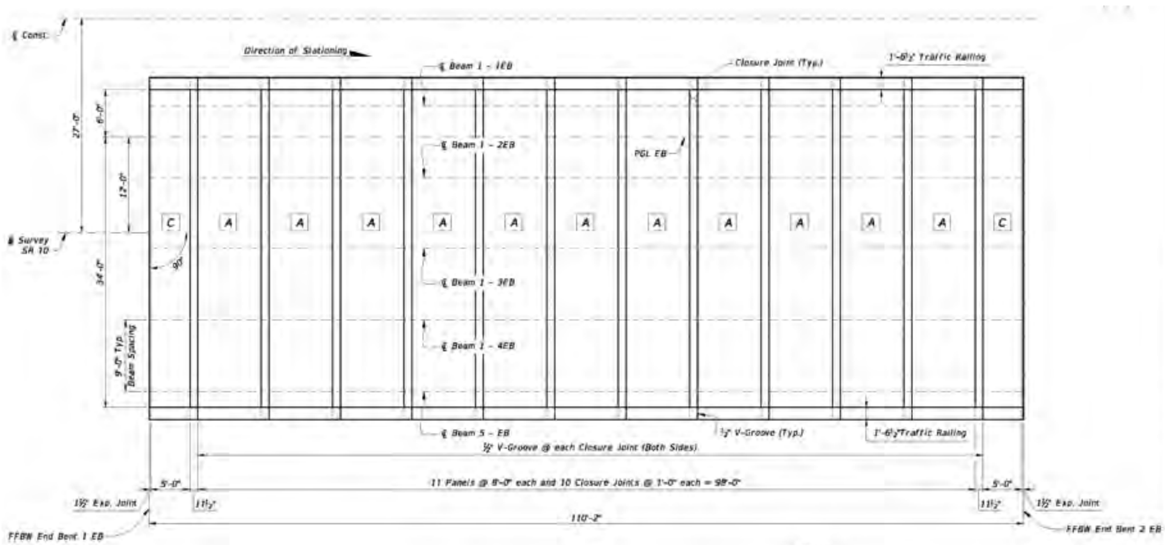


Fig. 4-2. Hurricane Creek EB Bridge, Non-prestressed Precast Deck Panel Layout (from Design Plans)

Fig. 4-3. Non-prestressed Precast Deck Panel, Twin Casting Beds

Fig. 4-4. Non-prestressed Precast Deck Panel, Reinforcing Steel

Fig. 4-5. Non-prestressed Precast Deck Panel, Reinforcement Around Shear Pocket, Positioning Leveling Bolt Embed

Fig. 4-6. Non-prestressed Precast Deck Panel Being Transported Around Casting Yard

Fig. 4-7. Storage of Non-prestressed Precast Deck Panels in Casting Yard


Fig. 4-8. Non-prestressed Precast Deck Panel , Example of Crack Adjacent to Shear Pocket


Fig. 4-9. Non-prestressed Precast Deck Panel, Example of Crack Adjacent to Lifting Loops

Fig. 4-10. Designations Marked on Precast Panel

Casting:

The 64 non-prestressed panels were cast in pairs in the casting bed. Casting occurred on:

- 2/2/2014 – 4/12/2014
- Over a period of 69 days (50 weekdays)

A period of about 11 days lapsed between casting the "A-type" panels and "B-" and "C-type" panels. Some of this time was needed to convert the forms from the longer "A" panel to the shorter "B" and "C" panels.

Stripping of the non-prestressed panels from their forms occurred shortly after casting (most within one (1) day), as summarized in **Table 4-3**.

Table 4-1. Non-prestressed Precast Deck Panel Designations and Casting/Shipping Dates

PANEL # *	CAST TOGETHER	CAST DATE	FORM STRIPPING DATE	AGE AT FORM STRIPPING (days)	SHIPPING DATE	AGE AT SHIPPING (days)
HC-A01		2/2/14	2/3/14	1	10/29/14	269
HC-A02		2/2/14	2/3/14	1	10/29/14	269
HC-A03		2/4/14	2/5/14	1	10/29/14	267
HC-A04		2/4/14	2/5/14	1	10/29/14	267
HC-A05		2/6/14	2/8/14	2	10/30/14	266
HC-A06		2/6/14	2/8/14	2	10/30/14	266
HC-A07		2/8/14	2/9/14	1	10/29/14	263
HC-A08		2/8/14	2/9/14	1	10/30/14	264
HC-A09		2/9/14	2/10/14	1	10/30/14	263
HC-A10		2/9/14	2/10/14	1	10/30/14	263
HC-A11		2/10/14	2/11/14	1	9/9/14	211
LR-A12		2/10/14	2/11/14	1	10/29/14	261
LR-A13		2/11/14	2/12/14	1	10/29/14	260
LR-A14		2/11/14	2/12/14	1	10/30/14	261
LR-A15		2/13/14	2/14/14	1	10/29/14	258
LR-A16		2/13/14	2/14/14	1	10/29/14	258
LR-A17		2/14/14	2/15/14	1	10/30/14	258
LR-A18		2/14/14	2/15/14	1	11/5/14	264
LR-A19		2/15/14	2/16/14	1	11/5/14	263
LR-A20		2/15/14	2/16/14	1	10/29/14	256
LR-A21		2/17/14	2/18/14	1	11/6/14	262
LR-A22		2/17/14	2/18/14	1	11/5/14	261
LR-A23		2/18/14	2/19/14	1	11/6/14	261
LR-A24		2/18/14	2/19/14	1	11/6/14	261
LR-A25		2/19/14	2/20/14	1	11/6/14	260
LR-A26		2/19/14	2/20/14	1	11/6/14	260
LR-A27		2/20/14	2/21/14	1	11/5/14	258
LR-A28		2/20/14	2/21/14	1	11/6/14	259
LR-A29		2/21/14	2/24/14	3	11/5/14	257
LR-A30		2/21/14	2/24/14	3	11/5/14	257
LR-A31		2/24/14	2/25/14	1	11/5/14	254
LR-A32		2/24/14	2/25/14	1	11/5/14	254
LR-A33		2/25/14	2/26/14	1	11/5/14	253
LR-A34		2/25/14	2/26/14	1	9/8/14	195
LR-A35		2/27/14	3/5/14	6	9/8/14	193
LR-A36		2/27/14	3/5/14	6	9/8/14	193
LR-A37		3/7/14	3/8/14	1	11/5/14	243
LR-A38		3/7/14	3/8/14	1	9/9/14	186
LR-A39		3/8/14	3/10/14	2	9/8/14	184
LR-A40		3/8/14	3/10/14	2	9/8/14	184
LR-A41		3/10/14	3/11/14	1	11/5/14	240
LR-A42		3/10/14	3/11/14	1	11/6/14	241
LR-A43		3/11/14	3/12/14	1	9/8/14	181
LR-A44		3/11/14	3/12/14	1	10/30/14	233
LR-A45		3/13/14	3/14/14	1	10/30/14	231
LR-A46		3/13/14	3/14/14	1	10/30/14	231
LR-A47		3/14/14	3/17/14	3	9/8/14	178
LR-A48		3/14/14	3/17/14	3	10/30/14	230
LR-A49		3/18/14	3/19/14	1	11/5/14	232
LR-A50		3/18/14	3/19/14	1	11/6/14	233
LR-A51		3/19/14	3/20/14	1	11/5/14	231
LR-A52		3/19/14	3/20/14	1	11/5/14	231
LR-A53		3/20/14	3/21/14	1	9/8/14	172
LR-A54		3/20/14	3/21/14	1	9/9/14	173
B01		4/1/14	4/2/14	1	10/29/14	211
C01		4/1/14	4/2/14	1	10/30/14	212
C02		4/2/14	4/3/14	1	10/30/14	211
C03		4/4/14	4/5/14	1	11/19/14	229
B02		4/4/14	Sat in forms 3 months before lifting	--	10/29/14	208
C04		4/8/14	4/9/14	1	11/5/14	211
C05		4/9/14	4/10/14	1	9/8/14	152
C06		4/10/14	4/11/14	1	9/9/14	152
C07		4/11/14	4/12/14	1	11/5/14	208
C08		4/12/14	4/14/14	2	11/5/14	207

* Panels were switched around for erection, so Designation HC or LR does not indicate which bridge the panel was constructed on.

Panels were constructed at same time in two (2) side-by-side casting beds.

Table 4-2. Non-prestressed Precast Deck Panel Concrete Strengths and Inspection Notes

PANEL # *	CAST TOGETHER	DESIGN MIX **	CDS LOT #	CDS CONCRETE STRENGTH AT STRIPPING (psi)	CDS CONCRETE STRENGTH AT 28 DAYS (psi)	FDOT SAMPLE # FOR VERIFICATION	FDOT CONCRETE STRENGTH AT 28 DAYS (psi)	FDOT SPECIAL NOTES ("post-pour" observations)
HC-A01		03-1721	GDS001AQ	8 690	14 670			Cracks
HC-A02		03-1721						Cracks
HC-A03		03-1722	GDS001Q	6 170	15 290	GDS001V	12 170	Cracks
HC-A04		03-1722	GDS001QR		11 520	GDS001VR	12 400	
HC-A05		03-1722	GDS002Q	8 040	12 520			
HC-A06		03-1722						
HC-A07		03-1722	GDS003Q	5 160	12 860			Cracks
HC-A08		03-1722						
HC-A09		03-1722	GDS004Q	5 940	12 080			
HC-A10		03-1722						Cracks
HC-A11		03-1722	GDS005Q	4 850	12 750			Cracks
LR-A12		03-1722						Cracks
LR-A13		03-1722	GDS006Q	6 380	11 840			
LR-A14		03-1722						Cracks
LR-A15		03-1722	GDS007Q	5 030	13 100			Cracks
LR-A16		03-1722						Cracks
LR-A17		03-1722	GDS008Q	5 090	11 940			Cracks/Spall
LR-A18		03-1722						Cracks
LR-A19		03-1722	GDS009Q	6 770	12 310			
LR-A20		03-1722						Cracks
LR-A21		03-1722	GDS010Q	5 620	11 840			Cracks
LR-A22		03-1722						
LR-A23		03-1722	GDS011Q	4 870	11 590	GDS011V	11 500	Cracks
LR-A24		03-1722						Cracks
LR-A25		03-1722	GDS012Q	5 490	12 810			Cracks
LR-A26		03-1722						
LR-A27		03-1722	GDS013Q	5 400	12 810			Cracks
LR-A28		03-1722						
LR-A29		03-1722	GDS014Q	8 730	12 990			Cracks
LR-A30		03-1722						Cracks
LR-A31		03-1722	GDS015Q	5 490	12 150	GDS015V	9 980	Cracks
LR-A32		03-1722	GDS015QR		12 040	GDS015VR	11 270	Cracks
LR-A33		03-1722	GDS016Q	5 460	12 900			Cracks
LR-A34		03-1722						
LR-A35		03-1722	GDS017Q	10 980				
LR-A36		03-1722						
LR-A37		03-1722	GDS018Q	5 830	12 080	GDS018V	12 540	Cracks
LR-A38		03-1722						Cracks
LR-A39		03-1722	GDS019Q	8 500	11 690			Cracks
LR-A40		03-1722						
LR-A41		03-1722	GDS020Q	7 120	13 140			Cracks
LR-A42		03-1722						Cracks
LR-A43		03-1722	GDS021Q	6 120	12 340	GDS021V	11 530	Cracks
LR-A44		03-1722						Cracks
LR-A45		03-1722	GDS022Q	7 100	13 530			Cracks
LR-A46		03-1722						Cracks
LR-A47		03-1722	GDS023Q	9 450	14 270			Cracks
LR-A48		03-1722						Cracks
LR-A49		03-1722	GDS024Q	5 480	12 250			Cracks
LR-A50		03-1722						Cracks
LR-A51		03-1722	GDS025Q	6 200	12 730	GDS025V	12 580	Cracks
LR-A52		03-1722						Cracks
LR-A53		03-1722	GDS026Q	6 100	12 600			Cracks
LR-A54		03-1722						Cracks
B01		03-1722	GDS027Q	7 440	13 100			Cracks
C01		03-1722						Cracks
C02		03-1722	GDS028Q	6 470	13 150			Cracks in mid area
								Non-compliance Report
C03		03-1722	GDS029Q	5 400	13 240			LIFTING AREA
								#7 bars replaced 6A1
B02		03-1722						Top Mat, RFI #13
C04		03-1722	GDS030Q	6 400	12 670	GDS030V	11 980	Cracks
C05		03-1722	GDS031Q	7 040	12 960			Cracks
C06		03-1768	GADS001Q	6 530	13 500	GADS001V	10 370	Cracks
			GADS001QR		11 020	GADS001VR	11 220	
C07		03-1768	GADS002Q	6 500	12 530			
C08		03-1768	GADS003Q	8 090	11 860			Cracks

* Panels were switched around for erection, so Designation HC or LR does not indicate which bridge the panel was constructed on

** MIX 03-1721 is FDOT Class V (6500 psi). MIXES 03-1722 and 03-1768 are FDOT Class VI (8500 psi).

Panels were constructed at same time in two (2) side-by-side casting beds.

Table 4-3. Non-prestressed Precast Deck Panels, Stripping and Number of Days After Casting

# of Days Panel was Stripped after Casting	# of Non-prestressed Panels
1	52
2	5
3	4
6	2
3 months*	1

* Panel sat in forms for 3 months before lifting, to determine if cracking could be reduced by allowing extra curing. The extra curing did not have a measurable impact on the cracking.

Materials:

The design plans specified Class II (Bridge Deck) (4500 psi) concrete for non-prestressed precast deck panels. The first two panels (A01 and A02) were cast with MIX 03-1721, which is Class V (6500 psi). The last two panels (C06, C07, and C08) were cast with MIX 03-1768, which is Class VI (8500 psi). All other non-prestressed panels were cast with MIX 03-1722, which is FDOT Class VI (8500 psi). **Table 4-4** summarizes the minimum, maximum, and average concrete strengths measured at stripping and at 28 days.

Table 4-4. Non-prestressed Precast Deck Panels, Measured Concrete Strengths (Minimum, Maximum, and Average)

Non-prestressed Panel Concrete Strength	At Stripping (psi)	At 28 days (psi)
Minimum	4 850	11 020
Maximum	10 980	15 290
Average	6 569	12 667

Erection:

For Hurricane Creek EB Bridge, the 13 panels were erected on 9/8/2014 – 9/9/2014. The age on erection day ranged from 152 to 211 days, with an average of 181 days. For Little River EB Bridge, the panels were erected on 10/29/2014 – 11/6/2014. The age on erection day ranged from 207 to 269 days, with an average of 248 days.

4.3 Cracking

Several of the Type A (non-prestressed) panels experienced cracking while at the precast yard. Of the 54 Type A panels, 30 had cracks as of May 2014. This required an Engineering Analysis and Report (EAR) to determine the extent of cracking and whether or not the panels were acceptable and/or needed to be repaired. Using FDOT Specification 400-21 Disposition of Cracked Concrete, the contractor's engineer classified the cracking as non-structural in nature. He identified five (5) typical cracking patterns and determined which pattern had occurred in each panel. Cracks occurred mostly along the short direction (longitudinally along the bridge) around the shear pockets and/or lifting lugs (see Figures 4-8 and 4-9). Table 2 in Specification 400-21 was used to determine if repairs were needed and, if so, the type of repair method that would be required. This determination was based on cracking significance, crack width, elevation above Mean High Water, and environment category (i.e., aggressiveness).

The contractor's engineer determined the cracking significance, which is a percentage of the total cracked surface area to the total surface area. The cracking significance was then rated in one of four (4) categories per Specification 400-21: Isolated, Occasional, Moderate, or Severe. The engineer determined that all but one of the cracked panels fell into the Isolated category, where less than 0.005% of the concrete area was cracked. One (1) panel, HC A10, was rated in the Occasional category, since 0.0070% of the concrete area was cracked. None of the cracks in any of the panels was wider than 0.016 inches, so no treatment was required per Specification 400-21. The engineer, however, recommended that the cracks be treated after being placed on the bridge and after all dead loads were applied.

After receiving updated crack maps in August 2014, the contractor's engineer revised his recommendation and concluded that the non-prestressed panels should not be installed and that prestressed panels be used instead. His conclusion was based on the updated crack maps and the fact that more cracks had occurred since his original analysis in May 2014. The change in the crack maps from May to August was "radical"; only three (3) panels had no cracks as of August.

In September 2014, the contractor's engineer revised his report; he recommended that the non-prestressed panels be installed as they were and that any repair be done after grinding and prior to grooving. This revised report came about after some clarifications were made by FDOT as to the test nature of the project, i.e., the behavior of the non-prestressed panels was being compared to the prestressed panels. The engineer's understanding was that the structural adequacy or safety of the panels was not being questioned, and that FDOT recognized that if the non-prestressed panels did not perform well in the future then they would be replaced. The cracking significance as of September 2014 is summarized in **Table 4-5**, which also shows that only one (1) panel had a crack that would require treatment (by epoxy injection or methyl methacrylate); the crack was 12 in. long and 0.018 in. wide.

Table 4-5. Cracking Significance of Non-prestressed Precast Deck Panels

NUMBER OF PANELS with Cracking and with Crack Widths ≥	PANEL TYPE A (54 total)	PANEL TYPE B (2 total)	PANEL TYPE C (8 total)
Cracking Significance:			
NO Cracks	3	1	0
ISOLATED*	44	0	6
OCCASIONAL**	7	1	2
Crack Width (in.):			
0.012	15	1	4
0.014	0	0	2
0.016	7	0	1
0.018	0	0	1***

*ISOLATED = Area of Cracking/Area of Panel < 0.005%
**OCCASIONAL = 0.005% ≤ Area of Cracking/Area of Panel < 0.017%
***Crack required treatment with epoxy injection or methyl methacrylate.

4.4 Related RFIs

RFI#13, dated 4/2/2014:

CDS has requested permission to use a #7 bar in place of the 6A1 in the TOP MAT OF STEEL only for this last Type B plank. CDS will consolidate the concrete as they have been with internal vibration. The #7 bar will be a one to one substitution for the #6 bar.

Response: It is acceptable to use #7 bars in place of the 6A1 (one to one substitution) in the TOP MAT OF STEEL only for the last Type B plank.

CHAPTER 5
PRESTRESSED PRECAST DECK PANELS

5.1 Description

Westbound bridges for Little River and Hurricane Creek have Prestressed Precast Deck Panels that contain mild reinforcement and prestressing strands. (Plan Views showing the precast deck panel layout are shown in **Figures 5-1 and 5-2**.) Each span consists mostly of 8-ft-long Type D panels, with either 7.5-ft-long Type E or 5-ft-long Type F panels at the ends of the span. Either 12 or 13 precast panels comprise a span, depending on the span length. Panel Type D is 8.5 in. thick, and Types E and F are 10 in. thick; each includes a 0.5-in. thickness for sacrificial wear. The 0.5-in. sacrificial thickness was included in the dead load of the deck slab but omitted from the section properties used for design. Types D, E, and F, respectively, contain 10, 12, and 8 prestressing strands that are in two (2) rows. For Type D, the strands are symmetric about the horizontal, centroidal axis of the cross section, excluding the 0.5-in. sacrificial thickness; for Types E and F, the strands are within 0.125-in. of being symmetric (**Figure 5-3**).

Photos of the casting bed are provided in **Figures 5-4 and 5-5**. Two panels were constructed at a time, with one set of prestressing strands extending along the length of both panels. Additional photos of the forms and stressing end are provided in **Figures 5-6 to 5-8**. Reinforcing steel and embedments for shear pockets and leveling bolts are shown in **Figures 5-9 to 5-12**. The panels were stored (not stacked) in the casting yard (**Figures 5-13 and 5-14**).

5.2 Casting Data

Table 5-1 provides casting information about the Prestressed Precast Deck Panels, including panel # designations and casting, prestress release, and shipping dates/ages.

Table 5-2 provides concrete strengths at prestress release and at 28 days, as well as FDOT inspection notes. A synopsis on the Prestressed Precast Deck Panel data in Tables 5-1 and 5-2 is as follows.

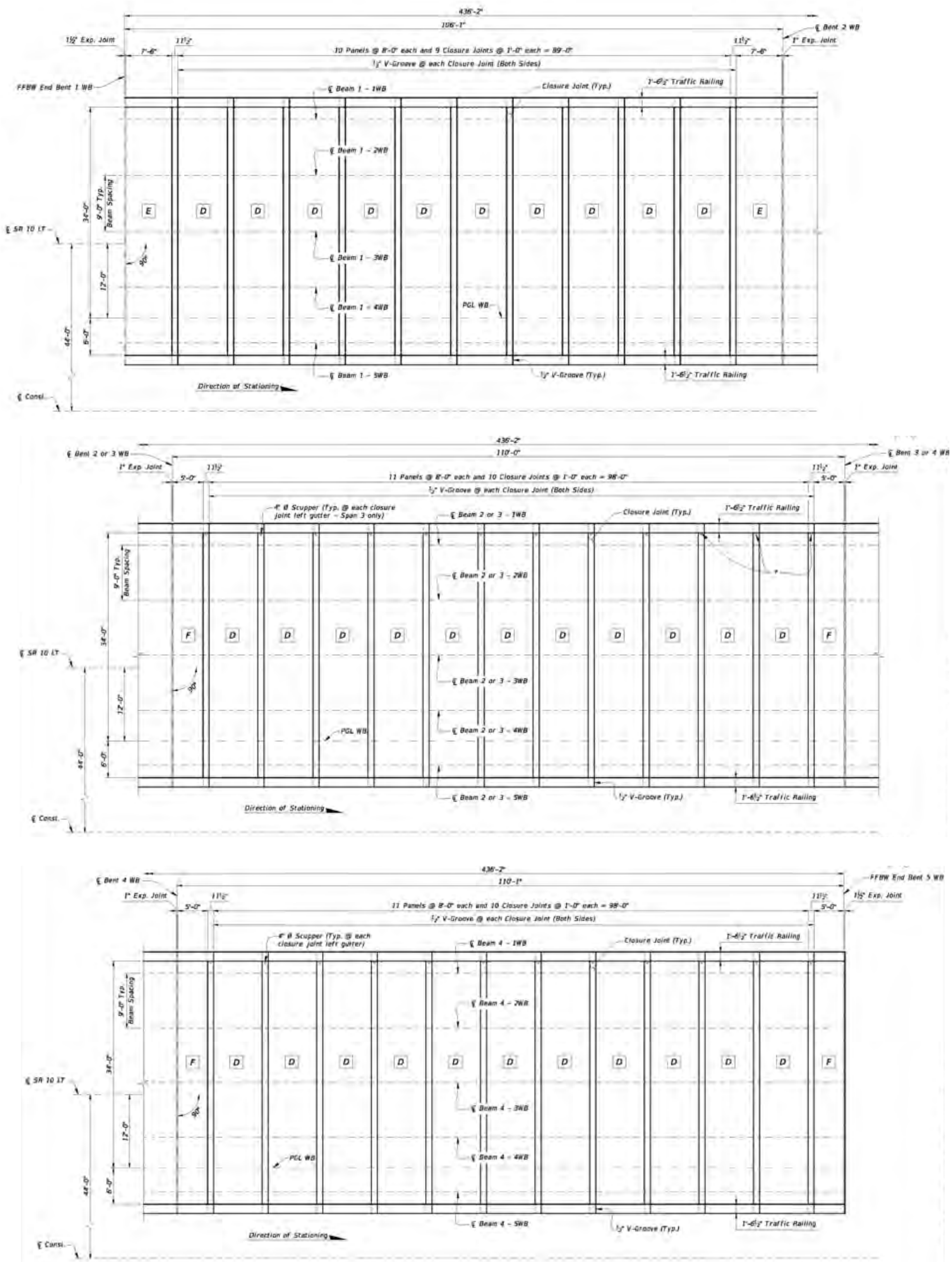


Fig. 5-1. Little River WB Bridge, Prestressed Precast Deck Panel Layout (from Design Plans)

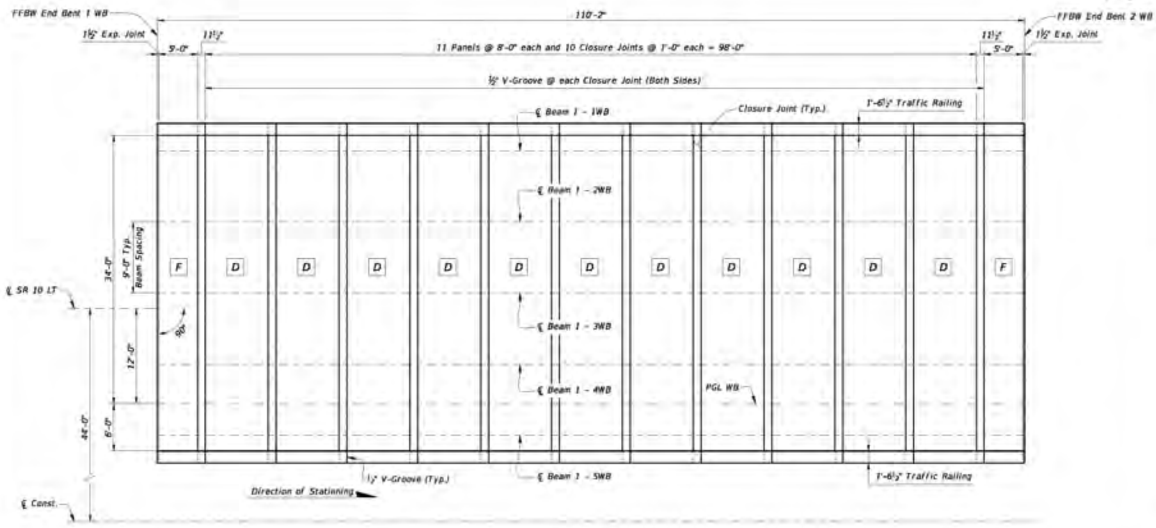
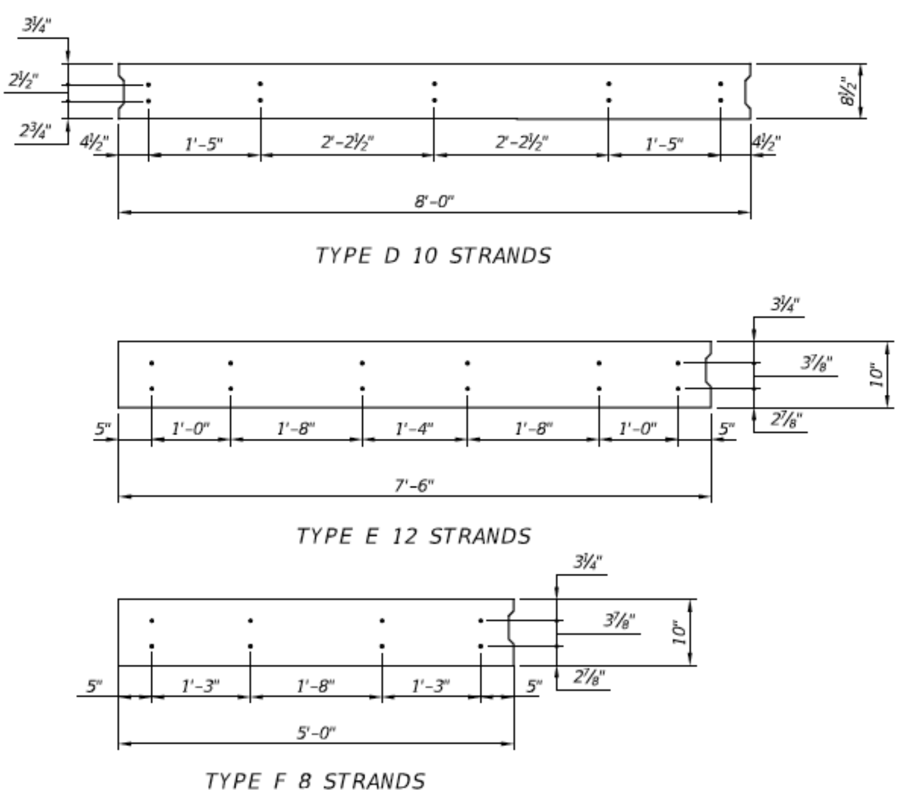


Fig. 5-2. Hurricane Creek WB Bridge, Prestressed Precast Deck Panel Layout (from Design Plans)



STRAND DESCRIPTION: Use 0.5" Diameter, Grade 270, Low Relaxation Strands stressed at 31.0 kips each. Area per strand equals 0.153 sq. in.

Fig. 5-3. Prestressed Precast Deck Panel Cross Section and Prestressing Strand Layout (from Design Plans)

Fig. 5-4. Prestressed Precast Deck Panel Casting Bed, Strands Extending in Foreground

Fig. 5-5. Prestressed Precast Deck Panel Casting Bed, Barrier Reinforcement in Foreground

Fig. 5-6. Prestressed Precast Deck Panel Casting Bed, Showing Prestressing Jack Hanging from Frame

Fig. 5-7. Prestressed Precast Deck Panel Casting Bed, Stressing End and Strand Pattern

Fig. 5-8. Prestressed Precast Deck Panel Casting Bed, Oblique View of Forms

Fig. 5-9. Prestressed Precast Deck Panel, Reinforcing Steel Along Long Edge

Fig. 5-10. Prestressed Precast Deck Panel, Reinforcing Steel Along Short Edge

Fig. 5-11. Prestressed Precast Deck Panel, Reinforcement in Vicinity of Future Barrier

Fig. 5-12. Prestressed Precast Deck Panel, Shear Pocket and Leveling Bolt Embeds

Fig. 5-13. Prestressed Precast Deck Panel, End View

Fig. 5-14. Prestressed Precast Deck Panel Casting Bed, Barrier Reinforcement in Foreground

Casting:

The 64 prestressed panels were cast in pairs in the casting bed. Casting occurred on:

- 3/7/2014 – 3/30/2014
- Over a period of 23 days (16 weekdays)

and again on:

- 7/17/2014 – 8/22/2014
- Over a period of 36 days (27 weekdays)

A period of about 4 days lapsed between casting the "D-type" panels and "E-type" panels, and another 3 days between "E-" and "F-type" panels. This time was needed to convert the forms from the longer, thinner "D" panel to the shorter, thicker "E" and "F" panels.

Prestress release occurred shortly after casting (most within one (1) day), as summarized in **Table 5-3**.

Table 5-1. Prestressed Precast Deck Panel Designations and Casting/Shipping Dates

PANEL # *	CAST TOGETHER	CAST DATE	RELEASE DATE	AGE AT RELEASE (days)	SHIPPING DATE	AGE AT SHIPPING (days)
HC-D01		3/7/14	3/8/14	1	6/26/15	476
HC-D02		3/7/14	3/8/14	1	6/26/15	476
HC-D03		3/9/14	3/11/14	2	6/26/15	474
HC-D04		3/9/14	3/11/14	2	6/26/15	474
HC-D05		3/12/14	3/13/14	1	6/26/15	471
HC-D06		3/12/14	3/13/14	1	6/26/15	471
HC-D07		3/14/14	3/18/14	4	6/26/15	469
HC-D08		3/14/14	3/18/14	4	9/18/15	553
HC-D09		3/21/14	3/22/14	1	9/18/15	546
HC-D10		3/21/14	3/22/14	1	9/18/15	546
HC-D11		3/22/14	3/24/14	2	6/26/15	461
LR-D01		3/22/14	3/24/14	2	6/26/15	461
LR-D02		3/24/14	3/25/14	1	9/18/15	543
LR-D03		3/24/14	3/25/14	1	9/18/15	543
LR-D04		3/25/14	3/26/14	1	9/18/15	542
LR-D05		3/25/14	3/26/14	1	9/18/15	542
LR-D06		3/26/14	3/27/14	1	8/27/15	519
LR-D07		3/26/14	3/27/14	1	8/27/15	519
LR-D08		3/27/14	3/29/14	2	8/31/15	522
LR-D09		3/27/14	3/29/14	2	8/31/15	522
LR-D10		3/30/14	3/31/14	1	6/26/15	453
LR-D11		3/30/14	3/31/14	1	6/26/15	453
LR-D12		7/17/14	7/18/14	1	8/31/15	410
LR-D13		7/17/14	7/18/14	1	9/18/15	428
LR-D14		7/18/14	7/21/14	3	8/31/15	409
LR-D15		7/18/14	7/21/14	3	8/31/15	409
LR-D16		7/21/14	7/22/14	1	9/18/15	424
LR-D17		7/21/14	7/22/14	1	9/18/15	424
LR-D18		7/22/14	7/23/14	1	8/31/15	405
LR-D19		7/22/14	7/23/14	1	9/18/15	423
LR-D20		7/23/14	7/24/14	1	9/18/15	422
LR-D21		7/23/14	7/24/14	1	9/18/15	422
LR-D22		7/24/14	7/25/14 ^	1	9/18/15	421
LR-D23		7/24/14	7/25/14 ^	1	8/27/15	399
LR-D24		7/25/14	7/28/14	3	8/27/15	398
LR-D25		7/25/14	7/28/14	3	8/27/15	398
LR-D26		7/28/14	7/29/14	1	9/18/15	417
LR-D27		7/28/14	7/29/14	1	9/18/15	417
LR-D28		7/29/14	7/30/14	1	9/18/15	416
LR-D29		7/29/14	7/30/14	1	9/18/15	416
LR-D30		7/30/14	7/31/14	1	9/18/15	415
LR-D31		7/30/14	7/31/14	1	8/27/15	393
LR-D32		7/31/14	8/1/14	1	8/27/15	392
LR-D33		7/31/14	8/1/14	1	8/27/15	392
LR-D34		8/4/14	8/5/14	1	8/27/15	388
LR-D35		8/4/14	8/5/14	1	8/27/15	388
LR-D36		8/5/14	8/6/14	1	8/31/15	391
LR-D37		8/5/14	8/6/14	1	8/31/15	391
LR-D38		8/6/14	8/7/14	1	8/31/15	390
LR-D39		8/6/14	8/7/14	1	8/27/15	386
LR-D40		8/7/14	8/8/14	1	9/18/15	407
LR-D41		8/7/14	8/8/14	1	9/18/15	407
LR-D42		8/9/14	8/11/14	2	8/31/15	387
LR-D43		8/9/14	8/11/14	2	8/31/15	387
LR-E01		8/15/14	8/16/14	1	9/18/15	399
LR-E02		8/15/14	8/16/14	1	9/18/15	399
LR-F01		8/19/14	8/20/14	1	8/27/15	373
LR-F02		8/19/14	8/20/14	1	8/31/15	377
LR-F03		8/20/14	8/21/14	1	8/27/15	372
LR-F04		8/20/14	8/21/14	1	9/18/15	394
LR-F05		8/21/14	8/22/14	1	8/31/15	375
LR-F06		8/21/14	8/22/14	1	9/18/15	393
LR-F07		8/22/14	8/25/14	3	6/26/15	308
LR-F08		8/22/14	8/25/14	3	6/26/15	308

* Panels were switched around for erection, so Designation HC or LR does not indicate which bridge the panel was constructed on.

^ FDOT inspection records show release on 7/24, but more likely it was 7/25.

Panels were in line in casting bed and used same set of prestressing strands.

Table 5-2. Prestressed Precast Deck Panel Concrete Strengths and Inspection Notes

PANEL # *	CAST TOGETHER	DESIGN MIX **	CDS LOT #	CDS CONCRETE STRENGTH AT RELEASE (psi)	CDS CONCRETE STRENGTH AT 28 DAYS (psi)	FDOT SAMPLE # FOR VERIFICATION	FDOT CONCRETE STRENGTH AT 28 DAYS (psi)	FDOT SPECIAL NOTES ("post-pour" observations)
HC-D01		03-1722	GPS001Q	7 170	11 590			2 minor spalls
HC-D02		03-1722						
HC-D03		03-1722	GPS002Q	7 650	11 210			
HC-D04		03-1722						
HC-D05		03-1722	GPS003Q	5 100	13 040			
HC-D06		03-1722						
HC-D07		03-1722	GPS004Q	9 310	13 520	GPS004V	12 760	
HC-D08		03-1722						
HC-D09		03-1722	GPS005Q	6 880	12 500	GPS005V	11 150	
HC-D10		03-1722						
HC-D11		03-1722	GPS006Q	7 380	11 280			
LR-D01		03-1722						
LR-D02		03-1722	GPS007Q	5 460	13 550			
LR-D03		03-1722						
LR-D04		03-1722	GPS008Q	5 300	13 550			Minor surface cracks
LR-D05		03-1722						
LR-D06		03-1722	GPS009Q	5 400	13 520			Minor surface cracks
LR-D07		03-1722						Surface cracks/foot
LR-D08		03-1722	GPS010Q	6 100	14 010			
LR-D09		03-1722						
LR-D10		03-1722	GPS011Q	7 380	13 950	GPS011V	12 980	Surface cracks/foot
LR-D11		03-1722						Surface cracks/foot
LR-D12		03-1722-01	GPS012Q	5 530	9 370			0 cracks
LR-D13		03-1722-01						0 cracks
LR-D14		03-1722-01	GPS013Q	6 630	8 730			Crack after removal
LR-D15		03-1722-01						Crack after removal
LR-D16		03-1722-01	GPS014Q	6 230	9 400	GPS014V	9 480	0 cracks
LR-D17		03-1722-01						0 cracks
LR-D18		03-1722-01	GPS015Q	6 490	9 050			0 cracks
LR-D19		03-1722-01						0 cracks
LR-D20		03-1722-01	GPS016Q	5 330	9 180			0 cracks
LR-D21		03-1722-01						0 cracks
LR-D22		03-1722-01	GPS017Q	4 540	8 600			0 cracks top surface
LR-D23		03-1722-01						0 cracks top surface
LR-D24		03-1722-01	GPS018Q	7 440	9 570			0 cracks
LR-D25		03-1722-01						0 cracks
LR-D26		03-1722-01	GPS019Q	5 845	9 620	GPS019V	9 670	0 cracks
LR-D27		03-1722-01						0 cracks
LR-D28		03-1722-01	GPS020Q	6 520	10 170			0 cracks
LR-D29		03-1722-01						Many shrinkage cracks
LR-D30		03-1722-01	GPS021Q	6 000	9 760	GPS021V	10 000	0 cracks
LR-D31		03-1722-01						0 cracks
LR-D32		03-1722-01	GPS022Q	5 910	10 360			0 cracks
LR-D33		03-1722-01						0 cracks
LR-D34		03-1722-01	GPS023Q	6 480	9 280			0 cracks
LR-D35		03-1722-01						0 cracks
LR-D36		03-1722-01	GPS024Q	5 660	9 400			0 cracks
LR-D37		03-1722-01						0 cracks
LR-D38		03-1722-01	GPS025Q	5 710	9 430			0 cracks
LR-D39		03-1722-01						0 cracks
LR-D40		03-1722-01	GPS026Q	4 840	9 670	GPS026V	8 610	REJECTED due to Flyash / Cracked while stored
LR-D41		03-1722-01						REJECTED due to Flyash / Cracked while stored
LR-D42		03-1722-01	GPS027Q	6 710	10 400			??CC
LR-D43		03-1722-01						??CC
LR-E01		03-1722-01	GPS028Q	7 720	10 100			REJECTED due to Flyash
LR-E02		03-1722-01						REJECTED due to Flyash
LR-F01		03-1722-01	GPS029Q	5 900	9 310	GPS029V	8 960	0 cracks
LR-F02		03-1722-01						0 cracks
LR-F03		03-1722-01	GPS030Q	6 690	10 210			0 cracks
LR-F04		03-1722-01						0 cracks
LR-F05		03-1722-01	GPS031Q	5 480	9 060			0 cracks
LR-F06		03-1722-01						0 cracks
LR-F07		03-1722-01	GPS032Q	7 750	9 910			
LR-F08		03-1722-01						

* Panels were switched around for erection, so Designation HC or LR does not indicate which bridge the panel was constructed on.
** MIXES 03-1722 and 03-1722-01 are FDOT Class VI (8500 psi).
Panels were in line in casting bed and used same set of prestressing strands.

Table 5-3. Prestressed Precast Deck Panels, Release and Number of Days After Casting

# of Days Prestress Release Occurred after Casting	# of Panels
1	48
2	8
3	6
4	2

<u>Materials</u>:

The design plans specified Class IV (5500 psi) concrete for prestressed precast deck panels, with a required strength of 4400 psi at the time of prestress release. All of these panels were cast with either MIX 03-1722 or MIX 03-1722-01, which are both Class VI (8500 psi). **Table 5-4** summarizes the minimum, maximum, and average concrete strengths measured at prestress release and at 28 days.

Table 5-4. Prestressed Precast Deck Panels, Measured Concrete Strengths (Minimum, Maximum, and Average)

Prestressed Panel Concrete Strength	At Prestress Release (psi)	At 28 days (psi)
Minimum	4 540	8 600
Maximum	9 310	14 010
Average	6 329	10 697

The design plans specified 0.5-inch diameter, Grade 270, low-relaxation strands stressed at 31.0 kips each. Material certifications for the strands used in the panels consistently showed a modulus of elasticity of 28,800 ksi.

<u>Erection</u>:

For Hurricane Creek WB Bridge, the 13 panels were erected on 6/26/2015. The age on erection day ranged from 308 to 476 days, with an average of 443 days. For Little River WB Bridge, the panels were erected on 8/27/2015 – 9/18/2015. The age on erection day ranged from 372 to 553 days, with an average of 430 days.

5.3 Cracking

Unlike the non-prestressed panels, the prestressed panels did not experience any significant cracking while at the precast yard or during transportation to the bridge site.

CHAPTER 6
PRECAST BENT CAPS

6.1 Description

Little River WB and EB Bridges each have Precast Bent Caps for their three (3) intermediate bents at Bent Locations 2, 3, and 4. The bent caps contain mild reinforcement but no prestressing strands. The caps are supported by two (2) 48-in. diameter drilled shafts (cast-in-place columns) (**Figure 6-1**). Each bent cap is 41'-6" long (transverse to the bridge), 4'-8" wide, and 4'-0" deep; the depth tapers to 3'-0" at the ends (**Figure 6-2**). The caps are level on the top and bottom. The details are similar to a bent cap/column system described in Strategic Highway Research Program 2, Project R04-RR-1 (SHRP2 2014), with the bent cap-to-column connection design being based on NCHRP Report 681 (NCHRP 2010). To reduce the precast cap weight, expanded polystyrene (EPS) voids were used, as detailed in **Figures 6-3 and 6-4**. The connection between the bent cap and drilled shaft/column (Figure 6-3) was designed to provide fixity for the governing-load-case moment.

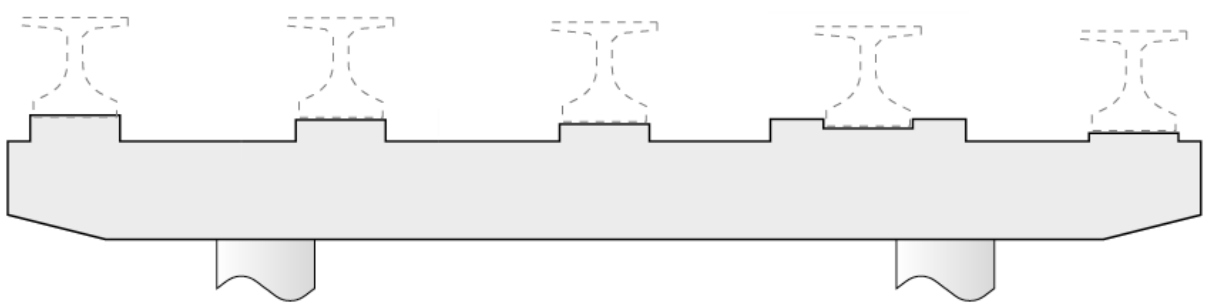


Fig. 6-1. Schematic of Intermediate Bent Cap, Elevation View

The bent caps were cast by CDS using steel forms (**Figures 6-5 to 6-7**). The reinforcing steel cage was tied outside of the forms but nearby (**Figures 6-8 to 6-10**). As shown in **Figure 6-11**, dowel bars extend from the top of the cast-in-place drilled shaft/column. The precast bent cap accommodates these dowel bars by holes formed with 4-in. diameter, 26 gauge, corrugated metal duct. (The holes were to be filled with precision grout later at the bridge site.) Steel pipes were placed inside the corrugated duct to stabilize the internal void and prevent the duct from deviating during concrete placement. A template was welded to the bottom of the forms, so that the pipes could be positioned in the proper location (**Figure 6-12 and 6-13**). The template also helped to secure the pipes (shown separately in **Figure 6-14**) during concrete casting. A bottom view of the completed bent cap, with the pipes slightly protruding, is shown in **Figure 6-15**. The pipes were removed a short time after casting, near the time of initial set, while they could still be removed easily.

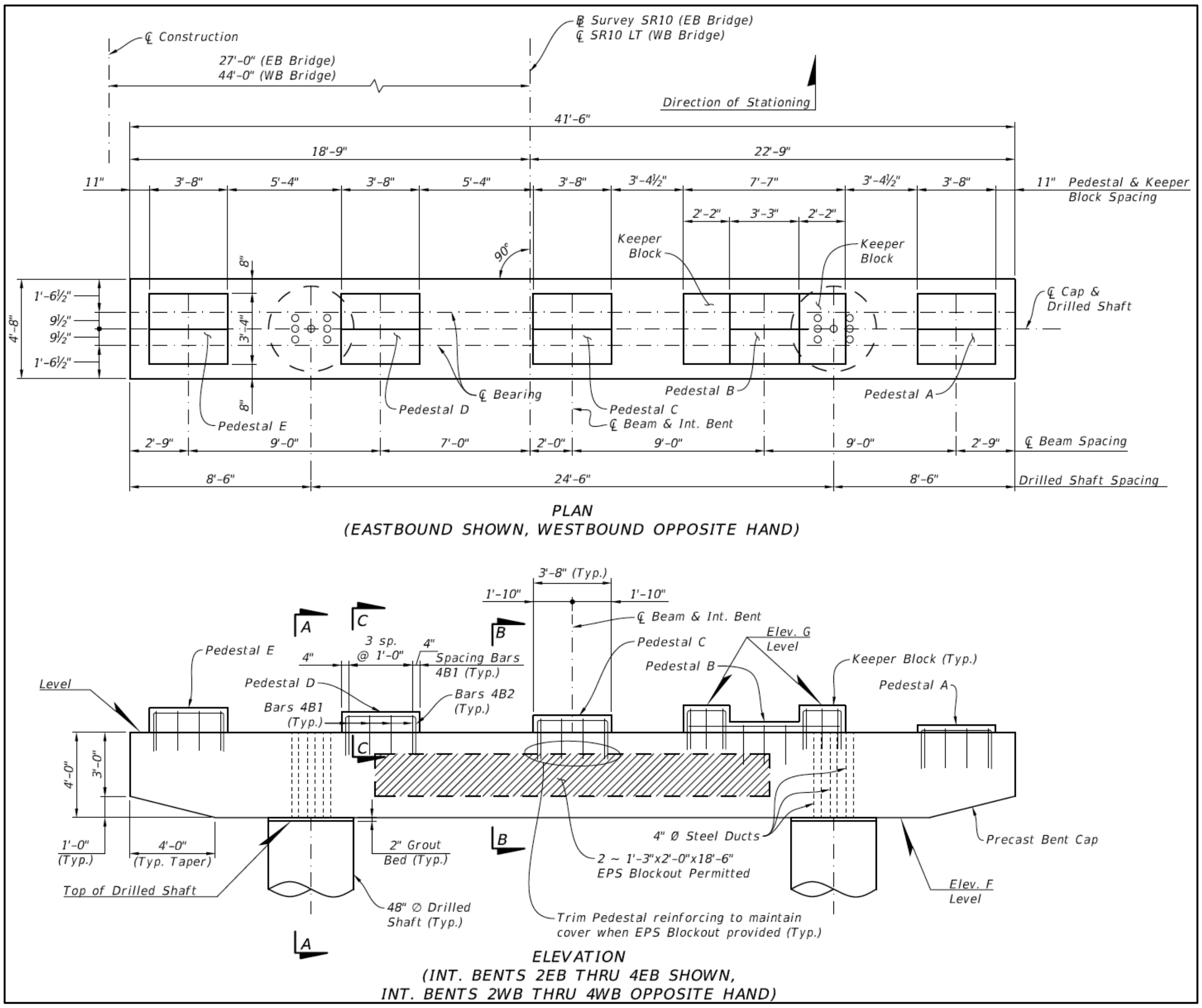


Fig. 6-2. Intermediate Bent Cap Dimensions and Details, Plan and Elevation Views (from Design Plans)

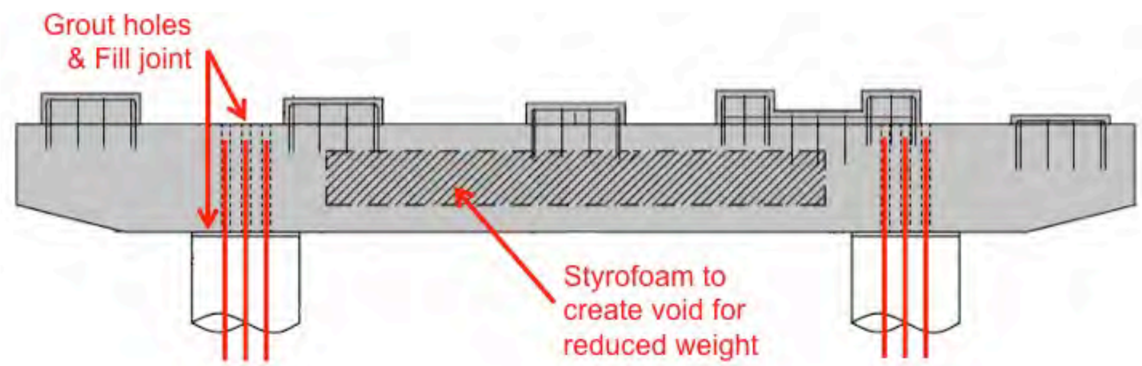


Fig. 6-3. Precast Bent Cap, Connection to Shaft/Column and Styrofoam

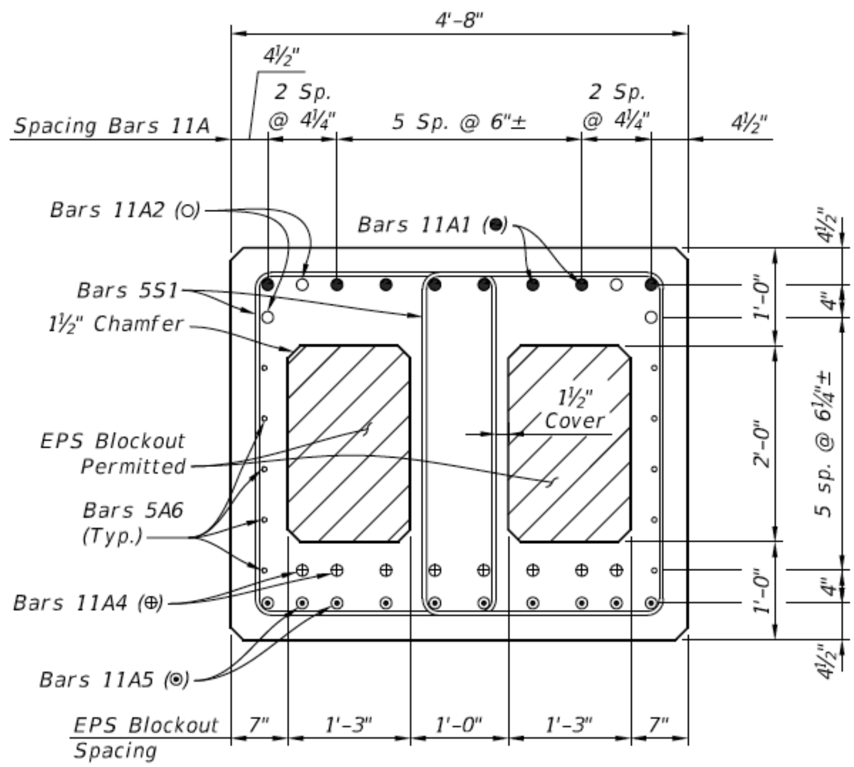


Fig. 6-4. Precast Bent Cap, Cross Section and Styrofoam Blockouts to Reduce Weight (from Design Plans)


Fig. 6-5. Precast Bent Cap Forms, with Left Side Not Yet Installed


Fig. 6-6. Precast Bent Cap Form, Oblique View

Fig. 6-7. Precast Bent Cap Forms, Getting Ready to Set Reinforcing Steel

Fig. 6-8. Precast Bent Cap, Tying Reinforcing Steel Cage (Concrete Blocks are Partially Obstructing View)

Fig. 6-9. Precast Bent Cap, Reinforcing Steel Cage

Fig. 6-10. Precast Bent Cap, Reinforcing Steel Cage, End View

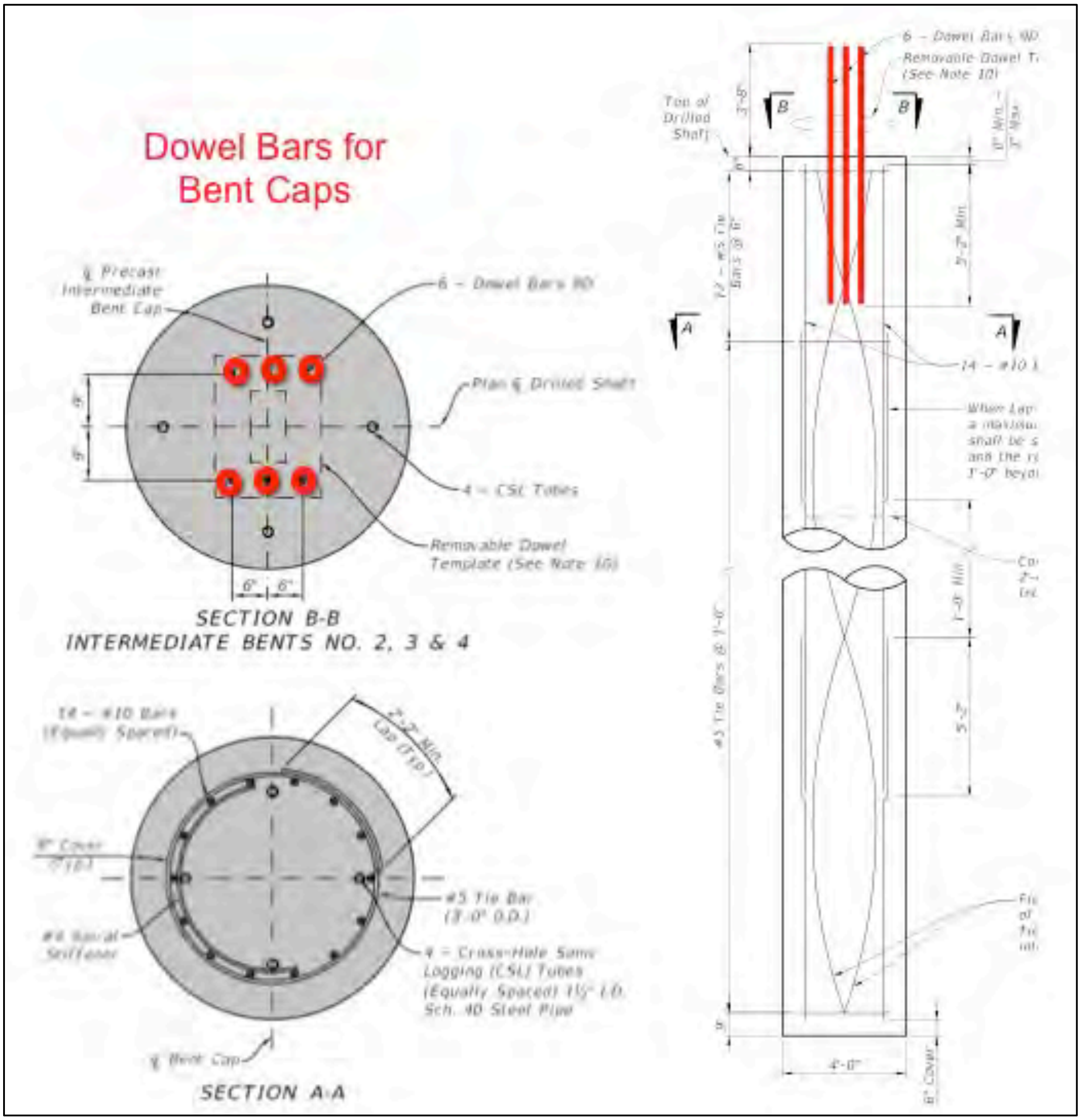


Fig. 6-11. Precast Bent Cap, Emphasizing Dowel Bars that Extend from Cast-in-Place Drilled Shaft/Columns

Fig. 6-12. Precast Bent Cap, End Taper in Background, and Template on Bottom to Position Pipe Forms for Dowel Bar Holes

Fig. 6-13. Precast Bent Cap, Close-up of Template to Secure Pipe Forms for Dowel Bar Holes

Fig. 6-14. Precast Bent Cap, Pipes for Stabilizing Dowel Bar Duct Forms (Two Shown)

Fig. 6-15. Underside of Precast Bent Cap, Showing Pipes Protruding

Reinforcing steel details per the design plans are provided in **Figure 6-16**. **Figure 6-17** is a cross section at the drilled shaft/column location, where the dowel bars extend. The design called for a spiral that surrounds the dowel bars; however, the precaster substituted the spirals with hoop ties (**Figures 6-18 and 6-19**), with permission from the Engineer. Also notice the EPS blockouts shown in the background of Figure 6-18 and in **Figure 6-20**.

Reinforcing steel for the bearing pad pedestals was cast into the bent cap, as shown in **Figure 6-21**. The pedestals were cast in place after the bent caps were erected at the bridge site. The pedestals varied in height to accommodate the 2% cross slope and beam grades. Casting the pedestals in the field, as opposed to precasting them with the bent cap, offered several advantages: the cap lifting weight was reduced; cap placement tolerances could be better accommodated; their heights could be adjusted for actual beam cambers; and they could help seal the grouted dowel bar holes and lifting points. Tolerances, per the design plans, are in **Figure 6-22**, and the support details (for pickup, storage, and transportation) are in **Figure 6-23**.

Figure 6-24 is a photo of a completed bent cap in the casting yard.

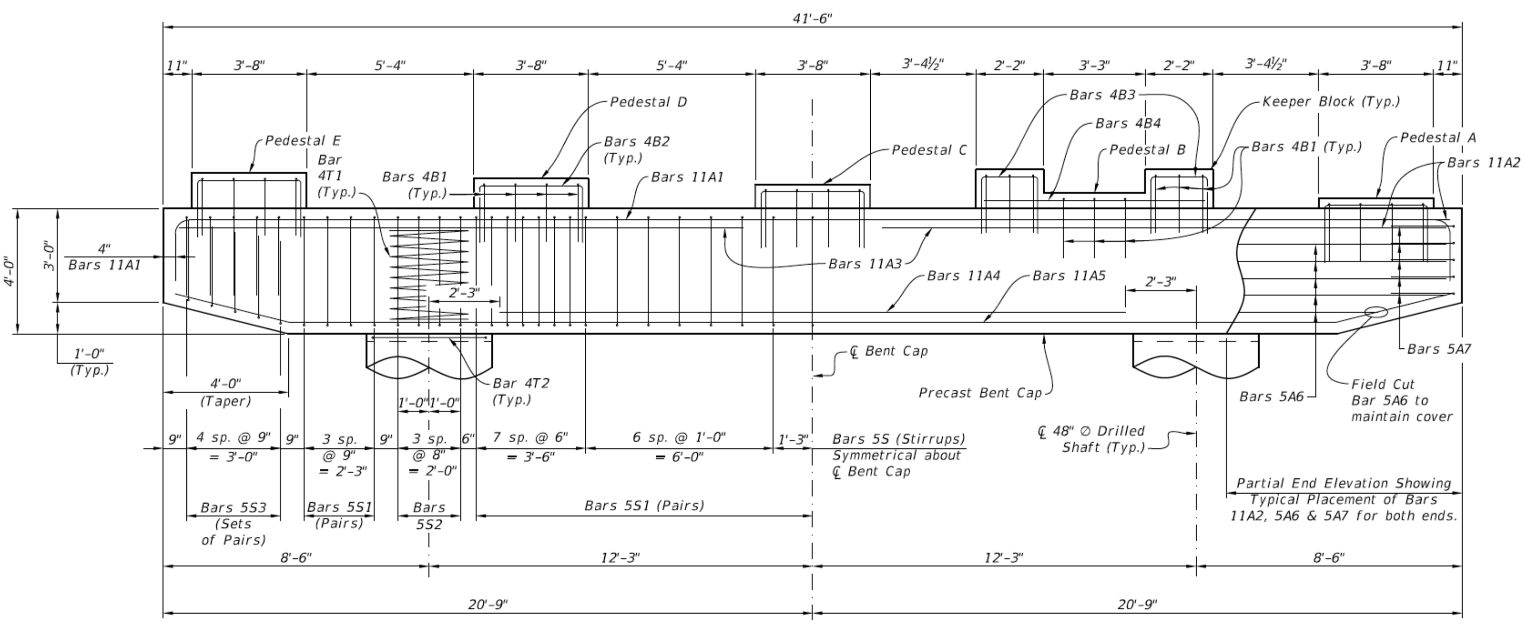


Fig. 6-16. Precast Bent Cap, Reinforcing Steel Details (from Design Plans)

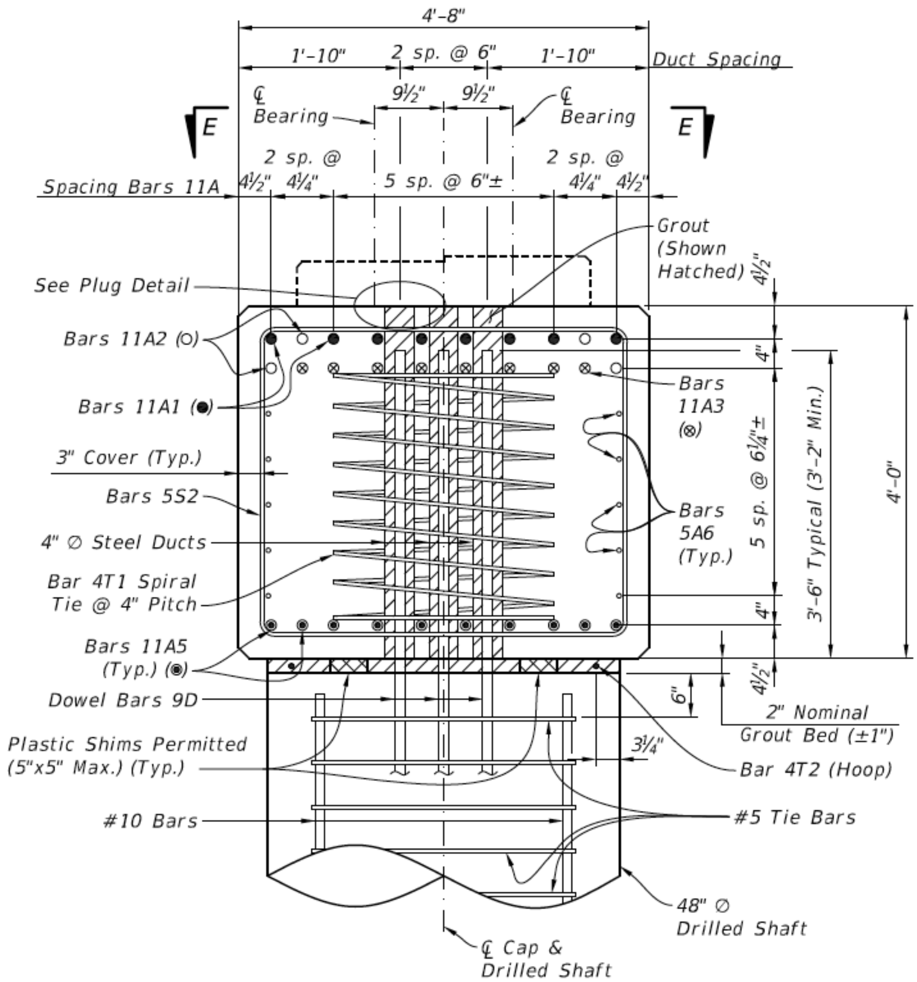


Fig. 6-17. Precast Bent Cap, Cross Section at Drilled Shaft, Showing Spiral (from Design Plans)

Fig. 6-18. Precast Bent Cap, Photo of Cross Section, Showing Hoop Ties (in Foreground) and EPS Blockout (in Background)

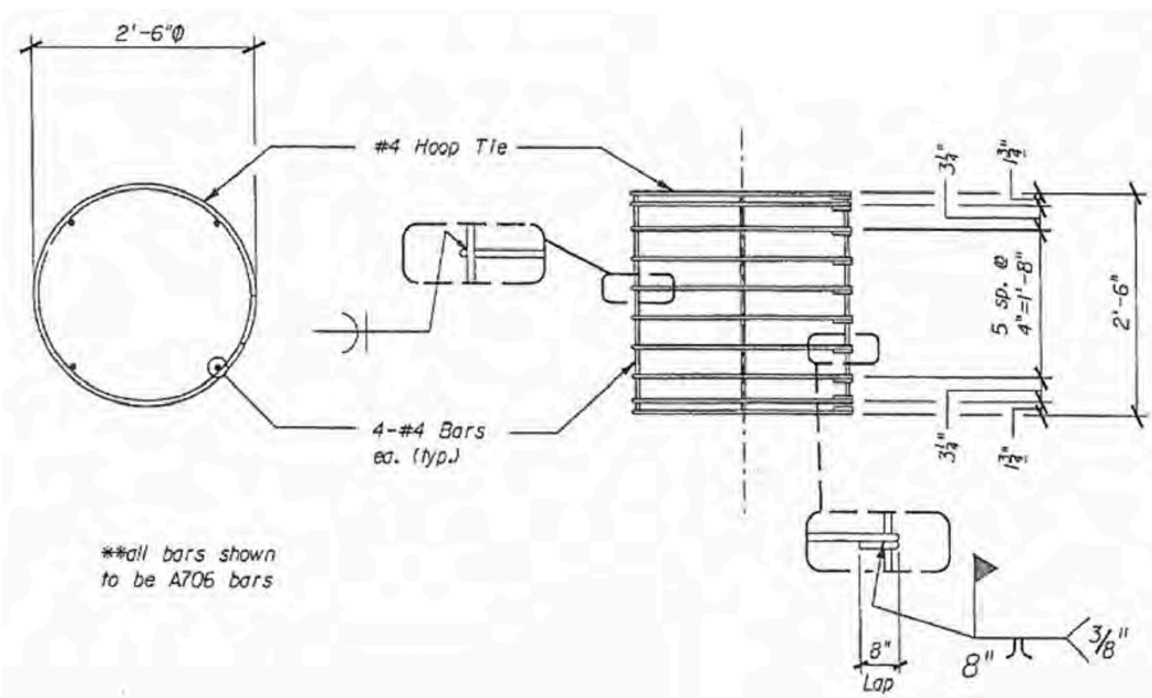
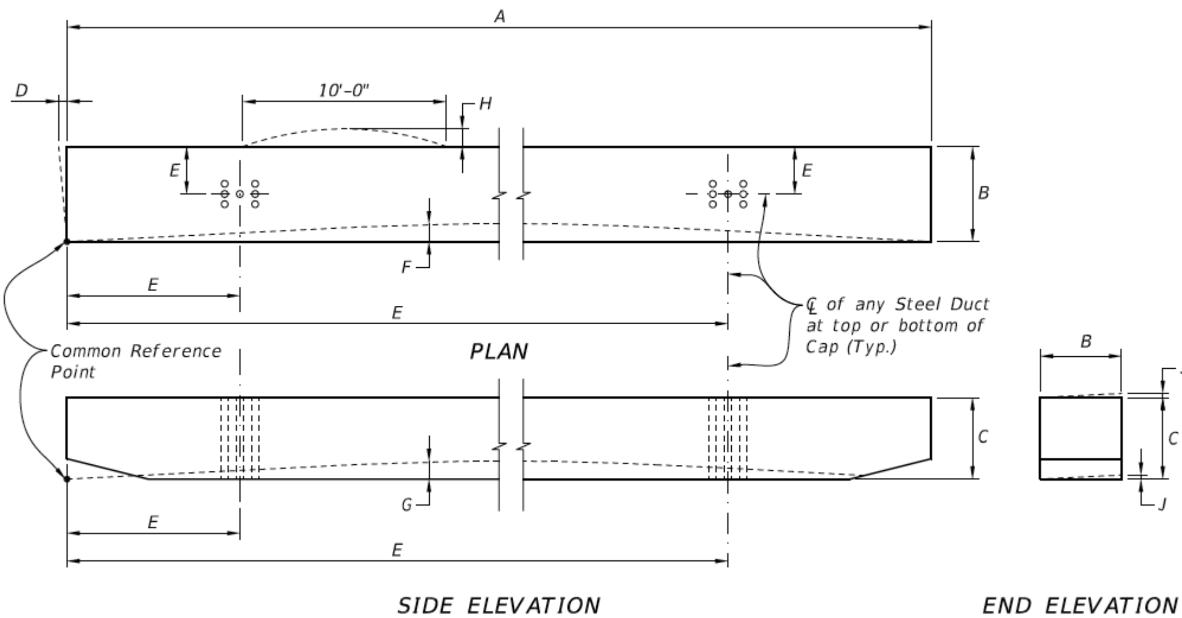


Fig. 6-19. Precast Bent Cap, Hoop Ties Used instead of Spirals (from Shop Drawings)


Fig. 6-20. Precast Bent Cap, Reinforcing Steel Cage and EPS Blockout, Side View


Fig. 6-21. Precast Bent Cap, Reinforcing Steel for Cast-in-Place Bearing Pad Pedestals



PRECAST BENT CAP FABRICATION TOLERANCES TABLE		
ITEM	DESCRIPTION	TOLERANCE
A	Length	± 1"
B	Width	± ½"
C	Depth	± ½"
D	Variation from Plan End Squareness	± ½"
E	Location of Steel Ducts	± ¼"
F	Sweep	± ⅜"
G	Variation from Specified Camber*	± ¼"
H	Local smoothness of any surface	¼" in 10 feet
J	Warping	± ⅛"

* Specified Camber = 0"

Fig. 6-22. Precast Bent Cap Tolerances (from Design Plans)

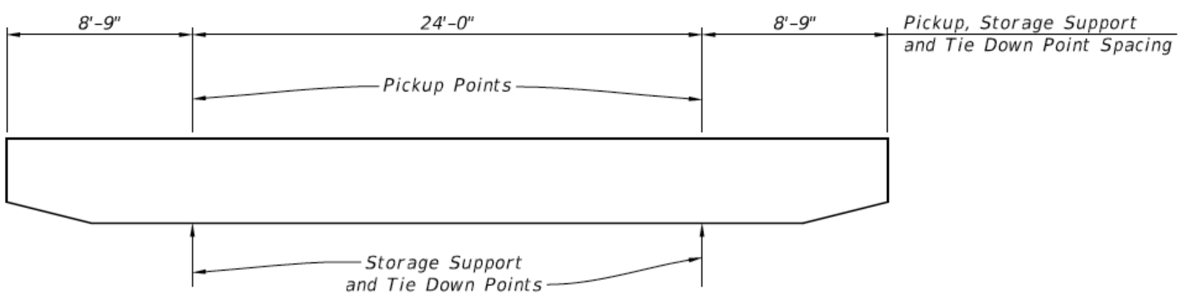


Fig. 6-23. Precast Bent Cap Pickup, Storage, and Transportation Support Details (from Design Plans)


Fig. 6-24. Precast Bent Cap Removed from Forms

6.2 Casting Data

Table 6-1 provides information about the precast bent caps, including bent cap # designations and casting, stripping and shipping dates/ages. **Table 6-2** provides concrete strengths at stripping and at 28 days, as well as FDOT inspection notes. A synopsis of the data in Tables 6-1 and 6-2 is as follows.

Casting:

The precast bent caps were cast one at a time in the casting bed. Casting occurred on:

- 3/30/2014, 4/4/2014, and 5/3/2014 for Caps EB-2, EB-3, and EB-4, respectively
- 7/23/2015, 7/11/2015, and 7/16/2015 for Caps WB-2, WB-3, and WB-4, respectively.

Forms were stripped 1 to 3 days after casting.

Table 6-1. Precast Bent Cap Designations and Casting/Shipping Dates

BRIDGE # / NAME	BENT CAP #	CAST DATE	FORM STRIPPING DATE	AGE AT FORM STRIPPING (days)	SHIPPING DATE	AGE AT SHIPPING (days)
500151	*WB-2*	7/23/15	7/24/2015 ?	1	7/27/15	4
LITTLE	*WB-3*	7/11/15	7/13/2015 ?	2?	7/24/15	13
RIVER WB	*WB-4*	7/16/15	7/18/2015 ?	2?	7/24/15	8
500152	*EB-2*	3/30/14	4/2/14	3	4/7/14	8
LITTLE	*EB-3*	4/4/14	4/5/14	1	4/7/14	3
RIVER EB	*EB-4*	5/3/14	5/5/14	2	5/7/14	4

Note: The symbol "?" means that the date was estimated based on a conversation with the precaster.

Table 6-2. Precast Bent Cap Concrete Strengths and Inspection Notes

BENT CAP #	DESIGN MIX **	CDS LOT #	CDS CONCRETE STRENGTH ON STRIPPING DAY (psi)	CONCRETE STRENGTH ON SHIPPING DAY (psi)	CDS CONCRETE STRENGTH AT 28 DAYS (psi)	FDOT SAMPLE # FOR VERIFICATION	FDOT CONCRETE STRENGTH AT 28 DAYS (psi)	FDOT SPECIAL NOTES ("post-pour" observations)
WB-2	*03-1722-04*	-	-	-	10 940	-	-	-
WB-3	*03-1722-04*	-	-	-	10 260	-	-	-
WB-4	*03-1722-04*	-	-	-	9 670	-	-	-
EB-2	*03-1722*	GBC001Q	9 340	-	12 830			0 cracks
EB-3	*03-1722*	GBC002Q	7 280	10 330 at 3 days	13 260			0 cracks
EB-4	*03-1722*	GBC003Q	9 260	-	13 600	GBC003V	12 000	0 cracks
		GBC003QR			12 460 at 31 days	GBC003VR	12 890 at 31 days	--

** All bent caps used MIX 03-1722, which is FDOT Class VI (8500 psi at 28 days).
- Information not available.

Materials:

The design plans specified Class III (5000 psi) concrete for precast bent caps. All bent caps were cast with MIX 03-1722, which is Class VI (8500 psi). (Class II concrete was specified in the design plans for the cast-in-place keeper blocks and pedestals.) **Table 6-3** summarizes the minimum, maximum, and average concrete strengths measured at stripping and at 28 days.

Table 6-3. Precast Bent Caps, Measured Concrete Strengths (Minimum, Maximum, and Average)

Bent Cap Concrete Strength	At Stripping (psi) (EB bents only)	At 28 days (psi)
Minimum	7 280	9 670
Maximum	9 340	13 600
Average	8 627	11 760

Erection:

The bent caps were transported from the precast yard to the construction site with an aluminum deck trailer. No issues or challenges were encountered. For Little River EB Bridge, Bent Caps EB-2 and EB-3 were erected on 4/7/2014. Bent Cap EB-4 was erected on 5/7/2014. The age on erection day was 8, 3, and 4 days, respectively. For Little River WB Bridge, Bent Caps WB-3 and WB-4 were erected on 7/24/2015, and Bent Cap WB-2 was erected on 7/27/2015. The age on erection day was 13, 8, and 4 days, respectively.

6.3 Observations

Dowel bar misalignment:

The dowel bars that extend out of the drilled shaft (Figure 2-18 in Report 1) must be securely positioned before the drilled shaft concrete is placed, to ensure proper fit with the precast bent cap. The design plans contained a note that stated the need to use a template and adjust it correctly:

> *Adjust Dowel Template laterally to match global location depicted on Intermediate Bent sheets and ensure fit up with Precast Bent Cap Ducts. Secure Dowel Template in position and verify correct location of Dowel Bars 9D prior to placing concrete for Drilled Shafts.*

The contractor used a wooden template to position the dowel bars (**Figure 6-25**). However, the template did not sufficiently secure the dowels while the concrete was placed in the drilled shaft extension on Bent 4 (the easternmost bent) of Little River EB Bridge. The dowel bars were out of position, to a condition in which the holes in the bent cap (as designed) would not be able to fit. **Figure 6-26** shows the constructed locations of these dowel bars. This misalignment was realized *before* the bent cap was cast, so the precaster was able to adjust the dowel hole ducts and reinforcing steel (bends and positions) in the bent cap.

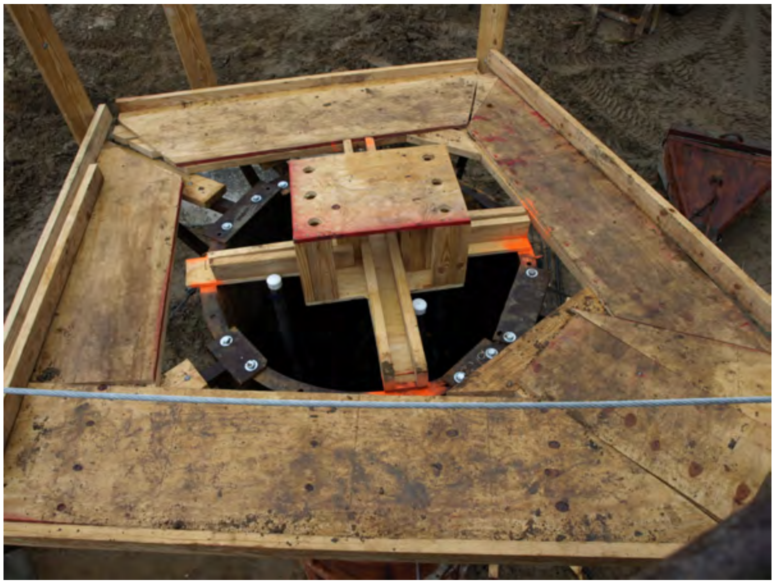


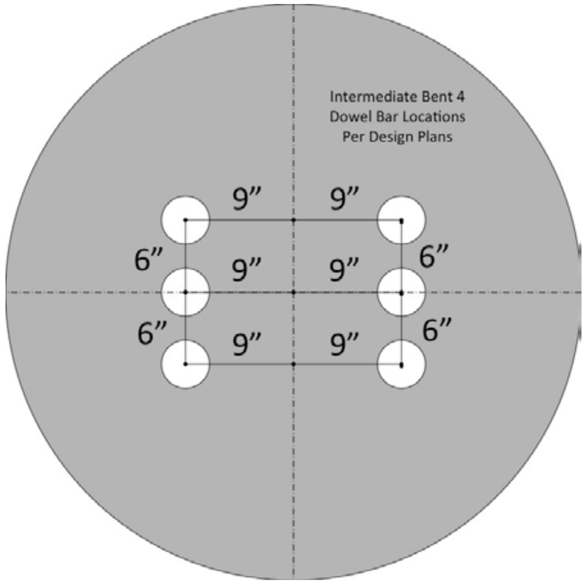
Fig. 6-25. Original Wooden Template Used to Position Drilled Shaft Dowel Bars that Will Extend into Precast Bent Cap (Drilled Shaft Steel Casing Shown Underneath)

Figure 6-27 shows the original shop drawings for the reinforcing steel in the bent cap cross section, and **Figure 6-28** shows the changes that were made to Bent 4EB (longitudinal bars 11A were shifted horizontally to avoid the shifted dowel ducts, and the stirrups were changed from two rectangular hoops to one large hoop and two single legs with hooks/bends at their ends. (See also information on RFI #12 below). For the remaining bents, the contractor made the holes smaller in the dowel bar template to better secure the bars. Later, the dowel bars for Bent 2 of Little River WB Bridge were out of position, but the bent cap had not yet been cast. The precaster was able to shift longitudinal bars 11A, along with the dowel holes; also, some of the stirrups were made 1.5 in. wider so that bars 11A would fit in the stirrup corners (**Figure 6-29**).

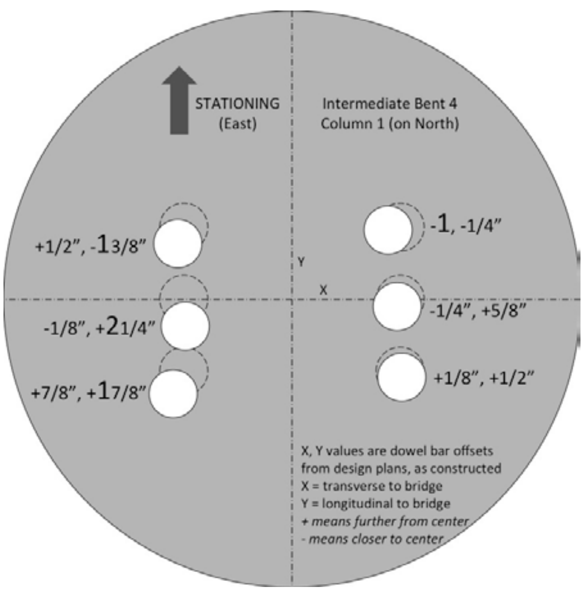
6.4 Related RFIs

RFI #11, dated 3/25/2014

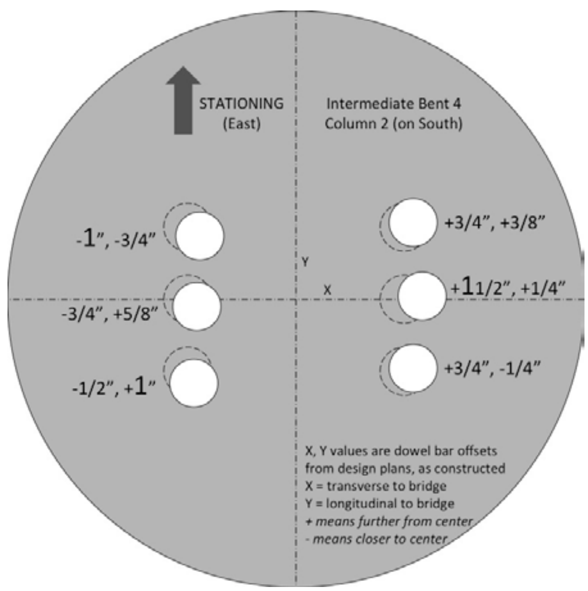
> Anderson Columbia has requested to change the rebar spiral to a hoop tie with vertical bars over the drilled shafts due to the spiral availability. Please see the attached revised drawing showing the change. Please review to see if this change is feasible.
>
> *Outcome: FDOT requested that Anderson Columbia submit revised shop drawings for this change. The revised shop drawings were approved as submitted.*



(a) Locations per Design Plans



(b) Column 1 As-Built



(c) Column 2 As-Built

Fig. 6-26. Constructed Locations of Dowel Bars on Little River EB Bridge, Intermediate Bent 4 (Diameters Are Drawn Larger for Clarity)

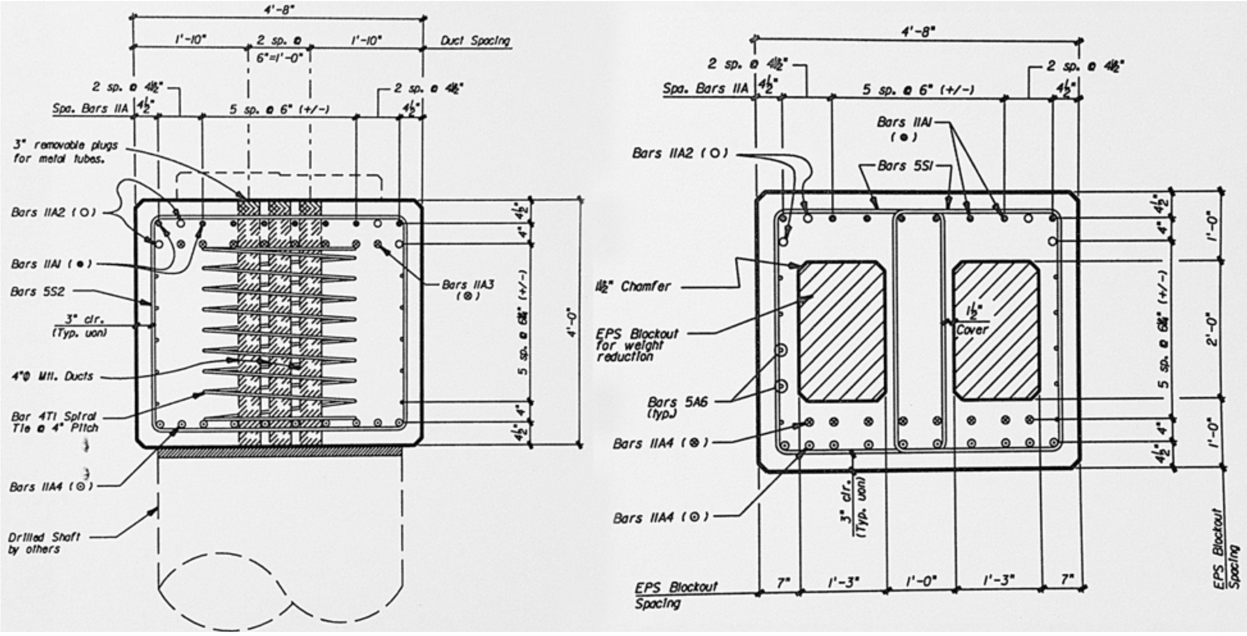


Fig. 6-27. Original Bent Cap Shop Drawings

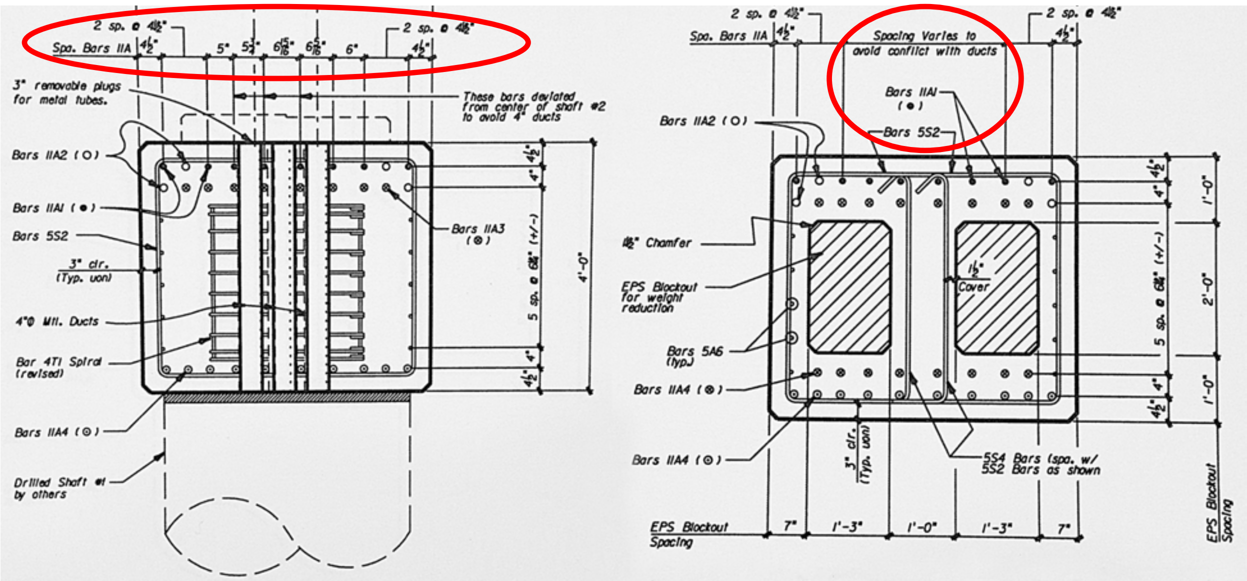


Fig. 6-28. Adjustment to Bent Cap 4EB (Reinforcing Steel Bars 11A, 5S2 and 5S4)

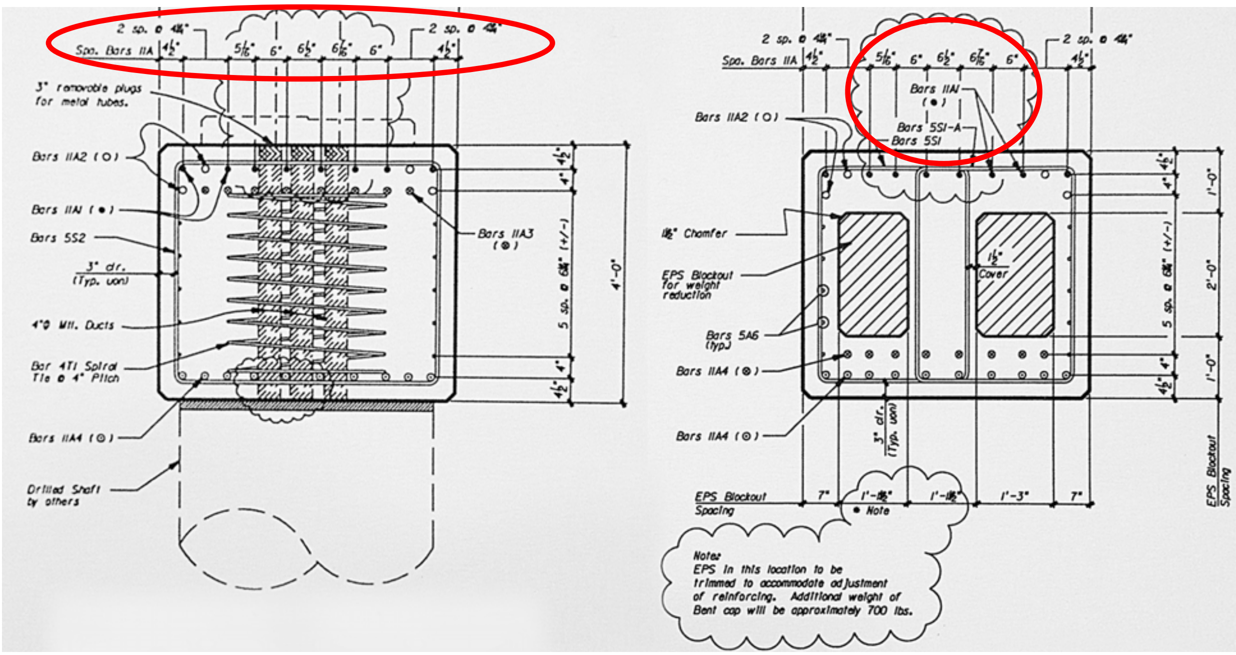


Fig. 6-29. Adjustment to Bent Cap 2WB (Reinforcing Steel Bars 11A and 5S1-A)

RFI #12, dated 3/28/2014 – Related to dowel bars in Bent 4 Little River EB Bridge

With the EPS form, there is not enough room to extend the stirrup to encase the #11 bars. Would reducing the size of the EPS help?

Response: Using the Bars C as substitutes for the internal legs of the shear stirrups (Bars 5S1) and reconfiguring Bars 5S1 as perimeter reinforcing only is acceptable. The bottom and top bar bend legs of the Bars C with Bars 5S1 should be at least 10" long. The Contractor will need to resubmit revised shop drawings detailing these stirrup bars, and to add to what I said below, the bars should be positioned so that there is a longitudinal bar in each corner of each single leg stirrup. That means the single leg stirrups could be positioned as [] or [[. And, the single leg stirrups should have one 90 degree hook and one 135 degree hook.

Figure 6-30 shows hand sketches of the changes that were made to the reinforcing steel as a result of RFI #12. (See also Figures 6-26 to 6-28.)

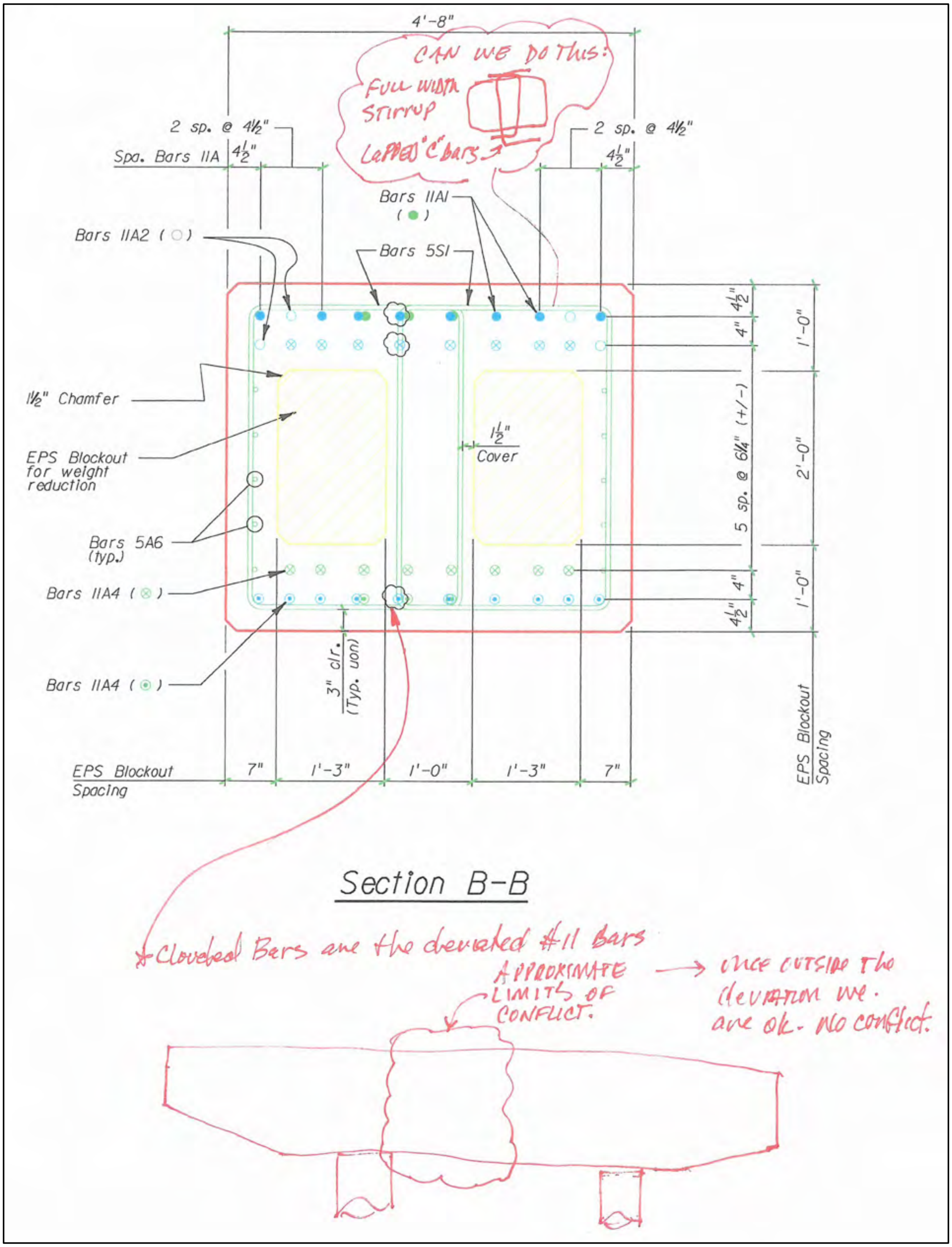


Fig. 6-30. Hand Sketch of Adjustments Made to Intermediate Bent 4 Reinforcing Steel

Precast Element Evaluation for the US 90 Bridges over Little River and Hurricane Creek

Contract Number BDV30-307-01
FSU Project ID 032819

Submitted to:

Florida Department of Transportation

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December 2017
REPORT 3: CONSTRUCTION OF PRECAST ELEMENTS

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REPORT 3: CONSTRUCTION OF PRECAST ELEMENTS

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CHAPTER 1
CONSTRUCTION PROJECT

1.1 Introduction

This report provides details on construction of the precast elements. The construction site was visited several times to observe and document the construction activities. The role of the researchers was *not* for quality control or quality assurance, as that was left to other parties of the construction contracts.

The precast elements that were used to construct the bridges included precast bent caps (**Figure 1-1**), prestressed Florida-I beams (**Figure 1-2**), and precast deck panels (**Figure 1-3**). As presented in this report, the research involved observing the construction process, contractor's means and methods, and equipment used. The researchers noted instances where the contractor was able to improve the process as experience was gained throughout the project duration. The researchers also looked for any overall quality issues and potential changes that could be made in the construction specifications.

Fig. 1-1. Setting a Precast Bent Cap Using a Two-Point Pick (July 24, 2015)


Fig. 1-2. Girders on a Span on Little River EB (September 18, 2014)


Fig. 1-3. Precast Deck Panels, Looking Transversely on Little River WB (August 31, 2015)

In the separate report, Report 2, Fabrication of Precast Elements, details were provided on the production of the elements including forms and casting yard activities. Some details from Report 2 are duplicated in this report, but repetitiveness is minimized as much as possible. The reports were written to complement each other. To best understand this report, the reader is encouraged to first read Report 2.

An outline of this report is as follows:

1. CONSTRUCTION OVERVIEW – Provides start and end dates, briefly explains the order of construction of EB and WB bridges, and contains photos of the construction site and finished bridges.
2. CONSTRUCTION SITE – Explains site access, temporary road, and work bridge.
3. DRILLED SHAFTS – Describes drilled shaft construction and revised tip elevations.
4. END BENTS – Provides description and photos of forms and a final constructed end bent.
5. INTERMEDIATE BENTS – Explains construction tolerances and their importance for fitting together drilled shafts, columns, and precast bent caps.
6. COLUMNS – Provides details on their construction. Explains some challenges associated with the dowel bars that extended from the top that connected the column to the precast bent cap.
7. PRECAST BENT CAPS – Explains delivery and erection, as well as grouting of the column-to-bent cap connection.
8. BEARING PAD PEDESTALS – Explains their construction and adjustments made to ensure constructable haunch thicknesses.
9. PRECAST PRESTRESSED FLORIDA-I BEAMS – Provides details on their erection and erection rate.
10. PRECAST DECK PANELS – Explains delivery and erection. Provides details on surveying and leveling the panels, and notes differences between the non-prestressed and prestressed panels. Describes the forms placed under the panels for grouting of shear pockets and haunches.

11. SHEAR POCKET AND HAUNCH GROUTING – Provides details on grouting the shear pockets and haunches – including materials, equipment, process, sampling requirements, and lessons learned.
12. CLOSURE JOINT CASTING – Describes forms and procedures for casting the closure joints between the precast deck panels.
13. FINISHING – Briefly describes the finishing touches, including barrier construction, milling and grooving of the bridge deck, and filling of the deck expansion joints.
14. GROUTING DEMONSTRATION TESTS – Provides details on the purpose of, and requirements for, the Grouting Demonstration Tests. The tests included mock-ups of the precast bent cap and precast deck panel grouting procedures.
15. CORES – Includes photos of cores that were drilled in the deck panels, to document the shear pocket-to-panel interface and the closure joint-to-panel interface.
16. CONSTRUCTION SCHEDULE – Provides an overall timeline for construction of the four (4) bridges, noting start and end dates for major activities.

1.2 Construction Overview

The US 90 bridge construction project was awarded on May 22, 2013, and Notice to Proceed was issued on July 31, 2013. The construction contract started on September 29, 2013. On the critical path for the overall project, roadway construction was performed concurrently with the bridge construction. Each of the four existing bridges was demolished before construction began on its replacement bridge.

Eastbound (EB) traffic was diverted to one of the westbound (WB) lanes during construction of the EB roadway and bridges. EB bridges were completed in late February 2015, and the EB lanes were opened to traffic on March 16, 2015. WB traffic was then diverted to one of the EB lanes so that WB roadway and bridges could be constructed. WB bridges were completed in late November 2015, and the WB lanes were opened to traffic in February 2016. FDOT issued final acceptance on March 1, 2016.

Figure 1-4 is an aerial view of the construction site, taken from Hurricane Creek EB bridge, looking west towards Little River. **Figure 1-5** is taken from the same vantage point, except looking east.

Fig. 1-4. Hurricane Creek EB, Looking West Towards Little River (November 20, 2014)

Fig. 1-5. Hurricane Creek EB, Looking East Towards Tallahassee (November 20, 2014)

Figure 1-6 is a photo of the finished bridges, looking east towards Tallahassee. The Little River bridges are in the foreground, and the Hurricane Creek bridges are in the background.

Fig. 1-6. Photo of Finished US 90 Bridges and Roadway

1.3 Construction Site

1.3.1 Hurricane Creek Bridges

The existing Hurricane Creek EB and WB bridges (**Figures 1-7 and 1-8**) were box culverts that spanned a significantly shorter length than their single-span replacement bridges. The box culverts were demolished in pieces. It took 3 months to remove the EB bridge. The WB bridge took only 1 month to remove; inspectors remarked that the shorter time frame was due to the WB bridge being fragmented into larger (fewer) pieces compared to the EB bridge.

Fig. 1-7. Existing Hurricane Creek EB Bridge, Southwest Corner (November 11, 2013)

Fig. 1-8. Hurricane Creek EB Bridge Site, Looking East from End Bent. Existing EB Bridge Has Been Demolished and Existing WB Bridge Remains. (February 3, 2014)

Construction of the replacement bridges was performed from above. The connecting roadway provided access for the cranes and other equipment needed. It was not necessary to work from creek level, and temporary roads were not needed.

1.3.2 Little River Bridges

Each existing Little River bridge had nine 36-foot spans and one 70-foot span (**Figure 1-9**). EB bridge had steel girders, and WB had prestressed concrete girders. Existing superstructure and substructure components were demolished, and the foundations were extracted.

Fig. 1-9. Existing Little River Bridge Site Before Construction: Northwest Corner, Looking East (January 13, 2013)

Construction of the replacement bridges was performed from above, from an access road below, and from a temporary work bridge. The temporary road and work bridge, shown in **Figures 1-10 and 1-11**, were built for construction equipment access. For EB bridge construction, the road and work bridge were constructed on the south side of the EB bridge. When construction was completed on the EB bridge, an access road was built on the north side of the WB bridge, and the temporary work bridge was relocated to there for WB bridge construction.

Little River was prone to severe flooding (**Figure 1-12**). Early during EB bridge construction, the temporary work bridge (**Figure 1-13**) was washed down the river during two separate flood events. The pieces had to be gathered, towed back to the bridge, and reassembled.

Fig. 1-10. Existing Little River Bridges, View from Underneath, and Temporary Work Bridge (November 11, 2013)

Fig. 1-11. Existing Little River EB Bridge During Construction: Temporary Work Bridge, Southwest Corner, Looking East (November 11, 2013)

Fig. 1-12. Flooding of Little River

Fig. 1-13. Temporary Work Bridge Damaged by Flooding

CHAPTER 2
SUBSTRUCTURE

2.1 Drilled Shafts

Drilled shafts, 48 inches in diameter, were constructed for the end bents of all four bridges, as well as for the three intermediate bent foundations on each of the Little River bridges. Typical construction methods were used, and no significant problems occurred. Drilling equipment on the site is shown in **Figure 2-1**, and **Figure 2-2** shows the reinforcing steel cage for one of the shafts. When installed, the reinforcing steel extended from the top of the shaft, so that later it could be spliced to the column (**Figure 2-3**).

Fig. 2-1. Drilled Shaft Construction

On Little River WB bridge, for drilled shaft #2 at Bent 4, the reinforcing cage was constructed inadvertently at an angle as it protruded from the top. This would cause the reinforcing steel to have less than the required 6-in. cover once the column was cast. As recommended by an Engineering Analysis Report, the cage was pulled into vertical alignment, within the 3-in. tolerance, and then supported with spacers at several points along the bars' length to keep the cage in place prior to placing the concrete.

Fig. 2-2. Existing Little River EB Bridge During Construction: Drilled Shaft Reinforcing Steel Cage, Southeast Corner, Looking West (November 11, 2013)

Fig. 2-3. Drilled Shaft Reinforcement for Little River EB Intermediate Bent (February 3, 2014)

The drilled shaft tip elevations, developed during design and based on laboratory testing of rock core samples, were +31 ft for Hurricane Creek end bents and ranged from +23 ft at end bents to +8.0 ft at interior bents for Little River. Results from the Osterberg Cell load test performed at Little River suggested that drilled shaft tip elevations could be optimized if similar subsurface conditions were encountered at individual shaft locations.

To evaluate subsurface conditions, a pilot hole boring (Standard Penetration Tests or SPT) was drilled within the footprint of each drilled shaft. Based on site-specific correlations between SPT, previous rock core testing results, and load test data, shaft embedment was revised. The updated tip elevations for Little River (shorter shafts) ranged from +30 ft at end bents to +25 ft at interior bents. Drilled shaft tip elevations for Hurricane Creek remained at +31 ft except for shaft No. 1 at end bent No. 1, where the tip elevation was extended to +35 ft based on the shaft-specific pilot hole results. Elevations are referenced to NAVD 88.

2.2 End Bents

The end bents were constructed using typical means and methods. They were formed with wooden boards and plywood (**Figures 2-4 to 2-6**). Steel reinforcing bars were tied mostly in cages. On the first end bent that was built, however, inspectors noted that it was difficult to get the reinforcing cage (for the cap and bearing seat) around the drilled shaft steel when the cage was being set in the forms. To avoid this problem when constructing subsequent end bents, they tied some of the rebar *after* setting the cage in. This mostly applied to the stirrup bars in the vicinity of the drilled shaft steel. The concrete was cast in place, and then the forms were removed.

Bank-and-shore rip rap rubble (**Figure 2-7**), 2.5 feet thick with 1 foot of bedding stone underneath it, was placed on the embankments of all four (4) bridges to prevent erosion from the creek or river floodwater. **Figure 2-8** shows how the soil was stabilized before the rip rap was placed. **Figure 2-9** is a photo of a completed end bent, including bearing pedestals and pads.

Fig. 2-4. Little River EB – Formwork for End Bent on East; High Water Level (March 18, 2014)

Fig. 2-5. Little River EB – Finishing Front Face of End Bent Cap on East (March 18, 2014)

Fig. 2-6. Hurricane Creek EB – Looking East at Formwork for End Bent (February 19, 2014)


Fig. 2-7. Completed End Bent and Rip Rap Rubble

Fig. 2-8. Hurricane Creek EB – Completed End Bent on West (March 18, 2014)

Fig. 2-9. Hurricane Creek EB – Completed End Bent on West and Rubble Rip Rap (March 24, 2014)

2.3 Intermediate Bents

2.3.1 Construction and Tolerances

Each Little River bridge had three (3) intermediate bents. Per bent, each of the two drilled shafts was vertically "extended" to a 48-inch-diameter column (forms are shown in **Figure 2-10**) that would support a precast bent cap. In this section, the specified construction tolerances and their importance will be discussed. In the next section, the column construction will be discussed, followed by the precast bent cap erection and their connection to the columns.

Construction tolerances were very important to ensure that the reinforcing steel protruding from the shaft would be in alignment with the column, and so that the reinforcing steel protruding from the column would fit in the preformed holes in the precast bent cap. Constructing within the tolerances was important, because too much misalignment would cause challenges with the elements fitting together.

Fig. 2-10. Both Columns Formed for Little River EB Intermediate Bent (February 6, 2014)

Tolerances were prescribed throughout the plans and specifications, a few excerpts from which are as follows:

Special Provision 455-20 Construction Tolerances

Meet the following construction tolerances for drilled shafts:

(a) Ensure that the top of the drilled shaft is no more than 3 inches laterally in the X or Y coordinate from the position indicated in the plans.

(b) Ensure that the vertical alignment of the shaft excavation does not vary from the alignment shown in the plans by more than 1/4 inch/ft of depth.

(c) After placing all the concrete, ensure that the top of the reinforcing steel cage is no more than 6 inches above and no more than 3 inches below plan position. When connecting to precast bent caps, cut any exposed reinforcing cage level with the top of the drilled shaft.

(d) Ensure that the reinforcing cage is concentric with the shaft within a tolerance of 1-1/2 inches. Ensure that concrete cover is a minimum of 4-1/2 inches unless shown otherwise in the Plans.

(e) All casing diameters shown in the Plans refer to I.D. (inside diameter) dimensions. However, the Contractor may use casing with an outside diameter equal to the specified shaft diameter if the extra length described in 455-15.7 is provided. In this case, ensure that the I.D. of the casing is not less than the specified shaft diameter less 1 inch. When approved, the Contractor may elect to provide a casing larger in diameter than shown in the Plans to facilitate meeting this requirement. When casing is not used, ensure that the minimum diameter of the drilled shaft is 1 inch less than the specified shaft diameter. When conditions are such that a series of telescoping casings are used, provide the casing sized to maintain the minimum shaft diameters listed above.

(f) Ensure that the top elevation of the drilled shaft concrete has a tolerance of plus 1 inch and minus 3 inches from the top of shaft elevation shown in the Plans. When connecting to precast bent caps, finish the top of the drilled shaft within plus or minus 1 inch of the elevation shown in the plans.

(g) The dimensions of casings are subject to American Petroleum Institute tolerances applicable to regular steel pipe.

(h) Use excavation equipment and methods designed so that the completed shaft excavation will have a flat bottom. Ensure that the cutting edges of excavation equipment are normal to the vertical axis of the equipment within a tolerance of plus or minus 3/8 inches per foot of diameter.

Plan Sheet No. B1-14 Drilled Shaft Notes and Details

Note #8: Adjust Dowel Template laterally to match global location depicted on Intermediate Bent sheets and ensure fit up with Precast Bent Cap Ducts. Secure Dowel Template in position and verify correct location of Dowel Bars 9D prior to placing concrete for Drilled Shafts.

Plan Sheet No. B1-19 Intermediate Bent General Notes (Sheet 1 of 2)

CONSTRUCTION:

General:

1. Chamfer all exterior corners 1½", except chamfer pedestal and keeper block exterior corners ¾".

2. Provide templates for positioning drilled shaft column dowel bars to ensure proper fit up with the precast bent cap.

Tolerances:

Precast Bent Cap:

See Precast Bent Cap Fabrication Tolerances Details on Sheet 2 of 2.

Drilled Shaft:

Top of Drilled Shaft Elevation = ± 1"

Drilled Shaft Cap Dowel Reinforcing:

Center of Group Template (Plan position) = ± ½"

Individual bars within Group template (Relative plan position) = ± ¼"

Plumbness = ± 1/8"

Pedestals:

Elevations = ± 1/8" (@ centerline of bearing)

Surface Slope = ± 1/16" per foot.

Steel Ducts:

Plan position = ± ¼" over full length.

Tolerances for the precast bent caps are provided in a separate report, Report 2, Fabrication of Precast Elements.

2.3.2 Columns

The formwork for the 48-inch-diameter columns tied in to the top of the drilled shaft casing column (**Figure 2-11**). The columns directly extended from the drilled shafts and were cast in place (**Figure 2-12**). It was critical for the columns to be plumb and in the correct position, so that they would fit with the precast bent caps that were later erected on top.

Fig. 2-11. Column Forms for Little River EB Intermediate Bent (February 6, 2014)

Six (6) #9 dowel bars extended from the top of each column, so that they could protrude into preformed holes in the precast bent cap when it was erected. Placement of the dowel bars was somewhat challenging for the contractor at first. A plywood template (**Figure 2-13**) was used to align the bars, but inspectors noted the difficulty of determining whether or not the dowel bars were in the correct position at their top ends. For two (2) of the 12 columns, the dowel bar positions did not meet tolerance. After having this problem on the first bent (Little River EB Bent 4), a remedy was made for EB Bents 2 and 3 and all WB bents: the template was moved upward towards the ends of the bars, to provide better lateral stability for the bars.

Fig. 2-12. Casting a Column for an Intermediate Bent

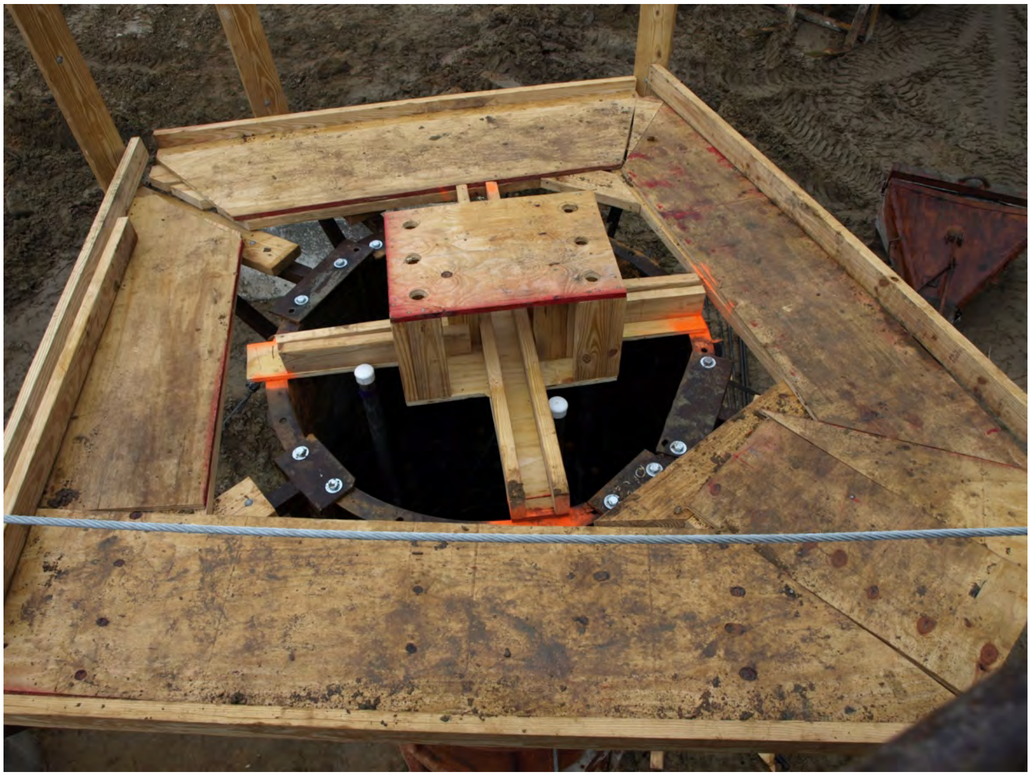


Fig. 2-13. Original Template on Column for Setting Dowel Bars for Little River EB (February 6, 2014)

Figures 2-14 to 2-16 are photos of the completed columns: respectively, a pair of columns for a bent, a close-up view of a column with the six (6) dowel bars at the top, and all six (6) columns for the three (3) Little River WB bridge intermediate bents. **Figure 2-17** is a photo of a column after the precast bent cap and Florida-I Beams (FIBs) have been placed. Bent Cap and FIB construction will be discussed in following sections.

Fig. 2-14. Completed Columns for a Little River EB Intermediate Bent (February 14, 2014)

Inspectors noted the importance of writing the specifications to account for the possibility that a drilled shaft specialty contractor is utilized. In which case, the specialty contractor should have some responsibility for the drilled shaft fit-up with the column. Inspectors also suggested that for column heights more than about 20 ft, it may be better to use driven piles instead of drilled shafts. It was opined that it is easier to get driven piles in the correct lateral position than it is for the column extensions.

Fig. 2-15. Column and Six Dowel Bars Extending from Top on Little River WB (July 21, 2015)

Fig. 2-16. Six (6) Columns for Little River WB Intermediate Bents (July 21, 2015)


Fig. 2-17. Column, Bent Cap, and FIBs on Little River EB (September 18, 2014)

Even if the drilled shaft plumbness is within tolerance (1/4 inch per foot) at ground level, the column might still be out of tolerance when it extends upward from the shaft. On the Little River bridges, in some cases, the cover to the reinforcing steel in the column had to be adjusted. The taller the column, the more this could be an issue.

On Little River EB bridge, the #9 dowel bars were misaligned in both drilled shafts for Bent 4. This would have caused the dowel bars to not fit up with the preformed holes in the bent cap. Fortunately, the bent cap had not yet been cast, so adjustments could be made to the bent cap details. More details are discussed in the following section.

2.3.3 Precast Bent Caps

Precasting of the bent caps is described in detail in Report 2, Fabrication of Precast Elements. In that report, adjustments that were made to the reinforcing steel in bent caps 4EB and 2WB were described. This was to address misalignment of the dowel bars in the columns. Other details about the construction issues and adjustments made to the bent caps are as follows:

- For Little River EB Bent 4, the #9 dowel bars were misaligned in both shafts. Because it had not yet been cast, the precast bent cap was revised for the as-built condition of the dowel bars. The as-built locations of the dowel bars are provided in Report 2, as are figures of the reinforcing steel adjustments that were made in the bent cap. In addition, several of the preformed dowel-bar holes in the bent cap were shifted by 0.5 in. (relative to the design condition) in one or both directions, longitudinally and/or transversely. The maximum amount that any bar was shifted was 1.25 in.

- For Little River WB Bent 2, drilled shaft and column 2 were constructed about 3.6 in. further upstation than design. The dowel bars were placed in the correct locations (i.e. at the correct stations, as surveyed). Correspondence from the engineer was as follows:

  > "It is preferred that the face of the column fall at least inside the bottom edge chamfer of the bent cap. Given the construction tolerances and only 2 1/2" distance between the edge of the chamfer and theoretical face of the column, I would suggest that the Contractor consider either shifting the ducts in the cap or adjusting the cap placement to ensure that column face does not encroach on the chamfer. Since there is only 1.375" clearance between the dowel and the insider face of the ducts, shifting the ducts may be the only option depending on the all other construction tolerances. Since the maximum out-of-position tolerance of the shaft is ± 3" (as assumed in the design), this alignment would normally be achievable without shifting the ducts."

As explained in Report 2, Fabrication of Precast Elements, adjustments were made to the reinforcing steel in the bent cap. In addition, several of the preformed dowel-bar holes in the bent cap were shifted.

- Bent cap 2 at Little River WB bridge was set 1.5 in. too far upstation and was partially grouted before the mistake was discovered. As a remedy, the bearing pads were shifted 1.5 in. so they would be in the correct location relative to the beam, although not to the bent cap. An Engineering Analysis Report was not required, per the engineer's response to the contractor: "The substructure is designed assuming the shafts are at the maximum construction tolerance, which is 3" out of position. Shifting the bearing pads has a similar effect, so I don't think we need any reanalysis."

Recommendations for future projects are as follows:

1. Bent caps should not be cast before the columns are cast. This will enable the precaster to remedy any misalignment issues with the drilled shaft, column, and/or dowel bar construction.
2. Adjustment details should be ready in advance, as part of the shop drawing requirements. This will help to avoid delays that would otherwise result due to time needed for approval of a Request for Information and subsequent shop drawings.
3. A different cap-to-column detail should be investigated.

Each cap was delivered to the site on the same day that it was erected. The bent cap was picked off the delivery truck using one crane on the access road. A two-point pick was used (**Figures 2-18 and 2-19**), so delivering on level ground was not necessary. To erect the caps onto the columns, the crane moved as necessary to the work bridge or ground level under the bridge (**Figures 2-20 and 2-21**).

The cap was then maneuvered over the dowel bars that extended from the tops of the two columns (**Figures 2-22 and 2-23**). Then, it was surveyed to ensure correct location and plumbness (**Figures 2-24 and 2-25**). Two high-density polyethylene (HDPE) pile driving cushions per column were used for shims, at the locations shown in the design plans. The shims were left in place. The cap was then unhooked from the crane. The lifting eyes were cut and grouted.

For Little River EB bridge, the contractor used friction collars with beams as temporary falsework (**Figure 2-26**). Some of the bent caps rested on the shims for several months before they were grouted. The shims did not compact or cause the cap to settle. Because the shims carried the load so well without deforming over time, friction collars were not used for WB bridge construction (**Figure 2-27**).

Fig. 2-18. Setting a Precast Bent Cap Using a Two-Point Pick (July 24, 2015)

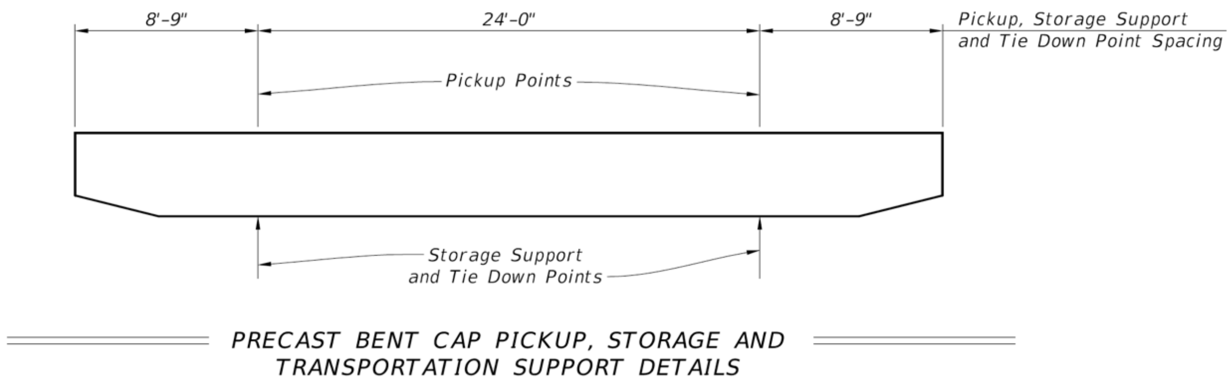


Fig. 2-19. Precast Bent Cap Pickup, Storage, and Transportation Support Details (from Design Plans)

Fig. 2-20. Crane Setting a Precast Bent Cap from Temporary Work Bridge

Fig. 2-21. Crane for Setting a Precast Bent Cap on Little River EB (May 7, 2014)

Fig. 2-22. Setting Precast Bent Cap over Dowel Bars (July 24, 2015)

Fig. 2-23. Tethering a Precast Bent Cap while Setting (May 7, 2014)

Fig. 2-24. Plumbing a Precast Bent Cap during Setting (July 24, 2015)

Fig. 2-25. Surveying a Precast Bent Cap

Fig. 2-26. Friction Collars for Precast Bent Caps on Little River EB (September 11, 2014)

Fig. 2-27. Friction Collars not Used on Little River WB (July 24, 2015)

The gap (connection) between the column and bent cap was then formed (**Figures 2-28 and 2-29**) and grouted (**Figures 2-30 and 2-31**) with a precision non-shrink grout. **Figure 2-32** is a top view of the cap, and **Figure 2-33** shows the grout being placed in the middle, seventh hole. The keeper block next to the bearing pedestal, on the low side of the superelevated bridge, was directly over the column. Pedestal reinforcing steel was placed in the bent cap at the casting yard, but it did not cause too much interference with grouting operations (**Figure 2-34**).

Fig. 2-28. Forms for Column-Bent Cap Connection

Fig. 2-29. Ready to Grout Bent Cap on Little River EB (September 11, 2014)

Fig. 2-30. Grouting Bent Cap on Little River EB, Grout Pump Equipment and Platform

Fig. 2-31. Grouting Bent Cap on Little River EB, Grout Pump Platform Next to Bent Cap (September 11, 2014)


Fig. 2-32. Top of Bent Cap on Little River EB (September 11, 2014)


Fig. 2-33. Grouting Bent Caps on Little River EB, Filling in 7th Hole (September 14, 2014)

Fig. 2-34. Bent Cap on Little River EB, Holes for Grouting 6 Dowel Bars (September 14, 2014)

Getting a good seal on the forms was difficult. The forms often leaked (**Figure 2-35**) and had to be tightened "on the fly", as shown in **Figures 2-36 and 2-37**. Sonotube forms were originally used, but they blew out. Then sonotubes with metal collars were used, but the metal leaked. The contractor ended up using sonotube – cut into two pieces to get it around the column – and clamped a two-piece sheet metal around it. They also caulked the top and bottom of the sonotube to help with the seal. The WB bridges had far less leakage than EB because of the improvements made in the process. The ability to freely tighten the forms, to stop leaks that occurred while grouting, was important.

Fig. 2-35. Leakage in Forms while Grouting Bent Cap on Little River EB (September 14, 2014)

Fig. 2-36. Tightening Forms while Grouting Bent Cap on Little River EB (September 14, 2014)

Fig. 2-37. Tightening Forms while Grouting Bent Cap on Little River EB (September 14, 2014)

Figure 2-38 is a photo of the completed intermediate bent, after forms were removed from the cap-column connection. In **Figure 2-39**, some FIBs have been placed on the bent cap; some of the (dark gray) sealant material that was used to form the connection is still visible at the top of the column.

Some voids occurred between the column and cap, where grout leaked out around the form during grouting. The voids were inspected (**Figure 2-40**) and then repaired by epoxy injection using the previously-approved Dayton Superior Pro-Poxy 100, an epoxy injection resin. This is a high-strength adhesive that is used for concrete repair, and it has low viscosity for deep penetration.

The non-shrink grout did shrink a little bit, causing a gap between the grouted area and the bottom of the bent cap. **Figure 2-41** shows this gap being measured. Voids were documented, as shown in **Figure 2-42**, but only one (1) of the 12 column-to-cap connections needed to be repaired. The contractor submitted a repair procedure: the outside of the gap was caulked, and the gap was pressure grouted with Dayton Superior Pro-Poxy 100, which is good for voids less than 0.25 in.

Fig. 2-38. Completed Intermediate Bent for Little River EB (September 18, 2014)


Fig. 2-39. FIBs Set on Little River EB Bent Cap, Some Sealant is Visible at Top of Column (September 18, 2014)

Fig. 2-40. Inspecting Cap-Column Connection on Little River WB (August 2015)

Fig. 2-41. Measuring Gap in Cap-Column Connection on Little River WB (August 2015)

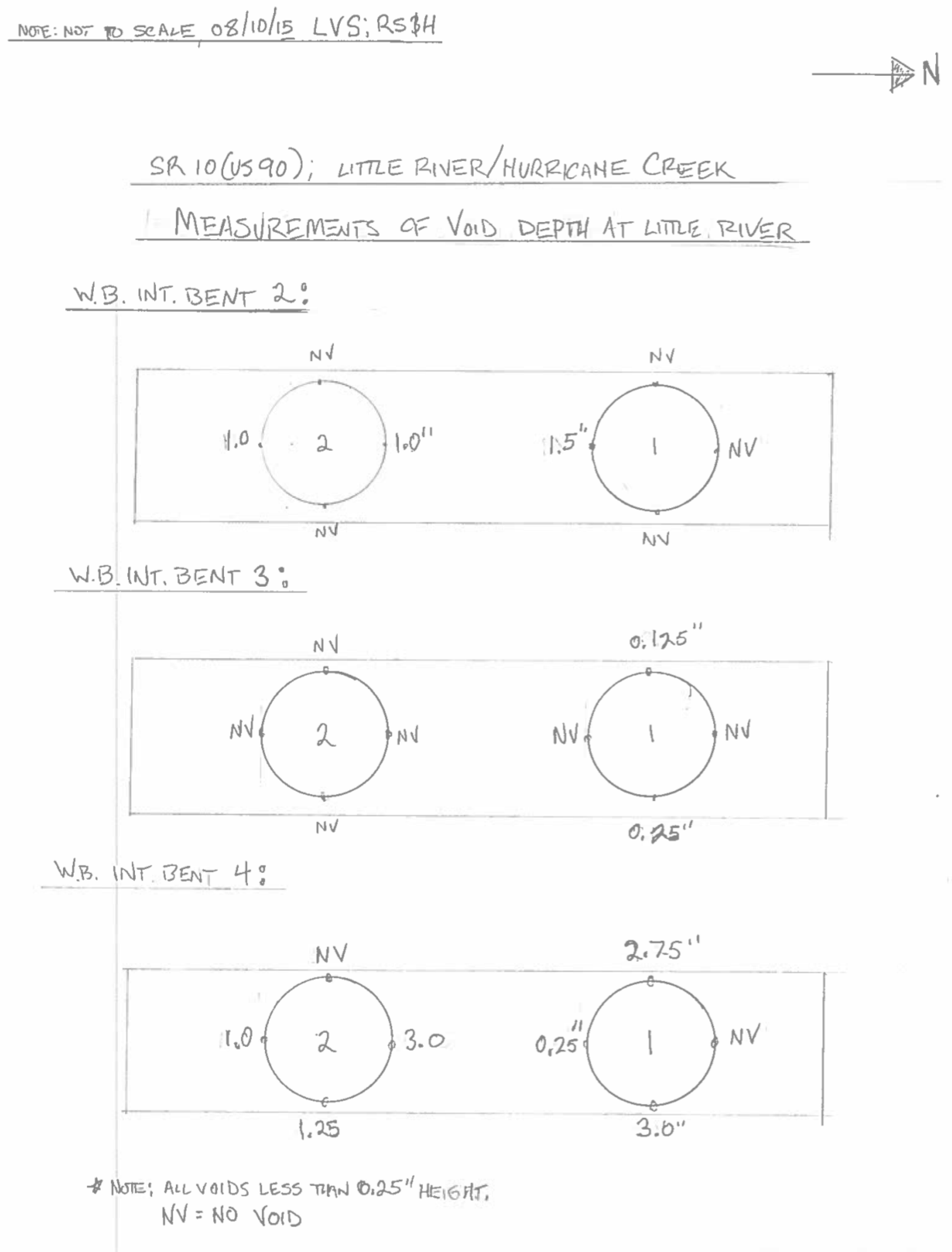


Fig. 2-42. Documentation of Voids in Column-Cap Connection for Little River WB (August 2015)

Overall, erection of the caps was rapid, even considering the care that had to be taken to slowly maneuver them, with the crane, over the dowel bars. For example, on the Little River WB bridge, two (2) bent caps were set in a single day. On another day, the two (2) caps were grouted, and a third cap was set. Inspectors opined that up to four (4) bent caps could easily be set in a single work day – possibly more depending on site conditions and delivery access. The contractor could potentially grout several bent caps in a day.

2.4 Bearing Pad Pedestals

Some "engineering" was required prior to casting the pedestals and setting girders. Before constructing the pedestals, the girder cambers were measured, so that accordingly the pedestal elevations could be revised from the original design plans if needed. Additional adjustments were made, as follows, to accommodate the girder camber and to ensure a reasonable (constructable) haunch thickness between the precast deck panels and girders:

- For Hurricane Creek EB bridge, a vertical curve was added to the profile grade line (PGL), which was originally designed as flat. This raised the deck 0.5 in., to provide a minimum 0.5-inch haunch thickness. Providing a minimum thickness is important for this new type of construction, because the grout needs to be able to flow well in the haunch, between the deck panel bottom and girder top flange. Whereas, for typical bridge construction, the haunch is simply cast with the slab, and therefore, a very thin haunch is tolerable.
- For span 4 of Little River EB bridge, the girders were not erected according to the "number" that they were assigned in the casting yard. They were erected in the order of their cambers, from most to least. This resulted in the pedestal heights being easier to adjust. Reinforcing steel can be more easily adjusted in taller pedestals, while maintaining required concrete cover.

Bearing pad pedestals were formed with plywood and constructed of cast-in-place concrete. On a single bent cap, the five (5) pedestals were constructed in two (2) days, including forming and casting. High-early strength concrete was used, and sufficient strength was gained within about two (2) days, after which time the forms could be stripped, the bearing pads installed, and the girders placed.

CHAPTER 3
SUPERSTRUCTURE – PRECAST ELEMENTS

3.1 Precast Prestressed Florida-I Beams

The girders were lifted using two (2) cranes, one (1) at each abutment (**Figures 3-1 and 3-2**). Each girder was delivered to the site on the same day that it was erected. For Hurricane Creek bridges, the delivery truck parked on the roadway in the opposite lanes (**Figure 3-3**). For Little River bridges, the girders were delivered to the temporary access road (**Figure 3-4**).

Fig. 3-1. Hurricane Creek EB Bridge – Ready for Girder Installation (March 24, 2014)

Photos of the girders being erected, along with the construction equipment used, are in **Figures 3-5 to 3-20**.

Fig. 3-2. Cranes at Abutments for Girder installation on Hurricane Creek EB Bridge – Looking North (March 24, 2014)

Fig. 3-3. Girder Being Lifted from Delivery Truck for Hurricane Creek EB (March 24, 2014)

Fig. 3-4. Delivering Girder to Little River EB (September 18, 2014)

Fig. 3-5. First Girder Installed for Project – Hurricane Creek EB Bridge (March 24, 2014)

Fig. 3-6. Installing Girder on Hurricane Creek EB Bridge (March 24, 2014)

Fig. 3-7. Hurricane Creek EB – View under Bridge (March 24, 2014)

Fig. 3-8. Hurricane Creek EB – Looking West at Installed Girders (March 24, 2014)

Fig. 3-9. Setting Girders on Little River EB (September 18, 2014)

Fig. 3-10. Girder Being Delivered by Truck to Little River EB (September 20, 2014)

Fig. 3-11. Setting Girders on Little River EB using Access Road (September 18, 2014)

Fig. 3-12. Setting Girder on Little River EB (September 18, 2014)

Fig. 3-13. Crane for Setting Girders on Little River EB (September 20, 2014)

Fig. 3-14. Three Spans of Girders Complete on Little River EB (September 20, 2014)

Fig. 3-15. Equipment for Setting Girders on Little River EB (September 18, 2014)


Fig. 3-16. Setting Girder on Little River EB – View under Bridge (September 20, 2014)


Fig. 3-17. Setting Girder on Little River EB – Side View (September 20, 2014)

Fig. 3-18. Maneuvering a Girder into Position

Fig. 3-19. Setting Girder on Bearing – Little River EB (September 18, 2014)

Fig. 3-20. Setting Girder on Little River EB – Camber is Highly Noticeable (September 19, 2014)

The shipping dates are provided in Report 2, Fabrication of Precast Elements. However, the Beam # in that report does not necessarily match the position of the beam as it was erected on the bridges. The beams were switched around according to their measured cambers, so that the cambers would decrease in the direction of the cross-slope.

For EB bridges, the largest-cambered beam was placed on the high side of the bridge, and the smallest-cambered beam on the low side. The actual erected positions are indicated in **Table 3-1** in the "Erected on Beam Line" column. For example, Beam # LR-45-1-1EB was originally intended for Little River Eastbound Bridge, Span 1, Beam Line 1; however, it was placed on Beam Line 5. (For all bridges, WB and EB, Beam Line 1 is to the north, and Beam Line 5 is to the south.) The WB girder erection was per plan; the pedestals were adjusted for the actual girder cambers prior to the cap/abutment construction.

Table 3-1. Beam #s and Erected Positions of FIBs

BRIDGE # / NAME	SPAN # / BEAM LENGTH	BEAM #	SHIPPING DATE	AGE AT SHIPPING (days)	ERECTED ON BEAM LINE (relative to North)
500151 LITTLE RIVER WB	**1 105' 11"**	*LR-45-1-1WB*	8/13/15	638	**1**
		LR-45-1-2WB	8/13/15	636	**2**
		LR-45-1-3WB	8/13/15	636	**3**
		LR-45-1-4WB	8/13/15	632	**4**
		LR-45-1-5WB	8/13/15	632	**5**
	2 109' 10.5"	*LR-45-2-1WB*	8/13/15	624	**1**
		LR-45-2-2WB	8/13/15	624	**2**
		LR-45-2-3WB	8/13/15	623	**3**
		LR-45-2-4WB	8/13/15	623	**4**
		LR-45-2-5WB	8/13/15	618	**5**
	3 109' 10.5"	*LR-45-3-1WB*	8/13/15	618	**1**
		LR-45-3-2WB	8/13/15	617	**2**
		LR-45-3-3WB	8/13/15	617	**3**
		LR-45-3-4WB	8/13/15	616	**4**
		LR-45-3-5WB	8/13/15	616	**5**
	4 109' 11"	*LR-45-4-1WB*	8/14/15	616	**1**
		LR-45-4-2WB	8/14/15	616	**2**
		LR-45-4-3WB	8/14/15	615	**3**
		LR-45-4-4WB	8/14/15	615	**4**
		LR-45-4-5WB	8/14/15	613	**5**
500152 LITTLE RIVER EB	**1 105' 11"**	*LR-45-1-1EB*	9/18/14	313	**5**
		LR-45-1-2EB	9/18/14	313	**3**
		LR-45-1-3EB	9/18/14	310	**4**
		LR-45-1-4EB	9/18/14	310	**1**
		LR-45-1-5EB	9/18/14	309	**2**
	2 109' 10.5"	*LR-45-2-1EB*	9/18/14	302	**2**
		LR-45-2-2EB	9/18/14	302	**3**
		LR-45-2-3EB	9/18/14	301	**5**
		LR-45-2-4EB	9/18/14	301	**4**
		LR-45-2-5EB	9/18/14	300	**1**
	3 109' 10.5"	*LR-45-3-1EB*	9/19/14	301	**2**
		LR-45-3-2EB	9/19/14	300	**1**
		LR-45-3-3EB	9/19/14	300	**5**
		LR-45-3-4EB	9/19/14	298	**3**
		LR-45-3-5EB	9/19/14	298	**4**
	4 109' 11"	*LR-45-4-1EB*	9/20/14	285	**3**
		LR-45-4-2EB	9/20/14	284	**2**
		LR-45-4-3EB	9/20/14	284	**4**
		LR-45-4-4EB	9/20/14	283	**1**
		LR-45-4-5EB	9/20/14	283	**5**
500153 HURRICANE CREEK WB	**1 109' 11.5"**	*HC-45-1-1WB*	6/15/15	614	**1**
		HC-45-1-2WB	6/15/15	613	**2**
		HC-45-1-3WB	6/15/15	608	**3**
		HC-45-1-4WB	6/15/15	608	**4**
		HC-45-1-5WB	6/15/15	606	**5**
500154 HURRICANE CREEK EB	**1 109' 11.5"**	*HC-45-1-1EB*	3/24/14	158	**5**
		HC-45-1-2EB	3/24/14	153	**1**
		HC-45-1-3EB	3/24/14	153	**2**
		HC-45-1-4EB	3/24/14	151	**3**
		HC-45-1-5EB	3/24/14	151	**4**

For Little River WB bridge, the best erection rate for the project was attained: ten (10) beams per day. That rate was achieved two (2) days in a row, meaning that the girders for all four (4) spans of the bridge were erected in two (2) days. After the girders were erected, they were surveyed.

Inspectors noted that the process could be further improved by working on bearing pedestals concurrently with setting beams. (Photos that emphasize the pedestals are in **Figures 3-21 to 3-23**.) As the bent caps are set and grouted, a crew could begin casting the pedestals, and within about two (2) days thereafter, girders could be set. Having separate crews for these two operations could expedite construction.

Completed spans of girders are in **Figures 3-24 to 3-27**.

Fig. 3-21. Girder and End Bent Pedestal – Hurricane Creek EB (March 24, 2014)

Fig. 3-22. Setting Girder on Bearing on Little River EB (September 19, 2014)

Fig. 3-23. Girder on Bearing Pedestal on Little River EB (September 20, 2014)

Fig. 3-24. Girders and Intermediate Bent on Little River EB (September 18, 2014)

Fig. 3-25. Girders on a Span on Little River EB (September 18, 2014)

Fig. 3-26. Girders Erected on All Four Spans of Little River

Fig. 3-27. Twenty Girders on Little River EB (September 20, 2014)

3.2 Precast Deck Panels

3.2.1 Delivery

After the girders were set, the precast deck panels could be erected. Before the panels were delivered, though, the tops of the girders were surveyed at the locations where the transverse closure joints would ultimately be. Each panel was delivered to the site on the same day that it was erected. One (1) panel at a time was delivered and erected, followed by the next panel.

For Little River bridges, the panels were delivered and lifted from the temporary access road (**Figure 3-28**). Each precast deck panel was lifted at eight (8) points, as described in the Report 2, Fabrication of Precast Elements, and shown in **Figures 3-29 and 3-30**. The contractor leveled the ground where the delivery truck (a flatbed trailer) parked for lifting of the panels. This was so that the panels could be picked uniformly without causing unnecessary stresses. Each panel was checked for cracks before it was lifted from the trailer.

Fig. 3-28. Precast Deck Panels Were Delivered to Access Road on Little River WB (August 31, 2015)

Fig. 3-29. Lifting Precast Deck Panel on Hurricane Creek EB (September 8, 2014)

Fig. 3-30. Getting Ready to Prepare Precast Deck Panel for Erection (September 8, 2014)

After a panel was lifted off the trailer, the leveling bolts were set to their theoretical elevations to account for girder camber and girder deflection due to panel weights (**Figure 3-31**). The bolts were immediately greased (**Figure 3-32**), the purpose of which was to help with their removal once the shear pockets were grouted. (The bolts were reused.)

Fig. 3-31. Measuring Leveling Bolt Extension for Adjustment

Fig. 3-32. Greasing Leveling Bolts on a Panel Before Erection (October 30, 2014)

The shipping dates are provided in Report 2, Fabrication of Precast Elements. Note that the Panel # (an example of the panel number markings is shown in **Figure 3-33**) does not necessarily match the position of the panel as it was erected on the bridge. The actual erected positions are indicated in **Tables 3-2 and 3-3** for EB and WB Bridges, respectively. For example, Panel # LR-A35 was originally intended for Little River EB bridge; however, it was placed on Hurricane Creek EB bridge in Panel Position 8, relative to the west end. The reason for switching on Hurricane Creek EB bridge was that some cracking had occurred in the non-prestressed panels. This was the first span that was erected on the project, so the least-cracked panels were placed on it until resolution of the cracking issues was made.

Fig. 3-33. Markings of Precast Panel Number and Casting Date

3.2.2 Erection

Next, the panel was erected on the girders and was checked for cracks that may have occurred during handling. No cracking as a result of the lifting operations was noticed in the panels. On Little River EB bridge, it took about 30 to 45 minutes to set a panel. On Little River WB bridge, 51 panels were set in 3 days: spans 1 and 2 were set on the first day; span 3 and part of span 4 was set on the second day; and span 4 was finished on the third day. Setting operations were not affected by the different panel types, non-prestressed and prestressed. **Figures 3-34 to 3-40** are photos of panels being erected.

Table 3-2. EB (Non-Prestressed) Panel #s and Erected Positions

PANEL # *	SHIPPING DATE	AGE AT SHIPPING (days)	ERECTED ON: LITTLE RIVER EB	ERECTED ON: HURRICANE CREEK EB	SPAN # ERECTED ON	ERECTION POSITION (from West)
HC-A01	10/29/14	269	✔		1	7
HC-A02	10/29/14	269	✔		1	6
HC-A03	10/29/14	267	✔		1	8
HC-A04	10/29/14	267	✔		1	5
HC-A05	10/30/14	266	✔		2	6
HC-A06	10/30/14	266	✔		2	5
HC-A07	10/29/14	263	✔		1	4
HC-A08	10/30/14	264	✔		2	8
HC-A09	10/30/14	263	✔		2	7
HC-A10	10/30/14	263	✔		2	3
HC-A11	9/9/14	211		✔	1	11
LR-A12	10/29/14	261	✔		1	9
LR-A13	10/29/14	260	✔		1	11
LR-A14	10/30/14	261	✔		2	4
LR-A15	10/29/14	258	✔		1	2
LR-A16	10/29/14	258	✔		1	10
LR-A17	10/30/14	258	✔		2	2
LR-A18	11/5/14	264	✔		3	2
LR-A19	11/5/14	263	✔		3	12
LR-A20	10/29/14	256	✔		1	3
LR-A21	11/6/14	262	✔		4	8
LR-A22	11/5/14	261	✔		4	9
LR-A23	11/6/14	261	✔		4	7
LR-A24	11/6/14	261	✔		4	6
LR-A25	11/6/14	260	✔		4	4
LR-A26	11/6/14	260	✔		4	10
LR-A27	11/5/14	258	✔		3	10
LR-A28	11/6/14	259	✔		4	5
LR-A29	11/5/14	257	✔		3	9
LR-A30	11/5/14	257	✔		4	3
LR-A31	11/5/14	254	✔		3	8
LR-A32	11/5/14	254	✔		3	11
LR-A33	11/5/14	253	✔		3	3
LR-A34	9/8/14	195		✔	1	12
LR-A35	9/8/14	193		✔	1	8
LR-A36	9/8/14	193		✔	1	6
LR-A37	11/5/14	243	✔		4	2
LR-A38	9/9/14	186		✔	1	9
LR-A39	9/8/14	184		✔	1	4
LR-A40	9/8/14	184		✔	1	5
LR-A41	11/5/14	240	✔		3	5
LR-A42	11/6/14	241	✔		4	12
LR-A43	9/8/14	181		✔	1	7
LR-A44	10/30/14	233	✔		2	10
LR-A45	10/30/14	231	✔		2	11
LR-A46	10/30/14	231	✔		2	12
LR-A47	9/8/14	178		✔	1	3
LR-A48	10/30/14	230	✔		2	9
LR-A49	11/5/14	232	✔		3	4
LR-A50	11/6/14	233	✔		4	11
LR-A51	11/5/14	231	✔		3	6
LR-A52	11/5/14	231	✔		3	7
LR-A53	9/8/14	172		✔	1	2
LR-A54	9/9/14	173		✔	1	10
B01	10/29/14	211	✔		1	12
C01	10/30/14	212	✔		2	1
C02	10/30/14	211	✔		2	13
C03	11/19/14	229	✔		4	13
B02	10/29/14	208	✔		1	1
C04	11/5/14	211	✔		3	1
C05	9/8/14	152		✔	1	1
C06	9/9/14	152		✔	1	13
C07	11/5/14	208	✔		3	13
C08	11/5/14	207	✔		4	1

* Panels were switched around for erection, so Designation HC or LR does not indicate which bridge the panel was constructed on.

Table 3-3. WB (Prestressed) Panel #s and Erected Positions

PANEL # *	SHIPPING DATE	AGE AT SHIPPING (days)	ON BRIDGE: LITTLE RIVER WB	ON BRIDGE: HURRICANE CREEK WB	SPAN # ERECTED ON	ERECTION POSITION (from West)	Panel # on CDS's Inspection Report
HC-D01	6/26/15	476		✔	1	2	LR-D1
HC-D02	6/26/15	476		✔	1	3	LR-D2
HC-D03	6/26/15	474		✔	1	5	LR-D3
HC-D04	6/26/15	474		✔	1	6	LR-D4
HC-D05	6/26/15	471		✔	1	9	LR-D5
HC-D06	6/26/15	471		✔	1	4	LR-D6
HC-D07	6/26/15	469		✔	1	11	LR-D7
HC-D08	9/18/15	553	✔		1	7	LR-D8
HC-D09	9/18/15	546	✔		1	8	LR-D9
HC-D10	9/18/15	546	✔		2	12	LR-D10
HC-D11	6/26/15	461		✔	1	10	LR-D11
LR-D01	6/26/15	461		✔	1	12	LR-D12
LR-D02	9/18/15	543	✔		2	5	LR-D13
LR-D03	9/18/15	543	✔		1	5	LR-D14
LR-D04	9/18/15	542	✔		1	3	LR-D15
LR-D05	9/18/15	542	✔		1	9	LR-D16
LR-D06	8/27/15	519	✔		4	2	LR-D17
LR-D07	8/27/15	519	✔		4	3	LR-D18
LR-D08	8/31/15	522	✔		3	4	LR-D19
LR-D09	8/31/15	522	✔		3	3	LR-D20
LR-D10	6/26/15	453		✔	1	7	LR-D21
LR-D11	6/26/15	453		✔	1	8	LR-D22
LR-D12	8/31/15	410	✔		3	2	LR-D23
LR-D13	9/18/15	428	✔		2	11	LR-D24
LR-D14	8/31/15	409	✔		3	9	LR-D25
LR-D15	8/31/15	409	✔		3	7	LR-D26
LR-D16	9/18/15	424	✔		1	6	LR-D27
LR-D17	9/18/15	424	✔		1	4	LR-D28
LR-D18	8/31/15	405	✔		3	5	LR-D29
LR-D19	9/18/15	423	✔		1	10	LR-D30
LR-D20	9/18/15	422	✔		2	3	LR-D31
LR-D21	9/18/15	422	✔		2	6	LR-D32
LR-D22	9/18/15	421	✔		2	7	LR-D33
LR-D23	8/27/15	399	✔		4	10	LR-D34
LR-D24	8/27/15	398	✔		4	12	LR-D35
LR-D25	8/27/15	398	✔		4	8	LR-D36
LR-D26	9/18/15	417	✔		1	11	LR-D37
LR-D27	9/18/15	417	✔		2	4	LR-D38
LR-D28	9/18/15	416	✔		2	2	LR-D39
LR-D29	9/18/15	416	✔		2	10	LR-D40
LR-D30	9/18/15	415	✔		1	2	LR-D41
LR-D31	8/27/15	393	✔		4	6	LR-D42
LR-D32	8/27/15	392	✔		4	11	LR-D43
LR-D33	8/27/15	392	✔		4	5	LR-D44
LR-D34	8/27/15	388	✔		4	9	LR-D45
LR-D35	8/27/15	388	✔		4	7	LR-D46
LR-D36	8/31/15	391	✔		3	6	LR-D47
LR-D37	8/31/15	391	✔		3	8	LR-D48
LR-D38	8/31/15	390	✔		3	12	LR-D49
LR-D39	8/27/15	386	✔		4	4	LR-D50
LR-D40	9/18/15	407	✔		2	9	LR-D51
LR-D41	9/18/15	407	✔		2	8	LR-D52
LR-D42	8/31/15	387	✔		3	11	LR-D53
LR-D43	8/31/15	387	✔		3	10	LR-D54
LR-E01	9/18/15	399	✔		1	1	LR-E01
LR-E02	9/18/15	399	✔		1	12	LR-E02
LR-F01	8/27/15	373	✔		4	12	LR-F01
LR-F02	8/31/15	377	✔		3	13	LR-F02
LR-F03	8/27/15	372	✔		4	1	LR-F03
LR-F04	9/18/15	394	✔		2	1	LR-F04
LR-F05	8/31/15	375	✔		3	1	LR-F05
LR-F06	9/18/15	393	✔		2	13	LR-F06
LR-F07	6/26/15	308		✔	1	13	LR-F07
LR-F08	6/26/15	308		✔	1	1	LR-F08

* Panels were switched around for erection, so Designation HC or LR does not indicate which bridge the panel was constructed on.

Fig. 3-34. View of Panel's Underside While Being Lifted to Set on Girders

Fig. 3-35. Setting Panels on Hurricane Creek WB (June 26, 2015)

Fig. 3-36. Setting Precast Deck Panels on Hurricane Creek EB (September 8, 2014)

Fig. 3-37. Setting Panels on Little River EB (October 30, 2014)

Fig. 3-38. Setting Panels on Girders, Leaving Gaps in Between for Closure Joints

Fig. 3-39. Precast Deck Panels on Little River WB – Showing Closure Joints in Between

Fig. 3-40. Setting Precast Deck Panels on Hurricane Creek EB (September 9, 2014)

When a panel was being placed on the girders, the hoop bars (shown in Figure 3-34) – which protruded from the panel's vertical face that would eventually form the closure joint – reportedly did not interfere with the hoop bars in the adjacently-placed panel. In the design, the hoop bar spacing was detailed to begin (relative to the panel's short edge) at 2 in. on one long edge and at 4 in. on the other long edge. This resulted in the hoop bars being staggered in the closure joint, as shown in Figure 3-39, with a nominal 1.5-inch clear space between them. The design spacing also permitted the panel to be placed in either direction (i.e. at 0° or 180°) without bar interference. Also, during panel erection, the shear pockets were placed relatively easily over the reinforcing steel that protruded from the girder top flanges that would eventually create the composite connection between the panels and girders (see the later section, "Shear Pocket and Haunch Grouting", for more details and photos).

The panels were not set directly on the girders, because space was needed for the haunch in between these two precast elements. Polyethylene forms were precut (with a hotwire) to height, considering the measured beam camber and final desired elevations. They were placed on top of the girders, and the panels were set on them (**Figure 3-41**). In addition to serving as a resting place for the panels, the forms were to also function as a seal during grouting operations. More details about grouting and the forms will be discussed later.

Fig. 3-41. Carefully Positioning Panel on Polystyrene Forms on Girders

3.2.3 Leveling Bolts

In the casting yard, the blockouts for the leveling bolts were anchored at the top, but they were not anchored at the bottom. It appears that the blockouts shifted at the bottom during panel casting, because some of the bolts were misaligned, as shown in **Figures 3-42 and 3-43**, and therefore did not make contact with the 6" x 6" Embedded Plate Assemblies in the FIB top flange (**Figure 3-44**). These steel plates were embedded during girder casting operations, specifically for the purpose of providing an area on which the panel could rest until the haunches were grouted. Tolerances, the sweep of the FIB, and shortening of the beam caused by creep and shrinkage could contribute to a condition in which the bolts do not contact the plates.


Fig. 3-42. Leveling Bolts Slightly Misaligned

The inspectors could not easily see the misaligned bolts, because they were underneath the panels in the haunch region. Inspectors used a smartphone (set on video mode) to look underneath the panels at the bolts, to check if they were making contact with the steel plates. The inspectors did this constantly as the panels were being set. To resolve the immediate issue, the panel was shifted so that the bolt would make contact; this slightly changed the adjacent closure joints' dimensions. Where the panel could not be shifted enough, the inspectors made notes of the misalignments. A retrofit was made at the location where the leveling bolt made contact with the girder – usually the next day. The panel had to be picked up, the girder top flange was grinded, and a new plate was epoxied onto the beam. This issue is also discussed in Report 2, Fabrication of Precast Elements.


Fig. 3-43. Close-up of Misaligned Leveling Bolts


Fig. 3-44. Leveling Bolt Misaligned and Not in Contact with Steel Plate on Girder Top Flange

3.2.4 Surveying and Leveling Bolt Adjustments

After initially setting all the panels in a span, the girders were surveyed (**Figure 3-45**). Survey measurements were taken at the closure joints in between the panels, on the girders' top flanges. It took about three (3) hours to survey a span, and another three (3) or four (4) hours to adjust the bolts until the survey on the girders appeared to be acceptable. The desired outcome of these survey adjustments was proper load distribution to the girders, so that all five (5) girders would be equally loaded.

Fig. 3-45. Surveying After Placing Deck Panels

Distributing the load equally to the girders was challenging, required iteration, and involved communication between the contractor and design engineer. The initial elevations described above were sent to FDOT for approval. FDOT responded back with bolt adjustments for every panel-beam intersection. Bolts were then turned accordingly, to raise or lower the panel, as shown in **Figure 3-46**. Adjustments ranging from 0 in. to 5/8 in. were necessary. The girders were then resurveyed, and the data was sent to FDOT.

Bolts were readjusted as needed. This typically required two iterations before final acceptance.


Fig. 3-46. Adjusting Leveling Bolts to Raise/Lower Deck Panels

The design plans provided theoretical values for build-up (i.e. haunch thicknesses), beam cambers, and beam deflections due to dead loads. The theoretical dead load deflections included the weights of the deck panels, haunches, and closure joints. This was to help the contractor with monitoring girder deflections as the panels were being placed, as well as with placing haunch forms to a proper height so that the final grade elevations of the panels' top surfaces could be achieved. However, surveying was performed when the deck panels were being placed; the haunches and closure joints were not yet cast. So, for construction purposes, the contractor needed the theoretical deflections due *only* to the deck panel weights. These theoretical numbers were not provided in the design plans (they were provided during construction when the contractor requested them), but perhaps should be in the design plans for future projects.

For the 110-ft span Hurricane Creek EB and WB bridges, the theoretical mid-span deflection due to the deck panel weight was 2.07 in. for the exterior girder and 2.32 in. for the interior girder – based on 8.5-ksi concrete strength. If the girder concrete strength significantly exceeds the design strength, however, then the expected deflections would need to be modified based on the measured strength. Perhaps for future projects a note could be made in the design drawings that **the cambers are based on 8.5 ksi and could vary significantly if the strengths are greater**. Nonetheless, FDOT provided the revised values to the contractor within a couple of days of being requested, so this did not cause construction delays.

3.2.5 Safety Lines

Before a panel could be placed, any safety line poles that were in the way had to be removed from the girders. **Figure 3-47** shows a safety line pole in the foreground. As each pole was removed, the embedded steel pipes that secured the poles to the girder top flange were cleaned and grouted (**Figure 3-48**). (Then the panel was set in place.)

Fig. 3-47. Setting Precast Deck Panels on Polystyrene Forms; Safety Line Pole in Foreground

Fig. 3-48. Filling Steel Pipes with Grout After Safety Line Pole Was Removed

If the pipe was located in a closure joint, then it could be cleaned and grouted later, since it would be accessible even after the panels were set. During construction, dirt and debris tended to fill the pipes, so it is advisable to put a cap on the pipe until it can be grouted. Also, in hindsight, it may have been better to put the safety line poles only at the closure joint locations, and not in the panel locations, so that the panel would not obscure the holes. This would enable easier cleaning, and all grouting could be done after all panels were placed. However, safety line pole design is typically considered part of a contractor's means and methods – nonetheless, the bridge design engineer could call attention to this issue in the design plans but leave it to the contractor's discretion.

3.2.6 Non-Prestressed vs. Prestressed Deck Panels

The EB bridges utilized non-prestressed precast deck panels. Many of the panels developed cracking during fabrication and storage, typically in the short direction along the shear pocket and lifting point locations. This issue is further discussed in Report 2, Fabrication of Precast Elements. Used on the WB bridges, the prestressed panels did not show significant cracking during handling in the casting yard or at the bridge construction site.

Inspectors noted that adjusting the WB panels based on survey data was more difficult than for EB. The EB (non-prestressed) panels were reportedly "more responsive" to bolt adjustments, whereas the WB (prestressed) panels were "stubborn". When both types of panels were erected, it seemed that they loaded the exterior girders proportionally more than the interior girders, and bolt adjustments were made to remove load from exterior girders and add it to interior girders. Adjustments were made more easily to the EB panels than the WB panels. The "stubbornness" of the WB panels could be due to camber or structural characteristics being different than for the EB panels. In the two paragraphs below, additional hypotheses are provided to explain the difficulties encountered with the WB panels.

The WB panels may have had some unintended camber because the prestressing strands were a little bit eccentric to the neutral axis when they were cast. (The strands were designed to not be eccentric, once the extra 0.5 in. was removed via milling and grooving the panel's top surface once the bridge construction was completed). The extra 0.5-in. thickness effectively causes a 0.25-in. strand eccentricity, which is minimal. However, some of the panels were cast thicker than the 8.5-inch design thickness. Some were as much as 9 in. thick (**Figure 3-49**). This would double the prestressing strand eccentricity to 0.5 in., assuming that the strand location relative to the bottom of the slab was consistently 4 in. regardless of the slab thickness.

As discussed previously, the bolts' theoretical elevations were set after the panel was lifted off the trailer. The bolt extensions were measured from the bottom surface of the panel. However, if the panel is cambered upward, due to reasons explained above, this would cause the bolts over the interior girders to be too short, relative to those over the exterior girders. This would cause the panel, when being erected, to put a disproportionate (larger) load on the exterior girders. For future projects, the contractor should consider pulling a string line underneath the panel (in the long direction), and measuring the bolt extensions relative to the string line, rather than relative to the panel's bottom surface. This could help accommodate any camber in the panel and ensure a more equal distribution of load to the girders.

Fig. 3-49. Some Panels Were Cast as Much as 9 in. Thick

3.2.7 Forms

Polyethylene forms were placed on top of the girders (**Figures 3-50 and 3-51**), and then the panels were placed on the forms (**Figures 3-52 to 3-54**). The forms were precut to height, considering measured beam camber and final desired deck panel elevations. Each form strip was precut to a height at which it would compress about 40% under the panel weight, so that it would form a seal for the grout that would be placed later. The 40% compression depended on the load that the panel placed on the girder, though.

Fig. 3-50. Laying Polystyrene Forms

Fig. 3-51. Close-up of Polystyrene Forms


Fig. 3-52. Setting Precast Deck Panels on Hurricane Creek EB – Polystyrene Forms in Foreground


Fig. 3-53. Setting Precast Deck Panels on Hurricane Creek EB (September 8, 2014)


Fig. 3-54. Setting Precast Deck Panels on Hurricane Creek EB (September 9, 2014)

Longitudinal direction (parallel to girders): The method of using polyethylene strips parallel to the girders was used on all bridges. However, the cutting method was changed to improve the process. Initially, each parallel strip was cut to about the length of the precast panel; strips were placed only along the panel lengths and not along the 1-ft-wide closure joints. For later spans, the strips were placed along the entire girder length – including along the closure joints, as shown in **Figure 3-55**. In lieu of using polyethylene strips, the contractor tried a method of dry packing the haunch (parallel to the girders), but it was time consuming. Too much time had to be spent in a manlift, working above the head.

Initially, caulk was used to adhere the forms to the girder top flange, but it secured the forms too well, making later removal of the forms difficult. Loctite Pro Line 300 Foamboard construction adhesive was then used instead, because it secured the forms well enough but made removing the polyethylene easier. The adhesive sealed well and did not leak during grouting. The good seal was made possible because the girder top flange was smoothed longitudinally along its edges while in the casting yard (**Figure 3-56**).

After grouting operations (to be discussed later) were completed, the polyethylene form strips that were parallel to the girders (shown in **Figure 3-57 and 3-58**) were manually pried out using a crowbar (**Figure 3-59**). This required access to underneath the bridge using a manlift and was time consuming. When the haunch was at least an inch thick, removal of the polyethylene was relatively easy (because the crowbar would fit), but when the haunch was less than about 0.5 in. thick, removal was difficult.

Figures 3-60 and 3-61 show the haunch after the polyethylene was removed. In Figure 3-60, the grout had fully filled the haunch, which was typically the case when spray foam sealant had not been added. However, in Figure 3-61, notice the yellow-colored remains: this is from the spray foam sealant that had been added to prevent grout leakage where the polystyrene did not form an adequate seal. The spray foam sealant expanded too much and caused the void that is visible in the photo. This is an example of the several voids that occurred as a result of overexpansion of the spray foam.

Fig. 3-55. Polystyrene Forms Placed Along Entire Girder Length on Hurricane Creek WB

Fig. 3-56. Edge of Girder Top Flange Was Smoothed Longitudinally During Precasting

Fig. 3-57. Polystyrene Forms in Haunch

Fig. 3-58. Polystyrene Forms (white) and Spray Foam Sealant (yellow) for Haunch

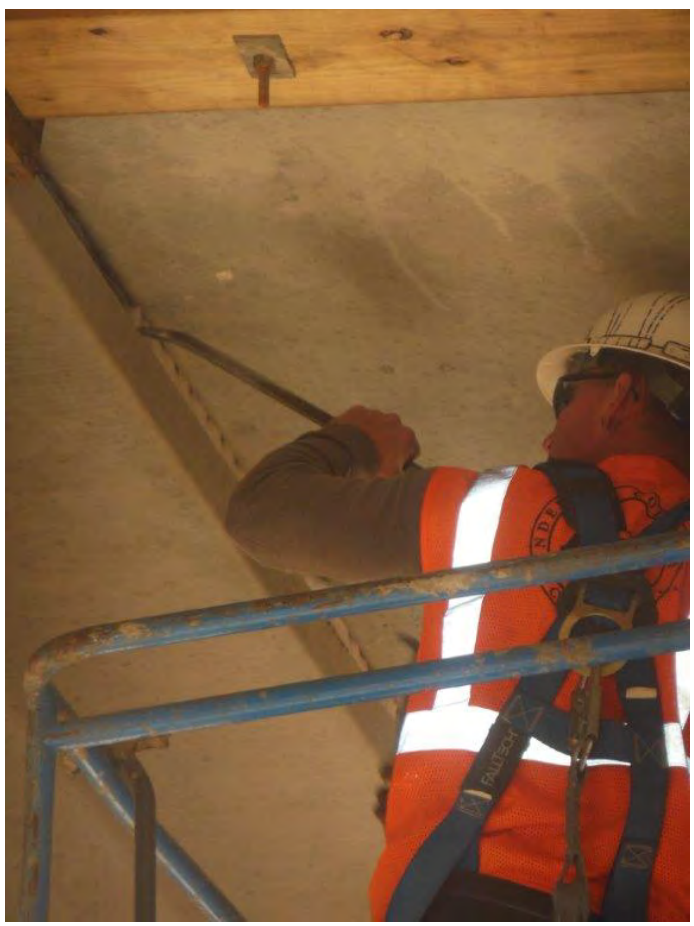


Fig. 3-59. Removing Polyethylene Forms from Haunch

later.

Fig. 3-60. Polyethylene Forms Removed from Haunch (to the Right of the Grout Port Left Behind)

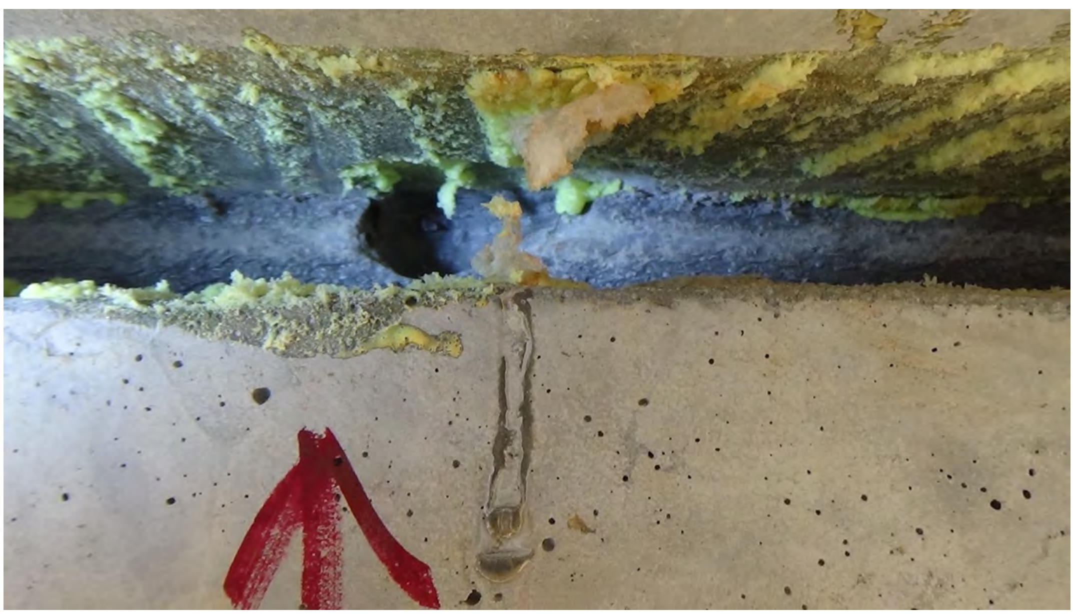


Fig. 3-61. Void in Haunch Left by the Spray Foam Sealant on Hurricane Creek EB

Because voids were found where the spray foam was used on Hurricane Creek EB bridge, the contractor was directed to remove the polyethylene forms and spray foam from *all* girders and spans. (Specification 404-5.3 required removal at only a few representative locations.) Removal of all forms was also required for the other three bridges constructed later.

The haunches were inspected, and any voids were documented – including their lengths, widths, and depths.

Void maps for Little River EB bridge Spans 1-4 are summarized as follows:

- Amplitudes were mostly 1/2 in.
- For Span 3, out of 130 panel lines (13 panels x 5 girders x 2 sides/girder line = 130 panel lines), 8 had voids that extended at least half the length of the panel.
- The typical void depth was 3 to 6 in.
- Some void depths were 12 in. or greater, at about 20 locations.

Void maps for Hurricane Creek WB bridge are summarized as follows:

- Amplitudes were 1/8 to 1/4 in.
- Locally, voids were 5 in. to 8 in. deep at some places.
- Out of 130 panel lines (13 panels x 5 girders x 2 sides/girder line = 130 panel lines), 26 had voids that extended at least half the length of the panel.
- The typical void depth was 2 to 5 in.
- Some void depths were 12 in. or greater, at about 25 locations.

The contractor was then instructed on which voids needed repair. The contractor submitted two (2 repair procedures to the designer for approval, depending on the void's size:

- Voids with heights less than 0.5 in.: Use Pro-Poxy 100.
- Voids with heights greater than 0.5 in.: Form with dry pack, grout through an insert (plus an outlet), then seal it off.

For future projects, the inspector recommended to use easily-removable forms, instead of dry pack, for the longer voids; however, this would require attachment to the girder top flange.

Transverse direction (perpendicular to girders): Initially, polyethylene form strips were used to seal the gap between the precast panel and girder top flange in the direction perpendicular to the girders, as shown in Figures 3-50 and 3-51. However, leaks occurred, partly because the top of the girder was intentionally roughened in the casting yard with a rake. Also, the forms needed to be removed so that the closure joints could be cast, but removal was difficult. The reinforcing steel that protruded from the sides of the precast panels and into the closure joint region (**Figure 3-62**) made it difficult to get in a crowbar to pry out the strips. (For this reason, inspectors noted that the *additional* reinforcing steel, as detailed in the design plans, should be placed in the closure joint *after* the forms are removed so that it does not make form removal even more difficult.) The process was later improved by using – instead of polyethylene – wooden forms and caulk perpendicular to the girders (**Figure 3-63**). However, the wood did not seal well because of the irregular rough surface of the girder top flange. Ultimately, the contractor improved the method further by using a dry pack to form the gap; this formed a good seal for grouting (**Figure 3-64**).

Fig. 3-62. Original Forming Method Using Polystyrene to Grout Shear Pockets on Hurricane Creek EB


Fig. 3-63. Revised Forming Method to Grout Shear Pockets on Little River EB


Fig. 3-64. Dry Pack Forming to Grout Shear Pockets on Little River WB

CHAPTER 4

SUPERSTRUCTURE – GROUTING, CLOSURE JOINTS, AND FINISHING

4.1 Shear Pocket and Haunch Grouting

After the panels were set, surveyed, and adjusted (**Figure 4-1**), the shear pockets and haunches were grouted. This formed a composite connection between the panels and girders, by way of reinforcing steel that protruded from the girder top flange into the shear pockets, as well as by horizontal shear friction between the panels, girders, and haunch surfaces.

Before the pockets were grouted on the EB bridges, additional #4 bars were wrapped around the "J" bars that protruded from the FIBs (**Figure 4-2** shows a pocket with the additional bars, and **Figure 4-3** shows a pocket without them). This fortified the connection to the FIB and alleviated some concern that any cracks in the panel would eventually propagate through the shear pockets. (As previously discussed, the non-prestressed panels on the EB bridges experienced some cracking while in the casting yard.)

Fig. 4-1. Panels Installed and Getting Ready to Grout Shear Pockets on Hurricane Creek EB


Fig. 4-2. Additional #4 Bars Wrapped Around the "J" Bars to Fortify Connection to the FIB – Hurricane Creek EB


Fig. 4-3. Shear Pocket Without Additional #4 Bars on Little River EB

In the subsections that follow, the grout material, equipment, process, and sampling requirements will be described. Problems that were encountered and lessons learned will also be discussed.

4.1.1 Material

According to Special Provision (SP) 934, the contractor had to submit test results for the selected grout material (see **Table 4-1**). This was regardless of whether or not the grout was on FDOT's Qualified Products List (QPL). As stated by Nolan (2014):

> *The grouting specifications for this project were developed using the NCHRP Report 681 Attachment CS with additional modifications recommended by the FDOT State Materials Office. The standard pre-approved non-shrink grouts on the FDOT Qualified Products List did not meet the minimum desired specification requirements, which necessitated the implementation of {Special Provision 934} and submittal of additional test reports for approval of the Contractor's proposed grout. Pre-approved grouts would expedite construction on more time-sensitive installations...*

Table 4-1. Precision Grout Performance Requirements per Special Provision 934-4.1

Testing Method	Property	Test Value
ASTM C109 or ASTM C942	Compressive Strength 3 Days	4,000 psi minimum
	Compressive Strength 28 Days	6,750 psi minimum
ASTM C496	Splitting Tensile Strength 28 Days	650 psi minimum
ASTM C191	Time of set, initial	4 hours minimum
	Time of set, final	8 hours maximum
FM 5-516	Chloride Content	0.40lb/yd^3 maximum
ASTM C1090	Volume Change at 1, 3, 14 Days	< Height Change at 28 Days
ASTM C940	Bleeding, Final	0.0% at 3 Hours
ASTM C939	Efflux Time	20 – 30 seconds
ASTM C666	Freeze Thaw	300 cycles, RDF 90%
ASTM C531	Coefficient of Thermal Expansion	3.0 – 10.0 x 10^{-6} psi

The contractor ultimately used MasterFlow® 928, a high-precision mineral-aggregate grout with extended working time, made by Master Builders. (They initially ordered Lambert Vibropruf® #11, but testing showed that it did not meet SP 934.)

4.1.2 Equipment

To mix and pump the grout, the following pieces of equipment were used: portable generator for power (model Multiquip GA-6H); portable air compressor (model Sullair 185); water storage unit; pump (model Wacker Neuson PT3); and mixer/pump (by Chem Grout). The same equipment was used for grouting the bent caps and is shown in **Figures 4-4 to 4-10**).

There were a few problems with the equipment. The air compressor sometimes broke and needed to be repaired. The lines got plugged, and the hose got dry packed. The auger (**Figure 4-11**) wore out and had to be replaced each time after mixing about 500 bags of grout. (This repair was expensive.)

Fig. 4-4. Equipment for Grouting

Fig. 4-5. Water Storage Unit for Grouting

Fig. 4-6. Pump for Grouting

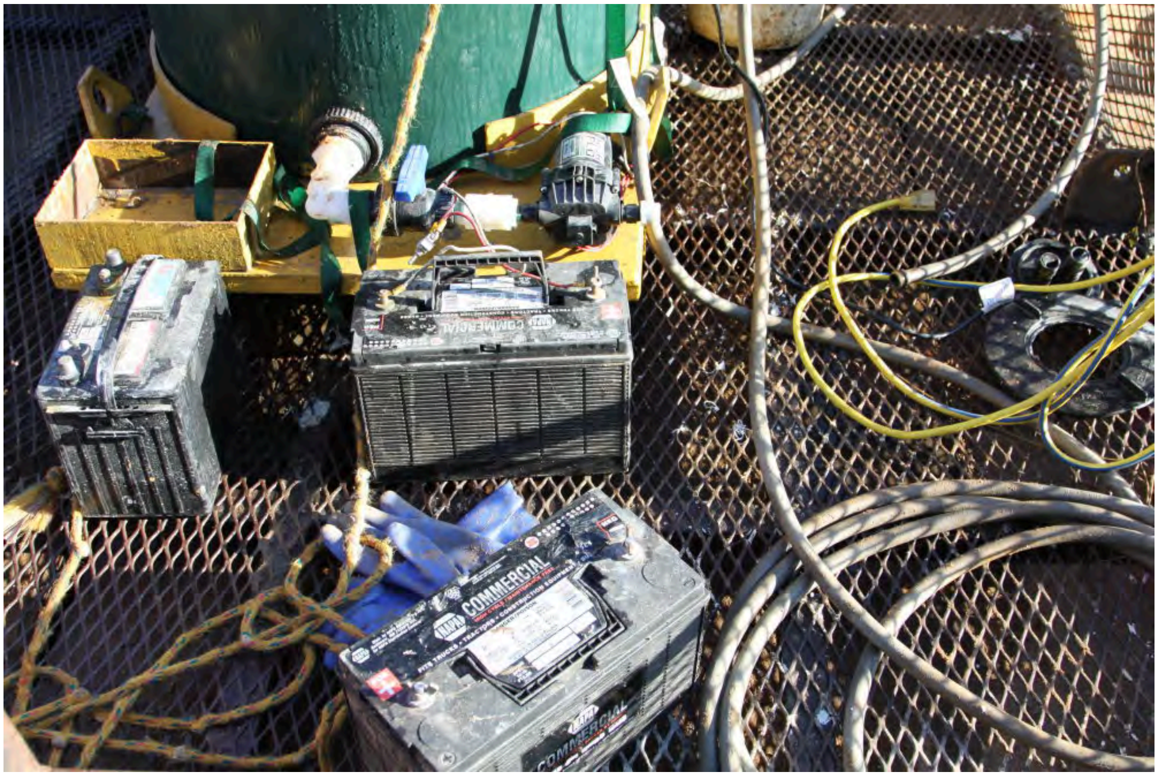


Fig. 4-7. Grout Pump Platform (Batteries, etc.)

Fig. 4-8. Grouting Platform and Mixer/Pump

Fig. 4-9. Grout Mixer/Pump and Bags of Grout

Fig. 4-10. Grout in Mixer/Pump


Fig. 4-11. Auger from Grout Mixing Equipment

4.1.3 Process

For a particular panel, each pair of shear pockets on a girder line was grouted along with its haunch in a single grouting operation (**Figure 4-12**). Then the next pair of pockets (with haunch) on that panel was grouted, and so on. For one of the spans, for example, 4.5 panels were grouted in one (1) day, and the rest of panels were grouted the next day.

Before grouting commenced, the perimeter of the shear pocket was sprayed with an epoxy bonding agent (**Figure 4-13**). The grout is a very fluid material, so it easily leaked if the forms between the panel and girder top flange were not very tightly sealed. A tarp was placed underneath the bridge to prevent grout from leaking into the creek or river below (**Figure 4-14**). Where the polystyrene form did not provide a good enough seal, sprayed foam sealant was added (**Figure 4-15**). This was problematic later on, though, when the forms were removed. In some locations, the sealant had expanded excessively into the haunch, so it left voids in the haunch where the grout did not flow.

Fig. 4-12. Grouting a Pair of Panel Pockets on Hurricane Creek WB

Fig. 4-13. Spraying Epoxy Bonding Agent Around Perimeter of Shear Pocket

Fig. 4-14. Tarp Placed Underneath Bridge to Catch Leaking Grout

Fig. 4-15. Spray Foam Sealant Being Added to Haunch Forms to Prevent Leaks While Grouting Shear Pockets on Hurricane Creek EB

Ports were placed protruding from the polystyrene forms. **Figure 4-16** shows the ports before the panels were placed, and **Figure 4-17** shows a port that has been plugged (see the plug in **Figure 4-18**). The process was later improved by using small pipes that extended into the closure joints. They were placed on both sides of the panel (**Figures 4-19 and 4-20**), so that grout could either be fed into them (**Figures 4-21 and 4-22**), or could flow out once the haunch and pockets were filled. After it appeared that the haunch was filled, and the grout began to rise into the shear pocket, then grout was placed directly into the pocket until filled (**Figure 4-23**). The grout was very fluid and tended to flow across the panel once the pocket was full. The contractor prevented the grout from flowing by placing a bead of caulk around the low side of the shear pocket (**Figures 4-24 and 4-25**).

The grouting process required several crew members: at least two (2) to run the grout machine; two (2) to hold the hoses and place the grout in the pockets; one (1) to watch for leaks underneath the bridge; and others. **Figure 4-26** shows a panel being completed.

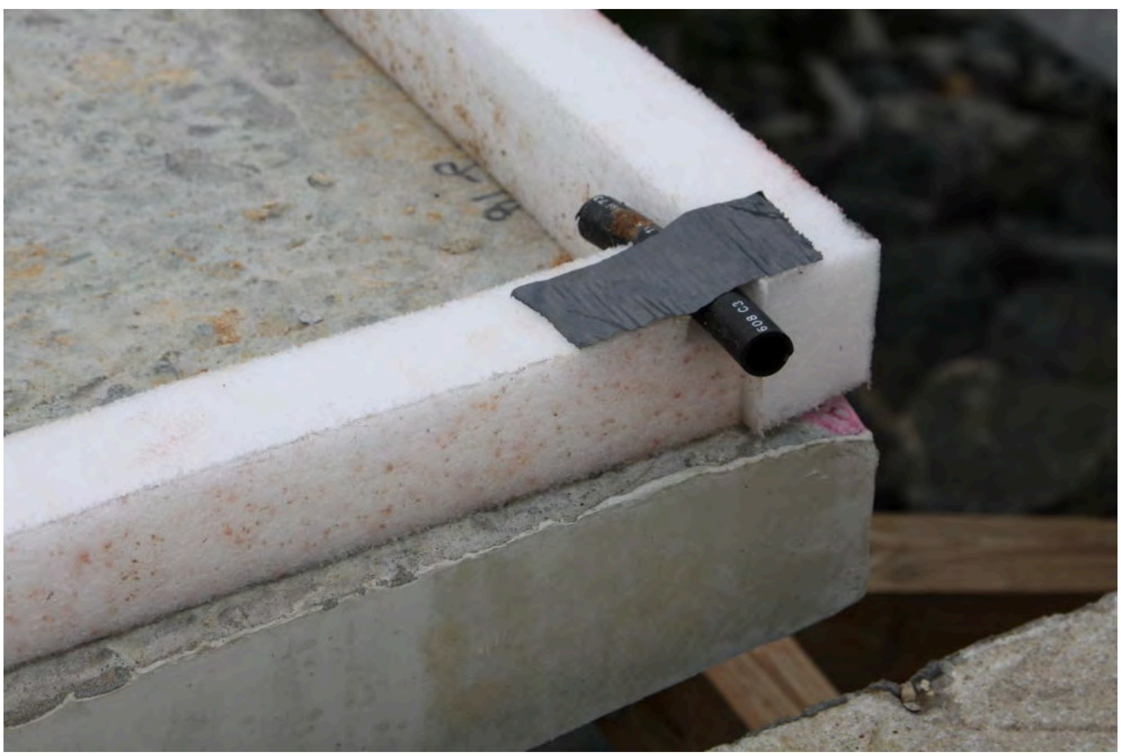


Fig. 4-16. Grout Port in Polystyrene Forms for Grouting Shear Pockets on Hurricane Creek EB, Before Placing Deck Panel

Fig. 4-17. Plugged Grout Port for Grouting Shear Pockets on Hurricane Creek EB

Fig. 4-18. Plug for Grout Port

Fig. 4-19. Pipes for Grouting Shear Pockets on Hurricane Creek WB

Fig. 4-20. Pipes for Grouting Shear Pockets on Little River WB


Fig. 4-21. Feeding Grout into Pipes for Shear Pockets


Fig. 4-22. Feeding Grout into Pipes for Shear Pockets on Hurricane Creek WB

Fig. 4-23. Placing Grout Directly into a Shear on Hurricane Creek EB

Fig. 4-24. Bead of Caulk Around Low Side of Shear Pocket to Stop Grout on Little River WB

Fig. 4-25. Grouting Shear Pockets on Little River WB – Caulk Prevented Flow

Fig. 4-26. Grouting Shear Pockets on Hurricane Creek EB, Full Grout Pocket in Foreground

After the grout was placed, it was left alone for a few minutes, and then a crew member would return to add grout if it had sunk any. Then the grout was smoothed until level with the surface of the precast deck panel. The grout tended to swell as it set, leaving small "humps" on the bridge deck (**Figures 4-27 and 4-28**). The finisher on the construction crew would sometimes let the grout set for a little while longer and would then go back and smooth out the humps. (The final grinding and grooving of the deck would ensure a flat surface for the final traveled roadway.) After initial set, wet burlap was placed over the pocket to facilitate curing (**Figure 4-29**).

A lot of grout was wasted, especially at the beginning of the project when the contractor was getting accustomed to the operations. It took some trial and error and debugging to master the grout proportioning and mixing. By the end of the project, one of the construction workers had become a master of sorts. Inspectors noted that an "expert" is really needed to mix the grout, and the person holding the end of the hose to dispense the grout needs to be knowledgeable on how to prevent voids.

Fig. 4-27. Grout Expands After Setting

Fig. 4-28. Slightly Raised Grout, After Setting

Fig. 4-29. Wet Burlap Placed Over Shear Pockets for Curing

By the end of the project, the grouting operations went relatively smoothly. It took six (6) days to grout the 13 panels on Hurricane Creek WB bridge. For Little River WB bridge's 51 panels, it took 17 days. This included grouting the shear pockets, as well as the leveling bolts and lifting eyes.

4.1.4 Sampling Requirements

The manufacturer's recommendation for the mixed grout temperature was 70 degrees. It was difficult to maintain the 70-degree temperature, though. The temperatures of the grout components were measured before mixing. Below is an example of the measurements that were taken for a particular batch of grout:

- 3 bags grout + 3.3 gallons water
- Temperature of dry grout in bag = 79 degrees
- Temperature of iced water = 43 degrees
- Temperature of air = 73 degrees
- Temperature of mixed grout = 70 degrees
- Efflux = 25 sec

The manufacturer's required efflux was 25 to 30 seconds, at 70 degrees. (SP 934 required 20 to 30 seconds.) **Figure 4-30** is a photo of efflux time being measured. Temperature and efflux measurements were recorded extensively. **Figure 4-31** is a sample of one of the logs.

The contractor requested to use cylinders for sampling requirements, but the breaks were too low and had a lot of variance. The contractor reverted to using 2-in. cubes (**Figure 4-32**), which gave good breaks with little variability.

Fig. 4-30. Measuring Efflux of Grout

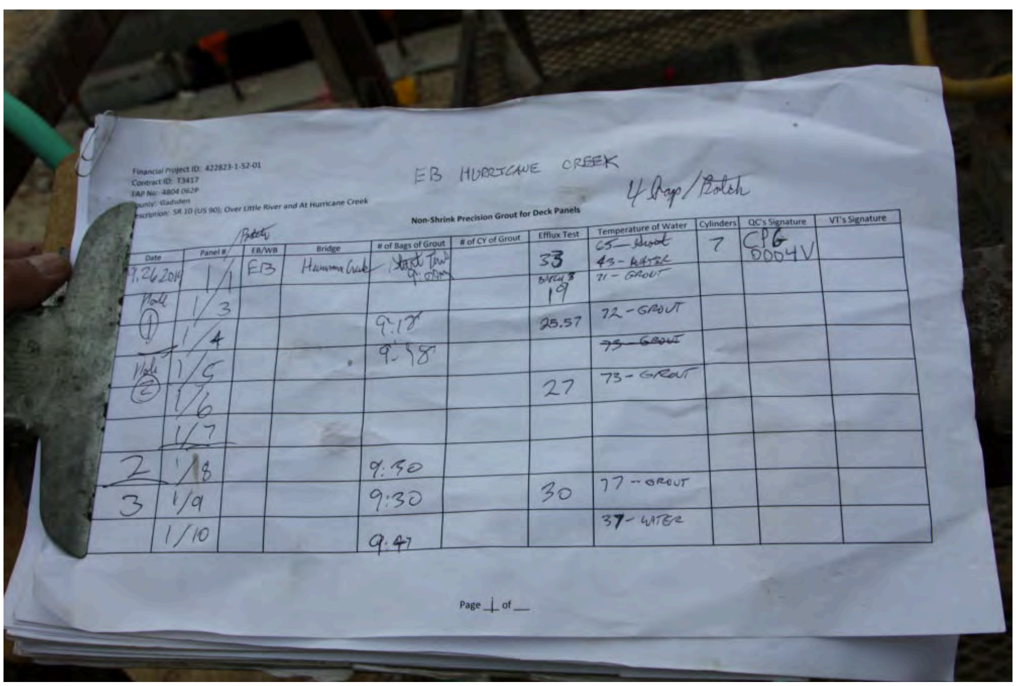


Fig. 4-31. Record Keeping for Grout Measurements


Fig. 4-32. Making Grout Cubes for Testing

The contractor made eight (8) cubes per panel. Inspectors working on behalf of FDOT also made eight (8) cubes per panel. Four (4) of the eight (8) samples were made using plastic molds; these were for the 3-day breaks. The other four (4) samples were made using brass molds; these were for the 28-day breaks. Each party broke three (3), and one (1) was left for resolution. The 3-day breaks were for determining if the leveling bolts could be removed.

4.1.5 Lessons Learned

Several lessons were learned during the grouting process. Inspectors offer the following suggestions to improve the process and quality:

1. Take care to ensure that the hose is completely submerged and inserted correctly in the haunch area to minimize air from being induced. This helps to prevent voids.

2. In hot weather, make sure that the shear pockets are dampened before placing the grout. On Little River EB bridge, 18 shear pockets needed to be repaired because the cured grout had contracted from the panel around the perimeter of the pocket. To repair a pocket, a couple of holes were drilled into the crack, and epoxy was poured in. Contraction did not occur on Little River WB bridge, because the contractor was careful about wetting the pockets, according to inspectors.
3. It is good practice to store the bags of grout in a steel container box. The contractor stored the bags outside at the bridge site, which made the grout hot. They added ice in an attempt to lower the temperature of the mixed grout.

The shear pockets on the WB bridges ended up with more cracks in them. Inspectors opined that not enough attention was paid to the finishing while the grout was wet. This resulted in "humps" once the grout cured. The milling machine had difficulty planing the deck, so the contractor had to chip down the pockets so that the milling machine could go over them. Perhaps the specifications could address this, by specifying an allowable amount of swelling of the finished shear pocket above the precast panel concrete (e.g., any swelling shall be no more than 0.25 in. higher than the surrounding panel concrete.)

Specifications called for a grout sample every 50 cubic yards or one-day's production. The samples are used to measure efflux, temperature, 28-day compressive strength, etc. Inspectors suggest that samples be required for *each element* (e.g., for *each* bent cap and for *each* panel). That way, if the material tests, such as the 28-day strength, did not pass, then only one element would be affected and would need resolution.

4.1.6 Leveling Bolts and Lifting Eyes

After it was demonstrated by testing that the grout had achieved 70% of the design strength (70% of 6750 psi = 4725 psi), the leveling bolts were removed from the panels. The blockouts for the leveling bolts and lifting eyes were then cleaned and grouted, using the same grout as for the shear pockets (**Figure 4-33**).

Fig. 4-33. Grouting Lifting Eyes on Deck Panel

4.2 Closure Joint Casting

After all of the shear pockets on a span were grouted, the closure joints between the precast deck panels were filled with concrete. This effectively connected the panels together and resulted in a continuous slab along the entire span length. Closure joints were as wide as the bridge in the transverse direction and 1 ft in the longitudinal direction. A cross section of the joint is shown in **Figure 4-34**.

Forms for the closure joints were partially put in position as the panels were placed (**Figure 4-35**). **Figure 4-36** is a view from underneath the panel, showing how the forms were left hanging. When it was time to prepare the joints for casting, the forms were tightened into position (**Figures 4-37 and 4-38**). Before the closure joints were cast, scuppers were installed. Then the reinforcing steel was placed in the joints per the design plans. The bars were fed in one at a time. (As shown in **Figure 4-39**, the form on the end of the closure pour was not put on until later, so that the bars could be slid in from the side.) As shown in **Figure 4-40**, the bars were not quite long enough to span the width of the bridge (there would be too much cover), so short bars were added at the ends. For future, inspectors recommended making the splice a little longer than required, as a precaution.

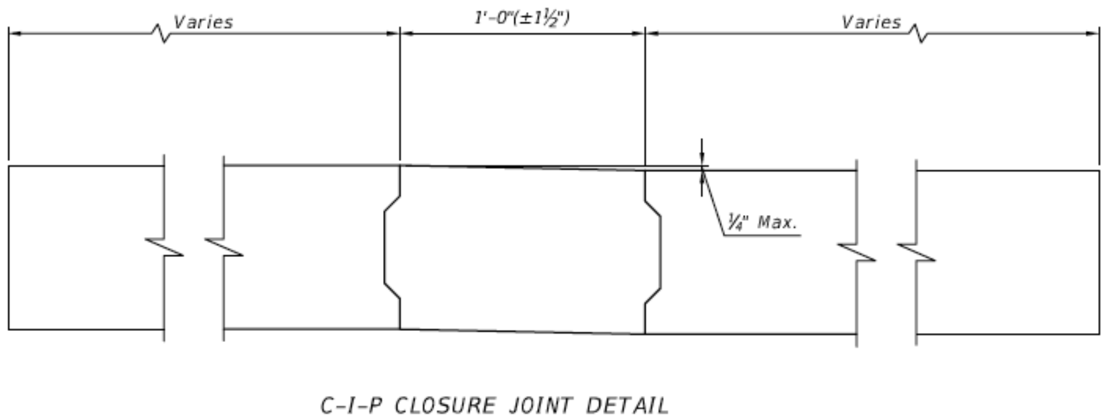


Fig. 4-34. Drawing of Closure Joint Cross Section Showing Shear Keys

Fig. 4-35. Positioning Closure Joint Forms on Hurricane Creek EB as Panels Were Placed

Fig. 4-36. Closure Joint Forms Left Hanging After Panels Were Placed (View from Underneath Panel)

Fig. 4-37. Closure joint Forms, View from Underneath Hurricane Creek EB

Fig. 4-38. Closure Joint Forms Suspended from Wood and Steel Rods, View from Above

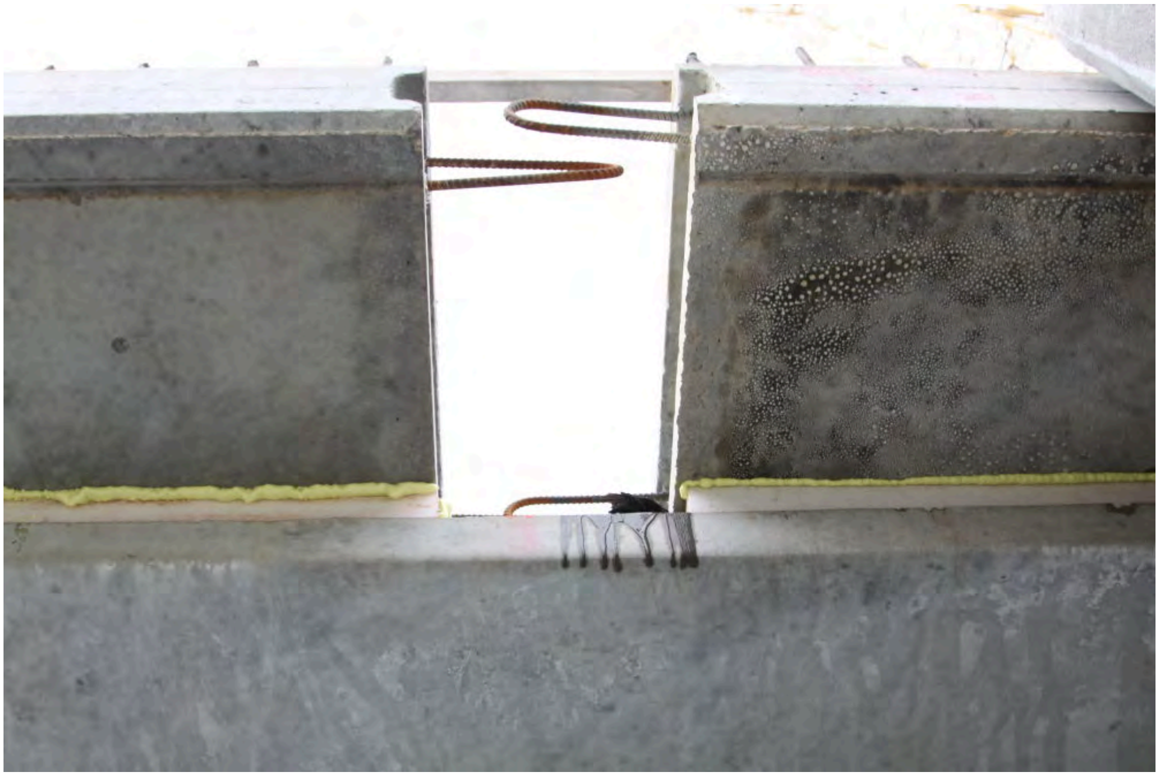


Fig. 4-39. View of Closure Joint from Underneath Bridge, at Panel Edge


Fig. 4-40. Closure Joint Reinforcing Steel (Horizontal Bars in Photo are Protruding from Panel, and Vertical Bars Were Added per the Design Plans)

The hooked bar that protruded from the precast panel, at the end, was set to 2 in. from the bar's center to the panel's edge. This left a cover of only 1.75 in. to the hooked bar – less than the 2 in. that was required.

At the edge of the closure joint, there was a lot of congestion: bars protruding from panels, straight bars in the joint, barrier bars, and scuppers in some cases. For future projects, putting the scuppers in the panels instead of in the joints could be considered.

An epoxy bonding agent was applied to the precast panel shear keys, to reduce development of shrinkage cracks at the interface of the precast and cast-in-place concretes (**Figure 4-41**). A conventional high-performance concrete (4,500-psi strength) with a shrinkage-reducing admixture was specified for the closure joints.

Fig. 4-41. Epoxy Bonding Agent was Sprayed to Panel Keys Before Casting Closure Joints

Casting and finishing closure joints (**Figures 4-42 to 4-44**) was relatively quick. As an example, all 47 closure joints in the four (4) spans were cast in five (5) days on Little River

WB bridge. Curing compound was applied to the joints (**Figure 4-45**). **Figure 4-46** shows the final deck as constructed, before the barriers were placed and before the surface was milled and grooved.

Fig. 4-42. Casting Closure Joints on Hurricane Creek EB

Fig. 4-43. Leveling Out Closure Joint Concrete on Hurricane Creek EB


Fig. 4-44. Screeding Concrete on Closure Joint


Fig. 4-45. Closure Pours and Curing Compound on Hurricane Creek EB


Fig. 4-46. Deck Surface of Hurricane Creek WB Bridge, Before Barriers and Milling

4.3 Finishing

4.3.1 Barriers

For both WB bridges, it took about a week to tie the reinforcing steel (**Figures 4-47 and 4-48**). Barriers were continuously cast using the conventional slip-forming technique (**Figures 4-49 and 4-50**). It took about 2 days to slip form the barriers on both sides of Hurricane Creek and Little River WB bridges. **Figure 4-51** shows a construction crew member finishing a barrier and adding vertical grooves on the face. A completed barrier is shown on the left (EB) bridge in **Figure 4-52**.

Fig. 4-47. Barrier Reinforcing Steel for Hurricane Creek WB

Fig. 4-48. Barrier Reinforcing Steel and Embedded Pipes for Hurricane Creek WB

Fig. 4-49. Slip Forming Barriers on Little River WB – Slip Former and Concrete Truck

Fig. 4-50. Slip Forming Barriers on Little River WB – Slip Former

Fig. 4-51. Finishing Barriers on Little River WB

Fig. 4-52. Completed Barrier on Hurricane Creek EB (on Left) While Load Test is Being Performed on Hurricane Creek WB (on Right)

4.3.2 Milling & Grooving

The deck was milled and grooved, as shown in **Figure 4-53**, using conventional methods. The milling equipment can mill the deck surface within about 12 to 18 inches from the barrier face. Because this region is typically within the shoulders, this does not affect the riding surface of the lanes. However, because precast deck panels were used, the shear pockets over the exterior girders did not get completely milled because the equipment could not reach them. This is mostly an issue of appearance. (With conventional cast-in-place deck construction, the unmilled portion just appears as relatively smooth concrete.) If this is a concern for future projects, then consider either: 1) moving the girder so that it is greater than 2 ft from the barrier or 2) offsetting the shear pocket from the centerline of girder.

Fig. 4-53. Deck Surface, Milled and Grooved

4.3.3 Expansion Joints

Deck expansion joints at the approach slabs and intermediate bents were filled with a poured compression seal (**Figures 4-54 to 4-56**).

Fig. 4-54. Preparing to Pour Expansion Joint Seal

Fig. 4-55. Silicone Sealant for Expansion Joints

Fig. 4-56. Filling Expansion Joint with Sealant

CHAPTER 5
GROUTING DEMONSTRATION TESTS

5.1 Introduction

Grouting operations were required for the drilled shaft/column-to-precast bent cap connections, as well as for the precast deck panel shear pockets and haunches between the panels and FIBs. The design plans required the contractor to submit a Precast Placement Plan and to perform a Grouting Demonstration Test (mock-up) in advance of grouting the members at the bridge site. The cost of the Grouting Demonstration Test was included in the pay items for the precast members.

The Precast Placement Plan included information on the grout material, equipment, and methods that the contractor planned to use, as well as mock-up details. The purpose of the mock-up was as follows: to demonstrate the grout properties and adequacy of equipment; to test the forming materials; and to familiarize the contractor's crew with the proposed grouting procedures. It was also to demonstrate that the procedures would result in minimal voids – less than the maximum void areas allowed in the contract plans. The grout material used for both the bent cap connections and precast deck panels was MasterFlow® 928, a high-precision mineral-aggregate grout with extended working time, made by Master Builders.

5.2 Precast Bent Cap Grout Mock-Up

Details and notes from the design plans are provided in **Figures 5-1 to 5-3**. The setup for the precast bent cap grout mock-up is shown in **Figures 5-4 and 5-5**. Grout was dispensed into the middle hole at the top of the forms (Figure 5-4). The grout flowed into the steel casing (representing the top of a drilled shaft/column) underneath wooden forms that mimicked a bent cap (Figure 5-5). After the "shaft" filled with grout, the grout flowed up the six (6) ducts that contained dowel bars.

Photos of the grouting equipment and mixing operations are shown in **Figures 5-6 to 5-8**. **Figure 5-9** shows the materials testing area (for measuring efflux time, measuring temperature, and making compression-test specimens). The next day, the wooden "bent cap" forms were removed, and the top of the "shaft" was inspected (**Figure 5-10**). The grout surface was very smooth, and there were no voids.

GROUTING DEMONSTRATION TEST:

1. *After approval of all Submittals required by Specification Section 404, and at least two weeks prior to Precast Bent Cap erection, prepare a full scale mock-up of the connection and place a trial batch of grout to demonstrate grout properties, adequacy of equipment, and to familiarize job site personnel with grouting procedures in the Contractor's Precast Placement Plan.*
2. *The full scale mockup shall consist of a representative drilled shaft with all corrugated steel ducts, dowels and bedding layer reinforcement, set in plywood or similar forms matching the Precast Bent Cap cross section.*
3. *After completion of the grouting operation and grout setup, the forms shall be carefully removed and the consolidation of the grout in the connection inspected by the Engineer.*
4. *The presence of significant voids in the grout shall require a resubmittal of the Precast Placement Plan and retesting by another full scale mock-up. Significant voids are defined as individual voids greater than two cubic inches in volume, or where the total surface area of voids in the bedding layer exceeds 1% of the plan view surface area for voids greater than ¼" width or length. In addition, the top surface of the bedding layer shall be sounded with a hammer, and as directed by the Engineer, portions of the bedding layer removed to verify grout consolidation.*
5. *Include the cost of the Grout Demostration Test in Pay Item 404-1 (Precast Bent Caps).*

Fig. 5-1. Grout Mock-up for Precast Bent Cap, Notes (from Design Plans)

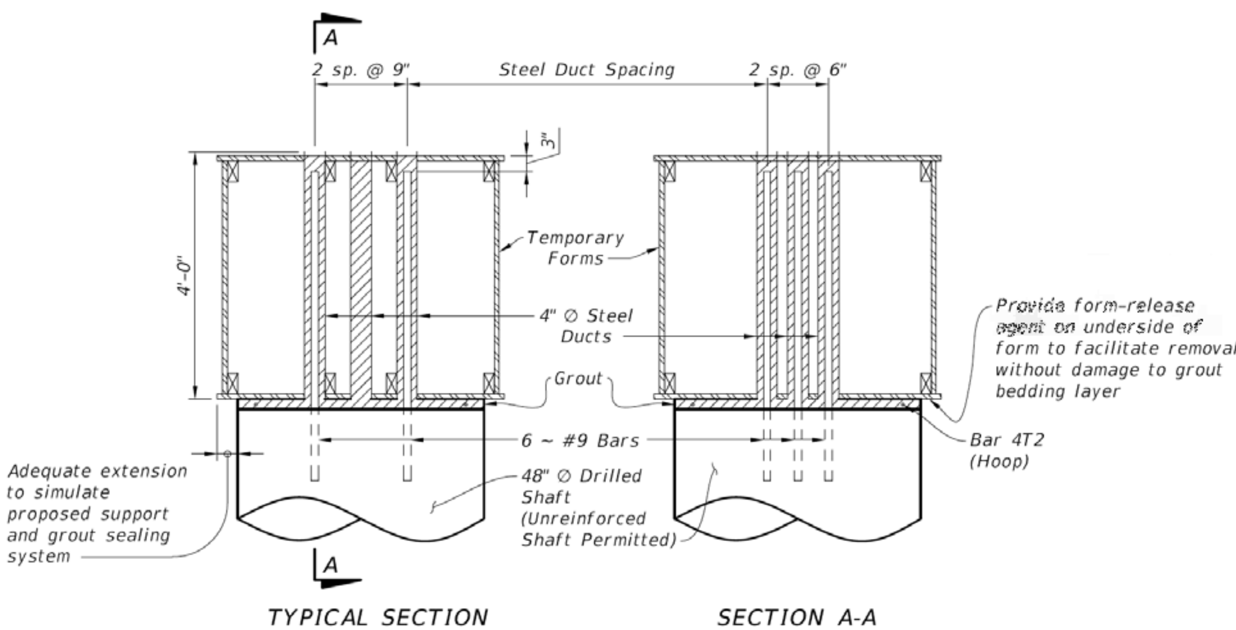


Fig. 5-2. Grout Mock-up for Precast Bent Cap, Elevation and Section Views (from Design Plans)

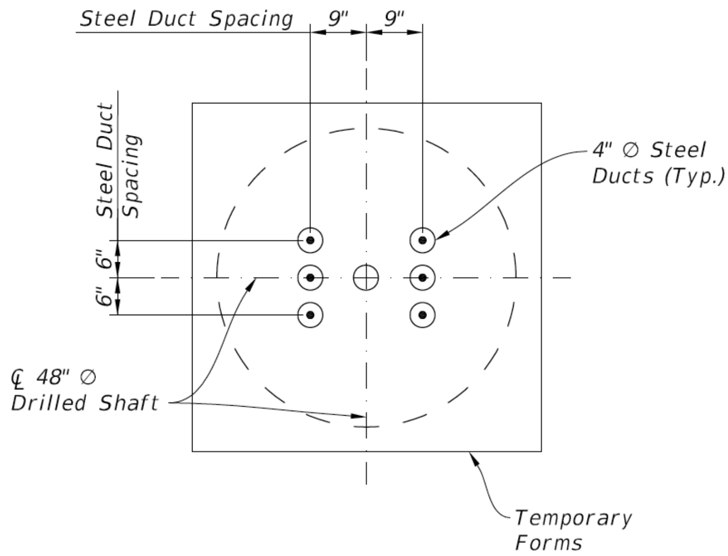


Fig. 5-3. Grout Mock-up for Precast Bent Cap, Plan View (from Design Plans)

Fig. 5-4. Grout Mock-up for Precast Bent Cap, Dispensing Grout, Top View

Fig. 5-5. Grout Mock-up for Precast Bent Cap, Side View, Wooden "Bent Cap" Forms on Top of Steel "Drilled Shaft" Casing

Fig. 5-6. Grouting Equipment

Fig. 5-7. Mixing Grout

Fig. 5-8. Grout Flowing in Pump

Fig. 5-9. Grout Testing Setup

Fig. 5-10. Grout Mock-up for Precast Bent Cap, after Casting and Removal of Forms

5.3 Precast Deck Panel Grout Mock-Up

For the precast panel demonstration, details and notes from the design plans are provided in **Figures 5-11 and 5-12**. The setup for the precast deck panel grout mock-up is shown in **Figures 5-13 and 5-14**. The bottom slab of concrete represented the top flange of a Florida-I Beam, and the top slab "panel" mimicked a portion of a precast deck panel with two (2) shear pockets/voids (**Figures 5-15 and 5-16**). Polyethylene forms were glued around the perimeter of the "FIB", to imitate forming for the haunches. The polyethylene was beveled differently along each edge (shown in Figures 5-13 and 5-14), to simulate the transverse and longitudinal slopes that would be encountered on the bridge. Grout was dispensed into the shear pockets at the top of the forms (Figure 5-16), and tubes pre-placed in the corners of the polyethylene/haunch layer (shown in Figure 5-14) were plugged after the grout flowed from them (**Figure 5-17**). See **Figure 5-18** for a photo of the panel after grouting was complete.

Test Setup

1. *After approval of all Submittals required by Specification Section 404, and at least two weeks prior to panel erection, a Grouting Demonstration Test using a full scale mock-up of the haunches shall be prepared and placed to demonstrate grout properties, adequacy of equipment, forming material, and to familiarize job site personnel with grouting procedures.*
2. *Reinforcing bars, lifting devices, leveling bolts, etc... shall be detailed, furnished and installed by the Contractor. Payment shall be incidental to the cost on the Grouting Demonstration Test. Concrete shall be Class II (Bridge Deck) for the top panel & bottom slab.*
3. *The full scale mock-up shall consist of a top panel and a bottom slab that represents a beam and panel as shown in the details on this sheet. The finish of the contact surface of the bottom of the top panel and the top of the bottom slab shall be in accordance with Section 404 and Sheet B1-46, respectively.*
4. *After completion of the grouting operation and setup of the grout, the top panel and forming material shall be carefully removed and the consolidation of the grout in the haunches inspected by the Engineer.*
5. *The presence of significant voids in the grout shall require a resubmittal of the Precast Placement Plan and retesting by using a full scale mock-up. Significant voids are defined as individual voids greater than two cubic inches in volume, or where the total surface area of voids greater than ¼" in width, length or diameter exceeds 1% of the plan area of the haunch. In addition, the top surface of the haunch shall be sounded with a hammer, and as directed by the Engineer, portions of the haunch removed to verify grout consolidation.*
6. *Include cost of Grout Demonstration Test in Pay Item 0404-6.*

Fig. 5-11. Grout Mock-up for Precast Deck Panel, Notes (from Design Plans)

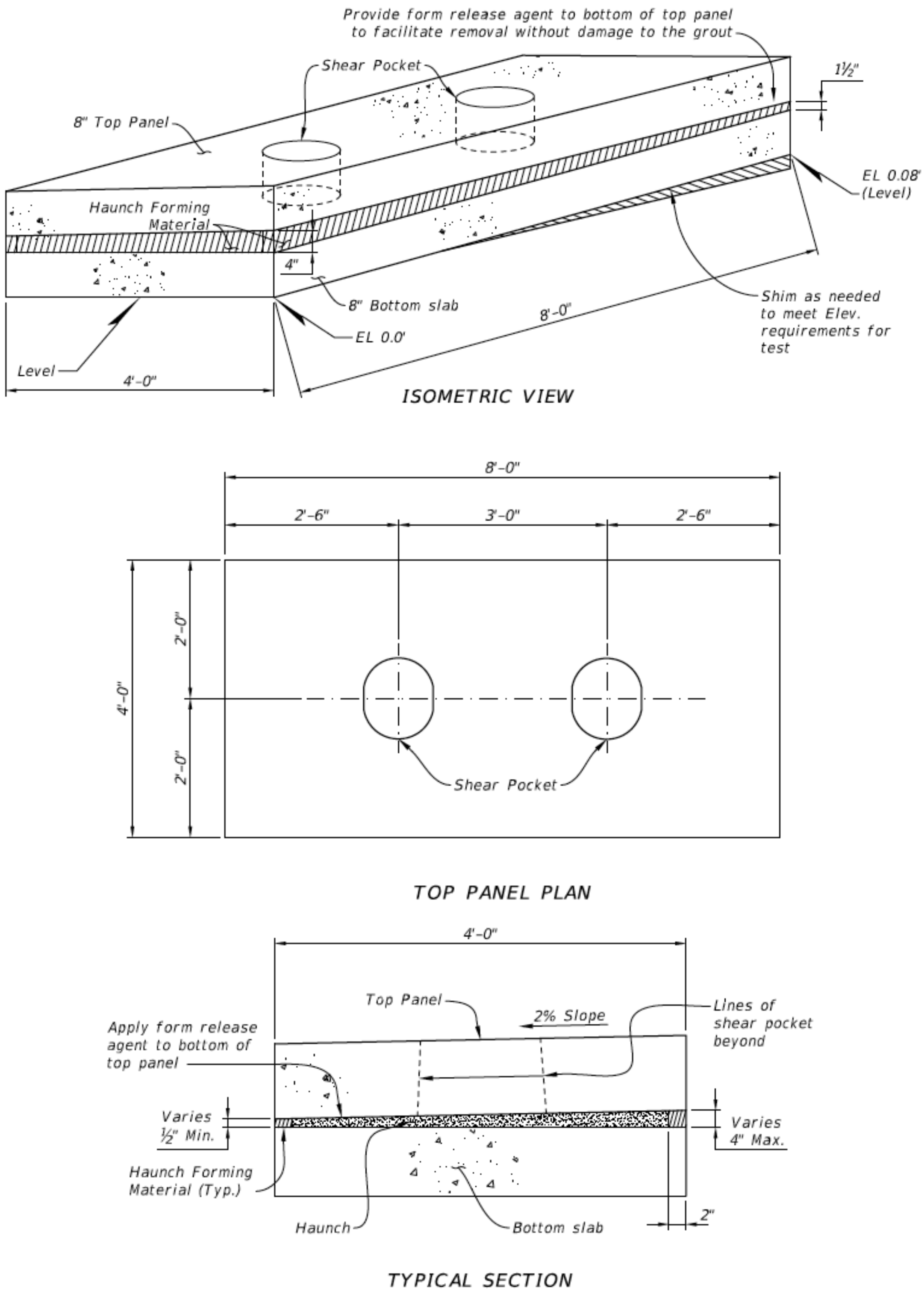


Fig. 5-12. Grout Mock-up for Precast Deck Panel, Isometric, Plan, and Section Views (from Design Plans)

Fig. 5-13. Grout Mock-up for Precast Deck Panel, “Panel” Being Lowered onto “FIB”

Fig. 5-14. Grout Mock-up for Precast Deck Panel, “Panel” and “FIB” in Place, with Polyethylene Forming the “Haunch”


Fig. 5-15. Grout Mock-up for Precast Deck Panel, Shear Pocket in “Panel”


Fig. 5-16. Grout Mock-up for Precast Deck Panel, Dispensing Grout in “Panel” Shear Pocket

Fig. 5-17. Grout Mock-up for Precast Deck Panel, Plugging Corner to Stop Grout Flow

Fig. 5-18. Grout Mock-up for Precast Deck Panel, Grouting Completed, Top View of Panel

The next day, the "panel" was removed (**Figure 5-19**), and the grouted surface was inspected (**Figure 5-20**). **Figures 5-21 to 5-28** are close-up photos of the voids, which were measured and recorded (see **Table 5-1**). The total area of voids was about 6.1% of the total grouted area (calculated in **Table 5-2**).

Fig. 5-19. Grout Mock-up for Precast Deck Panel, "Panel" Removed for Grout Inspection ("Panel" Is Flipped Over in Foreground; "FIB" and Grouted "Haunch" Are in Background)

Fig. 5-20. Grout Mock-up for Precast Deck Panel, Grout Surface


Fig. 5-21. Grout Mock-up for Precast Deck Panel, End 1


Fig. 5-22. Grout Mock-up for Precast Deck Panel, End 2

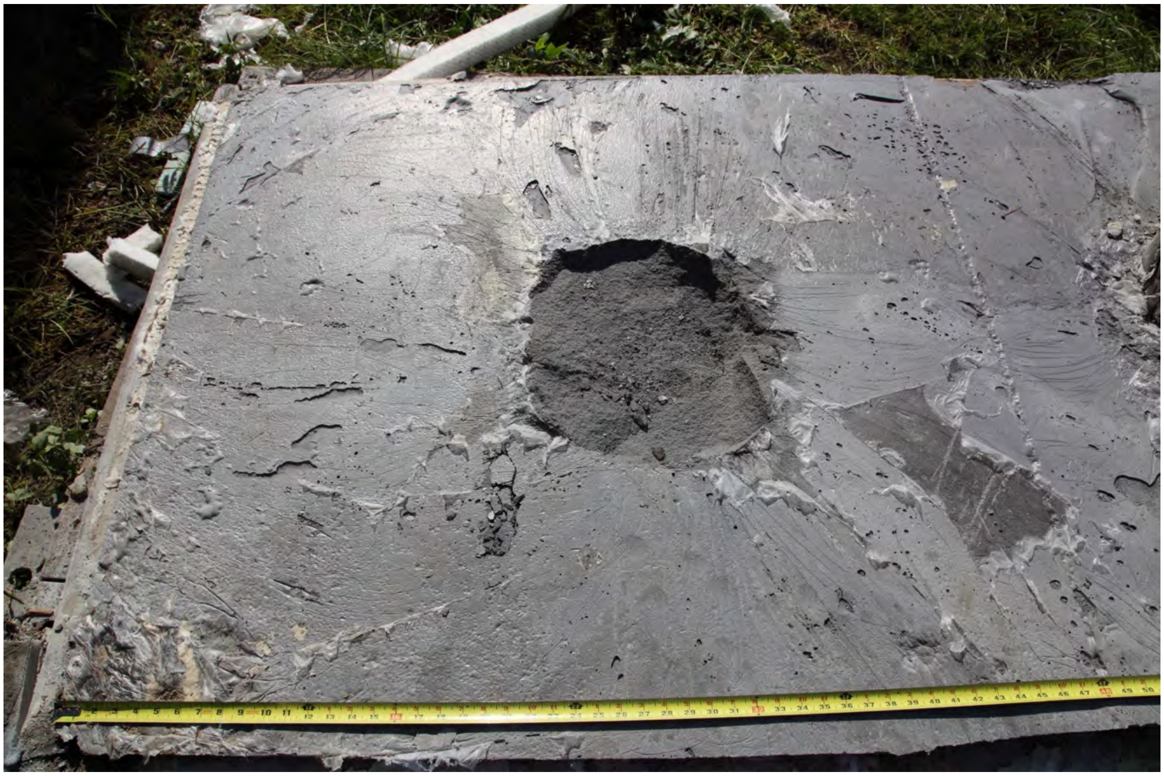


Fig. 5-23. Grout Mock-up for Precast Deck Panel, End 1 Close-up

Fig. 5-24. Grout Mock-up for Precast Deck Panel, Area between Shear Pockets

Fig. 5-25. Grout Mock-up for Precast Deck Panel, End 2 Close-Up

Fig. 5-26. Grout Mock-up for Precast Deck Panel, End 1

Fig. 5-27. Grout Mock-up for Precast Deck Panel, Middle Region

Fig. 5-28. Grout Mock-up for Precast Deck Panel, End 2

Table 5-1. Grouting Demonstration Test for Precast Deck Panel, Void Dimensions & Areas

VOID NUMBER	WIDTH (in.)	LENGTH (in.)	AREA OF VOID (in^2)	PERCENTAGE OF TOTAL GROUTED AREA
1	1.25	0.75	0.938	0.02%
2	3	0.75	2.250	0.06%
3	2.75	1.75	4.813	0.13%
4	1.75	0.6	1.050	0.03%
5	1.5	1.75	2.625	0.07%
6	0.85	1	0.850	0.02%
7	2	1.5	3.000	0.08%
8	1.8	0.75	1.350	0.04%
9	2	1	2.000	0.05%
10	1.5	0.75	1.125	0.03%
11	6.5	14	91.000	2.41%
12	3.5	0.5	1.750	0.05%
13	2.75	1.75	4.813	0.13%
14	10	3.25	32.500	0.86%
15	3.5	1	3.500	0.09%
16	3.25	1.25	4.063	0.11%
17	1.5	1	1.500	0.04%
18	3.25	1	3.250	0.09%
19	1.2	0.5	0.600	0.02%
20	1	0.75	0.750	0.02%
21	1.5	0.5	0.750	0.02%
22	2.5	0.75	1.875	0.05%
23	4	0.75	3.000	0.08%
24	3	0.8	2.400	0.06%
25	4.5	0.75	3.375	0.09%
26	2.75	0.5	1.375	0.04%
27	0.75	1	0.750	0.02%
28	2	0.75	1.500	0.04%
29	1	0.75	0.750	0.02%
30	4.5	2.75	12.375	0.33%
31	1.25	0.5	0.625	0.02%
32	2	0.5	1.000	0.03%
33	1.75	1	1.750	0.05%
34	1	0.5	0.500	0.01%
35	2	1	2.000	0.05%
36	1.5	1	1.500	0.04%
37	1.75	0.5	0.875	0.02%
38	1	0.6	0.600	0.02%
39	1.5	0.75	1.125	0.03%
40	7	4	28.000	0.74%
		TOTAL VOIDS:	**229.85 in^2**	**6.09% of Total Grouted Area**

Table 5-2. Grouting Demonstration Test for Precast Deck Panel, Total Grouted Area

Area of 8' x 4' Panel (1)	Area of Shear Pockets (2)	Approx. Area of Forming Material (3)	Total Grouted Area = (1) - (2) - (3)
4608 in^2	274 in^2	560 in^2	3774 in^2

5.4 Conclusions of Grouting Demonstration Tests

The mock-up was a good opportunity for the contractor to learn how to mix and place the grout. The tests were done on a very hot summer day. The contractor had to ensure that the grout temperature was about 70 degrees F to meet the efflux requirements and for the grout mix to be flowable. It was decided to use four (4) grout tubes, one (1) at each corner of the haunch area, rather than only two (2) at the high points as originally planned.

The Engineer of Record (EOR) recommends that, on future projects, the mockup schematic clarify that the bedding layer sealing system be formed similar to that proposed in the contractor's Grouting Plan, to verify the efficacy and facilitate better evaluation of the mockup sample to reveal any voids on the column face after form removal.

5.5 Related RFIs

RFI #17, dated 5/2/2014:

Was the polyethylene foam open or closed cell that was used on the FDOT mockup?

Response: It was a closed cell material.

RFI #18, dated 5/2/2014:

Anderson Columbia has requested the option to use polystyrene ASTM C 578 Type IV or higher for the haunch material. Whatever material is used for the mockup will be used in the field. Will this be feasible for the project?

Outcome: Anderson Columbia has decided they are going to use the closed cell polyethylene. Please disregard the request to use polystyrene for the project.

RFI #19, dated 5/20/2014:

At this time on the project ACCI has come to the point where we are ready to perform the "Grout Mock Up" presentation so to proceed with the project. As of now the project cannot move forward due to the grout not meeting all the specified test requirements. The specifications require the testing requirements listed below. In an effort to provide the Department with what is required, ACCI was unsuccessfully been able to find a grout product that meets all of the testing requirements on or off the QPL list. In an effort to expedite the progress of the project ACCI asks the following questions based on the job being developmental.

1) Do the results of these tests have any bearing on whether or not the product can serve its intended purpose?
Response: Yes.

2) Since this product is developmental, is the additional tests based upon developmental and experimental questioning? If so, can we proceed with the grout mock up and await the test results?
Response: We evaluated several research reports to determine which type of material would provide a quality product and the associated test for that product. We do not object to the Contractor proceeding at his own risk in performing the grout mock-up (i.e. Grouting Demonstration Test) as they await the test results. If the grout tests do not pass, then another GDT will be required using another selected material.

3) What grout product did the FDOT use for their mock up of the precast deck slabs? Knowing this product would assist the project to identify and obtain a grout product that meets the desired testing requirements.
Response: MasterFlow® 928 (Precision) was used at the FDOT mock up.

4) How and why did the FDOT place the additional performance requirements listed below?

Response: A modified special provision, MSP 934, was developed to provide the requirements for a "precision" grout. The MSP 934 was included in the Specification package for FPM 422823 Contract T3417. The MSP 934 was discussed at the pre-bid meeting that required mandatory attendance for all proposers.

5) Can the Bleeding, Final, Efflux Time, Freeze Thaw, and the Coefficient of Thermal Expansion testing be eliminated?

Response: No, precision grout testing is per MSP 934.

CHAPTER 6
CORES

One of the research project tasks was to drill cores in the deck panels after the shear pockets were grouted and the closure joints were cast. FDOT personnel drilled the cores using their equipment. Cores were taken from Little River EB and Hurricane Creek EB bridges. They were taken from the panels, at the shear pocket-to-panel interface, and at the closure joint-to-panel interface. **Figures 6-1 and 6-2** are of the coring machine and of a core being pulled from the panel. After being removed, the cores were visually inspected. The holes in the panels were filled with grout by FDOT personnel.

Fig. 6-1. Coring Machine for Extracting Concrete Samples

Fig. 6-2. Core Being Pulled from Deck Panel

Photos of the cores are provided in **Figures 6-3 to 6-27**. In general, it appeared that the grout filled the shear pockets and haunches well. The cast-in-place concrete filled the closure joints well, also. A few specific comments are as follows:

- Core 4 (Figures 6-12 to 6-14) was cut through an area where the grout in the haunch had been repaired, and good consistency was apparent.
- The panel-to-closure pour surfaces were tightly bonded on almost all cores. Some surfaces had minor gaps, probably from air getting trapped on the underside of the panel joint (see **Figure 6-28**).
- The surfaces between the girder and closure pour were rough, but they fit together well and seemed to provide good interlock and friction bond.
- **Figure 6-29** is of a core through the panel and shear pocket, extending into the girder top flange; it shows good bond between the three elements.
- Cores that were made through areas with grout repairs showed that the repairs had good fit with the concrete, meaning that the grout flowed well into the repaired areas.

Fig. 6-3. Core 1, Little River EB, Span 1, Panel 3 (in Panel) (1)

Fig. 6-4. Core 1, Little River EB, Span 1, Panel 3 (in Panel) (2)

Fig. 6-5. Core 1, Little River EB, Span 1, Panel 3 (in Panel) (3)

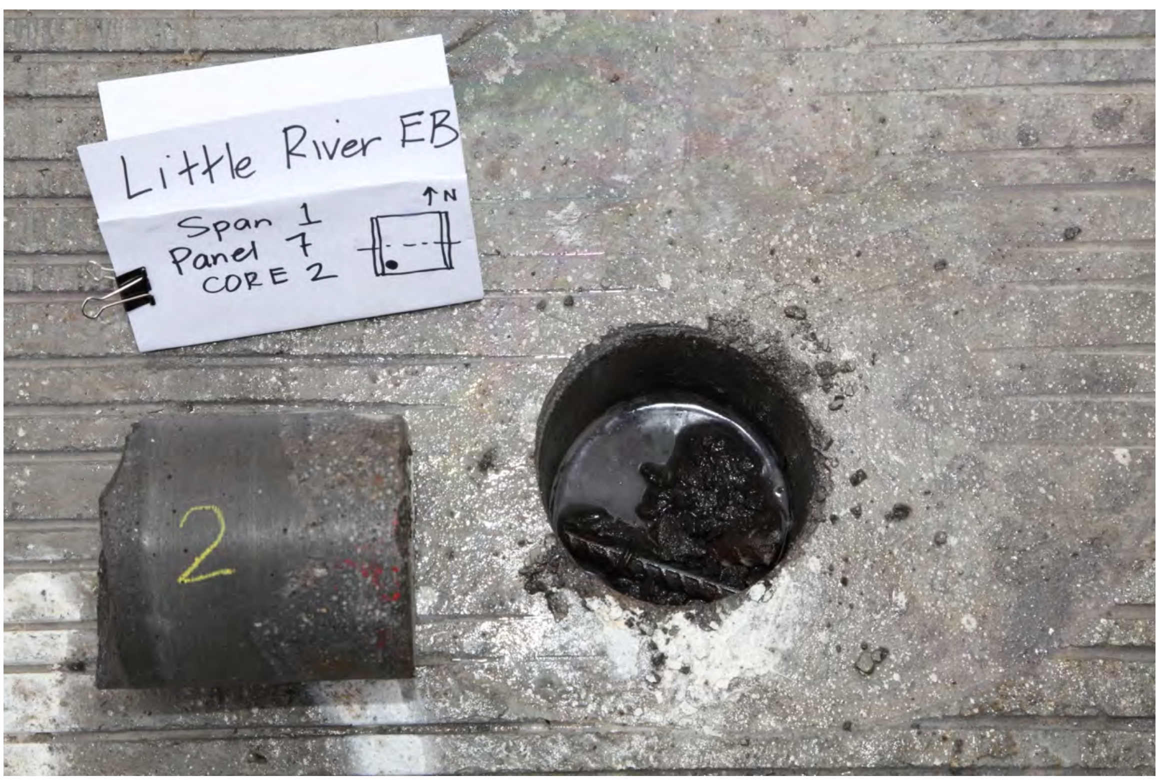


Fig. 6-6. Core 2, Little River EB, Span 1, Panel 7 (Shear Pocket-to-Panel Interface) (1)

Fig. 6-7. Core 2, Little River EB, Span 1, Panel 7 (Shear Pocket-to-Panel Interface) (2)

Fig. 6-8. Core 2, Little River EB, Span 1, Panel 7 (Shear Pocket-to-Panel Interface) (3)

Fig. 6-9. Core 3, Little River EB, Span 1, Panel 10 (Closure Joint-to-Panel Interface) (1)

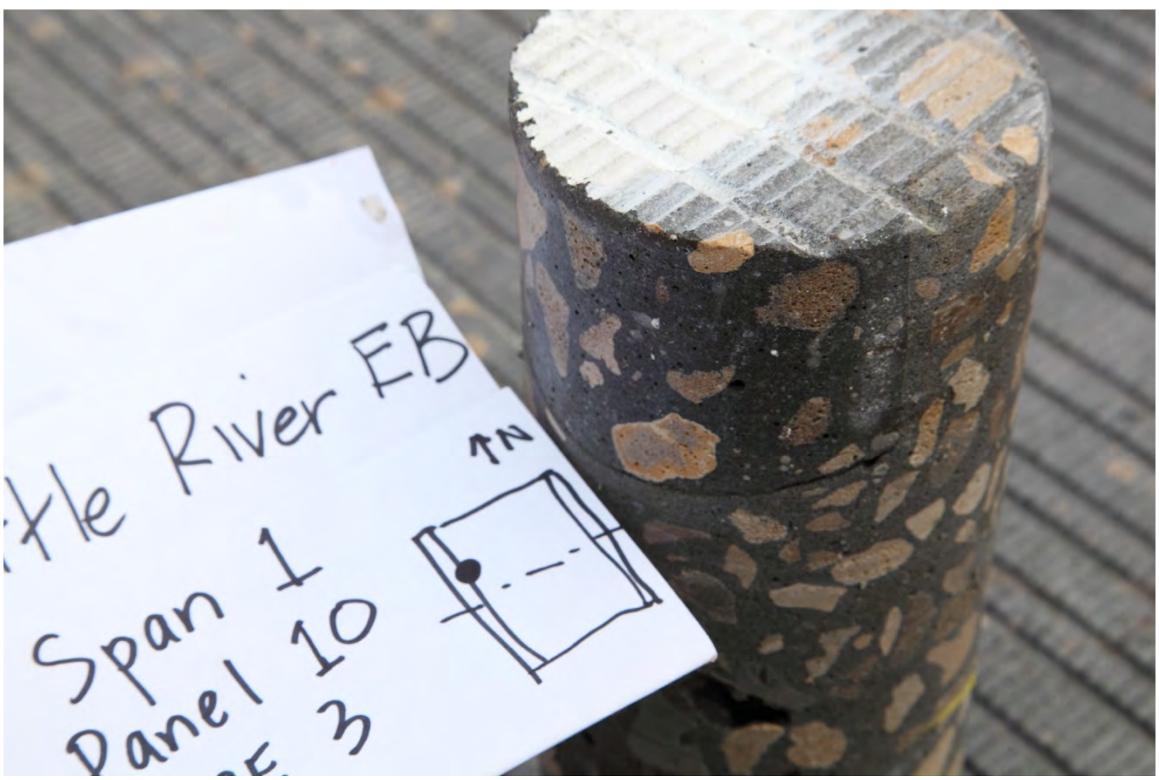


Fig. 6-10. Core 3, Little River EB, Span 1, Panel 10 (Closure Joint-to-Panel Interface) (2)

Fig. 6-11. Core 3, Little River EB, Span 1, Panel 10 (Closure Joint-to-Panel Interface) (3)

Fig. 6-12. Core 4, Little River EB, Span 2, Panel 6 (Closure Joint-to-Panel Interface) (1)

Fig. 6-13. Core 4, Little River EB, Span 2, Panel 6 (Closure Joint-to-Panel Interface) (2)

Fig. 6-14. Core 4, Little River EB, Span 2, Panel 6 (Closure Joint-to-Panel Interface) (3)

Fig. 6-15. Core 7, Little River EB, Span 3, Panel 2 (in Panel, at Edge of the Shear Pocket) (1)

Fig. 6-16. Core 7, Little River EB, Span 3, Panel 2 (in Panel, at Edge of the Shear Pocket) (2)

Fig. 6-17. Core 7, Little River EB, Span 3, Panel 2 (in Panel, at Edge of the Shear Pocket) (3)

Fig. 6-18. Core 9, Little River EB, Span 3, Panel 11 (Closure Joint-to-Panel Interface) (1)

Fig. 6-19. Core 9, Little River EB, Span 3, Panel 11 (Closure Joint-to-Panel Interface) (2)


Fig. 6-20. Core 11, Little River EB, Span 4, Panel 4 (Closure Joint-to-Panel Interface) (1)

Fig. 6-21. Core 11, Little River EB, Span 4, Panel 4 (Closure Joint-to-Panel Interface) (2)

Fig. 6-22. Core 12, Little River EB, Span 4, Panel 7 (Closure Joint-to-Panel Interface) (1)

Fig. 6-23. Core 12, Little River EB, Span 4, Panel 7 (Closure Joint-to-Panel Interface) (2)

Fig. 6-24. Core 13, Hurricane Creek EB, Panel 9 (Closure Joint-to-Panel Interface) (1)

Fig. 6-25. Core 13, Hurricane Creek EB, Panel 9 (Closure Joint-to-Panel Interface) (2)

Fig. 6-26. Core 14, Hurricane Creek EB, Panel 10 (in Panel) (1)

Fig. 6-27. Core 14, Hurricane Creek EB, Panel 10 (in Panel) (2)

Fig. 6-28. Core 9 (left) and 11 (right), Showing Minor Gaps Between Panel and Closure Joint

Fig. 6-29. Core 7, Showing Core through Panel and Shear Pocket and into Girder Top Flange

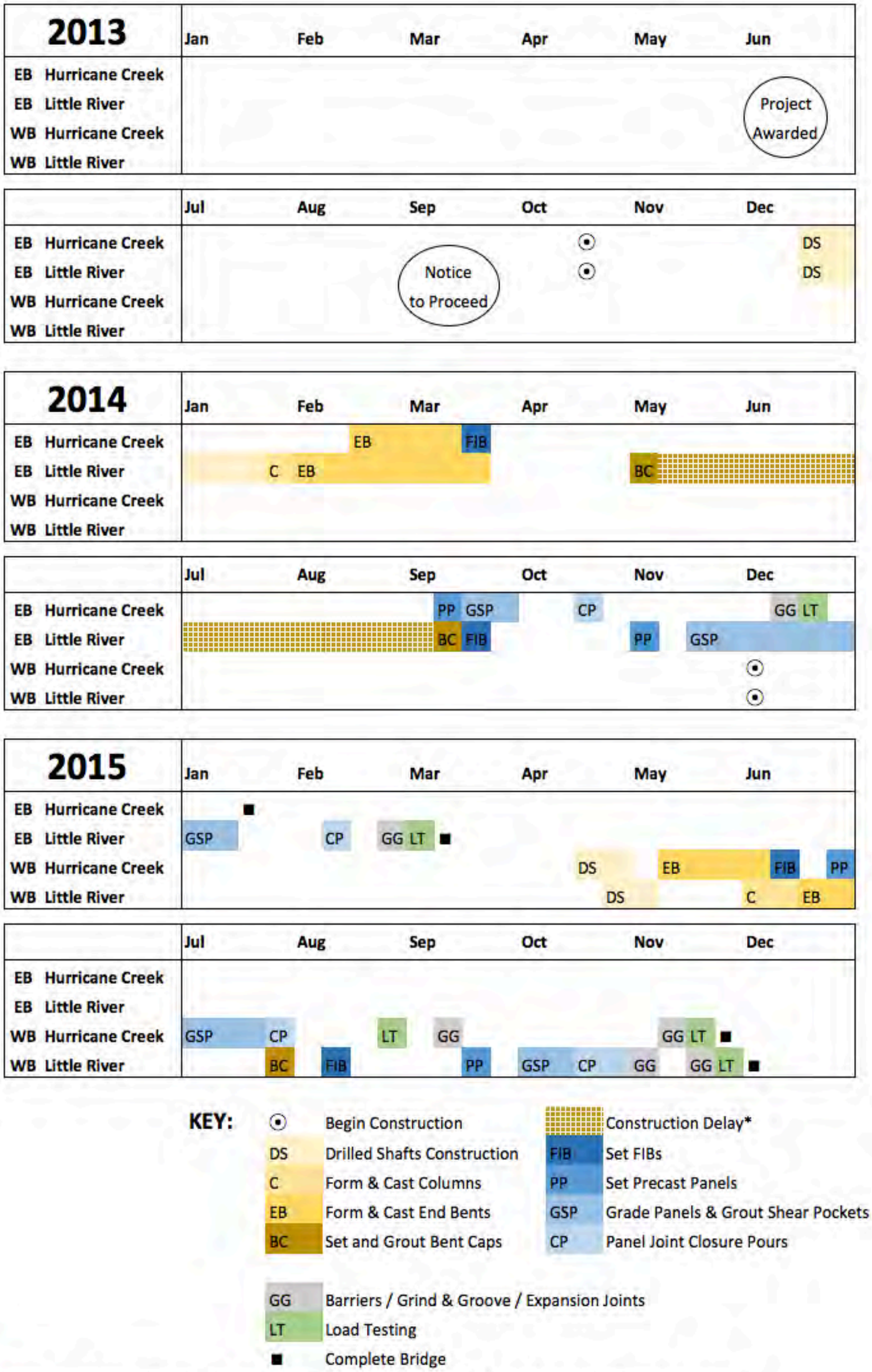
CHAPTER 7
CONSTRUCTION SCHEDULE

Construction of the two (2) EB bridges took about 12 months. The two (2) WB bridges were constructed more quickly – in about 7 months. The timeline for construction of the four (4) bridges is provided in **Figure 7-1**. There was about a 6-month delay at the beginning of the project, while the contractor tried to determine which grout to use and while the testing requirements per SP 934 were met.

The contractor's Precast Placement Plan was revised and resubmitted several times before it was approved. Inspectors recommend splitting the various aspects of the Plan into several different submittals. This way, if a part of the plan were rejected, then all activities would not be delayed.

Construction Engineering and Inspection (CEI) personnel noted that the inspection process was much more intense than for a typical bridge construction project. A lot more bookkeeping, forms, records, and surveys were required.

For future projects that involve precast deck panels and special grouting techniques, inspectors recommend that the bridge be classified as "complex". This is so that more experienced contractors would bid on the project. Inspectors also suggest giving incentives for on-time or early completion.



Note: Hurricane Creek Bridges have single spans and, therefore, do not have columns or bent caps.

*Much of the construction delay was due to the contractor not having completed the Grouting Demonstration Tests, which were required before grouting the bent caps.

Fig. 7-1. Timeline for Construction of the Four (4) US 90 Bridges

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Precast Element Evaluation for the US 90 Bridges over Little River and Hurricane Creek

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REPORT 4: MONITORING OF CRACKS IN PRECAST PANELS

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REPORT 4: MONITORING OF CRACKS IN PRECAST PANELS

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CHAPTER 1
INTRODUCTION

1.1 Background

As explained in Report 2, Fabrication of Precast Elements, and Report 3, Construction of Precast Elements, non-prestressed panels were used on the eastbound bridges (Hurricane Creek EB and Little River EB), and prestressed panels were used on the westbound bridges (Hurricane Creek WB and Little River WB). Report 2 discusses the panel performance, including the subject of pre-service cracking. To recapitulate, the prestressed panels did not experience any significant cracking while at the precast yard or during transportation to the bridge site. However, several of the non-prestressed panels experienced cracking. This required an Engineering Analysis and Report (EAR) to determine the extent of cracking and whether or not the panels were acceptable and/or needed to be repaired. The cracks were evaluated by the contractor's engineer according to the method outlined in FDOT Specification 400-21, Disposition of Cracked Concrete. More details about the findings and the Engineer's Disposition of Cracks are included in Report 2.

The research scope for the US 90 bridge construction project included the task of mapping cracks after the bridges were placed in service for a period of two (2) years. This information can be used by FDOT to evaluate the difference between the performance of non-prestressed precast deck panels and prestressed precast deck panels. Prestressed panels cost more than non-prestressed panels. (Bid price data is provided in Report 1, Introduction to Construction Project. For example, the winning bid was $231.42/sq. yd. for non-prestressed panels and $271.35/sq. yd. for prestressed panels.) However, prestressed members are designed so that the concrete stresses do not exceed allowable stresses (which are less than the cracking stress) for the service condition. This means that prestressed members might perform better in the long run and result in a lower life cycle cost for the bridge.

1.2 Overview

This report provides the results of post-service crack mapping for the first two (2) years of service. Several students and the principal investigator mapped the bridges approximately every three (3) months. This resulted in eight (8) mappings, which hereafter will be referred to as mappings #1 – #8. Typically, all four (4) bridges – ten (10) spans – were mapped. However, for mappings #5 and #7, only the two (2) eastbound bridges – five (5) spans – were mapped. The research project manager from FDOT and the co-principal investigator oftentimes accompanied them and assisted.

Table 1-1 provides the crack mapping dates, along with footnotes about when the bridges were placed in service. The average temperature and weather conditions on each day of mapping are provided in **Table 1-2**. When both eastbound and westbound bridges were mapped, it took two (2) days to complete: the first day was spent on the eastbound bridges, and the second day was spent on the westbound bridges. The two days were consecutive for mappings #1, #4, and #6; there was a one-day gap in between for mappings #2, #3, and #8; and mappings #5 and #7 required one day to complete because only the eastbound bridges were mapped. Mapping #6 required a third day because work had to be halted on the second day due to issues with maintenance of traffic that was being handled by a contractor.

For almost all mappings, FDOT provided maintenance of traffic services while the crack mapping was being performed. One span at a time was mapped, and when it was completed, everyone moved to the next span. The order in which the spans were mapped on a given day (for a given direction – eastbound or westbound) was as follows:

South lane and shoulder of

1. Hurricane Creek
2. Little River Span 4
3. Little River Span 3
4. Little River Span 2
5. Little River Span 1

North lane and shoulder of

6. Hurricane Creek
7. Little River Span 4
8. Little River Span 3
9. Little River Span 2
10. Little River Span 1

Table 1-1. Dates that Crack Mapping Was Performed on US 90 Bridges

		EASTBOUND BRIDGES*		WESTBOUND BRIDGES**	
Map #	Date Performed	Hurricane Creek	Little River	Hurricane Creek	Little River
1	June 21, 2016	✔	✔		
	June 22, 2016			✔	✔
2	September 20, 2016	✔	✔		
	September 22, 2016			✔	✔
3	January 17, 2017	✔	✔		
	January 19, 2017			✔	✔
4	April 24, 2017	✔	✔		
	April 25, 2017			✔	✔
5	July 5, 2017	✔	✔		
6	October 2, 2017	✔	✔		
	October 3, 2017			✔	✔ (Span 1)
	October 17, 2017				✔ (Spans 2-4)
7	January 24, 2018	✔	✔		
8	April 9, 2018	✔	✔		
	April 11, 2018			✔	✔

* Eastbound bridges were completed in February 2015 and opened to traffic in March 2015.

** Westbound bridges were completed in November 2015 and opened to traffic in February 2016.

Table 1-2. Average Temperature and Weather Conditions on Crack Mapping Days

Map #	Date Performed	Time	Temp-erature	Conditions
1	June 21, 2016	8:53 a.m.	78	Passing clouds
		9:53 a.m.	82	Passing clouds
		10:53 a.m.	84	Passing clouds
		11:53 a.m.	88	Scattered clouds
		12:53 p.m.	91	Partly sunny
		1:53 p.m.	89	Partly sunny
		2:53 p.m.	89	Partly sunny
1	June 22, 2016	8:53 a.m.	78	Passing clouds
		9:53 a.m.	83	Passing clouds
		10:53 a.m.	85	Passing clouds
		11:53 a.m.	87	Passing clouds
		12:53 p.m.	90	Scattered clouds
		1:53 p.m.	91	Overcast
		2:53 p.m.	92	Partly sunny
2	September 20, 2016	8:53 a.m.	76	Fog
		9:53 a.m.	78	Fog
		10:53 a.m.	85	Passing clouds
		11:53 a.m.	90	Passing clouds
		12:53 p.m.	92	Partly sunny
		1:53 p.m.	93	Scattered clouds
		2:53 p.m.	94	Scattered clouds
2	September 22, 2016	8:53 a.m.	79	Scattered clouds
		9:53 a.m.	83	Passing clouds
		10:53 a.m.	84	Passing clouds
		11:53 a.m.	86	Passing clouds
		12:53 p.m.	88	Passing clouds
		1:53 p.m.	89	Passing clouds
		2:53 p.m.	91	Passing clouds
3	January 17, 2017	8:53 a.m.	58	Partly sunny
		9:53 a.m.	64	High level clouds
		10:53 a.m.	70	Overcast
		11:53 a.m.	73	Overcast
		12:53 p.m.	76	More clouds than sun
		1:53 p.m.	75	Cloudy
		2:53 p.m.	74	Overcast
3	January 19, 2017	8:53 a.m.	60	Fog
		9:53 a.m.	62	Fog
		10:53 a.m.	63	Fog
		11:53 a.m.	66	Low clouds
		12:53 p.m.	71	Partly sunny
		1:53 p.m.	74	Partly sunny
		2:53 p.m.	75	Broken clouds
4	April 24, 2017	8:53 a.m.	58	Mostly cloudy
		9:25 a.m.	58	More clouds than sun
		10:53 a.m.	59	More clouds than sun
		11:53 a.m.	62	Cloudy
		12:53 p.m.	64	Mostly cloudy
		1:53 p.m.	64	Overcast
		2:53 p.m.	63	Cloudy
4	April 25, 2017	8:53 a.m.	61	Sunny
		9:53 a.m.	68	Passing clouds
		10:53 a.m.	72	Passing clouds
		11:53 a.m.	74	Scattered clouds
		12:53 p.m.	76	Partly sunny
		1:53 p.m.	76	Partly sunny
		2:53 p.m.	78	Passing clouds
5	July 5, 2017	8:53 a.m.	80	Passing clouds
		9:53 a.m.	84	Passing clouds
		10:53 a.m.	87	Passing clouds
		11:53 a.m.	89	Scattered clouds
		12:53 p.m.	91	Scattered clouds
		1:53 p.m.	94	Partly sunny
		2:53 p.m.	94	Partly sunny
6	October 2, 2017	8:53 a.m.	71	Overcast
		9:53 a.m.	73	Overcast
		10:53 a.m.	76	Overcast
		11:53 a.m.	78	Overcast
		12:53 p.m.	82	Broken clouds
		1:53 p.m.	84	Broken clouds
		2:53 p.m.	84	Partly sunny
6	October 3, 2017	8:53 a.m.	72	Passing clouds
		9:53 a.m.	75	Scattered clouds
		10:53 a.m.	79	Sunny
		11:53 a.m.	83	Partly sunny
		12:53 p.m.	82	Partly sunny
		1:53 p.m.	85	More clouds than sun
		2:53 p.m.	86	Broken clouds
6	October 17, 2017	8:53 a.m.	63	Partly sunny
		9:53 a.m.	63	Broken clouds
		10:53 a.m.	68	Partly sunny
		11:53 a.m.	70	Scattered clouds
		12:53 p.m.	72	Passing clouds
		1:53 p.m.	74	Passing clouds
		2:53 p.m.	75	Passing clouds
7	January 24, 2018	8:53 a.m.	40	Sunny
		9:53 a.m.	47	Scattered clouds
		10:53 a.m.	50	Scattered clouds
		11:53 a.m.	53	Sunny
		12:53 p.m.	53	Partly sunny
		1:53 p.m.	54	Partly sunny
		2:53 p.m.	55	Partly sunny
8	April 9, 2018	9:20 a.m.	61	Overcast
		9:53 a.m.	62	Overcast
		10:53 a.m.	67	Overcast
		11:53 a.m.	71	Overcast
		1:05 p.m.	74	Broken clouds
		1:53 p.m.	75	Broken clouds
		2:53 p.m.	77	Overcast
8	April 11, 2018	8:53 a.m.	53	Sunny
		9:53 a.m.	62	Sunny
		10:53 a.m.	65	Sunny
		11:53 a.m.	70	Passing clouds
		12:53 p.m.	72	Passing clouds
		1:53 p.m.	74	Passing clouds
		2:53 p.m.	75	Passing clouds

Note: Data was obtained from www.timeanddate.com for Tallahassee, FL.

Figure 1-1 shows a layout of the bridges, spans, and panels, and **Figure 1-2** indicates the order in which the spans were mapped. Each span has 13 panels (except for Little River Span 1, which has 12 panels), for a total of 128 panels. Typically, each student tended to two (2) panels per span.

The US 90 bridges have nearly 44,000 square feet of bridge deck area (precast panel top surface area) and hundreds of cracks, and therefore a full crack evaluation according to FDOT specifications would be laborious. Instead, an evaluation was performed that was part quantitative and part qualitative. The approach that was used to analyze the cracks is described in the remaining sections of this report, and observations from each stage of the approach are provided.

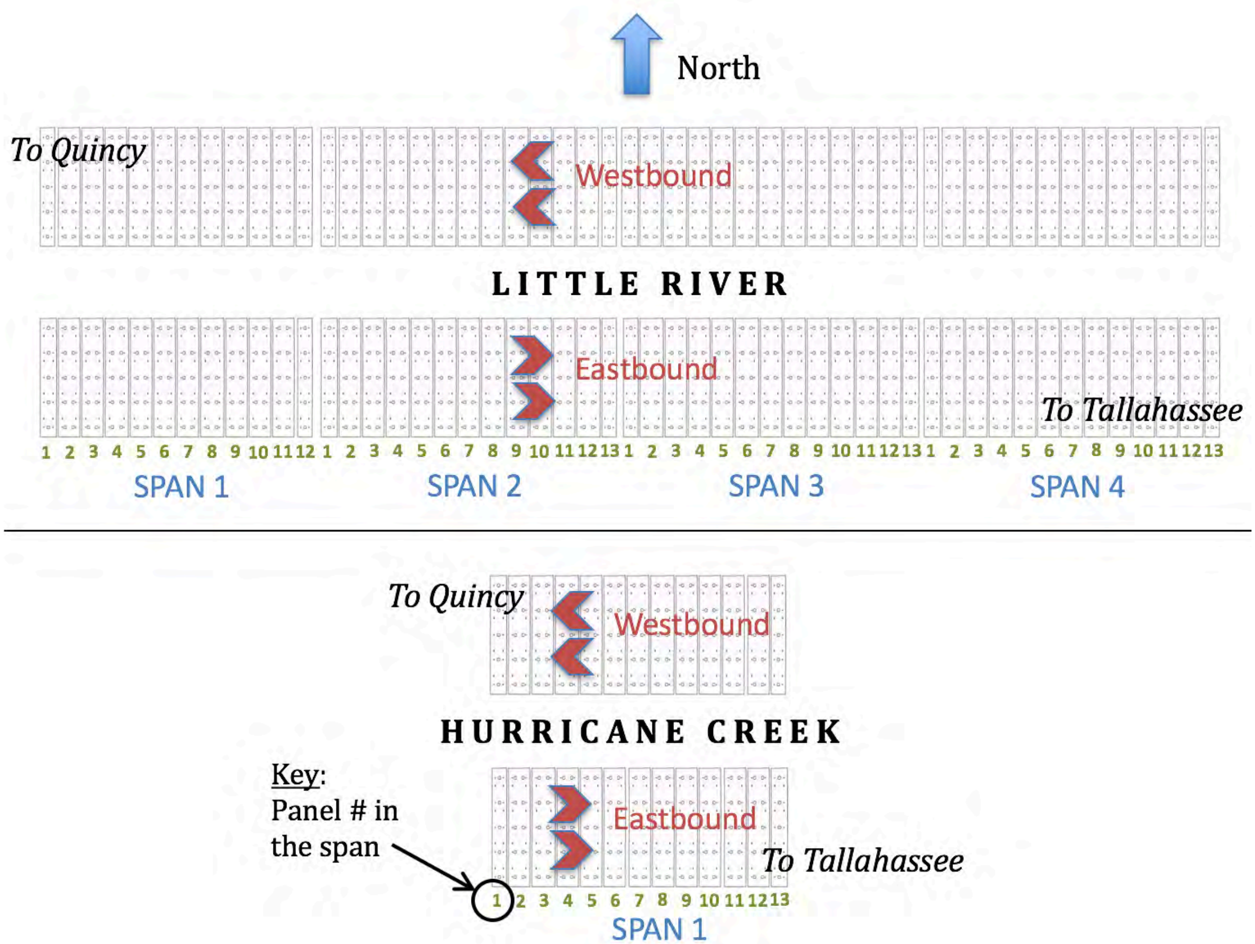


Fig. 1-1. Layout of Bridges, Spans, and Panels

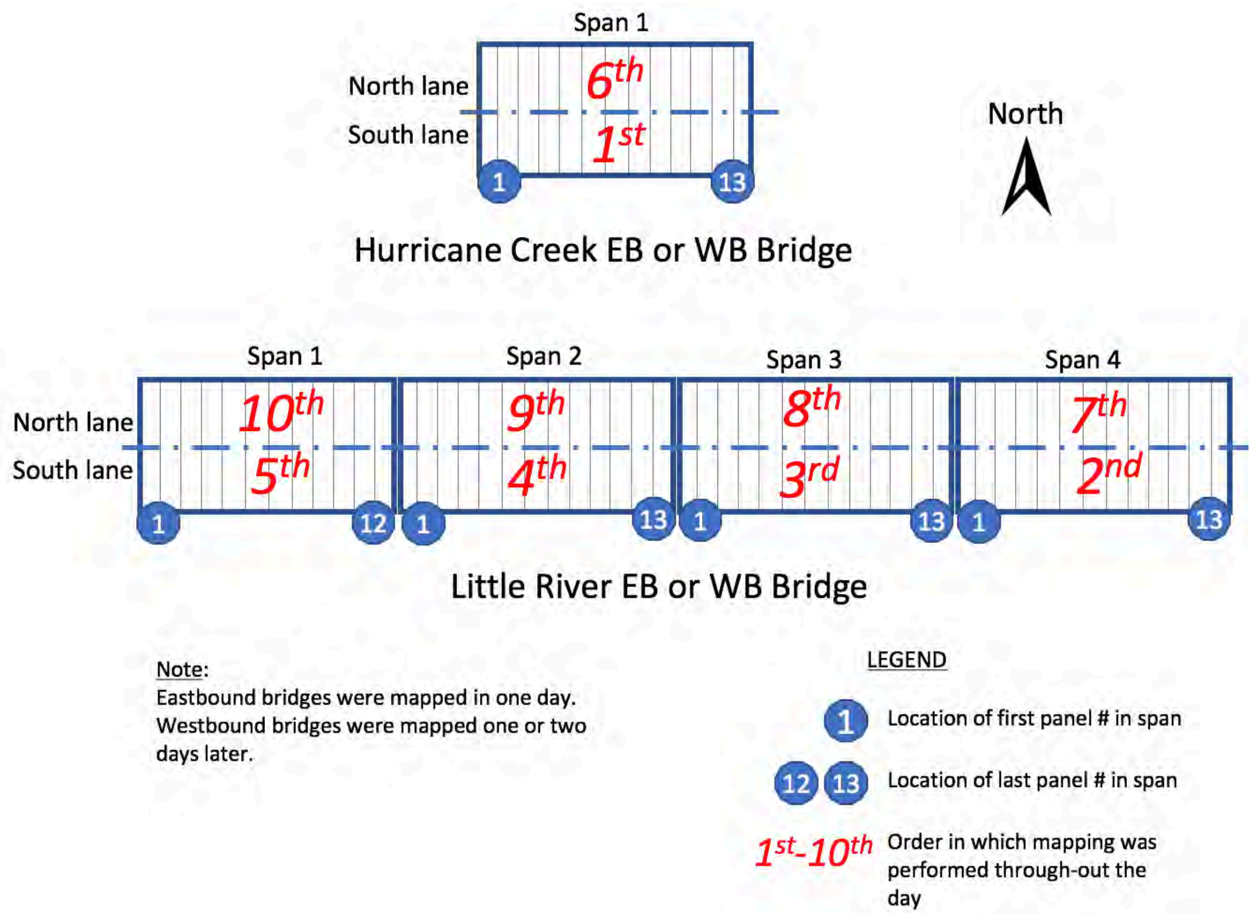


Fig. 1-2. Order in Which Spans Were Mapped

CHAPTER 2
CRACK MAPPING DATA

2.1 Crack Mapping of Panels

The cracks were sketched on a letter-sized sheet of paper – on a pre-printed diagram of a single panel. The diagram also included, on each side of the panel, a thin rectangle on which to record any cracks that were noticed in the closure joints. A number was written next to each crack, and information about that crack was documented in a table under the diagram. Lengths of shorter cracks were documented; the lengths of longer cracks were typically not documented if it was clear from the sketch how long the cracks were. For example, many cracks extended the entire panel width, were between the panels' edge and a shear pocket, or were between two (2) shear pockets.

Widths were inspected visually – sometimes with the use of a crack comparator. The widths were documented (in inches) on the sheet. Very fine, "hairline" cracks were recorded as "HL". Locations where spalling had occurred were also marked.

After a mapping (e.g., #1) was completed, the handwritten information was neatened, and the sheets were scanned to PDF files. The original sheets were reused for the next mapping (e.g., #2) approximately three (3) months later. Likewise, for subsequent mappings #3 – #8.

For six (6) of the eight (8) mappings, there are 128 panel maps; for mapping #5 and #7, there are 64 maps.

General observations are as follows:

- The maps clearly indicate a tendency for the panels to crack along the panels' short direction (which is longitudinal to the bridge, in the direction of traffic).

- Many of the cracks extend along the lines of the shear pockets. Cracking was much more likely to occur over the three (3) interior girders than over the two (2) exterior girders.
- Several cracks extend along the line of lifting loops that were installed for construction purposes and later removed and grouted. Cracking was more prevalent along the two (2) interior lines of lifting loops than along the outer two (2) lines that are adjacent to the exterior girders.
- The shear pocket and lifting loop locations were the most problematic places where cracking occurred during precasting and construction. However, new cracks and crack growth have not been limited to those locations.
- Cracking has occurred mostly in the middle half of the bridge width – between girders 2 and 4.
- There are far fewer cracks in the westbound bridges than in the eastbound bridges.
- The joints between the panels and closure joints were in very good condition. The two components were not separated (other than a fine "crack" between the two concrete materials), nor was there any spalling or damage.
- Several of the shear pockets had spalls. This seemed to occur in the shear pockets that had been repaired during construction. These repairs are discussed in some detail in Report 3, Construction of Precast Elements, in the section titled "Shear Pocket and Haunch Grouting", in the "Lessons Learned" subsection. There were also spalls in some of the lifting eye locations, perhaps also because of repairs made during construction.

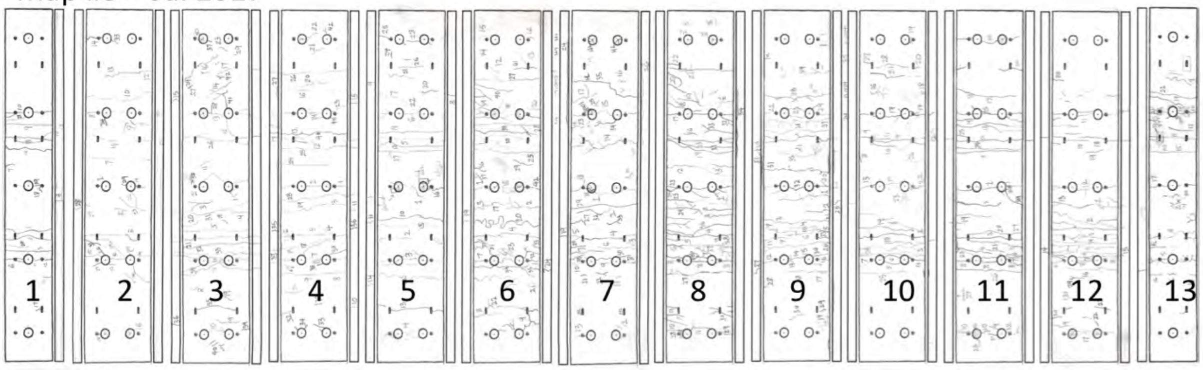
2.2 Composite Crack Maps of Spans

For each span, a composite map (JPG or PNG format) was created by digitally extracting the panel sketches from the PDF files described in the section above and placing the images beside each other in their arrangement in the span. The composite maps are provided in **Figures 2-1 to 2-20**, to help the reader quickly compare the six (6) or eight (8) mappings for a given span.

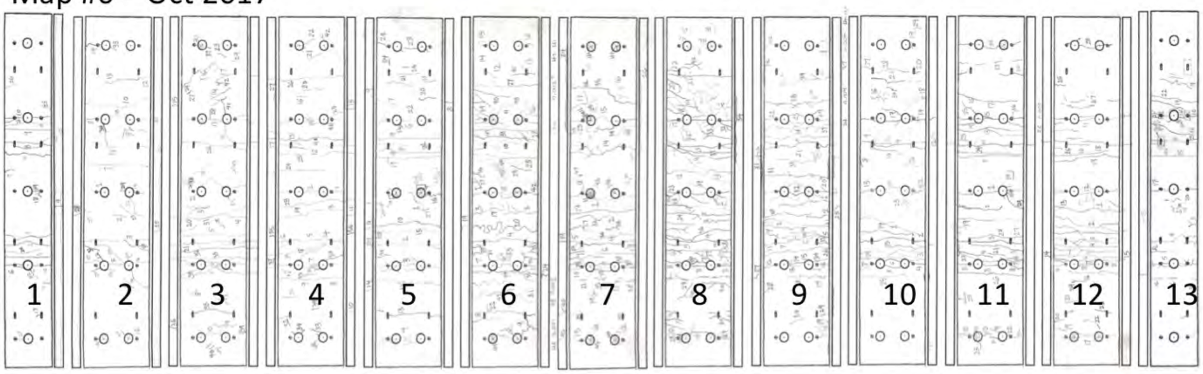
On the maps, the two thin rectangles between every two adjacent panels represent a closure joint. (Theoretically, one of the two rectangles would be redundant. However, both were left in, to preserve the original documentation which could have been made by one or two students.)

Fig. 2-1. Crack Maps for Hurricane Creek Eastbound Bridge

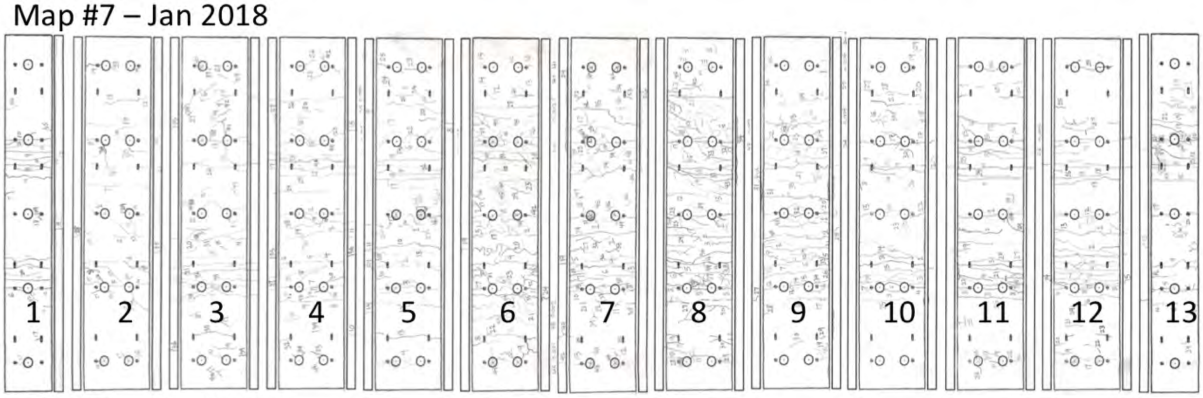
Map #5 – Jul 2017



Map #6 – Oct 2017



Map #7 – Jan 2018



Map #8 – Apr 2018

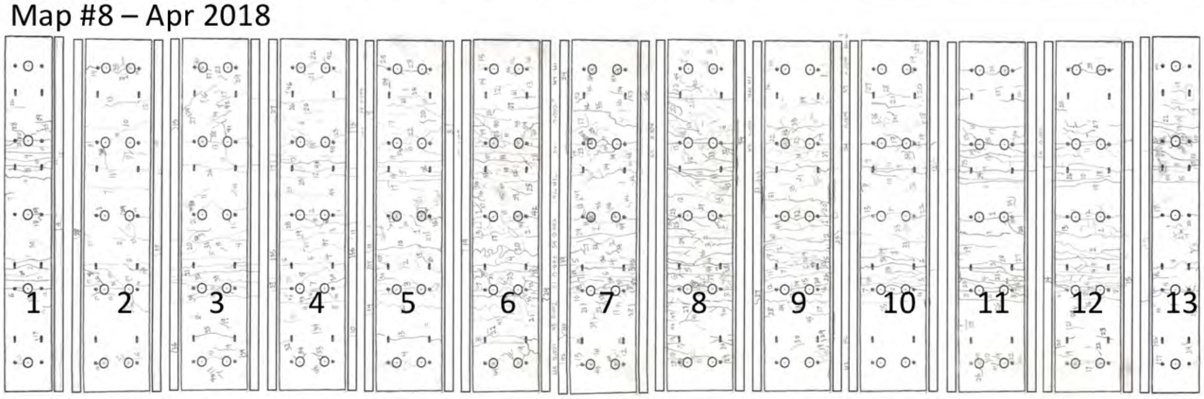


Fig. 2-2. Crack Maps for Hurricane Creek Eastbound Bridge (cont'd)

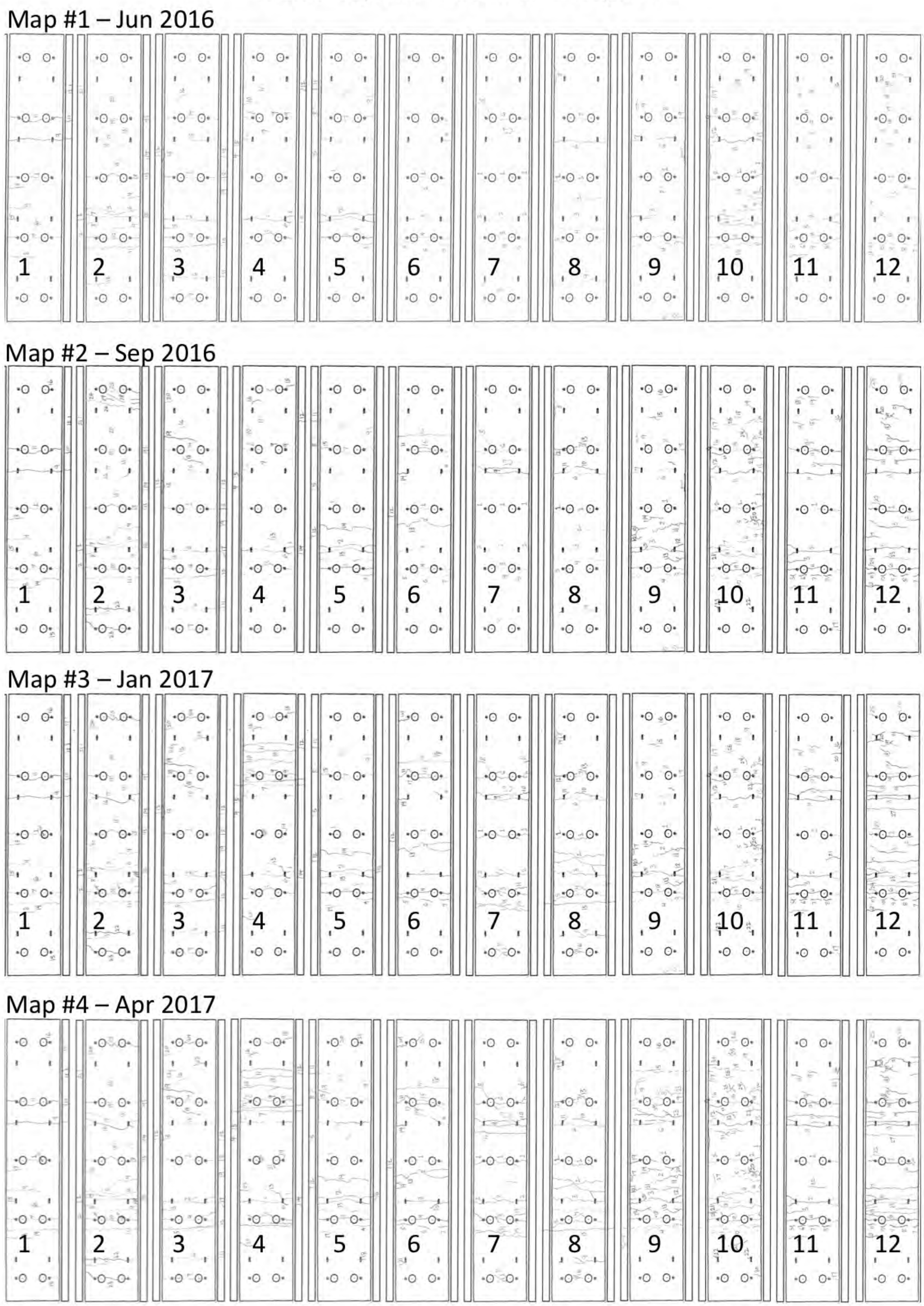
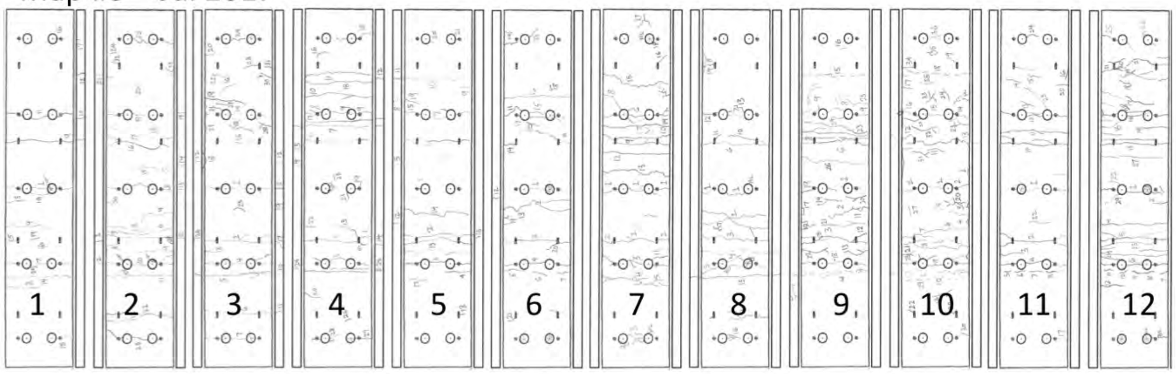


Fig. 2-3. Crack Maps for Little River Eastbound Span 1

Map #5 – Jul 2017



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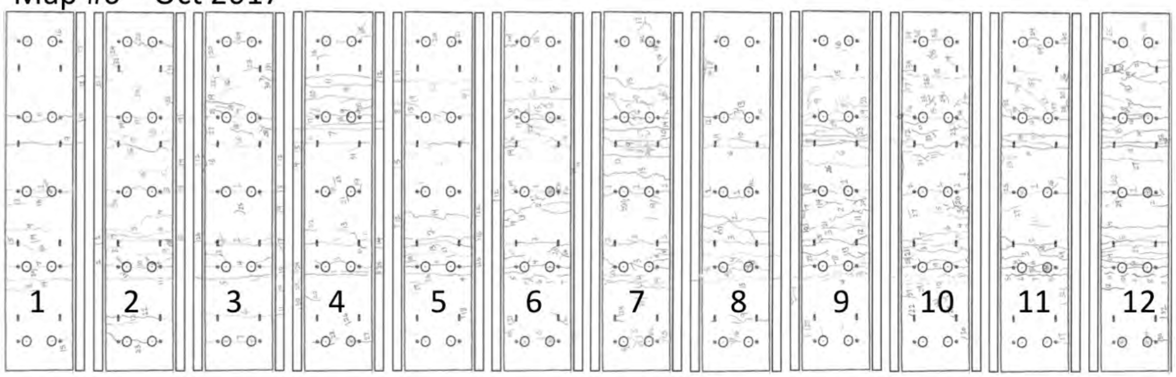

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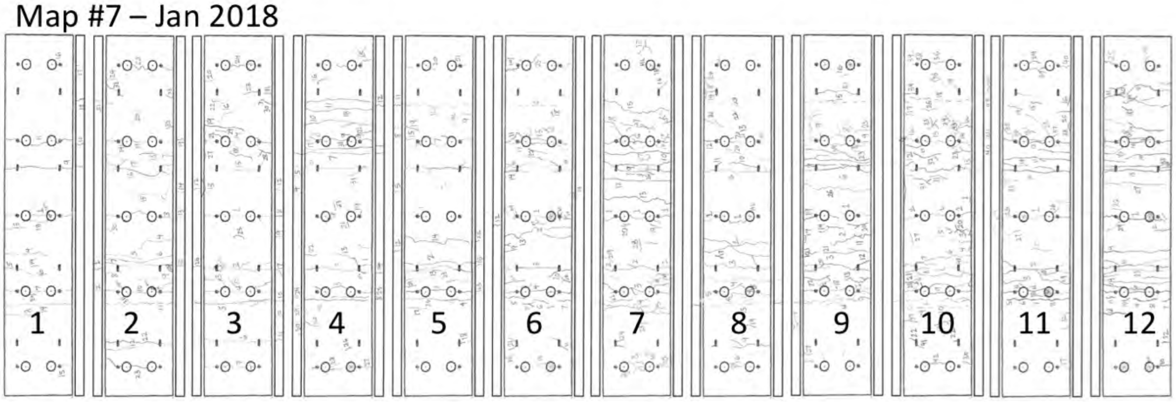

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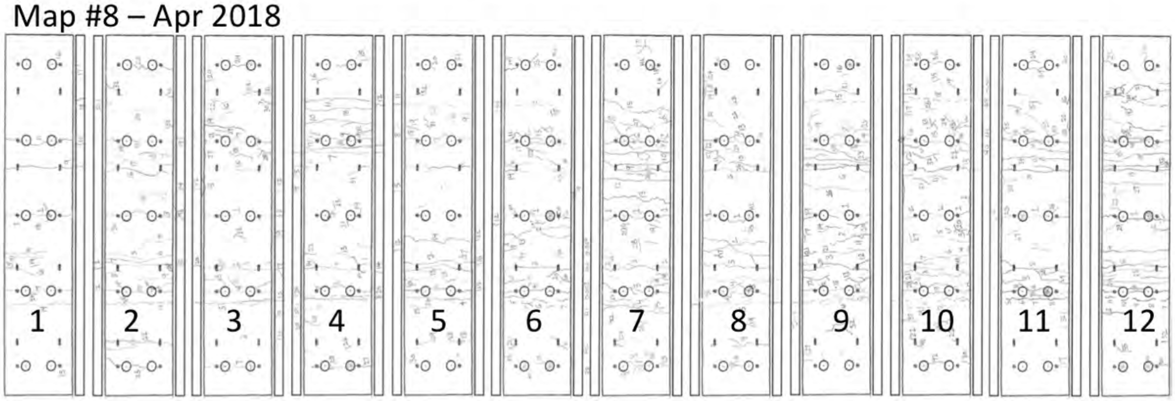


Fig. 2-4. Crack Maps for Little River Eastbound Span 1 (cont'd)

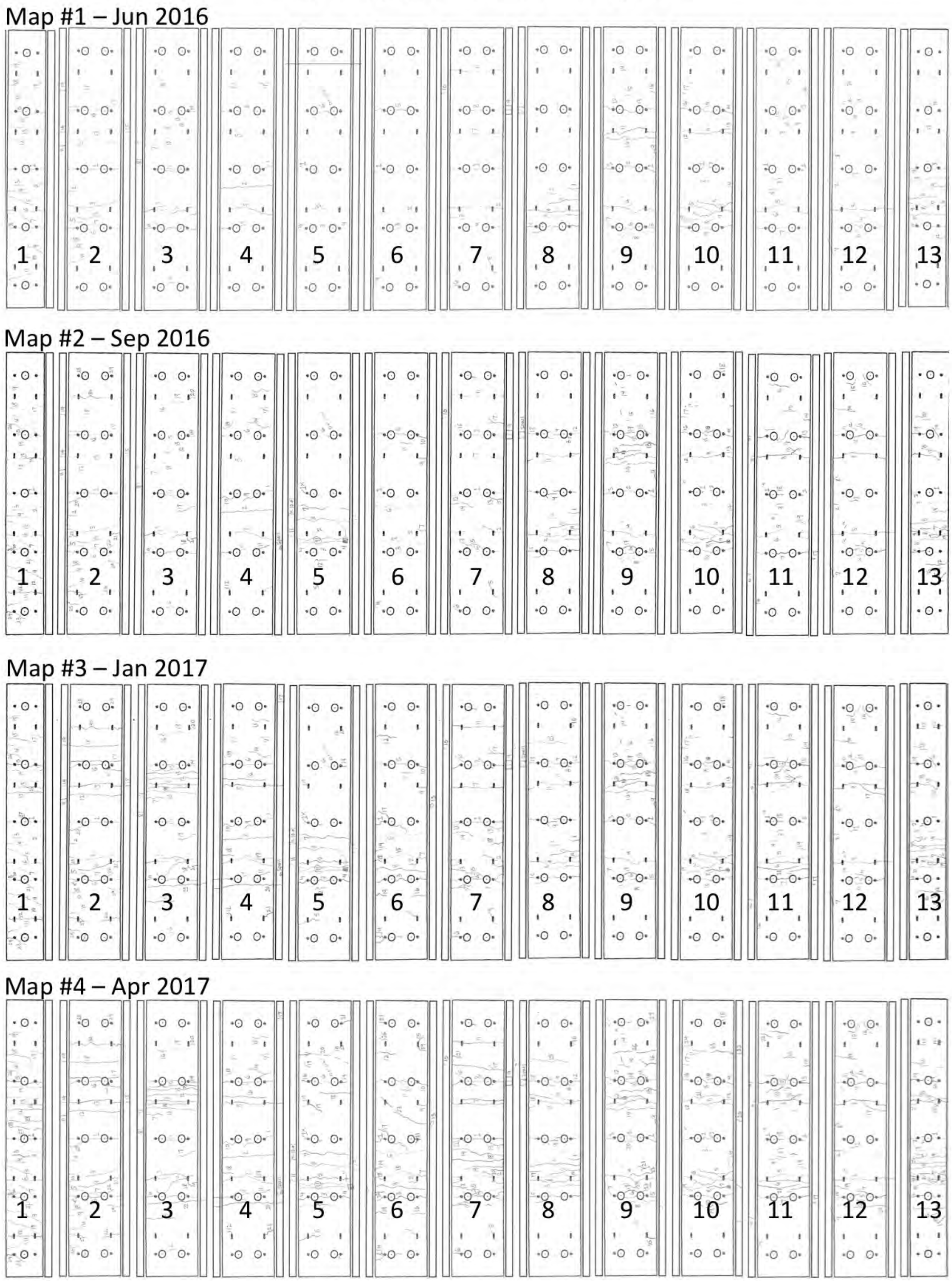


Fig. 2-5. Crack Maps for Little River Eastbound Span 2

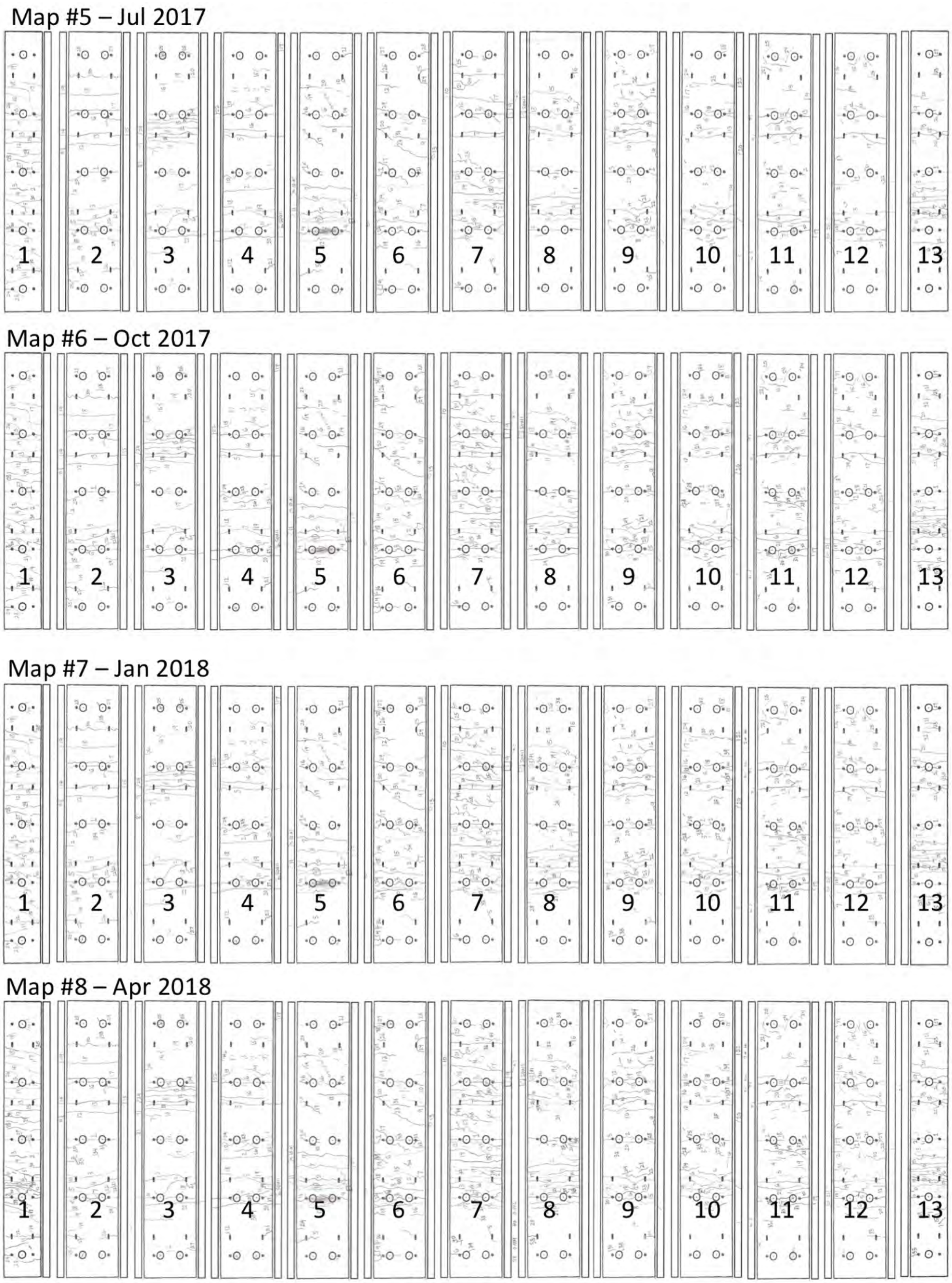


Fig. 2-6. Crack Maps for Little River Eastbound Span 2 (cont'd)

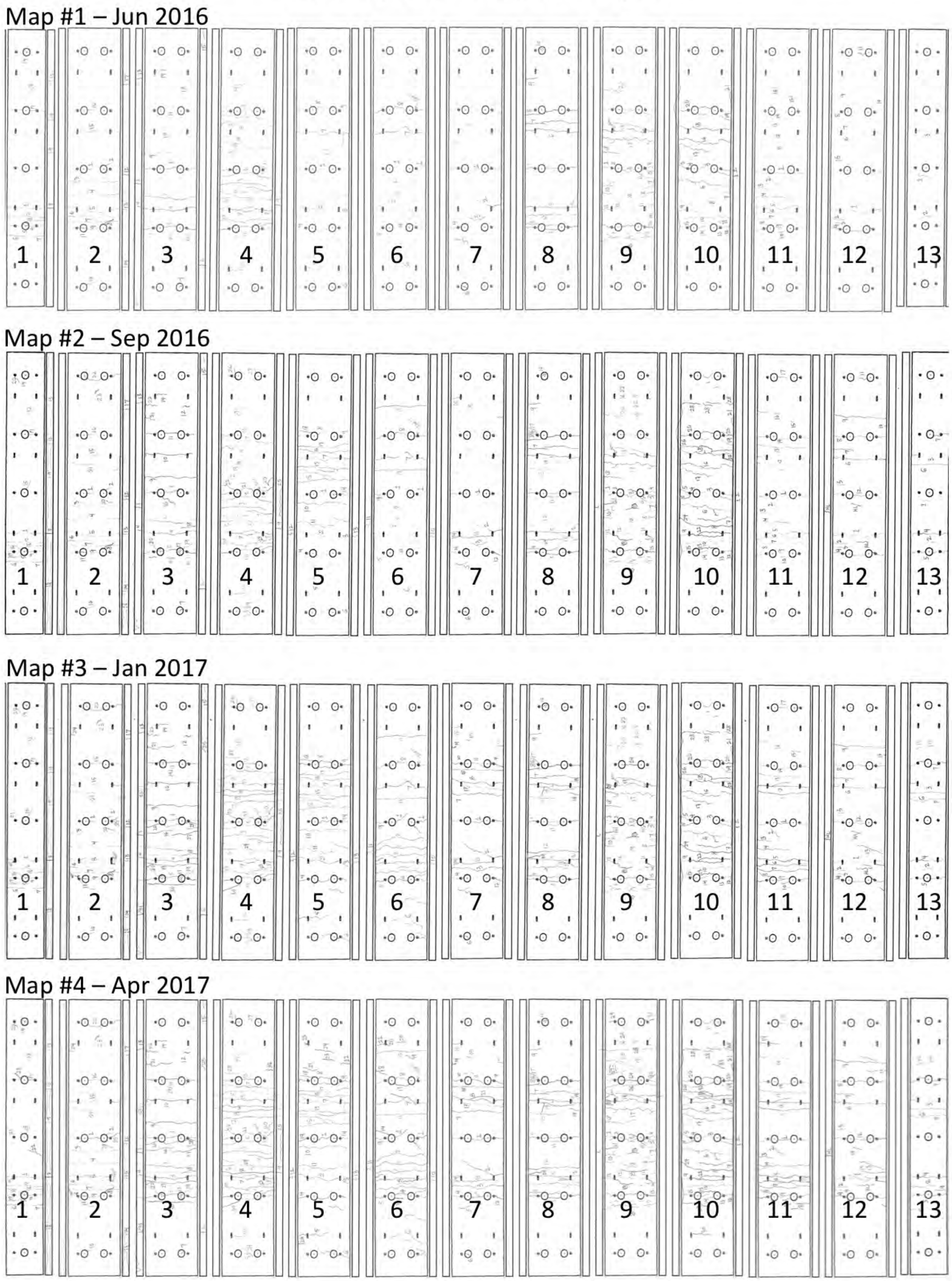
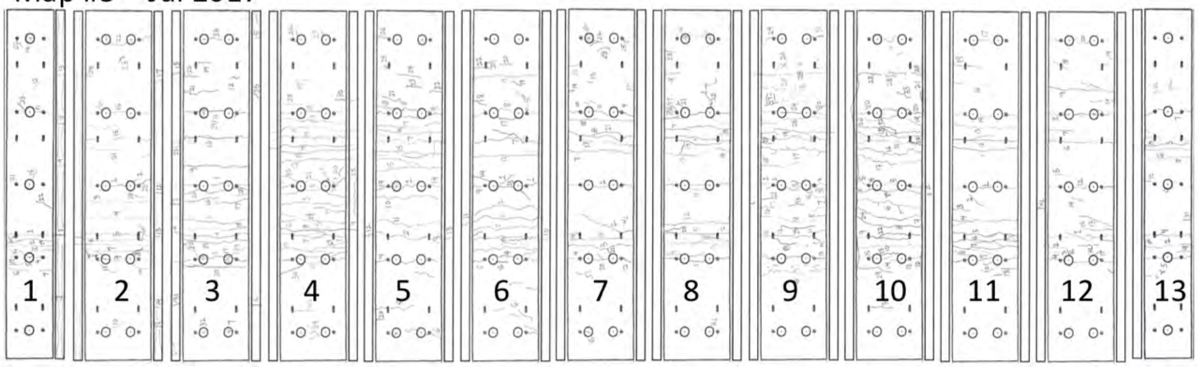
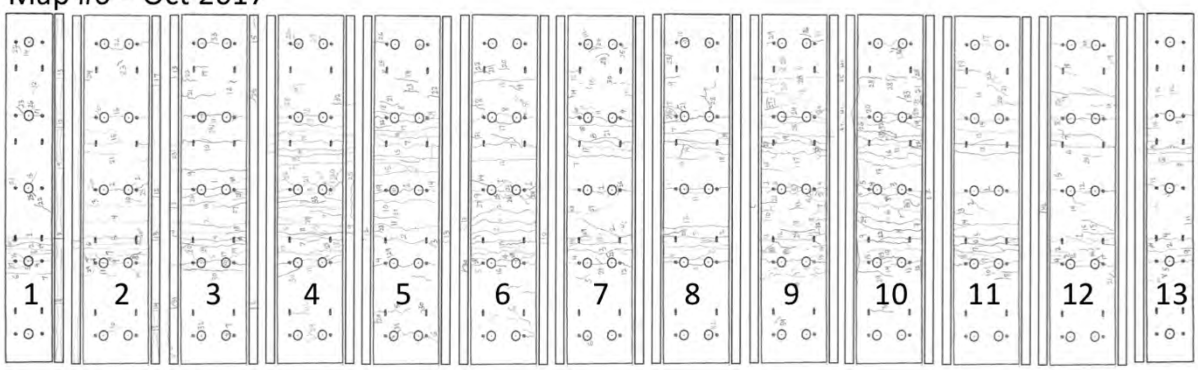


Fig. 2-7. Crack Maps for Little River Eastbound Span 3

Map #5 – Jul 2017



Map #6 – Oct 2017



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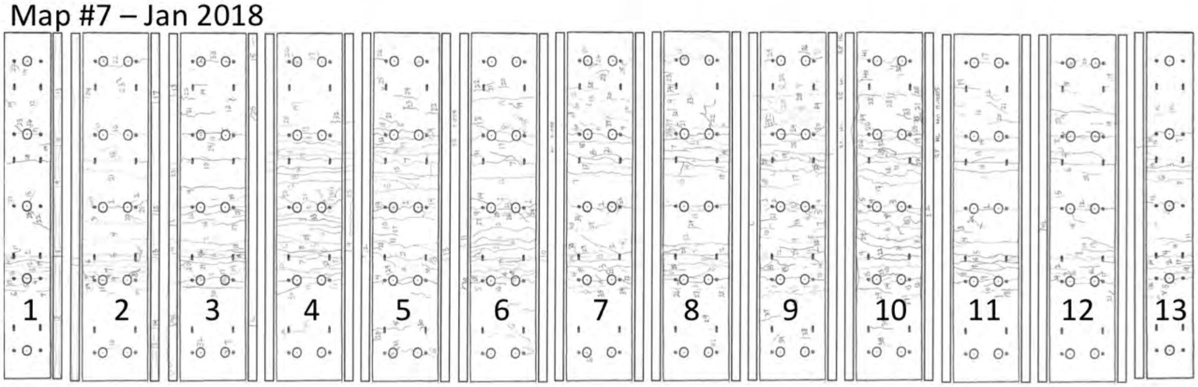

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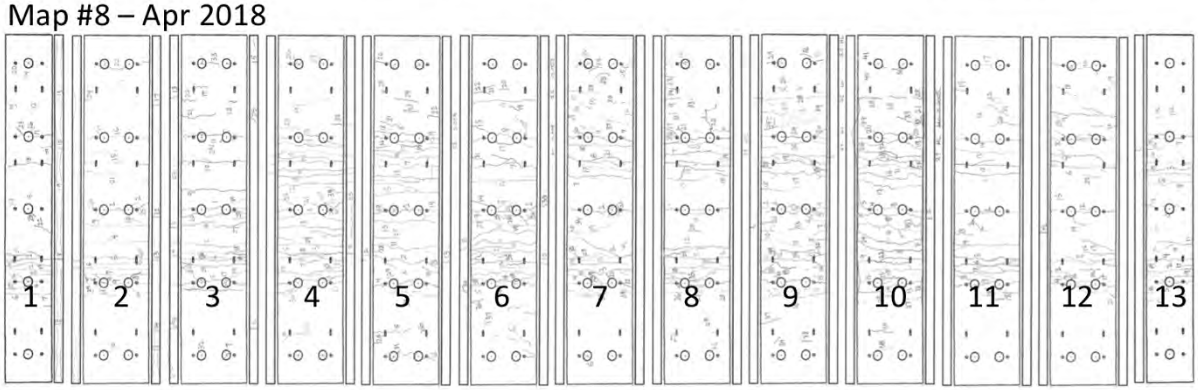


Fig. 2-8. Crack Maps for Little River Eastbound Span 3 (cont'd)

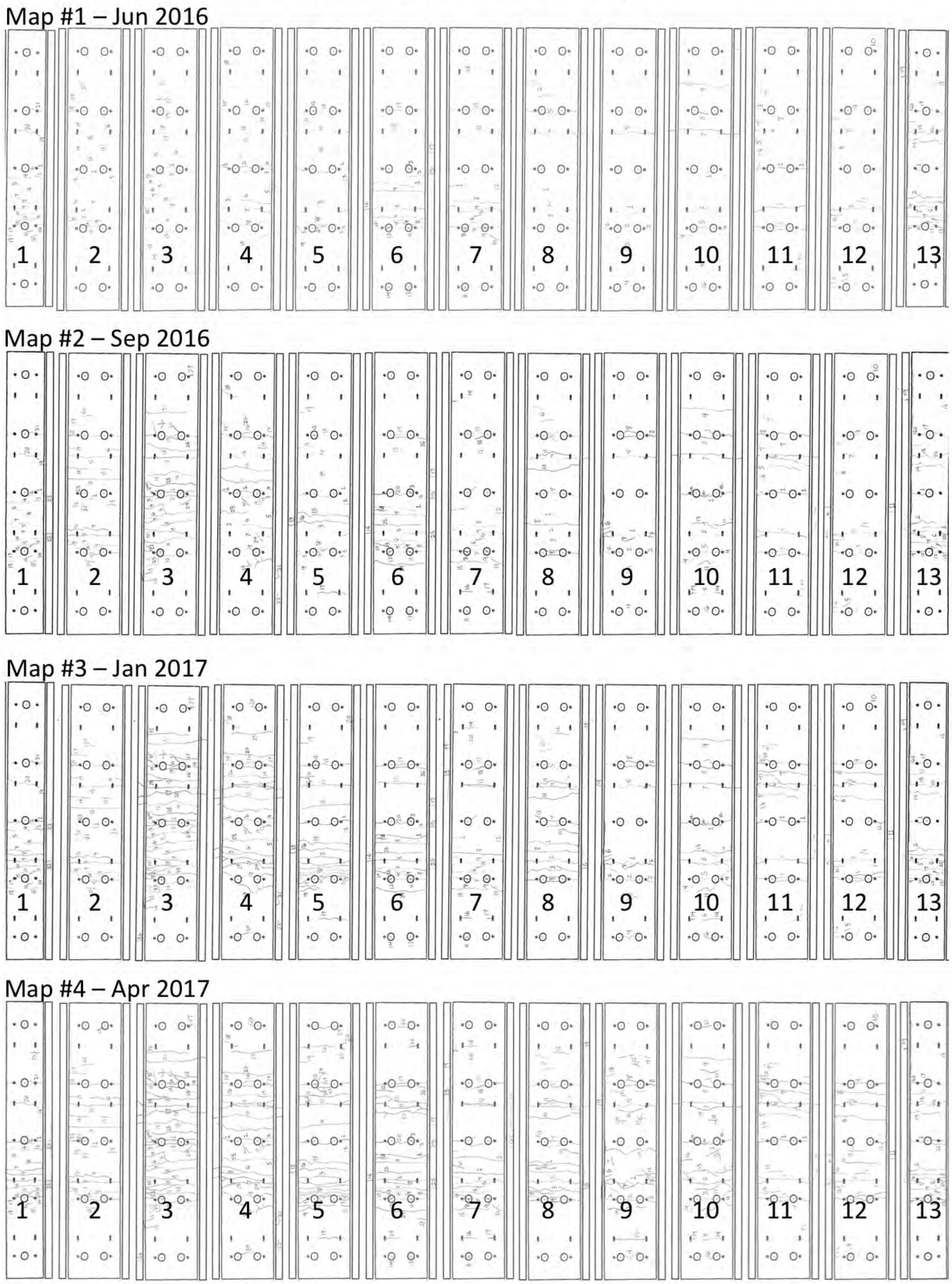
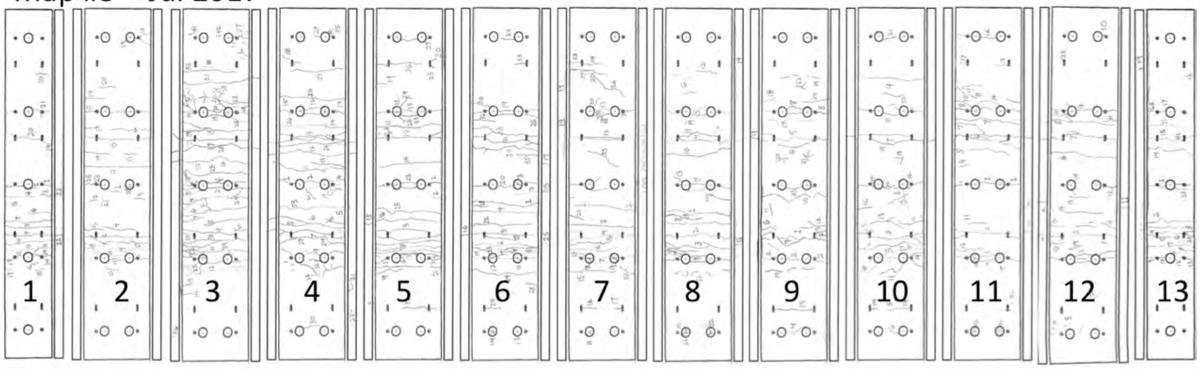
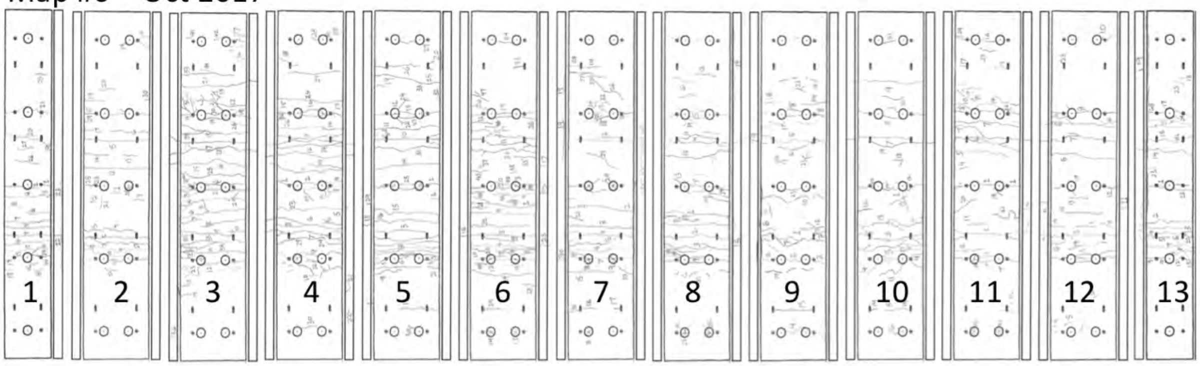


Fig. 2-9. Crack Maps for Little River Eastbound Span 4

Map #5 – Jul 2017



Map #6 – Oct 2017



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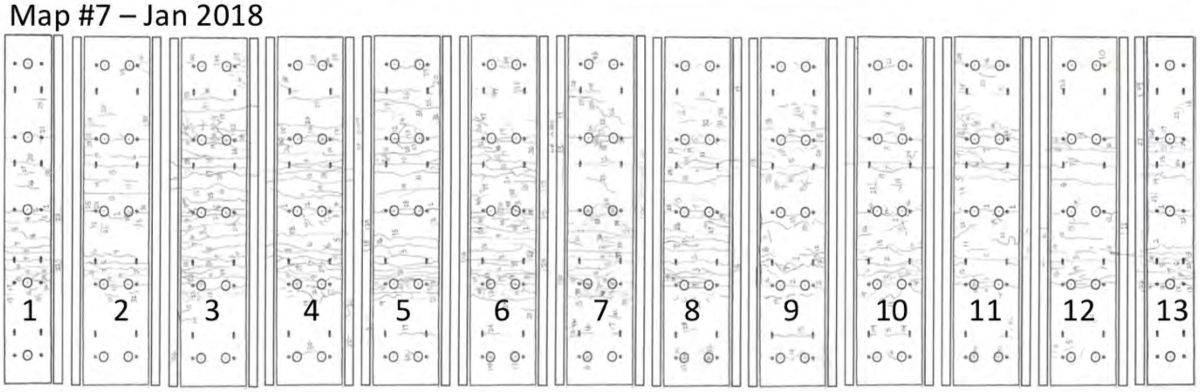


Map #8 – Apr 2018

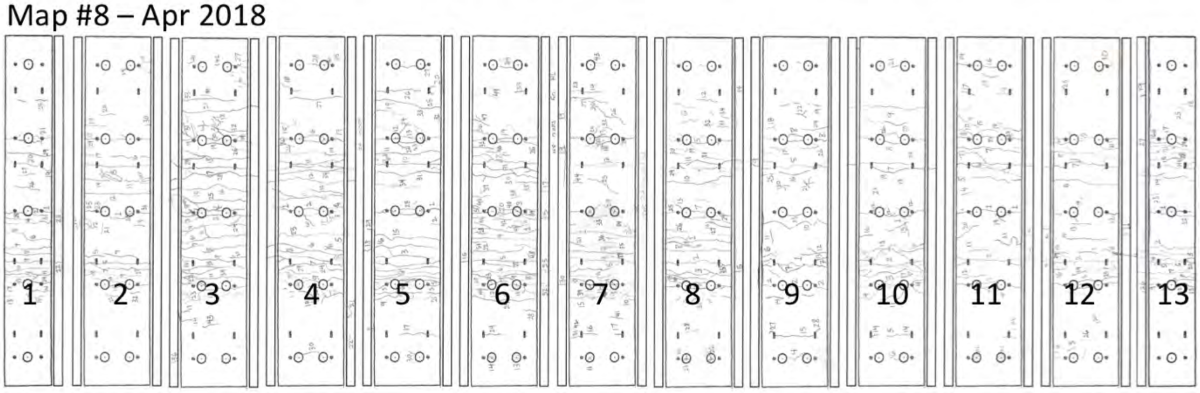


Fig. 2-10. Crack Maps for Little River Eastbound Span 4 (cont'd)

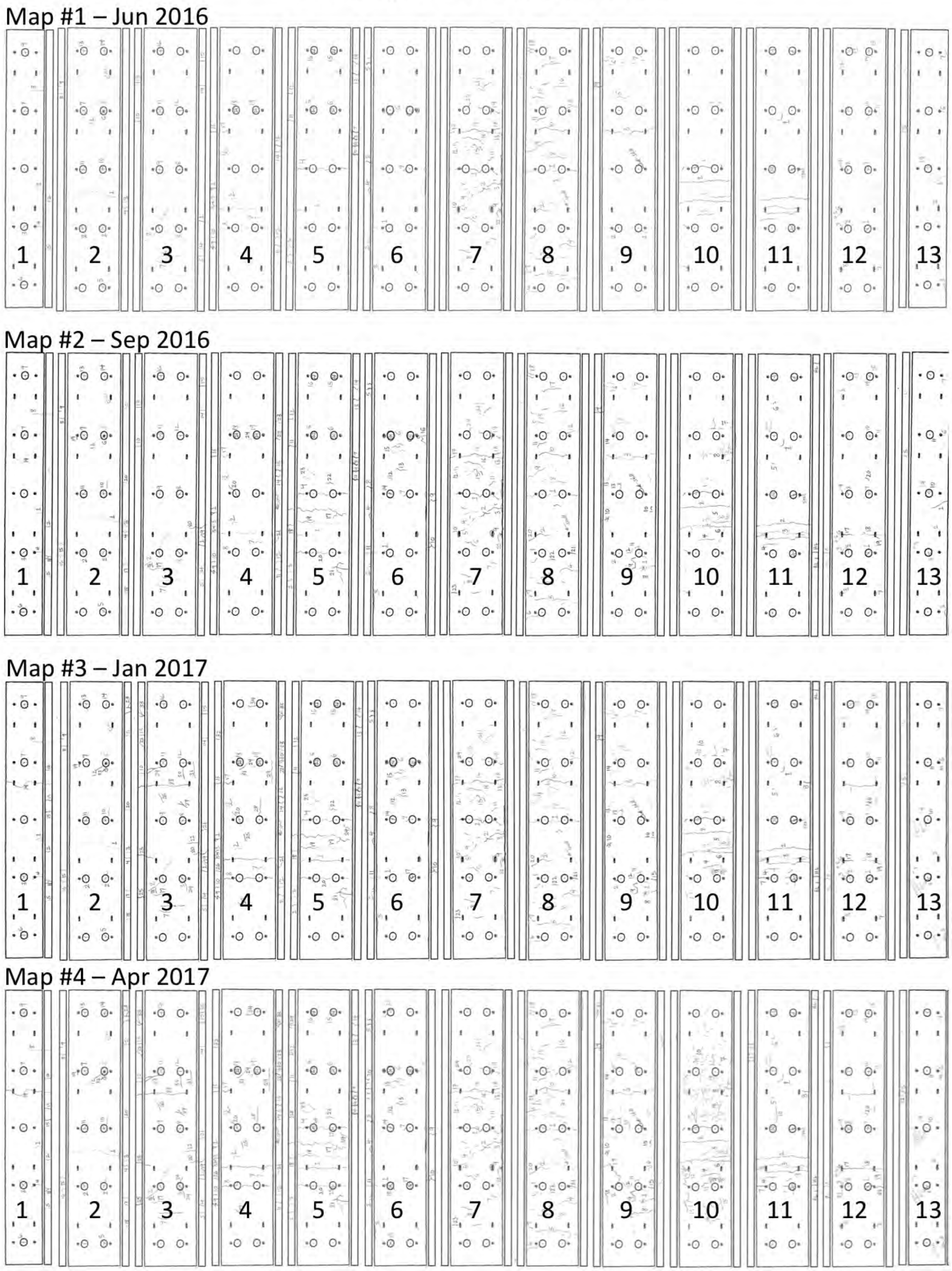
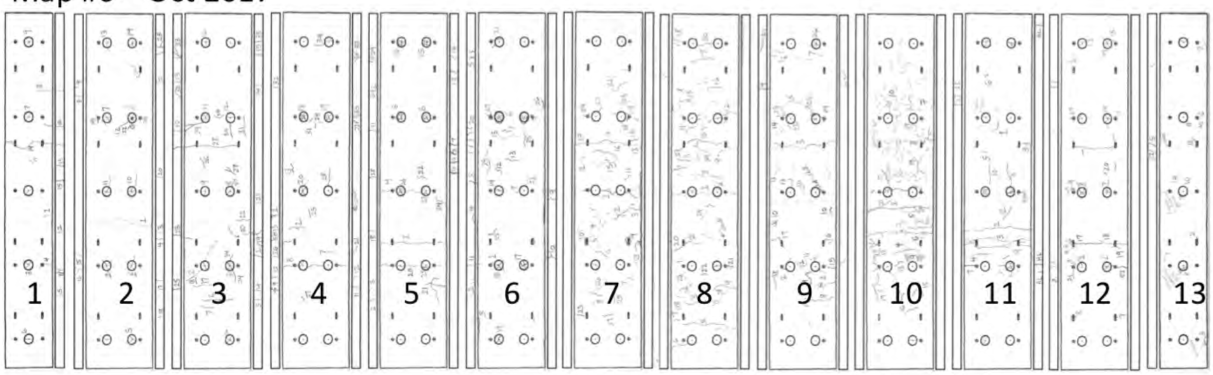


Fig. 2-11. Crack Maps for Hurricane Creek Westbound Bridge

Map #5 – Jul 2017

Mapping #5 was
not performed

Map #6 – Oct 2017



Map #7 – Jan 2018

Mapping #7 was
not performed

Map #8 – Apr 2018

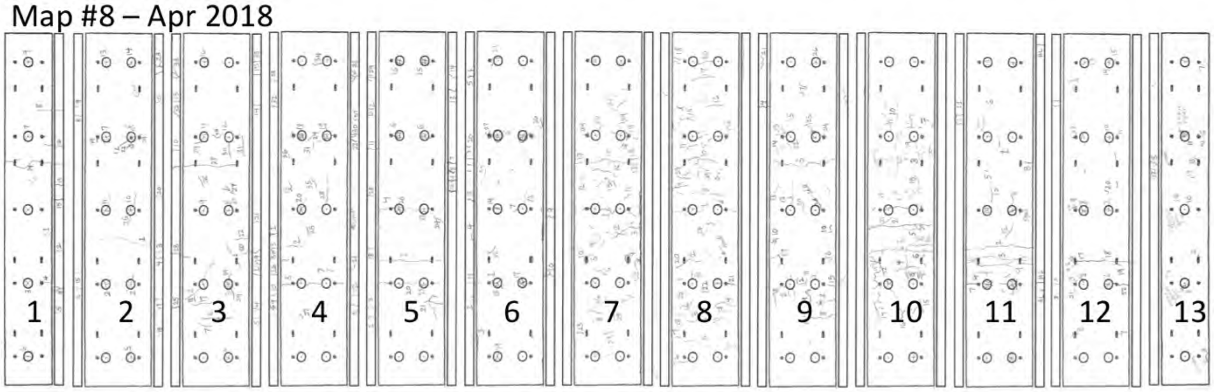


Fig. 2-12. Crack Maps for Hurricane Creek Westbound Bridge (cont'd)

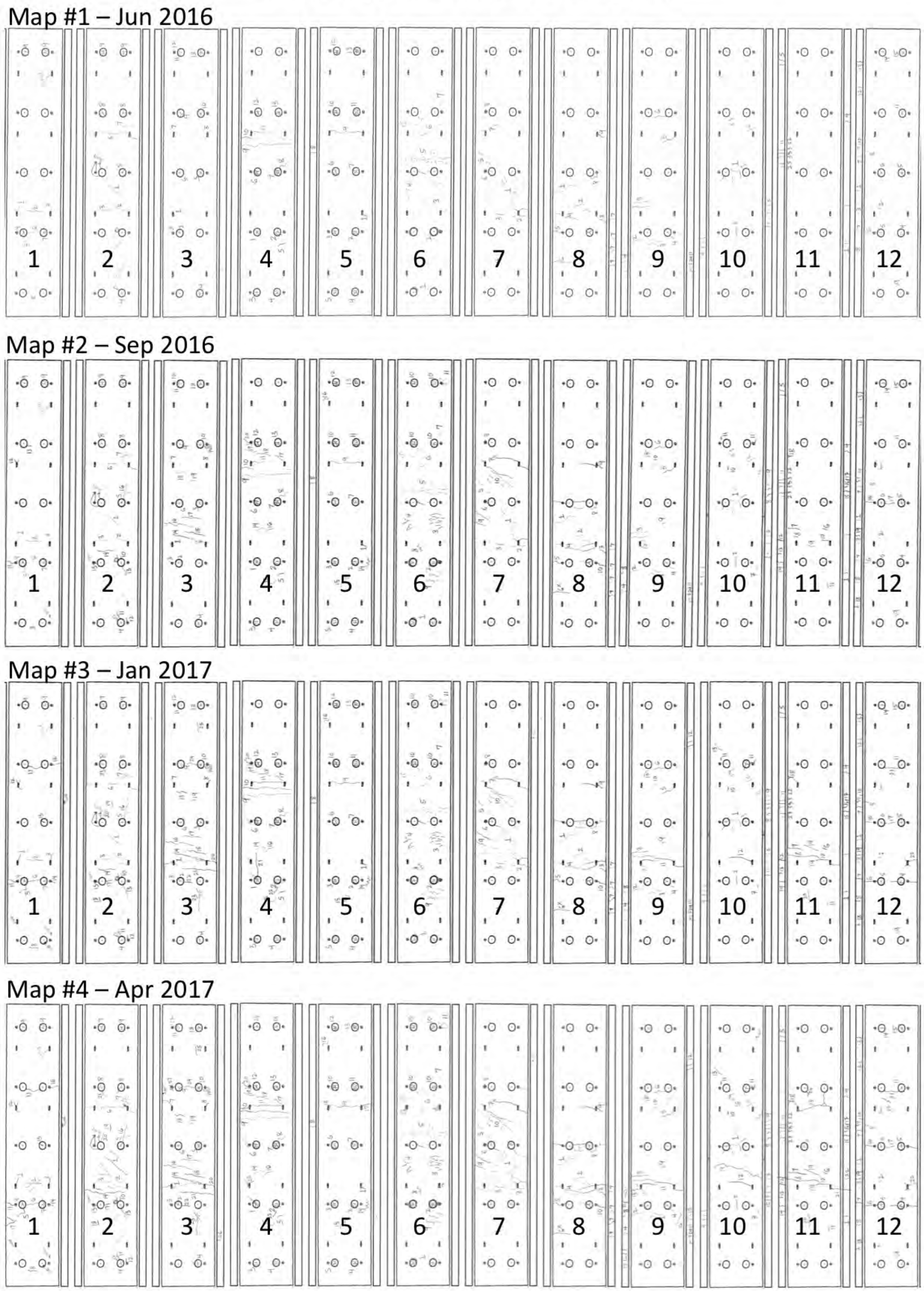
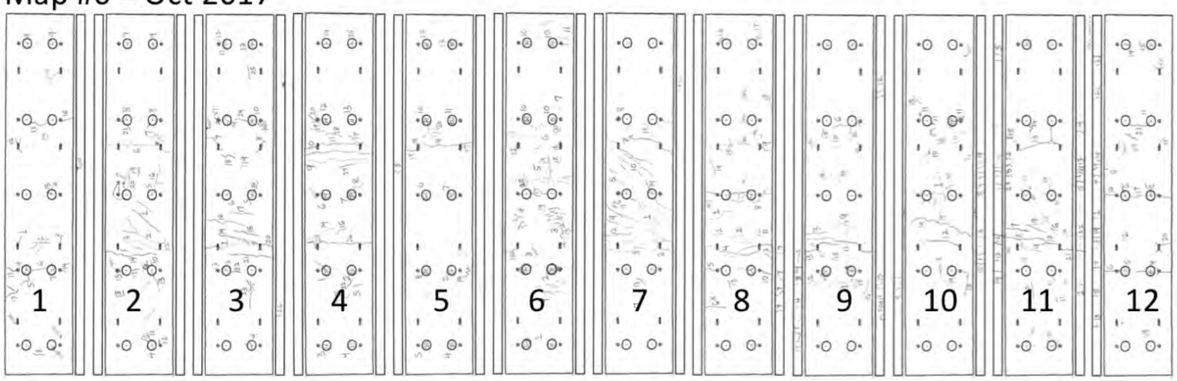


Fig. 2-13. Crack Maps for Little River Westbound Span 1

Map #5 – Jul 2017

Mapping #5 was
not performed

Map #6 – Oct 2017



Map #7 – Jan 2018

Mapping #7 was
not performed

Map #8 – Apr 2018

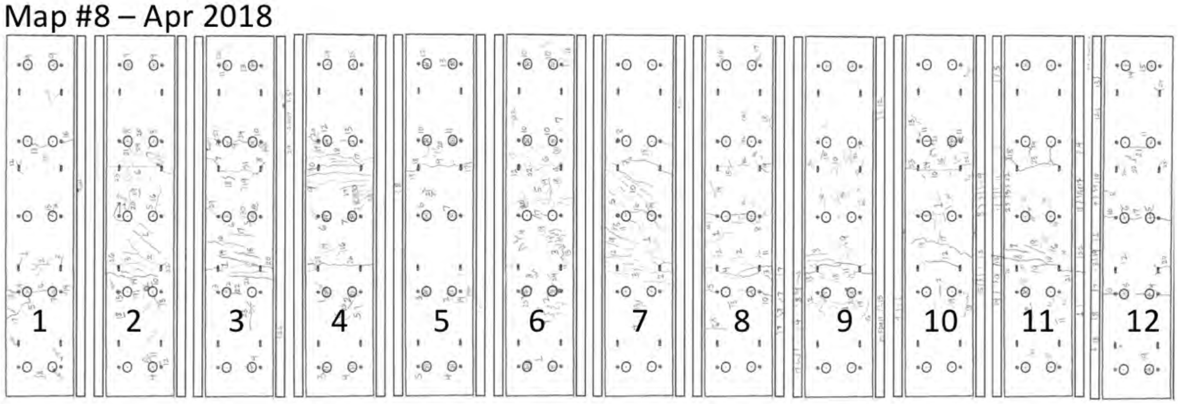


Fig. 2-14. Crack Maps for Little River Westbound Span 1 (cont'd)

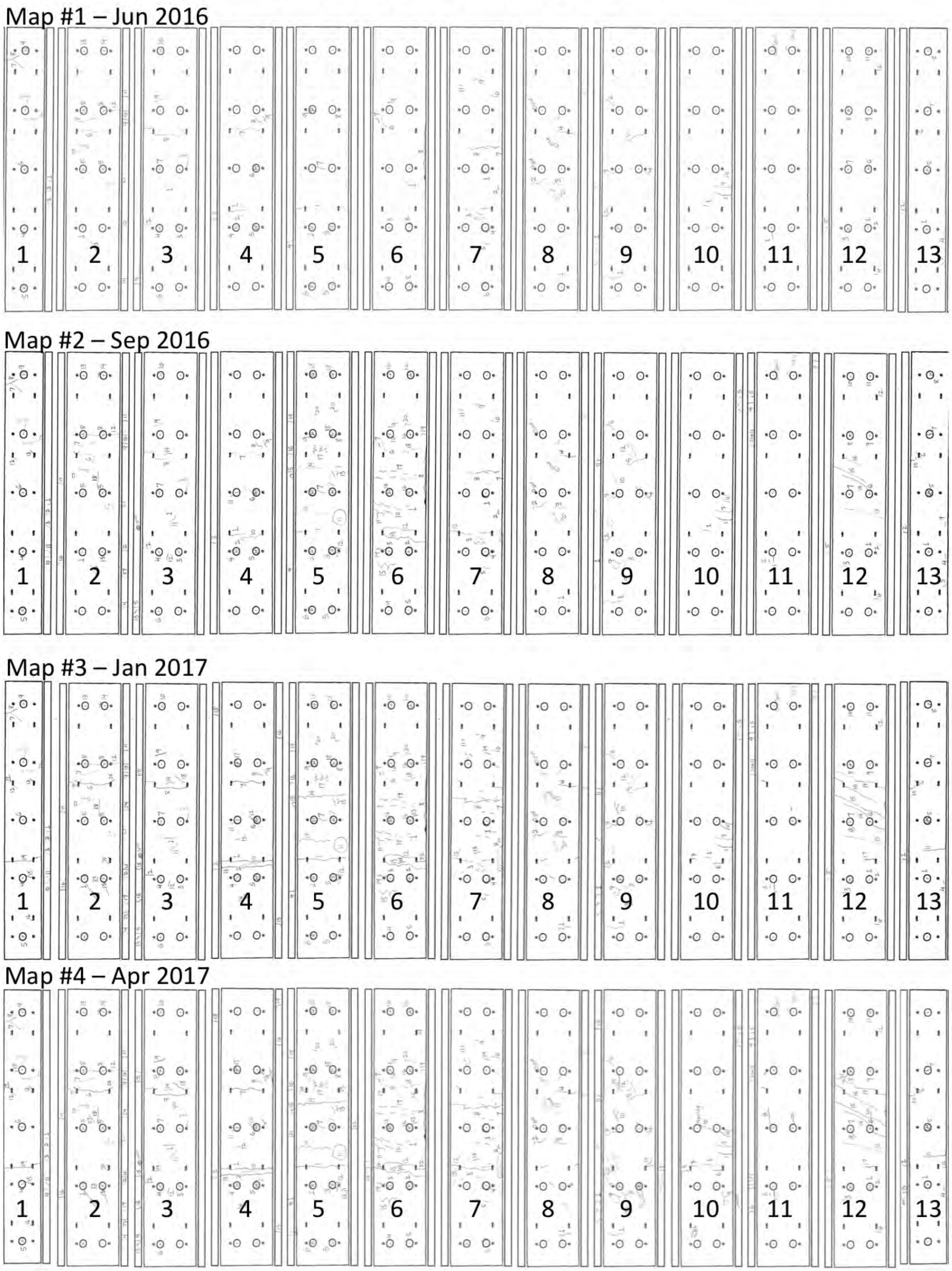
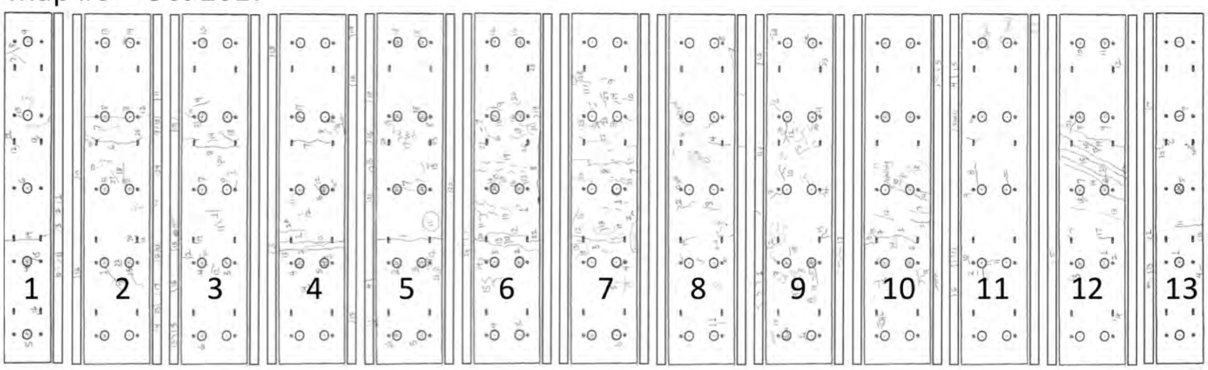


Fig. 2-15. Crack Maps for Little River Westbound Span 2

Map #5 – Jul 2017

Mapping #5 was
not performed

Map #6 – Oct 2017



Map #7 – Jan 2018

Mapping #7 was
not performed

Map #8 – Apr 2018

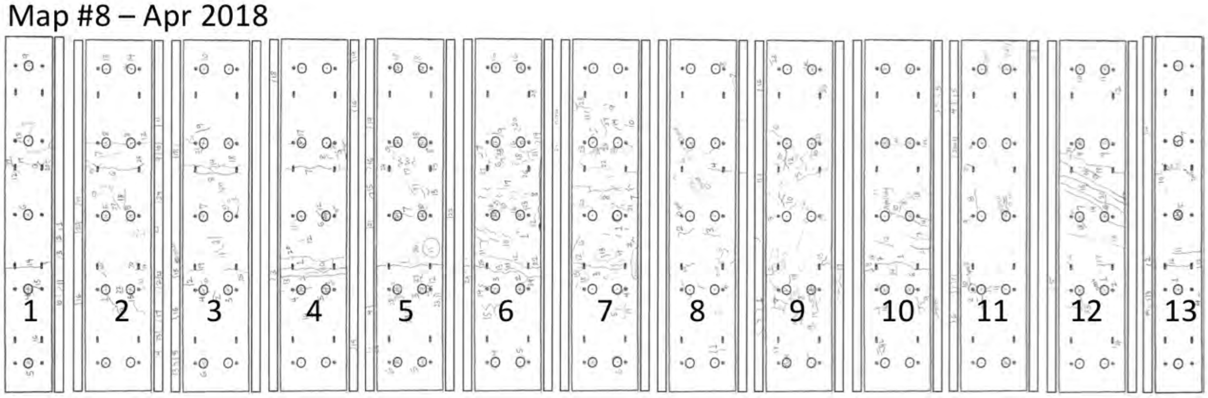


Fig. 2-16. Crack Maps for Little River Westbound Span 2 (cont'd)

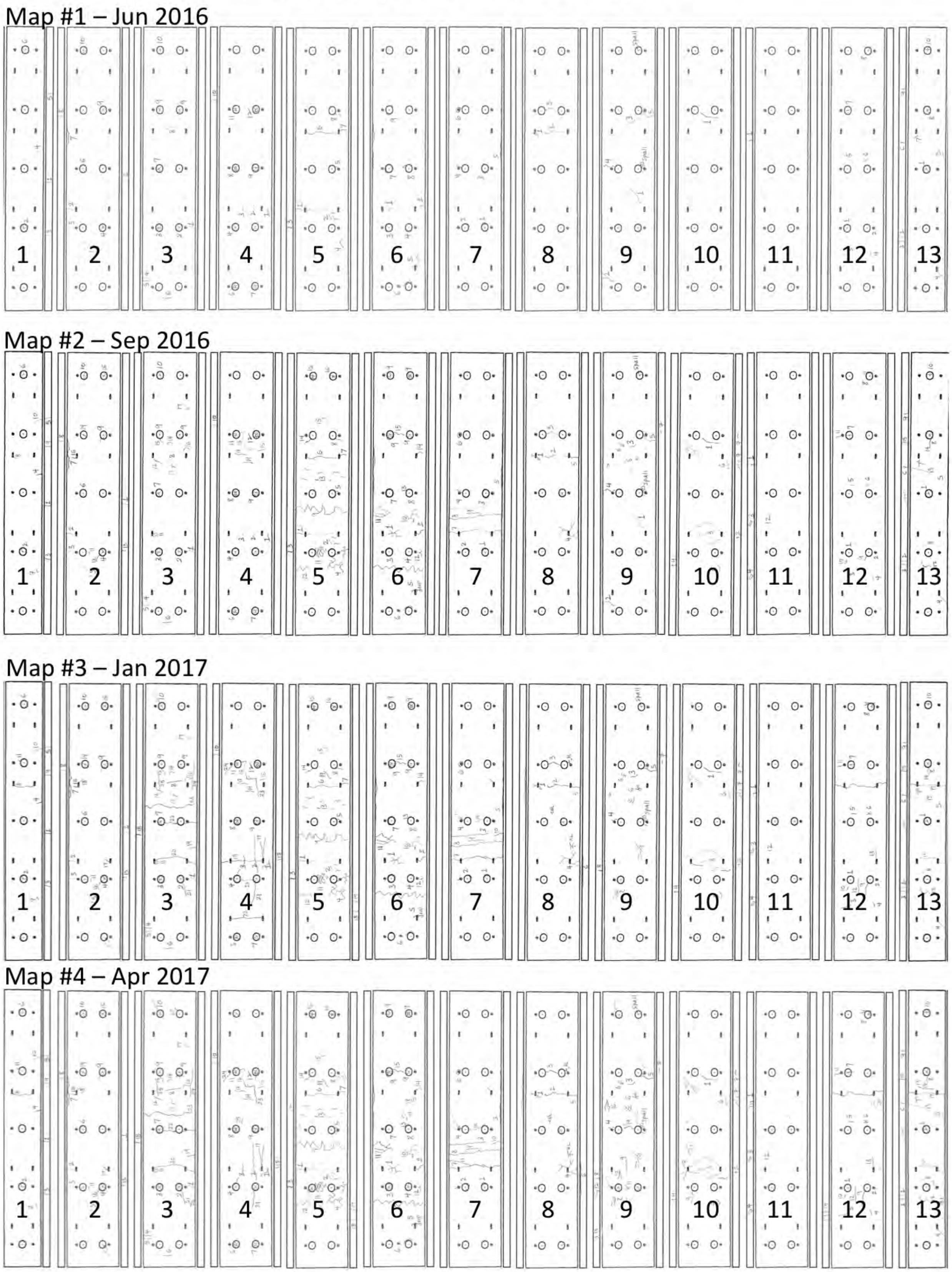
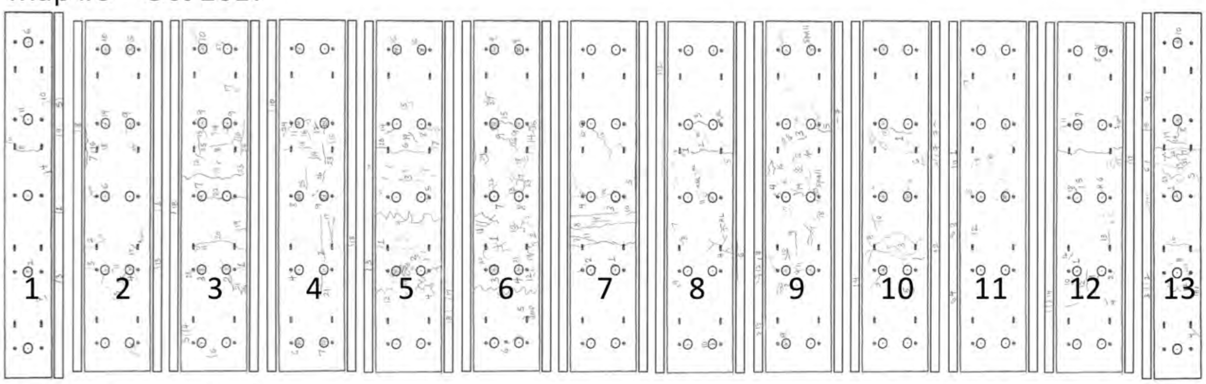


Fig. 2-17. Crack Maps for Little River Westbound Span 3

Map #5 – Jul 2017

Mapping #5 was not performed

Map #6 – Oct 2017



Map #7 – Jan 2018

Mapping #7 was not performed

Map #8 – Apr 2018

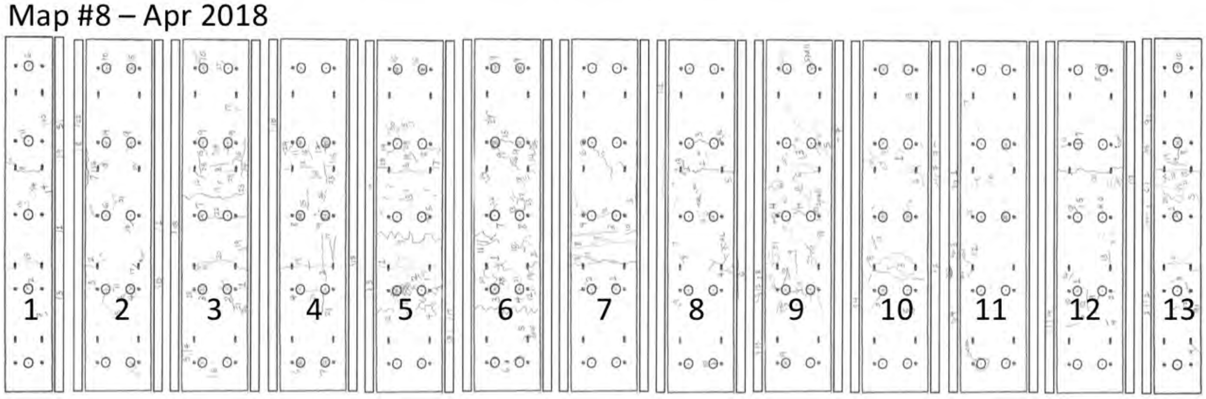


Fig. 2-18. Crack Maps for Little River Westbound Span 3 (cont'd)

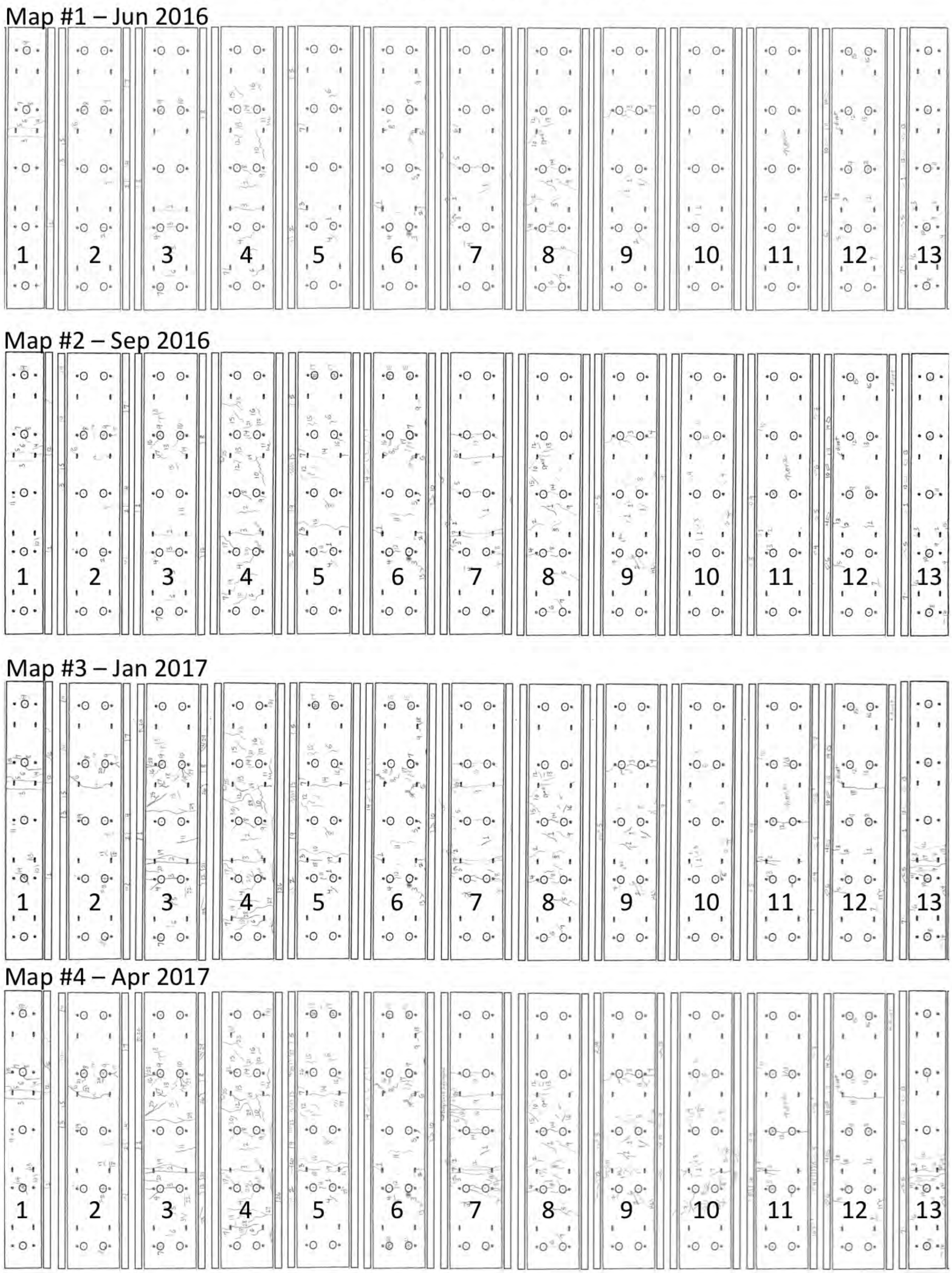
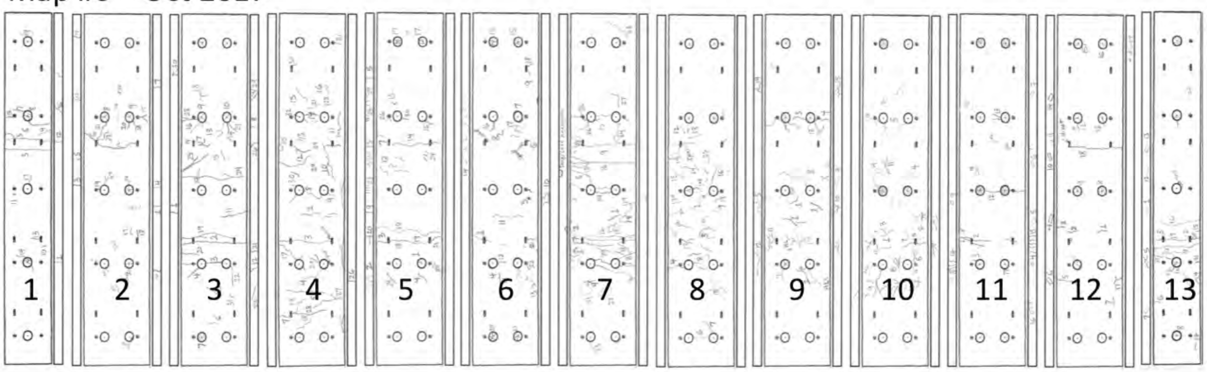


Fig. 2-19. Crack Maps for Little River Westbound Span 4

Map #5 – Jul 2017

Mapping #5 was
not performed

Map #6 – Oct 2017



Map #7 – Jan 2018

Mapping #7 was
not performed

Map #8 – Apr 2018

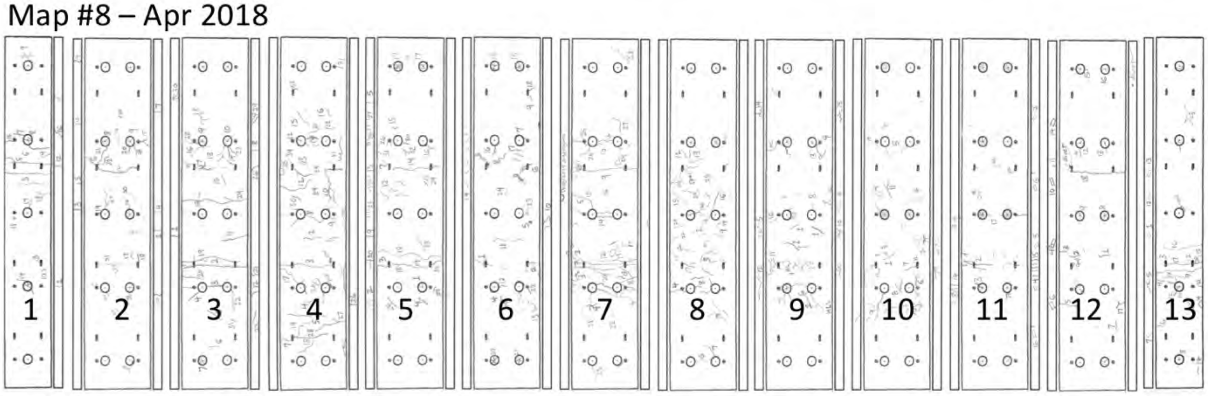


Fig. 2-20. Crack Maps for Little River Westbound Span 4 (cont'd)

CHAPTER 3
ANALYSIS OF CRACK MAPS

3.1 Number of Recorded Cracks in Each Span

The number of cracks recorded for each panel was tabulated in a spreadsheet. This was done for each crack width: hairline, 0.002, 0.004, 0.006, 0.008, 0.010, 0.012, and greater than 0.012 in. Subtotals were calculated for each of the ten (10) spans, for each mapping (#1 – #8) (see **Tables 3-1 and 3-2**). The subtotals include cracks in both the panels and joints. Some of the cracks in the joints could be counted twice, since the cracks were sometimes documented on two crack mapping sheets – one sheet for each of the two panels adjacent to the joint. There are very few instances where the joint cracks were counted twice.

The subtotals are shown in radar-type plots in **Figures 3-1 to 3-10**. As shown in the figures, all spans had an increase in the number of recorded cracks over time, from mapping #1 through mapping #8. However, there are clear differences in cracking between the eastbound and westbound bridges. The differences are summarized as follows:

- The radar plots indicate more considerable cracking in the *eastbound* bridges, with the plots skewed to the 0.002, 0.004, and 0.006 in. widths.
- For the *westbound* bridges, the radar plots are skewed mostly to the 0.002 in. width.
- A typical *eastbound* bridge span has around 80 – 140 cracks that are 0.002 in. wide, 100 – 140 cracks that are 0.004 in. wide, and 60 cracks that are 0.006 in. wide.
- A typical *westbound* bridge span has around 100 – 120 cracks that are 0.002 in. wide, 40 – 60 cracks that are 0.004 in. wide, and 20 cracks that are 0.006 in. wide.
- Westbound spans had fewer and narrower cracks than eastbound spans.

Table 3-1. Number of Cracks Documented on Each Span, for Each Mapping Performed (Eastbound Bridges)

Crack Map #	EB or WB	Bridge Name	Span #	Map # and Date	Number of Cracks Documented: Hairline	0.002	0.004	0.006	0.008	0.010	0.012	> 0.012	Total # Cracks
1	EB	HCEB	1	#1 Jun 2016	10	35	47	51	34	11	5	5	198
2	EB	HCEB	1	#2 Sep 2016	13	66	85	66	42	14	5	7	298
3	EB	HCEB	1	#3 Jan 2017	10	82	100	74	45	12	4	10	337
4	EB	HCEB	1	#4 Apr 2017	10	93	117	80	56	13	3	11	383
5	EB	HCEB	1	#5 Jul 2017	18	102	133	87	59	12	5	11	427
6	EB	HCEB	1	#6 Oct 2017	34	132	138	91	57	12	5	11	480
7	EB	HCEB	1	#7 Jan 2018	45	134	141	97	57	13	4	11	502
8	EB	HCEB	1	#8 Apr 2018	54	149	143	96	58	13	4	11	528
1	EB	LREB	1	#1 Jun 2016	15	22	59	42	18	3	2	4	165
2	EB	LREB	1	#2 Sep 2016	15	56	74	45	19	4	2	4	219
3	EB	LREB	1	#3 Jan 2017	15	71	74	49	19	5	3	4	240
4	EB	LREB	1	#4 Apr 2017	19	95	72	52	18	7	2	4	269
5	EB	LREB	1	#5 Jul 2017	32	110	76	52	19	6	2	4	301
6	EB	LREB	1	#6 Oct 2017	50	141	84	53	20	7	2	4	361
7	EB	LREB	1	#7 Jan 2018	65	142	87	55	18	7	2	4	380
8	EB	LREB	1	#8 Apr 2018	78	146	85	55	17	7	2	4	394
1	EB	LREB	2	#1 Jun 2016	7	22	64	54	17	2	3	0	169
2	EB	LREB	2	#2 Sep 2016	7	50	89	63	19	4	3	0	235
3	EB	LREB	2	#3 Jan 2017	6	67	98	57	32	5	3	0	268
4	EB	LREB	2	#4 Apr 2017	12	88	103	57	36	6	5	0	307
5	EB	LREB	2	#5 Jul 2017	27	100	102	57	35	7	5	0	333
6	EB	LREB	2	#6 Oct 2017	54	133	101	54	45	9	6	0	402
7	EB	LREB	2	#7 Jan 2018	67	148	95	55	48	8	5	1	427
8	EB	LREB	2	#8 Apr 2018	76	159	94	52	50	8	6	1	446
1	EB	LREB	3	#1 Jun 2016	9	25	53	38	31	10	4	3	173
2	EB	LREB	3	#2 Sep 2016	9	51	76	54	33	11	4	3	241
3	EB	LREB	3	#3 Jan 2017	9	63	76	60	38	10	5	3	264
4	EB	LREB	3	#4 Apr 2017	15	71	93	61	43	11	5	3	302
5	EB	LREB	3	#5 Jul 2017	20	82	102	61	42	11	5	3	326
6	EB	LREB	3	#6 Oct 2017	32	98	108	67	43	11	6	3	368
7	EB	LREB	3	#7 Jan 2018	42	111	117	71	44	12	6	3	406
8	EB	LREB	3	#8 Apr 2018	53	116	116	72	41	10	4	3	415
1	EB	LREB	4	#1 Jun 2016	13	36	52	40	26	9	6	3	185
2	EB	LREB	4	#2 Sep 2016	15	49	70	50	29	11	6	3	233
3	EB	LREB	4	#3 Jan 2017	12	61	82	49	34	13	6	3	260
4	EB	LREB	4	#4 Apr 2017	21	81	90	49	38	18	8	3	308
5	EB	LREB	4	#5 Jul 2017	37	95	103	53	38	17	6	3	352
6	EB	LREB	4	#6 Oct 2017	51	111	107	51	41	17	6	3	387
7	EB	LREB	4	#7 Jan 2018	66	118	112	53	43	18	6	3	419
8	EB	LREB	4	#8 Apr 2018	79	128	105	54	40	18	6	3	433

Table 3-2. Number of Cracks Documented on Each Span, for Each Mapping Performed (Westbound Bridges)

Crack Map #	EB or WB	Bridge Name	Span #	Map # and Date	Number of Cracks Documented Hairline	0.002	0.004	0.006	0.008	0.010	0.012	> 0.012	Total # Cracks
1	WB	HCWB	1	#1 Jun 2016	3	30	43	29	28	5	2	18	158
2	WB	HCWB	1	#2 Sep 2016	6	58	61	33	31	7	3	20	219
3	WB	HCWB	1	#3 Jan 2017	7	77	73	35	30	7	3	20	252
4	WB	HCWB	1	#4 Apr 2017	9	89	84	34	32	13	4	20	285
5	WB	HCWB	1	#5 not done									
6	WB	HCWB	1	#6 Oct 2017	16	97	88	41	33	13	4	18	310
7	WB	HCWB	1	#7 not done									
8	WB	HCWB	1	#8 Apr 2018	17	100	89	40	31	11	4	19	311
1	WB	LRWB	1	#1 Jun 2016	17	42	15	17	18	1	1	2	113
2	WB	LRWB	1	#2 Sep 2016	18	76	25	21	25	3	1	4	173
3	WB	LRWB	1	#3 Jan 2017	20	95	29	20	25	3	1	4	197
4	WB	LRWB	1	#4 Apr 2017	22	98	40	25	26	4	1	4	220
5	WB	LRWB	1	#5 not done									
6	WB	LRWB	1	#6 Oct 2017	45	116	41	28	25	4	1	5	265
7	WB	LRWB	1	#7 not done									
8	WB	LRWB	1	#8 Apr 2018	61	117	43	29	24	4	1	5	284
1	WB	LRWB	2	#1 Jun 2016	21	35	20	17	11	0	1	1	106
2	WB	LRWB	2	#2 Sep 2016	32	47	37	21	18	1	2	2	160
3	WB	LRWB	2	#3 Jan 2017	33	59	53	22	19	1	2	2	191
4	WB	LRWB	2	#4 Apr 2017	37	81	56	23	19	2	2	2	222
5	WB	LRWB	2	#5 not done									
6	WB	LRWB	2	#6 Oct 2017	47	114	61	18	17	2	2	2	263
7	WB	LRWB	2	#7 not done									
8	WB	LRWB	2	#8 Apr 2018	55	123	60	18	17	2	2	2	279
1	WB	LRWB	3	#1 Jun 2016	25	26	16	11	5	3	2	1	89
2	WB	LRWB	3	#2 Sep 2016	40	50	31	13	8	3	2	1	148
3	WB	LRWB	3	#3 Jan 2017	34	81	31	18	7	3	2	1	177
4	WB	LRWB	3	#4 Apr 2017	47	89	34	20	8	3	3	0	204
5	WB	LRWB	3	#5 not done									
6	WB	LRWB	3	#6 Oct 2017	64	105	38	20	8	3	3	0	241
7	WB	LRWB	3	#7 not done									
8	WB	LRWB	3	#8 Apr 2018	72	111	39	20	8	3	3	0	256
1	WB	LRWB	4	#1 Jun 2016	18	34	22	15	15	2	1	8	115
2	WB	LRWB	4	#2 Sep 2016	35	59	40	21	18	3	1	8	185
3	WB	LRWB	4	#3 Jan 2017	37	87	48	22	19	3	1	8	225
4	WB	LRWB	4	#4 Apr 2017	45	113	49	24	17	8	1	8	265
5	WB	LRWB	4	#5 not done									
6	WB	LRWB	4	#6 Oct 2017	71	115	54	25	18	10	1	8	302
7	WB	LRWB	4	#7 not done									
8	WB	LRWB	4	#8 Apr 2018	78	115	55	25	18	10	1	8	310

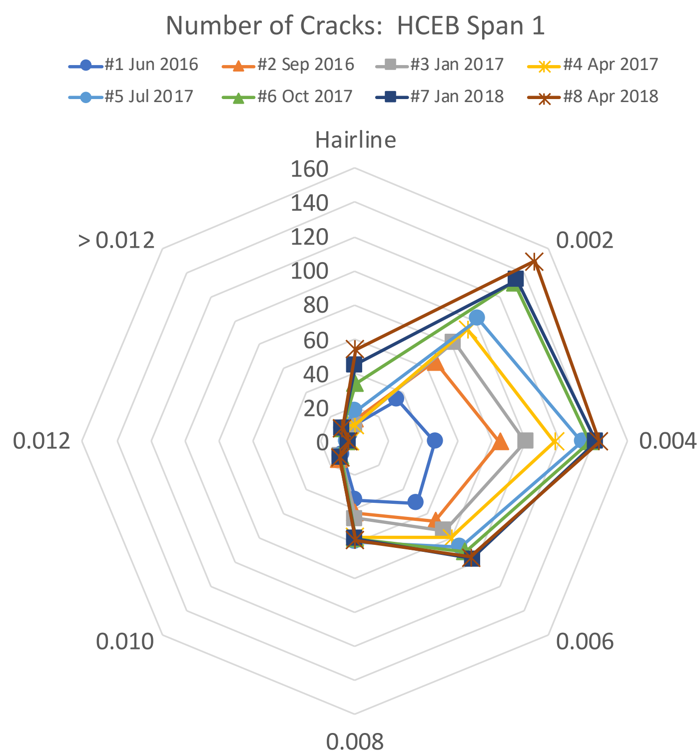


Fig. 3-1. Number of Cracks for Each Width: Hurricane Creek Eastbound Bridge

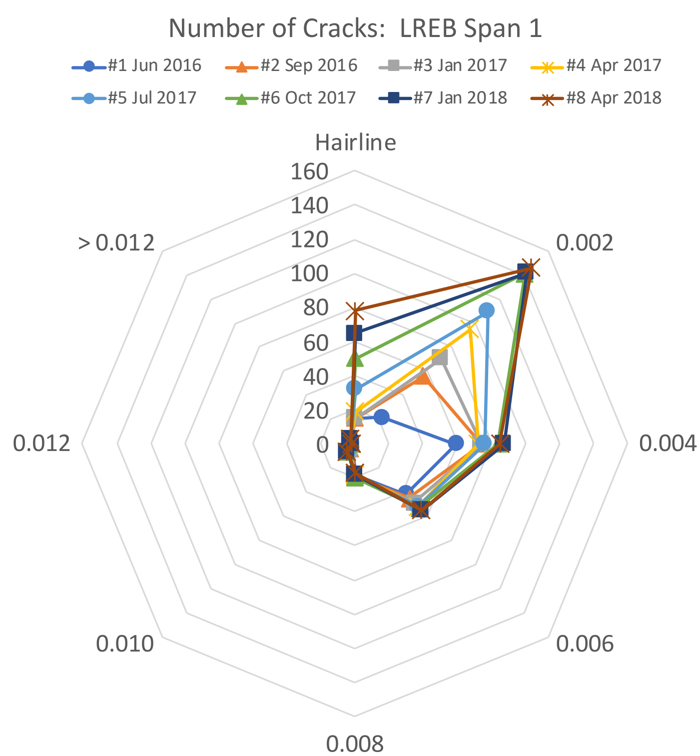


Fig. 3-2. Number of Cracks for Each Width: Little River Eastbound Span 1

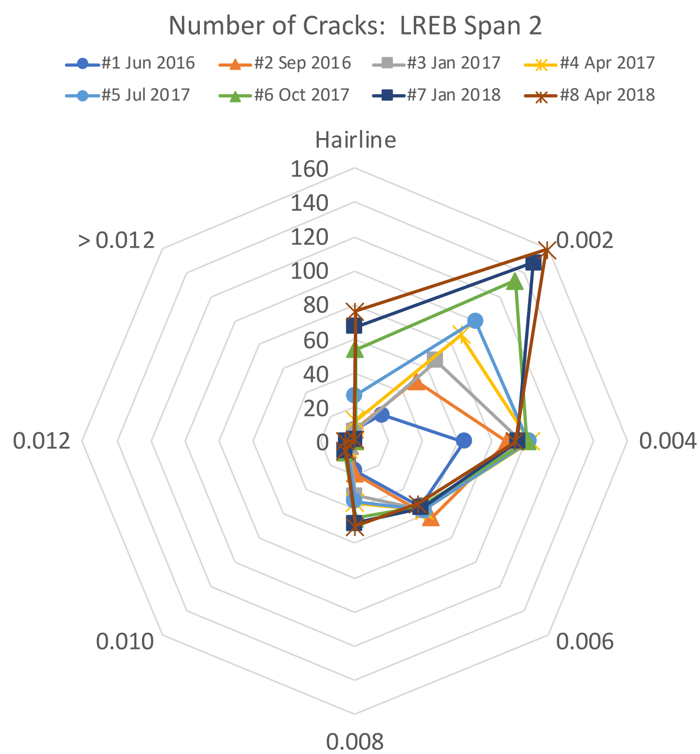


Fig. 3-3. Number of Cracks for Each Width: Little River Eastbound Span 2

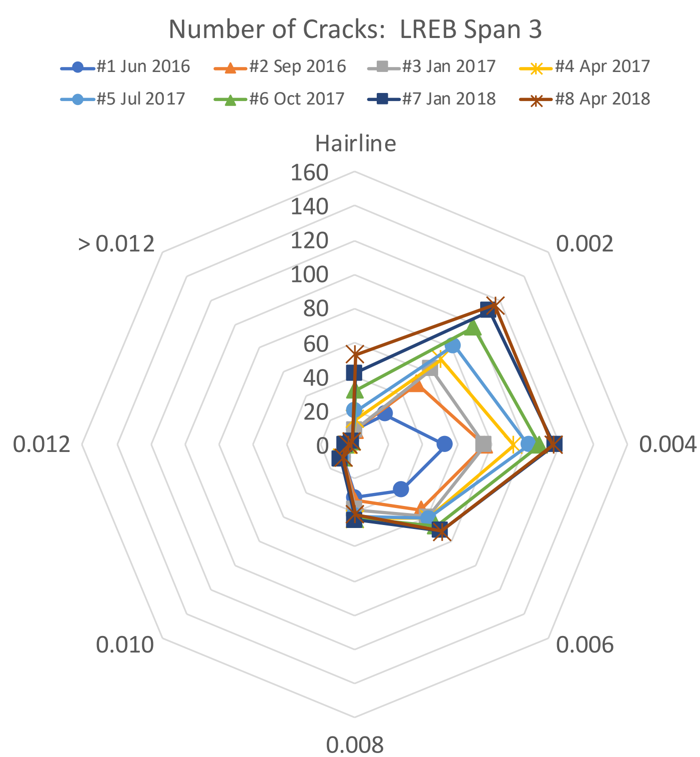


Fig. 3-4. Number of Cracks for Each Width: Little River Eastbound Span 3

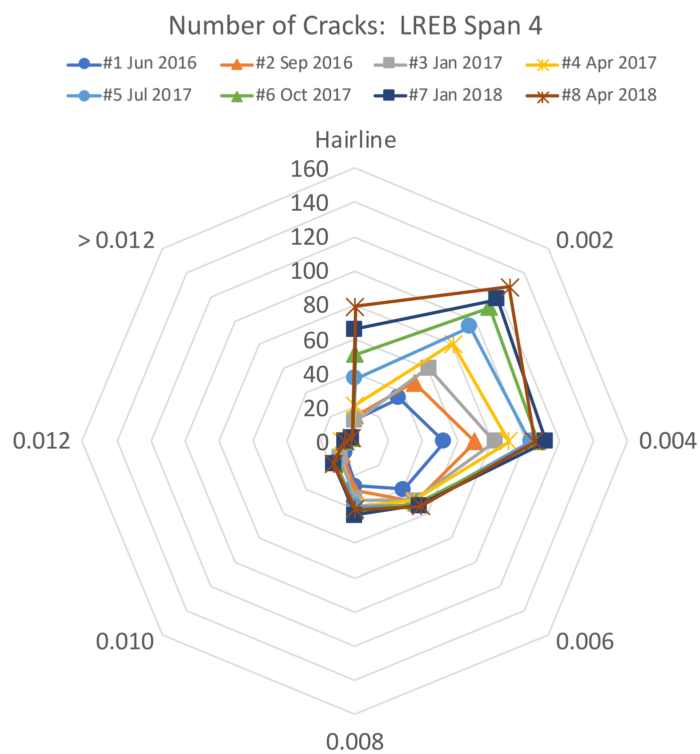


Fig. 3-5. Number of Cracks for Each Width: Little River Eastbound Span 4

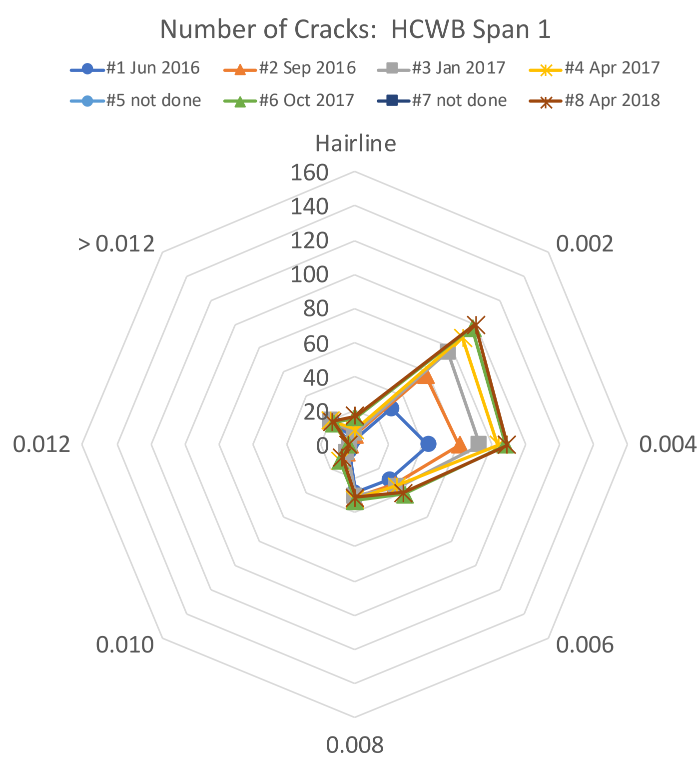


Fig. 3-6. Number of Cracks for Each Width: Hurricane Creek Westbound Bridge

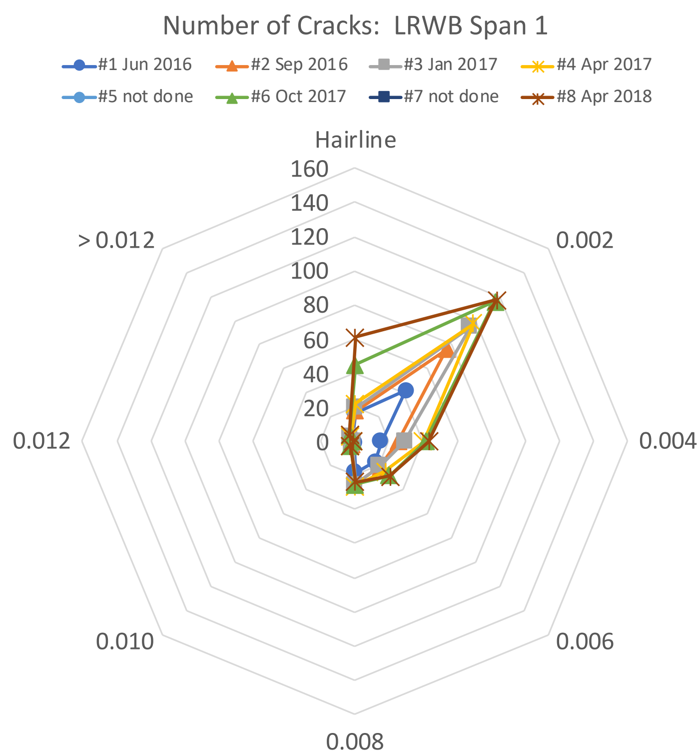


Fig. 3-7. Number of Cracks for Each Width: Little River Westbound Span 1

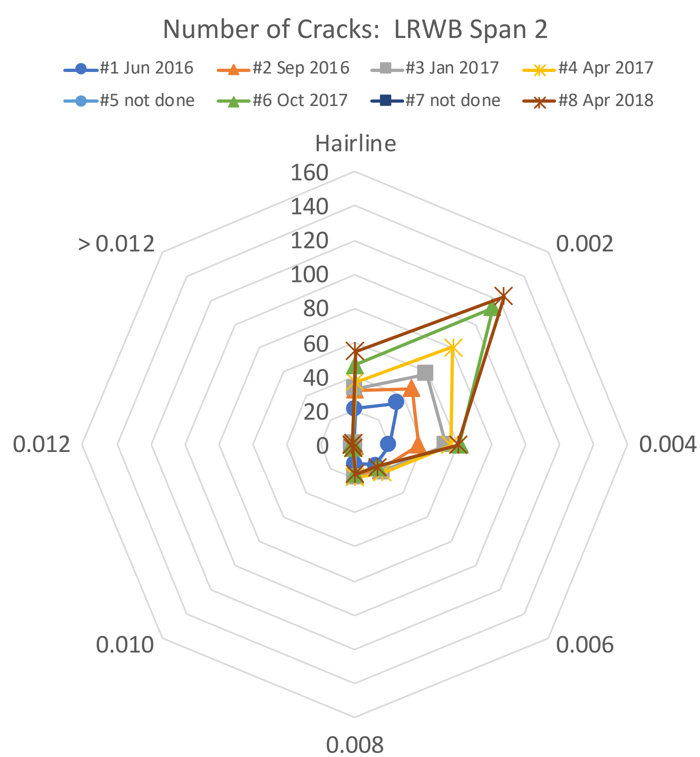


Fig. 3-8. Number of Cracks for Each Width: Little River Westbound Span 2

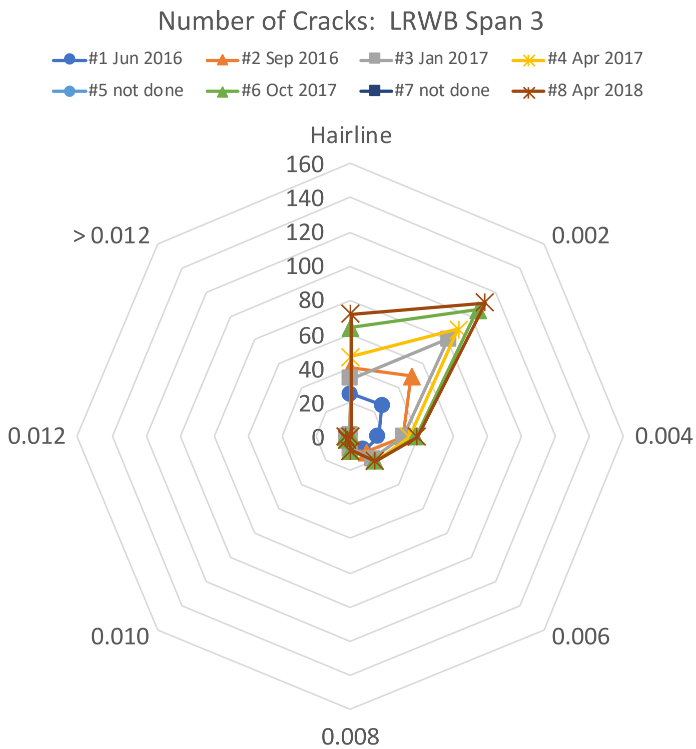


Fig. 3-9. Number of Cracks for Each Width: Little River Westbound Span 3

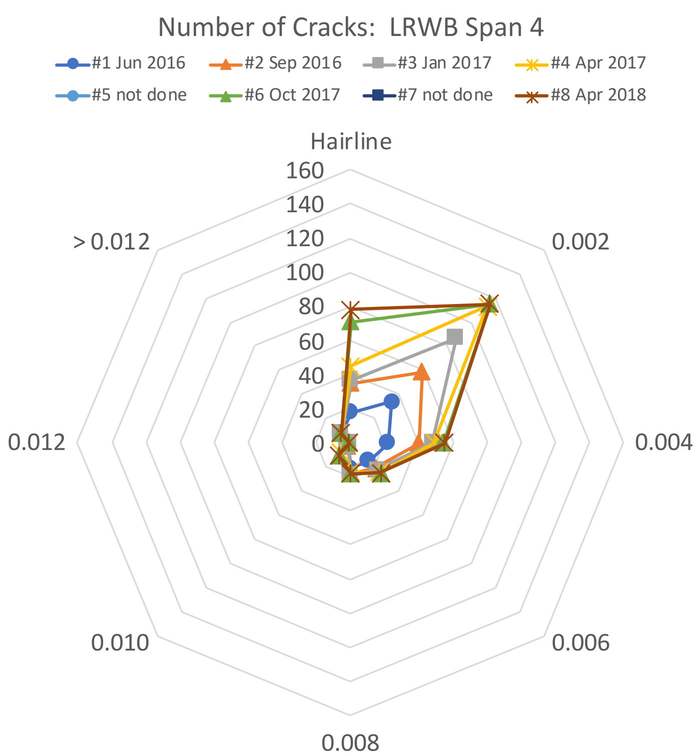


Fig. 3-10. Number of Cracks for Each Width: Little River Westbound Span 4

3.2 Extent of Cracking

The radar plots in Figures 3-1 to 3-10 depict the *number* of cracks, but they do NOT include any tabulation or analysis of the crack *lengths*. If lengths were accounted for, the westbound bridges would have even less significant cracking than the eastbound bridges. Based on a qualitative (visual) analysis of the crack maps, many of the recorded cracks on the westbound bridges were very short in length, whereas the eastbound bridge cracks were much longer and most often extended the full width of the panel. Rough observations are as follows:

- Most *eastbound* panels had **over a dozen cracks that extended the full width** of the panel.
- Most *westbound* panels had **only 0 to 2 full-length cracks**.

An abridged quantitative analysis of the eastbound-westbound difference was performed by counting the number of shear pockets that had cracks along them. The results are shown in **Figures 3-11 and 3-12** for the eastbound and westbound bridges, respectively. In summary:

- Each eastbound span had 25 or 29 cracks along the shear pockets at the time of the first crack mapping. The number grew to 39 or 41 by the fourth mapping and to 43 – 46 by the eighth mapping.
- Each westbound span initially had 0 to 2 cracked shear pockets, had 1 to 11 at the fourth mapping, and had 4 – 14 by the eighth mapping – **far fewer than for the eastbound spans**.

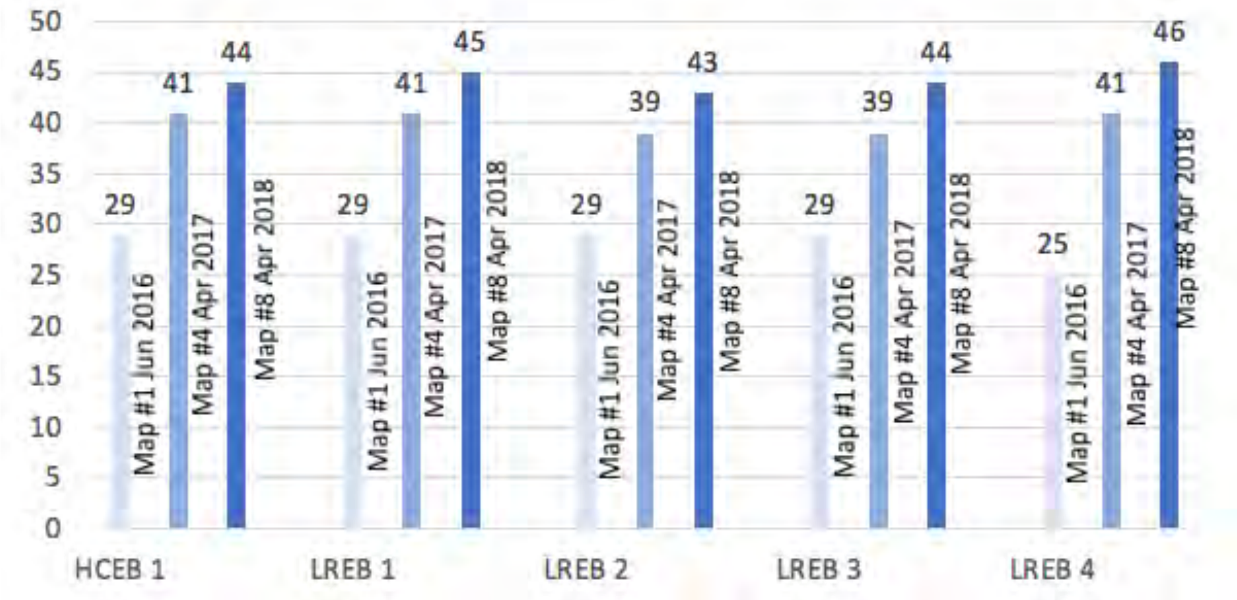


Fig. 3-11. Number of Cracked Shear Pockets on Eastbound Spans

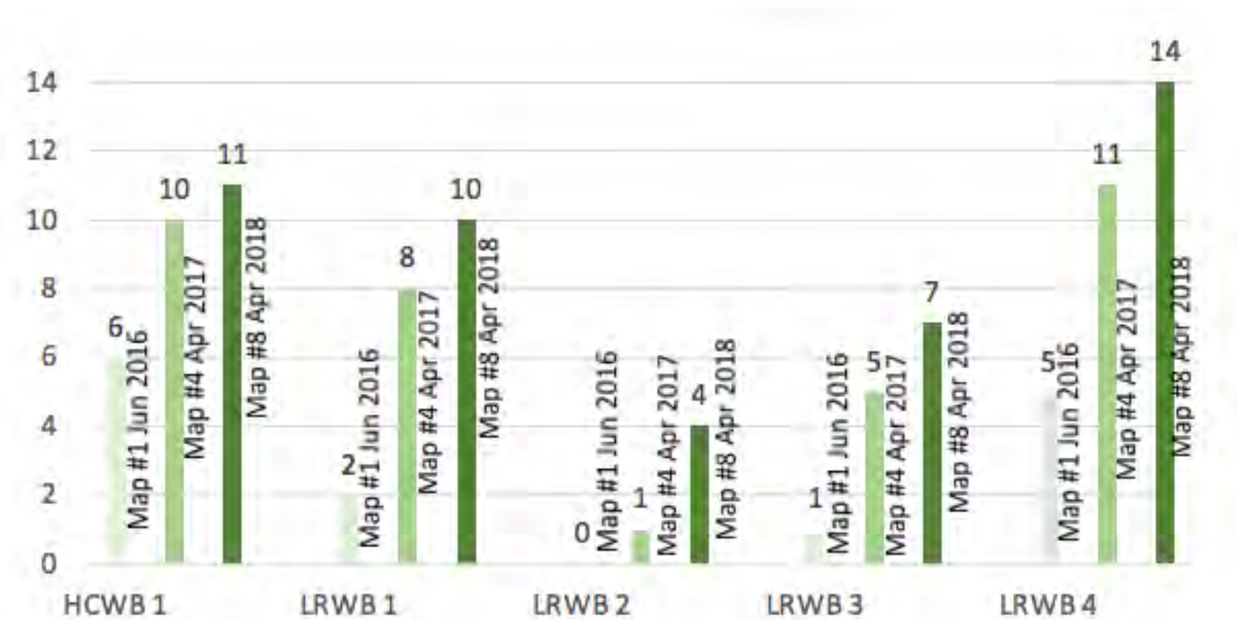


Fig. 3-12. Number of Cracked Shear Pockets on Westbound Spans

CHAPTER 4
CONCLUSIONS

4.1 Disposition of Cracked Panels

Based on the radar plots in Figures 3-1 to 3-10, Hurricane Creek EB Bridge had the most cracking in the panels. A crack disposition analysis was performed on the April 2017 (fourth) mapping. The analysis was done per FDOT Specification 400-21 Disposition of Cracked Concrete. The bridge superstructure is classified in the Environment Category of Slightly Aggressive, and it is more than 12 feet above mean high water (MHW). Panels 1 through 9 were analyzed, and a summary is provided in **Table 4-1**. Cracks in the adjacent joints were not included in the analysis; only cracks in the panels were included.

The disposition for these panels was No Treatment Required, except for Panels 3 and 6. More details about these panels are as follows:

- Panel 3 had Cracking Significance in the Severe category. Five (5) cracks on this panel had widths greater than 0.020 in. All other cracks were less than or equal to 0.012 in. wide. A separate analysis was done on the panel, as if those cracks were not present or were repaired, and the crack disposition is that the panel would not require treatment. Repair of these wider cracks should be considered.
- Panel 6 has Cracking Significance in the Moderate category. The largest crack on this panel was 0.026-in.-wide by 10-ft long. All other cracks were less than or equal to 0.008 in. wide. A separate analysis was done on the panel, as if that crack were not present or were repaired, and the crack disposition is that the panel would not require treatment. Repair of this wider crack should be considered.

In summary, FDOT should further investigate the cracking on all four (4) bridges and consider repairing the wider cracks.

Table 4-1. Disposition of Panels with Most Cracking (HCEB Bridge Panels 1-9), Based on Crack Mapping #4 in April 2017

Panel #	Crack Width Range in.	Total Cracked Surface Area (TCSA) in^2	Total Surface Area (TSA) in^2	(TCSA)/(TSA) x 100 %	Cracking Significance	Crack Disposition*	Comment
1	0.002-0.006	2.28	28800	0.0079	Occasional	NT	
2	0.002-0.008	4.55	46080	0.0099	Occasional	NT	
3	0.002-0.028	13.53	46080	0.0294	Severe	Replace	
3**	0.002-0.012	6.80	46080	0.0148	Occasional	NT	5 cracks had widths greater than 0.020 in. All other cracks were ≤ 0.012 in. wide. The values in this row neglect the 5 cracks, as if they were not present or were repaired.
4	0.002-0.008	6.43	46080	0.0140	Occasional	NT	
5	0.002-0.008	5.36	46080	0.0116	Occasional	NT	
6	0.002-0.026	11.99	46080	0.0260	Moderate	Investigate to Determine Appropriate Repair	
6***	0.002-0.008	8.87	46080	0.0192	Moderate	NT	Panel had one 0.026-in. wide crack, 10 ft long. All other cracks were ≤ 0.008 in. wide. The values in this row neglect the 0.026-in. wide crack, as if it was not present or were repaired.
7	0.002-0.014	5.76	46080	0.0125	Occasional	NT	
8	0.002-0.008	6.86	46080	0.0149	Occasional	NT	
9	0.002-0.012	7.38	46080	0.0160	Occasional	NT	
Analysis of Panels 1-9 combined:							
1 - 9	0.002-0.028	79.81	489600	0.0163	Occasional	Investigate to Determine Appropriate Repair	

* Bridge is more than 12 feet AMHW, and superstructure is in slightly aggressive environment.
NT = No Treatment Required
** This row is an alternate analysis of Panel 3, where the 5 largest cracks were neglected in the calculations.
*** This row is an alternate analysis of Panel 6, where the largest crack was neglected in the calculations.

4.2 FDOT Inspections and Recommendation

FDOT has performed routine inspections of all four (4) bridges. To date, inspections have been performed around February/March 2016, February 2017, and February 2018. Overall NBI Ratings from the February 2018 inspection reports are provided in **Table 4-2**. The condition of the deck, superstructure, and substructure elements were either Good or Very Good. Sufficiency ratings were all above 94, and health indices were all above 95.

Table 4-2. Overall NBI Ratings for Hurricane Creek and Little River Bridges, February 2018 Inspection

	Little River WB #500151	Little River EB #500152	Hurricane Creek WB #500153	Hurricane Creek EB #500154
Deck	7 Good	7 Good	7 Good	7 Good
Superstructure	7 Good	7 Good	8 Very Good	7 Good
Substructure	8 Very Good	8 Very Good	8 Very Good	7 Good
Performance Rating	2 Good	2 Good	2 Good	2 Good
Sufficiency Rating	99.7	94.7	99.7	99.7
Health Index	96.42	95.21	96.9	96.04

The inspection reports contain notes on the condition of the deck panels, closure pours, joints, approach slabs, girders, bridge railings, and substructure. Specifically, the deck/slab inspections noted:

- The deck panels and closure pours throughout the structure exhibit random cracks up to 0.016 in., 0.030 in., 0.016 in., and 0.016 in. wide for Bridges Little River WB, Little River EB, Hurricane Creek WB, and Hurricane Creek EB, respectively.
- The grout in the shear pockets exhibit random cracks up to 0.020 in. wide for all bridges except Hurricane Creek WB, for which the report noted 0.040 in. wide cracks. There is minor spalling in the shear pockets and no exposed steel.
- The grinding of the deck surface has left an uneven and irregular riding surface on both westbound bridges.

For all four (4) bridges, the inspector recommended sealing the deck surface with T-70 Methacrylate.

Precast Element Evaluation for the US 90 Bridges over Little River and Hurricane Creek

Contract Number BDV30-307-01
FSU Project ID 032819

Submitted to:

Florida Department of Transportation

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June 2019
REPORT 5: LOAD TEST REPORT

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CHAPTER 1
OVERVIEW

1.1 Introduction

This report provides details and analyses of load tests that were performed on Hurricane Creek and Little River bridges, both eastbound (EB) and westbound (WB). Design details, cross sections, and elevations are provided in a separate report, Report 1, Introduction to Construction Project. In another report, Report 2, Fabrication of Precast Elements, details are provided on the production of the elements including forms and casting yard activities. Additionally, construction activities are detailed in Report 3, Construction of Precast Elements. Some details from these reports are important to fully understand this load test report, but repetitiveness is minimized as much as possible. The reader is encouraged to read these previous reports to gain further insight on the uniqueness of the bridges studied.

Because the bridges were described in detail in other reports, a brief description of the bridges is as follows. The bridges are straight and have a width of 43’-1”. Each bridge carries two (2) striped lanes, but the width is such that up to three (3) lanes were considered in design. Each bridge consists of five (5) 45" Florida-I Beams spaced at 9 ft on center, cast-in-place barriers, and precast bridge deck panels most of which are 8.5 in. thick. The 8.5-in. thickness includes a 0.5-inch-thick sacrificial wearing surface that was considered in the dead load calculations but not in section property calculations for design.

The Hurricane Creek and Little River EB bridges have reinforced (non-prestressed) precast concrete deck panels, and the Hurricane Creek and Little River WB bridges have prestressed (pretensioned) precast concrete deck panels. Each of the three (3) intermediate bents of Little River EB and WB bridges consist of two cast-in-place columns that support a reinforced precast concrete bent cap – a unique feature of the bridges.

The Florida-I beams (FIBs) used in this project were modified from FDOT standards to accommodate for the precast panels and transverse joints connecting the panels.

Composite action is facilitated by grouted shear pockets with reinforcement that connects the panels to the girders, as well as transverse cast-in-place concrete joints between the panels. The shear connectors were spaced at 3'-0", which is larger than the spacing that is specified in FDOT's FIB design standards. The spans associated with the load tests only involve the FIBs with a Type 1 strand pattern (see Report 2 discussed earlier).

The goal of the research was to evaluate the structural behavior of the newly-developed bridge system that includes precast concrete deck panels and precast concrete bent caps. Through load testing, the objectives were to:

- Compare the behavior of the two different precast concrete panel types (non-prestressed and prestressed)
- Evaluate live load distribution to the girders. For design, it is necessary to know how well the live load distribution is predicted by the *AASHTO LRFD Bridge Design Specification*'s distribution factors for the interior and exterior girders (*AASHTO* 2017).
- Evaluate the composite behavior of the superstructure (i.e., the girder and deck panel system). It is important to know whether the design assumption of composite action between the deck panels and the girders is accurate.
- Evaluate the transverse behavior of the panel, as it spans across the girders
- Evaluate the transverse behavior of the panel joints
- Evaluate load sharing between the panels and adjacent joints
- Evaluate the long-term behavior of the system, by comparing the pre-service and in-service load tests on the bridges
- Evaluate the effect of the barriers on the live load distribution to the girders. This was a side study that was performed out of curiosity.
- Evaluate the behavior of the precast bent caps

1.2 Bridge Naming

In this report, the bridge names are sometimes shortened as follows:

HCEB	Hurricane Creek Eastbound Bridge
HCWB	Hurricane Creek Westbound Bridge
LREB	Little River Eastbound Bridge
LRWB	Little River Westbound Bridge

1.3 Background

1.3.1 Overview of Load Test Plan

This chapter describes the nine (9) load tests that were performed, including the load test procedures and instrumentation placed on the bridge. Load testing was performed on span 3 of each Little River bridge and on the single span of each Hurricane Creek bridge. The lengths of the tested spans for the Little River bridges were 110'-0", and the Hurricane Creek bridges were 110'-2". The beam lengths were 2" to 2.5" shorter than the span length. The panels that were instrumented were types panel A (non-prestressed) and panel D (prestressed), as described in previous reports. On the Little River bridges, Intermediate Bent 4 was instrumented – this bent supports spans 3 and 4.

Pre-Service Tests: Five (5) load tests were performed on the four (4) bridges before they were placed in service. With the exception of the first test on Hurricane Creek WB, the barriers had already been cast at the time of the load test. The order of the tests was as follows:

- Hurricane Creek Eastbound on December 19, 2014
- Little River Eastbound (Span 3) on March 4, 2015
- Hurricane Creek Westbound (before barriers) on September 1, 2015
- Hurricane Creek Westbound (after barriers) on November 19, 2015
- Little River Westbound (Span 3) on December 1, 2015

***In-Service Tests*:** An additional four (4) load tests were performed after the bridges had been in service for 1.5 – 2.5 years, on the following dates:

- Hurricane Creek Eastbound on April 20, 2017
- Little River Eastbound (Span 3) on April 20, 2017
- Hurricane Creek Westbound on June 29, 2017
- Little River Westbound (Span 3) on June 29, 2017

Note that while the WB bridges were being constructed, the EB bridges carried one lane of EB traffic and one lane of WB traffic. This means they were subjected to a higher traffic volume than normal. Dates of construction activities and initial service are provided in a previous report.

More details about the instrumentation, test procedure, and data analysis are provided in subsequent chapters. However, an overview is provided in the three paragraphs below.

The test plan was formulated by FDOT design engineers, with input from the researchers and from engineers at the FDOT Structures Research Center (SRC). The bridges were instrumented with strain and deflection gauges to measure the deformational response of each bridge (girders, deck panels, and precast bent caps) during the load tests. Deflection gages were placed at mid span and at approximately third points along the span. Strain gauges were placed in the longitudinal direction on the girders, deck panels, and barriers. Additional strain gages were placed transverse to the bridge, on the underside of the panels, to evaluate the interaction between panels and the joints. One bent (cap and columns) on each Little River bridge (EB and WB) was instrumented with strain and deflection gauges.

Instrumentation for each bridge was installed over a two- or three-day period, followed by testing on another day. The SRC assisted with instrumentation and data acquisition. The FDOT Office of Maintenance provided equipment (snooper truck) to access underneath the bridge, and SRC provided the test truck and weights. The load test trucks were positioned to produce the maximum load effects (longitudinal or transverse bending) at specific longitudinal positions along the span. Multiple load positions were employed for each bridge test, and generally the bridge response was measured under three different loads (referred to hereinafter as "block cases") for each load position. The strain and deflection responses were checked for linearity as the load was increased.

The test data was then analyzed for the following: the composite behavior between the deck panels and the girders; comparison of the theoretical distribution factors to measured ones; the difference in behavior between prestressed deck panels and non-prestressed deck panels; compatibility of strains in the joints and panels; the barrier effects on the bridge response including live load distribution; and comparison of the pre-service and in-service responses of the bridges.

1.3.2 Distribution Factors

In the design of a slab-on-girder bridge such as the Hurricane Creek and Little River bridges, distribution factors are used to determine the amount of live load that will be carried by the interior and exterior girders of a bridge. The distribution factors (DFs) are calculated using equations from the *AASHTO LRFD Bridge Design Specifications* (*AASHTO* 2017), as long as each parameter is within the range of applicability. Because these equations do not account for stiffening elements (e.g., traffic railings, diaphragms, etc.), the calculated values are somewhat conservative (Eamon 2002 and Eamon 2004).

Because the bridges employ a new type of precast panel, one of the purposes of this research was to determine how well AASHTO predicts DFs for this bridge type. Therefore, the AASHTO theoretical DFs were calculated and compared to those calculated from the load test data. These analyses on distribution factors are presented in later sections. For now, the AASHTO DF equations are provided below.

For the interior girder of a prestressed concrete girder bridge, AASHTO DFs are calculated using Eq. 1 for one (1) design lane loaded and Eq. 2 for two (2) or more lanes loaded.

$$DF_{int}^{1+lane} = 0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L_{bb}}\right)^{0.3}\left(\frac{K_g}{12.0L_{bb}t_s^3}\right)^{0.1} \qquad \text{Eq. 1}$$

$$DF_{int}^{2+lanes} = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L_{bb}}\right)^{0.2}\left(\frac{K_g}{12.0L_{bb}t_s^3}\right)^{0.1} \qquad \text{Eq. 2}$$

where:

S = girder spacing, ft

L_{bb} = span length (center of bearing pads), ft

t_s = slab thickness, in.

K_g = longitudinal stiffness parameter, in^4

The stiffness parameter factor K_g is calculated using Eq. 3:

$$K_g = n\left(I_{nc} + A_{nc}e_g^2\right) \qquad \text{Eq. 3}$$

where:

n = modular ratio

I_{nc} = moment of inertia of non-composite girder, in^4

A_{nc} = area of non-composite girder, in^2

e_g = distance between centers of gravity of girder and deck, in.

These AASHTO equations inherently include a Multiple Presence Factor (MPF), but the MPF can be excluded by dividing the calculated DF by the MPF. The purpose of the MPF is to account for the probability that the trucks would be fully loaded when next to each other. As the number of lanes increases, this probability decreases.

To design an exterior girder for one lane loaded, the lever rule is used to determine the DF. The first wheel line is placed 2 ft from the face of the barrier. The reaction on the exterior girder is then calculated by summing the moments about the first interior girder (assuming a hinge is there). The lever rule does not account for the multiple presence factor, so the factor must be applied manually.

If two or more lanes are loaded, then the DF for the exterior girder is calculated using Eq. 4.

$$DF_{ext}^{2+lanes} = e * DF_{int}^{2+lanes} \qquad \text{Eq. 4}$$

where:

$e \quad = 0.77 + \frac{d_e}{9.1}$

1.3.3 Material Properties

The material properties for the bridge elements were presented and discussed in a previous report.

For the load test analyses in this report, the modulus of elasticity of the concrete, in ksi, was calculated using Eq. 5, from the *AASHTO LRFD Bridge Design Specifications* (*AASHTO* 2017).

$$E_c = 120{,}000 K_1 w_c^2 (f_c')^{0.33} \qquad \text{Eq. 5}$$

where:

K_1 = correction factor for source aggregate

w_c = unit weight of concrete, kcf

f_c' = 28-day compressive strength of concrete, ksi

The unit weight of concrete for the purpose of calculating modulus of elasticity was 0.145 kcf. Also, as specified by the FDOT Structures Design Guidelines, the correction factor for source aggregate, K_1, is 1.0 (FDOT 2019).

Table 1-1 shows the materials properties of the girders in the tested spans.

For the panels, the concrete compressive strength used in the data analysis was taken as the average 28-day strength of all the panels in the span.

The cast-in-place barriers were assumed to have a 28-day concrete compressive strength of 4,500 psi, with a calculated modulus of elasticity of 4,145 ksi.

For the closure joints in between the deck panels, the average 28-day concrete compressive strength was 5,687 psi and calculated modulus of elasticity was 4,477 ksi.

Table 1-1. Florida-I Beam Material Properties

Bridge Name	Girder Location	Original Girder Label	28-day Concrete Strength (psi)	Calculated Concrete Modulus (ksi)
Hurricane Creek Eastbound	FIB 1	HC-45-1-2EB	11 840	5703
	FIB 2	HC-45-1-3EB	11 840	5703
	FIB 3	HC-45-1-4EB	12 480	5803
	FIB 4	HC-45-1-5EB	12 480	5803
	FIB 5	HC-45-1-1EB	11 060	5576
Hurricane Creek Westbound	FIB 1	HC-45-1-1WB	11 940	5719
	FIB 2	HC-45-1-2WB	9790	5356
	FIB 3	HC-45-1-3WB	12 290	5774
	FIB 4	HC-45-1-4WB	12 290	5774
	FIB 5	HC-45-1-5WB	11 060	5576
Little River Eastbound	FIB 1	LR-45-3-2EB	11 330	5621
	FIB 2	LR-45-3-1EB	12 440	5797
	FIB 3	LR-45-3-4EB	11 420	5636
	FIB 4	LR-45-3-5EB	11 420	5636
	FIB 5	LR-45-3-3EB	11 330	5621
Little River Westbound	FIB 1	LR-45-3-1WB	11 180	5596
	FIB 2	LR-45-3-2WB	12 460	5800
	FIB 3	LR-45-3-5WB	10 890	5548
	FIB 4	LR-45-3-4WB	10 890	5548
	FIB 5	LR-45-3-3WB	12 460	5800

CHAPTER 2

LOAD TEST PROCEDURES AND INSTRUMENTATION

2.1 Test Load and Procedure

2.1.1 Load Truck

The load was applied to the bridges using two (2) load test trucks provided by FDOT; one truck is shown in **Figure 2-1**. Note that the gage of the wheels is 6'-5", which is slightly more than the 6'-0" gage for the AASHTO HL93 truck. **Table 2-1** shows the axle weights of the load truck. For this research, three (3) different block configurations (block cases) were used: 12, 18, and 24. Each block weighed 2000 pounds. In general, all three (3) block cases were tested for each load position; load positions are described in the next section.

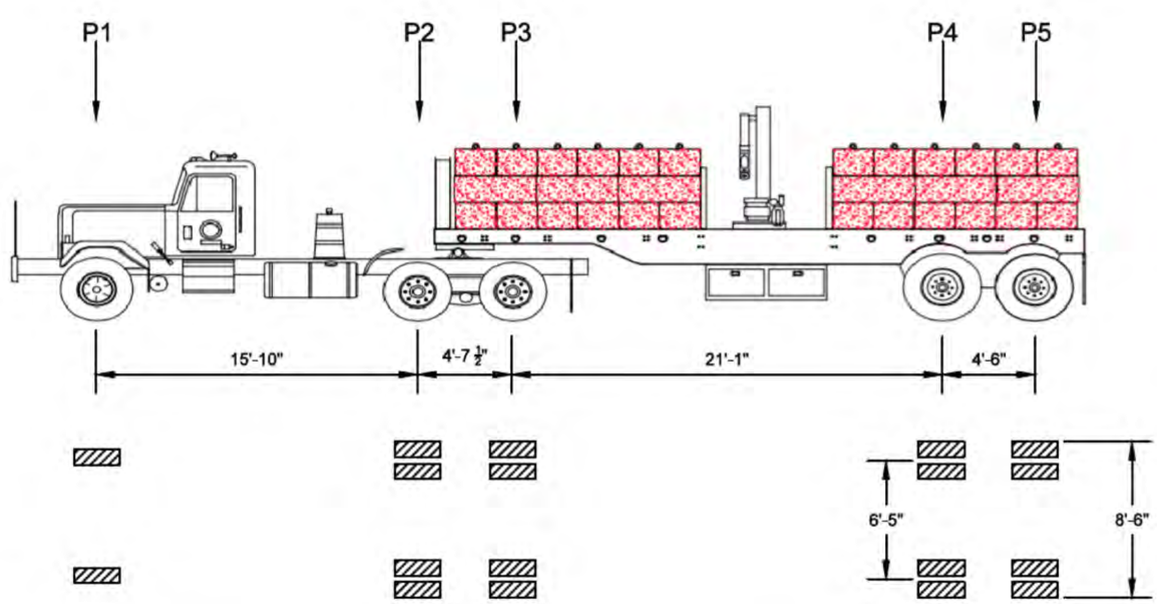


Fig. 2-1. FDOT Load Truck

When blocks were added to the truck, the resultant force location changed slightly because the load was concentrated more over the rear axles than the front. For simplicity in loading, the truck location relative to mid span was kept constant for all block cases, for a particular load position.

Table 2-1. FDOT Load Truck Axle Weights

No. Blocks	Front Axle	Front Tandem		Rear Tandem	
	P1 (kip)	P2 (kip)	P3 (kip)	P4 (kip)	P5 (kip)
0	10.60	9.59	9.59	9.59	9.59
6	10.20	11.55	11.55	12.24	12.24
12	10.20	14.02	14.02	15.67	15.67
18	10.68	16.49	16.49	19.10	19.10
24	10.87	19.32	19.32	22.63	22.63
30	10.51	21.24	21.24	25.81	25.81
36	10.65	23.99	23.99	29.24	29.24
42	11.53	26.17	26.17	32.82	32.82
48	11.41	28.42	28.42	36.27	36.27
54	11.59	31.05	31.05	39.53	39.53
60	11.21	33.89	33.89	43.28	43.28

2.1.2 Load Positions

Seven (7) load positions were used in this study and were labeled as follows: 1A, 1B, 2, 3A, 3B, 4, and 5. The load positions and their purpose, e.g. for maximizing a particular load effect, are described below. For load positions 1A, 1B, 3A, 3B, and 4, when two side-by-side trucks were placed, the wheels were 4'-0" apart. For load positions 2 and 5, they were placed 5'-7" apart.

Load Positions 1A and 1B: The load trucks were placed symmetrically about the middle of the bridge transversely, straddling girder 3. These positions maximized the deflection and strains on the interior girder (FIB3). The longitudinal position of the trucks was different for the two load positions, 1A and 1B. Load position 1A maximized the load effects at mid span by placing the third axle 6'-8" from mid span. For load position 1B, the third axle was 17'-3.5" from mid span; the purpose was to load equally, in the transverse direction, the closure joint adjacent to mid span and the center of the next panel.

Hurricane Creek WB bridge load position 1A (for the before barrier load test) is shown in **Figures 2-2 and 2-3**. These figures show plan and cross-sectional views of the truck placement. Truck 1 was placed 13'-1.5" from the north edge of the deck panel to the centerline of the wheel load, and load truck 2 was 13'-1.5" from the south edge.

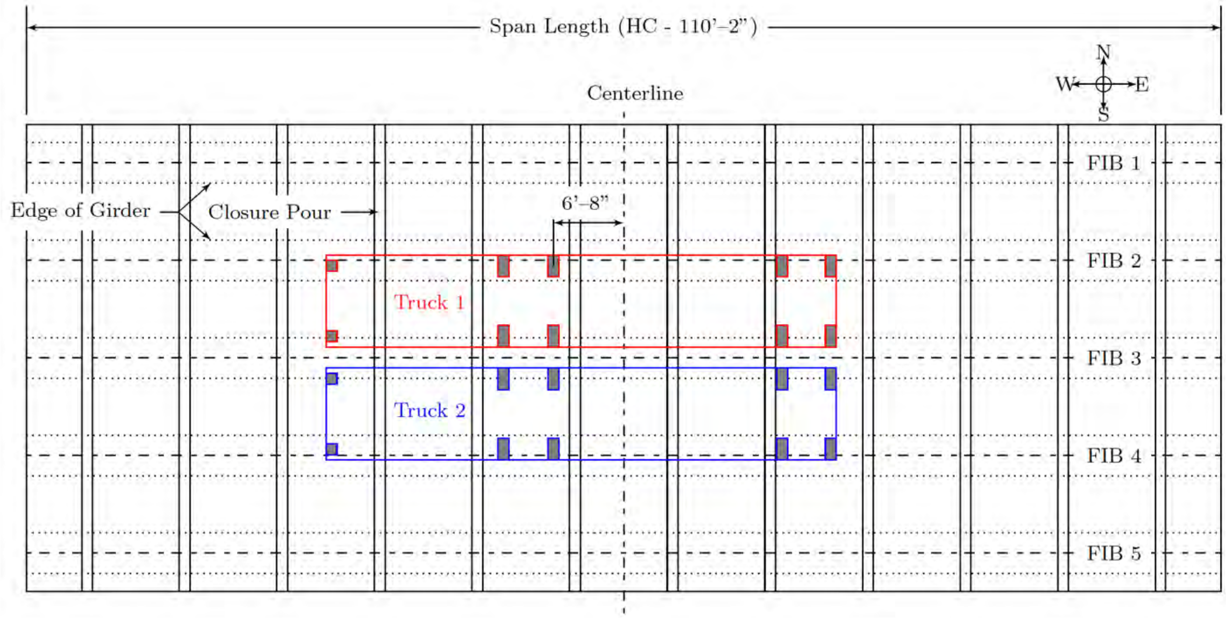


Fig. 2-2. Load Position 1A Plan View - Before Barrier

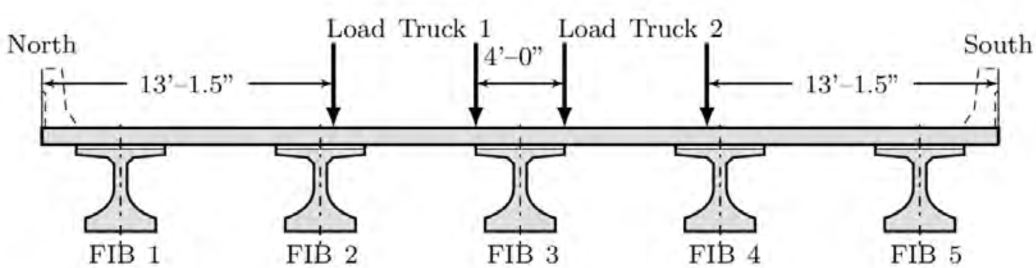


Fig. 2-3. Load Position 1A Cross Section - Before Barrier

For the tests performed after the barriers were constructed on each of the four (4) bridges, **Figures 2-4 and 2-5** show plan views of load positions 1A and 1B. As shown in **Figure 2-6**, truck 1 was placed 11'-7" from the face of the north barrier to the centerline of the wheel load. Likewise, truck 2 was placed 11'-7" from the face of the south barrier, leaving a 4'-0" spacing between the two trucks' wheel loads.

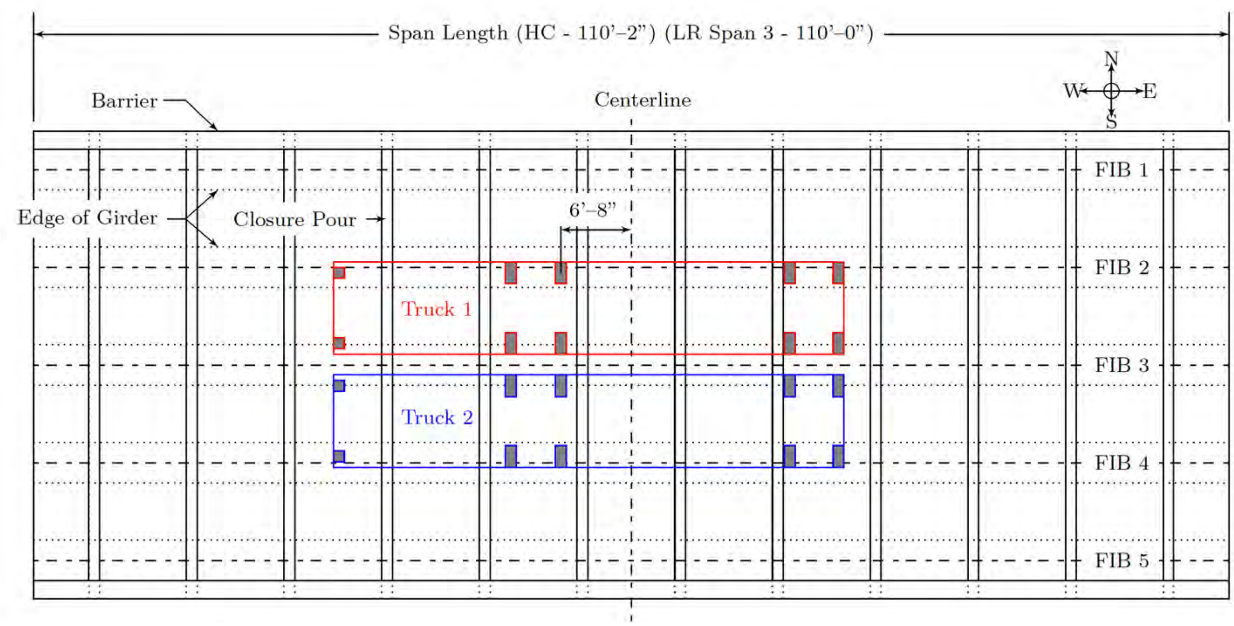


Fig. 2-4. Load Position 1A Plan View - After Barrier

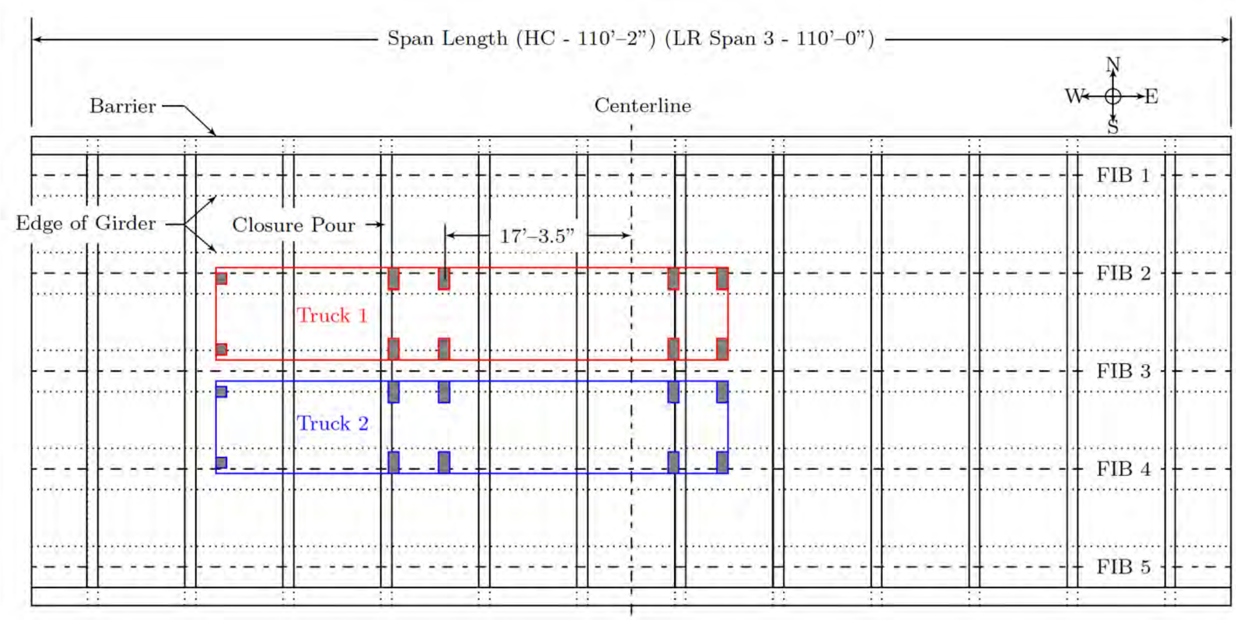


Fig. 2-5. Load Position 1B Plan View - After Barrier

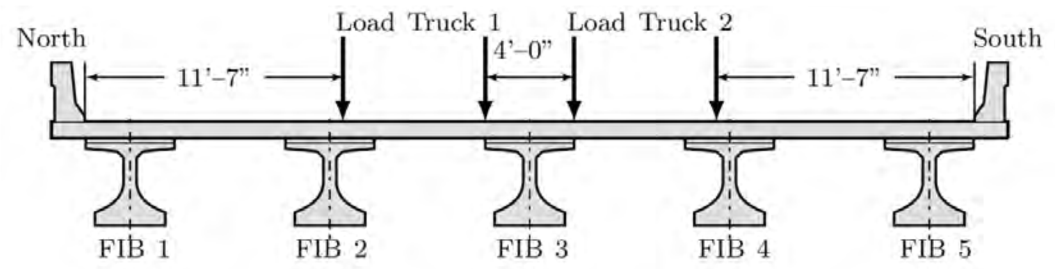


Fig. 2-6. Load Position 1 A/B Cross Section - After Barrier

Load Position 2: This load position was to maximize the load on one of the exterior girders. **Figures 2-7 and 2-8** show the truck locations for Hurricane Creek, before the barriers were constructed. Truck 1 was located 21'-4" from the north edge of the panel to the centerline of the wheel load, and truck 2 was located 3'-4" from the south edge of the panel. Similarly, **Figures 2-9 and 2-10** are for the load tests performed after the barriers were constructed.

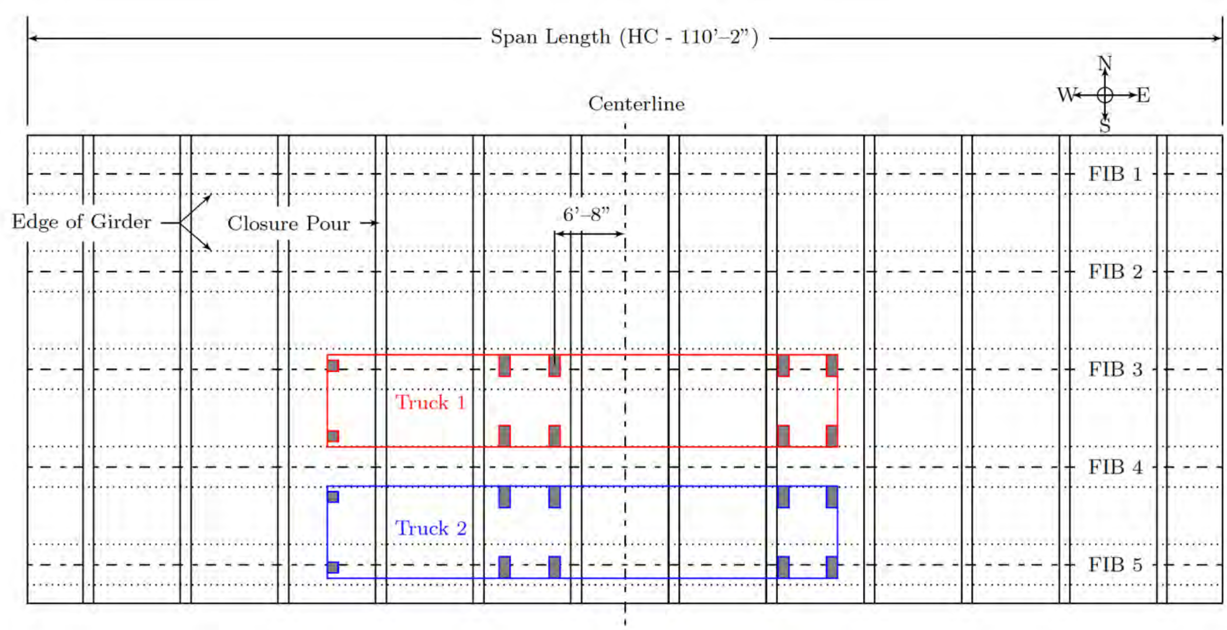


Fig. 2-7. Load Position 2 Plan View - Before Barrier

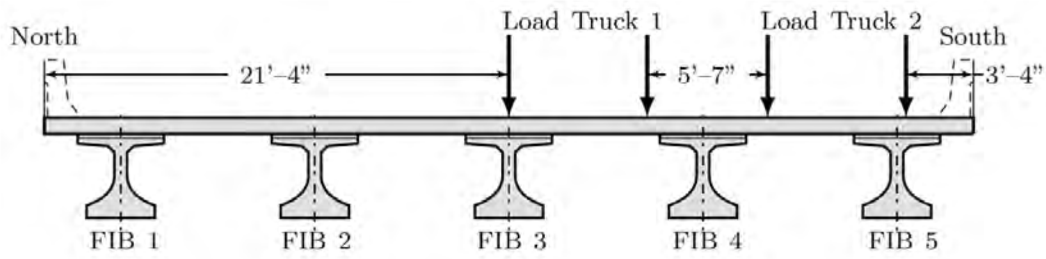


Fig. 2-8. Load Position 2 Cross Section - Before Barrier

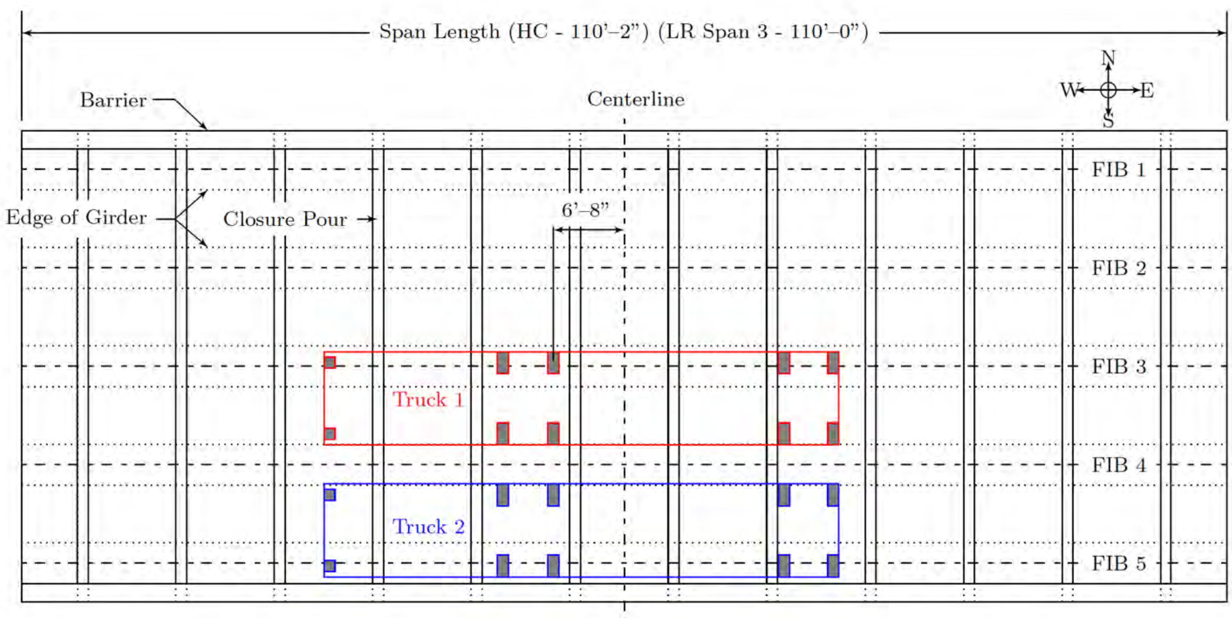


Fig. 2-9. Load Position 2 Plan View - After Barrier

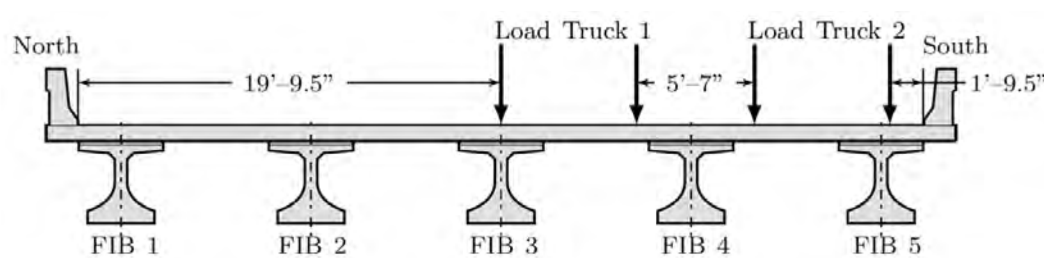


Fig. 2-10. Load Position 2 Cross Section - After Barrier

Load Positions 3A and 3B: As shown in **Figures 2-11 to 2-13**, these load positions asymmetrically loaded the bridge. Truck 1 was placed 16'-8" from the face of the north barrier to the centerline of the wheel load. Truck 2 was placed 6'-6" from the face of the south barrier, leaving a 4'-0" space between the two trucks' wheel loads. The truck position varied longitudinally for the two load positions, 3A and 3B.

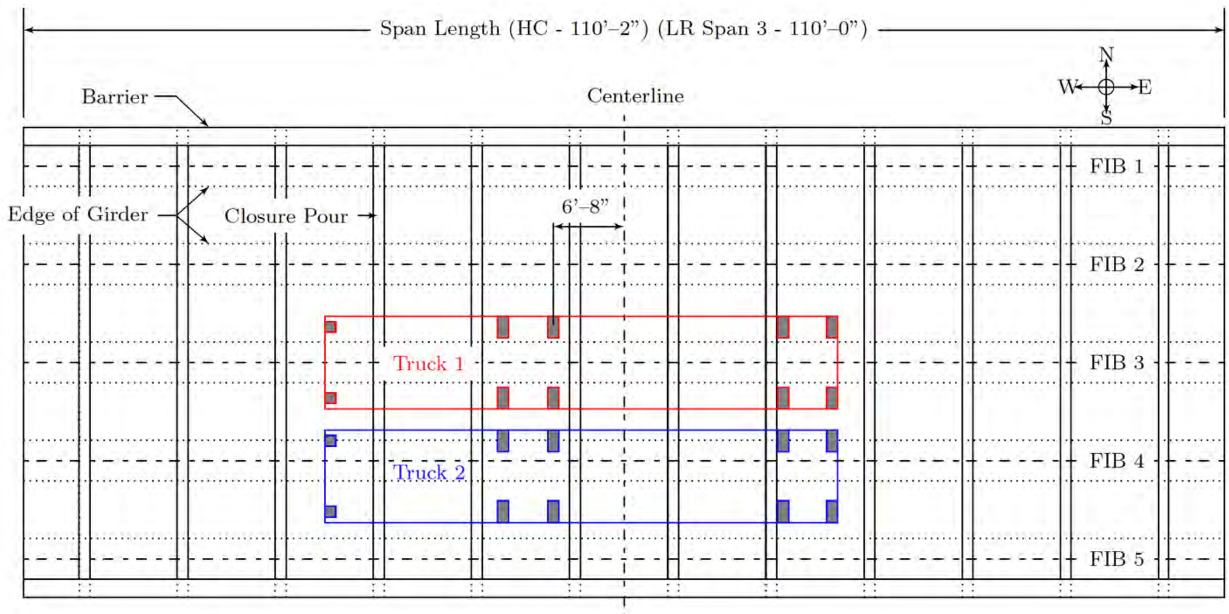


Fig. 2-11. Load Position 3A Plan View - After Barrier

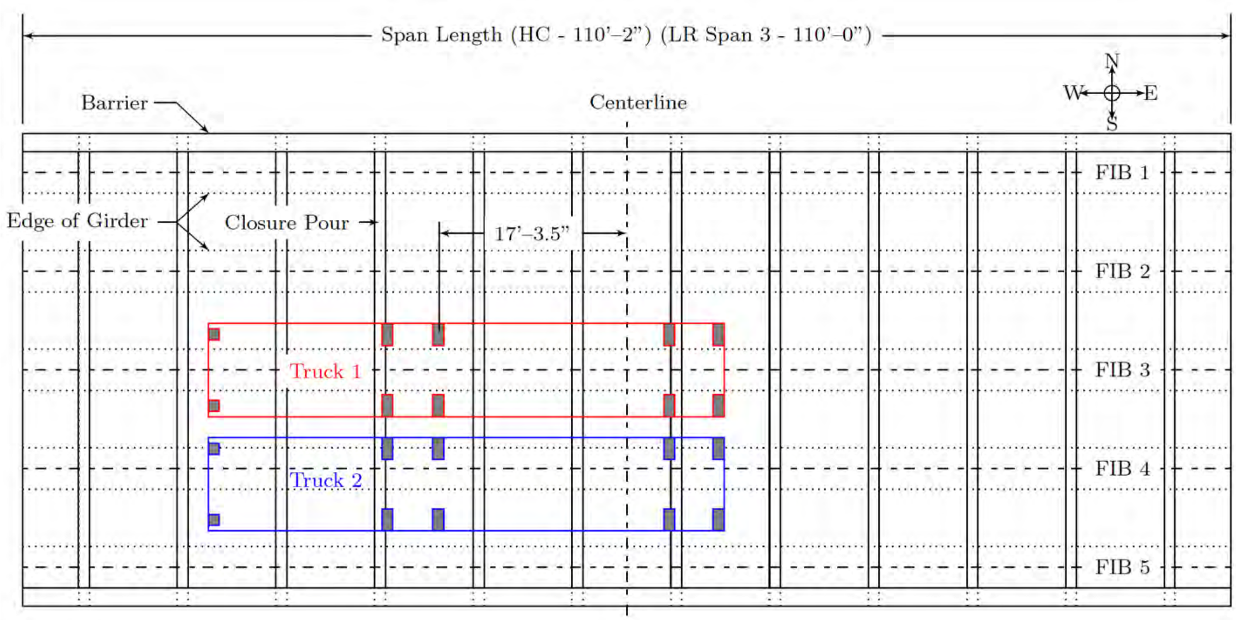


Fig. 2-12. Load Position 3B Plan View - After Barrier

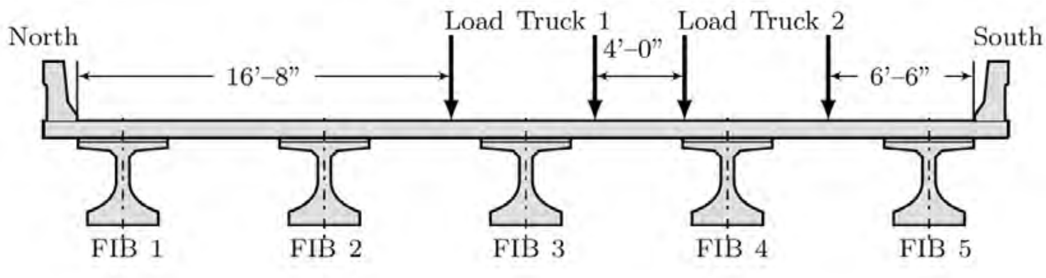


Fig. 2-13. Load Position 3 A/B Cross Section - After Barrier

Load Position 4: **Figures 2-14 and 2-15** show this load position, which was to maximize loading on the precast bent caps for the Little River bridges.

Load Position 5: **Figures 2-16 and 2-17** show this load position, which was to maximize loading on the precast bent caps for the Little River bridges.

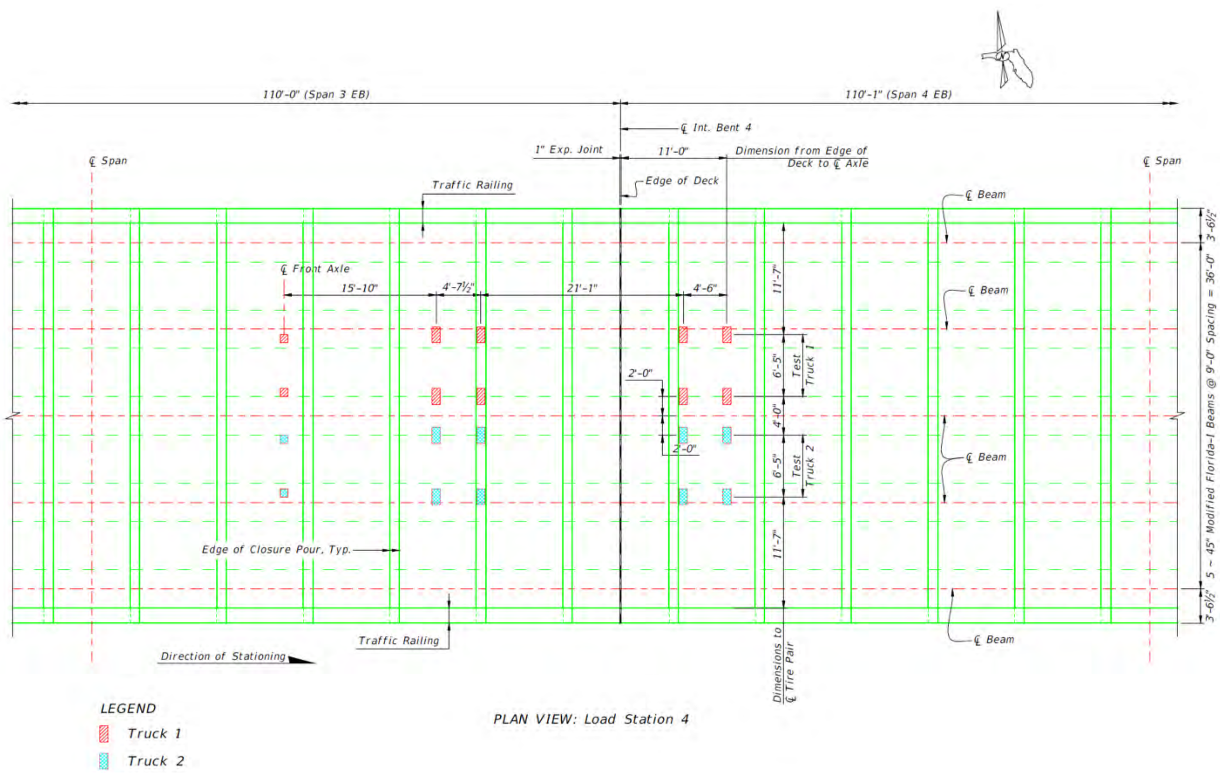


Fig. 2-14. Load Position 4 Plan View

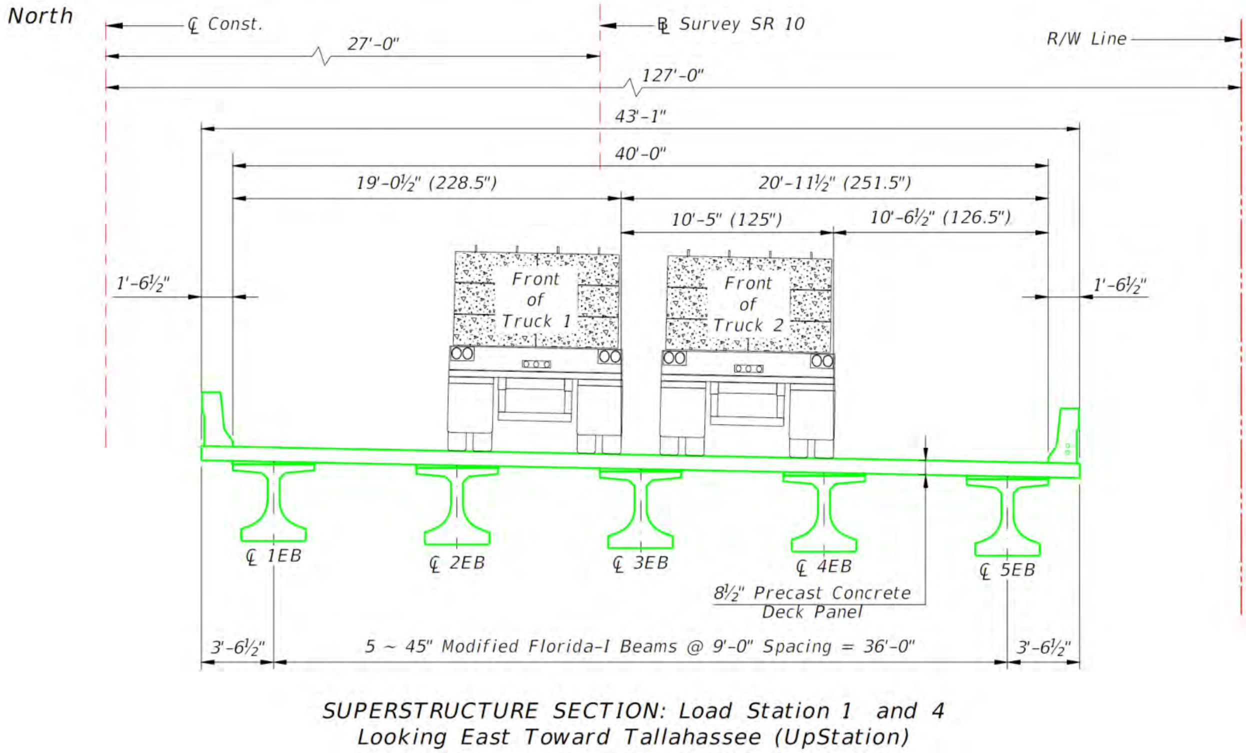


Fig. 2-15. Load Position 4 Cross Section

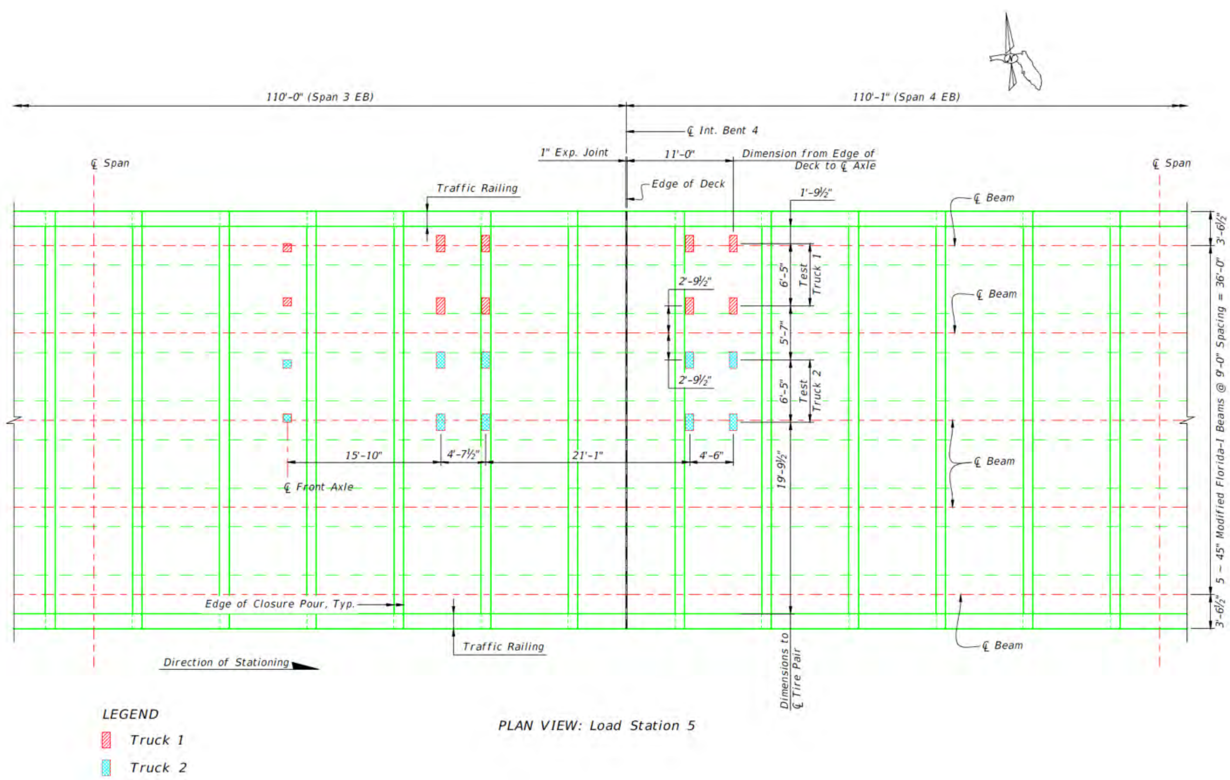


Fig. 2-16. Load Position 5 Plan View

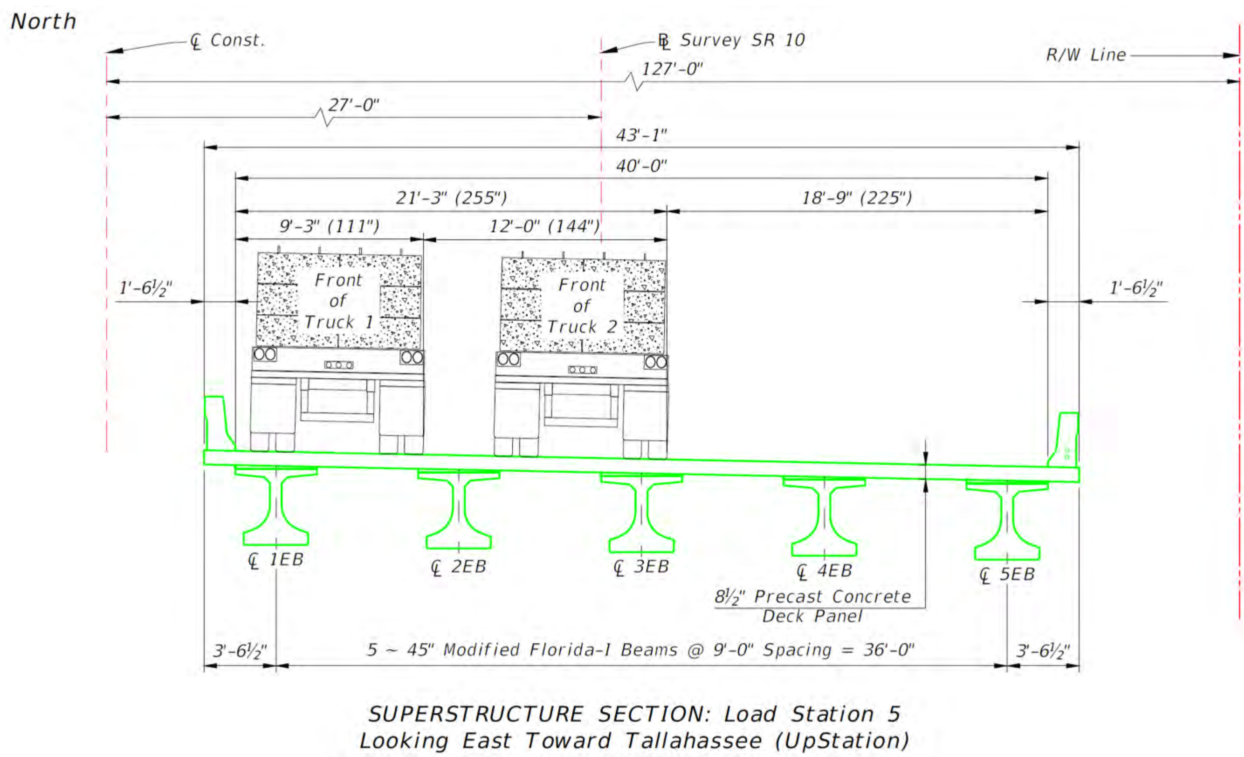


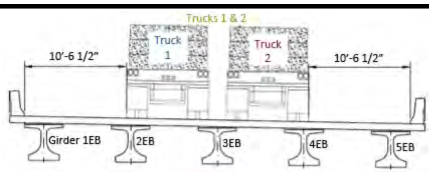
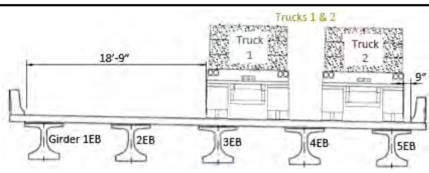
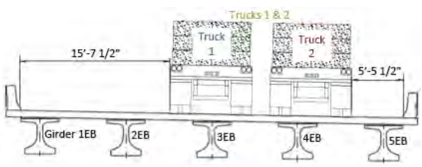
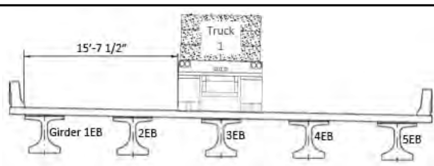
Fig. 2-17. Load Position 5 Cross Section

2.1.3 Test Procedure

Pre-service Load Tests: Similar steps and procedures were used for all load tests, as described in this section. **Tables 2-2 to 2-6** show the test steps for all load positions, for the pre-service load tests. For each test step, a 60-second recording was taken at one reading per second.

The test procedures for all the load tests followed the same pattern. For a given load position and block case, the cycle was as follows. The gauges were zeroed initially. Then, recordings were made with the first truck (truck 1) placed on the bridge, followed by recordings with both the first and second trucks (trucks 1 and 2), and finally recordings were made after removing the first truck (with only truck 2 remaining on the bridge). This completed the cycle for a given load position.

Table 2-2. Test Procedure - Hurricane Creek Eastbound

Load Station *		Truck 1	Truck 2	Zero ⦸ Recording	TEST STEP # 12 blocks	18 blocks	24 blocks
1		-	-	⦸	1	17	33
		✔	-	-	2	18	34
		✔	✔	-	3	19	35
		-	✔	-	4	20	36
		-	-	-	5	21	37
2		-	-	⦸	6	22	38
		✔	-	-	7	23	39
		✔	✔	-	8	24	40
		-	✔	-	9	25	41
		-	-	-	10	26	42
3		-	-	⦸	11	27	43
		✔	-	-	12	28	44
		✔	✔	-	13	29	45
		-	✔	-	14	30	46
		-	✔	-	-	-	46a**
		-	-	-	15	31	47
Slow Roll down center of bridge		✔	-	-	-	-	"Slow Roll"
Last Zero Recording before adding blocks or ending test.		-	-	⦸	16	32	48

Notes:

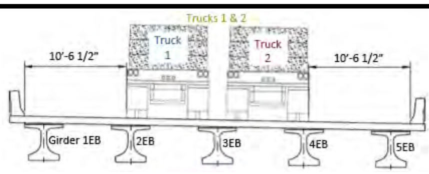
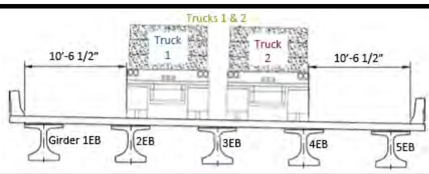
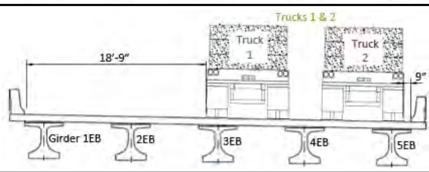
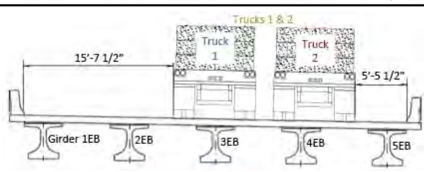
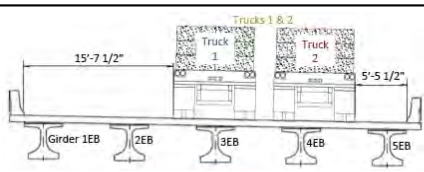
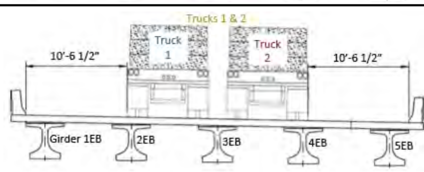
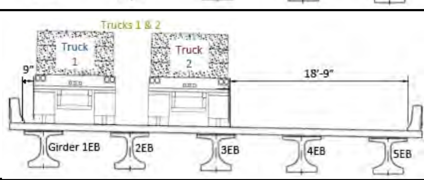
* For Load Stations 1-3, excluding Test Step #46a, the rear tires (Axle 5) were 36'-2" from the east end of bridge.

** For Test Step #46a, Sections B & C were straddled by Axles 2/3 on the left (west) and Axles 4/5 on the right (east). i.e, The rear tires (Axle 5) were 46'-9.5" from the east end of bridge.

After each load position was completed, the cycle was repeated for the next load position, starting with a reading just before truck 1 was placed; ideally, the measurements would read "zero", if there were no drift in the gauge readings nor residual strains.

After all load positions were completed for the 12-block case, the entire procedure was repeated for the 18- and 24-block cases.

Table 2-3. Test Procedure - Little River Eastbound

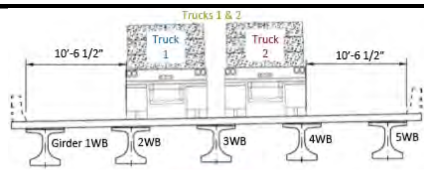
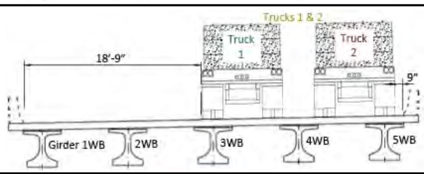
Load Station *	Truck 1	Truck 2	Zero ⊘ Recording	TEST STEP # 12 blocks	TEST STEP # 18 blocks	TEST STEP # 24 blocks
1A 	-	-	⊘	1	37	73
	✔	-	-	2	38	74
	✔	✔	-	3	39	75
	-	✔	-	4	40	76
	-	-	-	5	41	77
1B 	-	-	⊘	6	42	78
	✔	-	-	7	43	79
	✔	✔	-	8	44	80
	-	✔	-	9	45	81
	-	-	-	10	46	82
2 	-	-	⊘	11	47	83
	✔	-	-	12	48	84
	✔	✔	-	13	49	85
	-	✔	-	14	50	86
	-	-	-	15	51	87
3A 	-	-	⊘	16	52	88
	✔	-	-	-	-	88a**
	✔	-	-	17	53	89
	✔	✔	-	18	54	90
	-	✔	-	19	55	91
	-	✔	-	-	-	91a**
	-	-	-	20	56	92
3B 	-	-	⊘	21	57	93
	✔	-	-	22	58	94
	✔	✔	-	23	59	95
	-	✔	-	24	60	96
	-	-	-	25	61	97
4 	-	-	⊘	26	62	98
	✔	-	-	27	63	99
	✔	✔	-	28	64	100
	-	✔	-	29	65	101
	-	-	-	30	66	102
5 	-	-	⊘	31	67	103
	✔	-	-	32	68	104
	✔	✔	-	33	69	105
	-	✔	-	34	70	106
	-	-	-	35	71	107
Last Zero Recording before adding blocks or ending test.	-	-	⊘	36	72	108

Notes:

* The longitudinal position of the trucks is not shown on this table. It is different for each load station.

** For Test Step #88a and 91a, Sections B & C were straddled by Axles 2/3 on the left (west) and Axles 4/5 on the right (east). i.e, The rear tires (Axle 5) were 46'-9.5" from the east end of the span.

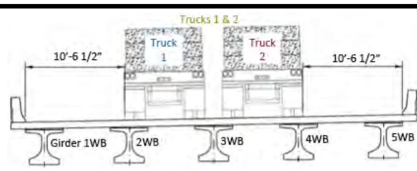
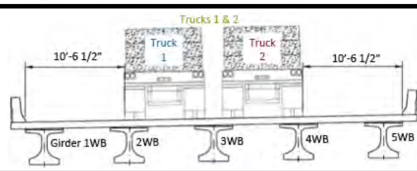
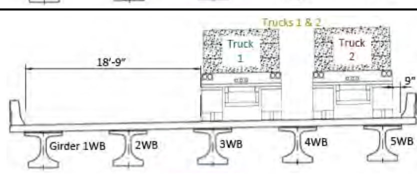
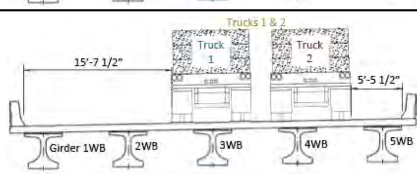
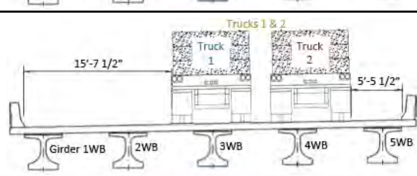
Table 2-4. Test Procedure - Hurricane Creek Westbound - Before Barriers

Load Station *		Truck 1	Truck 2	Zero ⊘ Recording	TEST STEP # 12 blocks	18 blocks	24 blocks
1A		-	-	⊘	1	12	23
		✔	-	-	2	13	24
		✔	✔	-	3	14	25
		-	✔	-	4	15	26
		-	-	-	5	16	27
2		-	-	⊘	6	17	28
		✔	-	-	7	18	29
		✔	✔	-	8	19	30
		-	✔	-	9	20	31
		-	-	-	10	21	32
Last Zero Recording before adding blocks or ending test.		-	-	⊘	11	22	33

Notes:

* For Load Stations 1A and 2, the rear tires (Axle 5) were 36'-2" from the east end of bridge.

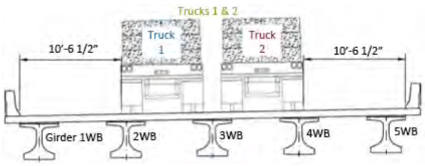
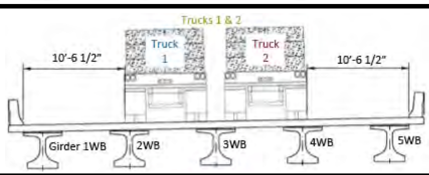
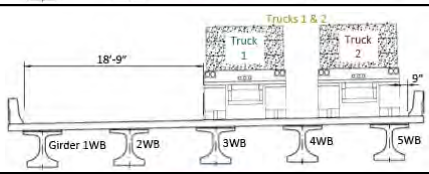
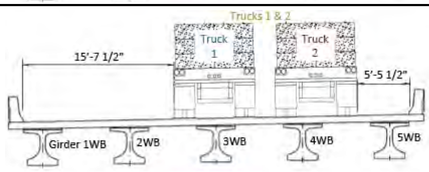
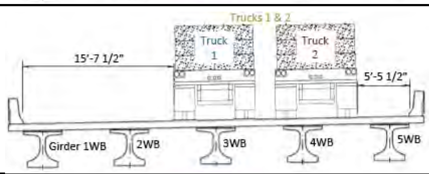
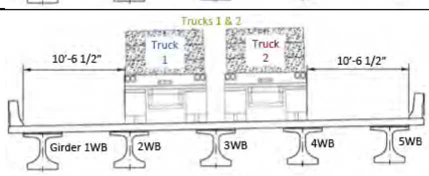
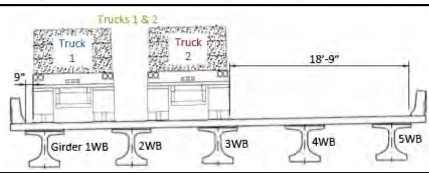
Table 2-5. Test Procedure - Hurricane Creek Westbound - After Barriers

Load Station *		Truck 1	Truck 2	Zero ⊘ Recording	TEST STEP # 12 blocks	18 blocks	24 blocks
1A		-	-	⊘	1	27	53
		✔	-	-	2	28	54
		✔	✔	-	3	29	55
		-	✔	-	4	30	56
		-	-	-	5	31	57
1B		-	-	⊘	6	32	58
		✔	-	-	7	33	59
		✔	✔	-	8	34	60
		-	✔	-	9	35	61
		-	-	-	10	36	62
2		-	-	⊘	11	37	63
		✔	-	-	12	38	64
		✔	✔	-	13	39	65
		-	✔	-	14	40	66
		-	-	-	15	41	67
3A		-	-	⊘	16	42	68
		✔	-	-	17	43	69
		✔	✔	-	18	44	70
		-	✔	-	19	45	71
		-	-	-	20	46	72
3B		-	-	⊘	21	47	73
		✔	-	-	22	48	74
		✔	✔	-	23	49	75
		-	✔	-	24	50	76
		-	-	-	25	51	77
Last Zero Recording before adding blocks or ending test.		-	-	⊘	26	52	78

Note:

* The longitudinal position of the trucks is not shown on this table. It is different for each load station.

Table 2-6. Test Procedure - Little River Westbound

Load Station *		Truck 1	Truck 2	Zero ⊘ Recording	TEST STEP # 12 blocks	18 blocks	24 blocks
1A		-	-	⊘	1	37	73
		✔	-	-	2	38	74
		✔	✔	-	3	39	75
		-	✔	-	4	40	76
		-	-	-	5	41	77
1B		-	-	⊘	6	42	78
		✔	-	-	7	43	79
		✔	✔	-	8	44	80
		-	✔	-	9	45	81
		-	-	-	10	46	82
2		-	-	⊘	11	47	83
		✔	-	-	12	48	84
		✔	✔	-	13	49	85
		-	✔	-	14	50	86
		-	-	-	15	51	87
3A		-	-	⊘	16	52	88
		✔	-	-	17	53	89
		✔	✔	-	18	54	90
		-	✔	-	19	55	91
		-	-	-	20	56	92
3B		-	-	⊘	21	57	93
		✔	-	-	22	58	94
		✔	✔	-	23	59	95
		-	✔	-	24	60	96
		-	-	-	25	61	97
4		-	-	⊘	26	62	98
		✔	-	-	27	63	99
		✔	✔	-	28	64	100
		-	✔	-	29	65	101
		-	-	-	30	66	102
5		-	-	⊘	31	67	103
		✔	-	-	32	68	104
		✔	✔	-	33	69	105
		-	✔	-	34	70	106
		-	-	-	35	71	107
Last Zero Recording before adding blocks or ending test.		-	-	⊘	36	72	108

Note:

* The longitudinal position of the trucks is not shown on this table. It is different for each load station.

In-service Load Tests: For the in-service tests, the steps were the same as the pre-service load tests, except for load positions 1B and 3B. For these two load positions, the 12- and 18-blocks cases were not tested (i.e., only the 24-blocks case was tested).

The naming convention used for the load position and trucks placed is LSA-TrB, where LS stands for load station (position), A represents the load position number, Tr stands for truck, and B is the truck number. For example, LS2B-Tr12 signifies load position 2B, with trucks 1 and 2 positioned on the bridge. Also, LSA-Zero represents zero gauge readings with no live load.

2.2 Instrumentation

2.2.1 Strain and Deflection Gauges

The measuring devices included strain gauges and deflection gauges. To measure strains in the longitudinal direction, mostly foil gauges were used. Transversely, "BDI" gauges were used. Vertical deflections were measured with wire deflection gauges (string pots), and horizontal slips were measured with slip deflection gauges. Additional strain and deflection gauges were placed on an intermediate bent on each Little River bridge. Each gauge type is described in the sections that follow.

Foil Strain Gauges: The gauges labeled 'F' followed by a number were all foil strain gauges. Foil strain gauges were used to measure longitudinal surface strains on the girders, deck panels, and barriers. These strain gauges work by measuring changes in the electric resistance running through a wire in the gauge. As the gauge is strained, the cross-sectional area of the wire changes, causing the resistance in the wire to change. The Data Acquisition System (DAS) converts this resistance into a strain reading.

The foil strain gauges were 60 mm long and manufactured by Kyowa Electronic Instruments Co., Ltd. Before applying a gauge, the concrete surface was prepared to remove any voids, dirt, and grime. The concrete was ground smooth, then cleaned with acetone. The gauges were then adhered to the concrete using "Zap Gel", the brand name of a cyanoacrylate glue. Two lead wires on each gauge were wired to an extension cable, for connection to the DAS.

BDI Gauges: Gauges labeled 'B' followed by a number were the BDI strain gauges, model ST350, manufactured by Bridge Diagnostics, Inc. (BDI). BDI gauges were used to measure the transverse strains on the deck panels and closure joints and some of the longitudinal strains. This gauge is very rugged and can be reused. The circuit within the gauge is a full wheatstone bridge with four (4) 350-ohm foil gauges, with an effective gauge length of 3 in.

Metal plates were attached to the BDI gauge and were adhered to the concrete surface using a quick-set epoxy. For the gauges in the transverse direction, applied to cast-in-place concrete, extensions were used to increase the effective gauge length.

***Wire Deflection Gauges*:** Gauges labeled 'D' followed by a number are the wire deflection gauges, which were Series 6 Miniature Position Transducers, by Firstmark Controls. The gauges were attached to the girder bottom flanges, and then the wires were anchored to the ground or abutment with weights.

***Slip Deflection Gauges*:** The gauges labeled 'SD' followed by a number were the slip deflection gauges. The slip deflection gauges were displacement transducers (CDP-50 by TML) that measure displacement using a spring-loaded metal rod in a housing. This rod can measure displacement in both directions along its axis. Each gauge was glued to the side of the girder's top flange, at the end of the span. The rod pressed against a metal plate that was mounted to the underside of the deck panel. This enabled the measurement of horizontal slippage between the deck and the girder.

2.2.2 Gauge Positions

***Longitudinal Gauges on Superstructure*:** Hurricane Creek EB was the first bridge to be instrumented and load tested. This test utilized the most gauges, and from the knowledge gained, the number of gauges was reduced in future load tests. See **Figures 2-18 to 2-21** for the longitudinal gauge numbers and locations on the panels and girders for the pre-service tests. There is an exception, however: on Hurricane Creek EB, BDI gauges B1-B5 were used in the longitudinal direction on the underside of the FIBs to validate the foil strain gauges F1-F5. For all other load tests, these BDI gauges were omitted and replaced with foil strain gauges F44, F46, F48, F50 and F52. Hurricane Creek EB also had strain gauges F44, F46, F48, F50 and F52; these gauges were not used on the other bridges.

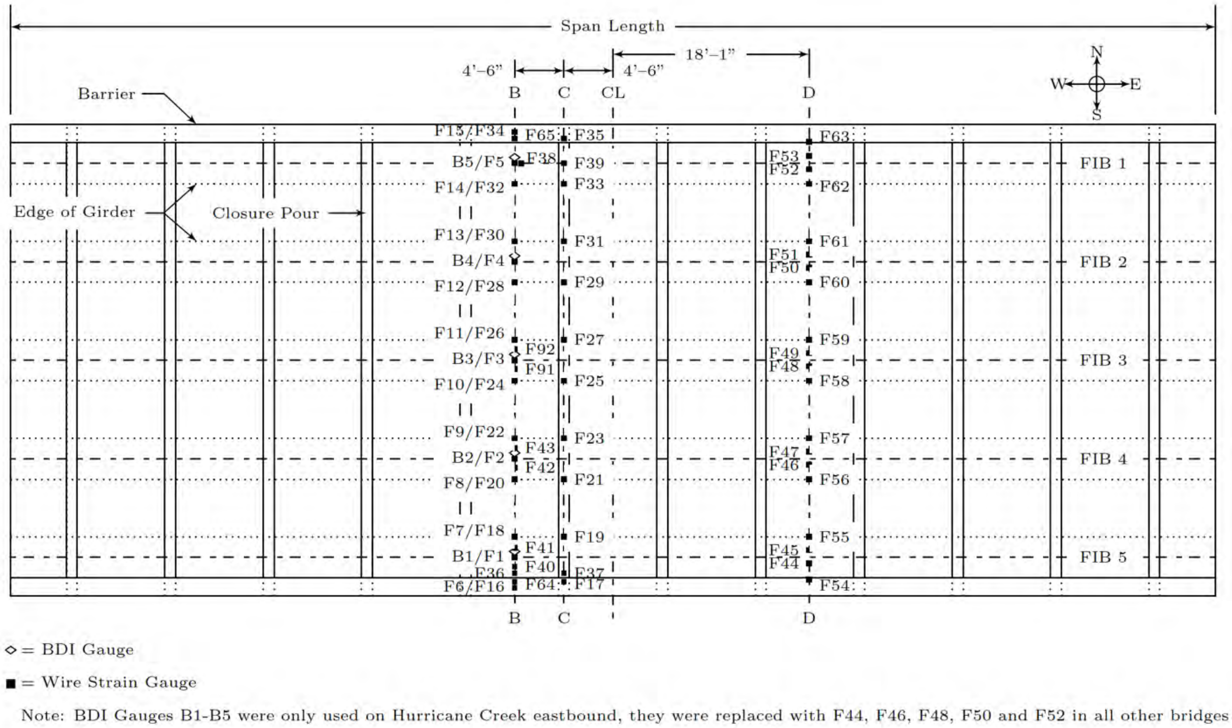


Fig. 2-18. Longitudinal Strain Gauge Locations Plan View

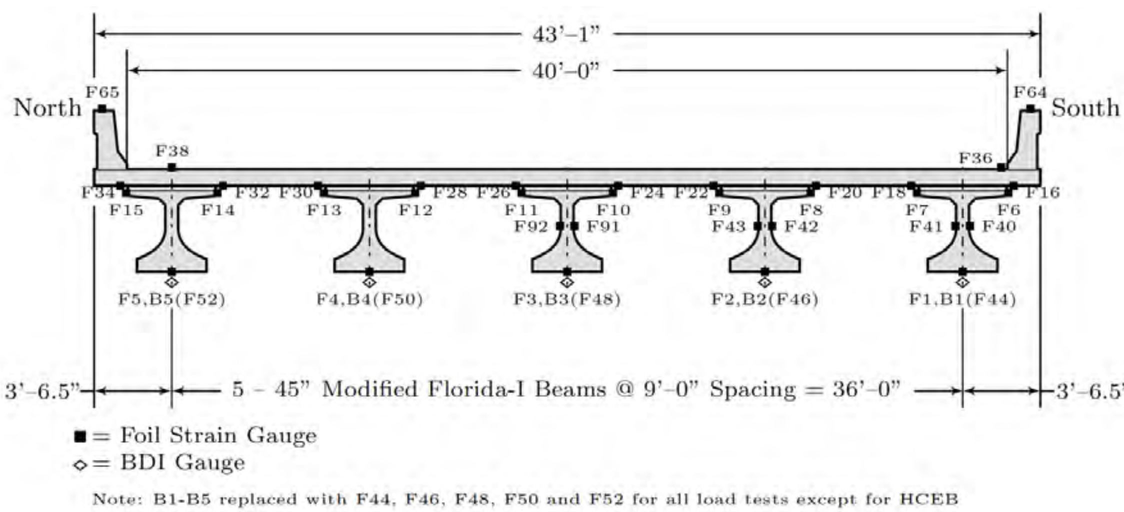


Fig. 2-19. Longitudinal Strain Gauge Locations - Section B

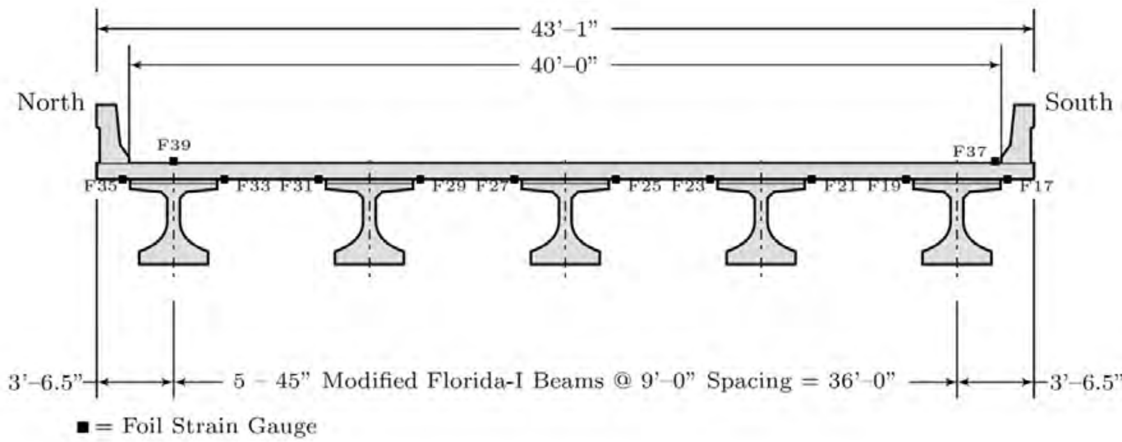


Fig. 2-20. Longitudinal Strain Gauge Locations - Section C

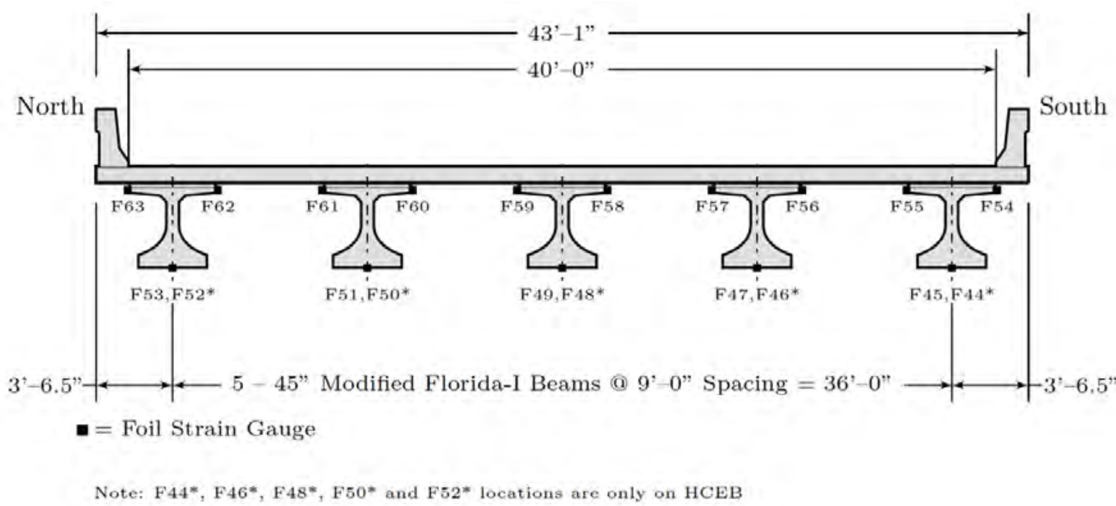


Fig. 2-21. Longitudinal Strain Gauge Locations - Section D

Most of the data analysis focused on section B for the longitudinal strain results. As shown in Figure 2-18, Section B is located 9'-0" from mid span. This gauge line had the heaviest concentration of gauges. As shown in Figure 2-19, gauges were placed on the bottom flanges of the FIBs, on the underside of the deck panel adjacent to the girder top flanges, on the top flanges of the FIBs, at mid height on the webs of the south side girders, on the top of the deck, and also on top of the barriers.

As shown in Figure 2-20, Section C only had a few gauges on the underside of the deck panel, adjacent to the girders, and some on the top of the deck panel. This gauge line was located along one of the 12-inch joints between two panels, which is 4'-6" west of mid span. These gauges were for evaluating the continuity of the joint and deck panel and to determine if they share load equally.

As shown in Figure 2-21, Section D is located 18'-1" east of mid span and had gauges placed on the girders' top and bottom flanges. Gauges F44, F46, F48, F50, and F52 were only used on section D for Hurricane Creek EB test. For the other bridges, those particular gauge numbers were used elsewhere.

***Transverse Gauges on Superstructure*:** Transverse gauges were used to evaluate the effectiveness of the transverse joints, and to compare their strains to the strains in the deck panels. Two different types of deck panels were used (non-prestressed and prestressed), and the strains on the two types were compared in the data analysis.

The plan view in **Figure 2-22** shows the transverse BDI strain gauge locations for the pre-service tests, and **Figure 2-23** shows a cross section for both gauge lines. BDI gages B7, B9, B11, and B13 were located along the centerline and on the underside of the panel, and B6, B8, B10, and B12 were located underneath the closure pour. The gauges were placed along the same gauge line at sections B and C.

***Deflection Gauges on Superstructure*:** Two types of deflection gauges, wire displacement gauges and slip deflections gauges, were used. The gauge locations are shown in **Figures 2-24 and 2-25**. The gauge lines were placed at mid span and 18'-1" east and west of mid span. Additional gauges were located at the ends of the spans.

Figures 2-24 and 2-25 show the maximum number of gauges used. As mentioned previously, some gauges were not used in all load tests. Also, most of the data analyses for the girder deflections focused on mid span.

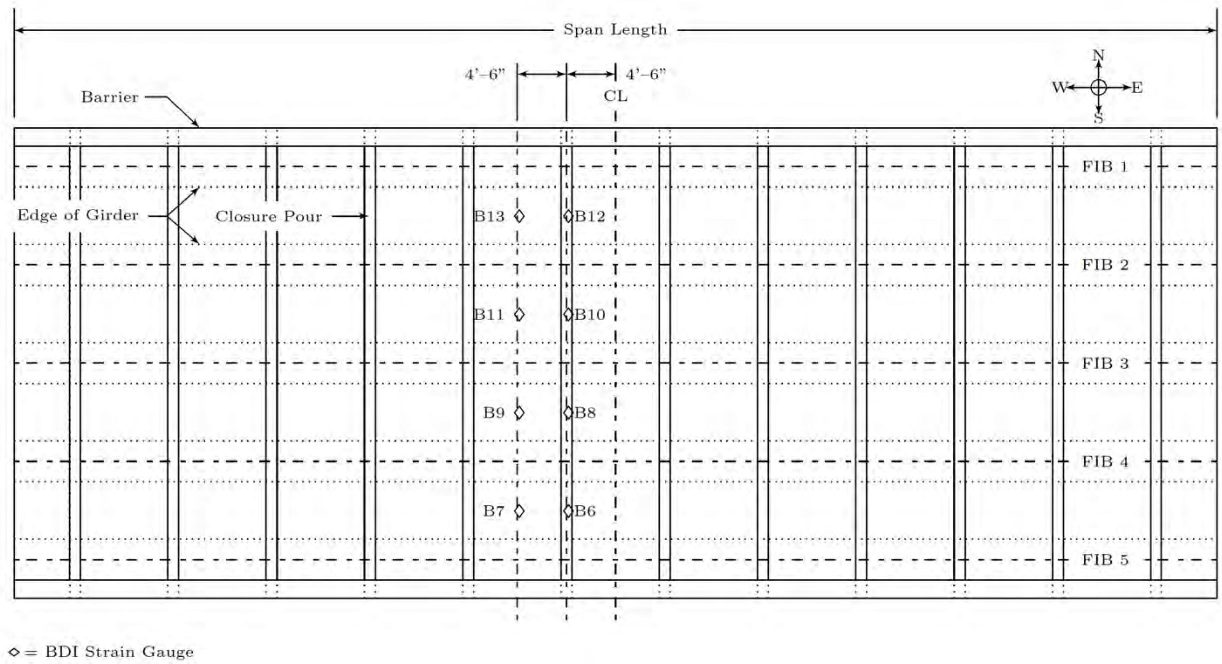


Fig. 2-22. Transverse Strain Gauge Locations Plan View

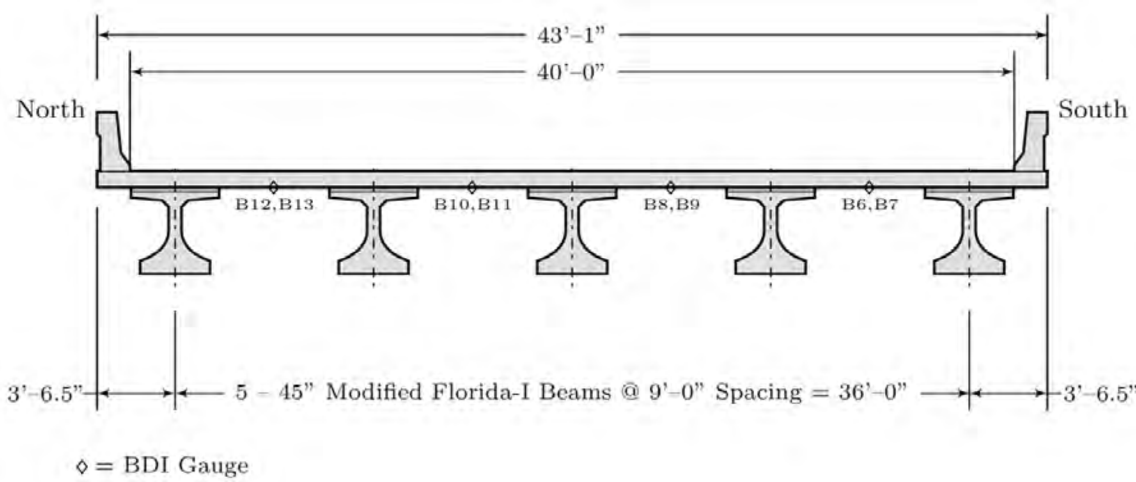


Fig. 2-23. Transverse Strain Gauge Locations Cross Section

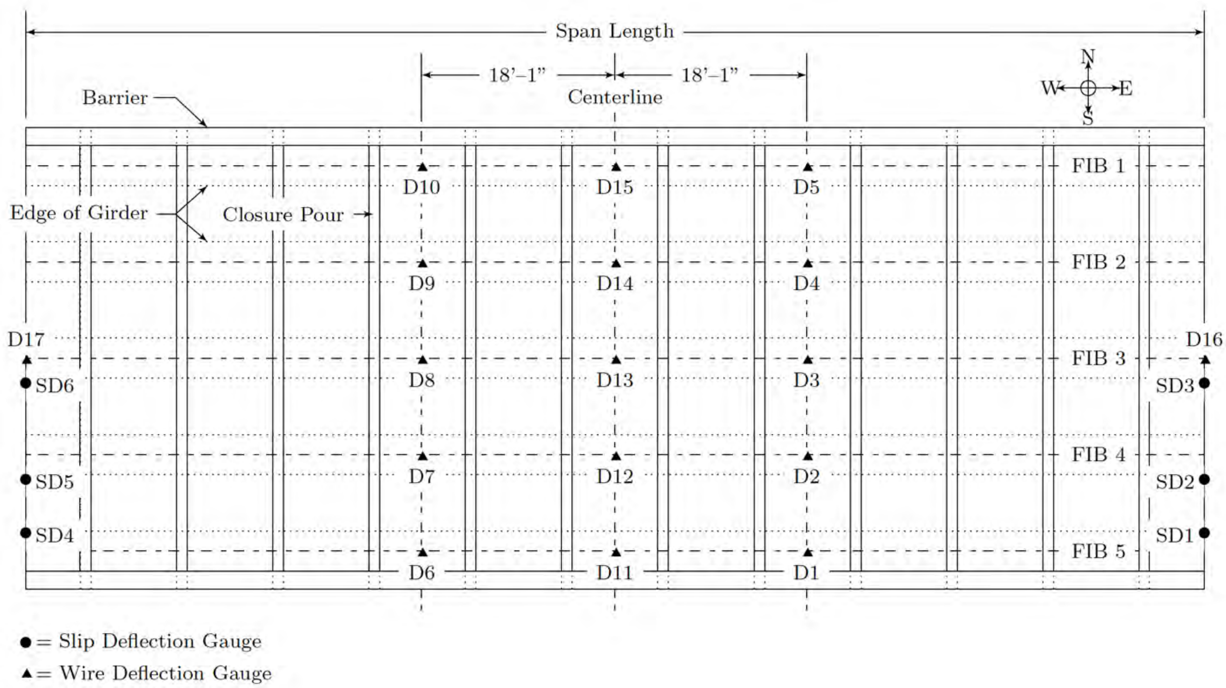


Fig. 2-24. Deflection Gauge Locations Plan View

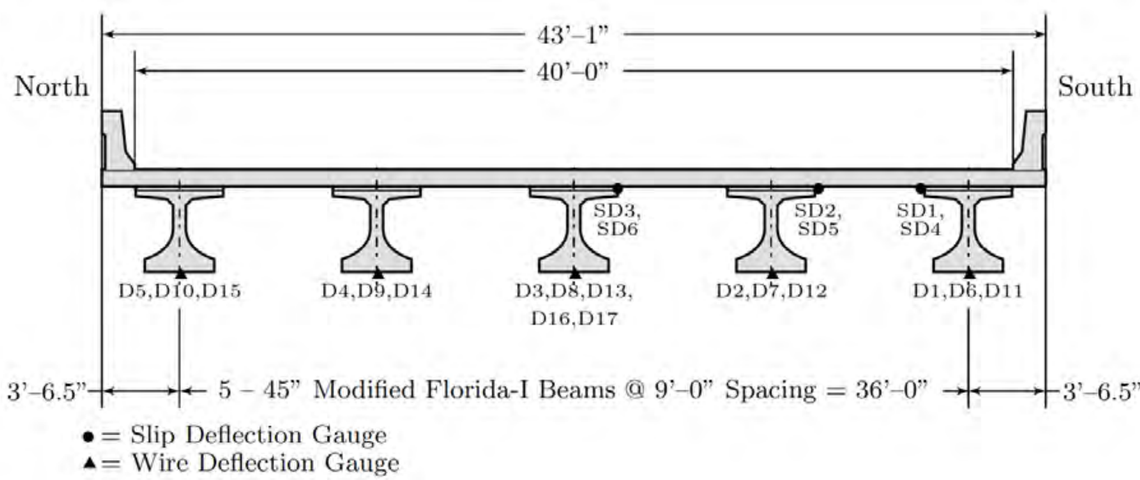


Fig. 2-25. Deflection Gauge Locations Cross Section

***Gauges on Substructure (Bent Caps)*: Figures 2-26 to 2-28** show the numbers and locations of the gauges on the bent columns and caps. Foil, deflection, and slip deflection gauges were used.

2.2.3 Gauge Numbers

Gauges numbers were kept consistent as much as possible throughout the test program, from bridge to bridge, and from pre-service to in-service tests. **Tables 2-7 to 2-11** show which gauges were used for each pre-service load test, as indicated by checkmarks next to the gauge numbers.

2.2.4 Data Acquisition

The gauges were connected to one of three nodes that were bolted to the FIBs. The nodes were then connected to the DAS, which was set up on the top of the bridge. Live readings were monitored throughout the load tests, to make sure the readings stayed in the linear-elastic range. For each test step recording, the data was collected at 1 Hz for at least 60 seconds, and the average of the approximately 60 readings was later calculated to represent a static measurement for the data analysis.

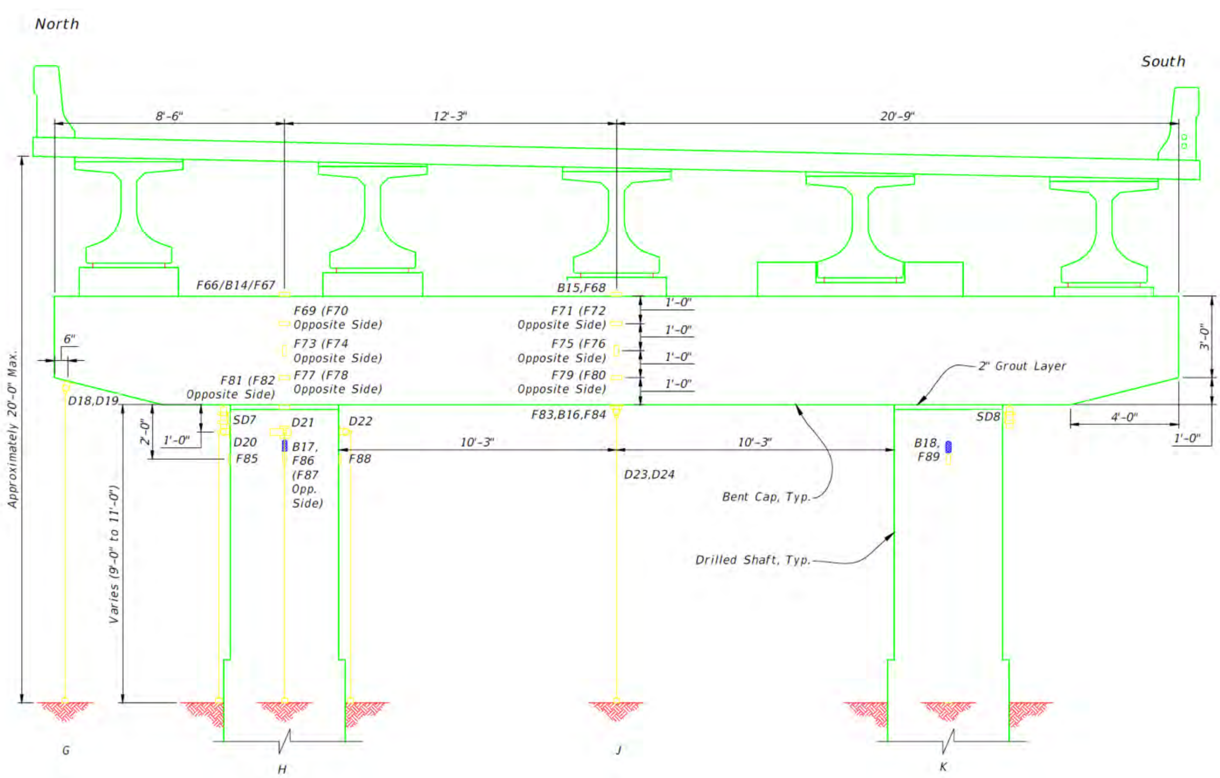


Fig. 2-26. Elevation of Bent Cap 3 with Attached Strain Gauges

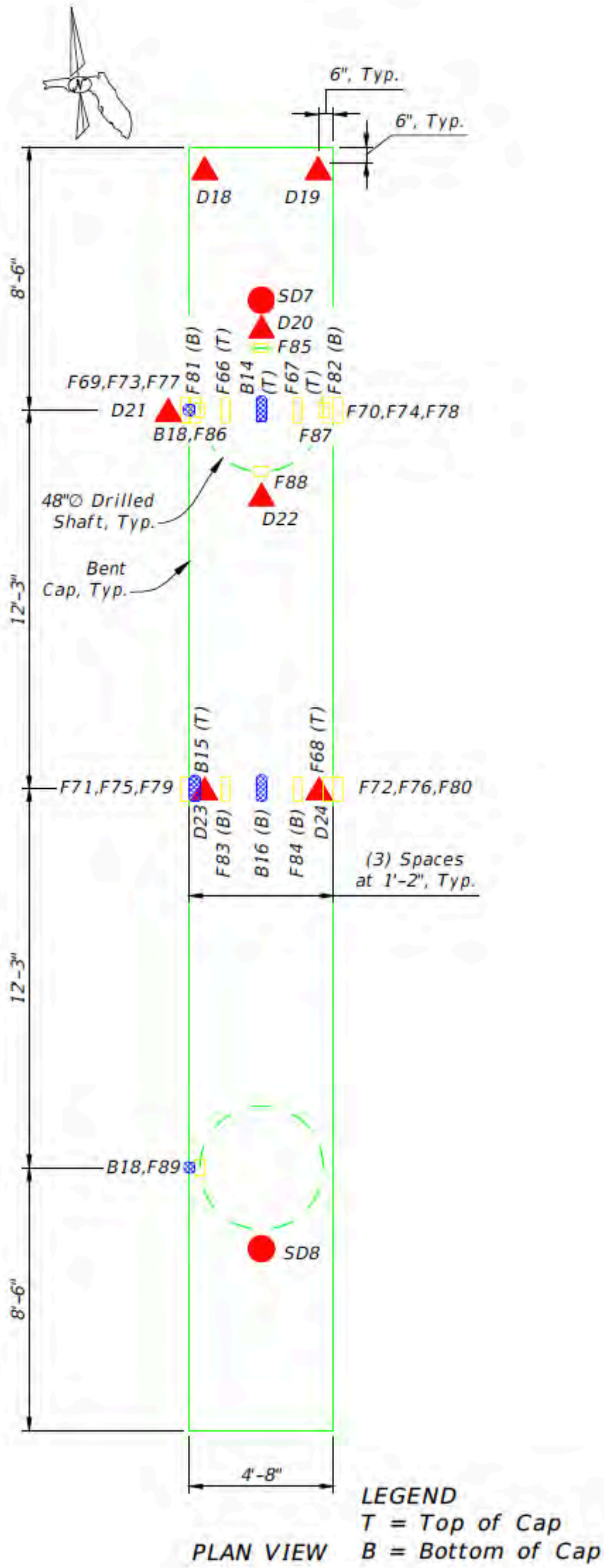


Fig. 2-27. Plan View of Bent Cap 3 with Attached Strain Gauges

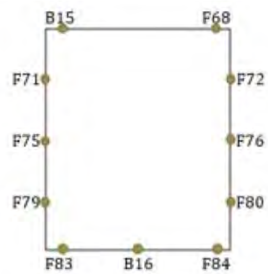


Fig. 2-28. Mid-section of Bent Cap Showing Gauges on Left and Right Face of Cap

Table 2-7. Hurricane Creek Eastbound Gauge Numbers

Deflection Gauges				Strain Gauges									
String Pot		Slip		Foil								BDI	
D1	✓	SD1	✓	F1	✓	F18	✓	F35	✓	F52*	✓	B1†	✓
D2	✓	SD2	✓	F2	✓	F19	✓	F36	✓	F53	✓	B2†	✓
D3	✓	SD3	✓	F3	✓	F20	✓	F37	✓	F54	✓	B3†	✓
D4	✓	SD4	✓	F4	✓	F21	✓	F38	✓	F55	✓	B4†	✓
D5	✓	SD5	✓	F5	✓	F22	✓	F39	✓	F56	✓	B5†	✓
D6	✓	SD6	✓	F6	✓	F23	✓	F40	✓	F57	✓	B6	✓
D7	✓			F7	✓	F24	✓	F41	✓	F58	✓	B7	✓
D8	✓			F8	✓	F25	✓	F42	✓	F59	✓	B8	✓
D9	✓			F9	✓	F26	✓	F43	✓	F60	✓	B9	✓
D10	✓			F10	✓	F27	✓	F44*	✓	F61	✓	B10	✓
D11	✓			F11	✓	F28	✓	F45	✓	F62	✓	B11	✓
D12	✓			F12	✓	F29	✓	F46*	✓	F63	✓	B12	✓
D13	✓			F13	✓	F30	✓	F47	✓	F64	✓	B13	✓
D14	✓			F14	✓	F31	✓	F48*	✓	F65	✓		
D15	✓			F15	✓	F32	✓	F49	✓	F91			
D16	✓			F16	✓	F33	✓	F50*	✓	F92			
D17	✓			F17	✓	F34	✓	F51	✓				
Total SP: 17		Total Slip: 6		Total Foil: 65								Total BDI: 13	
Total Number of Gauges: 101													

* Strain gauge located at 18' -1" east of mid span of bridge
† BDI gauge located 9'-0" west of mid span on bottom flange of girders

Table 2-8. Little River Eastbound Gauge Numbers

Deflection Gauges				Strain Gauges									
String Pot		Slip		Foil								BDI	
D1	✓	SD1		F1	✓	F18	✓	F35	✓	F52*	✓	B1	
D2	✓	SD2	✓	F2	✓	F19	✓	F36	✓	F53		B2	
D3	✓	SD3		F3	✓	F20		F37	✓	F54		B3	
D4	✓	SD4		F4	✓	F21		F38		F55		B4	
D5	✓	SD5	✓	F5	✓	F22	✓	F39		F56		B5	
D6		SD6		F6	✓	F23	✓	F40	✓	F57		B6	✓
D7				F7	✓	F24		F41	✓	F58		B7	✓
D8				F8	✓	F25		F42	✓	F59		B8	✓
D9				F9	✓	F26	✓	F43	✓	F60		B9	✓
D10				F10		F27	✓	F44*	✓	F61		B10	✓
D11	✓			F11		F28		F45		F62		B11	✓
D12	✓			F12		F29		F46*	✓	F63		B12	✓
D13	✓			F13		F30	✓	F47		F64	✓	B13	✓
D14	✓			F14		F31	✓	F48*	✓	F65	✓		
D15	✓			F15		F32		F49		F91			
D16	✓			F16		F33		F50*	✓	F92			
D17	✓			F17		F34	✓	F51					
Total SP: 12		Total Slip: 2		Total Foil: 32								Total BDI: 8	
Total Number of Gauges: 54													

* Strain gauge located at section B

Table 2-9. Hurricane Creek Westbound - Before Barrier Gauge Numbers

Deflection Gauges				Strain Gauges									
String Pot		Slip		Foil								BDI	
D1	✓	SD1		F1	✓	F18	✓	F35		F52*	✓	B1	
D2	✓	SD2	✓	F2	✓	F19		F36	✓	F53		B2	
D3	✓	SD3		F3	✓	F20		F37		F54		B3	
D4	✓	SD4		F4	✓	F21		F38	✓	F55		B4	
D5	✓	SD5	✓	F5	✓	F22	✓	F39		F56		B5	
D6		SD6		F6	✓	F23		F40	✓	F57		B6	
D7				F7	✓	F24		F41	✓	F58		B7	
D8				F8	✓	F25		F42	✓	F59		B8	
D9				F9	✓	F26	✓	F43	✓	F60		B9	
D10				F10		F27		F44*	✓	F61		B10	
D11	✓			F11	✓	F28		F45		F62		B11	
D12	✓			F12		F29		F46*	✓	F63		B12	
D13	✓			F13	✓	F30	✓	F47		F64		B13	
D14	✓			F14		F31		F48*	✓	F65			
D15	✓			F15	✓	F32		F49					
D16	✓			F16		F33		F50*	✓				
D17	✓			F17		F34	✓	F51					
Total SP: 12		Total Slip: 2		Total Foil: 28								Total BDI: 0	
Total Number of Gauges: 42													

* Strain gauge located at section B

Table 2-10. Hurricane Creek Westbound - After Barrier Gauge Numbers

Deflection Gauges				Strain Gauges									
String Pot		Slip		Foil								BDI	
D1	✓	SD1		F1	✓	F18	✓	F35	✓	F52*	✓	B1	
D2	✓	SD2	✓	F2	✓	F19	✓	F36	✓	F53		B2	
D3	✓	SD3		F3	✓	F20		F37	✓	F54		B3	
D4	✓	SD4		F4	✓	F21		F38		F55		B4	
D5	✓	SD5	✓	F5	✓	F22	✓	F39		F56		B5	
D6		SD6		F6	✓	F23	✓	F40	✓	F57		B6	✓
D7				F7	✓	F24		F41	✓	F58		B7	✓
D8				F8	✓	F25		F42	✓	F59		B8	✓
D9				F9	✓	F26	✓	F43	✓	F60		B9	✓
D10				F10	✓	F27	✓	F44*	✓	F61		B10	✓
D11	✓			F11	✓	F28		F45		F62		B11	✓
D12	✓			F12		F29		F46*	✓	F63		B12	✓
D13	✓			F13		F30	✓	F47		F64	✓	B13	✓
D14	✓			F14		F31	✓	F48*	✓	F65	✓		
D15	✓			F15		F32		F49		F91	✓		
D16	✓			F16		F33		F50*	✓	F92	✓		
D17	✓			F17		F34	✓	F51					
Total SP: 12		Total Slip: 2		Total Foil: 36								Total BDI: 8	
Total Number of Gauges: 58													

* Strain gauge located at section B

Table 2-11. Little River Westbound Gauge Numbers

Deflection Gauges				Strain Gauges									
String Pot		Slip		Foil								BDI	
D1	✓	SD1		F1	✓	F18	✓	F35	✓	F52*	✓	B1	
D2	✓	SD2	✓	F2	✓	F19	✓	F36	✓	F53		B2	
D3	✓	SD3		F3	✓	F20		F37	✓	F54		B3	
D4	✓	SD4		F4	✓	F21		F38		F55		B4	
D5	✓	SD5	✓	F5	✓	F22	✓	F39		F56		B5	
D6		SD6		F6	✓	F23	✓	F40	✓	F57		B6	✓
D7				F7	✓	F24		F41	✓	F58		B7	✓
D8				F8	✓	F25		F42	✓	F59		B8	✓
D9				F9	✓	F26	✓	F43	✓	F60		B9	✓
D10				F10	✓	F27	✓	F44*	✓	F61		B10	✓
D11	✓			F11	✓	F28		F45		F62		B11	✓
D12	✓			F12		F29		F46*	✓	F63		B12	✓
D13	✓			F13		F30	✓	F47		F64	✓	B13	✓
D14	✓			F14		F31	✓	F48*	✓	F65	✓		
D15	✓			F15		F32		F49		F91	✓		
D16	✓			F16		F33		F50*	✓	F92	✓		
D17	✓			F17		F34	✓	F51					
Total SP: 12		Total Slip: 2		Total Foil: 36								Total BDI: 8	
Total Number of Gauges: 60													

* Strain gauge located at Section B

For the in-service load tests, see **Tables 2-12 to 2-15** for a side-by-side list of gauges that compares the gauges in the in-service tests to those in the pre-service tests.

Table 2-12. Hurricane Creek Eastbound Gauges (Pre-service and In-service)

Description of Location	Gauge Numbers	HCEB Pre-Service	HCEB In-Service
Deflection gauges at 18' east of mid span	D1	✓	✓
	D2	✓	✓
	D3	✓	✓
	D4	✓	✓
	D5	✓	✓
Deflection gauges at 18' west of mid span	D6	✓	
	D7	✓	
	D8	✓	
	D9	✓	
	D10	✓	
Deflection gauges at mid span	D11	✓	✓
	D12	✓	✓
	D13	✓	✓
	D14	✓	✓
	D15	✓	✓
Deflection gauges at the east end of the span	SD1	✓	
	SD2	✓	✓
	SD3	✓	
	D16/SD16	✓	✓
Deflection gauges at the west end of the span	SD4	✓	
	SD5	✓	✓
	SD6	✓	
	D17/SD17	✓	✓
Longitudinal Strain gauges at Section B: 9' west of mid span	F1	✓	✓
	F2	✓	✓
	F3	✓	✓
	F4	✓	✓
	F5	✓	✓
	F6	✓	✓
	F7	✓	✓
	F8	✓	✓
	F9	✓	✓
	F10	✓	✓
	F11	✓	✓
	F12	✓	
	F13	✓	
	F14	✓	
	F15	✓	
	F16	✓	
	F18	✓	✓
	F20	✓	
	F22	✓	✓

Table 2-12 (cont'd). Hurricane Creek Eastbound Gauges (Pre-service and In-service)

Description of Location	Gauge Number	HCEB Pre-Service	HCEB In-Service
Longitudinal Strain gauges at Section B: 9' west of mid span continued	F24	✓	
	F26	✓	✓
	F28	✓	
	F30	✓	✓
	F32	✓	
	F34	✓	✓
	F36	✓	✓
	F38	✓	
	F40	✓	✓
	F41	✓	✓
	F42	✓	✓
	F43	✓	✓
	F64	✓	✓
	F65	✓	✓
	F91		✓
	F92		✓
	B1/F44	✓	✓
	B2/F46	✓	✓
	B3/F48	✓	✓
	B4/F50	✓	✓
	B5/F52	✓	✓
Longitudinal Strain gauges at Section C: 4'-6" west of mid span	F17	✓	
	F19	✓	✓
	F21	✓	
	F23	✓	✓
	F25	✓	
	F27	✓	✓
	F29	✓	
	F31	✓	✓
	F33	✓	
	F35	✓	✓

Table 2-12 (cont'd). Hurricane Creek Eastbound Gauges (Pre-service and In-service)

Description of Location	Gauge Number	HCEB Pre-Service	HCEB In-Service
	F37	✓	✓
	F39	✓	
Longitudinal Strain gauges at Section D: 18'-1" east of mid span	F44	✓	
	F45	✓	
	F46	✓	
	F47	✓	
	F48	✓	
	F49	✓	
	F50	✓	
	F51	✓	
	F52	✓	
	F53	✓	
	F54	✓	
Longitudinal Strain gauges at Section D: 18'-1" east of mid span continued	F55	✓	
	F56	✓	
	F57	✓	
	F58	✓	
	F59	✓	
	F60	✓	
	F61	✓	
	F62	✓	
	F63	✓	
Transverse Strain gauges at Section B: 9' west of mid span	B7	✓	✓
	B9	✓	✓
	B11	✓	✓
	B13	✓	✓
Transverse Strain gauges at Section C: 4'-6" west of mid span	B8	✓	✓
	B10	✓	✓
	B12	✓	✓
	B14	✓	✓
		Total = 101	Total = 58

Table 2-13. Hurricane Creek Westbound Gauges (Pre-service and In-service)

Description of Location	Gauge Numbers	HCWB Pre-Service	HCWB In-Service
Deflection gauges at 18' east of mid span	D1	✓	✓
	D2	✓	✓
	D3	✓	✓
	D4	✓	✓
	D5	✓	✓
Deflection gauges at mid span	D11	✓	✓
	D12	✓	✓
	D13	✓	✓
	D14	✓	✓
	D15	✓	✓
Deflection gauges at the east end of the span	SD2	✓	✓
	D16/SD16	✓	✓
Deflection gauges at the west end of the span	SD5	✓	✓
	D17/SD17	✓	✓
Strain gauges at Section B: 9' west of mid span	F1	✓	✓
	F2	✓	✓
	F3	✓	✓
	F4	✓	✓
	F5	✓	✓
	F6	✓	✓
	F7	✓	✓
	F8	✓	✓
	F9	✓	✓
	F10	✓	✓
	F11	✓	✓
	F18	✓	✓
	F22	✓	✓
	F26	✓	✓

Table 2-13 (cont'd). Hurricane Creek Westbound Gauges (Pre-service and In-service)

Description of Location	Gauge Number	HCWB Pre-Service	HCWB In-Service
Strain gauges at Section B: 9' west of mid span continued	F30	✓	✓
	F34	✓	✓
	F36	✓	✓
	F40	✓	✓
	F41	✓	✓
	F42	✓	✓
	F43	✓	✓
	F44	✓	✓
	F46	✓	✓
	F48	✓	✓
	F50	✓	✓
	F52	✓	✓
	F64	✓	✓
	F65	✓	✓
	F91	✓	✓
	F92	✓	✓
Strain gauges at Section C: 4'-6" west of mid span	F19	✓	✓
	F23	✓	✓
	F27	✓	✓
	F31	✓	✓
	F35	✓	✓
	F37	✓	✓
Transverse Strain gauges at Section B: 9' west of mid span	B7	✓	✓
	B9	✓	✓
	B11	✓	✓
	B13	✓	✓
Transverse Strain gauges at Section C: 4'-6" west of mid span	B8	✓	✓
	B10	✓	✓
	B12	✓	✓
	B14	✓	✓
		Total = 58	Total = 58

Table 2-14. Little River Eastbound Gauges (Pre-service and In-service)

Description of Location	Gauge Numbers	LREB Pre-Service	LREB In-Service
Deflection gauges at 18' east of mid span	D1	✓	✓
	D2	✓	✓
	D3	✓	✓
	D4	✓	✓
	D5	✓	✓
Deflection gauges at mid span	D11	✓	✓
	D12	✓	✓
	D13	✓	✓
	D14	✓	✓
	D15	✓	✓
Deflection gauges at the east end of the span	SD2	✓	✓
	D16/SD16	✓	✓
Deflection gauges at the west end of the span	SD5	✓	✓
	D17/SD17	✓	✓
Strain gauges at Section B: 9' west of mid span	F1	✓	✓
	F2	✓	✓
	F3	✓	✓
	F4	✓	✓
	F5	✓	✓
	F6	✓	✓
	F7	✓	✓
	F8	✓	✓
	F9	✓	✓
	F10		✓
	F11		✓
	F18	✓	✓
	F22	✓	✓
	F26	✓	✓

Table 2-14 (cont'd). Little River Eastbound Gauges (Pre-service and In-service)

Description of Location	Gauge Number	LREB Pre-Service	LREB In-Service
Strain gauges at Section B: 9' west of mid span	F30	✓	✓
	F34	✓	✓
	F36	✓	✓
	F40	✓	✓
	F41	✓	✓
	F42	✓	✓
	F43	✓	✓
	F44	✓	✓
	F46	✓	✓
	F48	✓	✓
	F50	✓	✓
	F52	✓	✓
	F64	✓	✓
	F65	✓	✓
	F91		✓
	F92		✓
Strain gauges at Section C: 4'-6" west of mid span	F19	✓	✓
	F23	✓	✓
	F27	✓	✓
	F31	✓	✓
	F35	✓	✓
	F37	✓	✓
Transverse Strain gauges at Section B: 9' west of mid span	B7	✓	✓
	B9	✓	✓
	B11	✓	✓
	B13	✓	✓
Transverse Strain gauges at Section C: 4'-6" west of mid span	B8	✓	✓
	B10	✓	✓
	B12	✓	✓
	B14	✓	✓
		Total = 54	Total = 58

Table 2-15. Little River Westbound Gauges (Pre-service and In-service)

Description of Location	Gauge Numbers	LRWB Pre-Service	LRWB In-Service
Deflection gauges at 18' east of mid span	D1	✓	✓
	D2	✓	✓
	D3	✓	✓
	D4	✓	✓
	D5	✓	✓
Deflection gauges at mid span	D11	✓	✓
	D12	✓	✓
	D13	✓	✓
	D14	✓	✓
	D15	✓	✓
Deflection gauges at the east end of the span	SD2	✓	✓
	D16/SD16	✓	✓
Deflection gauges at the west end of the span	SD5	✓	✓
	D17/SD17	✓	✓
Strain gauges at Section B: 9' west of mid span	F1	✓	✓
	F2	✓	✓
	F3	✓	✓
	F4	✓	✓
	F5	✓	✓
	F6	✓	✓
	F7	✓	✓
	F8	✓	✓
	F9	✓	✓
	F10	✓	✓
	F11	✓	✓
	F18	✓	✓
	F22	✓	✓
	F26	✓	✓

Table 2-15 (cont'd). Little River Westbound Gauges (Pre-service and In-service)

Description of Location	Gauge Number	LRWB Pre-Service	LRWB In-Service
Strain gauges at Section B: 9' west of mid span	F30	✓	✓
	F34	✓	✓
	F36	✓	✓
	F40	✓	✓
	F41	✓	✓
	F42	✓	✓
	F43	✓	✓
	F44	✓	✓
	F46	✓	✓
	F48	✓	✓
	F50	✓	✓
	F52	✓	✓
	F64	✓	✓
	F65	✓	✓
	F91	✓	✓
	F92	✓	✓
Strain gauges at Section C: 4'-6" west of mid span	F19	✓	✓
	F23	✓	✓
	F27	✓	✓
	F31	✓	✓
	F35	✓	✓
	F37	✓	✓
Transverse Strain gauges at Section B: 9' west of mid span	B7	✓	✓
	B9	✓	✓
	B11	✓	✓
	B13	✓	✓
Transverse Strain gauges at Section C: 4'-6" west of mid span	B8	✓	✓
	B10	✓	✓
	B12	✓	✓
	B14	✓	✓
		Total = 58	Total = 58

2.3 Load Test Data Preparation and Results

2.3.1 Data Preparation

This section explains how the data was prepared for plotting and analyses. First, recall that for each load test step, data was recorded at 1 Hz (one data point per second) for at least 60 seconds. For data analysis, the approximately 60 values were averaged for each gauge, and the average values were used.

As described previously, before each load position started (when there was no live load on the bridge), the gauges were zeroed. For the data analysis, the zeroed value was subtracted from gauge readings that followed the zeroed result, to eliminate any fluctuation in the gauge during the load test. Also, all gauges were zeroed after all load positions were completed for a particular block case, and they were zeroed again before the start of the next block case. These readings were checked to make sure there were no extreme fluctuations in the gauge readings. If a gauge showed abnormal readings, it was considered a faulty gauge and was omitted from the results. A faulty strain gauge could be attributed to debonding from the concrete surface. A faulty wire deflection gauge could be due to unintended settlement of the anchor in the ground.

On the bottom flange of the FIBs, there were two (2) strain gauges placed side by side (separated by 2 in., 1 in. off the centerline of the girder) for each gauge line. For analysis, the two gauges were compared to make sure they showed equivalent results, and if the results were similar, they were averaged for the data analysis. If they were not similar, the zeroed readings were checked again to make sure the gauge had not failed during the load test.

The deflection gauges (such as gauges D16 and D17) at the ends of the span were to measure any displacement at the abutments or bents. However, the recorded values were insignificant and therefore were not included in the data analysis.

2.3.2 Presentation of Test Results

In subsequent chapters, the results of the in-service tests are presented and discussed by bridge type, beginning with the prestressed deck panel bridges, followed by the non-prestressed deck panel bridges. Later, comparisons between the pre-service and in-service tests are made. The presented data includes the longitudinal strains plotted along the depth of the composite girder's cross section, deflection data at mid span, longitudinal strain data at section B, and transverse strain data at sections B and C.

CHAPTER 3
PRE-SERVICE TEST RESULTS

3.1 Pre-Service Test Data For Prestressed Concrete Deck Panel Bridges

3.1.1 Introduction

This section presents the pre-service test data for HCWB and LRWB bridges. These bridges were constructed with **prestressed** concrete deck panels. The data analysis was done for load positions 1A and 2 for the deflection and longitudinal strain data, and for load positions 1A, 1B, 3A and 3B for the transverse strain data. The results are presented either at mid span, section B, or section C. As discussed previously, section B is located 9'-0" west of mid span, and section C is located 4'-6" west of mid span.

3.1.2 Composite Action

To determine if composite action was achieved between the panels and the FIBs, the longitudinal strains in FIB5, deck panel and barrier were plotted relative to the gauge's vertical location.

Figures 3-1 and 3-2 show the strains before the barriers were added on HCWB for load positions 1A and 2, for all block cases. As the number of blocks increased, so did the strain at the bottom of the girder; likewise, the strain on the top of the deck increased negatively. The plotted strains vary linearly from the top of the deck panel to the bottom of the FIB. It appears that composite action was achieved between the panels and FIBs.

Figures 3-3 and 3-4 show the strains on HCWB after the barriers were added, with very similar results as before the barriers. There is still a linear relationship between strains and girder depth. This relationship is more evident for load position 2, where the trucks were shifted close to the barrier to maximize load on FIB5. As expected, the strains were greater for load position 2 than for 1, because the trucks were positioned closer to the barriers in the former. In addition, the strains transition from negative to positive at approximately the same vertical locations relative to the girder bottom for both load positions 1 and 2. This transition indicates the neutral axis for the composite girder.

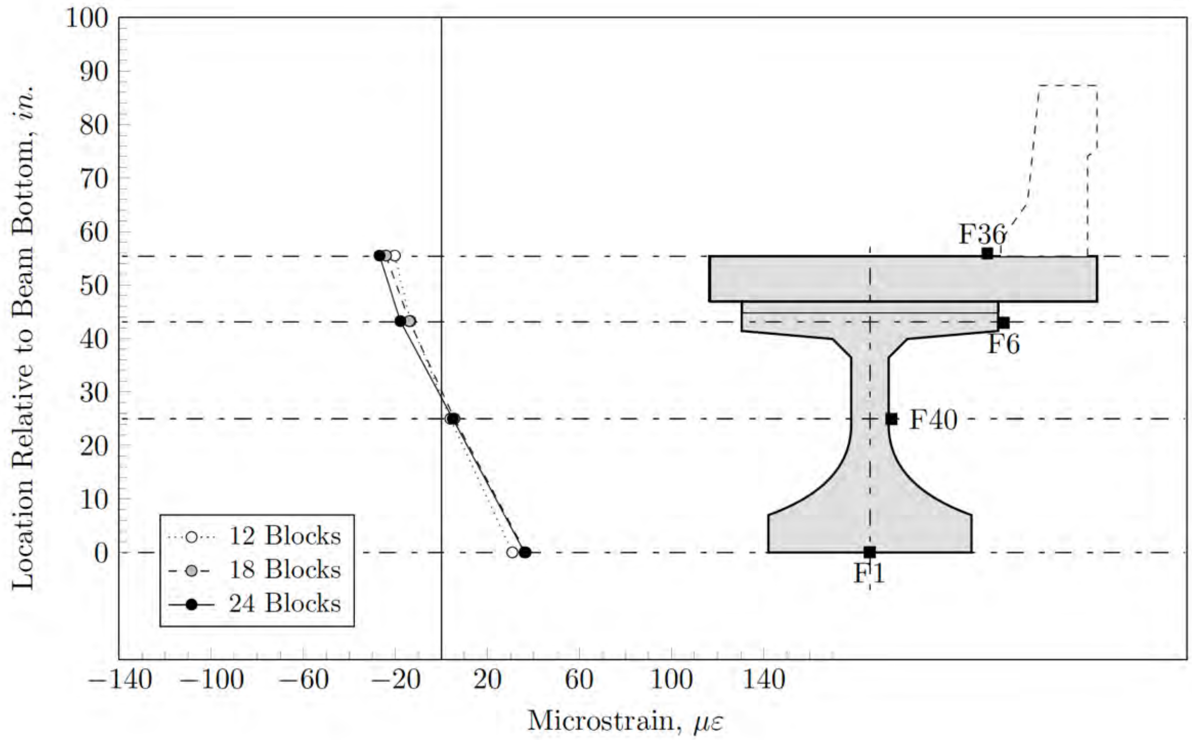


Fig. 3-1. Longitudinal Strain - HCWB Before Barrier - FIB5 - LP 1A

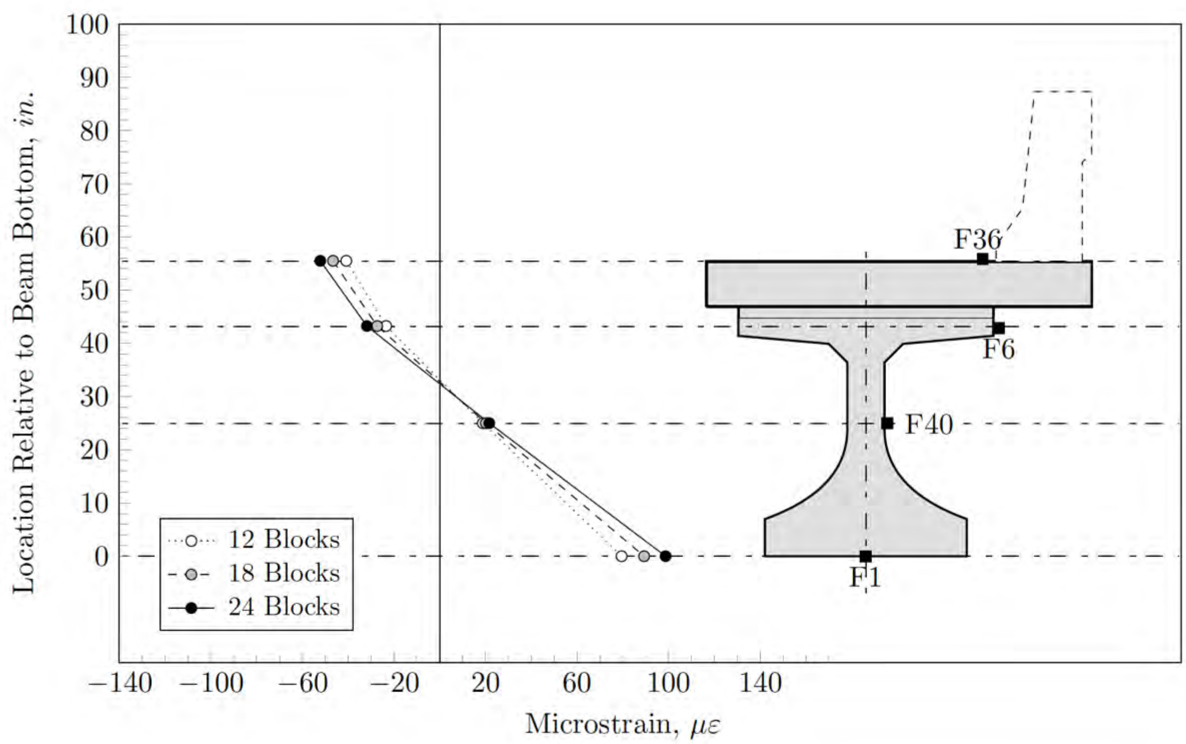


Fig. 3-2. Longitudinal Strain - HCWB Before Barrier - FIB5 - LP 2

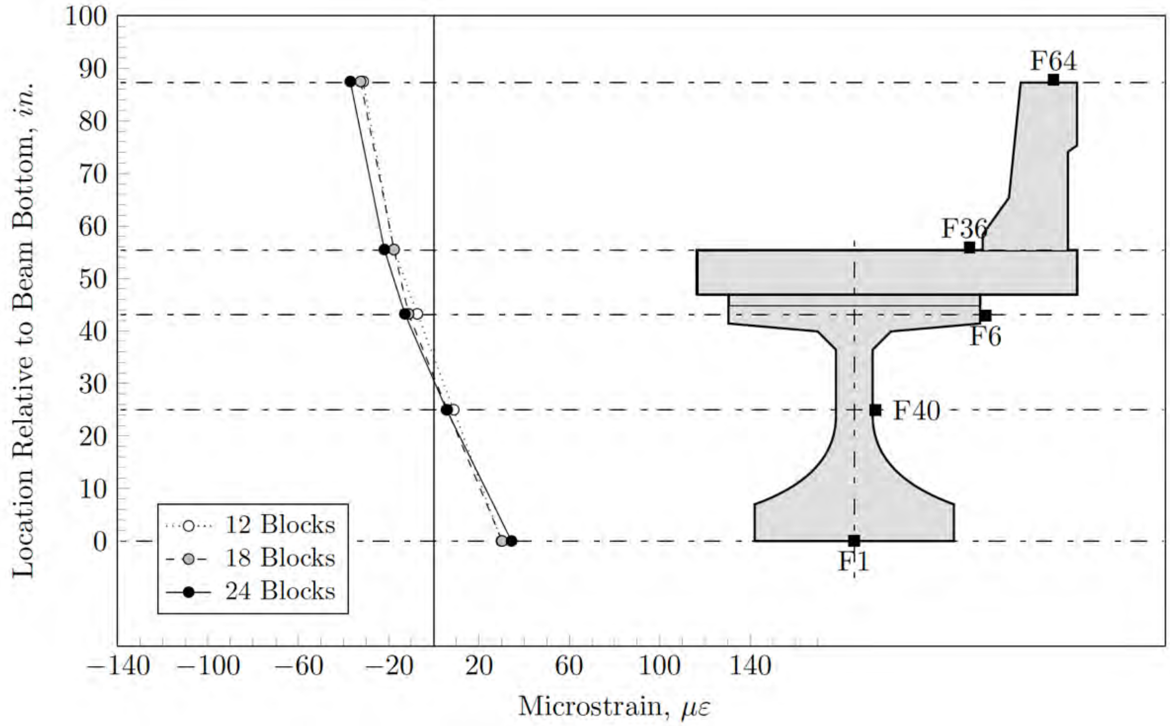


Fig. 3-3. Longitudinal Strain - HCWB - FIB5 - LP 1A

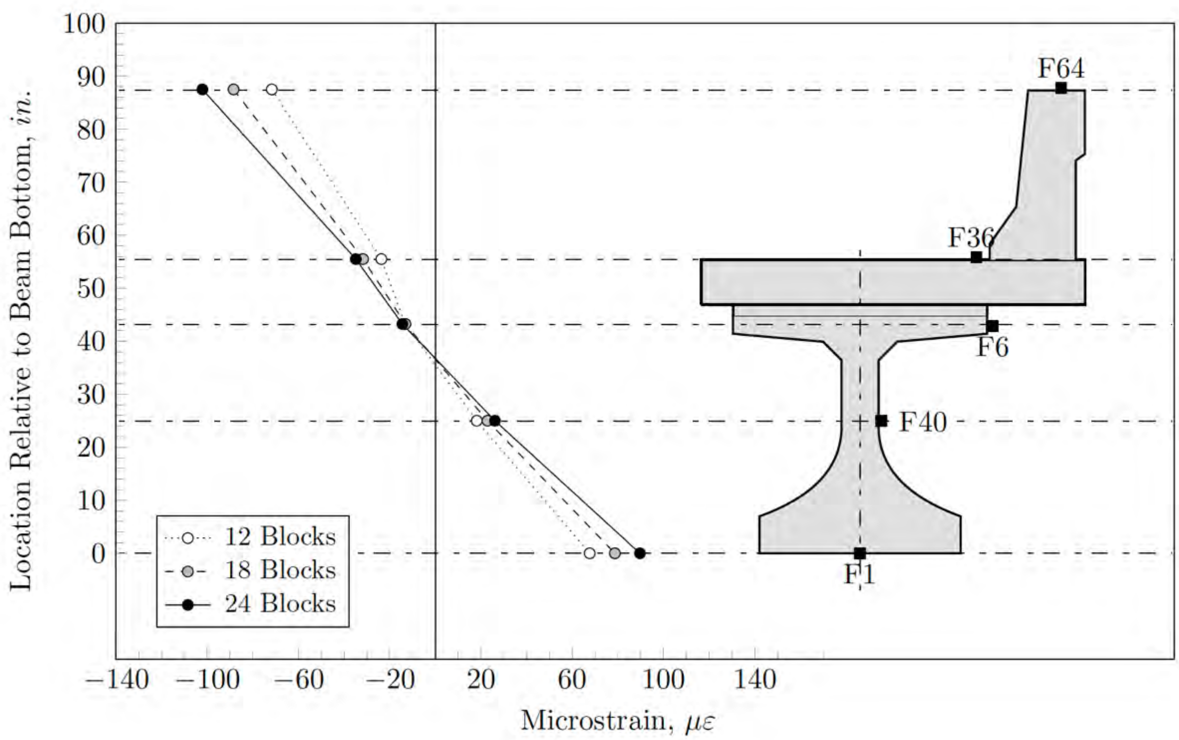


Fig. 3-4. Longitudinal Strain - HCWB - FIB5 - LP 2

Figures 3-5 and 3-6 show the LRWB load test strains. Again, these both show a linear relationship between strains and girder depth. The neutral axis locations are also similar, except that there is more variability for load position 2 when the number of blocks changes.

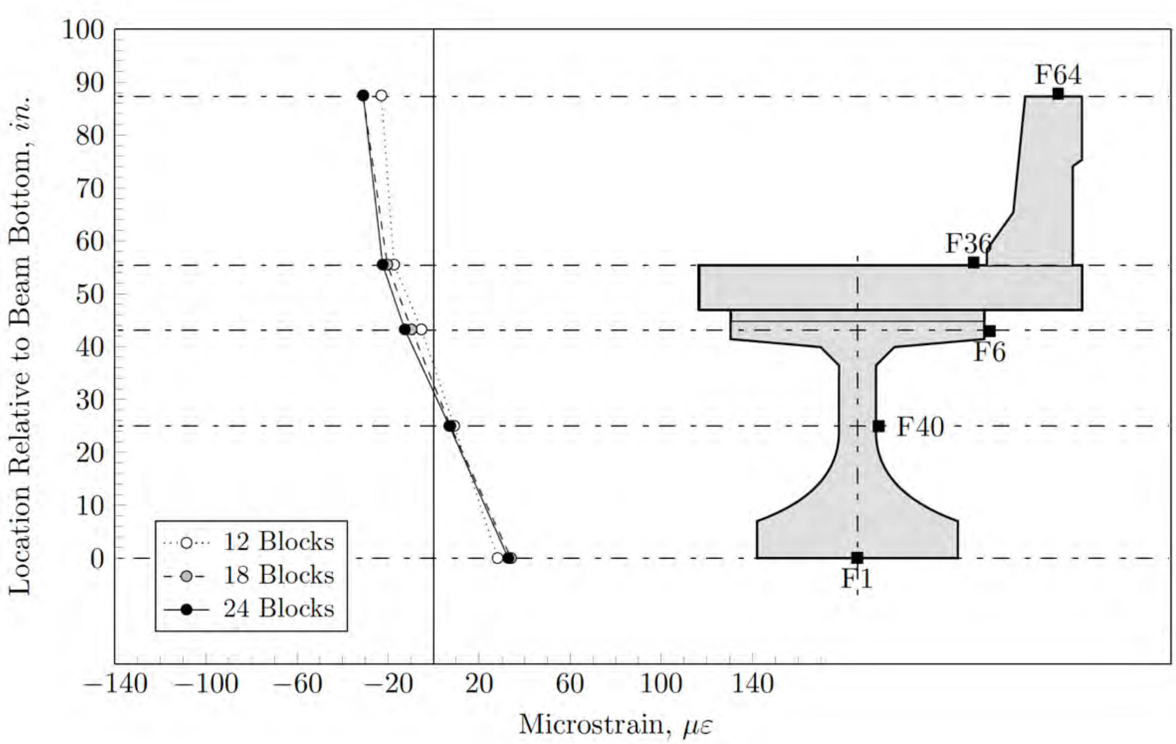


Fig. 3-5. Longitudinal Strain - LRWB - FIB5 - LP 1A

3.1.3 Deflection Results

Figures 3-7 to 3-9 show the deflections on HCWB and LRWB for load position 1A, with Truck 1 and Truck 2 on the bridge individually and together. This position loads the bridge symmetrically about its centerline.

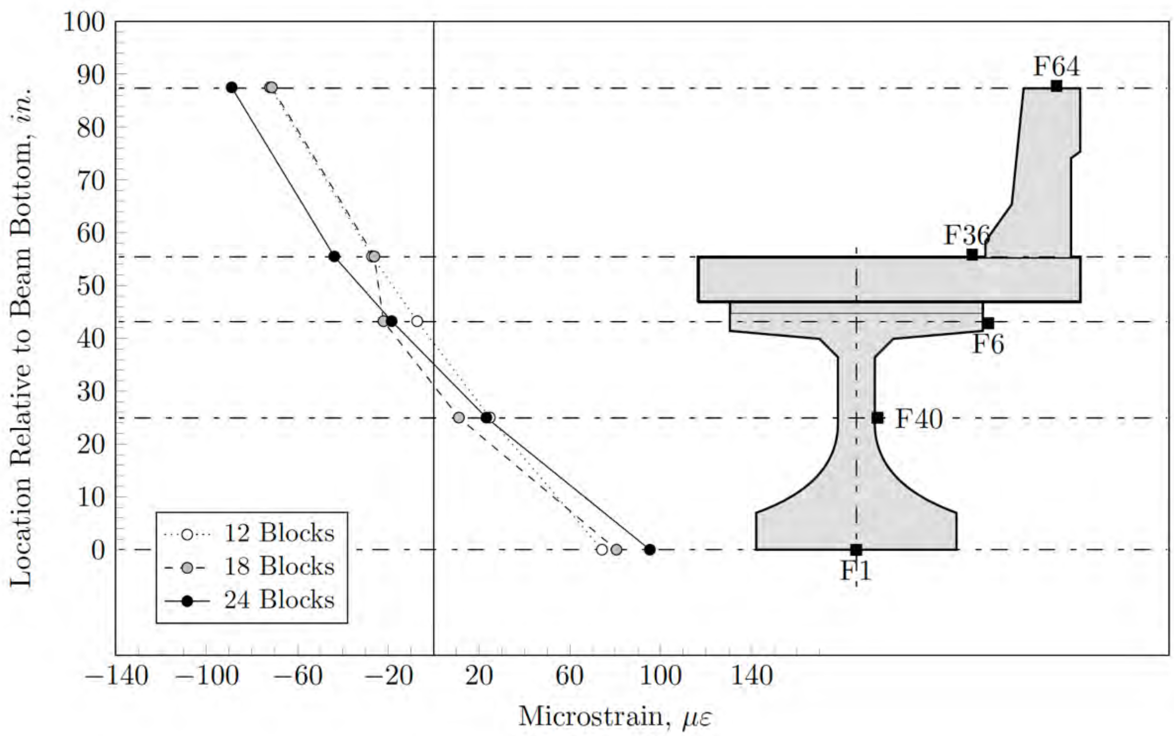


Fig. 3-6. Longitudinal Strain - LRWB - FIB5 - LP 2

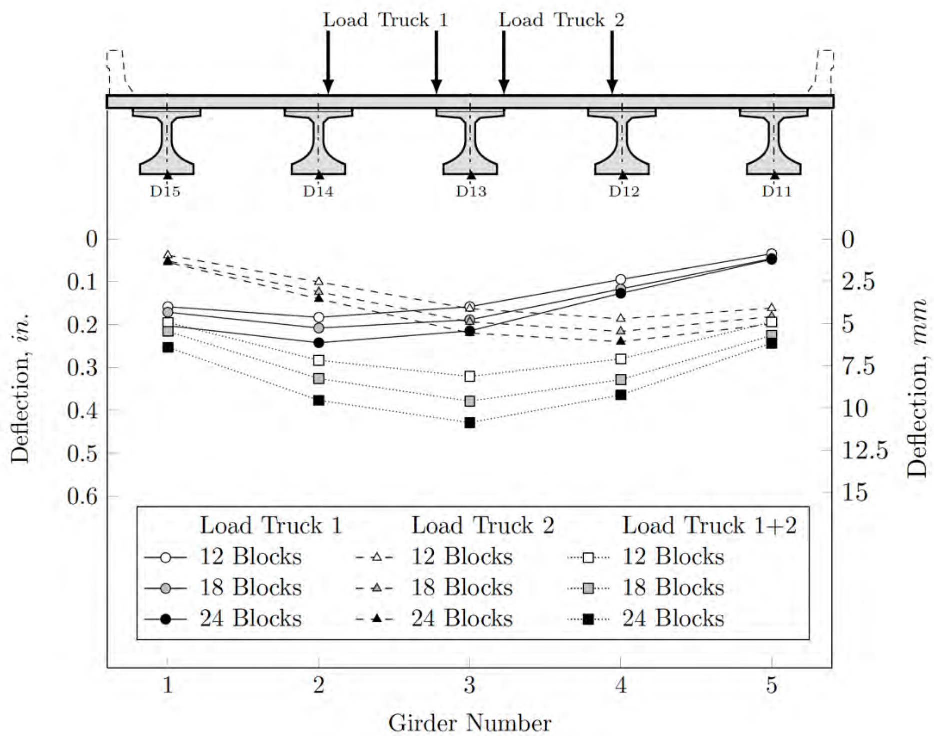


Fig. 3-7. Deflection - HCWB Before Barrier - Mid Span - LP 1A

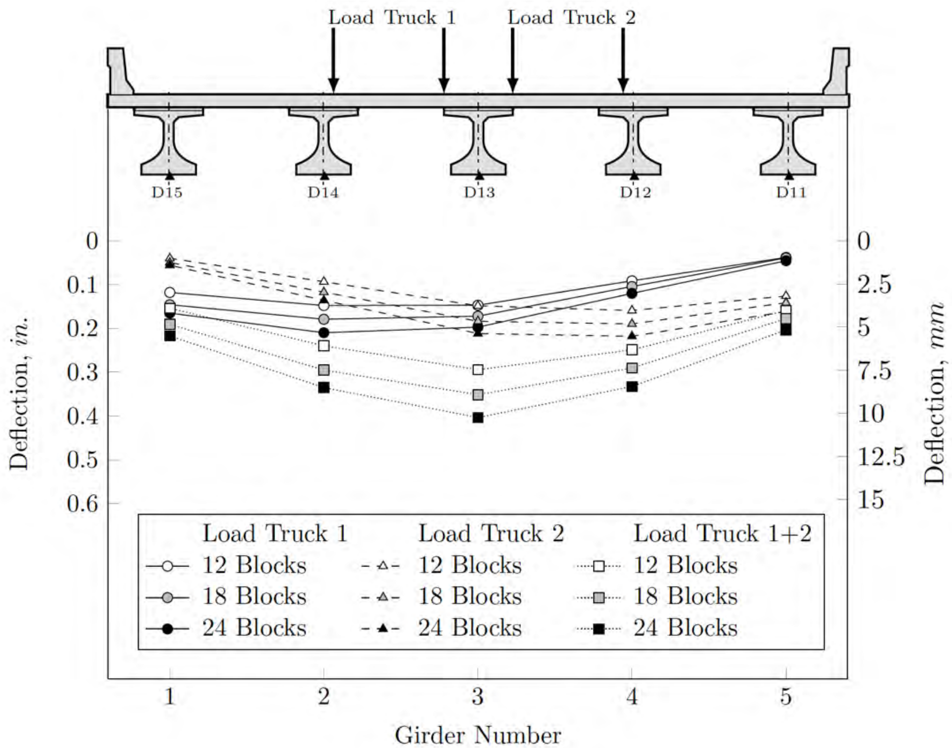


Fig. 3-8. Deflection - HCWB - Mid Span - LP 1A

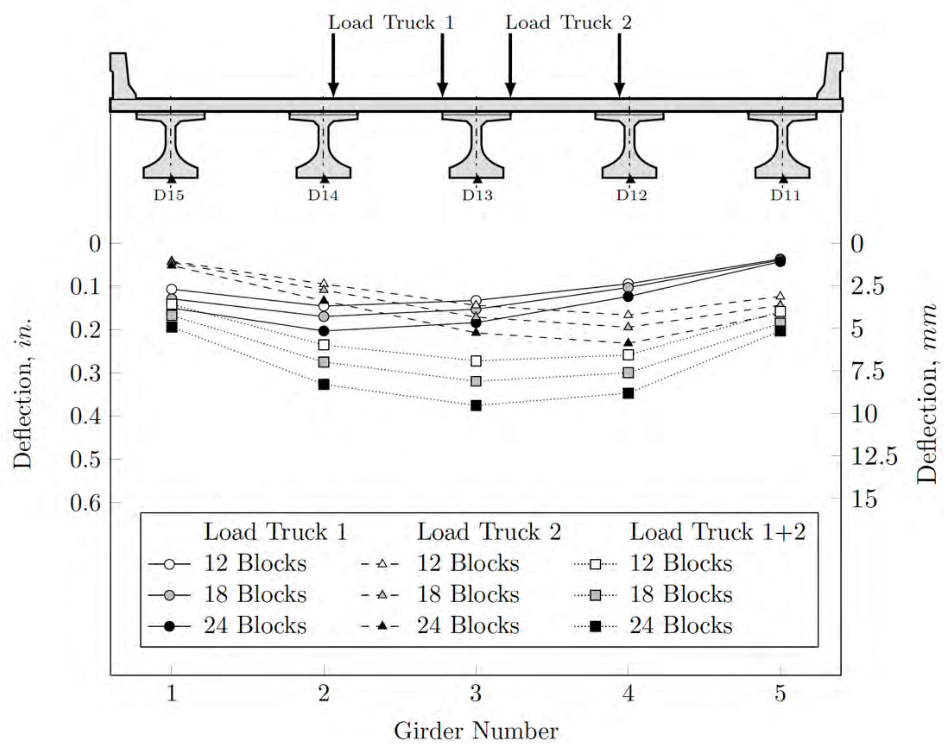


Fig. 3-9. Deflection - LRWB - Mid Span - LP 1A

The overall behavior was as expected. When truck 1 was on the bridge, the north side (left on the graph) exhibited the most deflection; likewise, the south side (right on the graph) deflected more when truck 2 was on the bridge. Superposition of truck 1 deflection and truck 2 deflection approximately equals the deflection due to both trucks being loaded on the bridge. This was true for all three (3) block cases that were tested. For the 24 block cases for HCWB before the barrier, the mid span deflection of FIB3 was 0.43"; for FIB1 and FIB5, the deflections were 0.25" and 0.24", respectively. After the barriers were added on HCWB, the deflections were very similar: FIB1 was 0.22", FIB5 was 0.20", and FIB3 was 0.40". LRWB exhibited similar deflections compared to HCWB bridges, with deflections of 0.19" for FIB1, 0.20" for FIB5, and 0.38" for FIB3. The difference among the three load tests is within 5 hundredths of an inch.

Figures 3-10 to 3-12 show the deflections of the bridge when the trucks were at load position 2. The purpose of this loading was to maximize deflections of the exterior girder, FIB5.

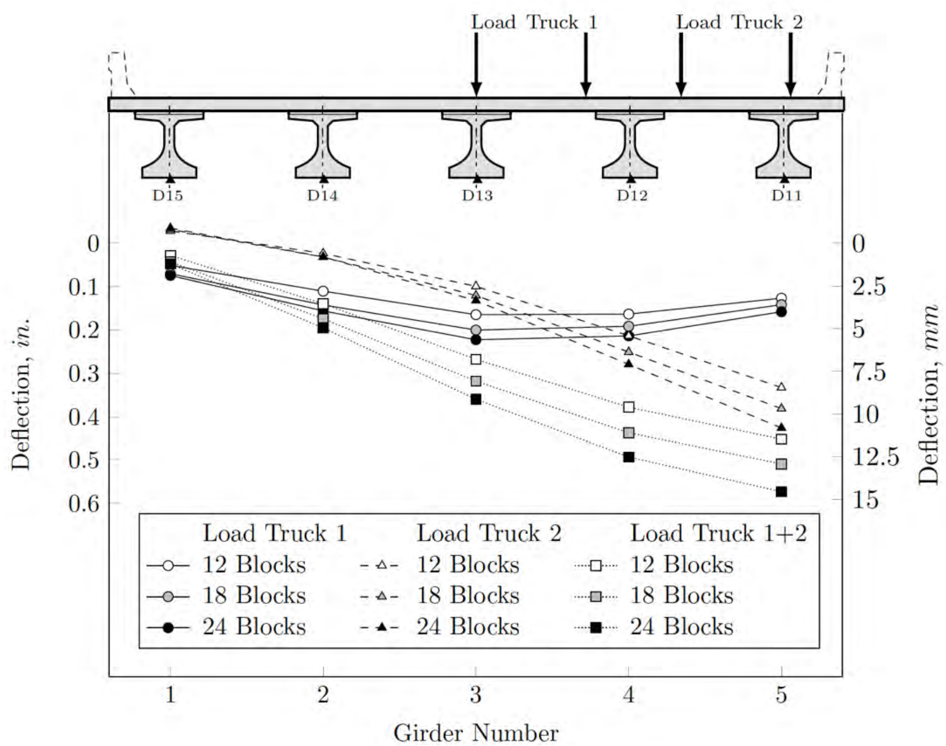


Fig. 3-10. Deflection - HCWB Before Barrier - Mid Span - LP 2

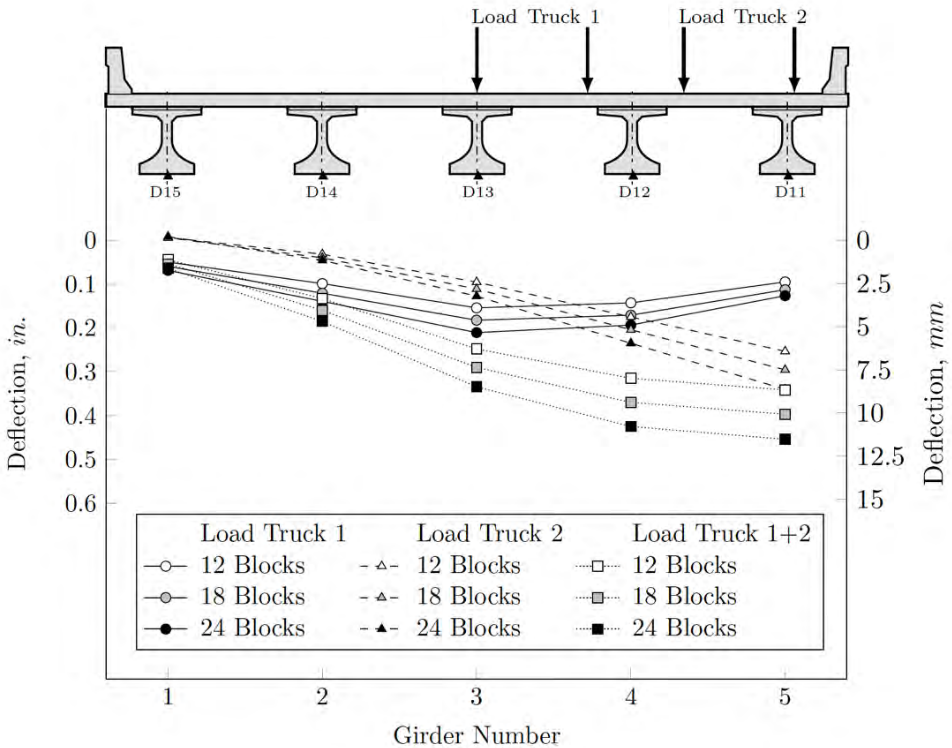


Fig. 3-11. Deflection - HCWB - Mid Span - LP 2

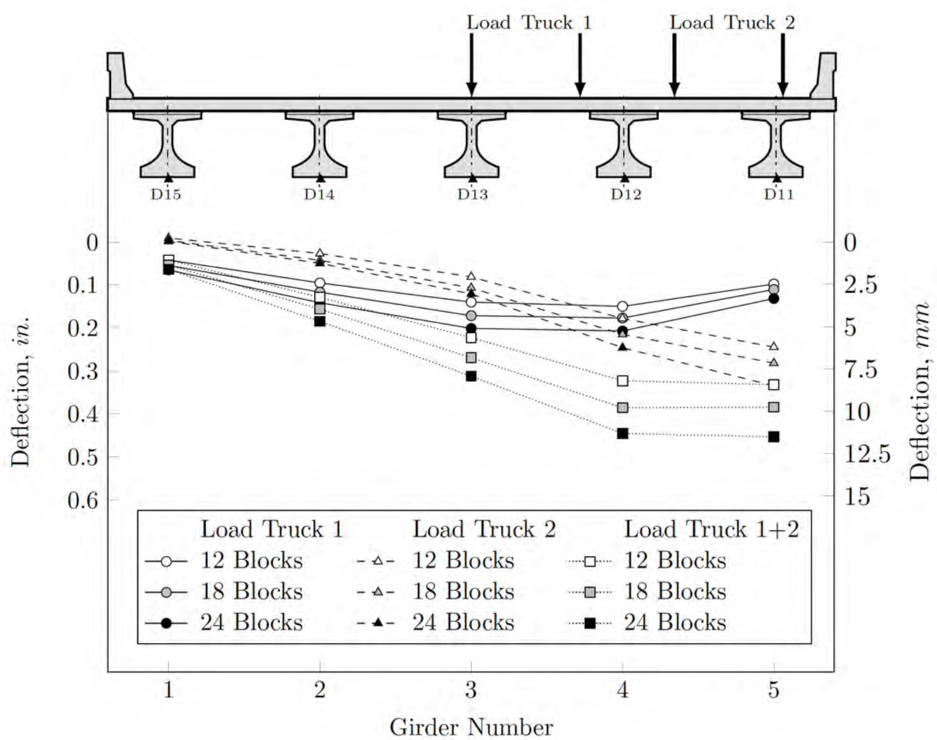


Fig. 3-12. Deflection - LRWB - Mid Span - LP 2

When 24 blocks were loaded on the two (2) trucks on HCWB before the barriers, FIB5 deflected 0.57", which was more than the 0.45" for both HCWB and LRWB after the barriers. When only truck 2 was on the bridge, FIB1 for all block cases lifted slightly, which is denoted by negative values. Figures 3-11 and 3-12 are very similar, meaning that HCWB and LRWB behaved the same.

3.1.4 Longitudinal Strain Results

The longitudinal strains were measured on the bottom flanges of the girders at section B. Two (2) gauges were placed side by side on each girder, an inch off the girder's centerline. The data presented here are averages of the two gauges on each girder except where noted.

Figures 3-13 to 3-15 show data for HCWB and LRWB loaded with truck 1, truck 2, and both trucks at load position 1A. This load position maximizes load to FIB3. From Figure 3-13, for HCWB, the maximum longitudinal strain occurred on FIB3, 82 $\mu\varepsilon$, when both trucks were loaded with 24 blocks. Loading HCWB after the barriers resulted in a maximum strain of 85 $\mu\varepsilon$ (Figure 3-14), and LRWB had 87 $\mu\varepsilon$ (Figure 3-15).

Load position 2 places the trucks on one side of the bridge, to maximize load on FIB5. From the test on HCWB before the barrier, the maximum strain on FIB5 was 104 $\mu\varepsilon$, HCWB after the barrier was 92 $\mu\varepsilon$, and LRWB was 91 $\mu\varepsilon$, as shown in **Figures 3-16 to 3-18**, respectively.

3.1.5 Transverse Strain Results

Transverse strains were measured for HCWB and LRWB after the barriers were placed. (They were not measured on HCWB before barriers were added.) The strain gauges were placed in between the girders and on the underside of the bridge deck panels (at Section B) and on the underside of the joint between panels (at Section C). Data is presented below for load positions 1A, 1B, 3A and 3B for both bridges at sections B and C.

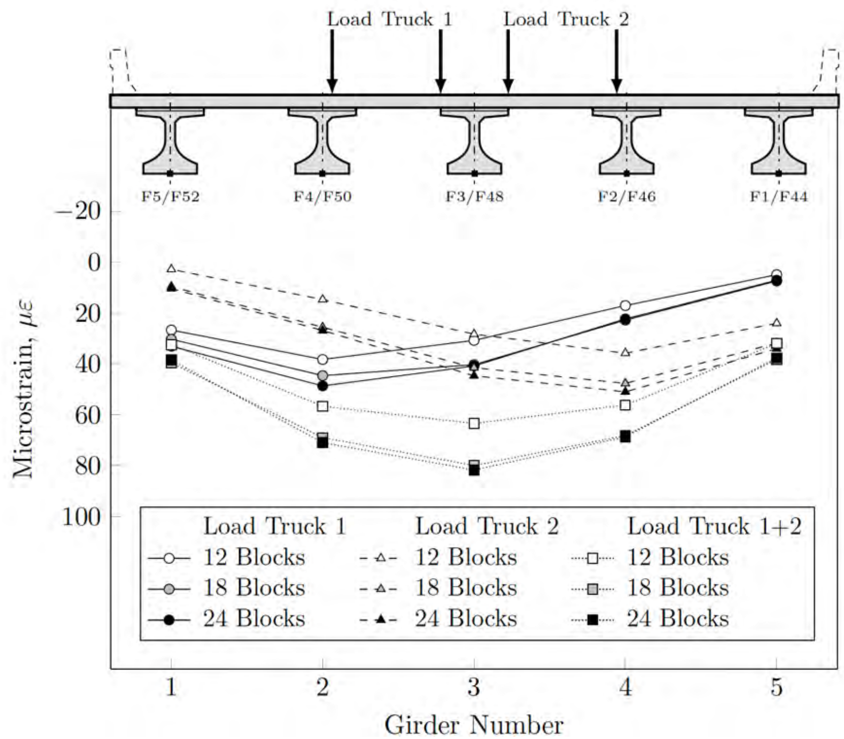


Fig. 3-13. Longitudinal Strain - HCWB - Section B - LP 1A - Before Barrier

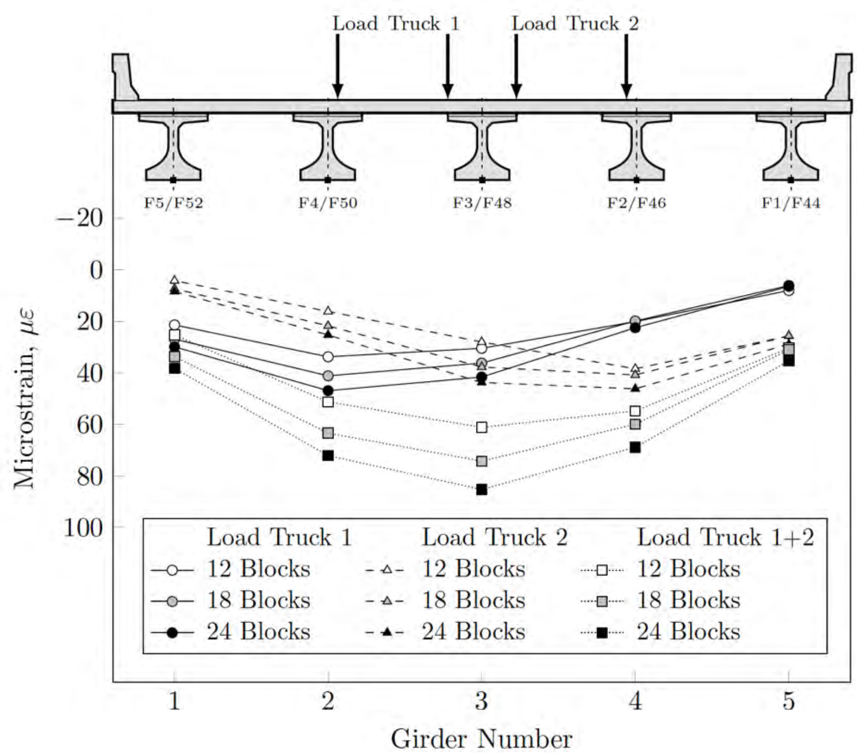


Fig. 3-14. Longitudinal Strain - HCWB - Section B - LP 1A

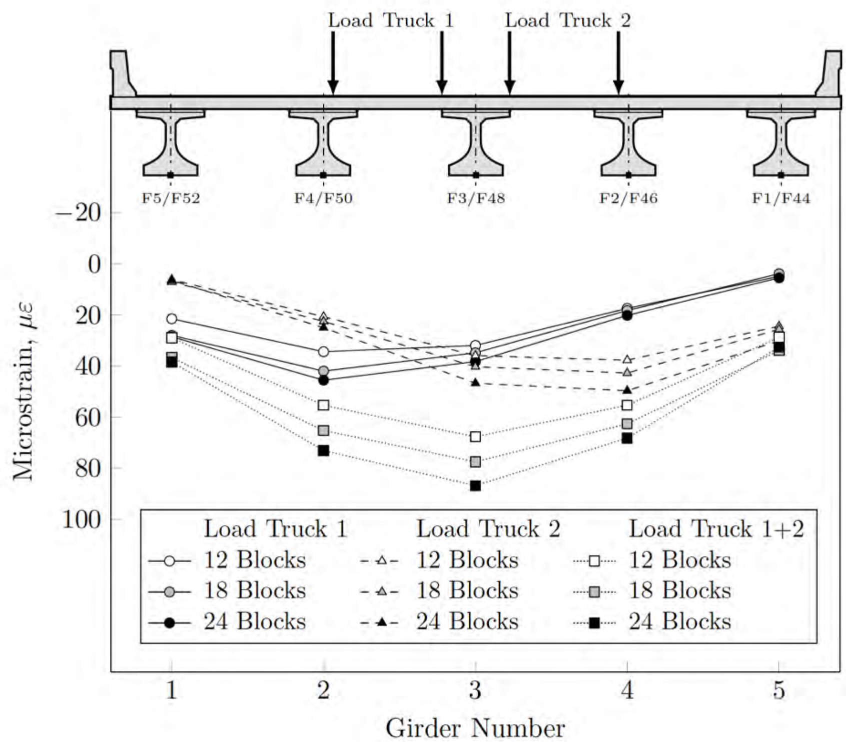


Fig. 3-15. Longitudinal Strain - LRWB - Section B - LP 1A

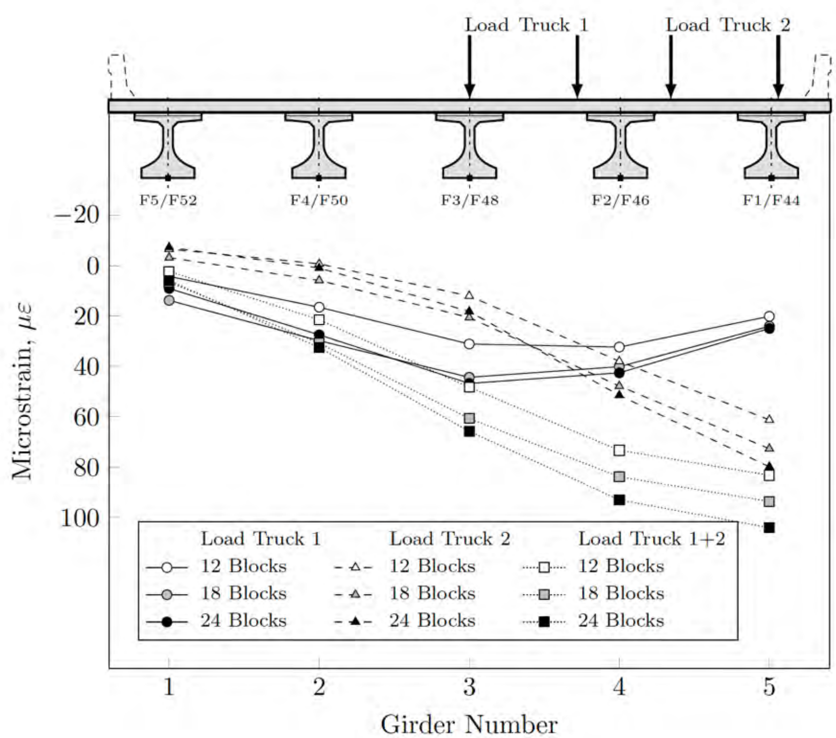


Fig. 3-16. Longitudinal Strain - HCWB - Section B - LP 2 - Before Barrier

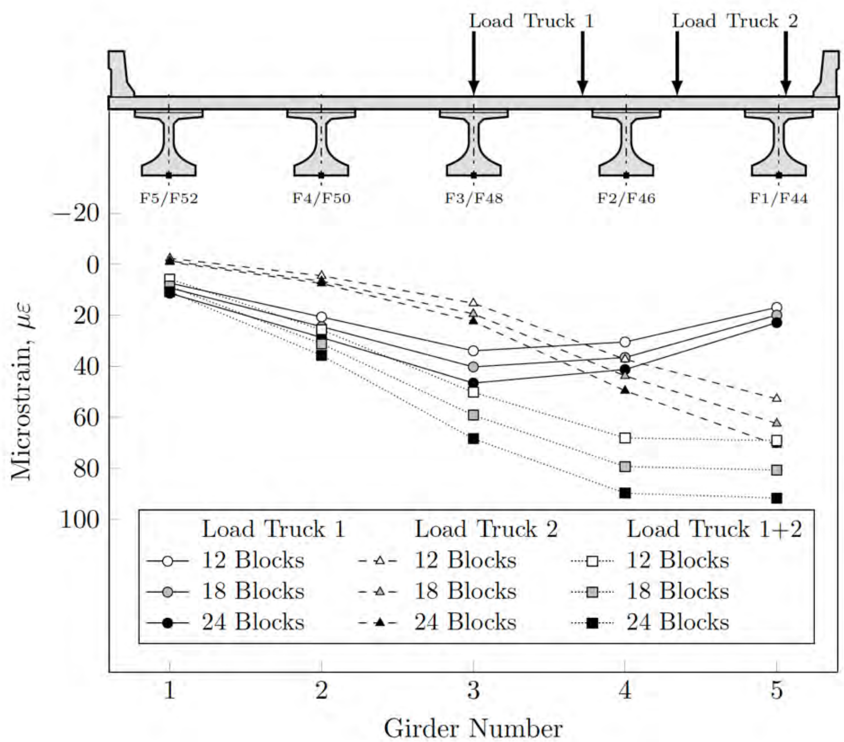


Fig. 3-17. Longitudinal Strain - HCWB - Section B - LP 2

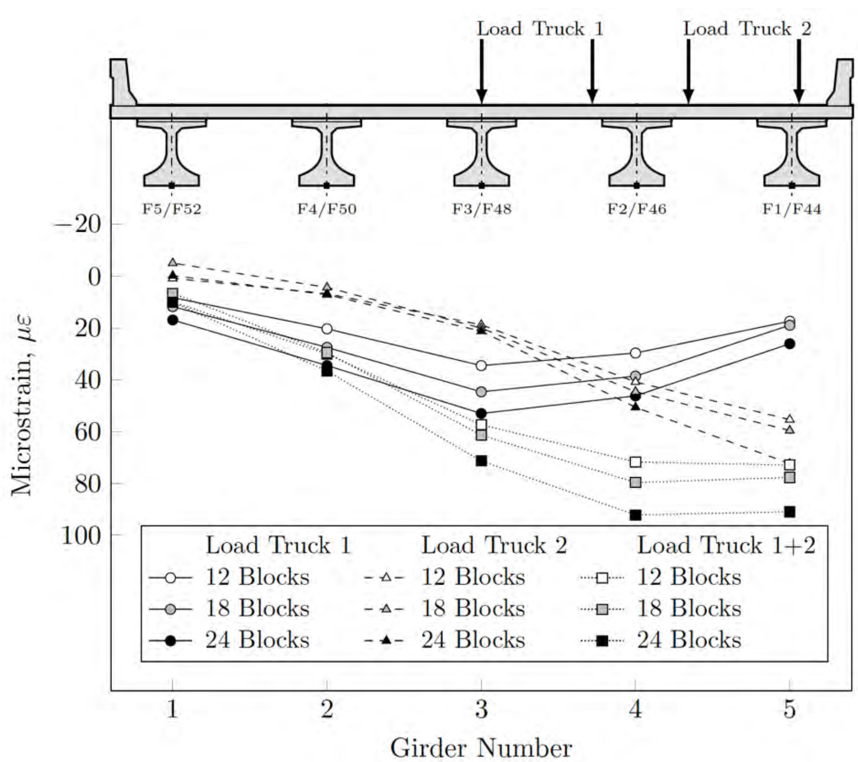


Fig. 3-18. Longitudinal Strain - LRWB - Section B - LP 2

As discussed previously, for load positions 1A and 3A, axle 3 of the truck was placed 6'-8" from mid span, and for load positions 1B and 3B, axle 3 was placed 17'-3.5" from mid span. When the trucks were placed at the 1A and 3A load positions, axles 2 and 3 were straddling section B. When the trucks were placed at the 1B and 3B load positions, axles 3 and 4 were straddling sections B and C, to load these sections equally, transversely.

Load Positions 1A and 1B: **Figure 3-19** shows the transverse strains for HCWB, load position 1A, section B. The bridge behaved symmetrically, as expected. The maximum transverse strains were at gauges B11 and B9, with values of 62 με and 63 με, respectively. This occurred when both trucks were loaded with 24 blocks. When a single truck was placed on the bridge, the transverse strains were greater at the location of the truck compared to the other gauges (i.e., strain in gauge B11 was highest for Load Truck 1, and gauge B9 was highest for Load Truck 2).

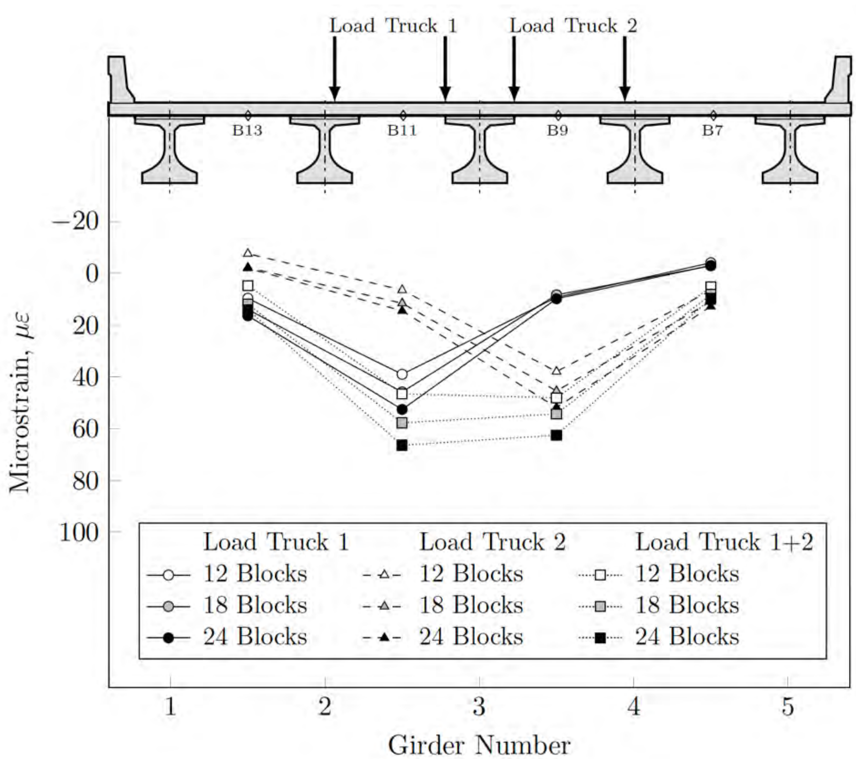


Fig. 3-19. Transverse Strain - HCWB - Section B - LP 1A

Figure 3-20 shows the transverse strain for HCWB, load position 1A, section C. The maximum transverse strains were at gauges B10 and B8, with values of 80 με and 65 με, respectively, when both trucks were loaded with 24 blocks. It was expected that B10 and B8 strains would be the same for this load condition, but they were not.

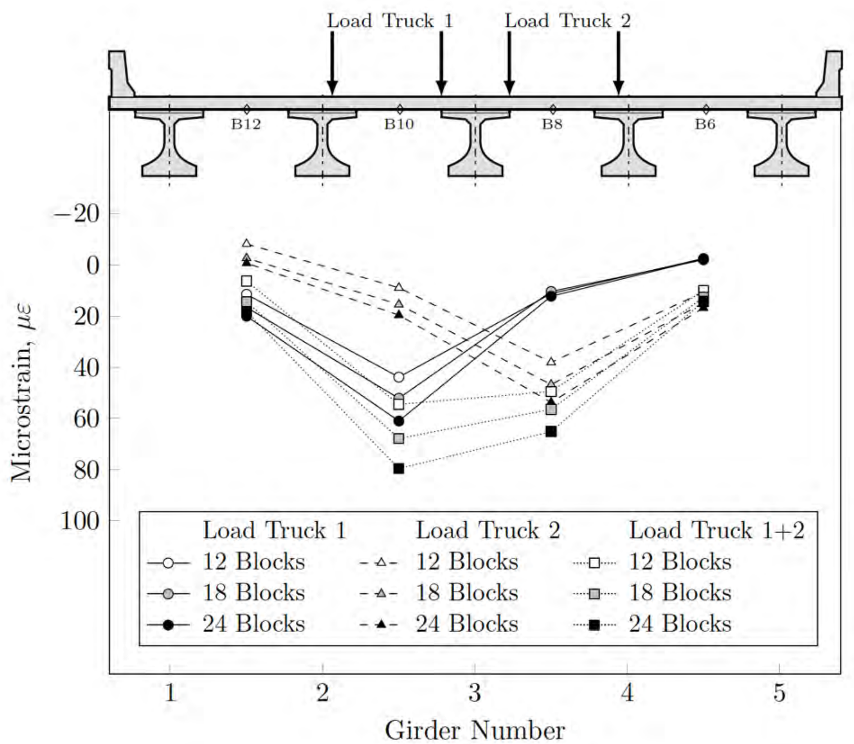


Fig. 3-20. Transverse Strain - HCWB - Section C - LP 1A

For load position 1B, the trucks were longitudinally more westward on the bridge than for load position 1A, but they still loaded the bridge symmetrically about the centerline. **Figure 3-21** shows symmetrical behavior, with maximums occurring at gauges B11 (58 με) and B9 (58 με) on the panels. These strains are less than those on the closure joints at section C, as shown in **Figure 3-22**.

HCWB load position 1B, section C, data is shown in Figure 3-22. There, the maximum strains occurred at gauges B10 and B8, with values slightly higher at gauge B10. The maximum value at B10 was 71 με whereas at B8 it was 61 με.

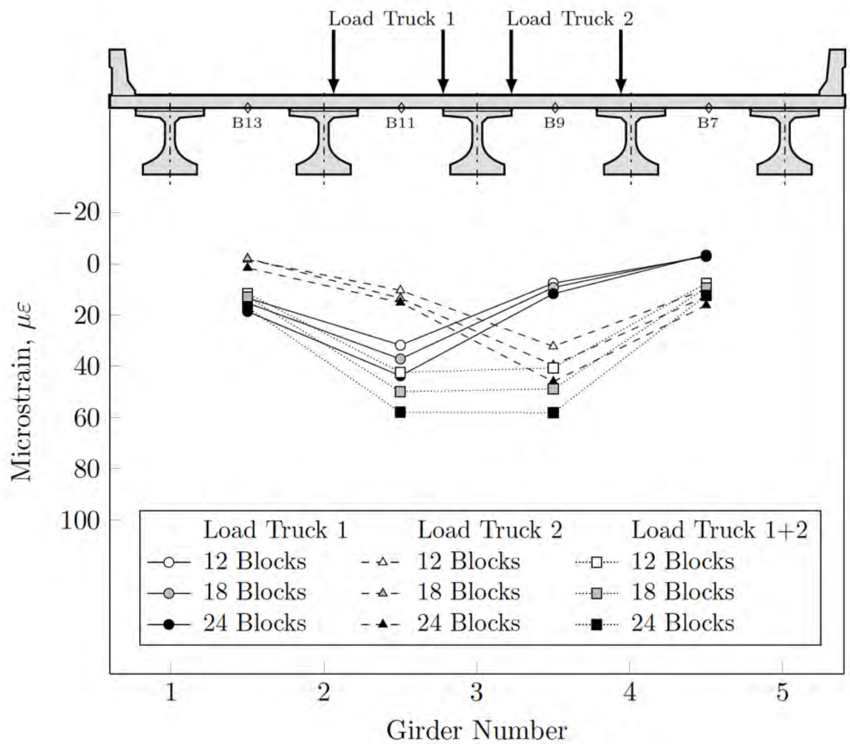


Fig. 3-21. Transverse Strain - HCWB - Section B - LP 1B

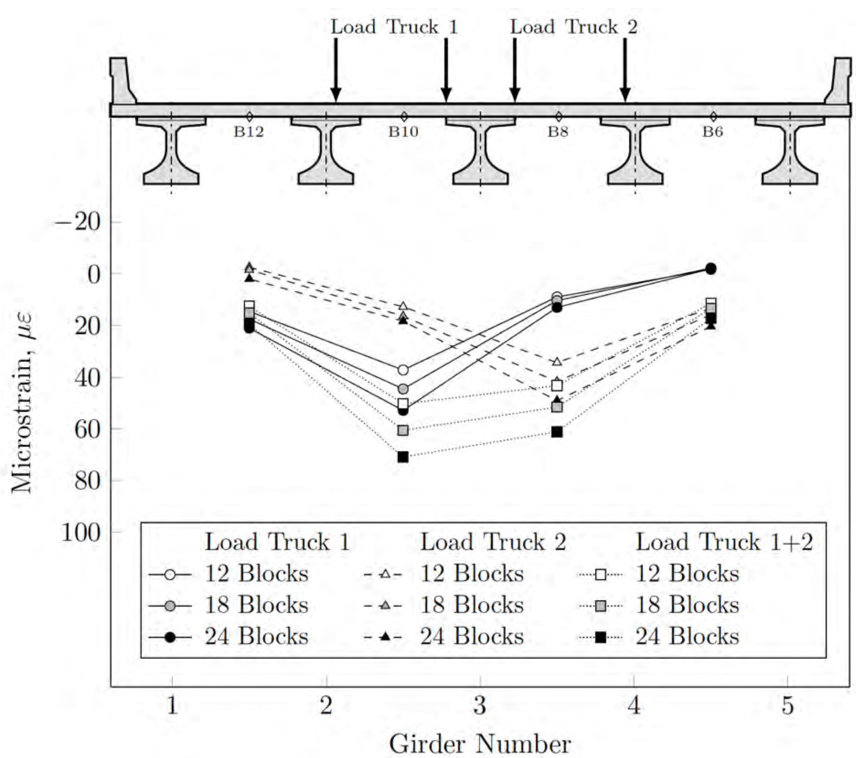


Fig. 3-22. Transverse Strain - HCWB - Section C - LP 1B

Figures 3-23 to 3-26 show the transverse strains for LRWB for load positions 1A and 1B at sections B and C. As shown in Figure 3-23, LRWB load position 1A section B resulted in much larger strain at gauge B11 compared to B9, with maximum values of 91 με and 69 με, respectively. Gauge B11 showed higher results for all the test steps. LRWB load position 1A section C showed maximum strains at B10 (76 με) and B8 (68 με), as shown in Figure 3-24. The values were expected to be equal, but B11 was higher than B9, and B10 was higher than B8. For load position 1B, the maximum transverse strains produced at each section were closer to being equal, but still had a small difference – 70 and 63 με in gauges B11 and B9, and 65 and 64 με in gauges B10 and B8 (Figures 3-25 and 3-26).

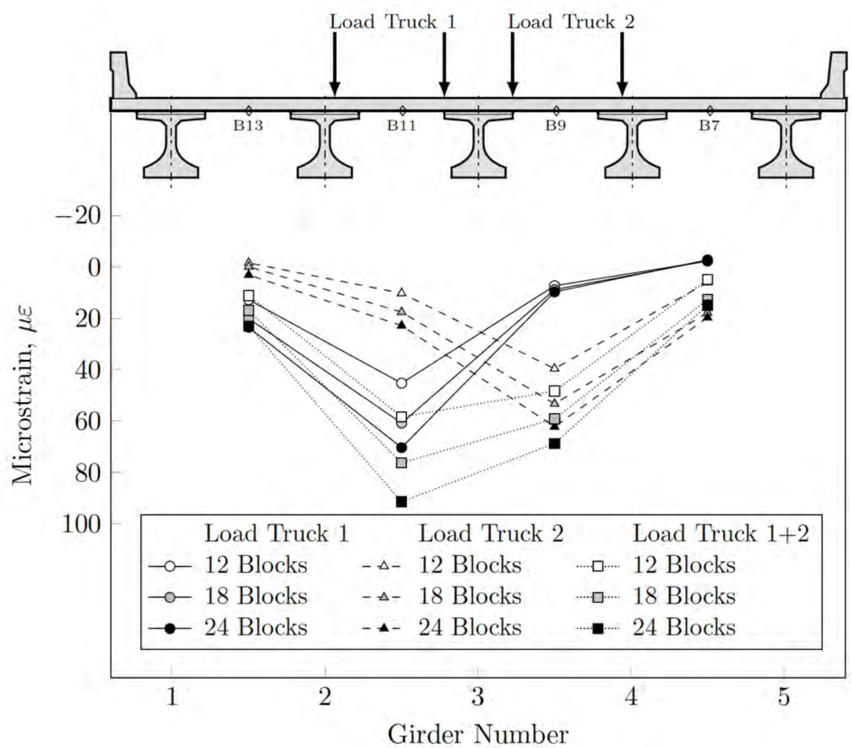


Fig. 3-23. Transverse Strain - LRWB - Section B - LP 1A

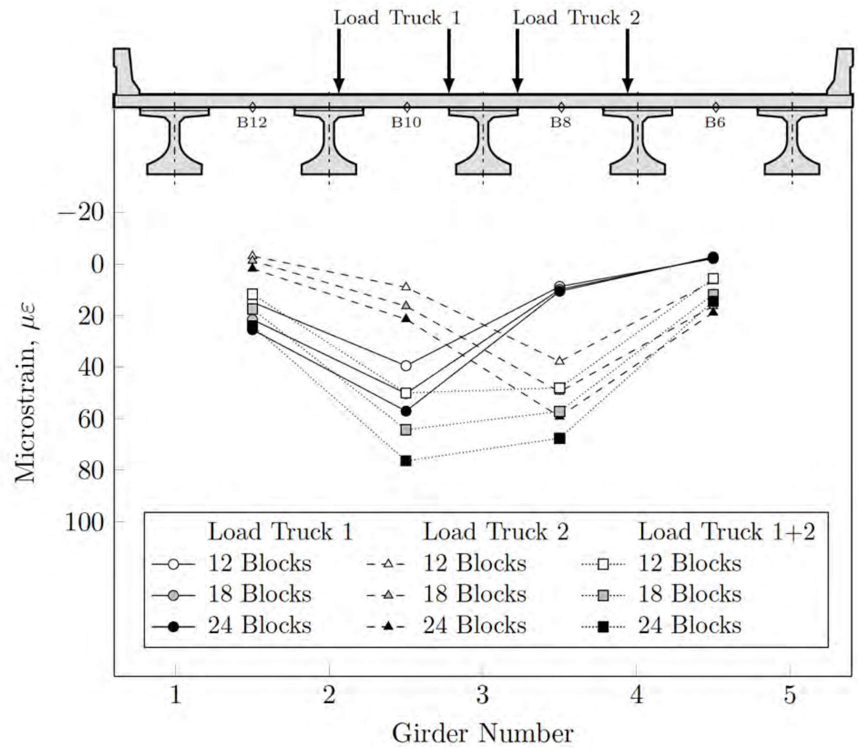


Fig. 3-24. Transverse Strain - LRWB - Section C - LP 1A

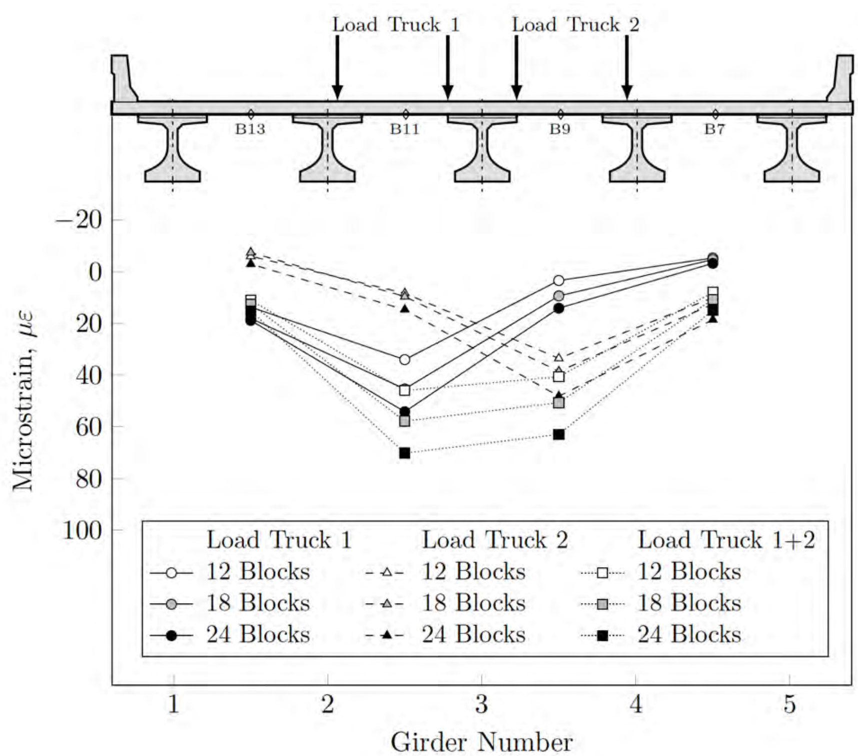


Fig. 3-25. Transverse Strain - LRWB - Section B - LP 1B

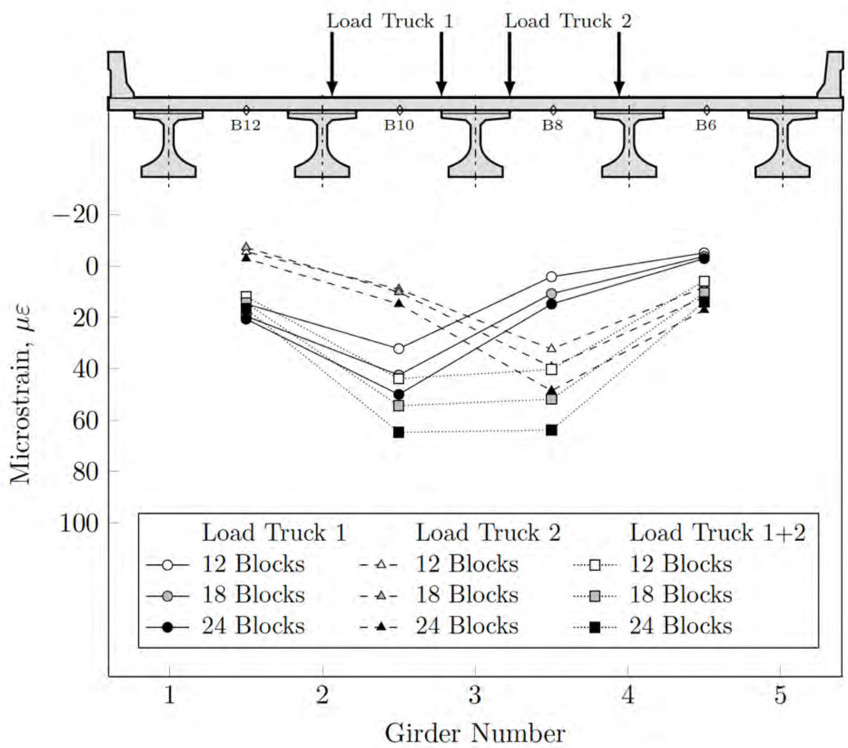


Fig. 3-26. Transverse Strain - LRWB - Section C - LP 1B

Load Positions 3A and 3B: Load position 3 is an asymmetrical load case, to further evaluate the transverse behavior of the panels and joints. **Figures 3-27 to 3-30** show the results for HCWB for load positions 3A and 3B at sections B and C. The maximum transverse strains occur in between FIB3 and FIB4. The maximum transverse strain for load position 3A section B was 74 με at gauge B9 and at section C was 73 με at gauge B8. Load position 3B resulted in smaller strains with section B equal to 60 με and section C equal to 64 με.

LRWB had some unique results where the maximum transverse strains would be expected at load position 3A. The results showed a strain reduction in gauges B8 and B9 once 24 blocks were loaded on the bridge (**Figures 3-31 and 3-32**). So in this case the maximum transverse strains occurred when 18 blocks were loaded and were at gauge B8 (73 με) and B9 (67 με).

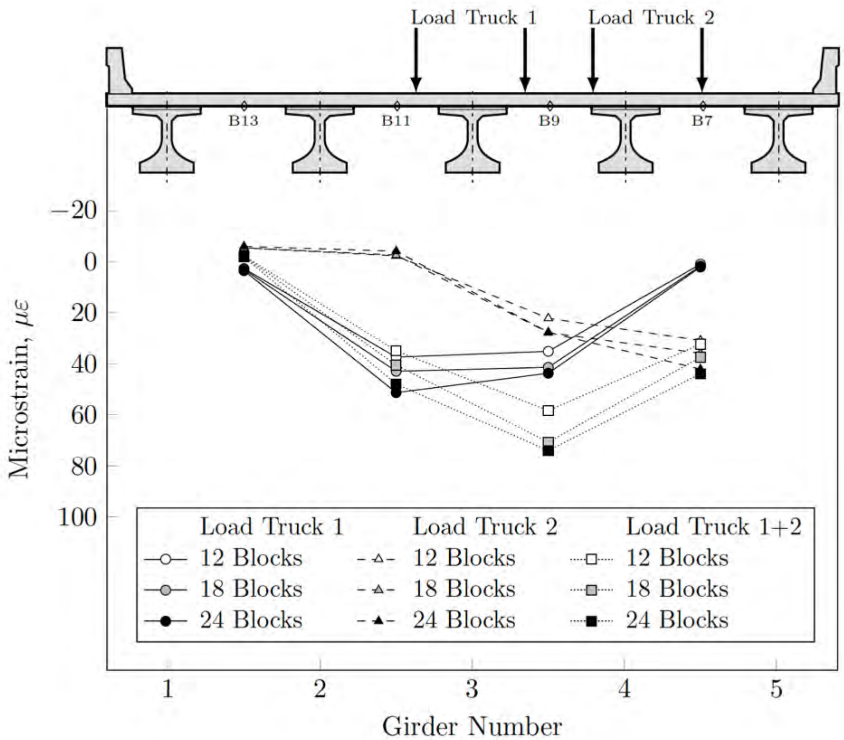


Fig. 3-27. Transverse Strain - HCWB - Section B - LP 3A

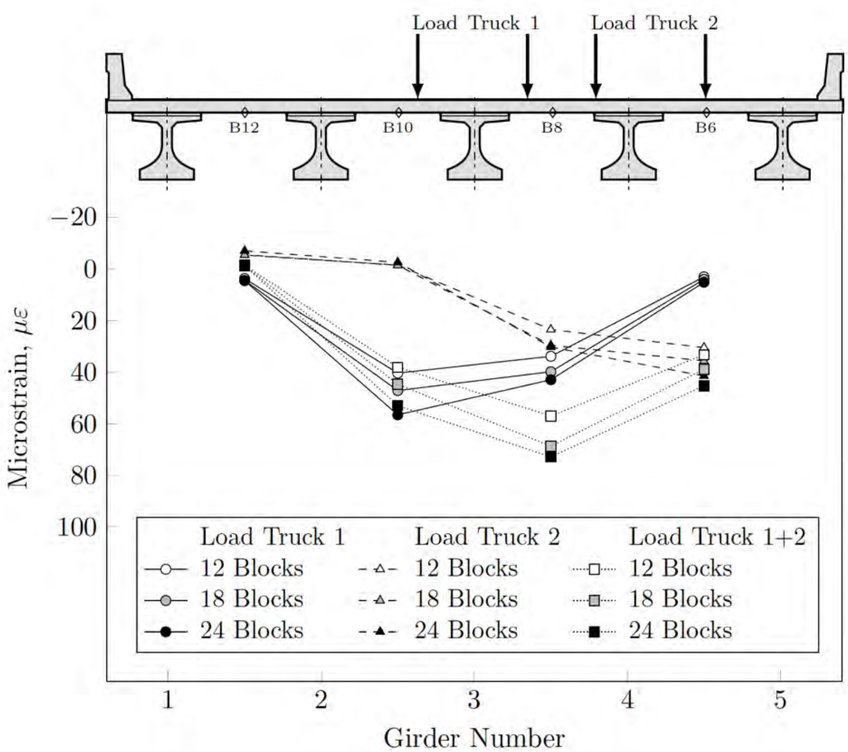


Fig. 3-28. Transverse Strain - HCWB - Section C - LP 3A

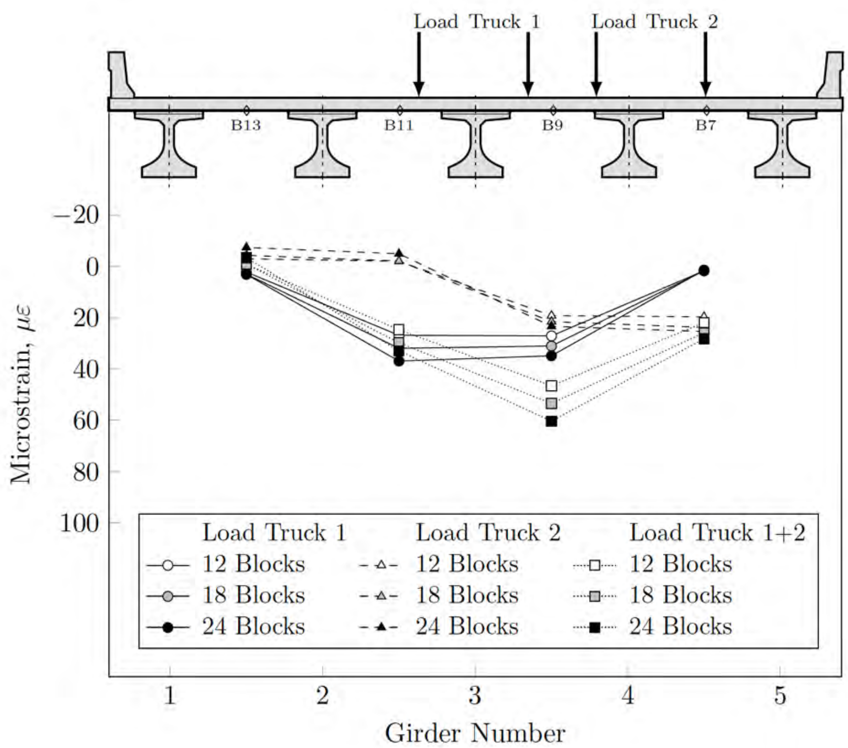


Fig. 3-29. Transverse Strain - HCWB - Section B - LP 3B

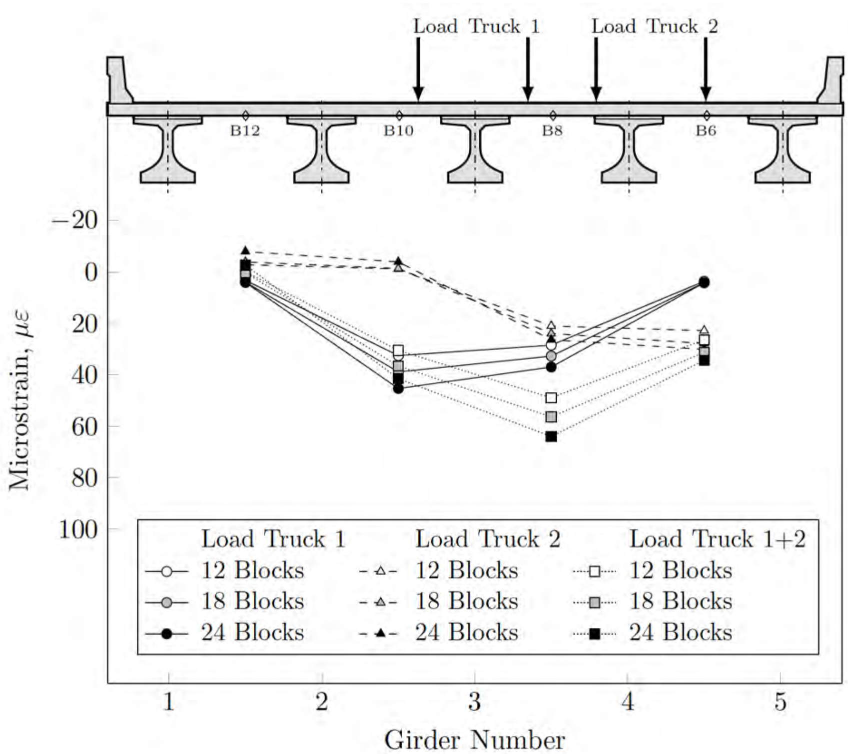


Fig. 3-30. Transverse Strain - HCWB - Section C - LP 3B

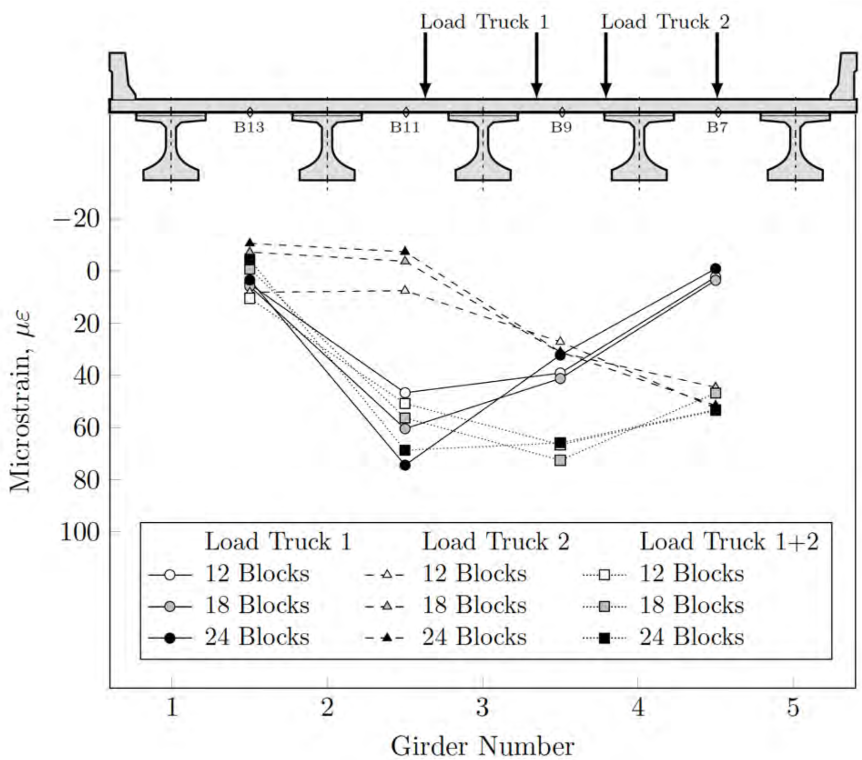


Fig. 3-31. Transverse Strain - LRWB - Section B - LP 3A

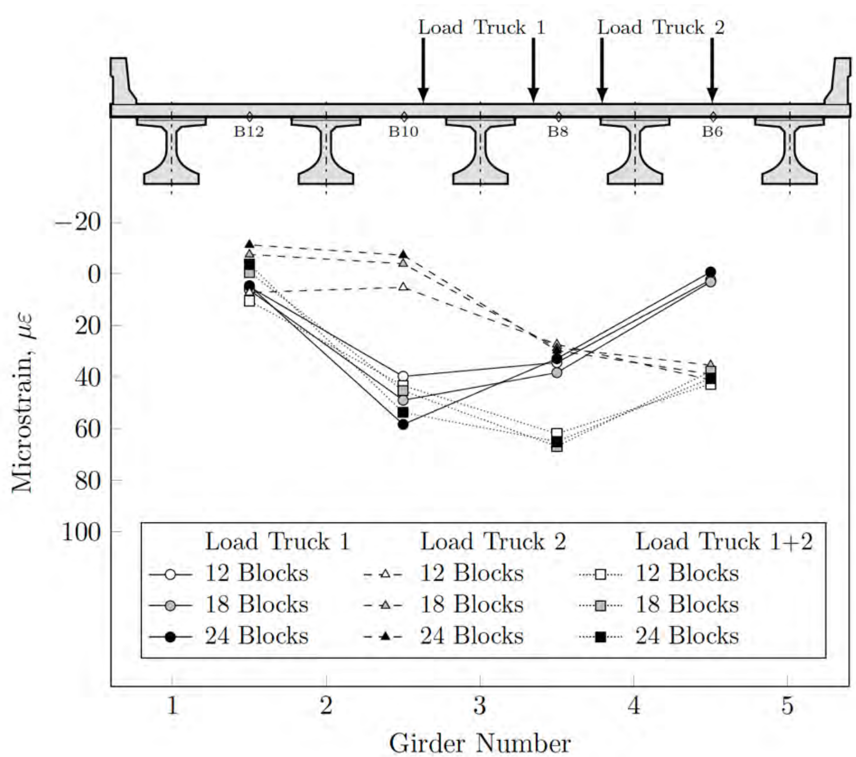


Fig. 3-32. Transverse Strain - LRWB - Section C - LP 3A

To conclude remarks on the transverse strain data for the westbound bridges, plots for LRWB for load position 3B are shown in **Figures 3-33 and 3-34**. The trends shown in these graphs are as expected. Maximum strains, located in between FIB3 and FIB4, were 70 με for gauge B8 and 68 με for gauge B9.

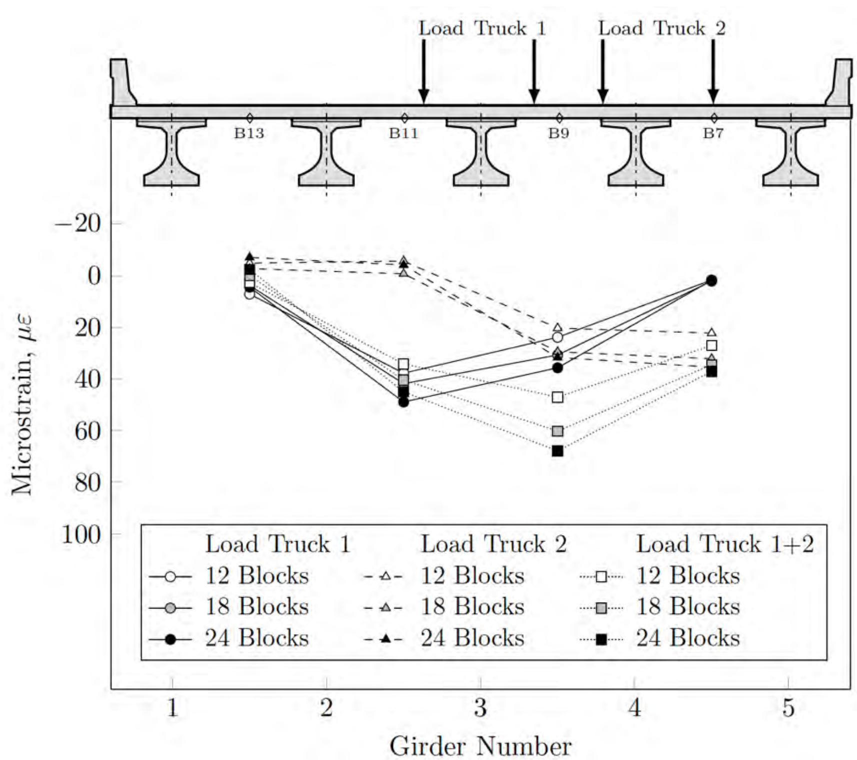


Fig. 3-33. Transverse Strain - LRWB - Section B - LP 3B

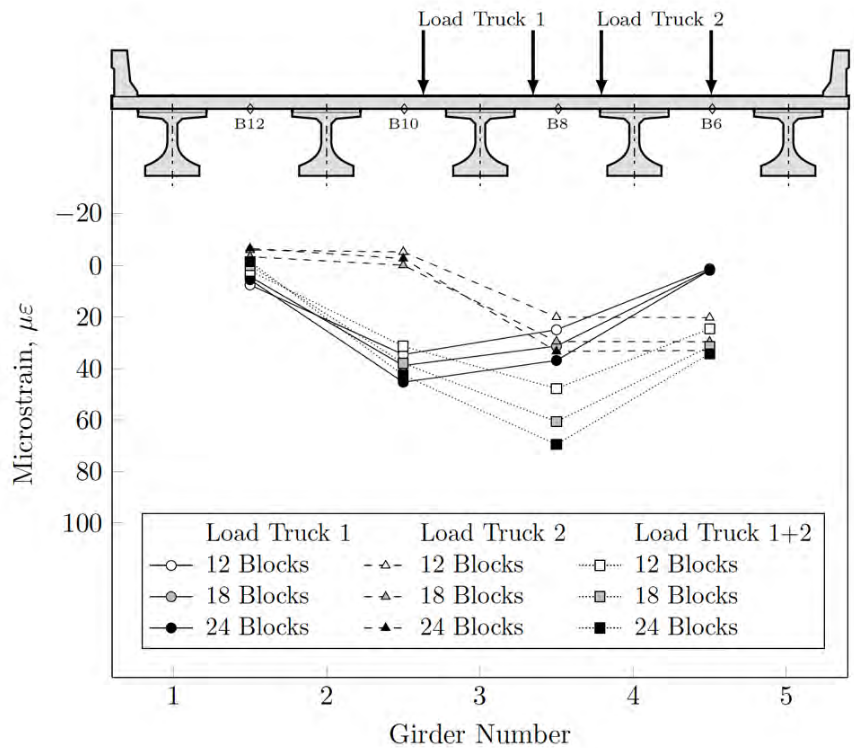


Fig. 3-34. Transverse Strain - LRWB - Section C - LP 3B

3.2 Pre-Service Test Data for Non-Prestressed Concrete Deck Panel Bridges

3.2.1 Introduction

This section presents the pre-service test data for HCEB and LREB. These bridges were constructed with **non-prestressed** concrete deck panels. The graphs presented in this section show data from two (2) loads tests, HCEB and LREB, after the barriers were constructed. The data analysis was done for load positions 1A and 2 for the deflection and longitudinal strain data and load positions 1A, 1B, 3A and 3B for the transverse strain data. The results are presented either at mid span, section B, or section C. As discussed previously, section B is located 9'-0" west of mid span, and section C is located 4'-6" west of mid span. Hurricane Creek EB, the first bridge that was load tested, only had three (3) load positions: 1, 2, and 3. Load positions 1 and 3 for HCEB correspond to 1A and 3A for the other load tests.

3.2.2 Composite Action

To determine if composite action was achieved between the panels and the FIBs, the longitudinal strains in FIB5, deck panel and barrier were plotted relative to the gauge's vertical location.

Figures 3-35 and 3-36 present the longitudinal strains on the cross section of HCEB FIB5 for load positions 1 and 2. The figures show strains in gauges F64, F36, F16, F6, F40, and F1, which are located on top of the barrier, top of the deck panel, bottom of the deck panel, top flange of the girder, midway up the girder from the bottom, and girder bottom, respectively. The strain in both graphs varies approximately linearly with the depth. Some gauges had higher readings for the 12 block cases than for 18 and 24 blocks. This could be due to drift in the gauge readings that occurred over time.

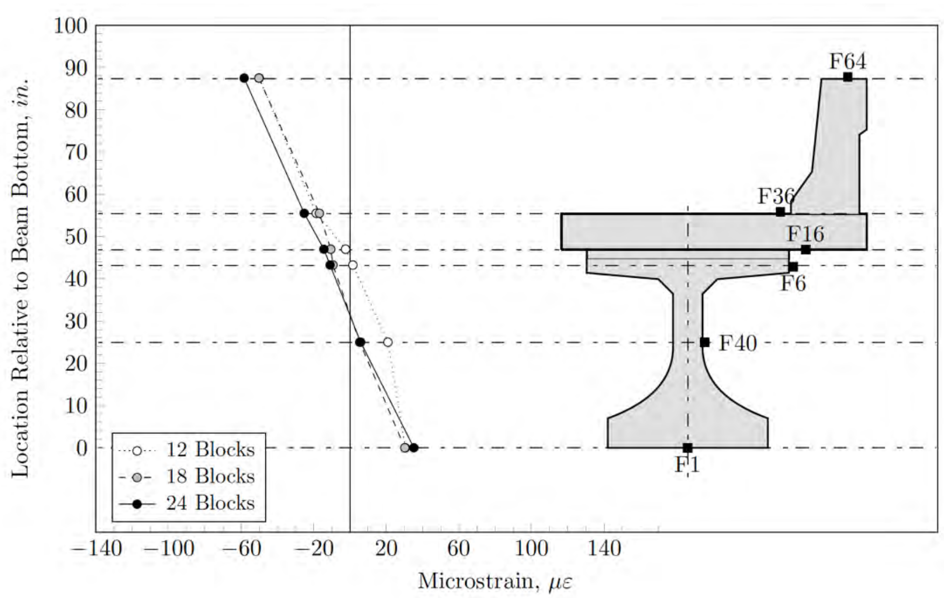


Fig. 3-35. Longitudinal Strain - HCEB - FIB5 - LP 1

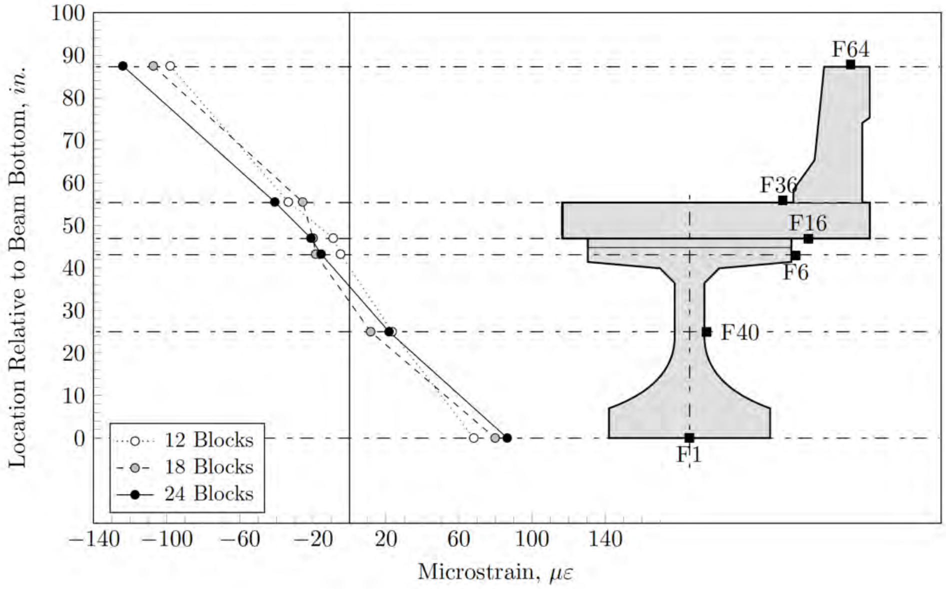


Fig. 3-36. Longitudinal Strain - HCEB - FIB5 - LP 2

LREB had the same gauges as HCEB, and the results were similar. The longitudinal strain through the cross section of the girder varied linearly, as shown in **Figures 3-37 and 3-38**. As the number of blocks increased, the compressive strain on the top of the barrier increased as the tensile strain on the girder bottom increased. The strains shifted from compression to tension at the same location, at approximately 32 in. to 35 in. from the girder bottom. This was true for both load positions (1A and 2).

3.2.3 Deflection Results

Presented in this section is the deflection data for HCEB and LREB (**Figures 3-39 to 3-42**). The anchor connected to the wire deflection gauge D11 on HCEB settled during the load tests and gave false readings, therefore that data was eliminated from the figures. Figures 3-39 and 3-40 show the girder deflections at mid span for the symmetric load positions 1 and 1A. The maximum deflections were for FIB3, with values of 0.39 in. for HCEB and 0.40 in. for LREB.

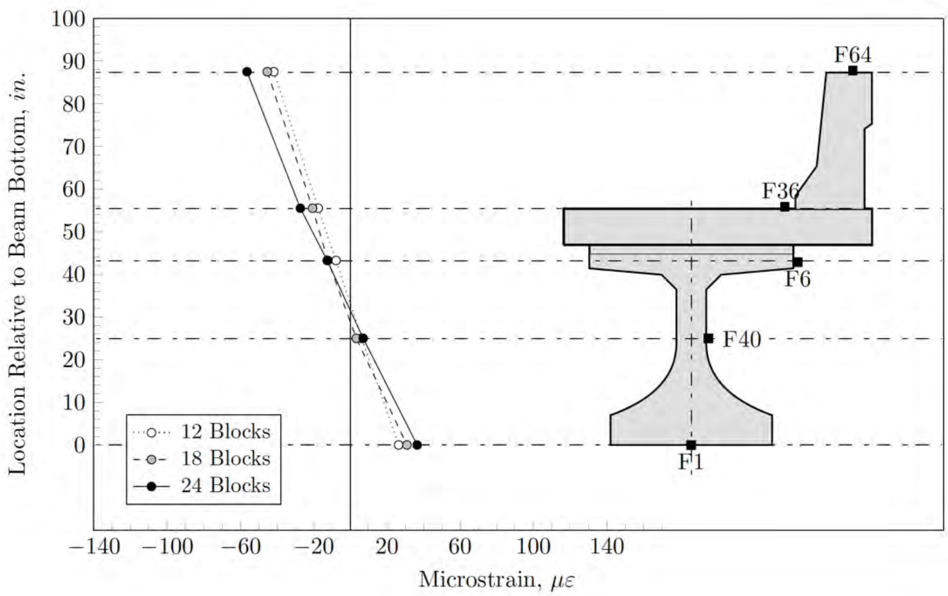


Fig. 3-37. Longitudinal Strain - LREB - FIB5 - LP 1A

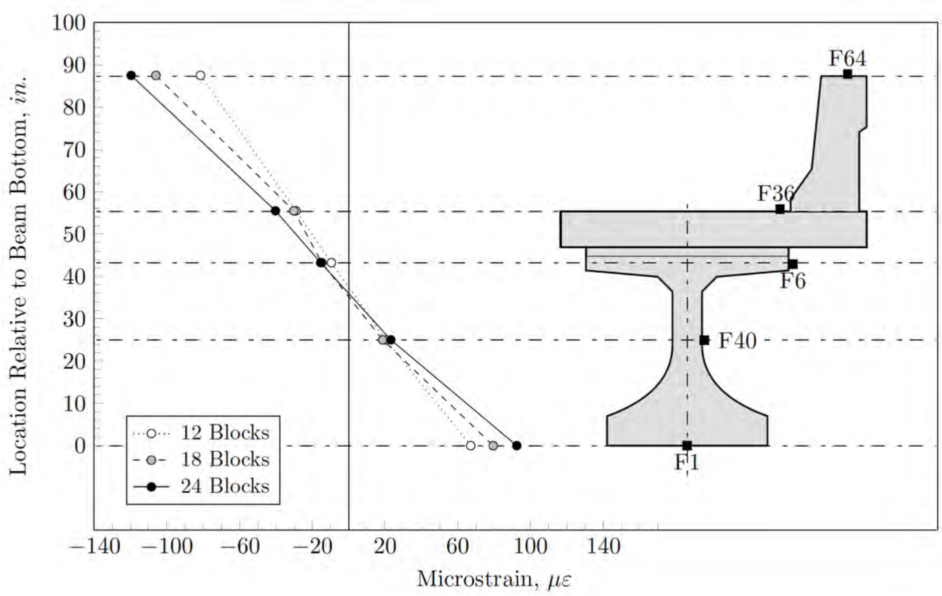


Fig. 3-38. Longitudinal Strain - LREB - FIB5 - LP 2

A comparison of Figures 3-41 and 3-42 show that the bridge response was repeatable from the HCEB to the LREB tests. The LREB FIB5 maximum deflection was about 0.44 in. when both trucks were loaded on the bridge at load position 2, as shown in Figure 3-42. The maximum deflection for HCEB FIB5 was not obtained for load positions 2 because gauge D11 malfunctioned during the load test.

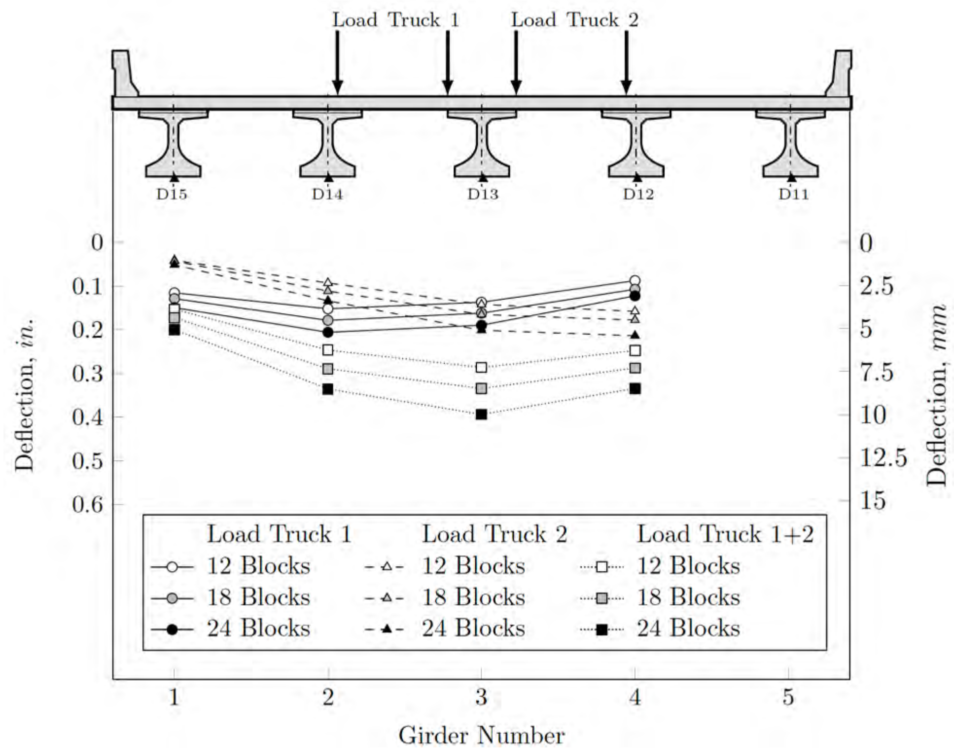


Fig. 3-39. Deflection - HCEB - Mid Span - LP 1

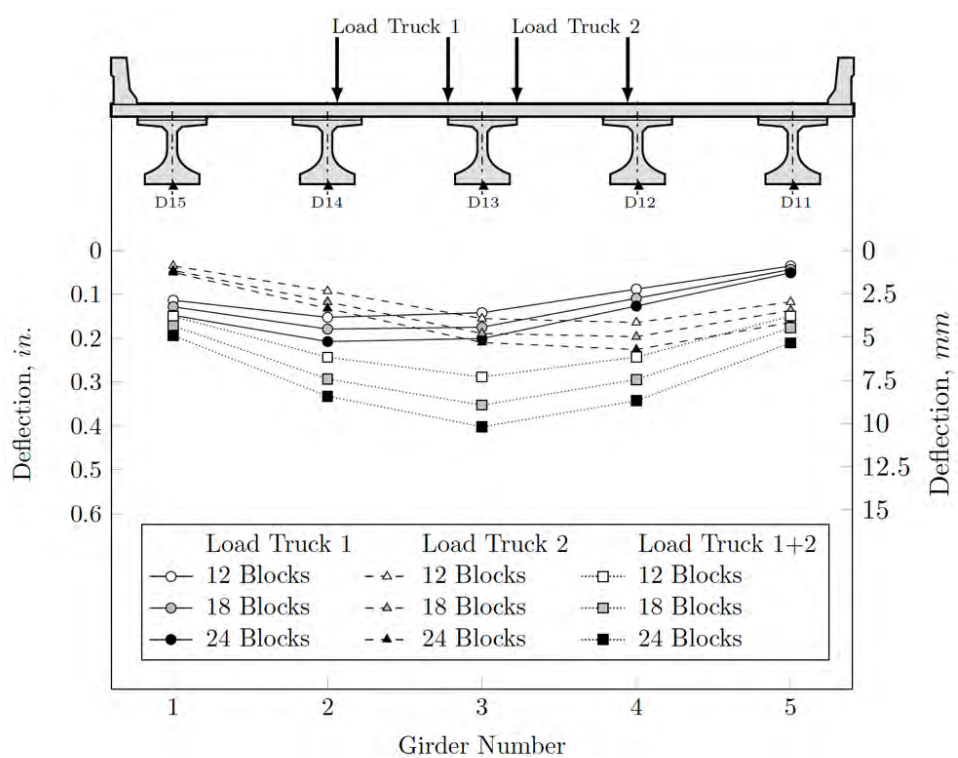


Fig. 3-40. Deflection - LREB - Mid Span - LP 1A

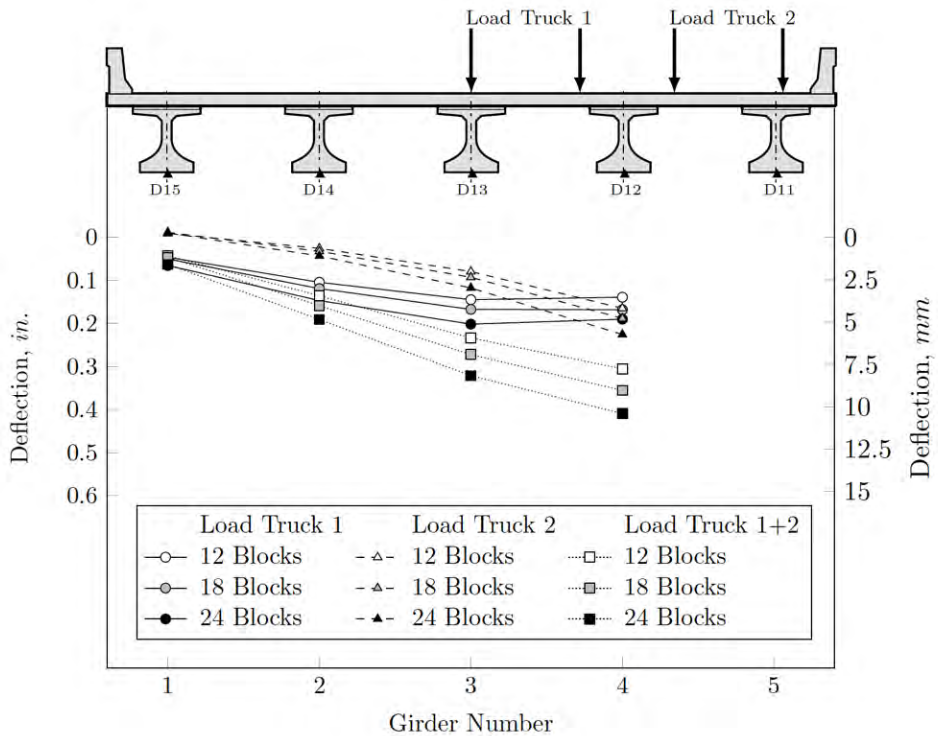


Fig. 3-41. Deflection - HCEB - Mid Span - LP 2

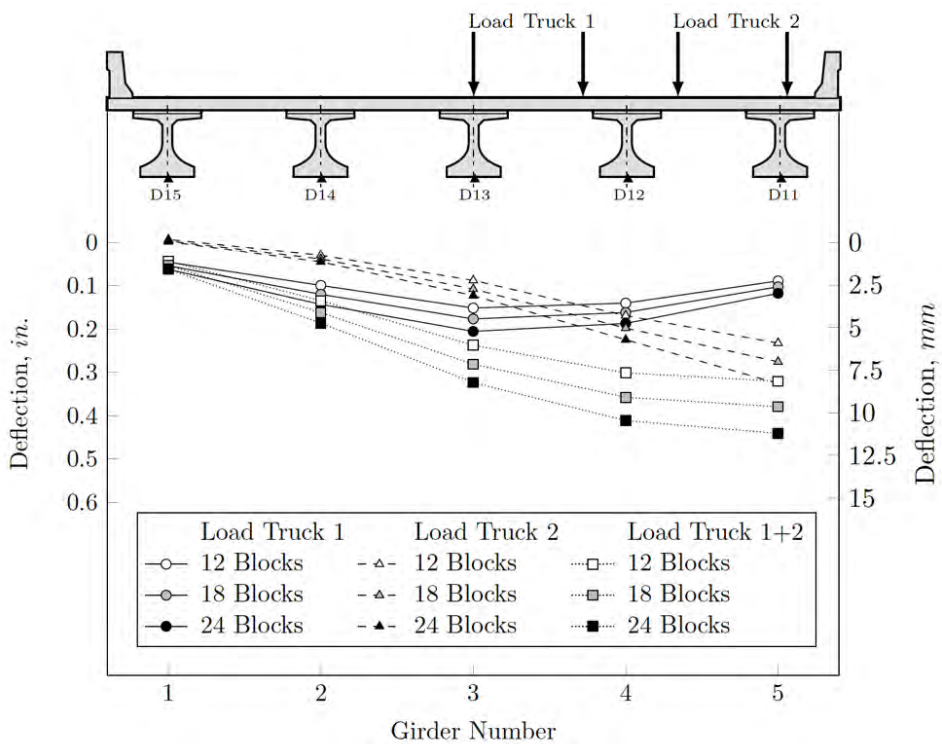


Fig. 3-42. Deflection - LREB - Mid Span - LP 2

3.2.4 Longitudinal Strain Results

The longitudinal strains for HCEB and LREB are presented in this section. **Figure 3-43** shows the results for HCEB for load position 1. (This load position is the same as 1A for LREB.) Gauge F2 on FIB4 malfunctioned during the test, so only gauge B2 results are plotted. At FIB5, gauge B1 also malfunctioned, so only F1 results are plotted. Otherwise, the averages of the two (2) gauges are plotted for each girder.

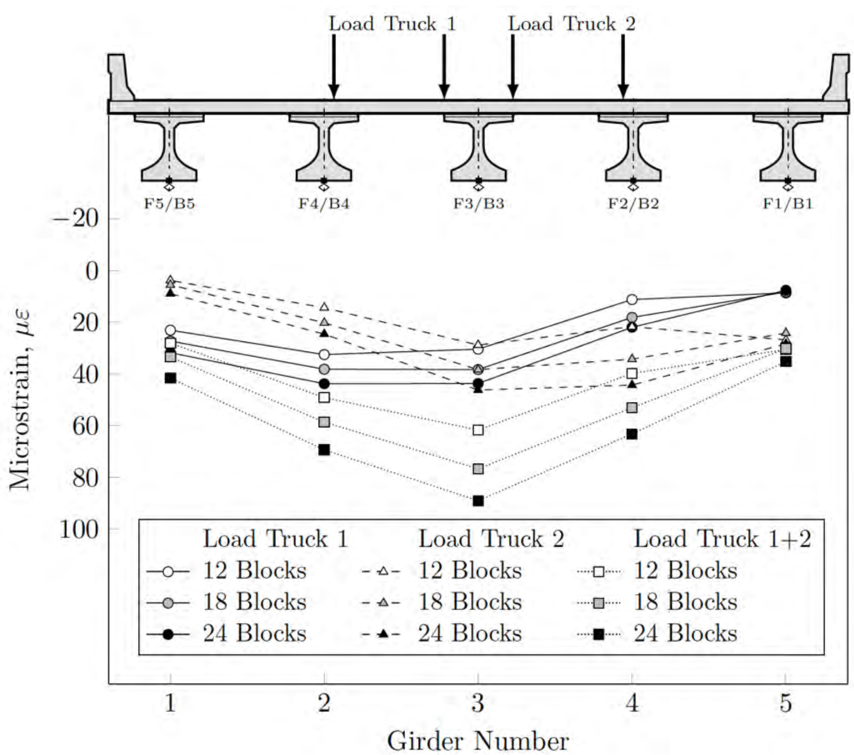


Fig. 3-43. Longitudinal Strain - HCEB - Section B - LP 1

HCEB had a maximum longitudinal strain of 89 με at FIB3. When truck 2 was placed on the bridge, the strains for the 12 block case were lower at FIB4 (22 με) than FIB5 (27 με), but this appears to be an anomaly. For all other block cases, this gauge (B2) had higher readings at FIB4 than FIB5, as expected.

During the LREB test, gauge F46 on FIB4 malfunctioned; therefore, the only data shown for FIB4 is from gauge F2. As shown in **Figure 3-44**, the maximum strain of 87 με was at FIB3 when 24 blocks were loaded. **Figures 3-45 and 3-46** show the longitudinal strains for load position 2 for both bridges. As expected, the maximum strains occurred at FIB5, with results of 86 με for HCEB and 93 με for LREB.

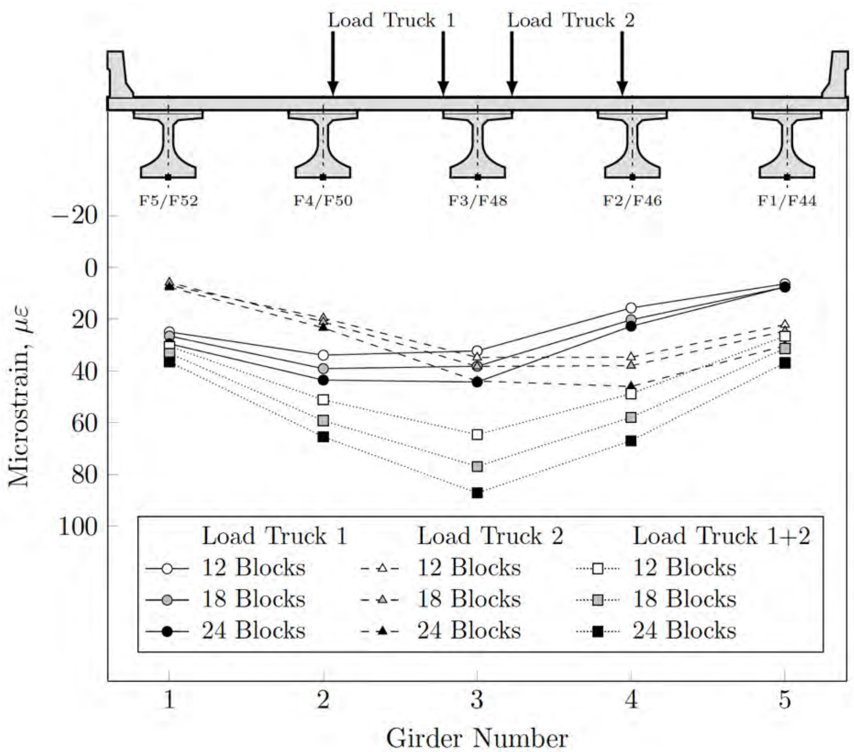


Fig. 3-44. Longitudinal Strain - LREB - Section B - LP 1A

3.2.5 Transverse Strain Results

Presented in this section are transverse strains for HCEB and LREB. **Figures 3-47 and 3-48** show the transverse strains for HCEB for load position 1 at sections B (in the panel) and C (in the joint). The loading in this case is symmetric transversely, so one would expect the strains in the gauges between FIB2 and 3 to equal the strains in the gauges between FIB3 and 4. However, as shown in Figure 3-47, gauge B11 has a much lower strain than B9, with a value of 40 με compared to 64 με. This difference occurs for all the block cases.

Figure 3-48 shows the data at section C of HCEB, where maximum strains were in gauges B10 (76 με) and B8 (64 με) for 24 blocks. The other block conditions follow the same trend, where gauge B10 is higher than B8 when both trucks are on the bridge. To explore these irregularities further, comparisons between EB and WB bridges, as well as between panels and joints, will be made in later sections of this report.

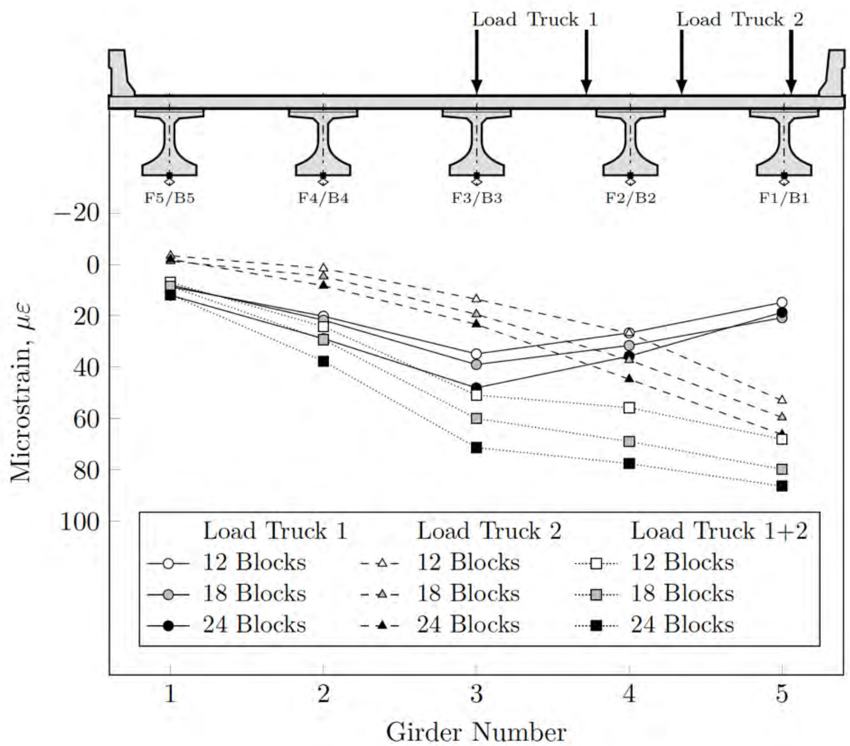


Fig. 3-45. Longitudinal Strain - HCEB - Section B - LP 2

Figure 3-49 shows the strains for LREB section B for load position 1A. The trend is almost symmetric, with maximum strains of 82 με in gauge B11 and 94 με in gauge B9. For gauge B9, the difference in strain from 12 blocks to 24 blocks is much larger than for gauge B11: 34 με for B9 and 20 με for B11.

Section C's gauge B8 went bad during the LREB load test, after the 18-block load step for load position 3B with truck 1 loaded. Therefore, **Figure 3-50** shows no data points for the 24-block load cases. The maximum strain in gauge B10 was 66 με.

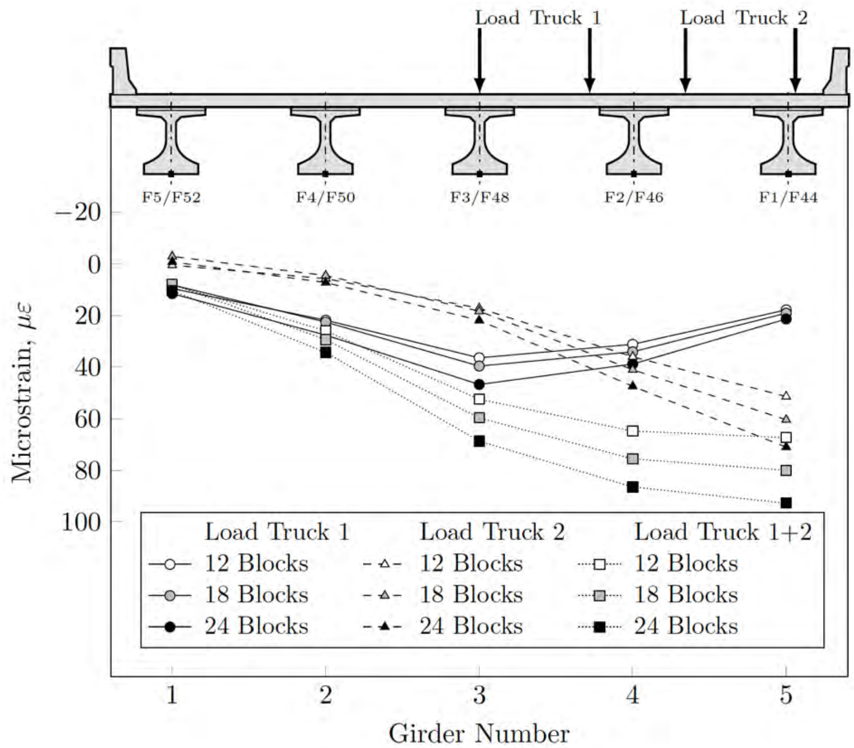


Fig. 3-46. Longitudinal Strain - LREB - Section B - LP 2

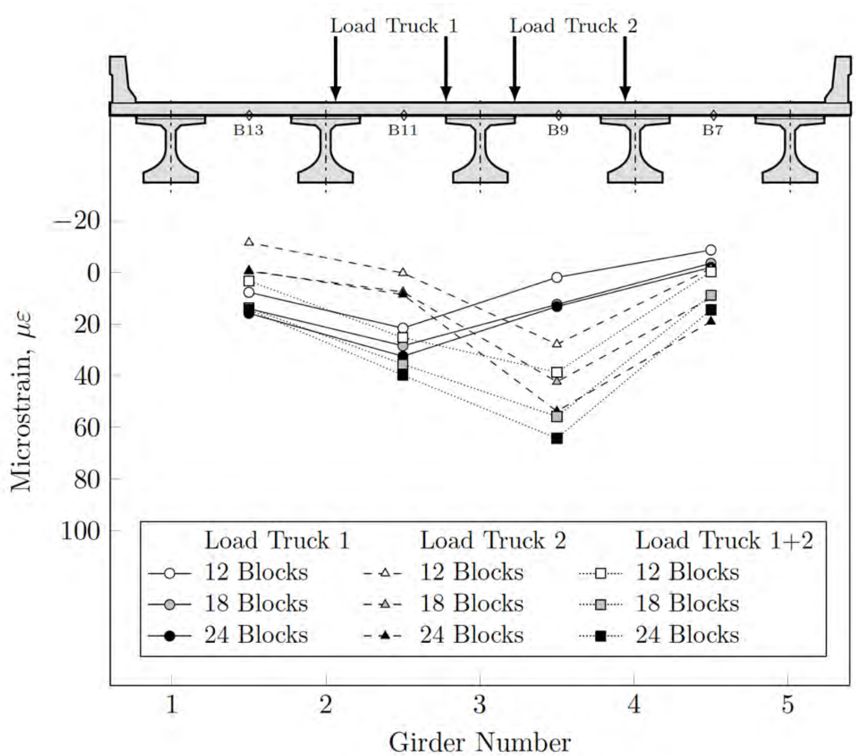


Fig. 3-47. Transverse Strain - HCEB - Section B - LP 1

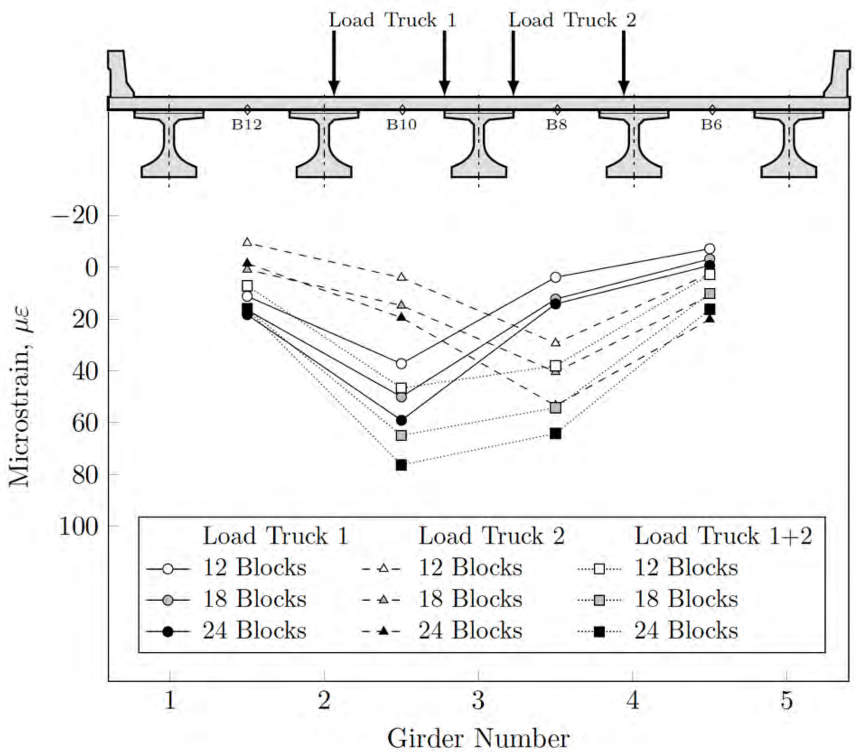


Fig. 3-48. Transverse Strain - HCEB - Section C - LP 1

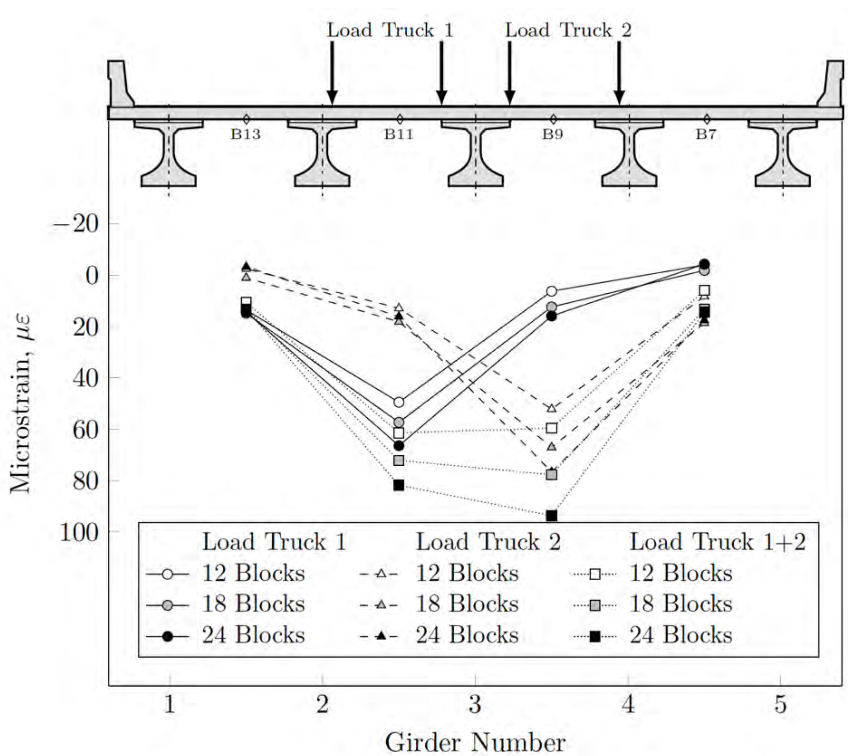


Fig. 3-49. Transverse Strain - LREB - Section B - LP 1A

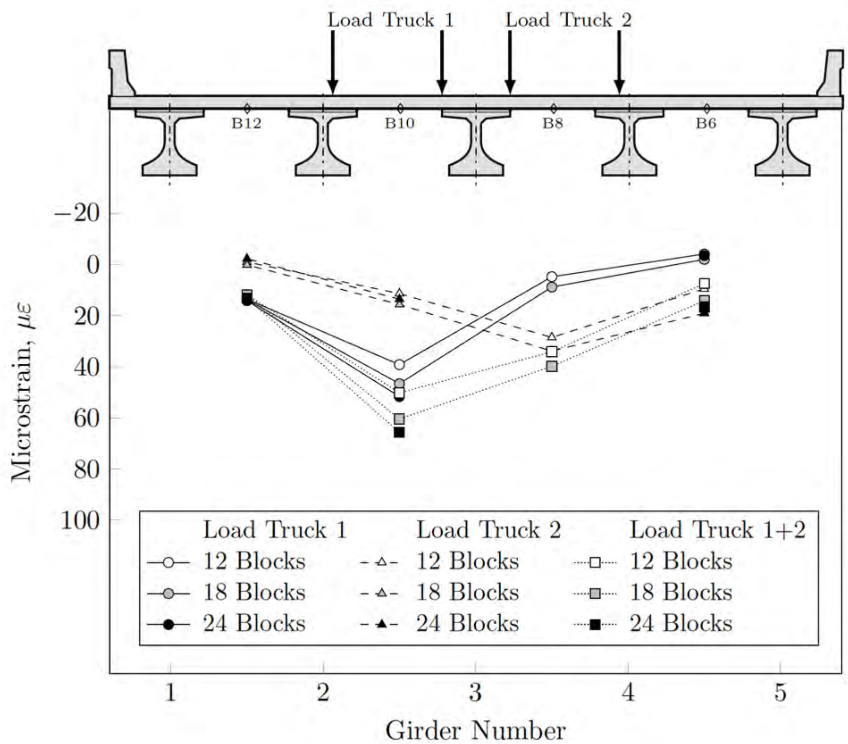


Fig. 3-50. Transverse Strain - LREB - Section C - LP 1A

The strains for LREB sections B and C for load position 1B are shown in **Figures 3-51 and 3-52**. In section B, the maximum strains occurred in gauges B11 and B9, with values of 82 με and 94 με. In section C, the maximum strain, in gauge B10, was 63 με.

As shown in **Figures 3-53 and 3-54**, load position 3 for HCEB again showed some fluctuations in both gauges B8 and B9 for sections C and B. The strains in these gauges were higher for the 18-block load cases than for the 24-block load cases.

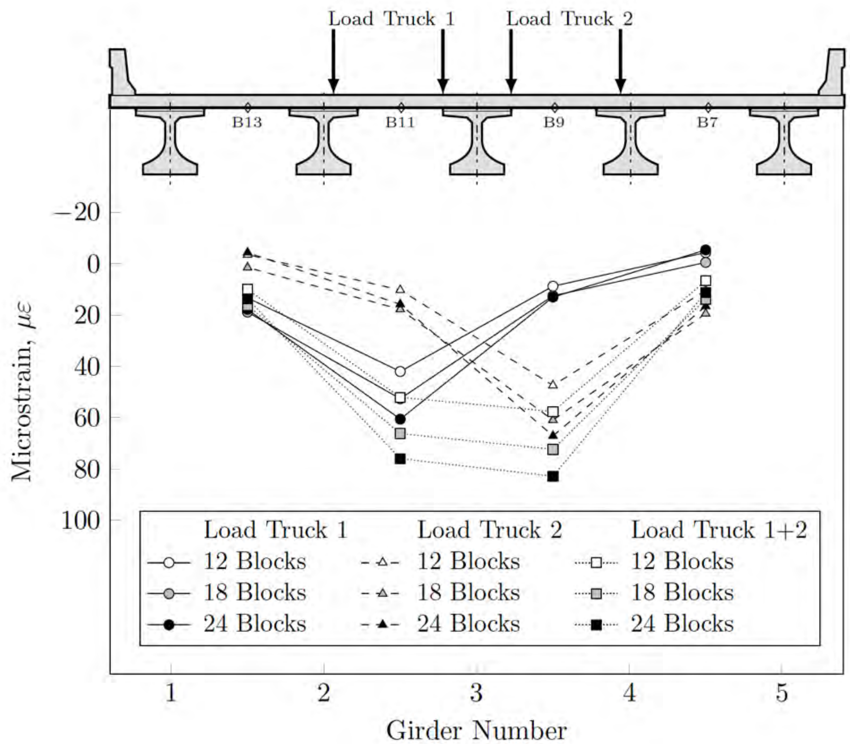


Fig. 3-51. Transverse Strain - LREB - Section B - LP 1B

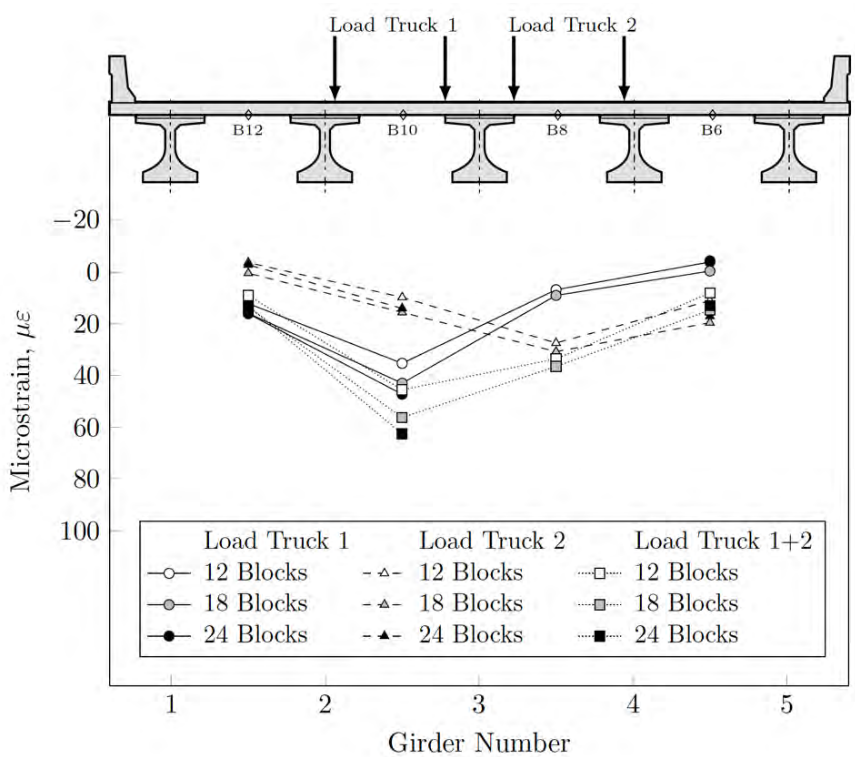


Fig. 3-52. Transverse Strain - LREB - Section C - LP 1B

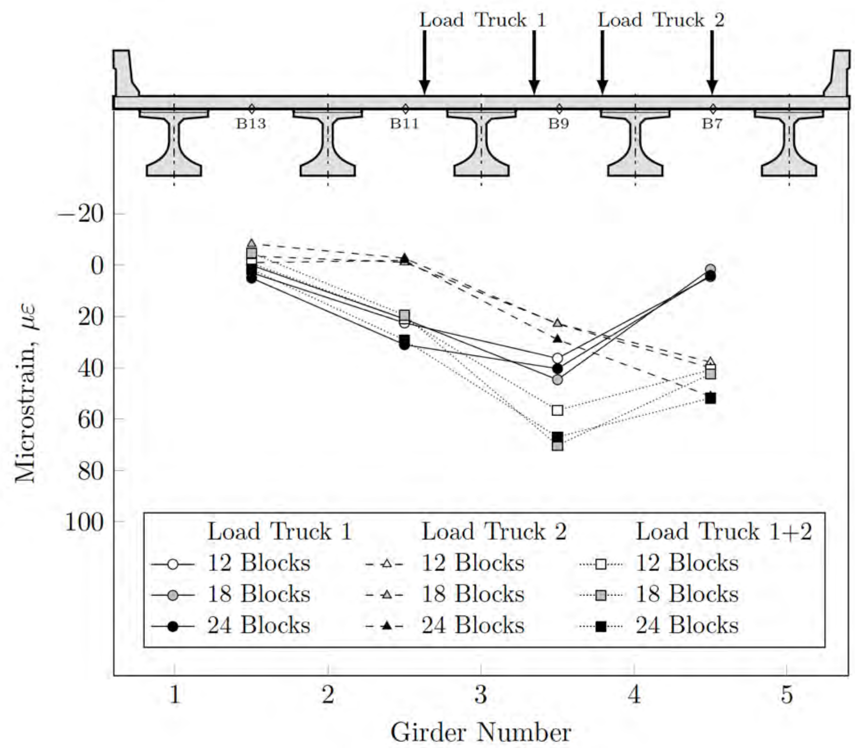


Fig. 3-53. Transverse Strain - HCEB - Section B - LP 3

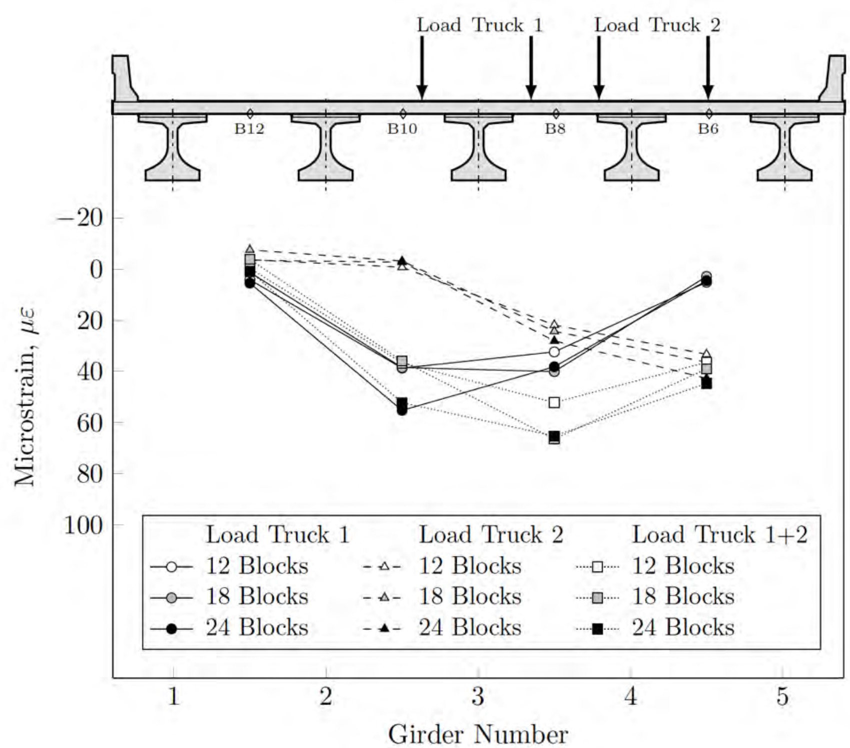


Fig. 3-54. Transverse Strain - HCEB - Section C - LP 3

Figures 3-55 to 3-58 plot all the results for LREB load positions 3A and 3B for both sections B and C. The load was not symmetric, and as expected, the maximum strains were in gauges B9 and B8. Unfortunately, gauge B8 malfunctioned, so no results are plotted for the 24-block case and only some are plotted for the 18-block cases. Comparing load positions 3A and 3B, the strain patterns were similar. This was in spite of two of the truck axles being close to sections B and C for load position 3A, whereas the axles were several feet away for load position 3B. From Figures 3-55 and 3-57, for load truck 1 which was centered on the bridge, the strains were mostly symmetric (i.e., gauges B9 and B11 had similar readings) as expected.

Section C showed data and both load position 3A and 3B behaved similarly with strain readings of 102.4 $\mu\varepsilon$ and 87.21 $\mu\varepsilon$.

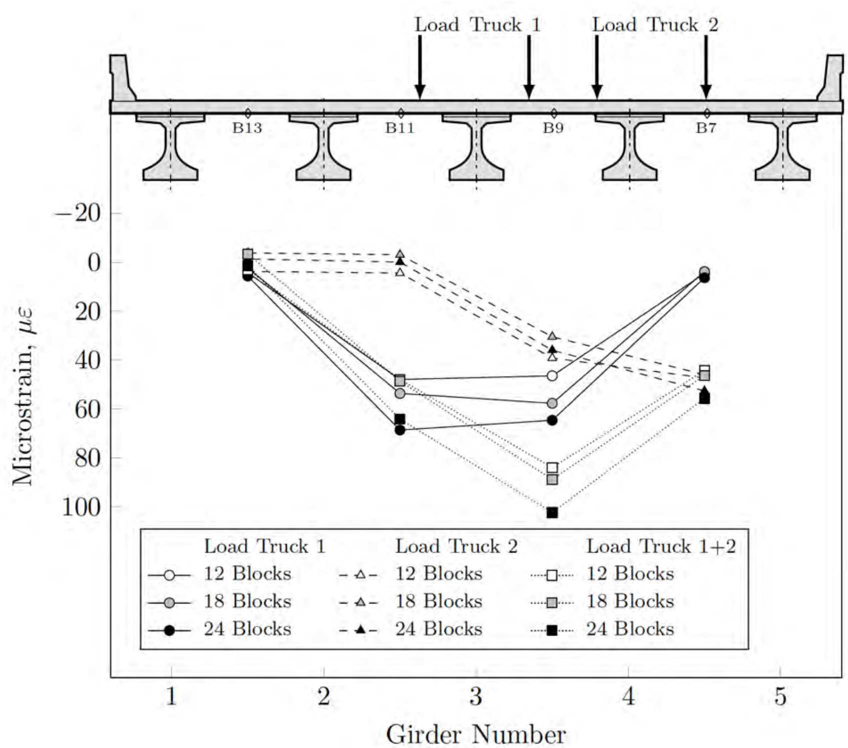


Fig. 3-55. Transverse Strain - LREB - Section B - LP 3A

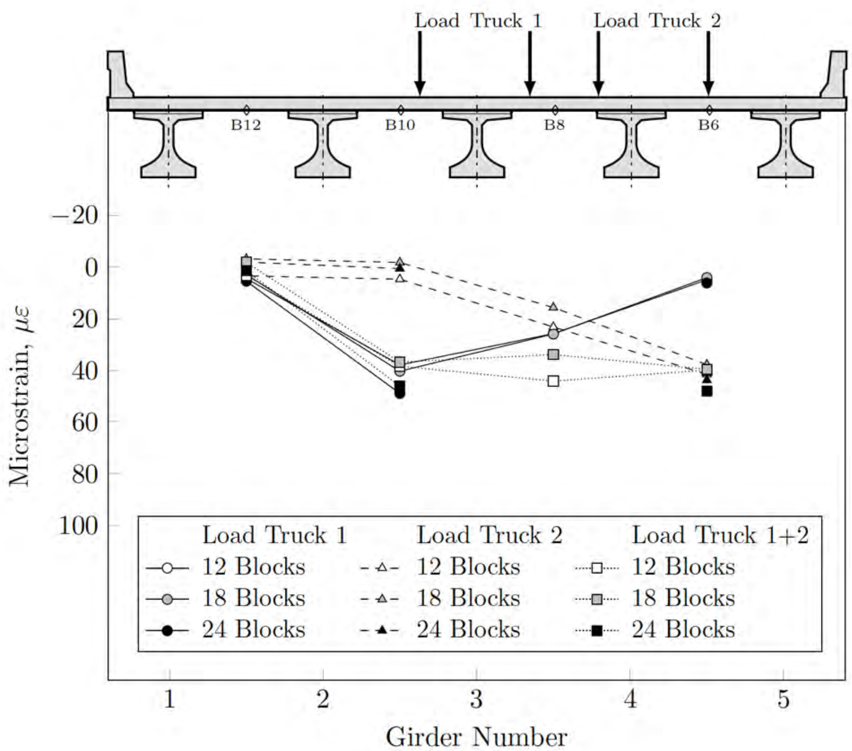


Fig. 3-56. Transverse Strain - LREB - Section C - LP 3A

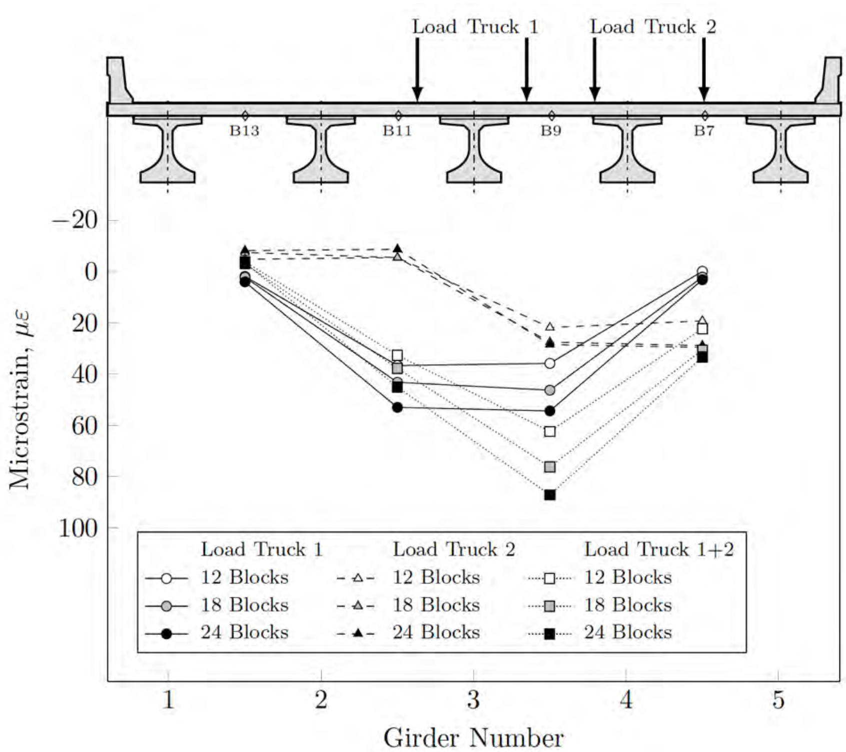


Fig. 3-57. Transverse Strain - LREB - Section B - LP 3B

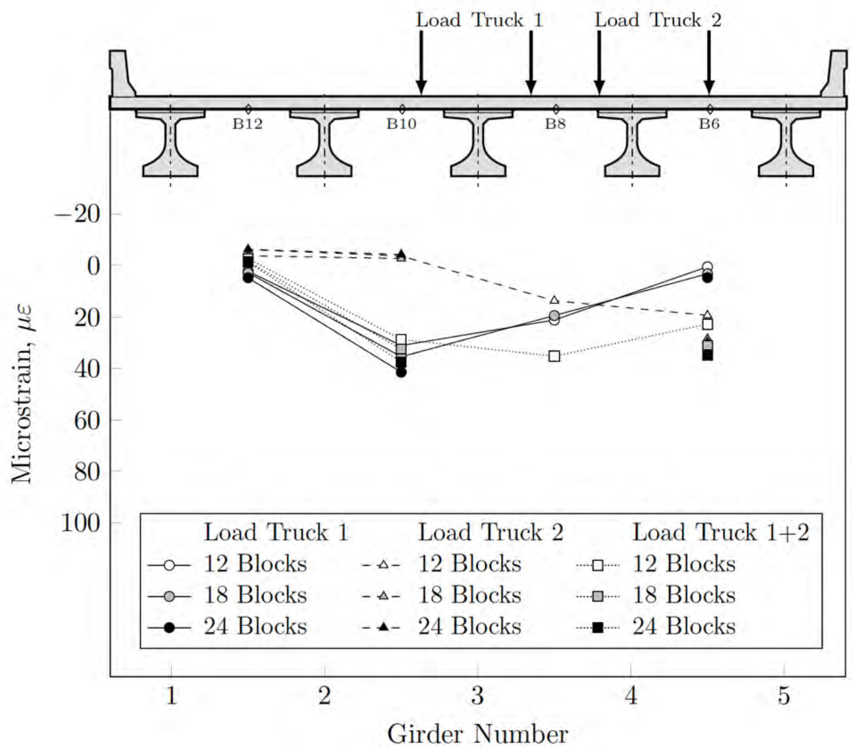


Fig. 3-58. Transverse Strain - LREB - Section C - LP 3B

3.3 Pre-Service Test Data for Bent Caps

This section presents the pre-service test data for the LREB and LRWB bent caps. **Figures 3-59 to 3-64** show the longitudinal strains on the vertical faces of the bent cap, at mid span, for Load Position 4. Figures 3-59 to 3-61 are for the EB bent cap, for 12, 18, and 24 blocks, respectively. Figures 3-62 to 3-64 are for the WB bent cap, for 12, 18, and 24 blocks, respectively.

For the EB bent cap, Load Position 4, the left and right faces responded similarly (i.e., the strain distributions from top to bottom were approximately equal) for all three block cases (see Figures 3-59 to 3-61), with the exception of the top gauge. However, for the WB bent cap, the right face had significantly higher tensile strains than the left face, and the neutral axis was much closer to the top (see Figures 3-62 to 3-64).

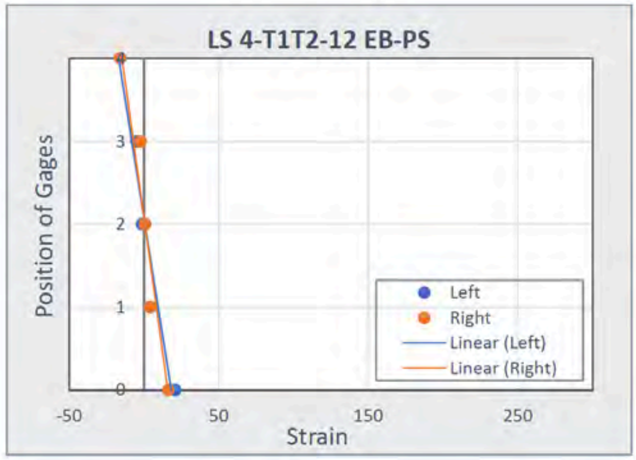


Fig. 3-59. Linear Strain Response of Bent Cap Faces to LS4-T1T2-12 EB-PS

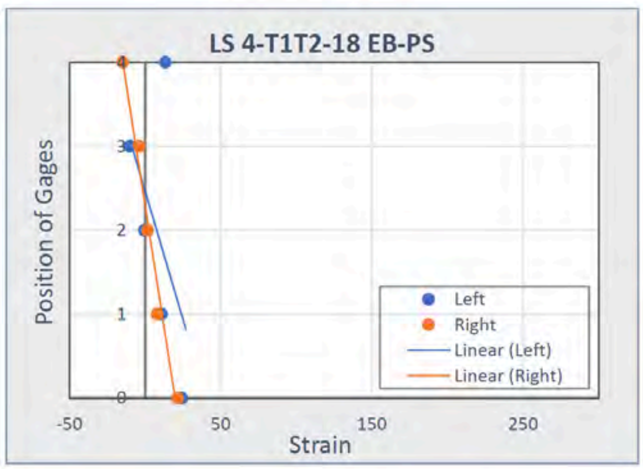


Fig. 3-60. Linear Strain Response of Bent Cap Faces to LS4-T1T2-18 EB-PS

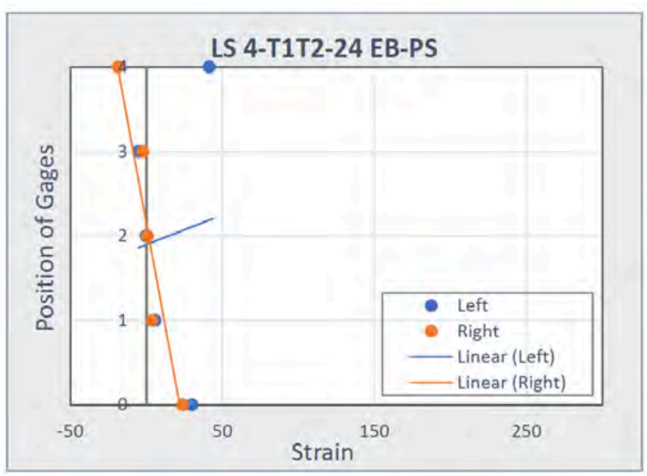


Fig. 3-61. Linear Strain Response of Bent Cap Faces to LS4-T1T2-24 EB-PS

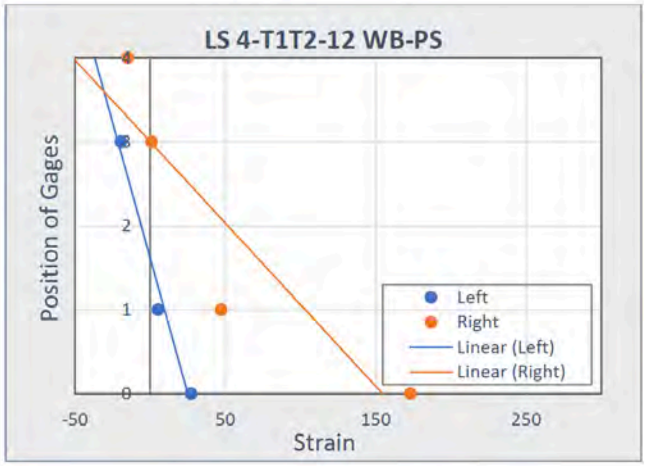


Fig. 3-62. Linear Strain Response of Bent Cap Faces to LS4-T1T2-12 WB-PS

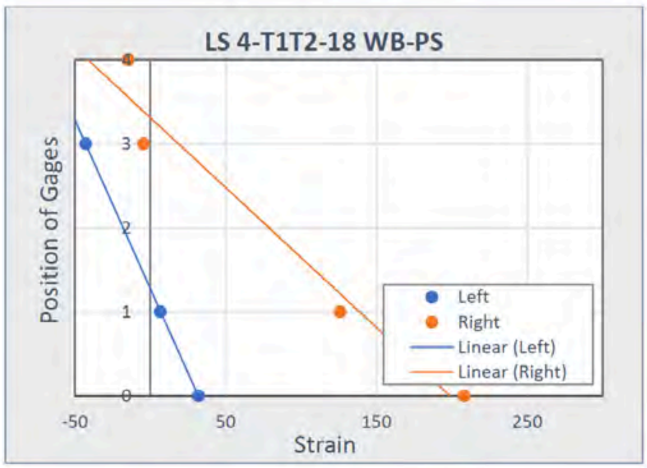


Fig. 3-63. Linear Strain Response of Bent Cap Faces to LS4-T1T2-18 WB-PS

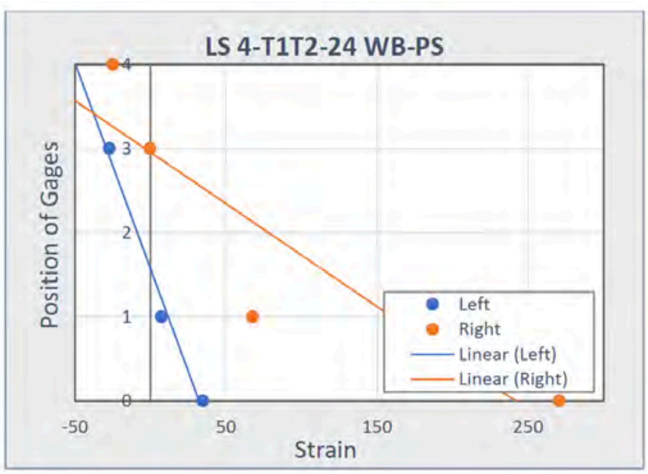


Fig. 3-64. Linear Strain Response of Bent Cap Faces to LS4-T1T2-24 WB-PS

Figures 3-65 to 3-70 show the longitudinal strains on the vertical faces of the bent cap, at mid span, for Load Position 5. Figures 3-65 to 3-67 are for the EB bent cap, for 12, 18, and 24 trucks, respectively. Figures 3-68 to 3-70 are for the WB bent cap, for 12, 18, and 24 trucks, respectively.

For the EB bent cap, Load Position 5, the response was similar to that for Load Position 4 as described above. The left and right faces responded similarly (i.e., the strain distributions from top to bottom were approximately equal) for all three block cases (see Figures 3-65 to 3-67), with the exception of the top gauge. However, for the WB bent cap, the right face had significantly higher tensile strains than the left face, and the neutral axis was much closer to the top (see Figures 3-68 to 3-70).

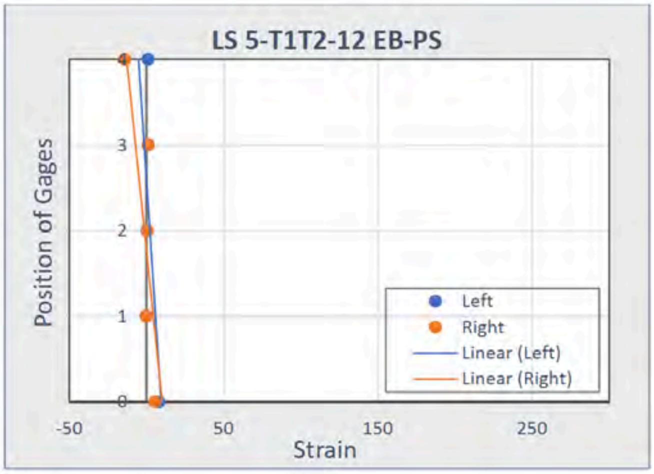


Fig. 3-65. Linear Strain Response of Bent Cap Faces to LS5-T1T2-12 EB-PS

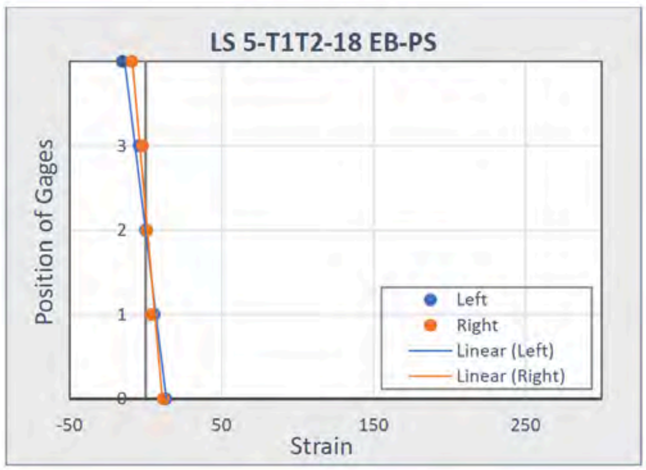


Fig. 3-66. Linear Strain Response of Bent Cap Faces to LS5-T1T2-18 EB-PS

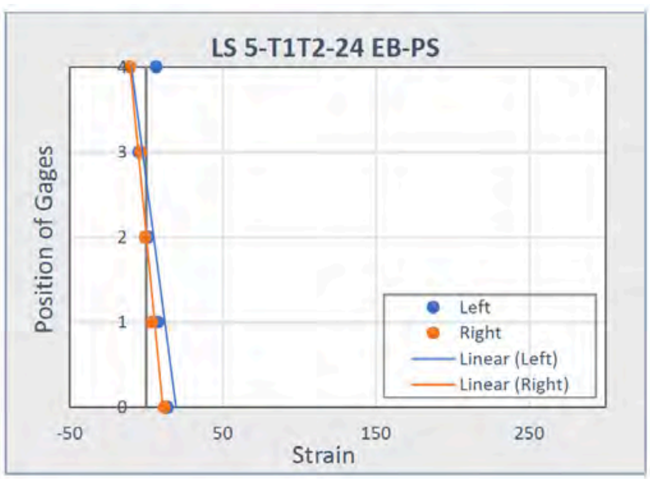


Fig. 3-67. Linear Strain Response of Bent Cap Faces to LS5-T1T2-24 EB-PS

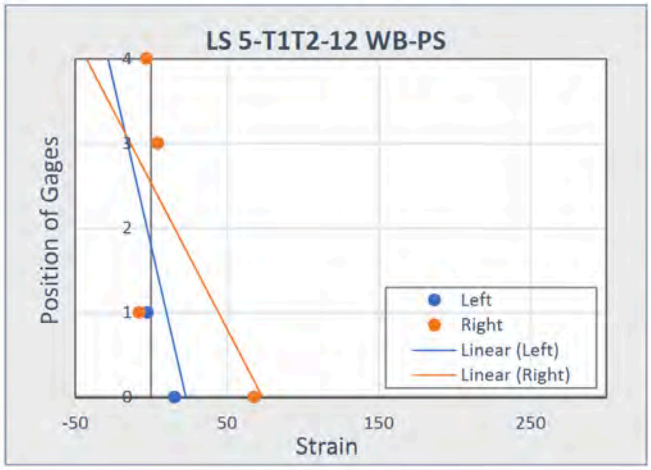


Fig. 3-68. Linear Strain Response of Bent Cap Faces to LS5-T1T2-12 WB-PS

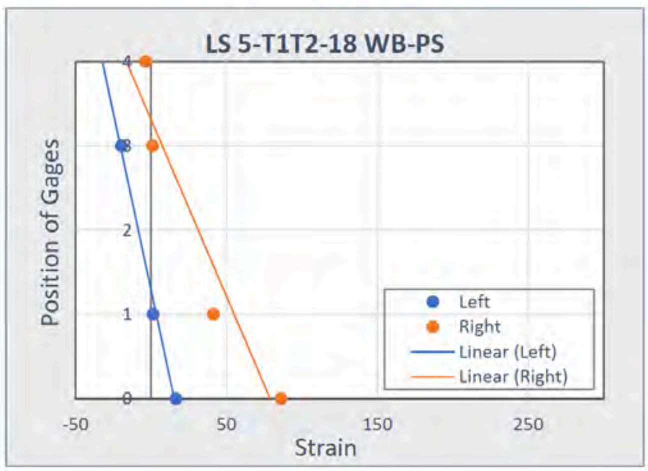


Fig. 3-69. Linear Strain Response of Bent Cap Faces to LS5-T1T2-18 WB-PS

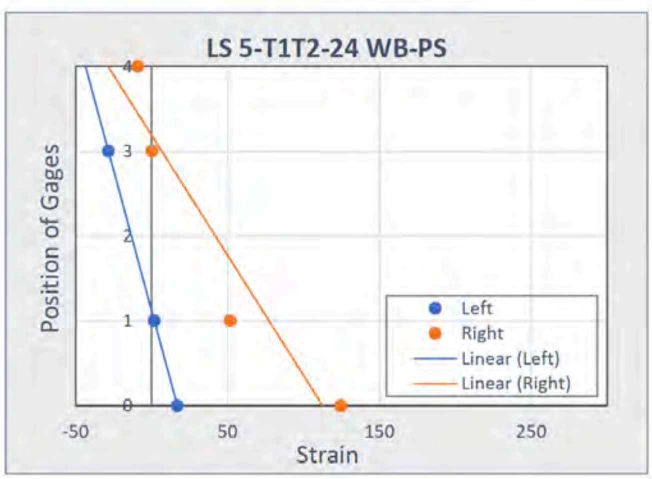


Fig. 3-70. Linear Strain Response of Bent Cap Faces to LS5-T1T2-24 WB-PS

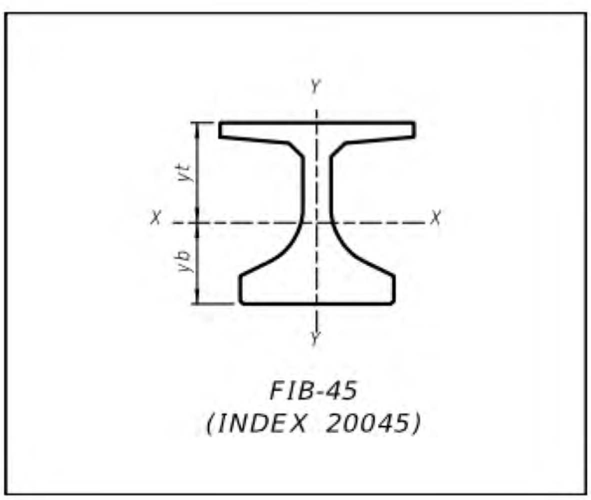
CHAPTER 4
PRE-SERVICE TEST DATA ANALYSIS

4.1 Analysis Approach

This chapter presents an analysis of the pre-service test results. First, theoretical section properties of the girders were calculated, followed by calculations of bending moments due to the test trucks. These theoretical values were used to analyze the data and make comparisons between the load tests. In subsequent chapters, the following analyses are presented: comparisons are made between the prestressed panel bridges and the non-prestressed panel bridges; distribution of load to the girders was determined from the load tests and compared to AASHTO predictions; and the HCWB load tests were analyzed for the effect that the barrier has on girder strains, deflections, and load distribution.

4.1.1 Theoretical Moment of Inertia

The theoretical moments of inertia of the composite girders were calculated. For all four (4) bridges, the material properties varied slightly from girder to girder, so section properties were calculated for each girder that was tested. For the non-composite 45-inch FIB, FDOT provides section properties, as shown in **Figure 4-1** (FDOT 2012).



FIB-45 SECTION PROPERTIES	
Area (in.^{2})	869.58
Perimeter (in.)	224.57
Ixx (in.^{4})	226,581
Iyy (in.^{4})	81,327
yt (in.)	24.79
yb (in.)	20.21

Fig. 4-1. 45" Florida-I Beam Properties (Source: FDOT Index 20010)

Listed in the figure are the gross geometric properties, including the area, moment of inertia, and centroid location. Each composite girder section included the girder, prestressing strands, haunch, effective slab, and barriers where applicable. Each of these components was transformed into girder concrete using the modular ratio, $n = E_{component}/E_{FIB}$. The modulus of elasticity of the girder, E_{FIB}, was calculated based on the girder's 28-day strength from cylinder tests. For example, FIB1 section properties consisted of FIB1, the haunch above the FIB, the slab with an effective width of 96.5 in., and also the barrier. (The barrier was included in section property calculations for the exterior girders. This was deemed acceptable because there were no open joints in the barrier for the entire span length). For an interior girder such as FIB2, the section properties were calculated using the haunch above the FIB and the effective slab which was 108 in. wide.

The haunches, for calculation purposes, were assumed to be 2 in. thick, and the deck panels were assumed to be 8.5 in. The nominal panel thickness was 8.5 in. including a 0.5-in. sacrificial wearing surface. However, some panels were cast as much as 9 in. thick. The thickness was reduced some when the deck was milled and grooved. The material properties that were used for the girders, haunches, and deck panels were provided previously.

To calculate the composite transformed section properties (moment of inertia and centroid location relative to the bottom of the girder), first, the transformed area of each component was calculated using Eq. 6. The steel prestressing strands were in multiple layers, and each layer was considered as a component when calculating the transformed properties.

$$A_{tr_i} = n_i(A_i) \qquad \text{Eq. 6}$$

where:

A_{tr_i} = transformed area of a component, in^2

n_i = modular ratio for a component, E_i/E_{FIB}

A_i = area of a component, in^2

After the areas were calculated, the moment of inertia for each component was calculated. The haunches and deck panels were considered to be rectangular, the girder's moment of inertia was provided by FDOT, and the barriers were approximated with "simple" shapes (rectangles and triangles) as shown in **Figure 4-2**.

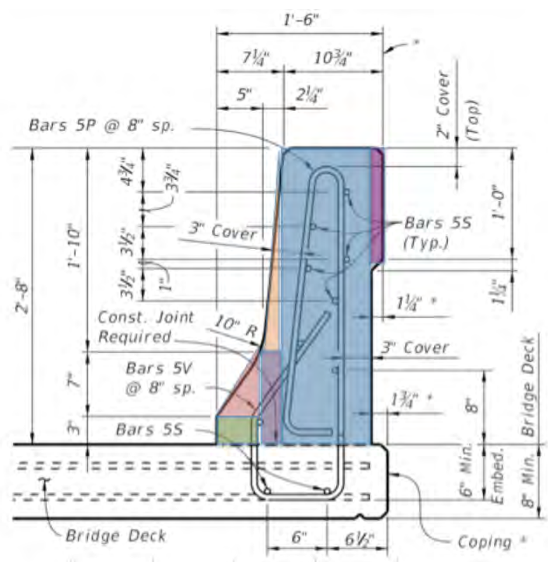


Fig. 4-2. Barrier Cross Section (Source: FDOT)

After the moments of inertia were calculated, the centroid of the composite section, c_{bot}, relative to the bottom girder was calculated using Eq. 7.

$$c_{bot} = \frac{\sum_{i=1}^{n}\left(\tilde{y}_i A_{tr_i}\right)}{\sum_{i=1}^{n} A_{tr_i}} \qquad \text{Eq. 7}$$

where:

$\tilde{y}_i$ = centroid of individual component, relative to bottom of girder, in.

The moment of inertia for the composite girder, I_c, was then calculated using the parallel axis theorem in Eq. 8. The distance d_i in the equation is the distance from the centroid of the composite section to the centroid of an individual component, $d_i = c_c - \tilde{y}_i$.

$$I_c = \sum_{i=1}^{n}\left(I_{tr_i} + A_{tr_i} d_i^2\right) \qquad \text{Eq. 8}$$

where:

d_i = distance from centroid of component to centroid of composite section, in.

I_{tr_i} = transformed moment of inertia of an individual component, in^4

Table 4-1 shows the calculated transformed areas A_{tr}, centroids from the bottom c_{bot} of the FIB, and composite moment of inertia I_c.

Table 4-1: Transformed Section Properties for All Bridges and Girders

Bridge	FIB1			FIB2			FIB3			FIB4			FIB5		
	A_{tr} in^2	c_{bot} in	I_c in^4	A_{tr} in^2	c_{bot} in	I_c in^4	A_{tr} in^2	c_{bot} in	I_c in^4	A_{tr} in^2	c_{bot} in	I_c in^4	A_{tr} in^2	c_{bot} in	I_c in^4
HCEB	2124	40.00	1017307	1934	36.11	716946	1916	35.99	712088	1916	35.99	712088	2152	40.18	1027830
LREB	2129	40.05	1022383	1903	35.89	709056	1933	36.08	716958	1933	36.08	716958	2129	40.05	1022383
HCWB	2151	40.50	1054868	1990	36.44	731214	1908	35.92	710025	1908	35.92	710025	2184	40.70	1067236
LRWB	2121	40.39	1058467	1841	35.37	693265	1886	35.68	705648	1886	35.68	705648	2077	40.09	1041219

4.1.2 Theoretical Bending Moment

Figure 4-3 is a schematic of the span and loads. For each load position, the truck axle loads were placed as discussed previously, and the moments were calculated at section B as shown in Figure 2-18. The distances between the truck axles were constant, but the truck position varied longitudinally depending on the load position and test step. The truck axle configuration and loads are shown in Figure 2-1 and Table 2-1.

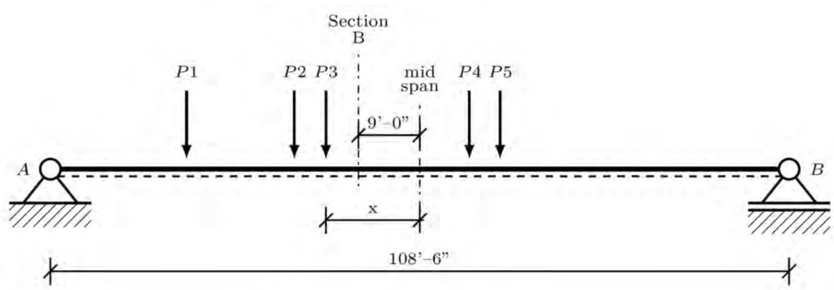


Fig. 4-3. Line Analysis Schematic

The theoretical bending moments due to the test loads were calculated using a line analysis. The span length used was the center-to-center distance between the bearing pads, 108'-6". The theoretical bending moment at Section B (which was 9'-0" west of the mid span), was 1856 kip-ft for 24 blocks loaded on one truck. For two (2) trucks, this number was doubled, resulting in 3712 kip-ft. The theoretical moments at section B and at mid span due to one and two trucks for all block cases and for load positions 1A, 1B, 2, 3A, and 3B are shown in **Table 4-2**.

Table 4-2. Theoretical Bending Moments at Section B and Mid Span

Load Position	x ft	Location of Moment	Number of Blocks	Moment - One Truck kip-ft	Moment - Two Trucks kip-ft
1A, 2, 3A	6'8"	Section B	12	1360	2720
			18	1600	3200
			24	1856	3712
		Midspan	12	1362	2724
			18	1609	3218
			24	1873	3746
1B, 3B	17'-3.5"	Section B	12	1295	2590
			18	1537	3074
			24	1795	3590
		Midspan	12	1325	2650
			18	1580	3160
			24	1850	3700

4.2 Load Distribution

As discussed previously, distribution factors are used in design to determine the amount of live load that will be carried by the interior and exterior girders. The load distribution factors were calculated for the four (4) bridges according to the *AASHTO LRFD Bridge Design Specifications* (*AASHTO* 2017). These theoretical values were then compared to distribution factors calculated from the load test data. They were calculated in two ways: using measured strains and using measured deflections. These DFs from the test data were multiplied by the multiple presence factor (MPF) so they could be compared to AASHTO predictions. In addition, when two trucks were on the bridge, the DFs from the test data were multiplied by 2.

4.2.1 AASHTO LRFD Distribution Factors

In a previous section, the AASHTO LRFD distribution factor equations for interior and exterior girders were provided. Using Eqs. 1 and 2, the interior girder distribution factors were calculated. The exterior girder distribution factors were calculated using either the lever rule for one lane loaded or Eq. 4 for two lanes loaded. **Table 4-3** presents the theoretical distribution factors for four (4) bridges. Recall that the material properties for the girders vary slightly, which explains the small differences in the DFs for girders that are otherwise identical. For example, the DF for HCEB FIB2 is 0.467 for one lane loaded, which is slightly different than 0.468 for FIB3 because of their different concrete strengths.

Table 4-3. Theoretical Distribution Factors (AASHTO LRFD)

Bridge	Girder	1 Lane		2+Lanes	
		With MPF	Without MPF	With MPF	Without MPF
HCEB	FIB1	0.800	0.667	0.672	0.672
	FIB2	0.467	0.389	0.678	0.678
	FIB3	0.468	0.390	0.680	0.680
	FIB4	0.468	0.390	0.680	0.680
	FIB5	0.800	0.667	0.670	0.670
LREB	FIB1	0.800	0.667	0.672	0.672
	FIB2	0.469	0.390	0.680	0.680
	FIB3	0.467	0.390	0.679	0.679
	FIB4	0.467	0.390	0.679	0.679
	FIB5	0.800	0.667	0.672	0.672
HCWB	FIB1	0.800	0.667	0.673	0.673
	FIB2	0.465	0.388	0.676	0.676
	FIB3	0.468	0.390	0.680	0.680
	FIB4	0.468	0.390	0.680	0.680
	FIB5	0.800	0.667	0.671	0.671
LRWB	FIB1	0.800	0.667	0.676	0.676
	FIB2	0.472	0.393	0.685	0.685
	FIB3	0.470	0.391	0.682	0.682
	FIB4	0.470	0.391	0.682	0.682
	FIB5	0.800	0.667	0.678	0.678

Note: FIB1 and FIB5 are exterior girders

All comparisons made in the upcoming sections include the MPF, which is 1.2 for one lane loaded and 1.0 for two lanes loaded. Also, the AASHTO DFs for two lanes are inherently doubled because in design they are multiplied by the effect (e.g., moment) due to 1 truck, typically obtained from a line analysis.

4.2.2 Distribution Factors from Measured Strains

To calculate the distribution of the truck load to each girder, the longitudinal strains measured on the bottom of the FIBs were first converted to stress. The strains in the two (2) side-by-side gauges on each FIB's underside were averaged together. Eq. 9 was used to convert the strain into stress, by multiplying it by the concrete's modulus of elasticity.

$$\sigma = E_{FIB}(\varepsilon) \qquad \text{Eq. 9}$$

where:

E_{FIB} = modulus of elasticity of FIB, ksi

σ = stress on bottom of girder, ksi

ε = measured strain on bottom of girder, in./in.

The transformed section properties of the composite girders and the stresses were used to calculate the moments in the girders, by multiplying the stress on the bottom of each FIB by the transformed moment of inertia and then dividing it by the distance from the girder bottom to the neutral axis, as shown in Eq. 10.

$$M = \frac{\sigma I}{c} \qquad \text{Eq. 10}$$

where:

M = moment, kip-in.

I = transformed moment of inertia of composite girder, in^4

c = distance from neutral axis of composite girder to bottom of girder, in.

For the tests where the barriers were present, the exterior girder section properties were calculated by including the barrier cross section. The transformed section properties that were used are presented in Table 4-1.

The moments in the girders were added together to get the total moment on the bridge cross section. These calculated moments at Section B are shown in **Appendix A** in **Tables A.1 – A.5**. Theoretically, these calculated values should be comparable to the theoretical moments calculated from the line analysis (shown in Table 4-2). However, the calculated moments at Section B (**Tables A.1 – A.5** in **Appendix A**) were much lower than the theoretical values (Table 4-2) by roughly 20% to 25%. The difference could be from stiffer materials than expected (i.e. higher modulus of elasticity than calculated) or from rotational restraint provided by the bearings (whereas the theoretical assumption was that the bearings act as pins and rollers).

To analyze the distribution of load to each girder, the moment in each girder was divided by the sum of the moments in all the girders, as shown in Eq. 11.

$$DF_{FIB} = \frac{M_{FIB}}{\sum_{i=1}^{5} M_{FIB_i}} \qquad \text{Eq. 11}$$

where:

M_{FIB_i} = moment in each girder, kip-ft

The DFs were then multiplied by the multiple presence factor, and when 2 trucks were on the bridge, the DFs were also multiplied by 2.0. This DF will be referred to as "moment distribution" in following sections. These calculated DFs at Section B are shown in **Appendix B** in **Tables B.1 – B.5**, and they are compared to the theoretical DFs (Table 4-3) in several figures presented in following sections.

4.2.3 Distribution Factors from Measured Deflections

The measured deflections were converted into distribution factors DF_Δ using Eq. 12. This equation was derived from the deflection equation $\Delta = \frac{PL^3}{48EI}$ for a simply supported span with a point load at mid span. Assuming that the modulus of elasticity E and span length L are constants, the load P carried by each girder is proportional to ΔI, the deflection and the moment of inertia of the composite FIB.

$$DF_\Delta = \frac{\Delta I_c}{\sum_{i=1}^{5}(\Delta_i I_{ci})} \qquad \text{Eq. 12}$$

where:

Δ_i = deflection of individual girder, in.

I_{ci} = composite moment of inertia of an individual girder, in^4

Values for I_c are provided in Table 4-1. The DFs were then multiplied by the multiple presence factor, and, when two (2) trucks were on the bridge, the DFs were also multiplied by 2.0. This DF will be referred to as "deflection distribution" in following sections. These calculated DFs at Section B are shown in **Appendix C** in **Tables C.1 – C.4**, and they are compared to the theoretical DFs (Table 4-3) in several figures presented in following sections.

4.3 Pre-Service, Prestressed Panel Bridges vs. Non-Prestressed Panel Bridges

Comparisons were made between the **prestressed** panel bridges (HCWB and LRWB) and the **non-prestressed** panel bridges (HCEB and LREB), with regard to their deflections, longitudinal strains, distribution factors, and transverse strains.

4.3.1 Measured Deflection Comparisons

In this section, the measured deflections for the prestressed and non-prestressed panel bridges are compared. Only load cases 1/1A and 2 were analyzed.

Figures 4-4 to 4-7 show that the westbound bridges (with prestressed panels) deflected similarly to the eastbound bridges (with non-prestressed panels), with very little difference in deflections. (For example, from Figure 4-7, LREB had a maximum deflection on FIB5 of 0.441 in., and LRWB had a maximum deflection on FIB5 of 0.453 in.) This was the case for both Hurricane Creek and Little River bridges, and for both load positions analyzed (1A and 2). The 24-block case was analyzed, but the other block cases (12 and 18) also showed that the westbound and eastbound bridges behaved similarly.

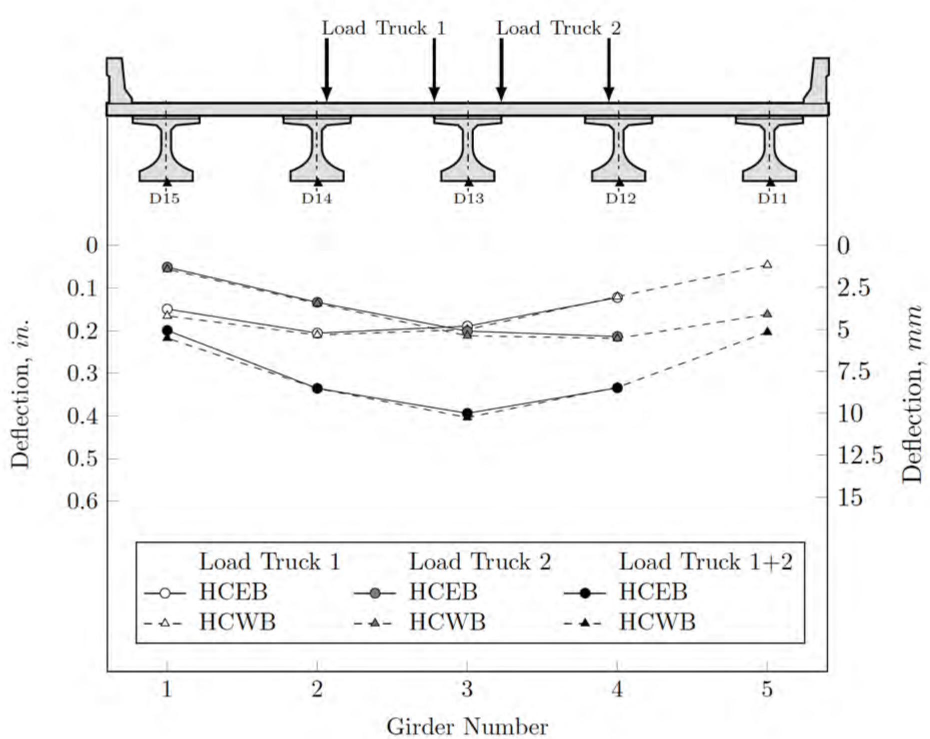


Fig. 4-4. Deflection HCWB vs. HCEB - LP 1A - 24 Blocks

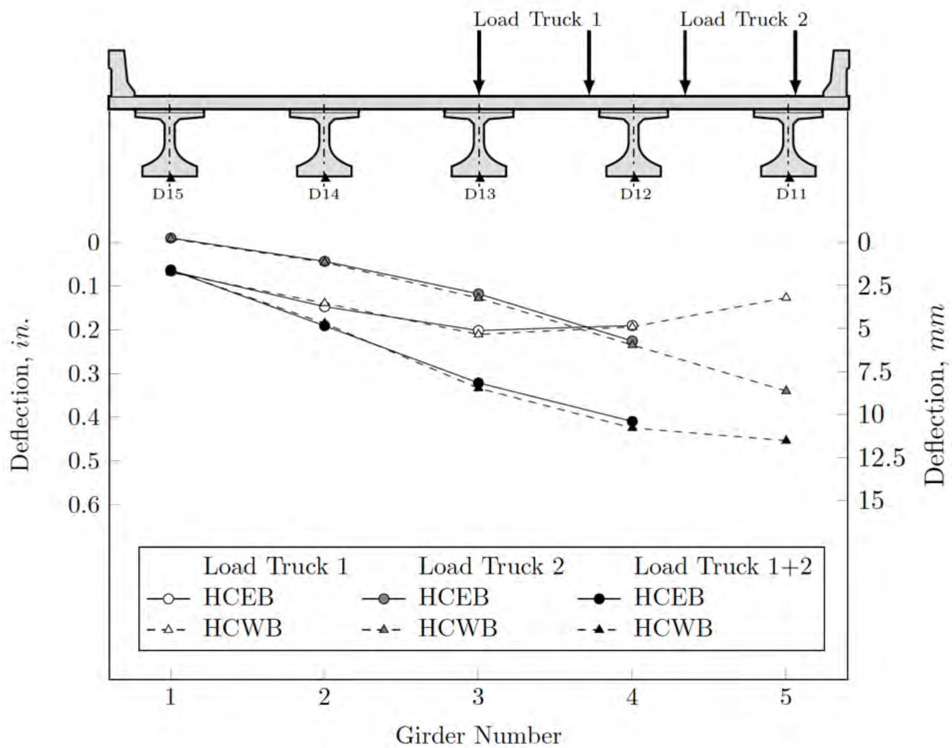


Fig. 4-5. Deflection HCWB vs. HCEB - LP 2 - 24 Blocks

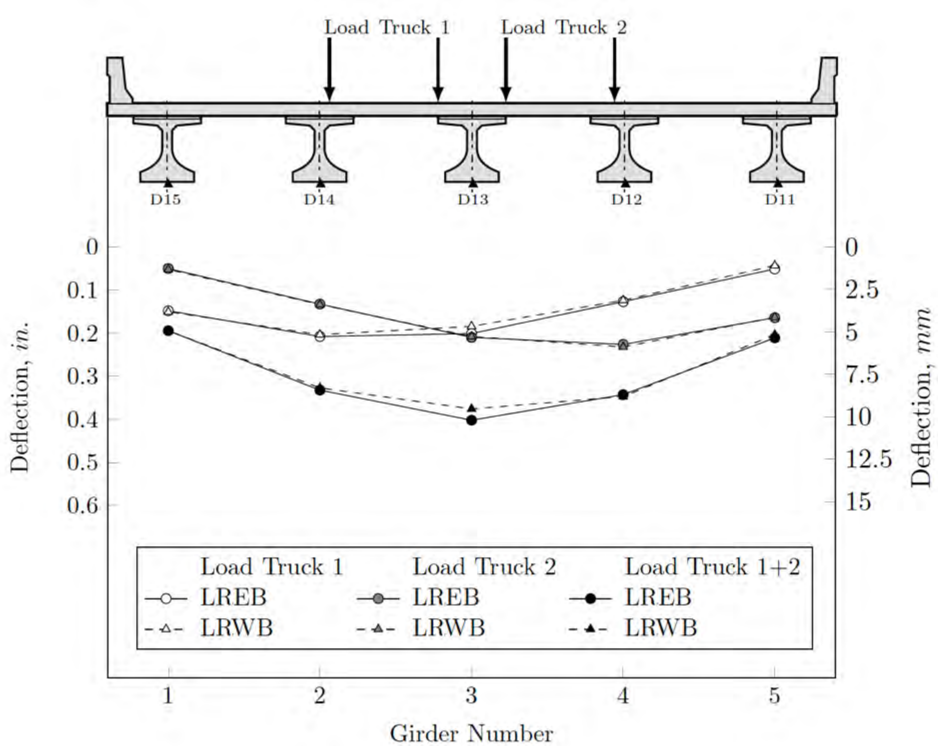


Fig. 4-6. Deflection LRWB vs. LREB - LP 1A - 24 Blocks

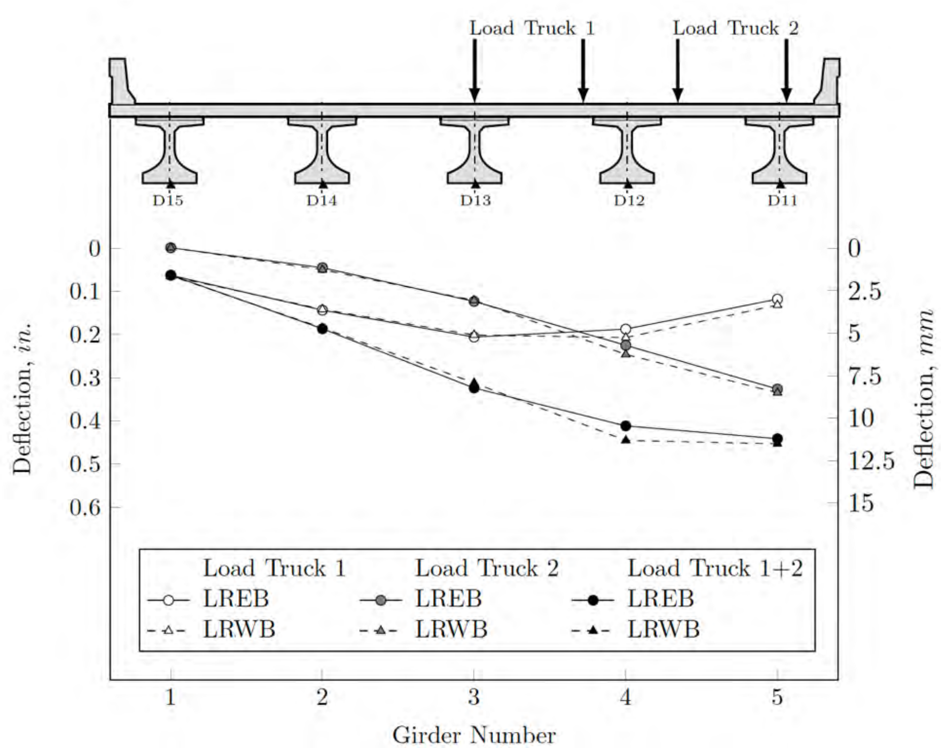


Fig. 4-7. Deflection LRWB vs. LREB - LP 2 - 24 Blocks

4.3.2 Longitudinal Strain Comparisons

This section presents a comparison of the longitudinal strains at section B for the prestressed and non-prestressed panel bridges. Only load positions 1A and 2 were analyzed.

The longitudinal strains for all the bridges were expected to be approximately the same other than small differences due to varying material and section properties. **Figures 4-8 to 4-11** show the strains in the bottom of the girders for Hurricane Creek and Little River bridges, for the 24-block case. In general, the westbound (prestressed panel) bridges had similar strains as the eastbound (non-prestressed panel) bridges. For example, when the load was at position 2, the strains in FIB4 and 5 were within 6% of each other, as shown in Figures 4-9 and 4-11. A larger variance was seen in HCWB at FIB4, due to a faulty gauge at this position (F2).

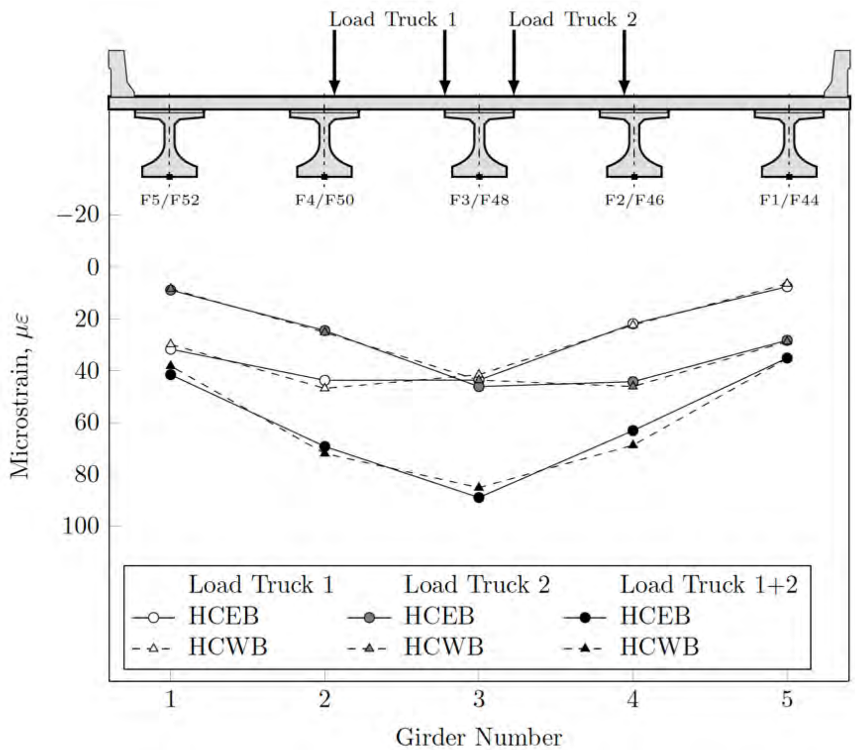


Fig. 4-8. Longitudinal Strain – HCWB vs. HCEB - LP 1A - 24 Blocks

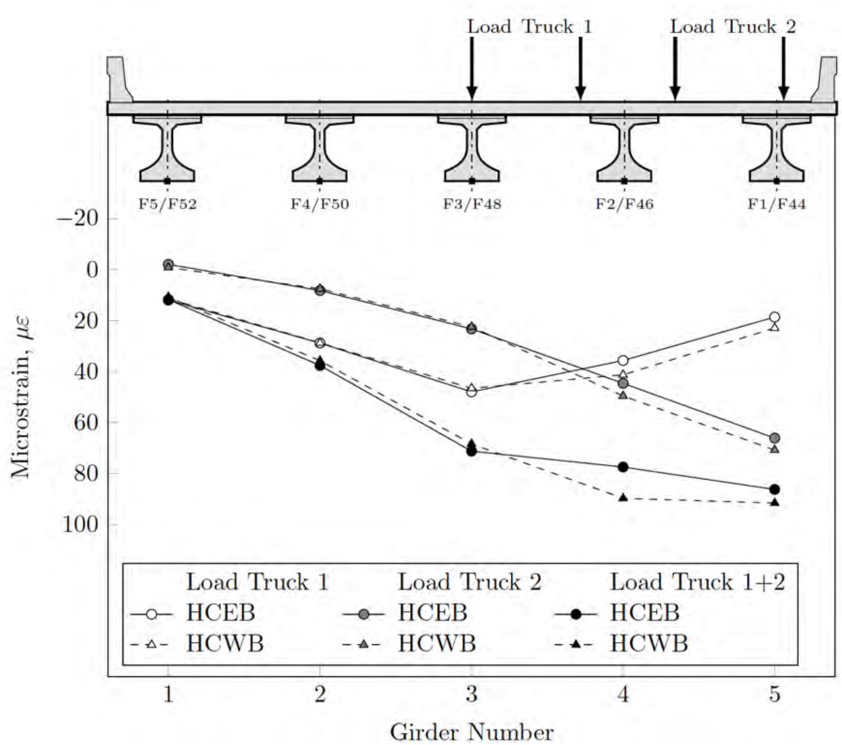


Fig. 4-9. Longitudinal Strain – HCWB vs. HCEB - LP 2 - 24 Blocks

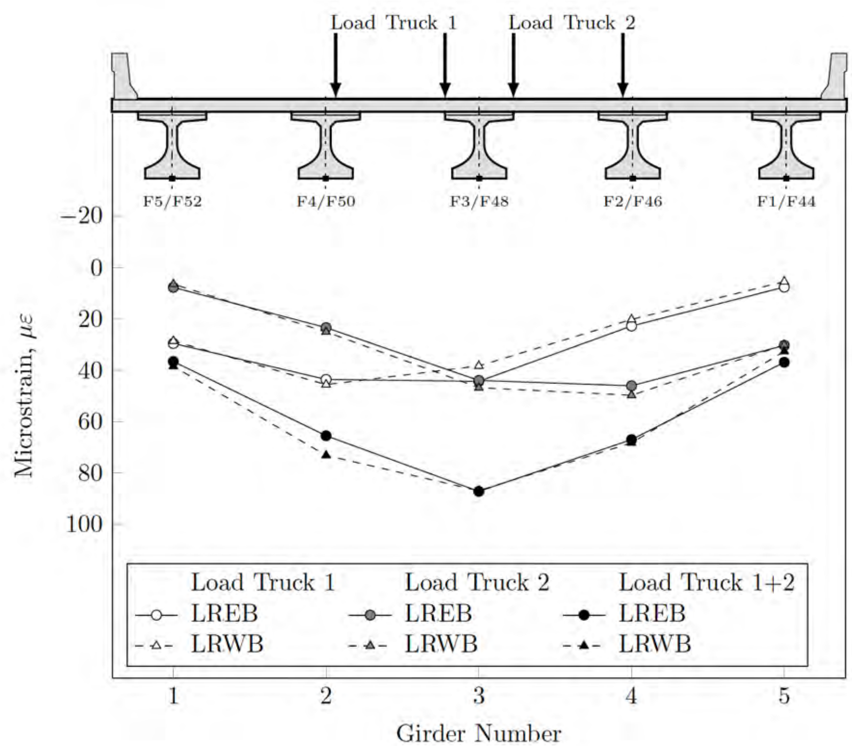


Fig. 4-10. Longitudinal Strain – LRWB vs. LREB - LP 1A - 24 Blocks

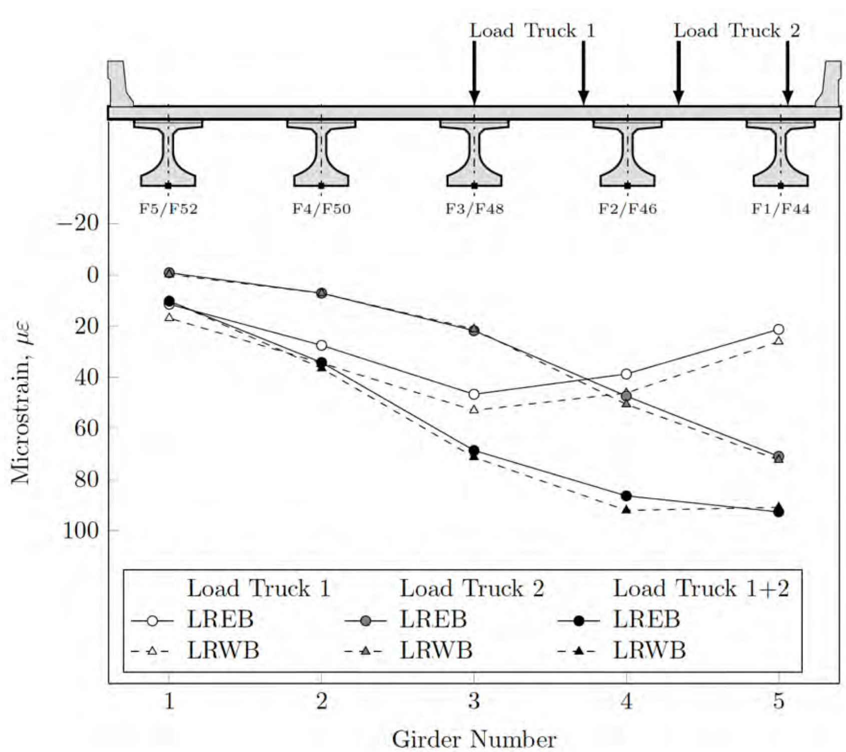


Fig. 4-11. Longitudinal Strain – LRWB vs. LREB - LP 2 - 24 Blocks

4.3.3 Distribution Factors from Measured Strains Comparisons

This section compares the distribution factors for prestressed and non-prestressed panel bridges for the load tests performed after the barriers were constructed. The distribution factors were calculated from the measured strains and corresponding calculated moments, as discussed previously. Also, the bridges had similar material properties as presented in a previous section, so they were expected to behave very similarly from a load distribution perspective.

The trends in the distribution factors for the eastbound and westbound bridges were almost identical, with only slight variance between the bridges, as shown in **Figures 4-12 to 4-19**.

For one truck loaded, the AASHTO DFs were conservative by 28% for the first interior girder and 21% for the exterior girder, on average for all bridges when the trucks were placed at load position 2. The only case when the DF exceeded the AASHTO DF was when both trucks were loaded and placed at load position 2, as shown in Figures 4-15 and 4-19. As explained previously, the AASHTO equations do not include the effect of the barrier stiffness. The DFs calculated from the load tests, however, included the barrier in the exterior girder section property calculations.

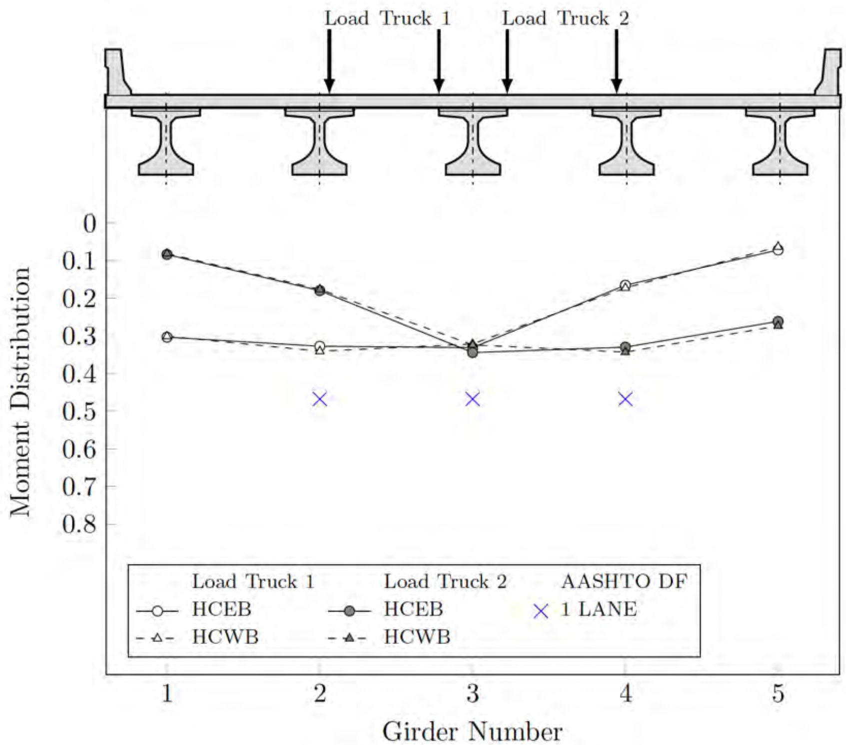


Fig. 4-12. Moment Distribution HCEB vs. HCWB - LP 1A - 24 Blocks with MPF - 1 lane

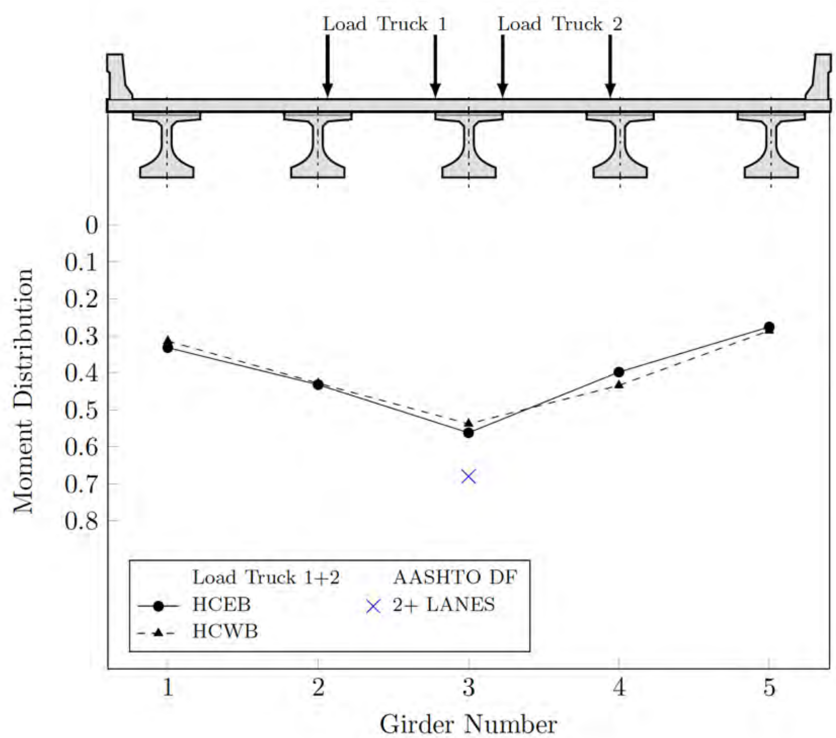


Fig. 4-13. Moment Distribution HCEB vs. HCWB - LP 1A - 24 Blocks with MPF - 2 lanes

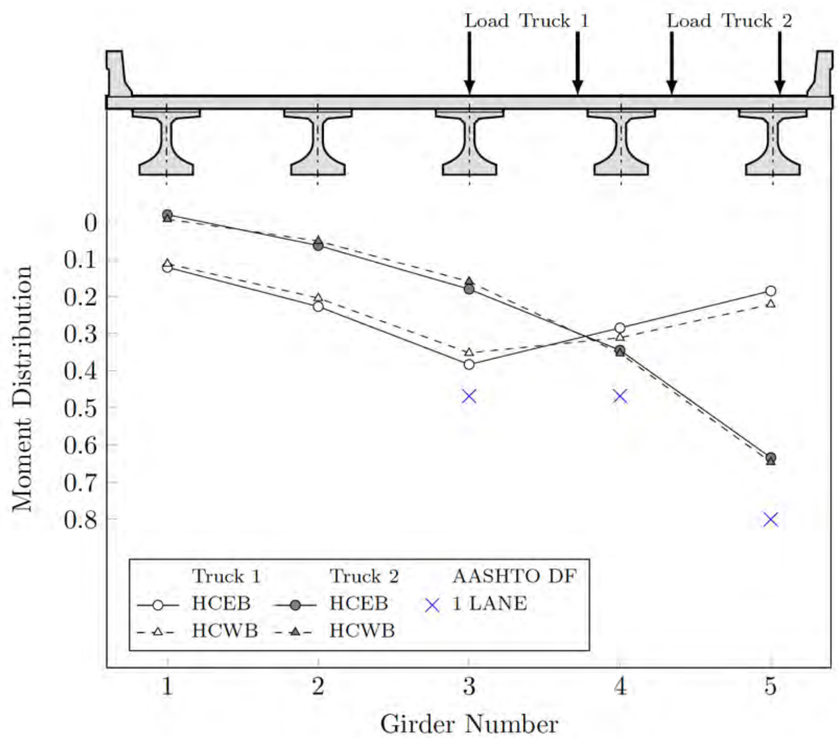


Fig. 4-14. Moment Distribution HCEB vs. HCWB - LP 2 - 24 Blocks with MPF - 1 lane

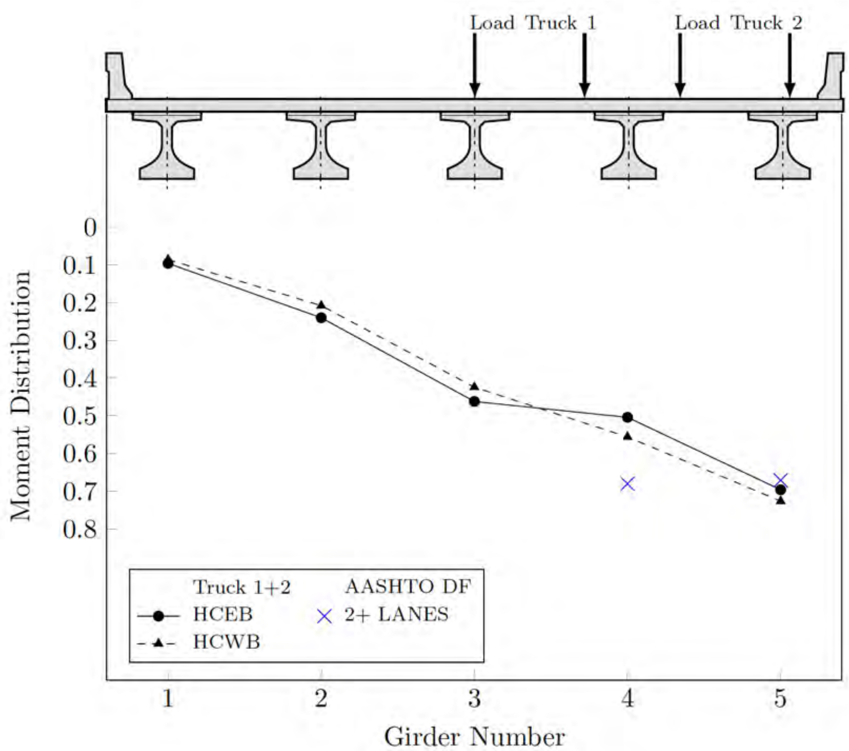


Fig. 4-15. Moment Distribution HCEB vs. HCWB - LP 2 - 24 Blocks with MPF - 2 lanes

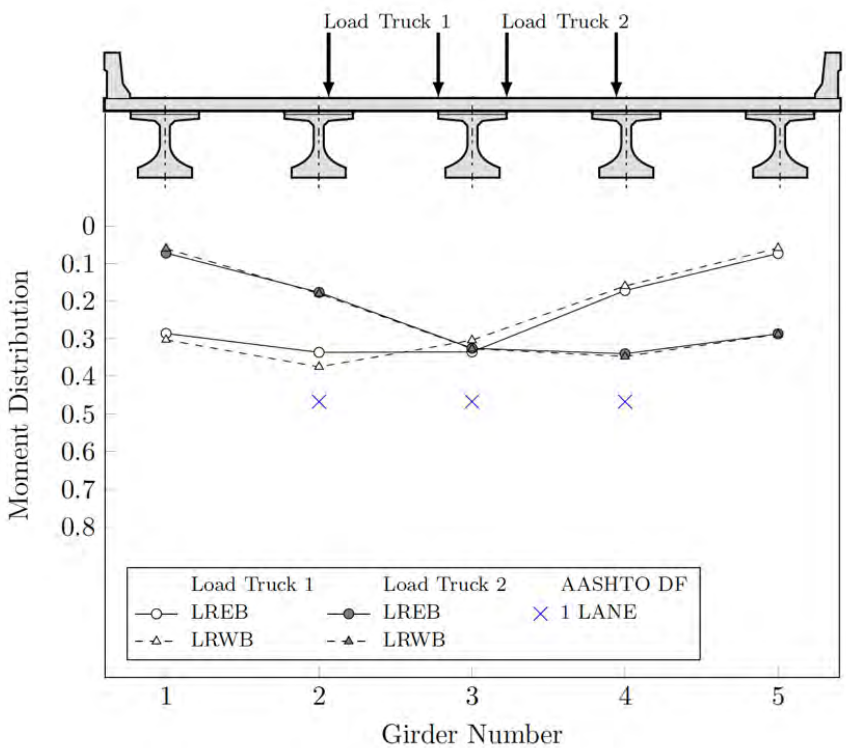


Fig. 4-16. Moment Distribution LREB vs. LRWB - LP 1A - 24 Blocks with MPF - 1 lane

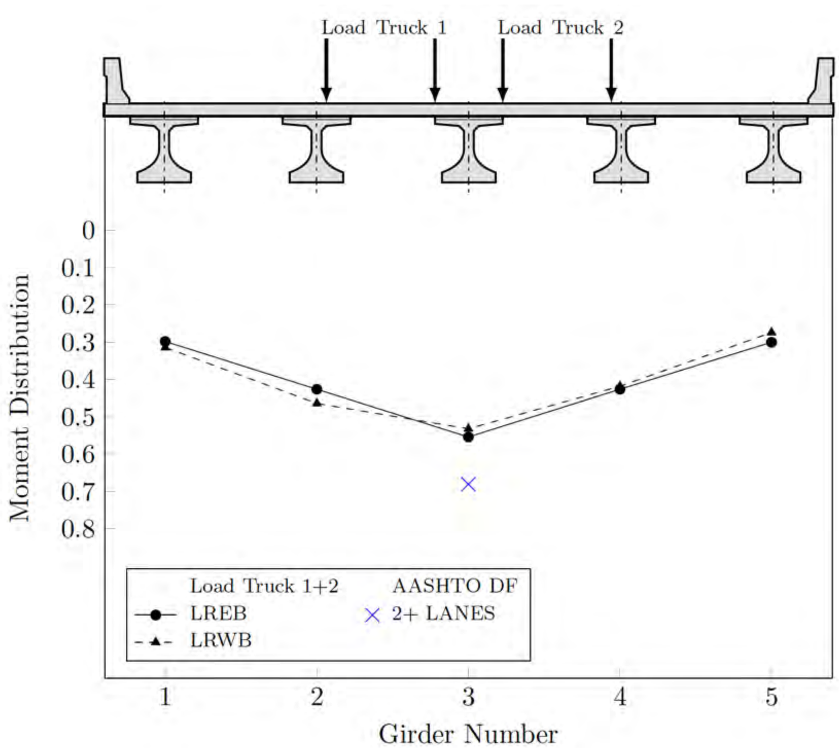


Fig. 4-17. Moment Distribution LREB vs. LRWB - LP 1A - 24 Blocks with MPF - 2 lanes

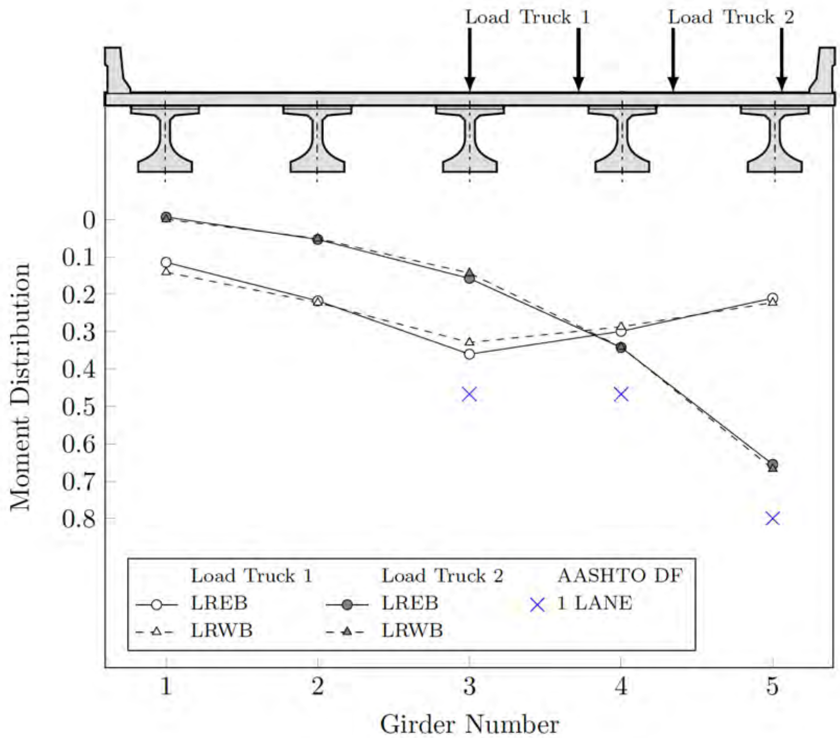


Fig. 4-18. Moment Distribution LREB vs. LRWB - LP 2 - 24 Blocks with MPF - 1 lane

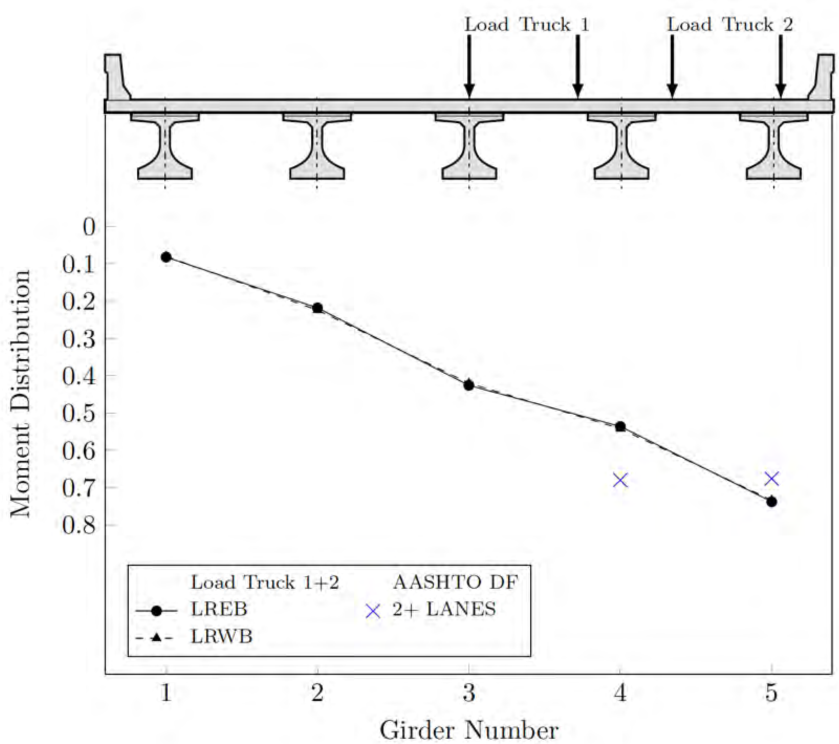


Fig. 4-19. Moment Distribution LREB vs. LRWB - LP 2 - 24 Blocks with MPF - 2 lanes

4.3.4 Distribution Factors from Measured Deflections Comparisons

This section presents a comparison of the deflection distribution for the prestressed panel and non-prestressed panel bridges. Because HCEB deflection gauge D11 malfunctioned during the test, a comparison between HCWB and HCEB was not made; however, **Figures 4-20 to 4-23** show the DFs for HCWB. **Figures 4-24 to 4-27** show the DFs for LRWB and LREB.

The AASHTO predictions were mostly conservative. However, load position 2 with both trucks loaded exceeded the AASHTO predictions, as shown in Figure 4-23. The DF for FIB5 was 16% greater, whereas the DF for FIB4 was 32% less than the AASHTO predictions.

Little River bridges behaved similarly to HCWB, where the first interior girder (FIB4) DF was 26% less than, whereas FIB5 was 15% more than AASHTO predictions (Figure 4-27).

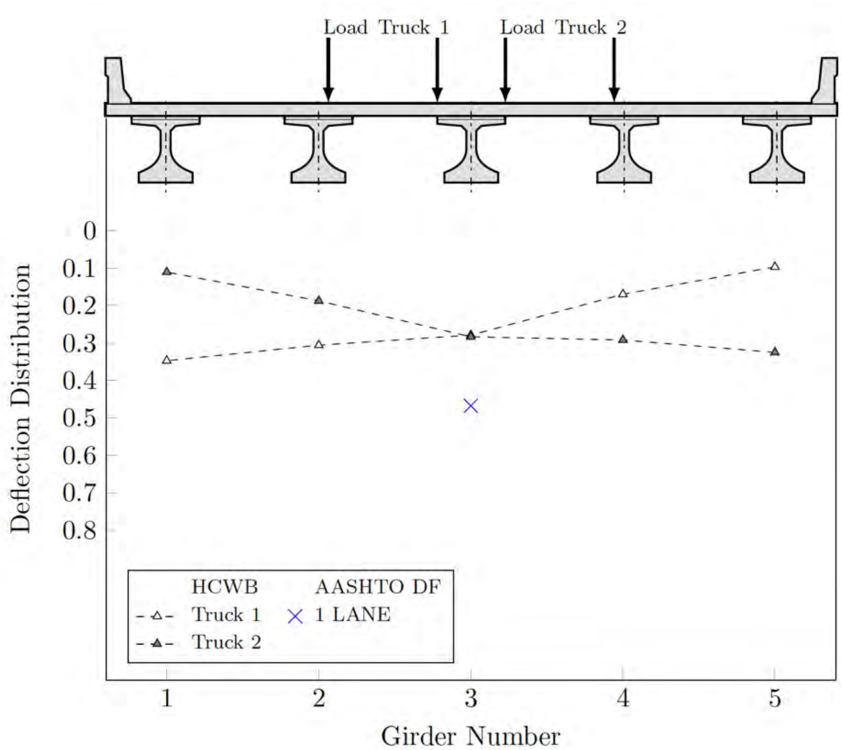


Fig. 4-20. Deflection Distribution HCWB - LP 1A - 24 Blocks - 1 lane

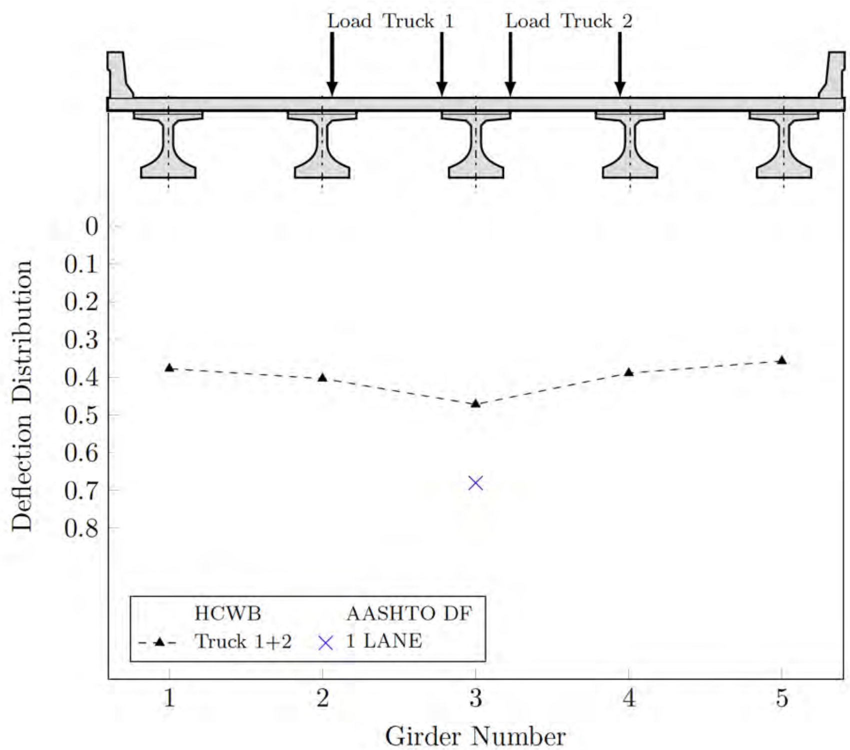


Fig. 4-21. Deflection Distribution HCWB - LP 1A - 24 Blocks - 2 lanes

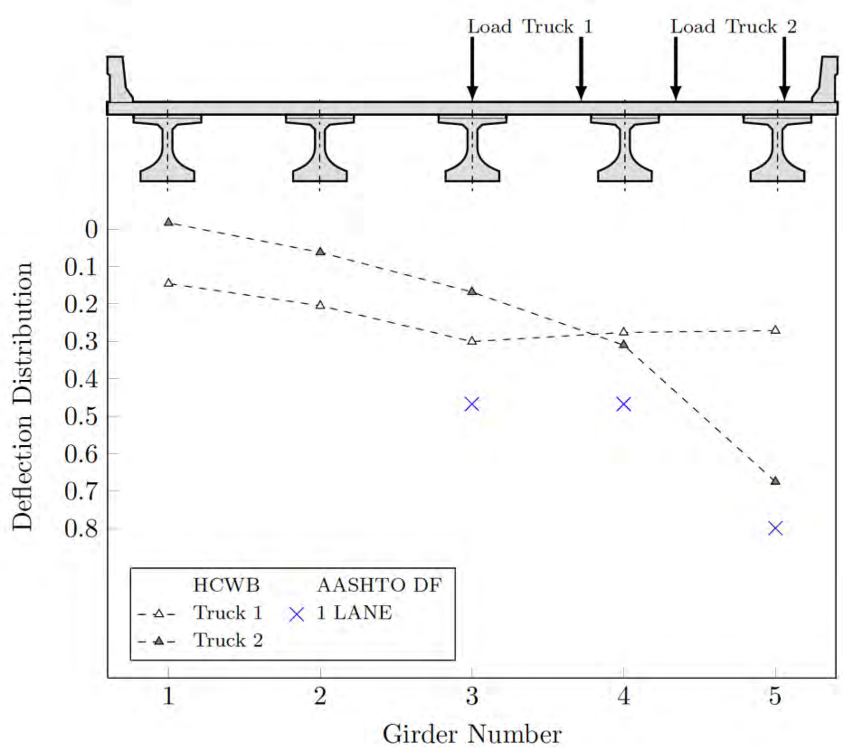


Fig. 4-22. Deflection Distribution HCWB - LP 2 - 24 Blocks - 1 lane

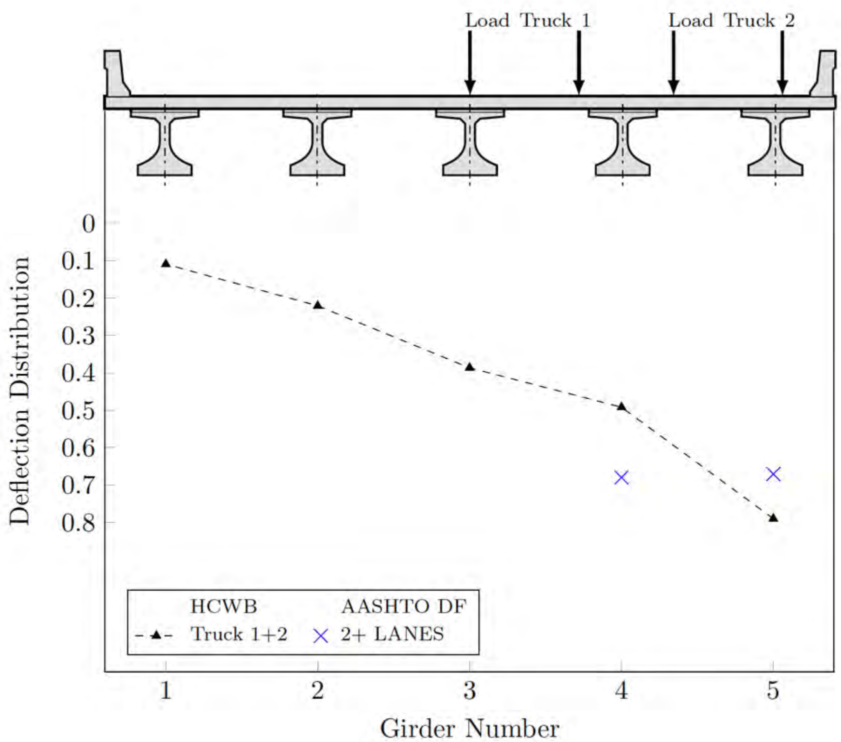


Fig. 4-23. Deflection Distribution HCWB - LP 2 - 24 Blocks - 2 lanes

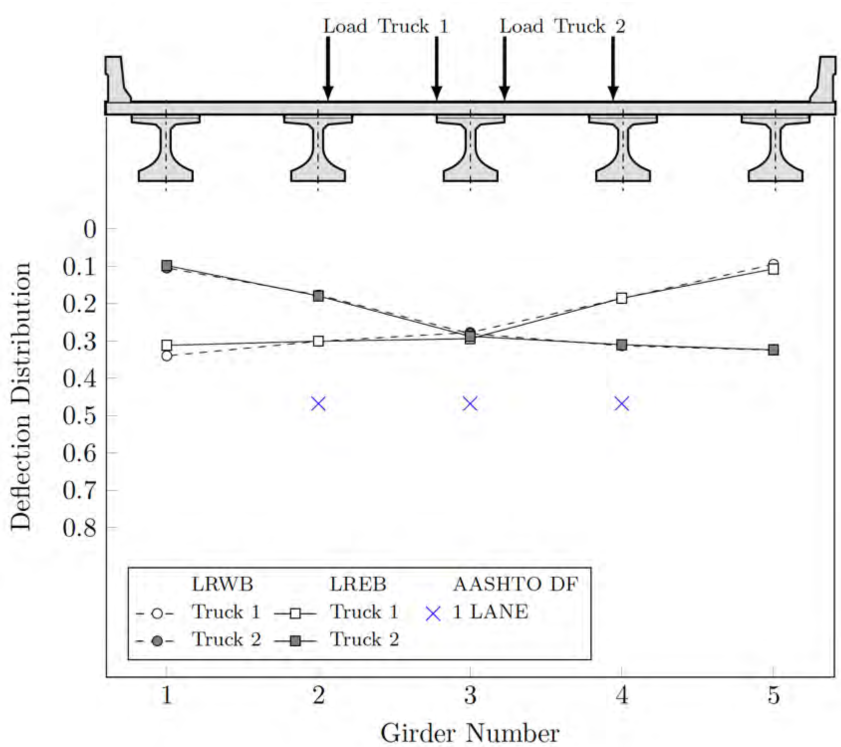


Fig. 4-24. Deflection Distribution LRWB vs. LREB - LP 1A - 24 Blocks - 1 lane

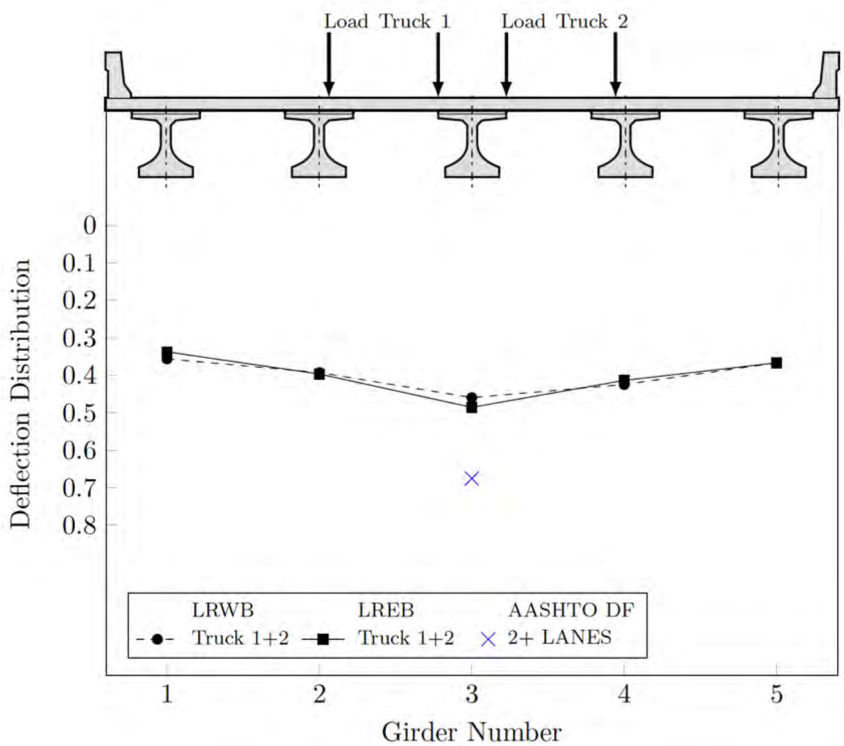


Fig. 4-25. Deflection Distribution LRWB vs. LREB - LP 1A - 24 Blocks - 2 lanes

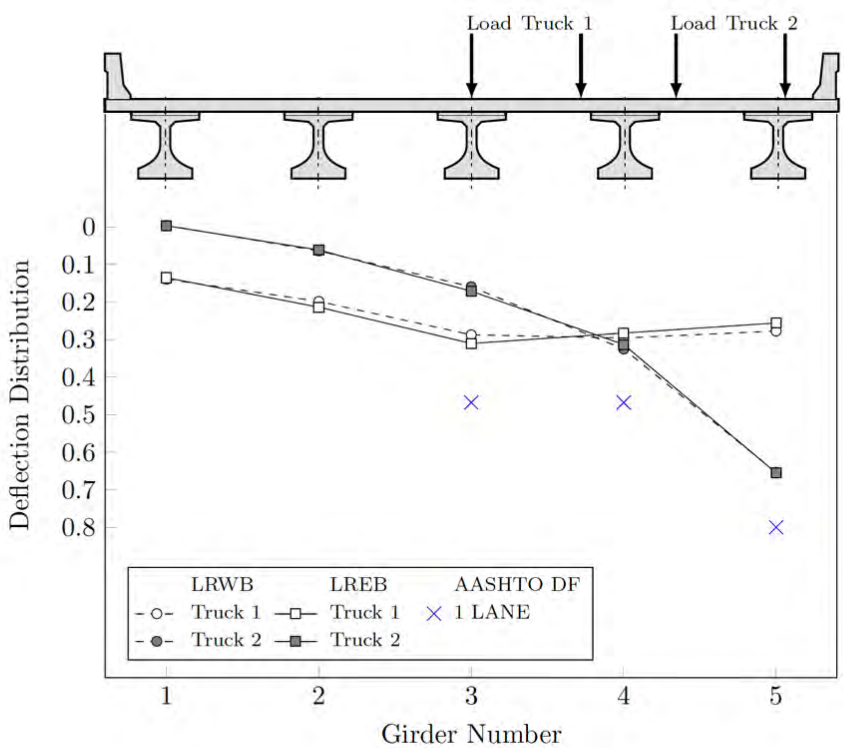


Fig. 4-26. Deflection Distribution LRWB vs. LREB - LP 2 - 24 Blocks - 1 lane

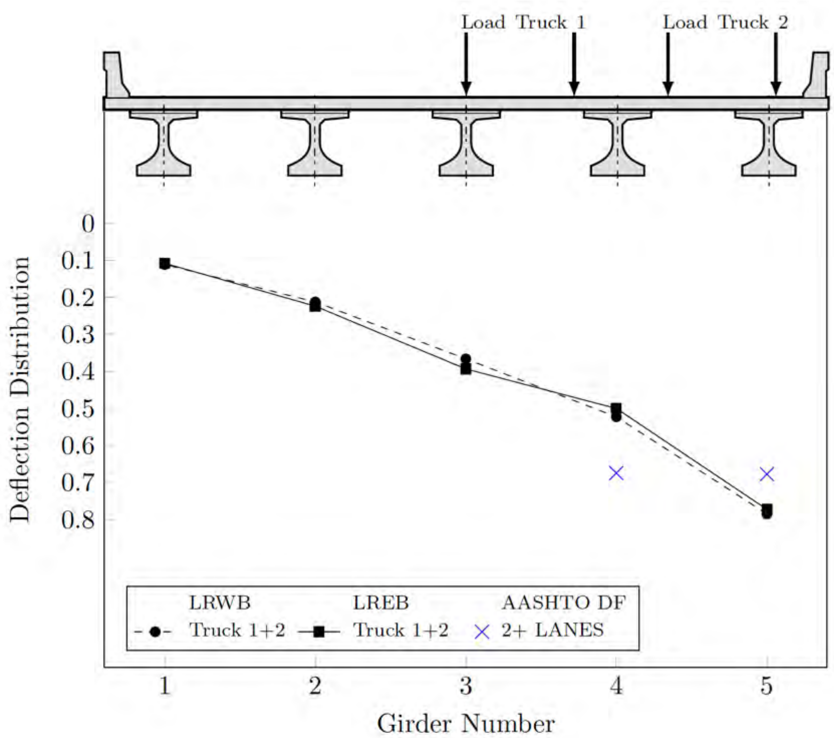


Fig. 4-27. Deflection Distribution LRWB vs. LREB - LP 2 - 24 Blocks - 2 lanes

4.3.5 Transverse Strain Comparison

The transverse strain data was compared for four (4) load positions: 1/1A, 1B, 3/3A, and 3B. However, for HCEB, the "B" load positions were not tested and therefore were not compared. The purpose of comparing these transverse strains was to evaluate if the prestressed and non-prestressed deck panels behaved similarly; to assess whether the cast-in-place joint (Section C) behaved similarly to the high-strength concrete deck panels (Section B); and to determine if the truck load was being shared equally between the joint and panel.

First, the Hurricane Creek WB and EB bridges were compared to each other (**Figures 4-28 to 4-31**). For the most part, the strains matched up. However, for load positions 1/1A and 3/3A, there was discrepancy in gauge B11 between FIB2 and FIB3. At this location, the transverse strain was less in the non-prestressed deck panel (EB) than in the prestressed panel (WB), for load truck 1, load truck 2, and both truck positions, as shown in Figure 4-

28. The strains in the other gauges for HCEB were similar to HCWB, so it is possible that gauge B11 malfunctioned during the HCEB test.

Next, HCEB and HCWB were compared when the trucks were placed at load position 3/3A, which asymmetrically loaded the bridge transversely. Figure 4-29 shows similar trends to load position 1/1A discussed above, where gauge B11 showed inconsistent results. The strains in gauge B11 for HCEB, when truck 1 was on the bridge, were less than for HCWB. Other than that, the other gauge readings were similar.

Figures 4-30 and 4-31 compare the transverse strains in the Hurricane Creek bridges at section C, a concrete joint. These results match up almost identically, when 24 blocks are loaded. The largest difference was for load position 3/3A, where the eastbound bridge (non-prestressed) had slightly lower transverse strains in gauge B8 of around 12%, compared to the westbound bridge (prestressed).

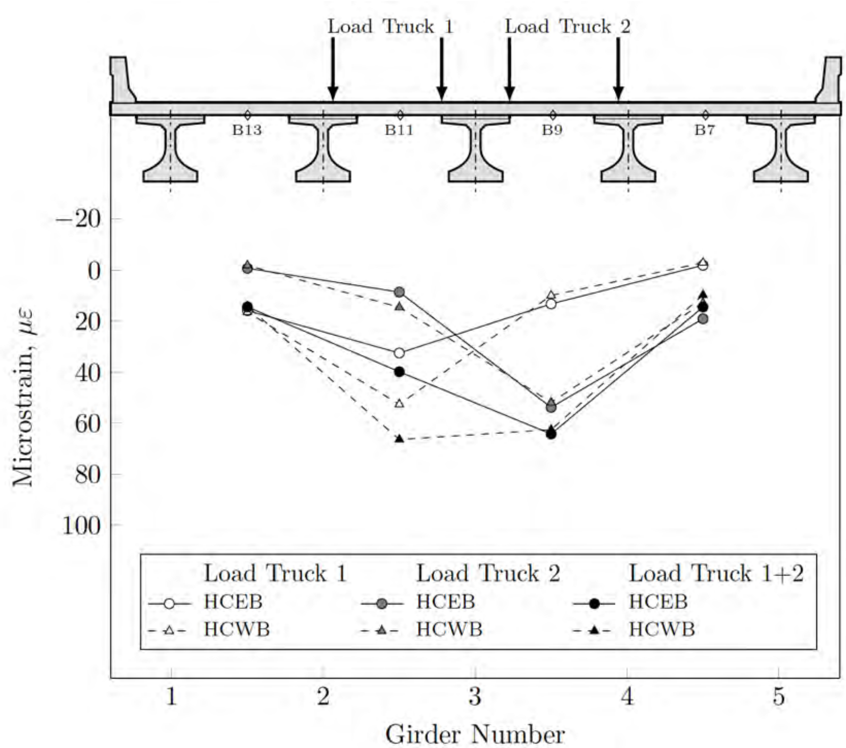


Fig. 4-28. Transverse Strain - HCWB vs. HCEB - LP 1/1A - 24 Blocks - Section B

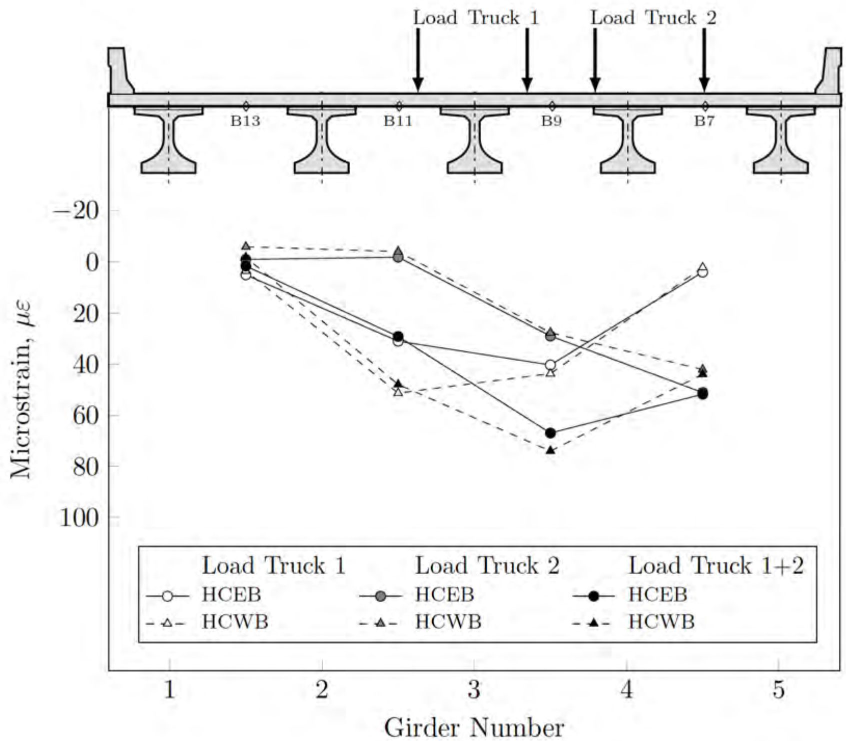


Fig. 4-29. Transverse Strain - HCWB vs. HCEB - LP 3/3A - 24 Blocks - Section B

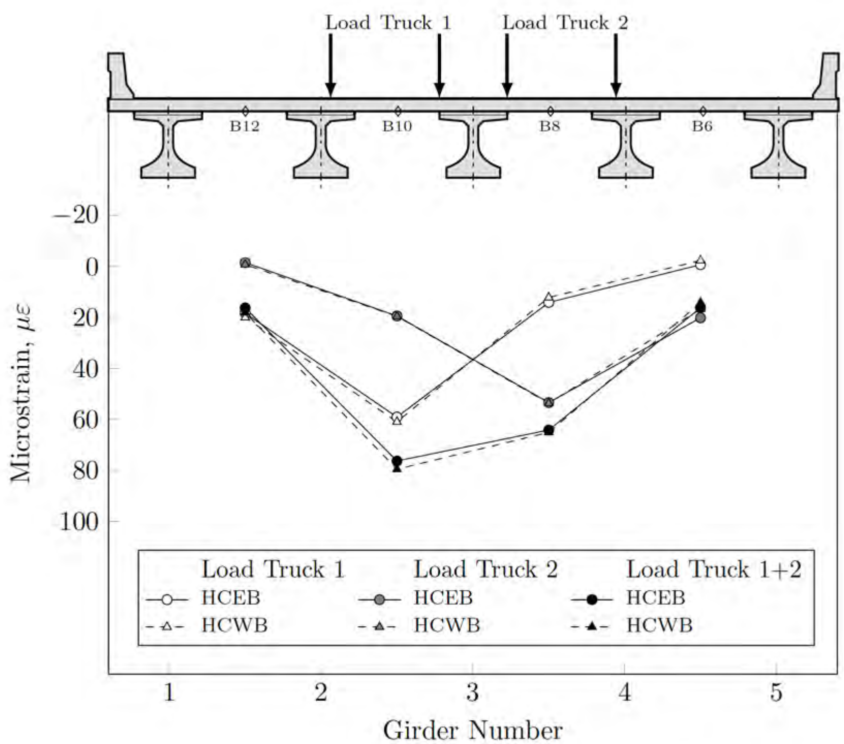


Fig. 4-30. Transverse Strain - HCWB vs. HCEB - LP 1/1A - 24 Blocks - Section C

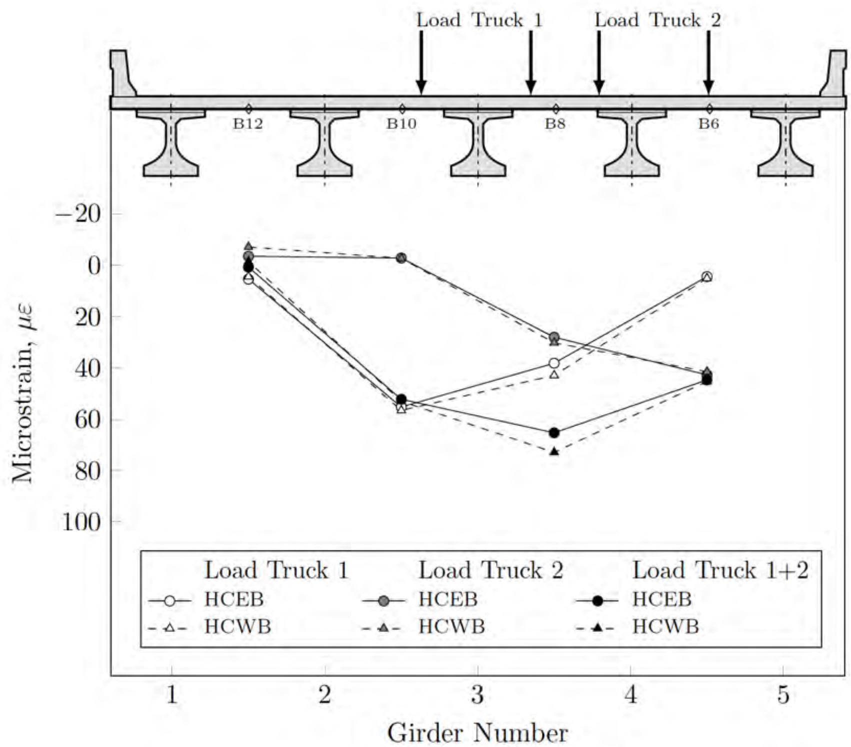


Fig. 4-31. Transverse Strain - HCWB vs. HCEB - LP 3/3A - 24 Blocks - Section C

The Little River bridges were compared to each other, for four (4) load positions (1A, 1B, 3A, and 3B). **Figures 4-32 and 4-33** show similar trends for load positions 1A and 1B at section B. For load position 1A, however, the load was placed much closer longitudinally to section B than for load position 1B. Therefore, as expected, the transverse strains were higher at this location for load position 1A than for 1B. Gauge B9 showed unexpected lower strains for LRWB compared to LREB.

Load positions 3A and 3B for the Little River bridges gave some odd results at gauge B9 as for load positions 1A and 1B discussed above. Gauge B9 had much lower strains for LRWB compared to LREB, but only when the truck 1 was placed on the bridge. The results from the other gauges matched closely for both bridges (**Figures 4-34 and 4-35**).

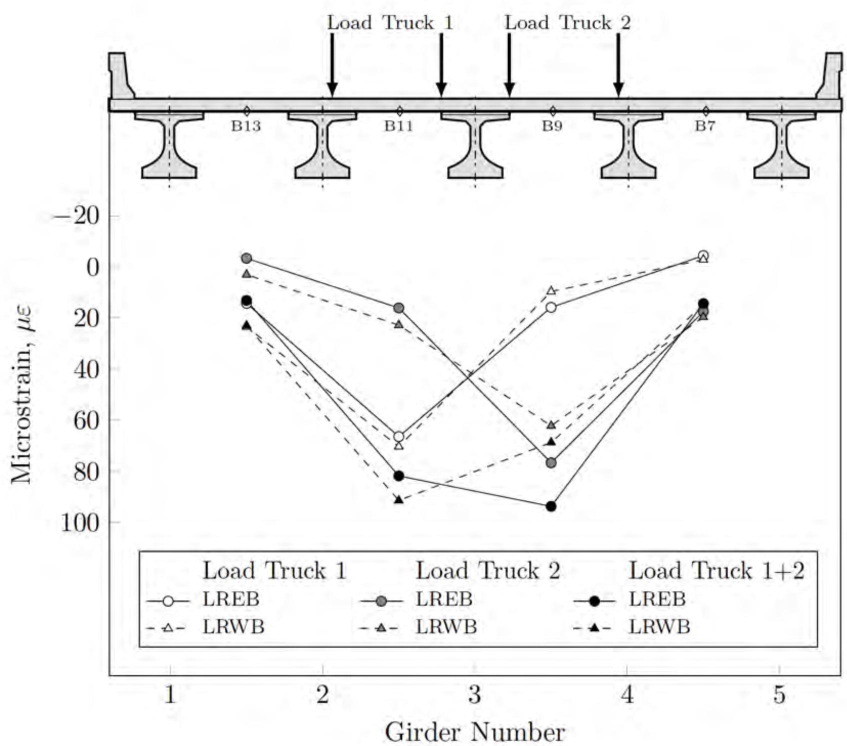


Fig. 4-32. Transverse Strain - LRWB vs. LREB - LP 1A - 24 Blocks - Section B

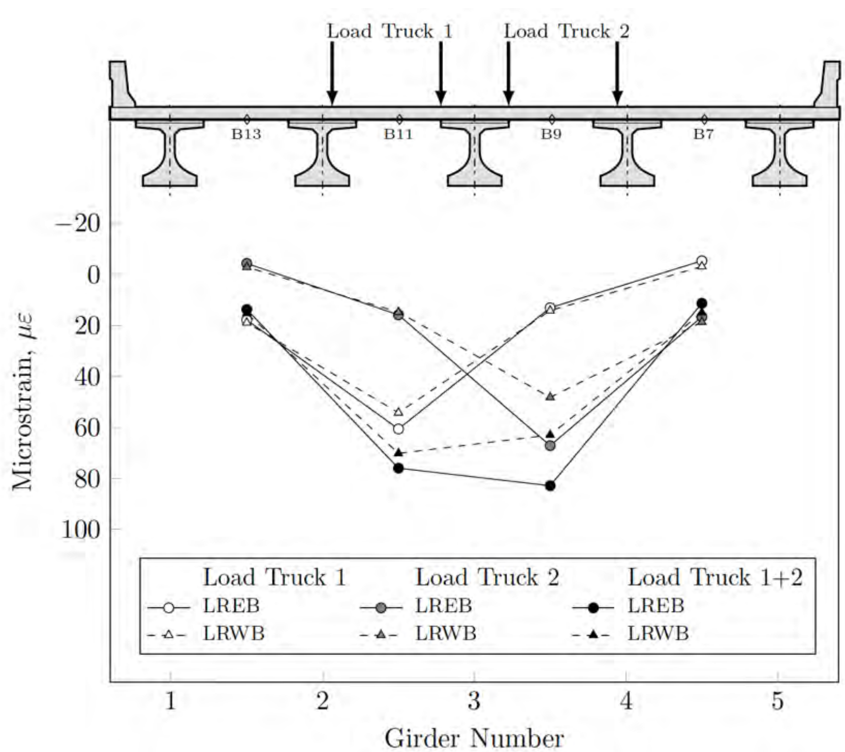


Fig. 4-33. Transverse Strain - LRWB vs. LREB - LP 1B - 24 Blocks - Section B

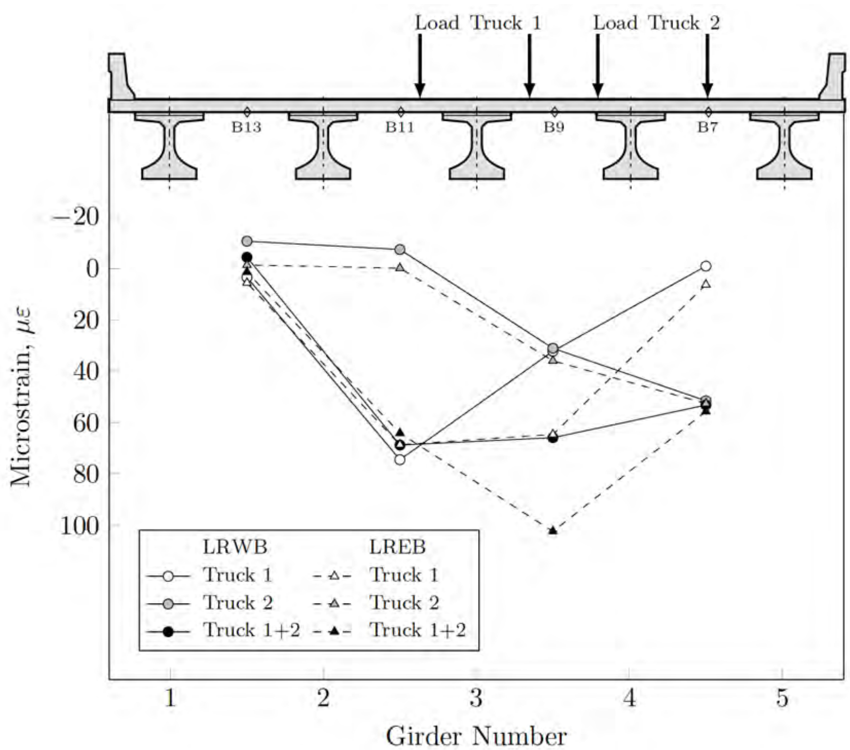


Fig. 4-34. Transverse Strain - LRWB vs. LREB - LP 3A - 24 Blocks - Section B

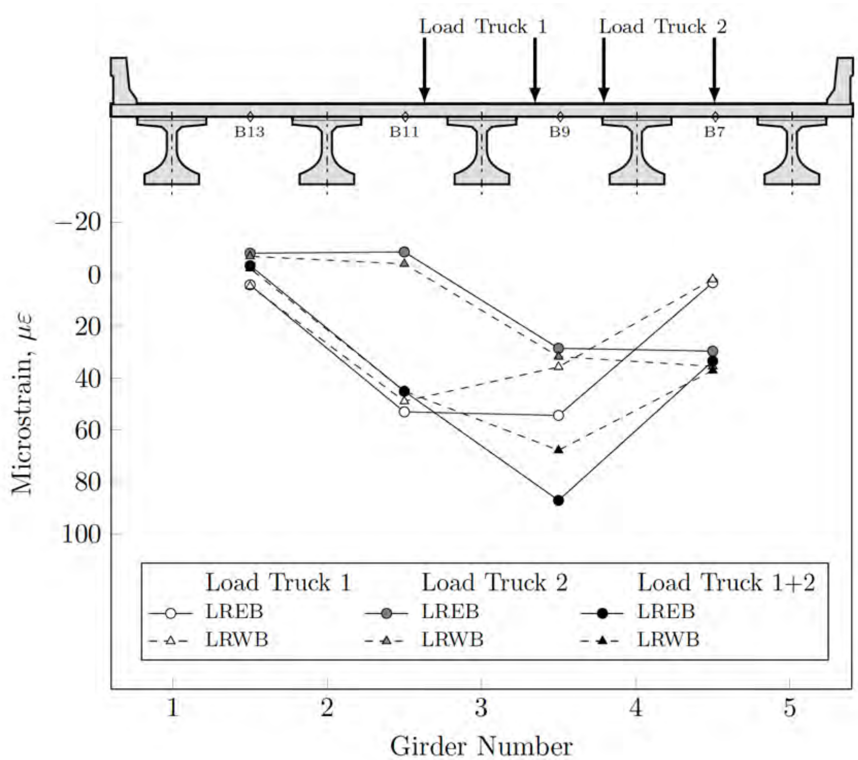


Fig. 4-35. Transverse Strain - LRWB vs. LREB - LP 3B - 24 Blocks - Section B

4.3.6 Joint vs. Panel – (Transverse Strain Comparison)

To compare the transverse strains in the precast panels relative to those in the cast-in-place joints, section B and section C results were plotted together. Load position 3B, where the axles longitudinally "straddled" the joint and panel, equally loaded the joint and the panel. HCEB was not tested with load position 3B; alternatively, test step 46a was performed which was equivalent to load position 3B when truck 2 was loaded. Comparing sections B and C for this case, the values very closely match each other except in between FIB4 and 5, where the joint experienced greater strains by 13.5%, as shown in **Figure 4-36**.

The next bridge analyzed was LREB. Unfortunately, a gauge on the joint malfunctioned during the load test, so no conclusion could be made with regard to the location in between FIB3 and 4, as shown in **Figures 4-37 and 4-38**.

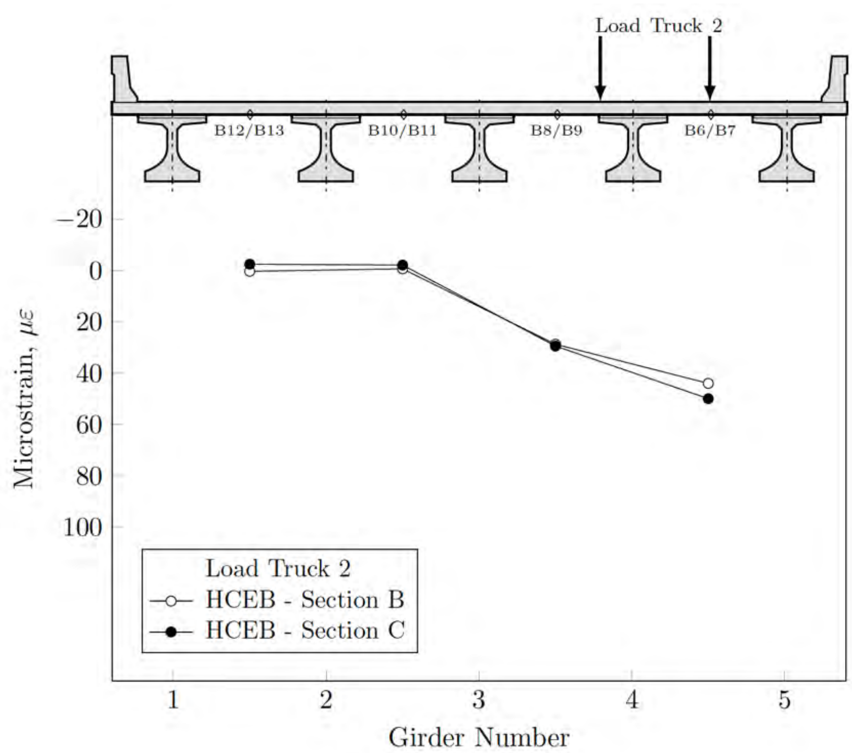


Fig. 4-36. Transverse Strain - HCEB - Test Step 46a - 24 Blocks - Section B/C

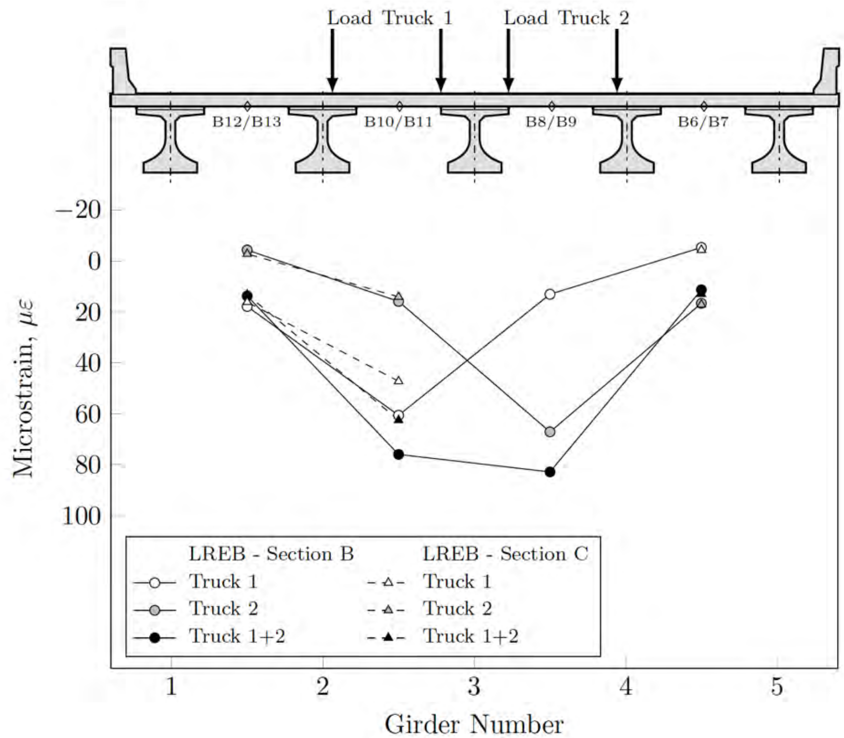


Fig. 4-37. Transverse Strain - LREB - LP 1B - 24 Blocks - Section B/C

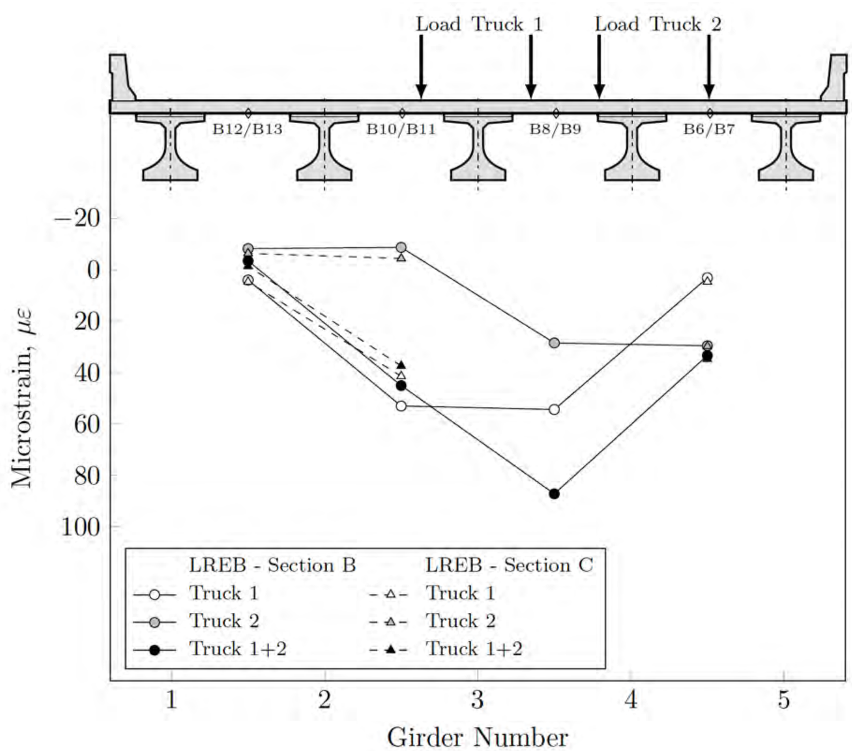


Fig. 4-38. Transverse Strain - LREB - LP 3B - 24 Blocks - Section B/C

Whenever truck 1 was loaded on the bridge the strains in between girder 2 and girder 3 were less at the joint compared to the middle of the panel for all load positions and in between the exterior and first interior girders the strains were very close for all load positions. Again, no assumptions could be made at the location of gauge B8, because it went bad. This is true for all the graphs associated with section C of LREB.

Figures 4-39 and 4-40 compare the joint and panel strains for HCWB. The joint (section C) had slightly higher strains in all the gauges, around 6%. This could be due to a lower modulus of elasticity for the cast-in-place joint concrete than for the precast panel concrete.

Lastly, LRWB was analyzed, as shown in **Figures 4-41 and 4-42**. Sections B and C behaved almost identically for all block cases.

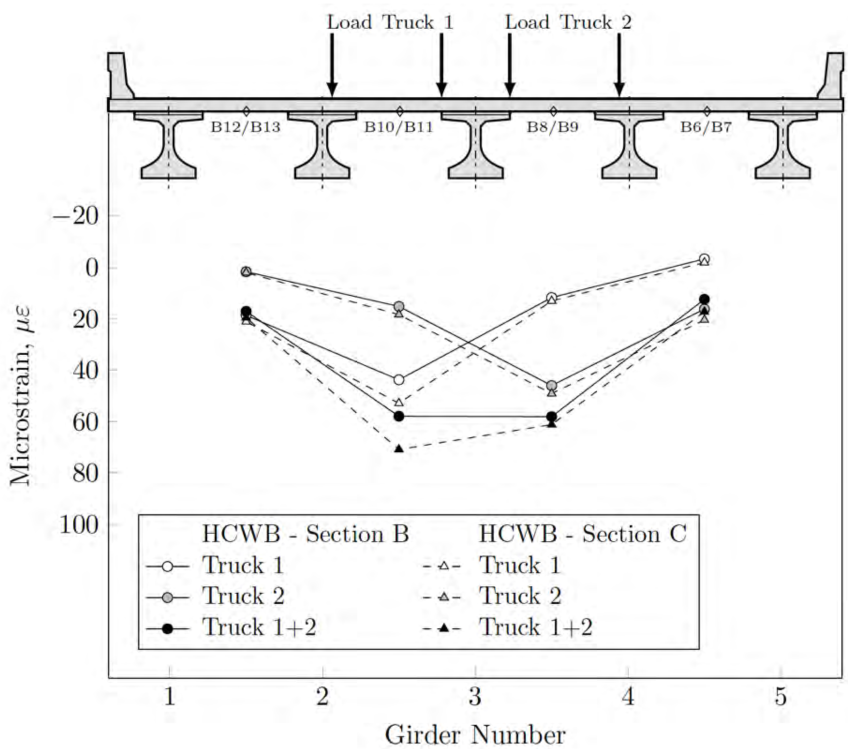


Fig. 4-39. Transverse Strain - HCWB - LP 1B - 24 Blocks - Section B/C

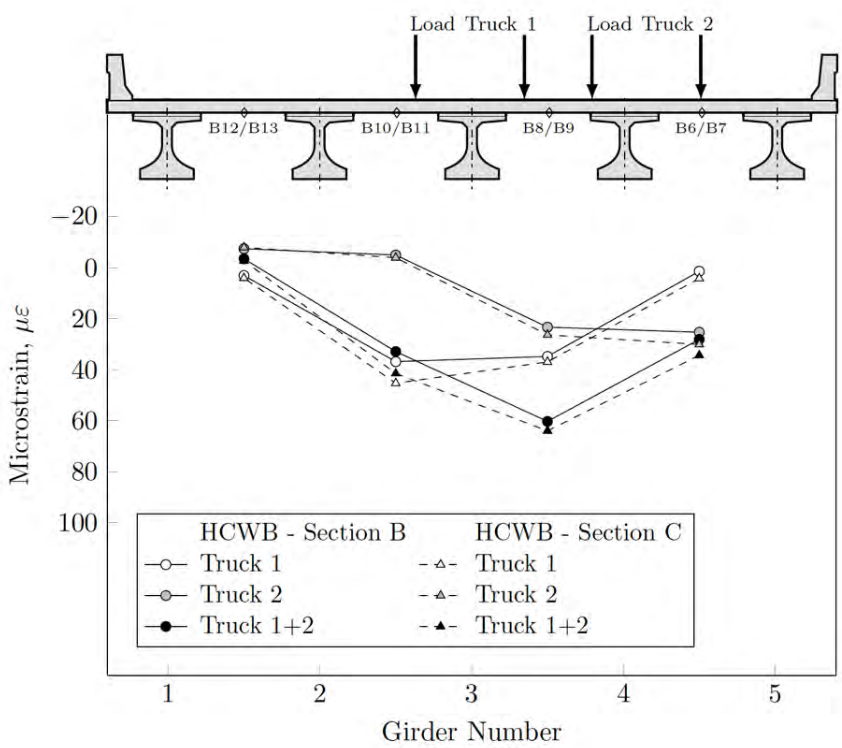


Fig. 4-40. Transverse Strain - HCWB - LP 3B - 24 Blocks - Section B/C

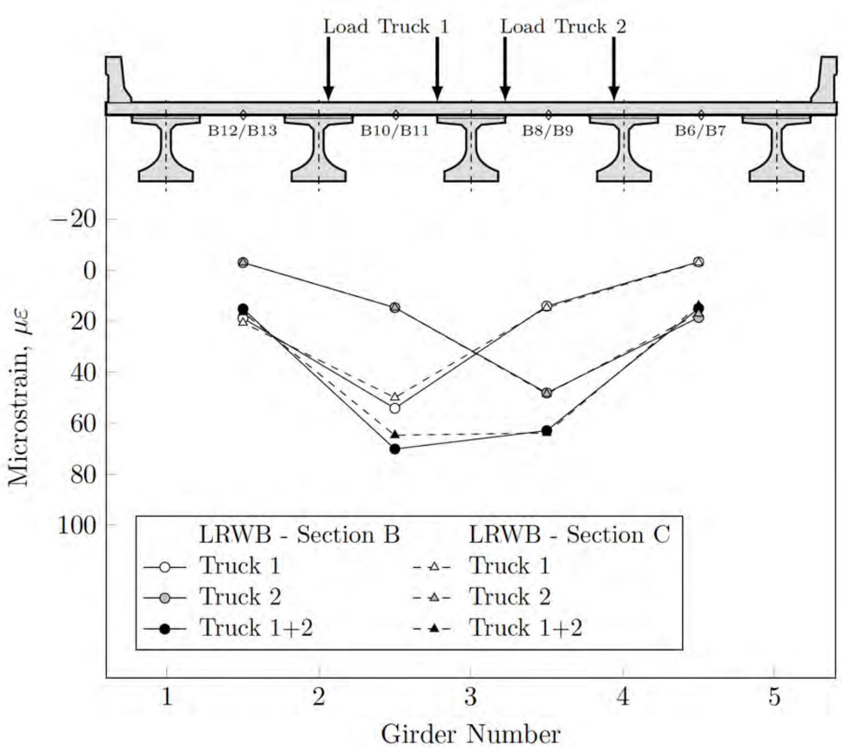


Fig. 4-41. Transverse Strain - LRWB - LP 1B - 24 Blocks - Section B/C

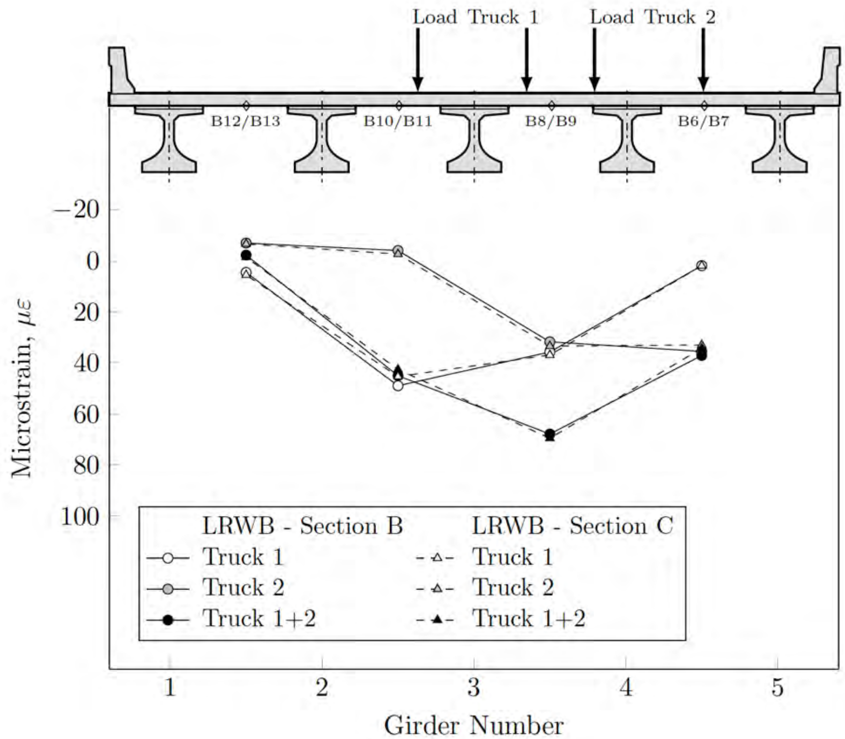


Fig. 4-42. Transverse Strain - LRWB - LP 3B - 24 Blocks - Section B/C

4.4 Pre-Service, Before Barrier vs. After Barrier

4.4.1 Measured Deflection Comparisons

In this section, the deflections are compared before and after the barriers were constructed on HCWB. Only load cases 1/1A and 2 are analyzed.

The data from the load tests performed on HCWB before and after the barriers were constructed was compared, to analyze the barriers' effect on the girder deflections. **Figures 4-43 and 4-44** show that HCWB deflected more before the barrier than after, for both load positions 1A and 2. This shows that the barrier provides additional stiffness to the bridge cross section. For load position 1A, where the trucks were placed symmetrically about the centerline of the bridge, the deflection curves had a similar shape before and after the barriers, as shown in Figure 4-43. However, for load position 2 (Figure 4-44), the exterior girder FIB5 deflected 27% more before the barrier (0.57 in.) than after the barrier (0.45 in.).

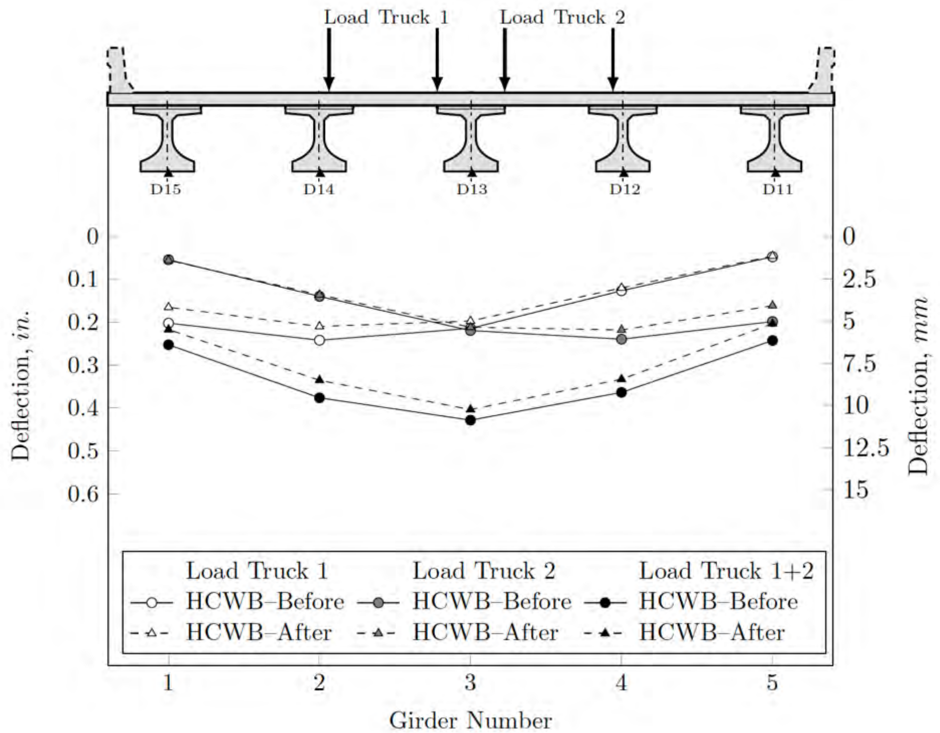


Fig. 4-43. Deflection HCWB Before vs. After Barrier - LP 1A - 24 Blocks

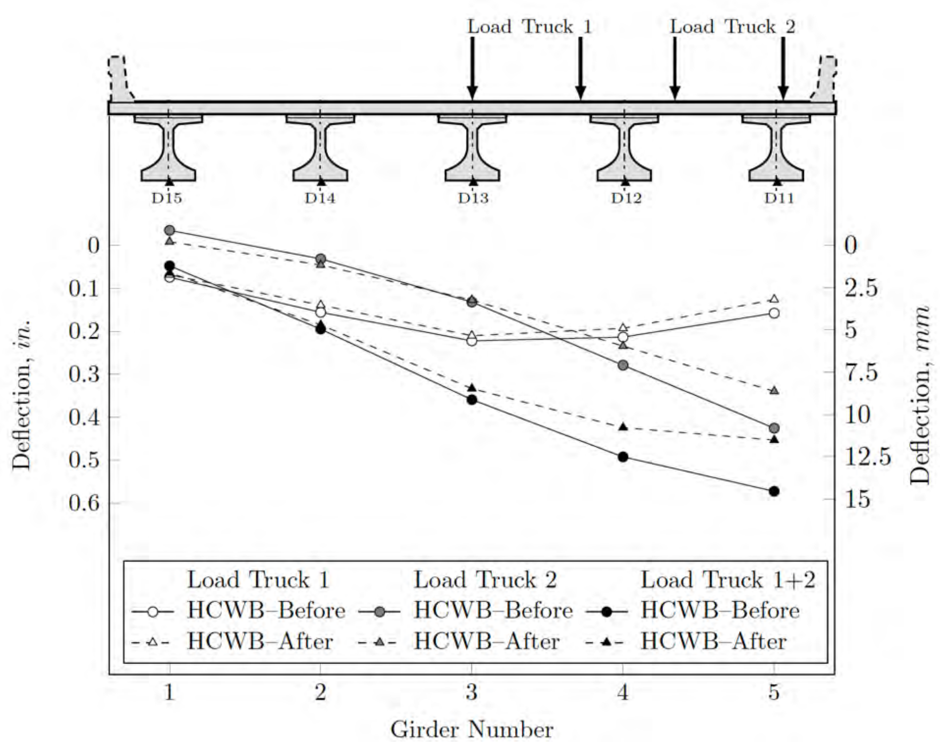


Fig. 4-44. Deflection HCWB Before vs. After Barrier - LP 2 - 24 Blocks

4.4.2 Longitudinal Strain Comparisons

This section presents a comparison of the strains before and after the barriers were constructed on HCWB. Only load positions 1A and 2 were analyzed.

As shown in **Figures 4-45 and 4-46**, the longitudinal strains were generally very similar before and after the barriers were constructed. However, for load position 2, the exterior girder FIB1 experienced more strain – although the strains were small – after the barriers were constructed. On the other hand, the exterior girder FIB5 experienced 12% *less* strain after the barriers were constructed – FIB5 strains decreased from 104 με to 92 με. This is due to the additional stiffness that the barrier provides, particularly to its nearest girder. The barrier effectively carries load, as demonstrated by the strains on the top of the barrier shown in Figures 3-3 and 3-4.

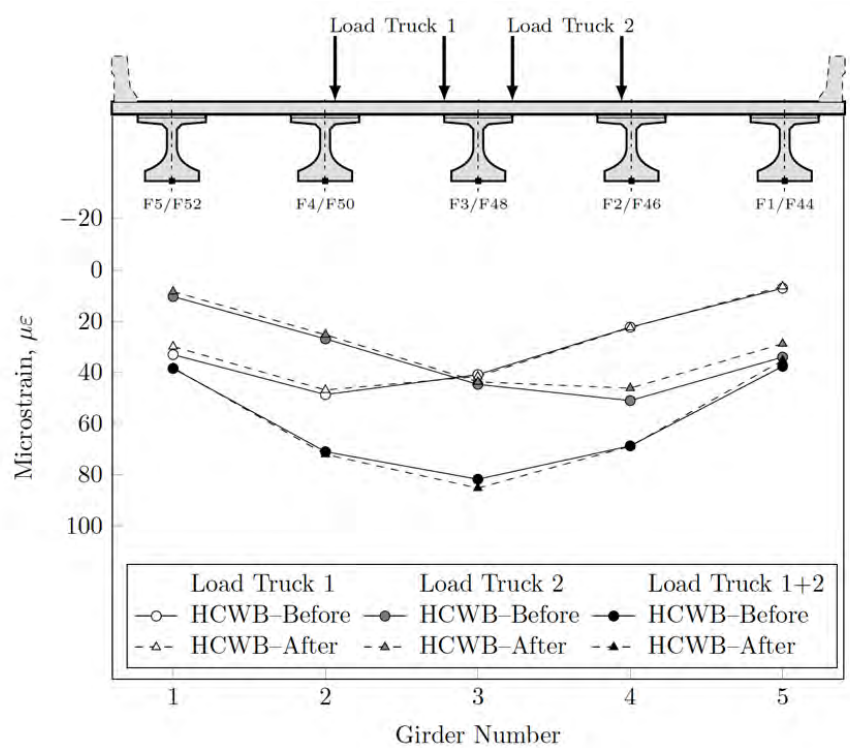


Fig. 4-45. Longitudinal Strain - HCWB Before vs. After Barrier - LP 1A - 24 Blocks

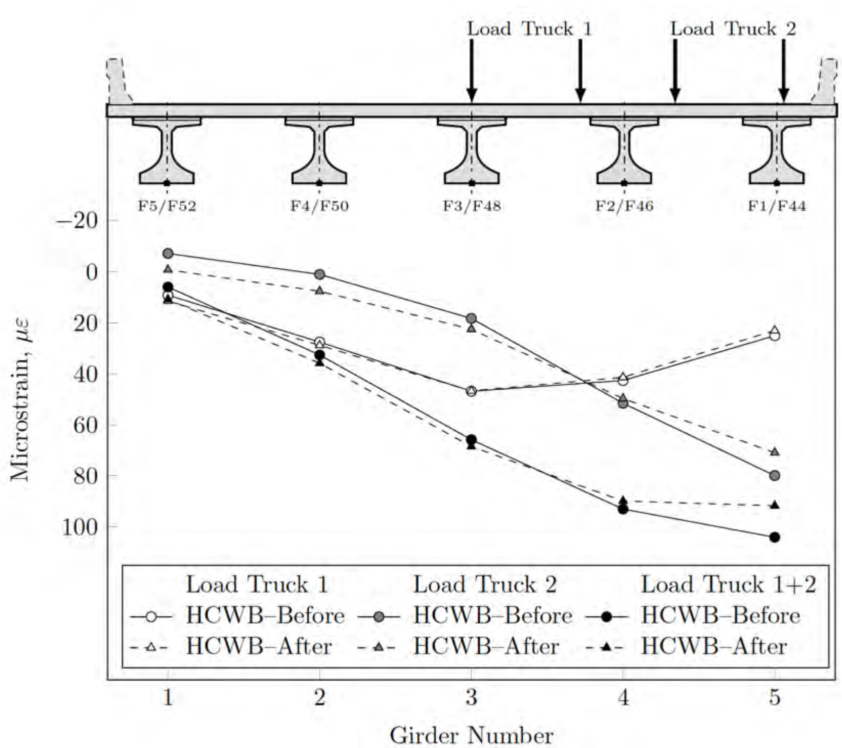


Fig. 4-46. Longitudinal Strain - HCWB Before vs. After Barrier - LP 2 - 24 Blocks

4.4.3 Distribution Factors from Measured Strains Before Barrier on HCWB

For design, the AASHTO equations do not consider the effect of the barriers. For comparison, the DFs calculated from the load test on HCWB before the barriers were plotted with the theoretical AASHTO DFs. **Figures 4-47 and 4-48** show the DFs for load position 1A which positioned the load trucks over FIB2, FIB3, and FIB4. One lane loaded corresponds to a single load truck, whereas two lanes corresponds to both trucks on the bridge. The DF was 0.371 for FIB2 when truck 1 was loaded, 0.373 for FIB4 when truck 2 was loaded, and 0.561 for FIB3 when both trucks were loaded. The theoretical DFs for one lane loaded from the AASHTO equations are 0.465 for FIB2 and 0.468 for FIB4, which is 25% greater than the load test. The theoretical DF for two lanes loaded is 0.680 for FIB3, which is 21% greater than the load test.

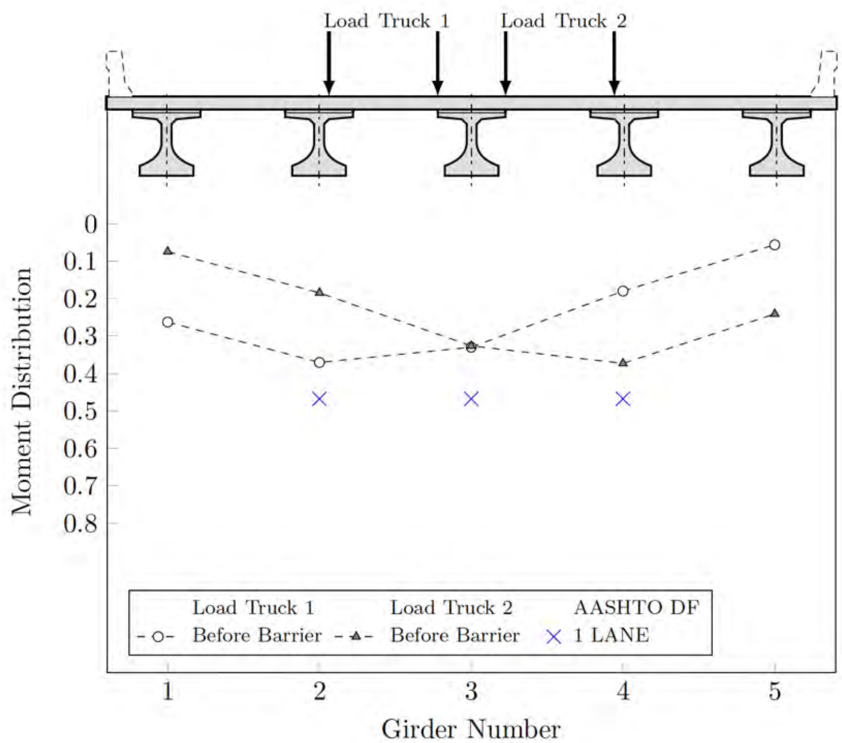


Fig. 4-47. Moment Distribution HCWB Before Barriers - LP 1A - 24 Blocks with MPF – 1 lane

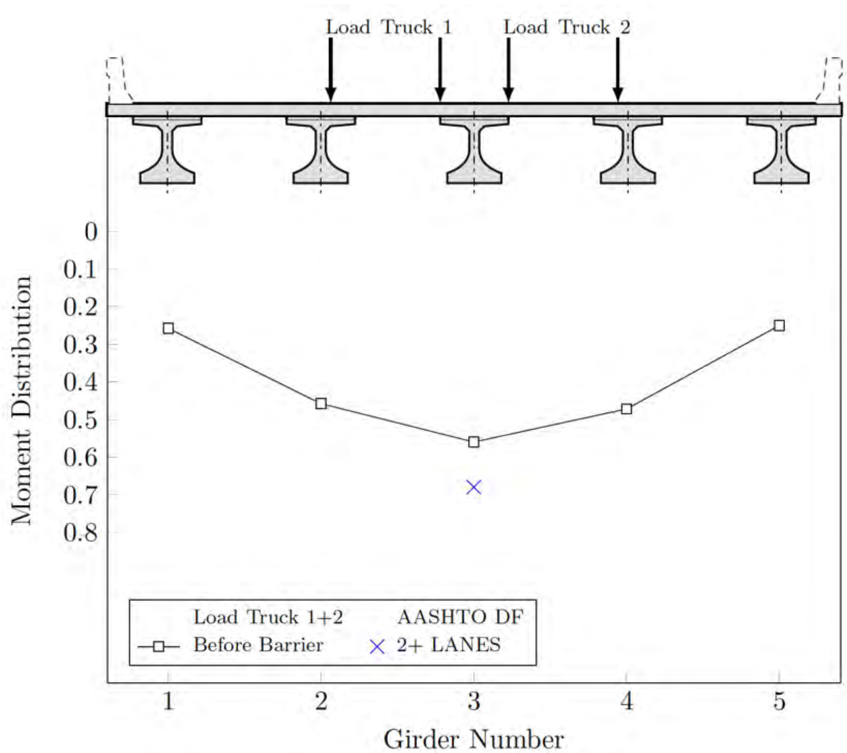


Fig. 4-48. Moment Distribution HCWB Before Barriers - LP 1A - 24 Blocks with MPF – 2 lanes

Load position 2 maximized the effects of the trucks on FIB3, FIB4, and FIB5. **Figures 4-49 and 4-50** show that the calculated DFs closely match the AASHTO DFs. The DF was 0.438 for FIB4 and 0.661 for FIB5 when truck 2 was loaded. The AASHTO DF for one lane is 0.468 for FIB4, which is 6.8% greater than the load test. For the exterior girder FIB5, the AASHTO DF of 0.800 is 21% greater than the load test. For both trucks, 1 and 2, FIB4 had a DF of 0.628, and the AASHTO DF of 0.680 is 8.3% greater. FIB5 DF was 0.683, and the AASHTO DF of 0.671 is 1.8% less.

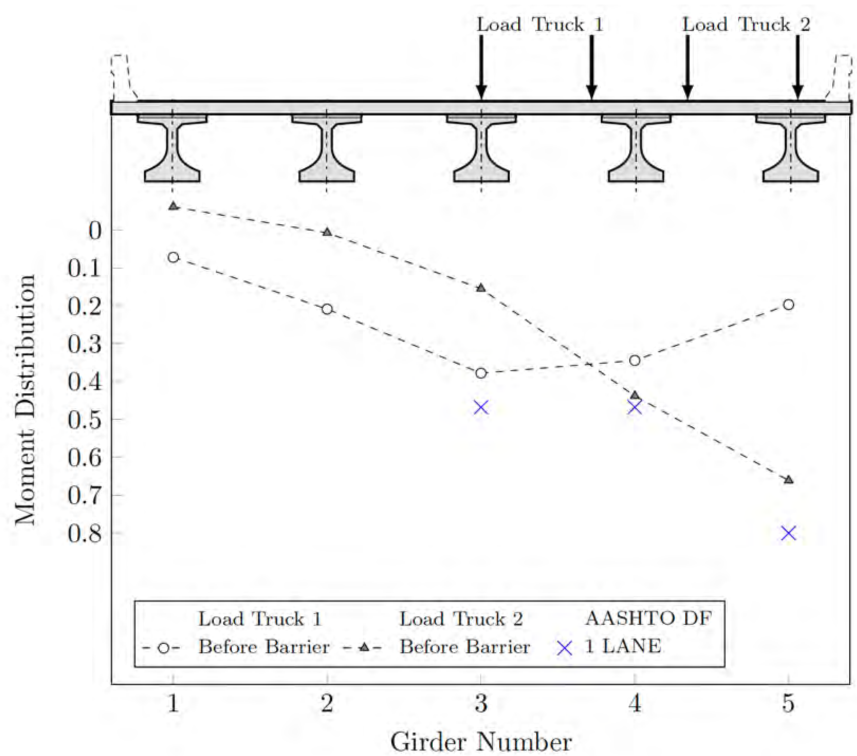


Fig. 4-49. Moment Distribution HCWB Before Barriers - LP 2 - 24 Blocks with MPF – 1 lane

4.4.4 Distribution Factors from Measured Strains Comparisons

In this section, the distribution factors calculated from strains from the before-barrier and after-barrier load tests on HCWB are compared.

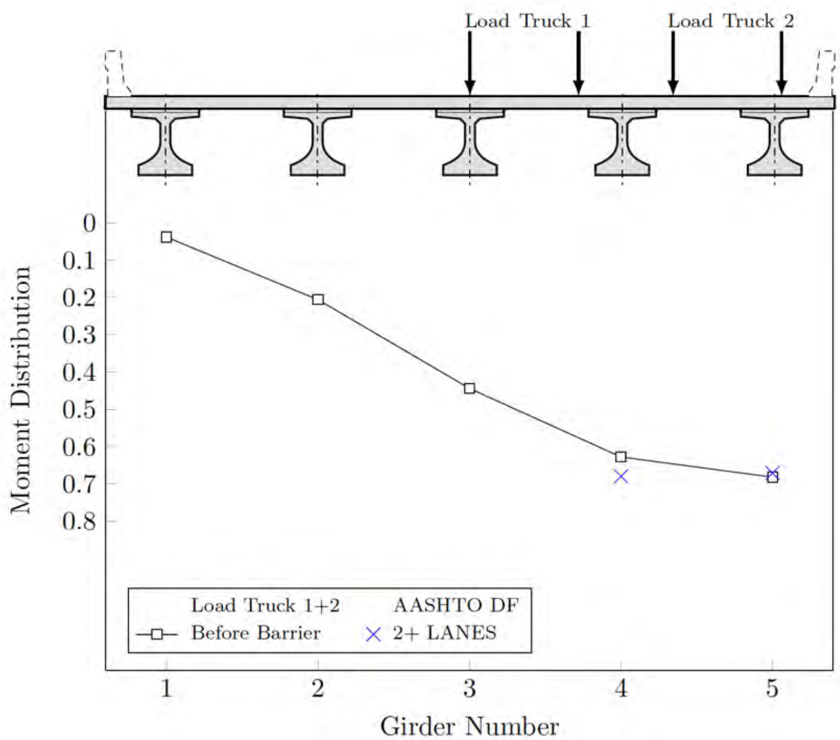


Fig. 4-50. Moment Distribution HCWB Before Barriers - LP 2 - 24 Blocks with MPF - 2 lanes

When the trucks were placed at load position 1A after the barriers were constructed, the DF for the exterior girders increased, compared to the test before the barriers, as shown in **Figures 4-51 and 4-52**. For two (2) trucks, FIB1 DF increased 22% from 0.259 to 0.315, and FIB5 increased 11% from 0.257 to 0.286, as shown in Figure 4-52. All of the interior girders' DFs were reduced slightly.

For load position 2, the DF for the first interior girder (FIB4) decreased after the barriers were constructed, compared to the before-barrier case, as shown in **Figures 4-53 and 4-54**. For two (2) trucks, the reduction was 12%. The exterior girder (FIB5) DF increased 6% as shown in Figure 4-54.

In summary, the barriers seem to attract load away from the first interior girder and towards the exterior girder. As noted previously, though, in the section on Longitudinal Strain Comparisons, the barriers also add stiffness to the exterior girder, effectively carrying load.

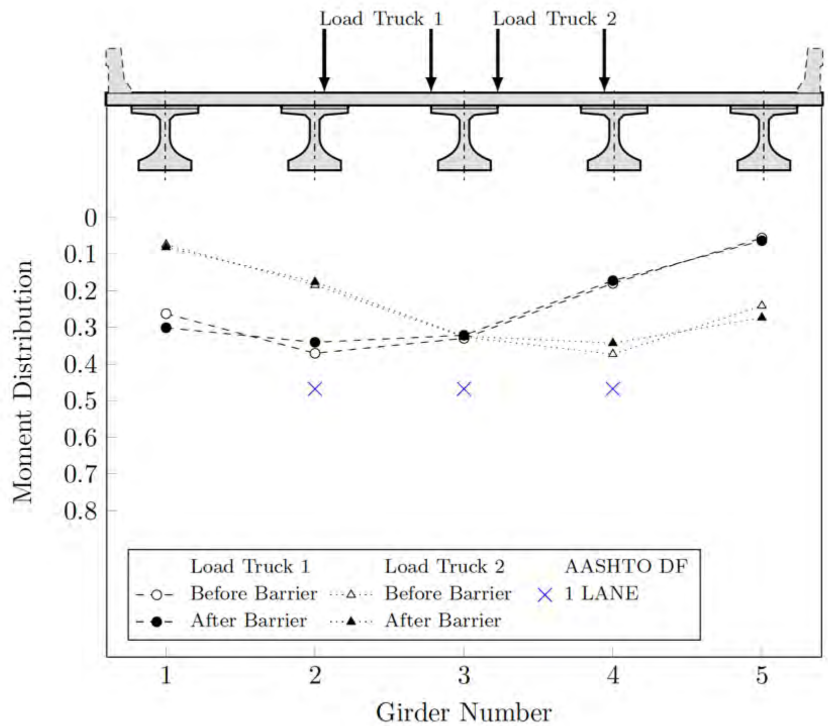


Fig. 4-51. Moment Distribution HCWB Before vs. After Barriers - LP 1A - 24 Blocks with MPF - 1 lane

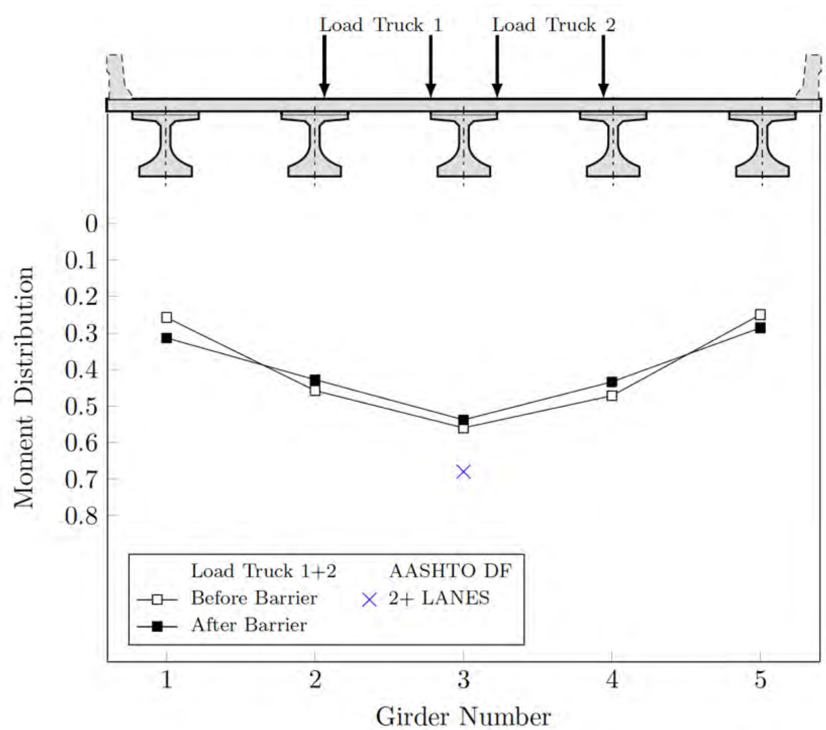


Fig. 4-52. Moment Distribution HCWB Before vs. After Barriers - LP 1A - 24 Blocks with MPF - 2 lanes

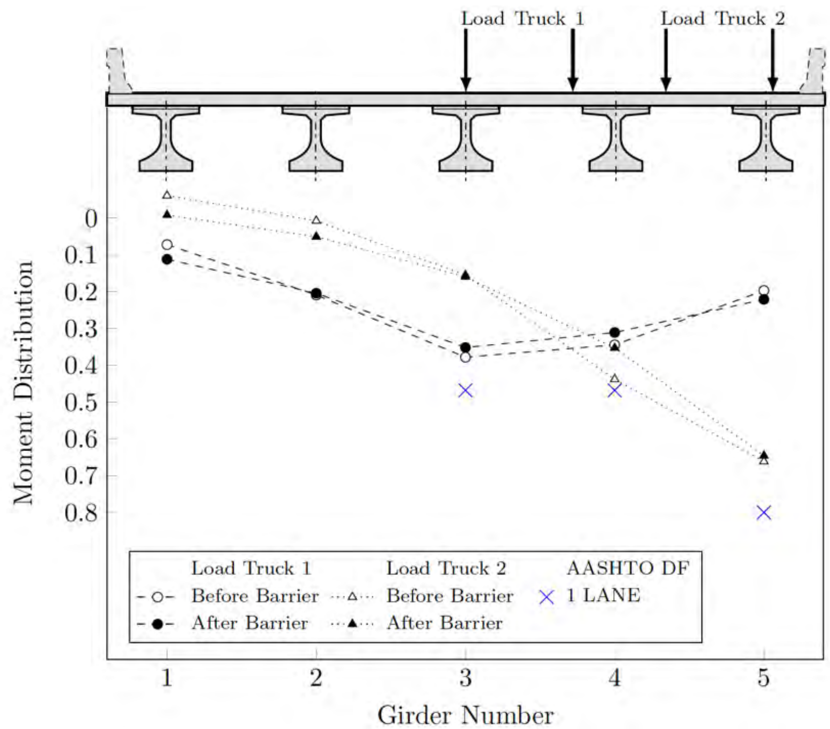


Fig. 4-53. Moment Distribution HCWB Before vs. After Barriers - LP 2 - 24 Blocks with MPF – 1 lane

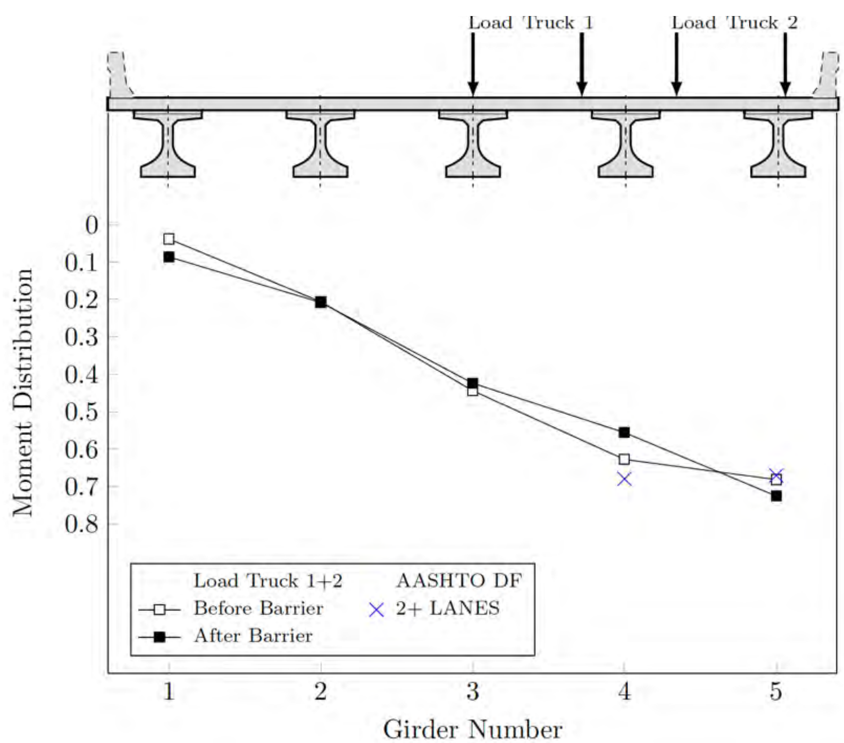


Fig. 4-54. Moment Distribution HCWB Before vs. After Barriers - LP 2 - 24 Blocks with MPF - 2 lanes

4.4.5 Distribution Factors from Measured Deflections Comparisons

In this section, the distribution factors calculated from deflections from the before-barrier and after-barrier load tests on HCWB are compared. **Figures 4-55 and 4-56** are for load position 1, and **Figures 4-57 and 4-58** are for load position 2.

As shown in Figures 4-55 and 4-57, the one lane AASHTO DFs are conservative for interior and exterior girders for both load positions. Also, the deflection DFs increased for the exterior girders and decreased for the interior girders with the addition of the barriers.

Figures 4-56 and 4-58 show DFs for both load positions with 2 lanes loaded. Similar to one lane loaded, the after barrier case resulted in higher distribution factors for the exterior girders compared to before the barriers, but lower DFs for the interior girders. This validates that the barrier has a predictable effect on the bridge.

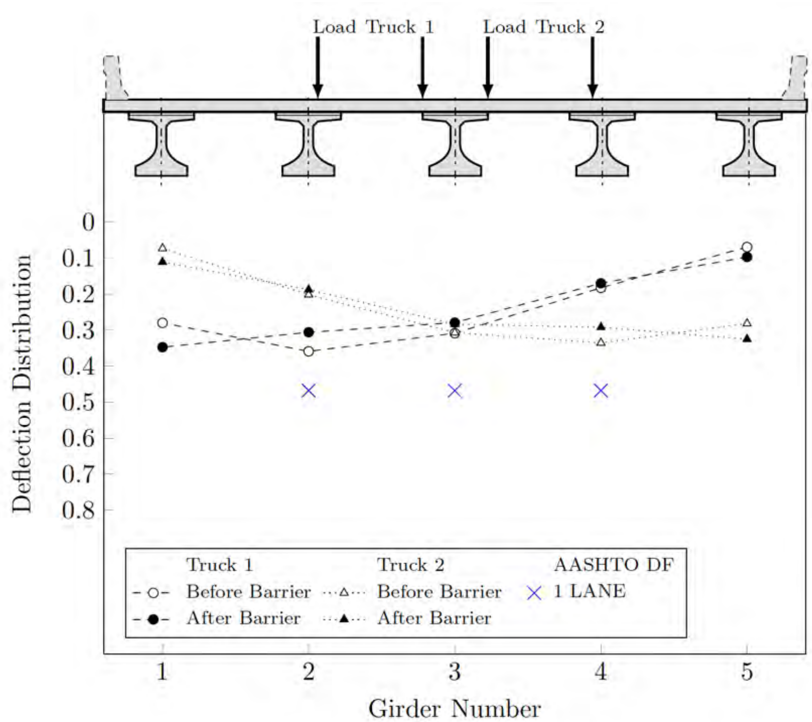


Fig. 4-55. Deflection Distribution HCWB Before vs. After Barriers - LP 1A - 24 Blocks - 1 lane

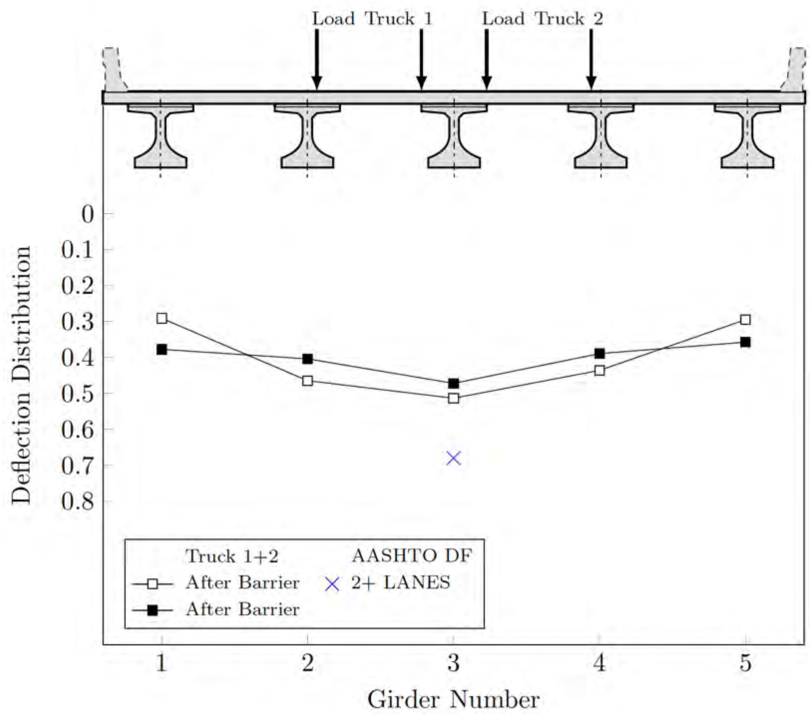


Fig. 4-56. Deflection Distribution HCWB Before vs. After Barriers - LP 1A - 24 Blocks - 2 lanes

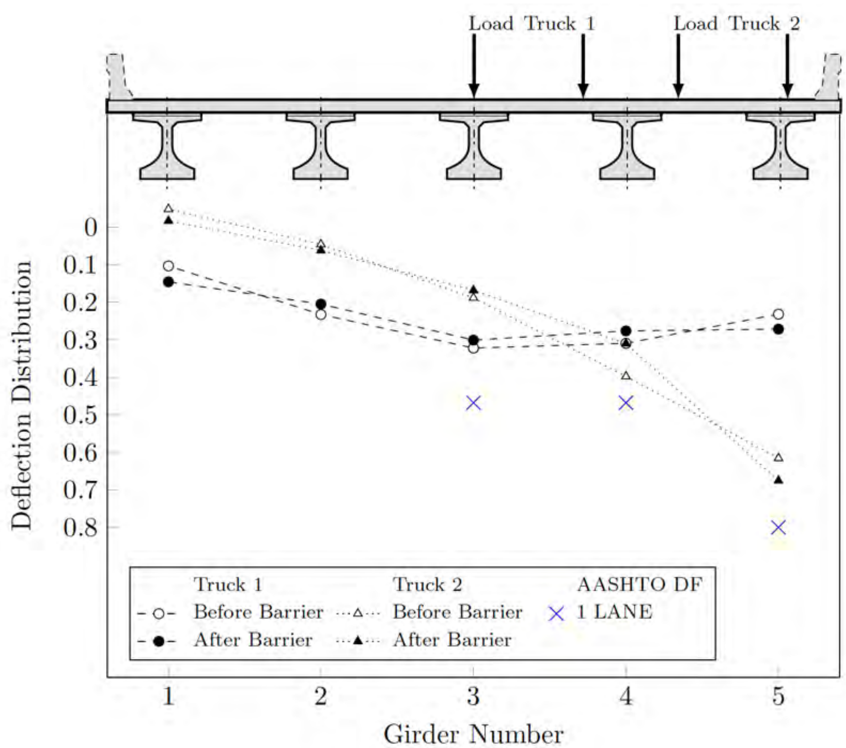


Fig. 4-57. Deflection Distribution HCWB Before vs. After Barriers - LP 2 - 24 Blocks - 1 lane

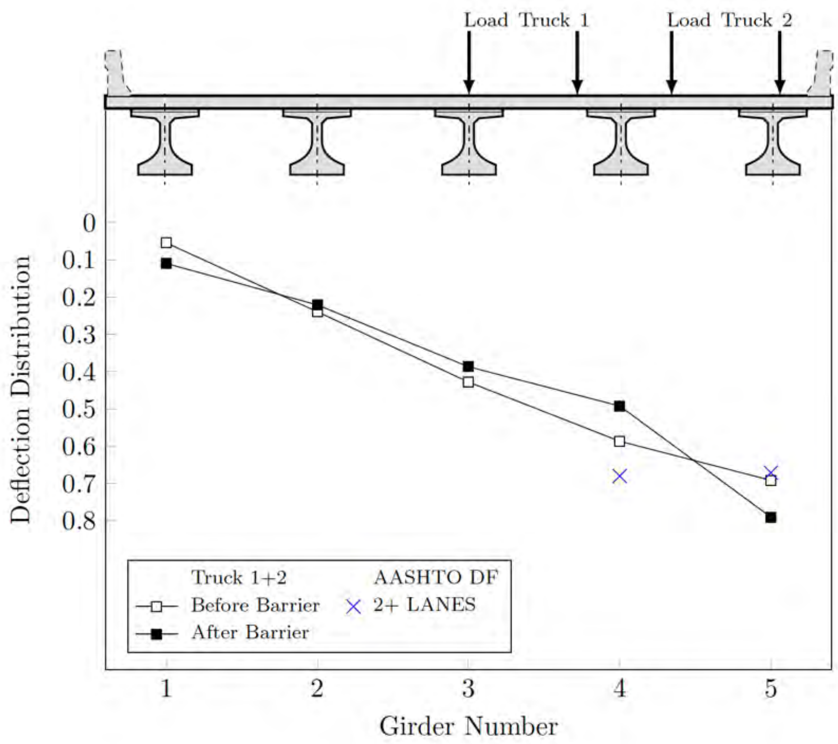


Fig. 4-58. Deflection Distribution HCWB Before vs. After Barriers - LP 2 - 24 Blocks - 2 lanes

4.4.6 Load Distribution (Deflections) vs. Load Distribution (Moments) for Before and After Barrier Tests

This section compares the load distribution factors calculated from the load test deflections to those calculated from the strains (moments). Because the maximum distribution factors occurred for load position 2, this section will focus on that data. **Figures 4-59 to 4-62** show comparisons for all bridges except HCEB which is excluded because of a malfunctioned deflection gauge. Figure 4-59 shows HCWB DFs before the barriers. The results of the two calculation methods are very similar, with the exterior girder FIB5 showing almost identical results for both methods, and the first interior girder only slightly differing.

After the barriers were added to HCWB (Figure 4-60) the deflection distribution factors were higher at FIB5, with a value of 0.790 for two (2) trucks compared to the moment distribution factor of 0.726, a difference of about 8%. As shown in Figures 4-61 and 4-62, deflection DFs and moment DFs matched somewhat more closely for LRWB and LREB test results.

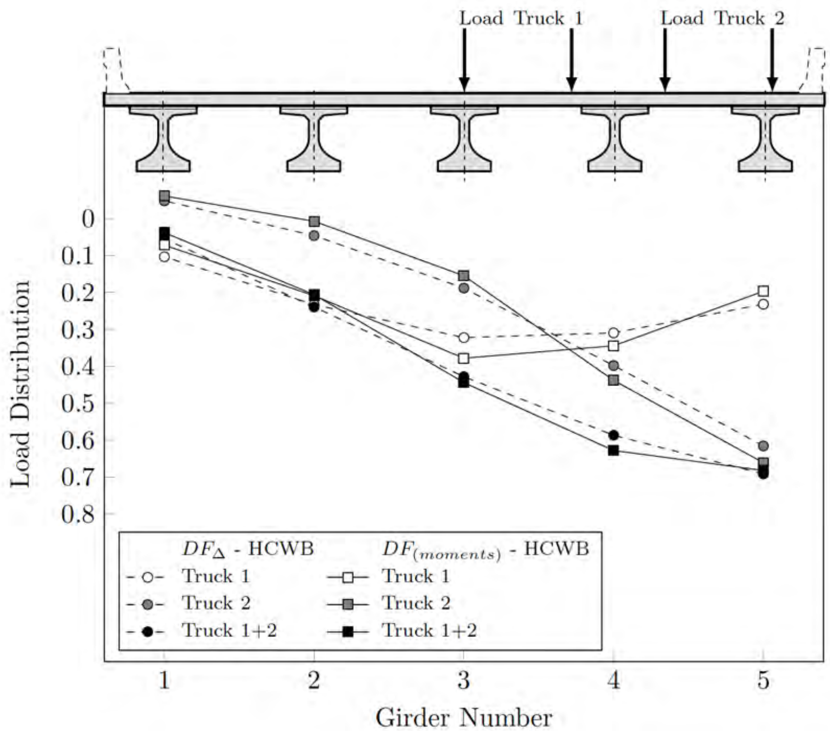


Fig. 4-59. Load Distribution Comparison - HCWB - Before Barrier - LP 2 - 24 Blocks

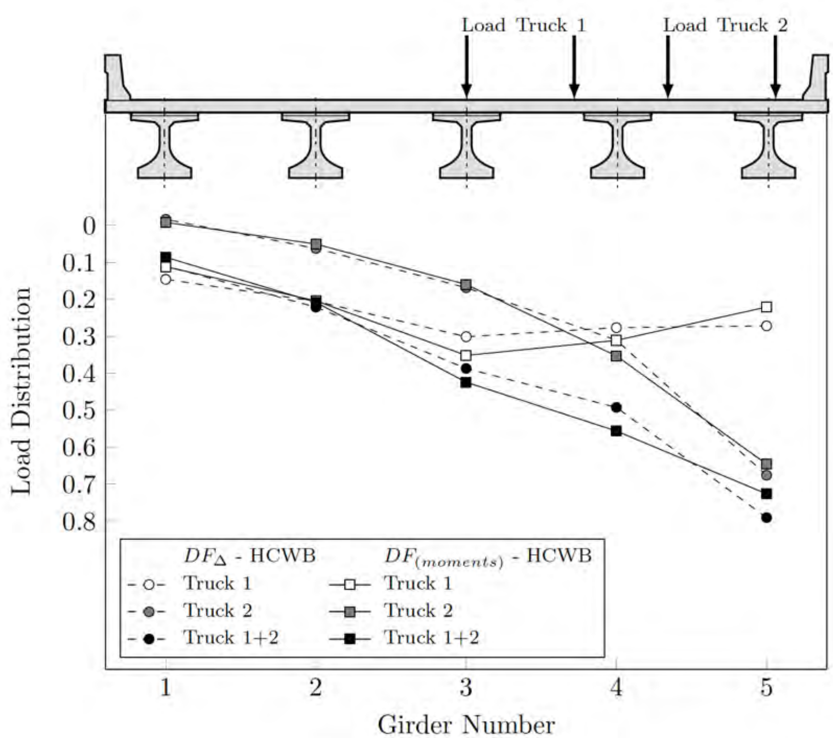


Fig. 4-60. Load Distribution Comparison - HCWB - After Barrier - LP 2 - 24 Blocks

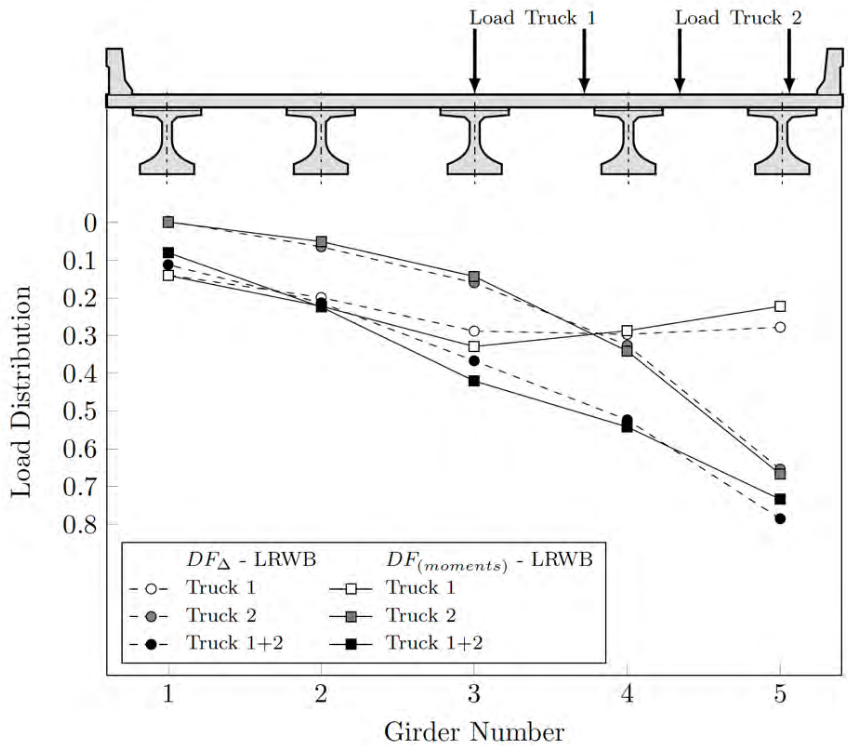


Fig. 4-61. Load Distribution Comparison - LRWB - LP 2 - 24 Blocks

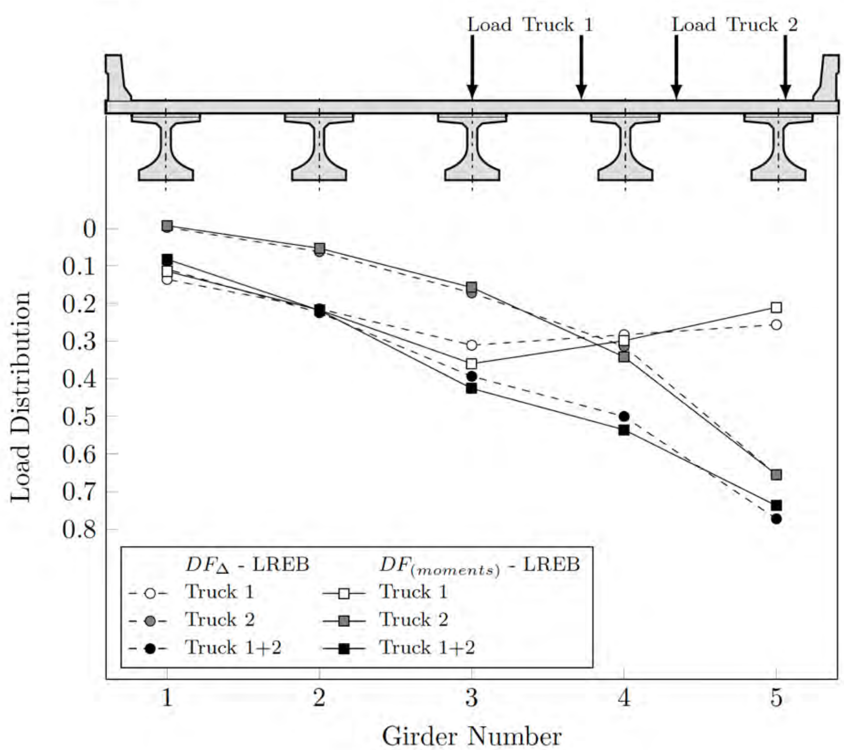


Fig. 4-62. Load Distribution Comparison - LREB - LP 2 - 24 Blocks

4.5 Bent Cap Behavior – EB and WB

Comparing Figure 3-59 to 3-62, 3-60 to 3-63, and 3-61 to 3-64, the bottom strain on the left face was approximately the same for EB and WB bent caps. However, for the other gauges on the left face, the EB and WB bent caps differed some.

As noted earlier, the right face of the WB bent cap (see Figures 3-62 to 3-64 and 3-68 to 3-70) experienced much higher strains than the left face. Whereas, the faces of the EB bent cap responded similarly to each other (see Figures 3-59 to 3-61 and 3-65 to 3-67).

Because the strains on the right face of the WB bent cap were significantly larger than those on both the right face of the EB bent cap and the left face of the WB bent cap, a closer look at the raw data is warranted, namely for the other truck cases (i.e., the figures presented herein show data for trucks 1 and 2 both being on the bridge at the same time). Of particular interest is whether the strains increased linearly with an increase in load; adding the measured response from truck 1 to the response from truck 2 should equal the measured response from trucks 1 and 2 being on the bridge at the same time. For Load Position 4, 24 blocks, the raw data shows the following:

- For the EB bent cap, gauge F83 (left face) and gauge F84 (right face) had strains within a couple $\mu\varepsilon$, so the averages can be used for analysis. The averages of gauges F83 and F84 were 13, 14, and 27 $\mu\varepsilon$ for truck 1, truck 2, and truck 1+2, respectively. This shows that the response was indeed linear as the load increased (i.e., 13 $\mu\varepsilon$ from truck 1, plus 14 $\mu\varepsilon$ from truck 2, equals the measured 27 $\mu\varepsilon$ for trucks 1 and 2).
- For the WB bent cap, gauge F83 (left face) was 16, 21, and 35 $\mu\varepsilon$ for truck 1, truck 2, and truck 1+2, respectively. This shows that the response was linear as the load increased (i.e., 16 $\mu\varepsilon$ from truck 1, plus 21 $\mu\varepsilon$ from truck 2, equals 37 $\mu\varepsilon$, which is very close to the measured 35 $\mu\varepsilon$ for trucks 1 and 2).
- Gauge F84 (right face) was 114, 165, and 270 $\mu\varepsilon$ for truck 1, truck 2, and truck 1+2, respectively. The response was approximately linear as the load increased (i.e., 114

με from truck 1, plus 165 με from truck 2, equals 279 με, which is very close to the measured 270 με for trucks 1 and 2).

Similar linearity was observed in the raw data for the 12 and 18 block cases for Load Position 4 (see **Table 4-3** and **Figure 4-63**), as well as for the 12, 18, and 24 block cases for Load Position 5.

Table 4-3. Strain on Bottom of EB and WB Bent Caps for Load Position 4

Load Case	Reaction (k)	Strain on Bottom of Bent Cap (με) EB (Ave. of F83 & F84)	WB F83	WB F84
LS4-T1-12	57.6	9	10	68
LS4-T1-18	68.2	10	18	93
LS4-T1-24	79.3	13	16	114
LS4-T2-12	57.6	10	13	88
LS4-T2-18	68.2	12	20	112
LS4-T2-24	79.3	14	21	165
LS4-T12-12	115.2	19	27	173
LS4-T12-18	136.3	23	32	208
LS4-T12-24	158.6	27	35	270

LS4 = Load Position 4
T1 = truck 1, T2 = Truck 2, T12 = Trucks 1 & 2
12, 18, 24 = number of blocks on each truck

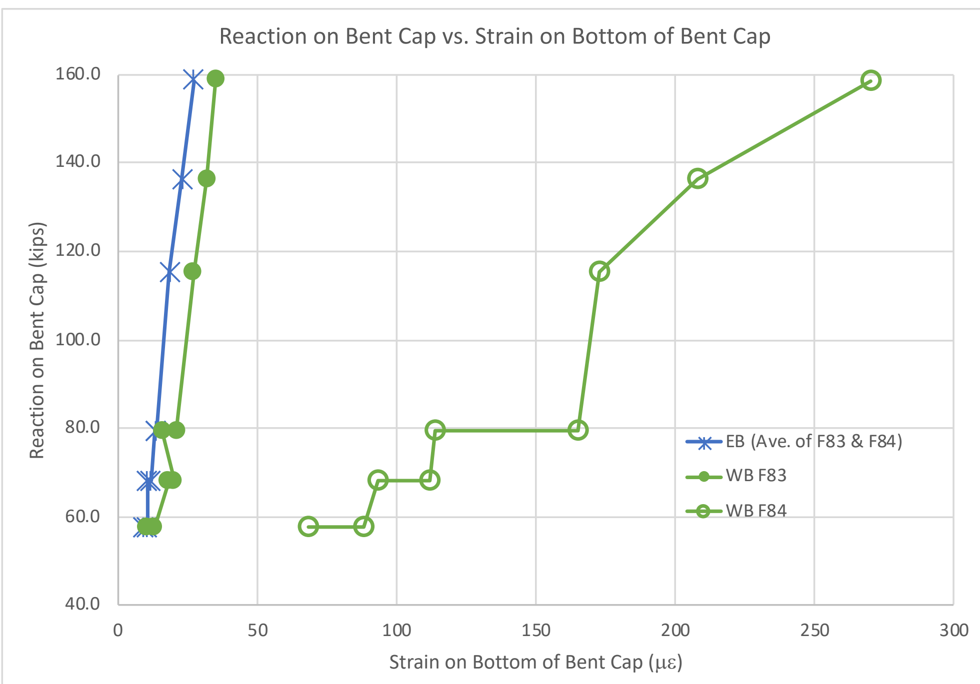


Fig. 4-63. Reaction on Bent Cap vs. Strain on Bottom of Bent Cap

CHAPTER 5
PRE-SERVICE vs. IN-SERVICE RESULTS AND ANALYSIS

5.1 Introduction

In this chapter, the results from the pre-service load tests will be compared to those from the in-service load tests, to see if the behavior changed over time and after being subjected to service loads. The results presented include plots to affirm composite action between the elements of the bridges, deflection results at mid span, longitudinal strain results at section B, and transverse strain results at sections B and C.

5.2 Non-Prestressed Concrete Panel Bridges

5.2.1 Non-Prestressed Concrete Panel Bridges - Overview

The eastbound bridges, HCEB and LREB, were constructed with non-prestressed deck panels. The graphs for HCEB show results for load positions 1, 2, and 3, while graphs for LREB show results for load positions 1A, 1B, 2, 3A, and 3B; load positions 1 and 3 for HCEB correspond to 1A and 3A on other load tests. The results presented are either at mid span, third points, section B (9'-0" west of mid span), or Section C (4'-6" west of mid span).

5.2.2 Non-Prestressed Concrete Panel Bridges - Composite Action

Composite action was checked using the in-service load test data. Gauge readings from the longitudinal strains on the cross sections along HCEB-FIB5 and LREB-FIB5 were plotted against their vertical locations. Results were only presented for load position 2, since this position maximized load effects on FIB5.

Figure 5-1 shows longitudinal strains on HCEB FIB5, measured with gauges F36, F6, F40, and F1, which are located on top of the deck panel, top flange of the girder, on the web of the girder, and below the bottom flange of the girder, respectively. Gauge F64 on top of the barrier malfunctioned during the load test, therefore it was excluded from this analysis. The graph shows that the strains vary approximately linearly with the depth, which confirms composite action was achieved. Generally, as the number of blocks increased from 12 to 24, the compression at the top and the tension at the bottom both increased.

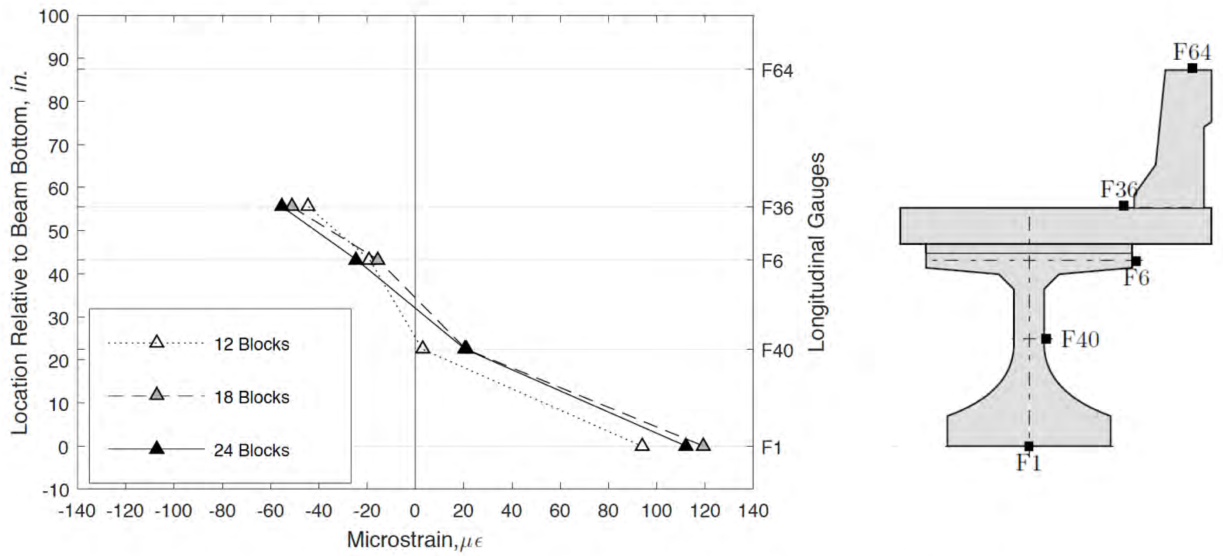


Fig. 5-1. Composite Action – HCEB – FIB5 – LS2

However, there was an exception as in gauge F1 (below the bottom flange of the girder), where more tensile strain was recorded for 18 blocks than the 24 block case. This could be attributed to the drift in F1 gauge readings that was observed; there was also some drift in F6 gauge readings. The average neutral axis position was approximately 31 in. measured from the bottom of the composite section, as shown in Figure 5-1. The neutral axis is the location where zero strain occurs in the cross section, as determined from interpolation. For the pre-service tests, the neutral axis was approximately 36 in. from the bottom of the section.

Longitudinal strains on LREB FIB5 are shown in **Figure 5-2**. The strains vary approximately linearly with the depth. Generally, as the number of blocks increased from 12 to 24, the compression at the top and the tension at the bottom both increased. The neutral axis for the composite section was approximately at 34 in. measured from the bottom of the composite section. For the pre-service tests, the neutral axis was approximately 35 in. from the bottom of the section.

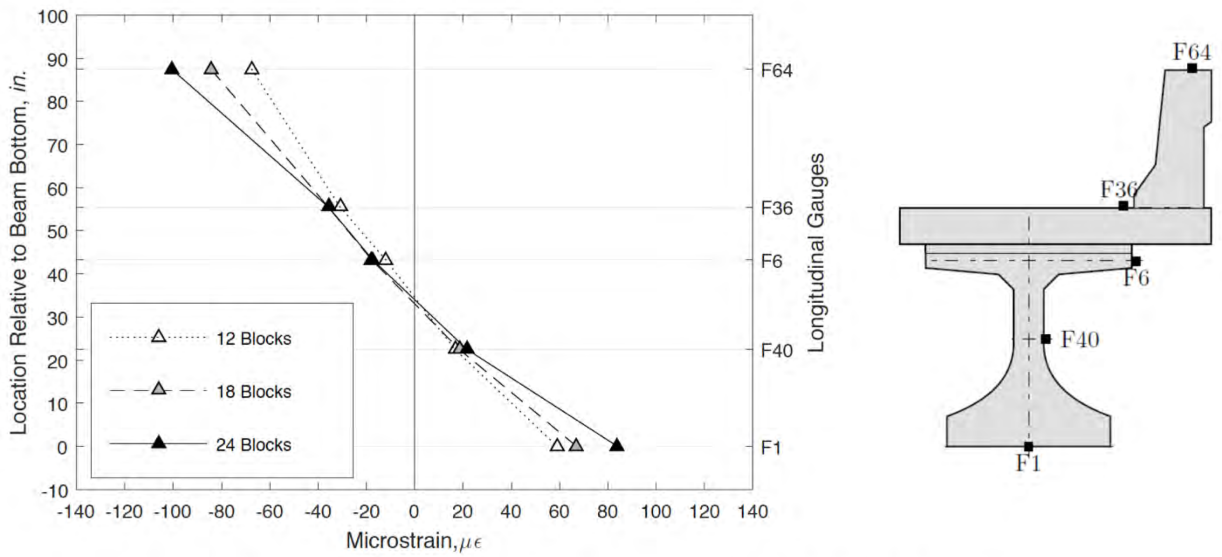
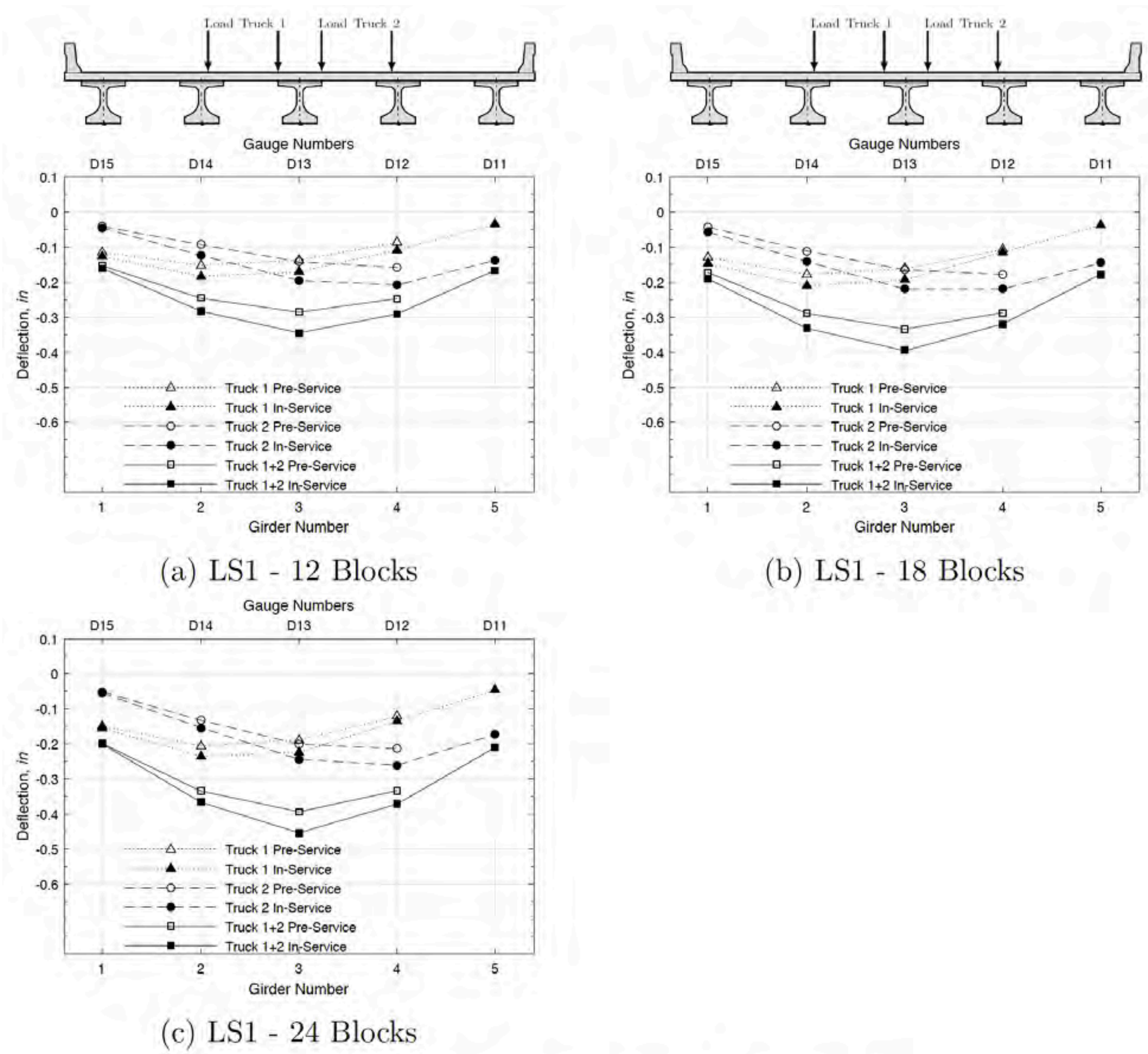


Fig. 5-2. Composite Action – LREB – FIB5 – LS2

5.2.3 Non-Prestressed Concrete Panel Bridges - Deflection Results

Global deflection of the superstructure was measured about the center line of each girder's bottom flange at mid span and section E (18' east of mid span). Deflections generally increased as the load increased. The results presented are at mid span for load positions 1/1A, 1B, and 2, for the 12, 18, and 24 blocks load cases. Deflection distributions (relative magnitudes) measured at mid span were similar to those measured at section E. Readings for gauge D11 in HCEB pre-service results were excluded because the gauge was faulty.

Load Position 1/1A/1B: **Figures 5-3 and 5-4** show the results for the pre-service and in-service load tests for HCEB and LREB at load position 1/1A/1B. These positions loaded the bridges symmetrically in the transverse direction and maximized deflection on FIB3, when both truck 1 and truck 2 were placed on the bridges. The overall behavior of the bridges show that the north side deflected more when only truck 1 was on the bridge as seen on the left side of the graphs, while the south end (right on the graph) deflected more with only truck 2 on the bridge.



(a) LS1 - 12 Blocks

(b) LS1 - 18 Blocks

(c) LS1 - 24 Blocks

Fig. 5-3. Deflection Mid Span – HCEB – LS1

From the pre-service and in-service plots for HCEB (Figure 5-3), the maximum deflection (at FIB3) was 0.29", 0.33", and 0.39" for 12, 18 and 24 blocks, respectively, for the pre-service test. The in-service test shows maximum deflections to be 0.35", 0.40", and 0.45" for those block cases. This is an average increase of 18.1% from pre-service to in-service.

Figure 5-4 shows the pre-service and in-service plots for LREB, for load position 1A/1B. For position 1A (Figures 5-4a, b and c), the maximum deflection (at FIB3) was 0.29", 0.35", and 0.40" for 12, 18, and 24 blocks, respectively, for the pre-service test. The in-service test shows maximum deflection to be 0.32", 0.38", and 0.43" for those block cases. This is an average increase of 7.7% from pre-service to in-service. Figure 5-4d shows that deflection increased from 0.39" to 0.41" for load position 1B, which is a 5.1% increase.

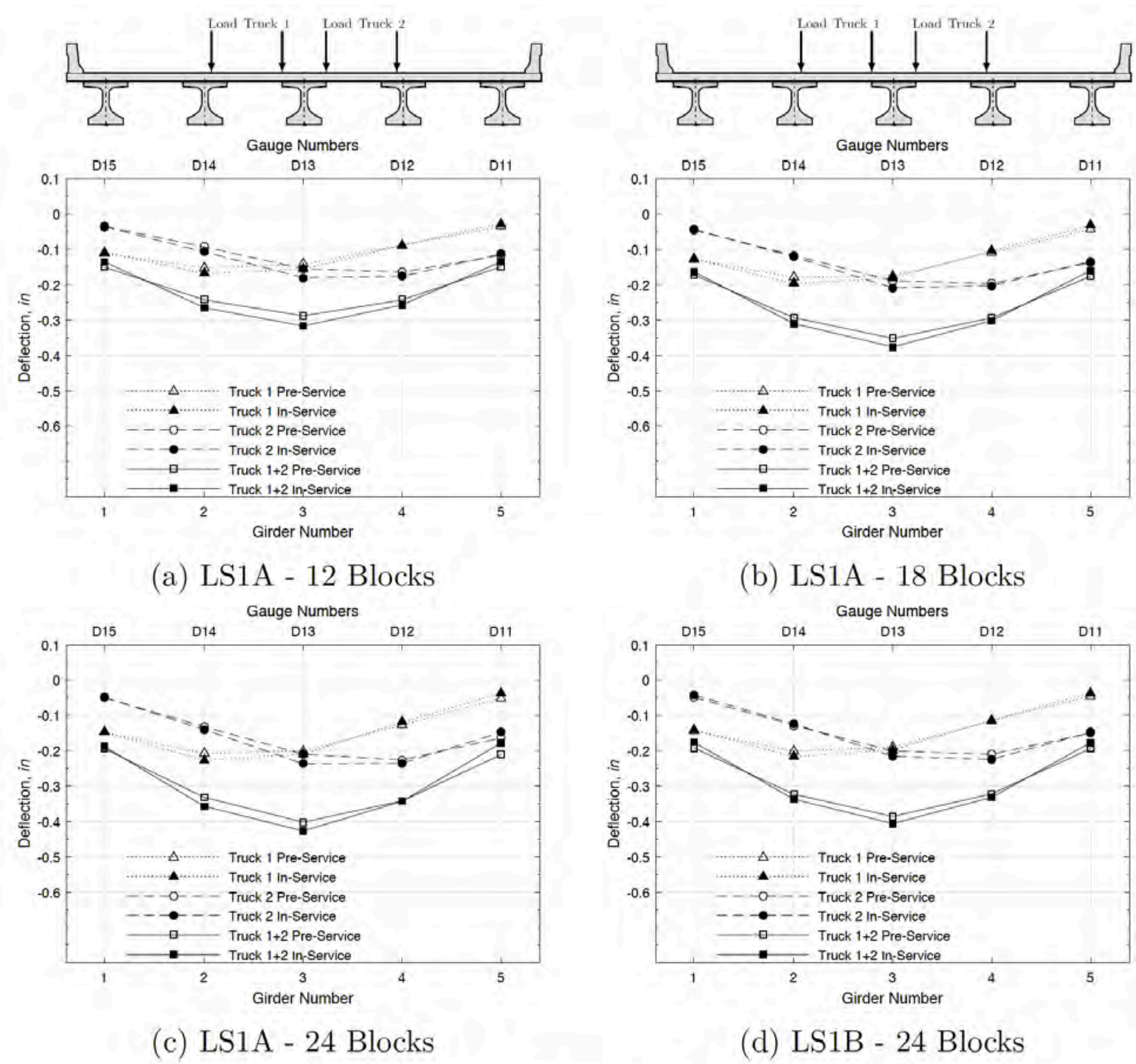


Fig. 5-4. Deflection Mid Span – LREB – LS1A/1B

Load Position 2: **Figures 5-5 and 5-6** show the results for the pre-service and in-service load tests for HCEB and LREB at load position 2. This position maximized deflection on one of the external girders (FIB5), when both truck 1 and truck 2 were placed on the bridges.

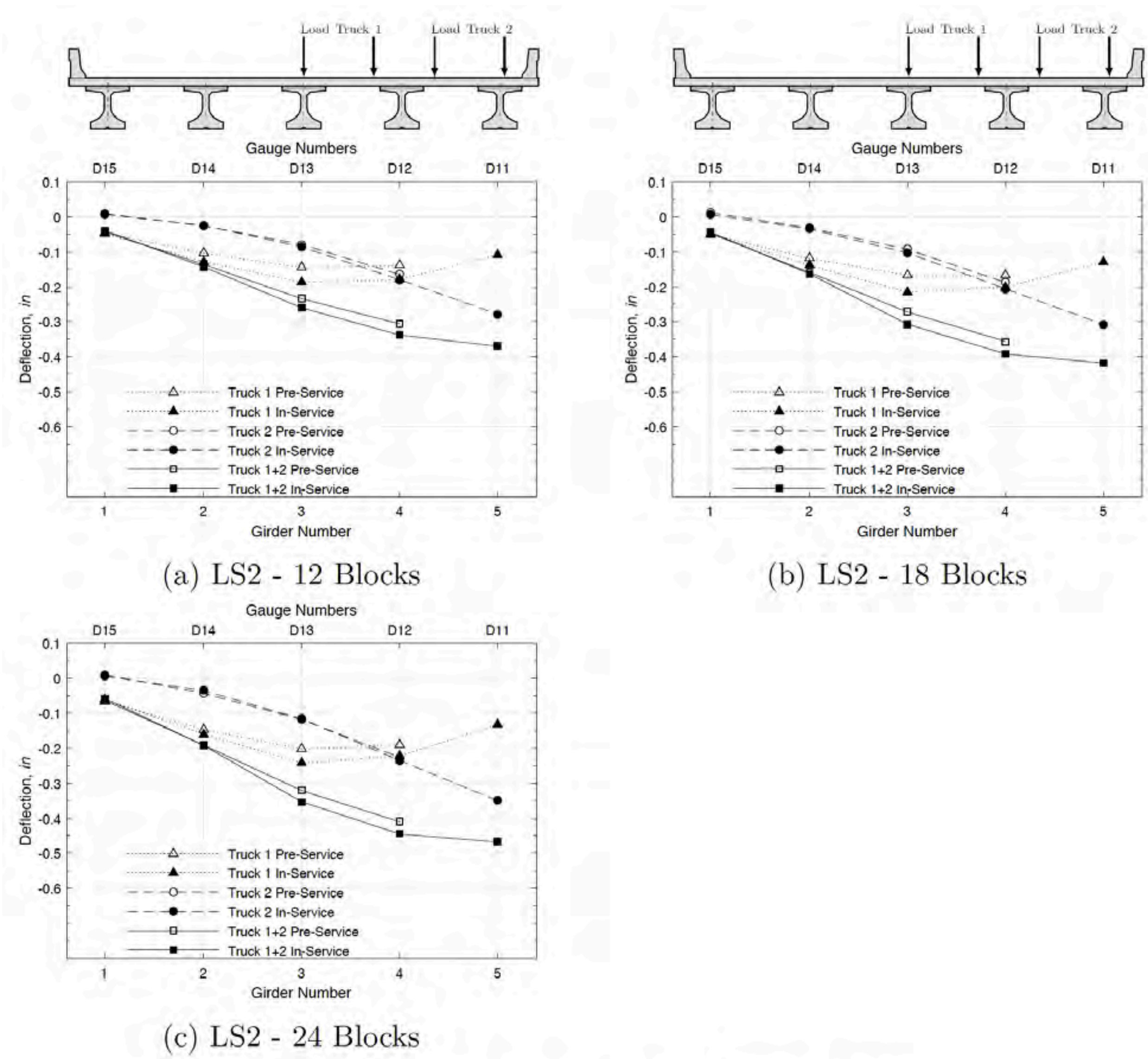


Fig. 5-5. Deflection Mid Span – HCEB – LS2

The pre-service and in-service plots for HCEB, load position 2, are shown in Figure 5-5. Gauge D11 was faulty during the pre-service test, hence its data is not included in the results. Maximum deflections (at FIB5) were 0.37", 0.42", and 0.47" for 12, 18, and 24 blocks, respectively, for the in-service test.

The pre-service and in-service plots for LREB, load position 2, are shown in Figure 5-6. When the plots of the two tests were compared, the change in maximum deflection was miniscule for each of the block cases.

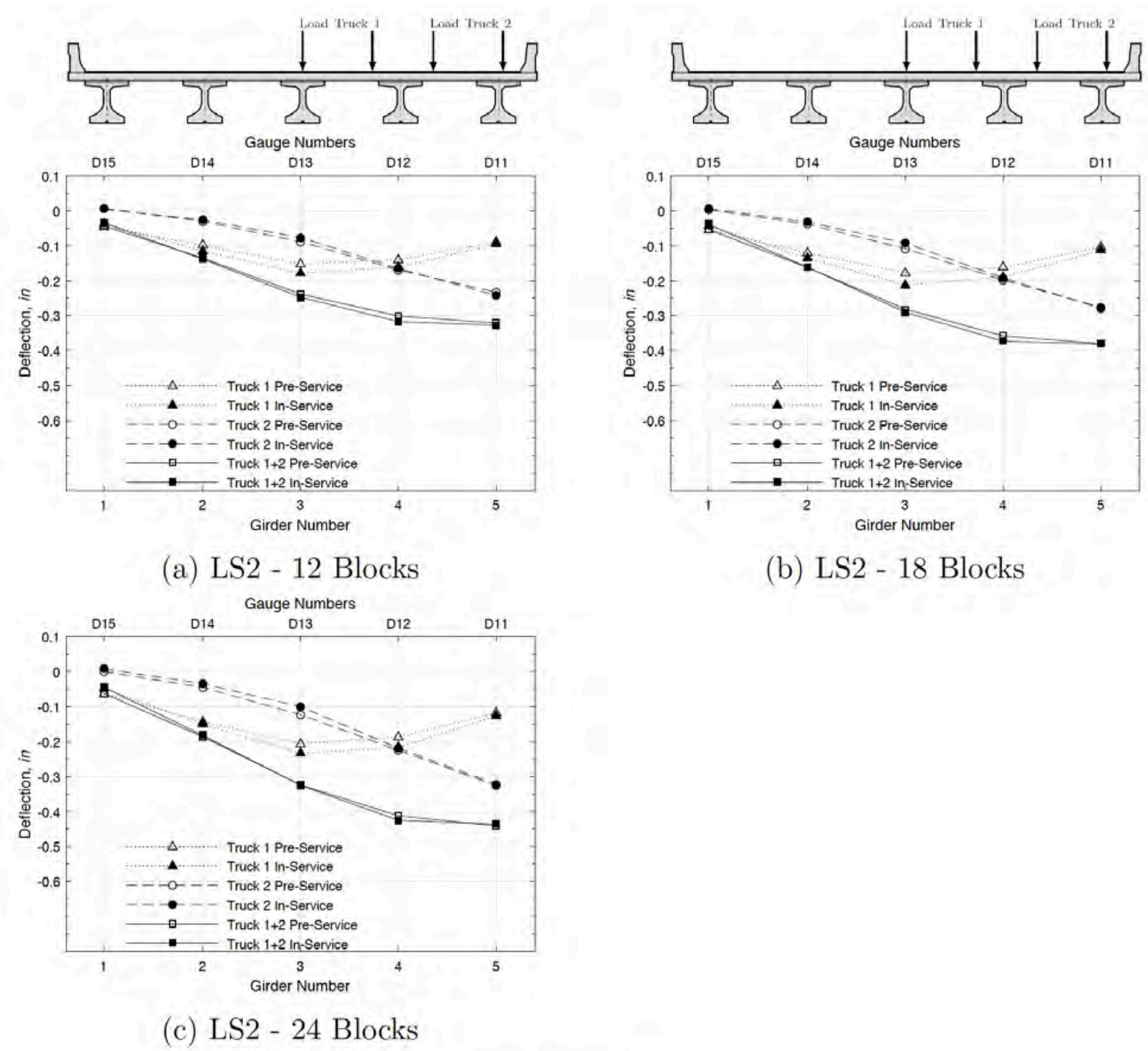


Fig. 5-6. Deflection Mid Span – LREB – LS2

5.2.4 Non-Prestressed Concrete Panel Bridges - Longitudinal Strain Results

Longitudinal strains were measured on both sides of the center line of the bottom flange of each girder at section B. The strain gauges were placed 1 in. off the center line on both sides. Average results of the gauge pairs were used for the plots except in cases where one of the gauges experienced drifts, in which case that gauge was excluded. Changes were made to gauge numbers used for the pre-service test on HCEB.

The pre-service test for HCEB utilized the following gauge numbers (in pairs, on five girders): F5/B5, F4/B4, F3/B3, F2/B2, and F1/B1, which were replaced by F5/F52, F4/F50, F3/F48, F2/F46, and F1/F44 for the in-service test. However, LREB maintained the same gauge numbers for both tests. These numbers are the same as those for the in-service test on HCEB. Results are presented for 12, 18, and 24 block cases at load positions 1/1A/1B and 2.

Load Position 1/1A/1B: Strains generally increased with the load, as expected. Maximum strains were recorded on FIB3 when both trucks were placed on the bridges for this load position. In-service strains were also higher than pre-service strains for the most part, although the in-service strain gauges on some exterior girders recorded lower strains than their pre-service counterparts.

For HCEB, load position 1, maximum strains (at FIB3) were 62 $\mu\varepsilon$, 77 $\mu\varepsilon$, and 89 $\mu\varepsilon$ for 12, 18, and 24 blocks, respectively, for the pre-service test (**Figure 5-7**). The in-service test showed maximum strains of 71 $\mu\varepsilon$, 84 $\mu\varepsilon$, and 91 $\mu\varepsilon$ for those block cases. This is an average increase of 8.8%.

For LREB, load position 1A, recorded maximum strains (at FIB3) were 65 $\mu\varepsilon$, 77 $\mu\varepsilon$, and 87 $\mu\varepsilon$ for 12, 18, and 24 blocks in the pre-service test, and 70 $\mu\varepsilon$, 79 $\mu\varepsilon$, and 98 $\mu\varepsilon$ in the in-service test (**Figures 5-8a, b, and c**). This is an average increase of 7.5%. For load position 1B, the maximum strain increased from 81 $\mu\varepsilon$ to 91 $\mu\varepsilon$ (**Figure 5-8d**).

Load Position 2: Maximum strains recorded on HCEB - FIB5 increased progressively after being in service. Pre-service, the maximum strains were 68 $\mu\varepsilon$, 80 $\mu\varepsilon$, and 86 $\mu\varepsilon$ for 12, 18, and 24 blocks, respectively. The in-service strains, however, were 94 $\mu\varepsilon$, 119 $\mu\varepsilon$, and 112 $\mu\varepsilon$. This is an average increase of 39% (**Figure 5-9**).

For LREB, load position 2, maximum strains were 67 με, 80 με, and 93 με for the pre-service test, for 12, 18, and 24 blocks, respectively. They were 63 με, 69 με, and 88 με for the in-service test. This is an average decrease of 8.2% (**Figure 5-10**).

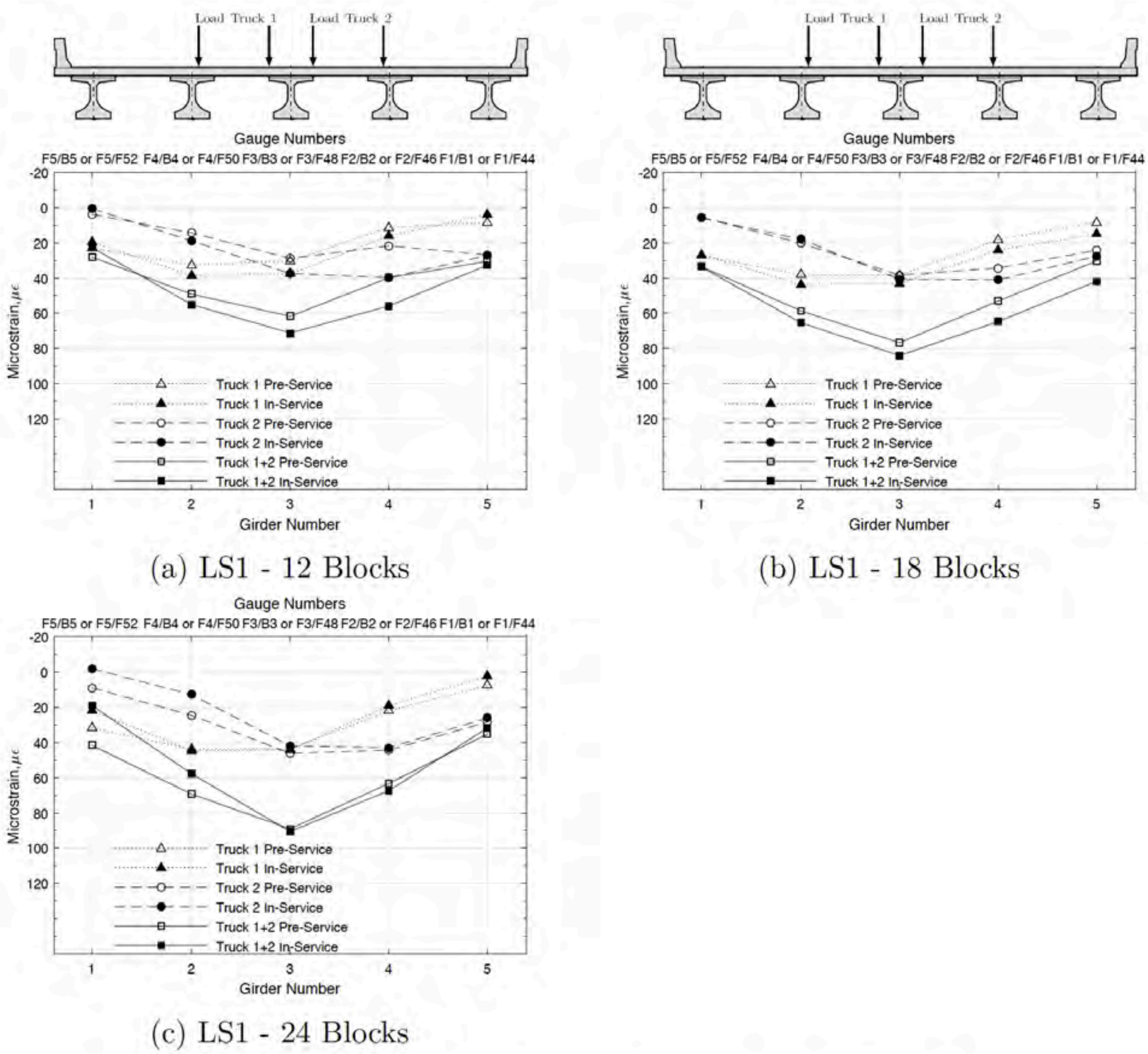


Fig. 5-7. Longitudinal Strain – Section B – HCEB – LS1

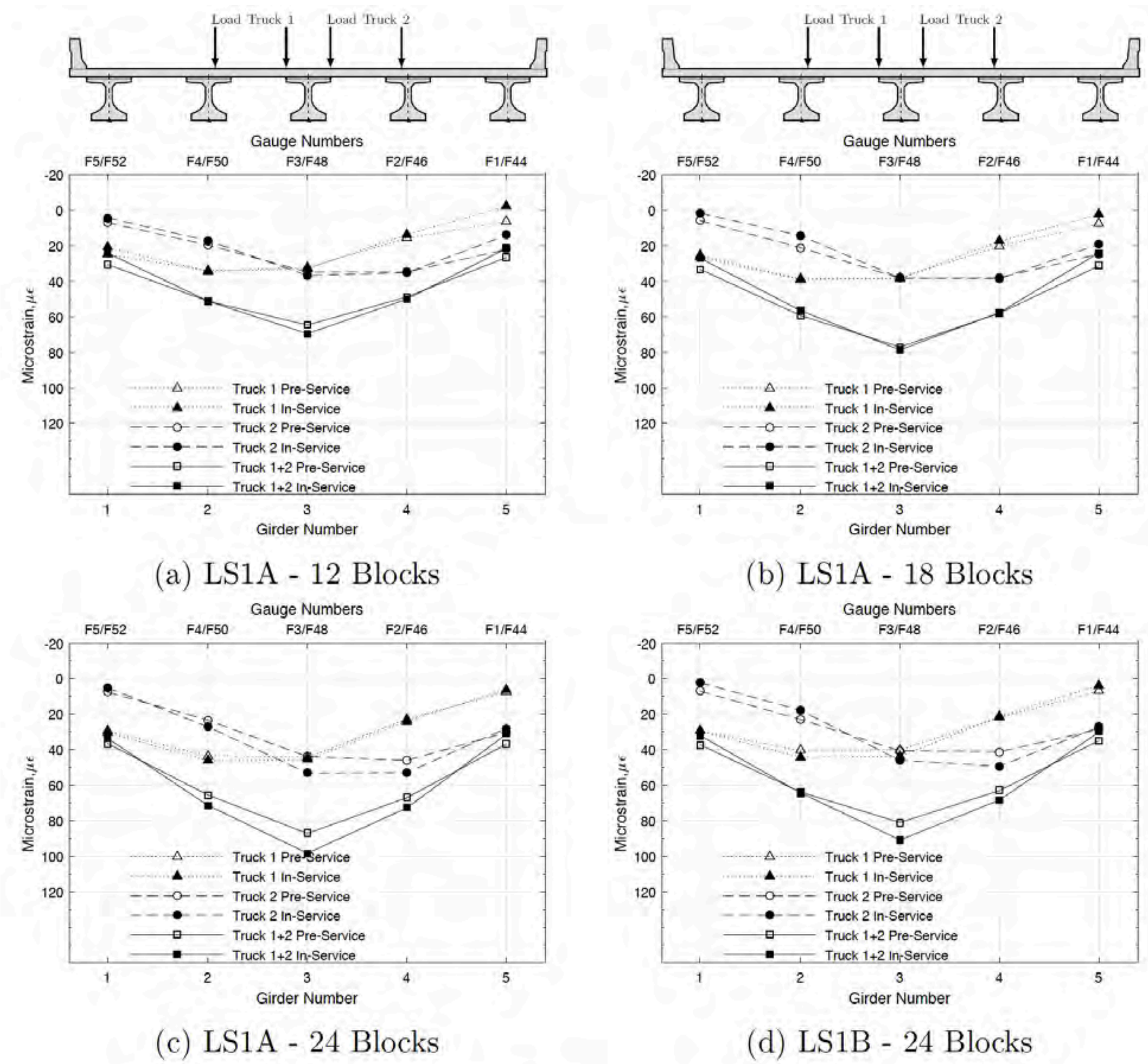


Fig. 5-8. Longitudinal Strain – Section B – LREB – LS1A/1B

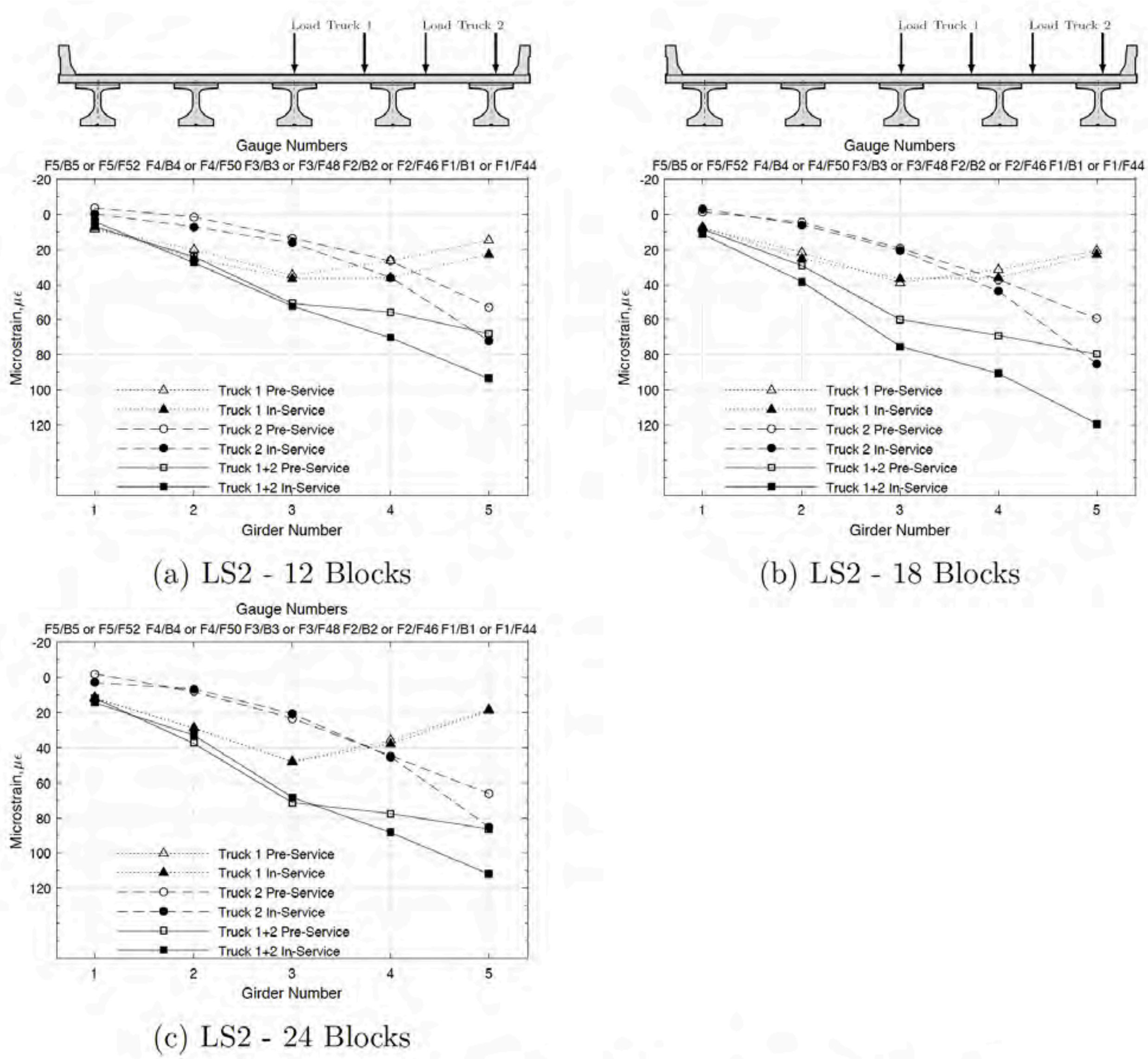


Fig. 5-9. Longitudinal Strain – Section B – HCEB – LS2

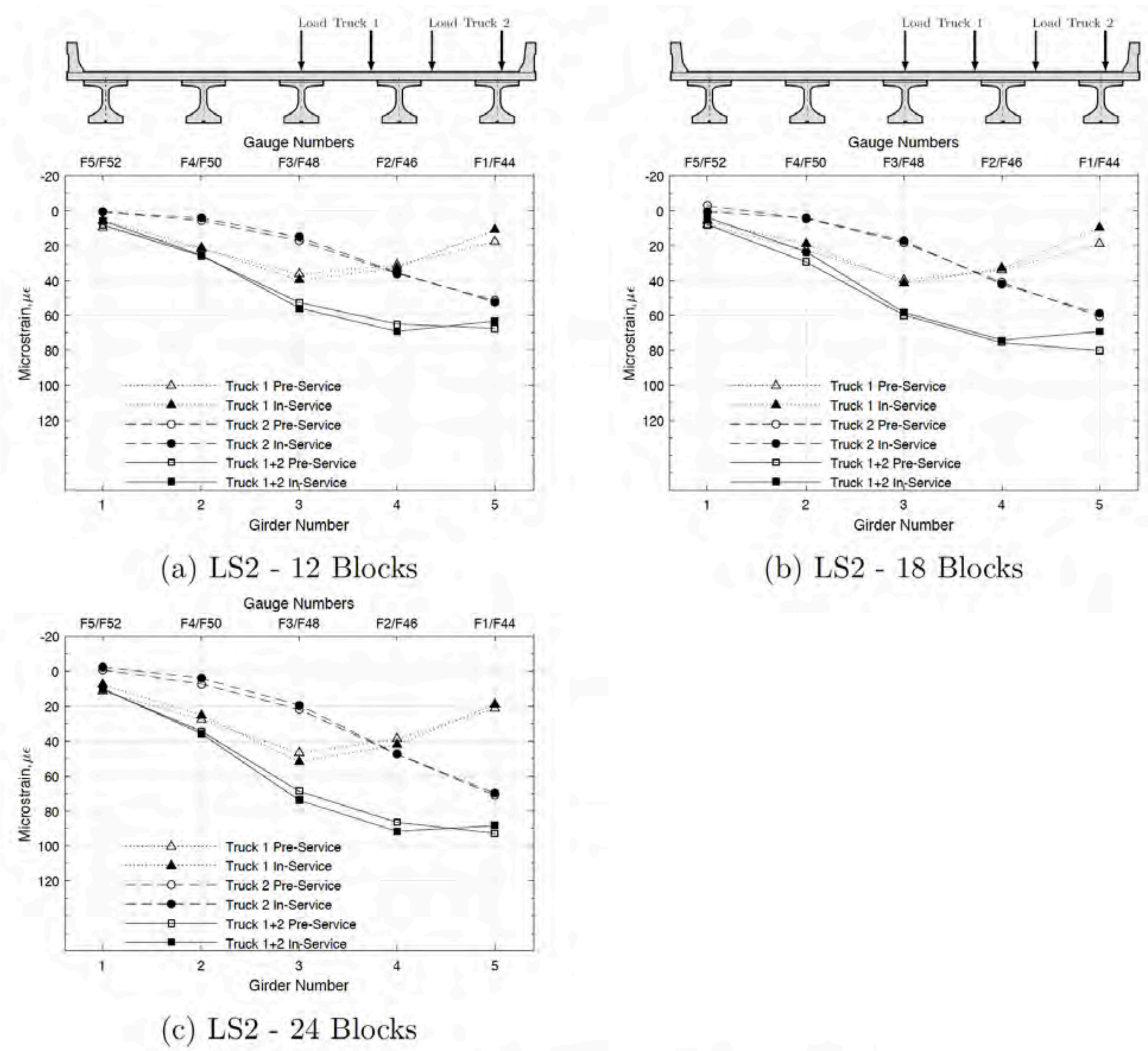


Fig. 5-10. Longitudinal Strain – Section B – LREB – LS2

5.2.5 Non-Prestressed Concrete Panel Bridges - Transverse Strain Results

Unlike the deflection and longitudinal strain data, which captured global responses of the bridge superstructures to applied live load, the transverse strain data are responses local to the bridges' deck panels. Transverse strain gauges were placed between adjacent girders on the underside of the bridges' deck panels at section B and on the underside of the joints between the panels at section C.

Results at both sections were inconclusive. As shown in previous chapters, there was some unexpected variability in the measurements. For example, the data was not symmetric, even when the loads were placed symmetrically. Therefore, the pre-service and in-service tests will not be explored here.

5.3 Prestressed Concrete Panel Bridges

5.3.1 Prestressed Concrete Panel Bridges - Overview

The westbound bridges, HCWB and LRWB, were constructed with prestressed deck panels. In this section, the pre-service and in-service test results for the different load positions will be considered. The graphs for HCWB and LRWB show results for load positions 1A, 1B, 2, 3A and 3B. The results presented are either at mid span, third points, section B (9'-0" west of mid span), or Section C (4'-6" west of mid span).

5.3.2 Prestressed Concrete Panel Bridges - Composite Action

Composite action was checked using the in-service load test data. Gauge readings from the longitudinal strains on the cross sections along HCWB FIB5 and LRWB FIB5 were plotted against their vertical locations. Results were only presented for load position 2, since this position maximized load effects on FIB5.

Figure 5-11 shows longitudinal strains on HCWB FIB5, measured with gauges F64, F36, F6, F40, and F1, which are located on top of the barrier, top of the deck panel, top flange of the girder, on the web of the girder, and below the bottom flange of the girder, respectively. The graph shows that the strains varied approximately linearly with the depth, which confirms composite action was achieved. Compression at the top and the tension at the bottom both increased with increasing number of blocks. The average neutral axis position was approximately 36 in. measured from the bottom of the composite section as shown in Figure 5-11. For the pre-service tests, the neutral axis was approximately 35 in. from the bottom of the section.

Longitudinal strains on LRWB FIB5 are shown in **Figure 5-12**. The strains varied approximately linearly with the depth. Generally, as the number of blocks increased from

12 to 24, the compression at the top and the tension at the bottom both increased. The neutral axis for the composite section was approximately at 34 in. measured from the bottom of the composite section. For the pre-service tests, the neutral axis was approximately 34 in. from the bottom of the section.

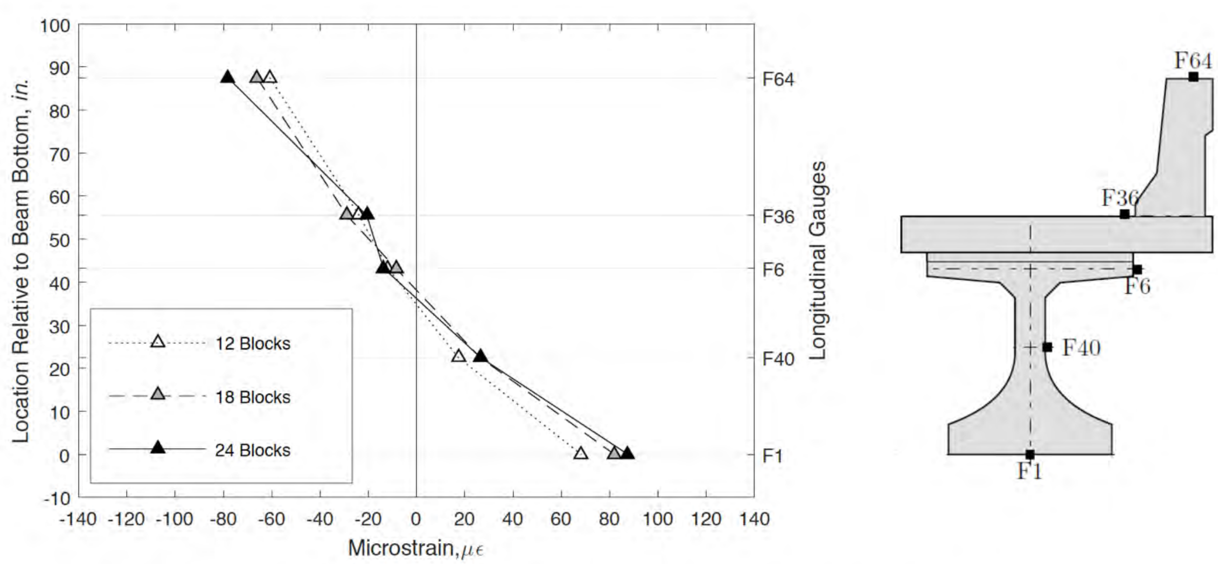


Fig. 5-11. Composite Action – HCWB – FIB5 – LS2

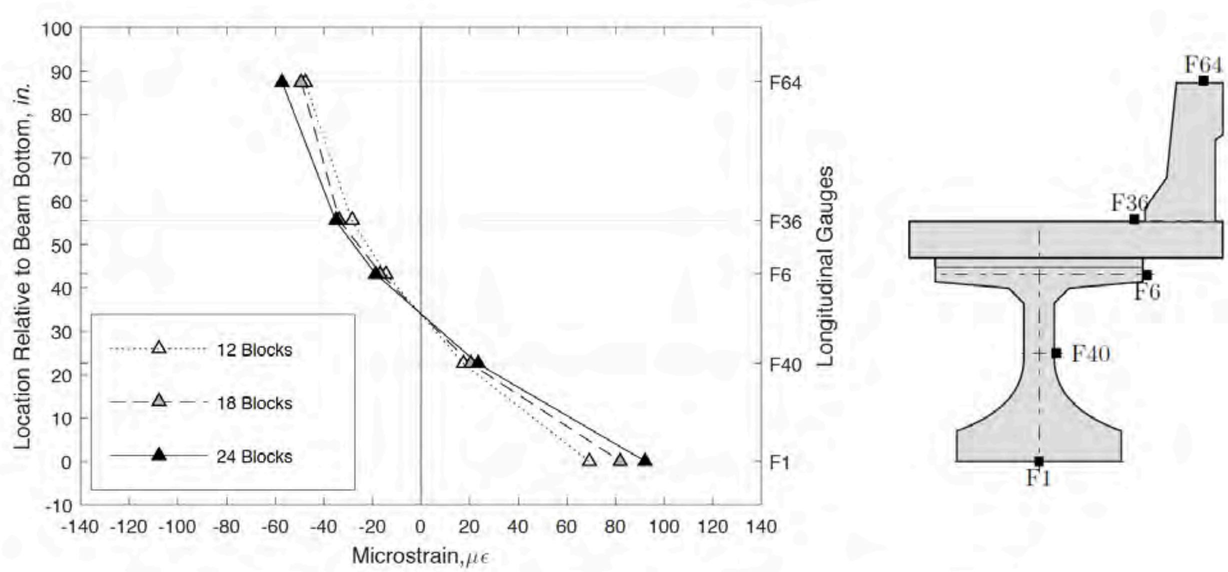


Fig. 5-12. Composite Action – LRWB – FIB5 – LS2

5.3.3 Prestressed Concrete Panel Bridges - Deflection Results

Deflections generally increased as the number of blocks in the trucks increased. The results presented are for the 12, 18, and 24 blocks load cases. These results show similar trends. Deflection distributions (relative magnitudes) measured at mid span are similar to those measured at section E.

Load Position 1A/1B: **Figures 5-13 and 5-14** show the results for the pre-service and in-service load tests for HCWB and LRWB at load position 1A/1B.

From the pre-service and in-service plots for HCWB, load position 1A (Figures 5-13a, b, and c), maximum deflections (at FIB3) were 0.30", 0.35" and 0.40" for 12, 18, and 24 blocks, respectively, for the pre-service test. The in-service test shows maximum deflections to be 0.32", 0.37" and 0.42" for those block cases. This was an average increase of 4.8% between the pre-service and in-service results. Figure 5-13d shows maximum pre-service strain to be 0.39" and the in-service strain to be 0.40", an increase of 2.6% for load position 1B.

Figure 5-14 shows the pre-service and in-service plots for LRWB, for load position 1A/1B. For position 1A (Figures 5-14a, b and c), maximum deflection (at FIB3) was 0.27", 0.32" and 0.38" for 12, 18 and 24 blocks, respectively, for the pre-service test. The in-service test showed maximum deflection of 0.32", 0.38" and 0.43" for those block cases. This reflected an average increase of 17% between the pre-service and in-service results. Figure 5-14d shows that deflection increased from 0.35" to 0.41" which is also a 17% increase for load position 1B.

Load Position 2: **Figures 5-15 and 5-16** show the results for the pre-service and in-service load tests for HCWB and LRWB at load position 2.

The pre-service and in-service plots for HCWB, load position 2, are shown in Figure 5-15. Maximum deflections (at FIB5) were 0.34", 0.40", and 0.45" for 12, 18, and 24 blocks, respectively, for the pre-service test. The in-service test produced maximum deflections of 0.32", 0.39" and 0.44", for an average decrease of 4%.

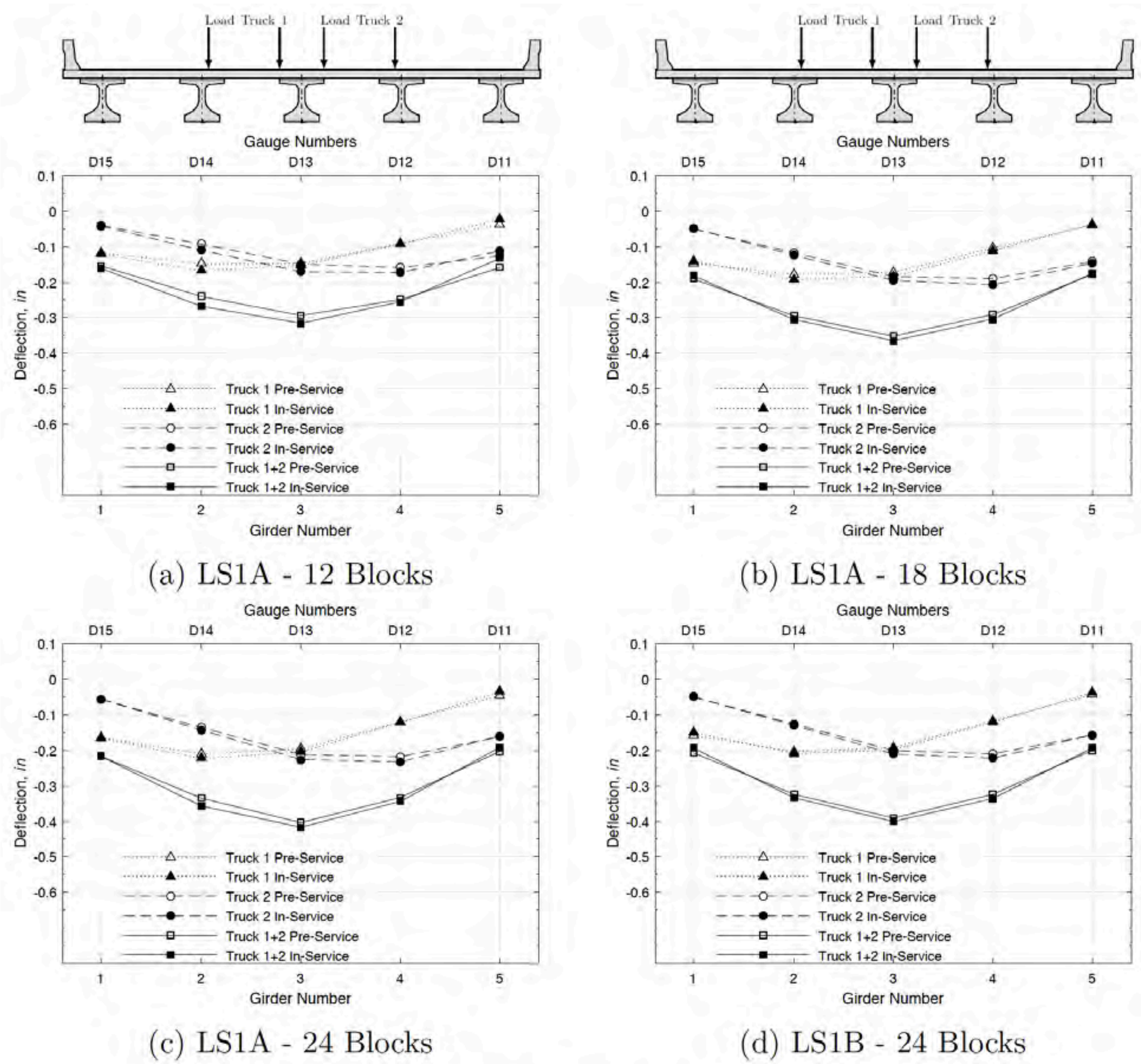


Fig. 5-13. Deflection Mid Span – HCWB – LS1A/1B

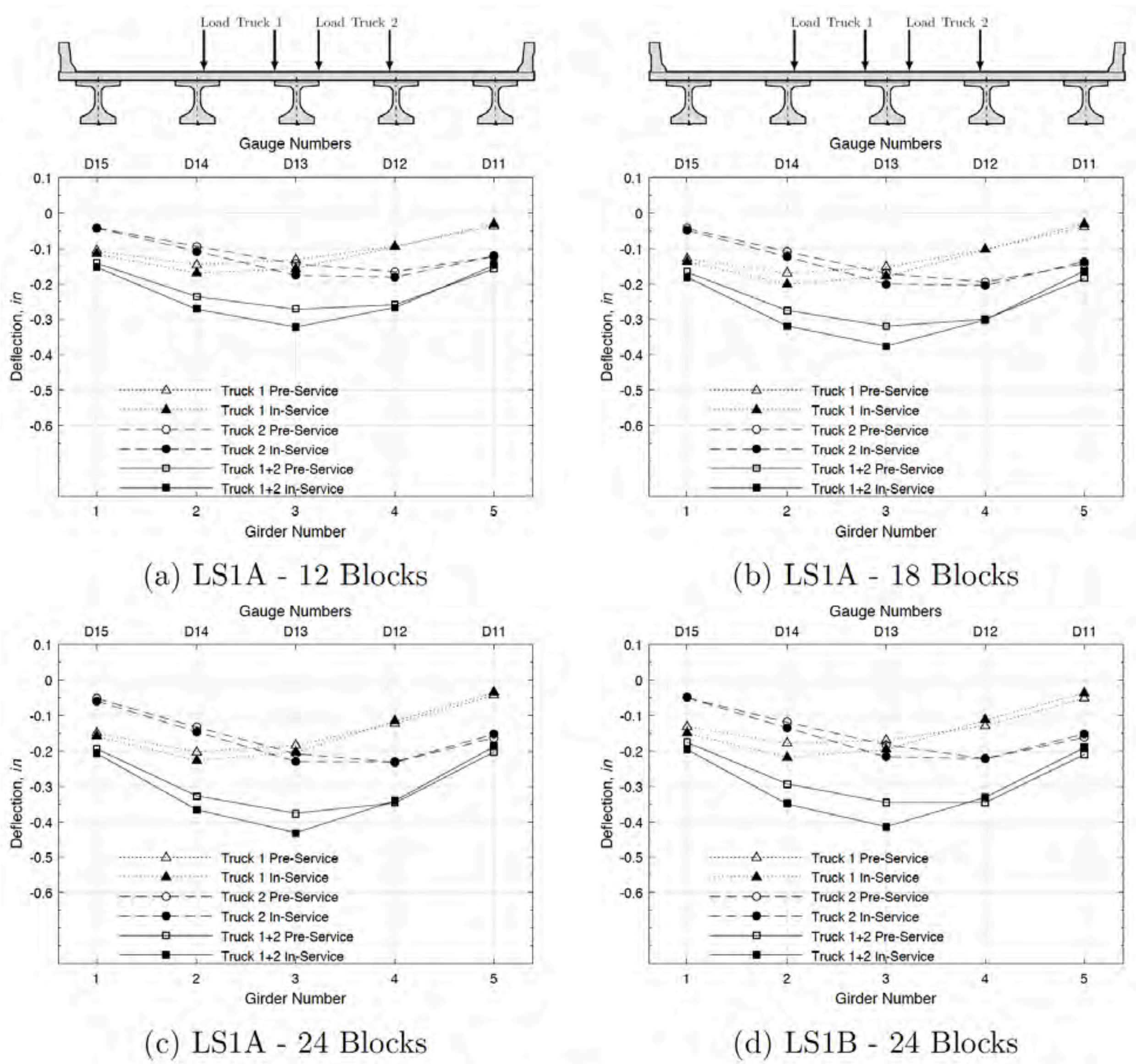


Fig. 5-14. Deflection Mid Span – LRWB – LS1A/1B

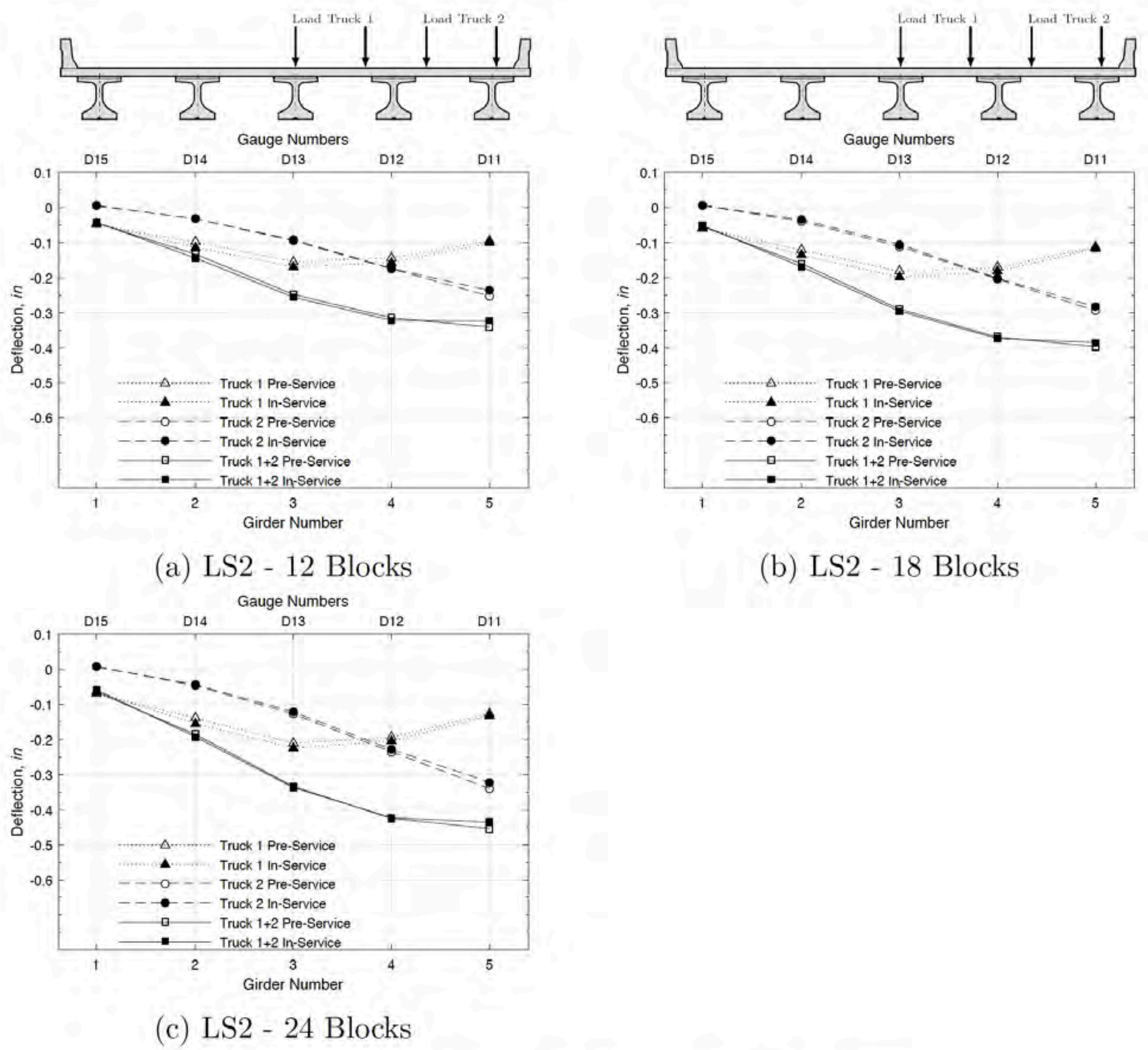


Fig. 5-15. Deflection Mid Span – HCWB – LS2

Figure 5-16 shows the pre-service and in-service plots for LRWB, load position 2. When the plots of the two tests were compared, the change in maximum deflection was very small for each of the block cases. Values for the pre-service test were 0.33", 0.39", and 0.45" for 12, 18, and 24 blocks, respectively. For the pre-service test, they were 0.34", 0.38", and 0.44", which is a slight decrease.

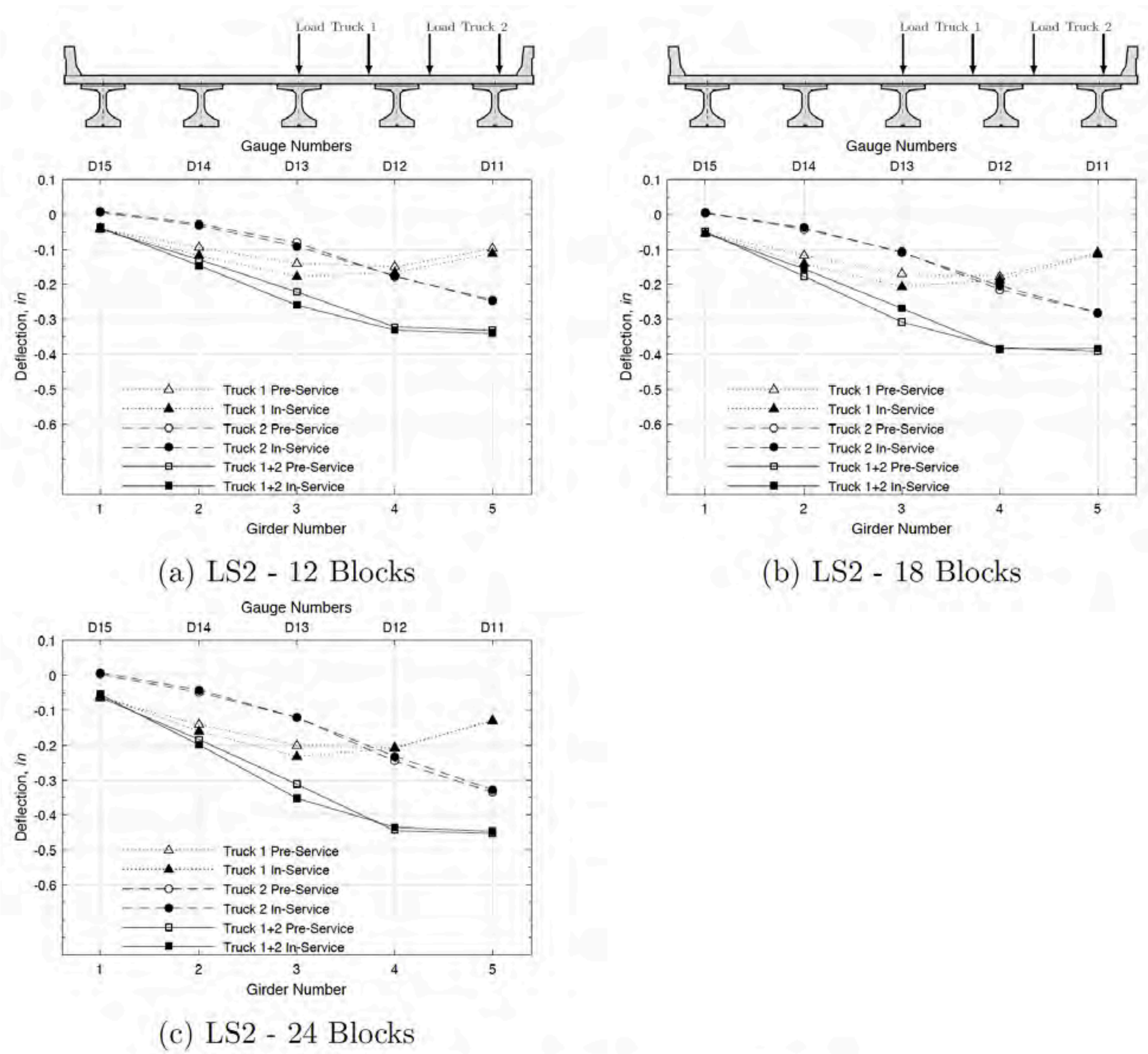


Fig. 5-16. Deflection Mid Span – LRWB – LS2

5.3.4 Prestressed Concrete Panel Bridges - Longitudinal Strain Results

Longitudinal strains were measured on both sides of the center line of the bottom flange of each girder at section B, just as for the eastbound bridges. Average results of the gauge pairs were used for the plots except in cases where one of the gauges experienced drifts. Gauge numbers for the westbound bridges (in pairs, on five girders) were F5/F52, F4/F50, F3/F48, F2/F46, and F1/F44 for both the pre-service and in-service tests. Results are presented for 12, 18, and 24 block cases at load positions 1A/1B and 2.

***Load Position 1A/1B*:** Maximum strains recorded on FIB3 for HCWB, load position 1A, were 61 με, 74 με, and 85 με for 12, 18, and 24 blocks, respectively, for the pre-service test. The in-service test shows maximum strains to be 66 με, 73 με, and 88 με for those block cases. This was an average increase of 3.3% (**Figure 5-17a, b, and c**). For HCWB, load position 1B, the maximum strain increased slightly as well (**Figure 5-17d**).

LRWB, load position 1A had maximum strains of 68 με, 78 με, and 87 με for 12, 18 and 24 block in the pre-service test, and 69 με, 79 με, and 91 με in the in-service test (**Figures 5-18a, b and c**). This is an average increase of 2.7%. For LRWB, load position 1B, the maximum strain went from 89 με to 85 με (**Figure 5-18d**).

***Load Position 2*:** Maximum strains recorded on HCWB FIB5 pre-service results were 69 με, 81 με, and 92 με for 12, 18, and 24 blocks, respectively. The in-service results, however, gave 68 με, 82 με, and 88 με. This is an average decrease of 1.6% (**Figure 5-19**).

For LRWB, load position 2, maximum strains were 73 με, 78 με, and 91 με for the pre-service test and 68 με, 81 με, and 91 με for the in-service test. This is an average decrease of 0.56% (**Figure 5-20**).

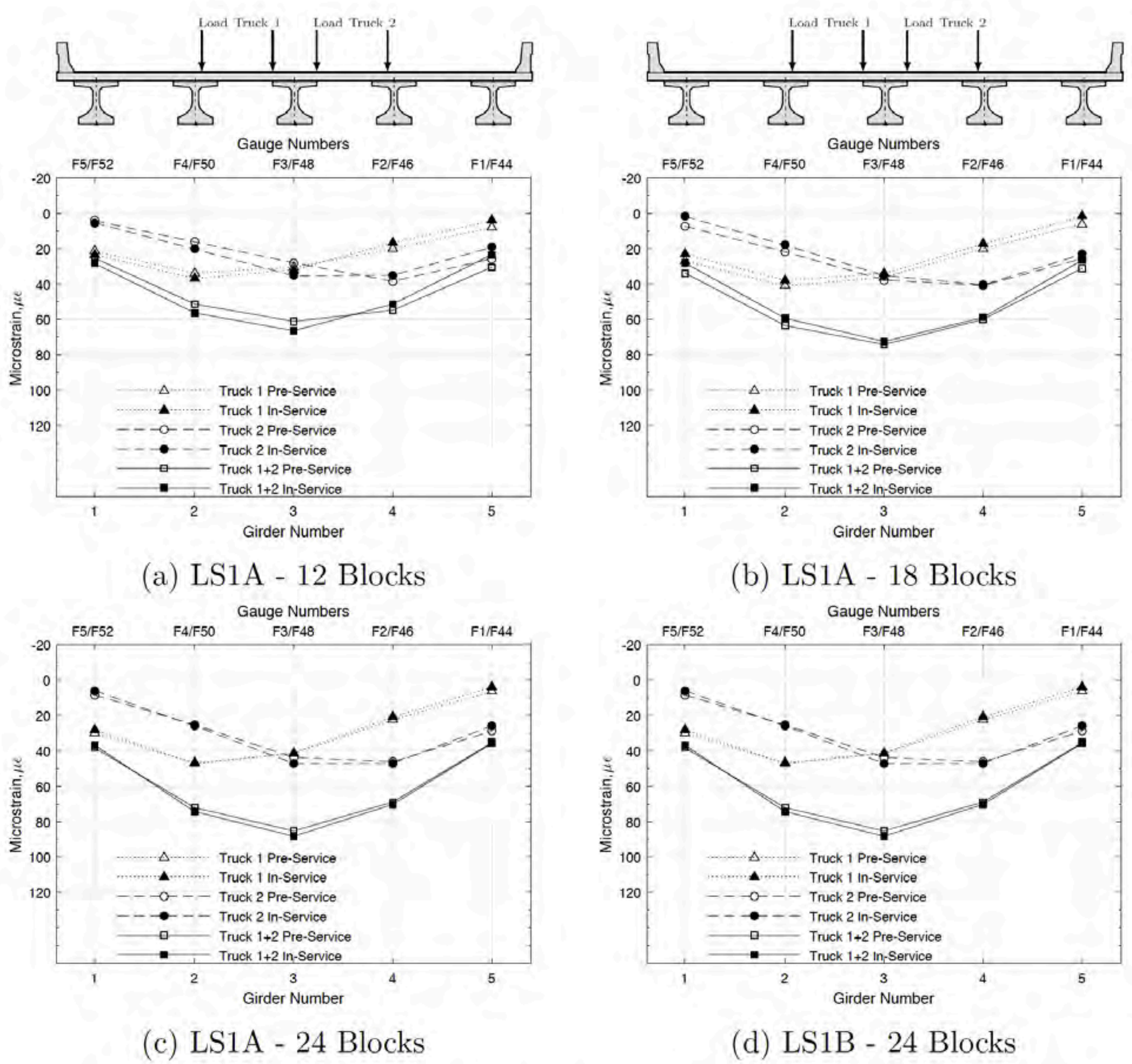


Fig. 5-17. Longitudinal Strain – Section B – HCWB – LS1A/1B

5.3.5 Prestressed Concrete Panel Bridges - Transverse Strain Results

Strains local to the deck panels and joint in HCWB and LRWB were measured by transverse strain gauges. These strain gauges were placed between adjacent girders on the underside of the deck panels of the bridges at section B and on the underside of the joint, between two panels, at section C. Results are presented for load positions 1A/1B and 3A/3B at sections B and C. For load positions 1A and 3A, the second and third axles of the trucks straddled section B, while for load positions 1B and 3B, the trucks straddled section B and C, with the goal of equally loading the sections.

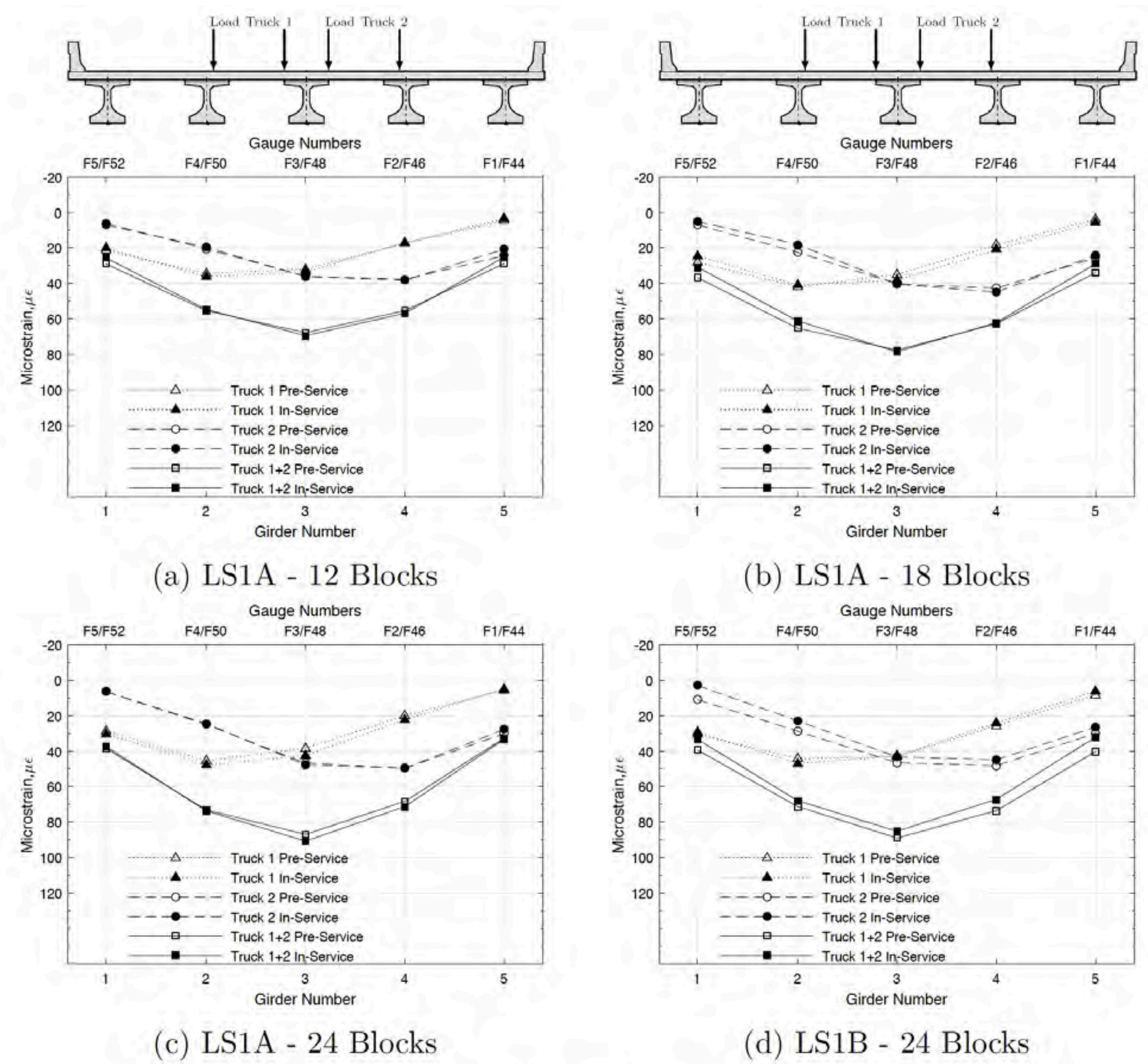
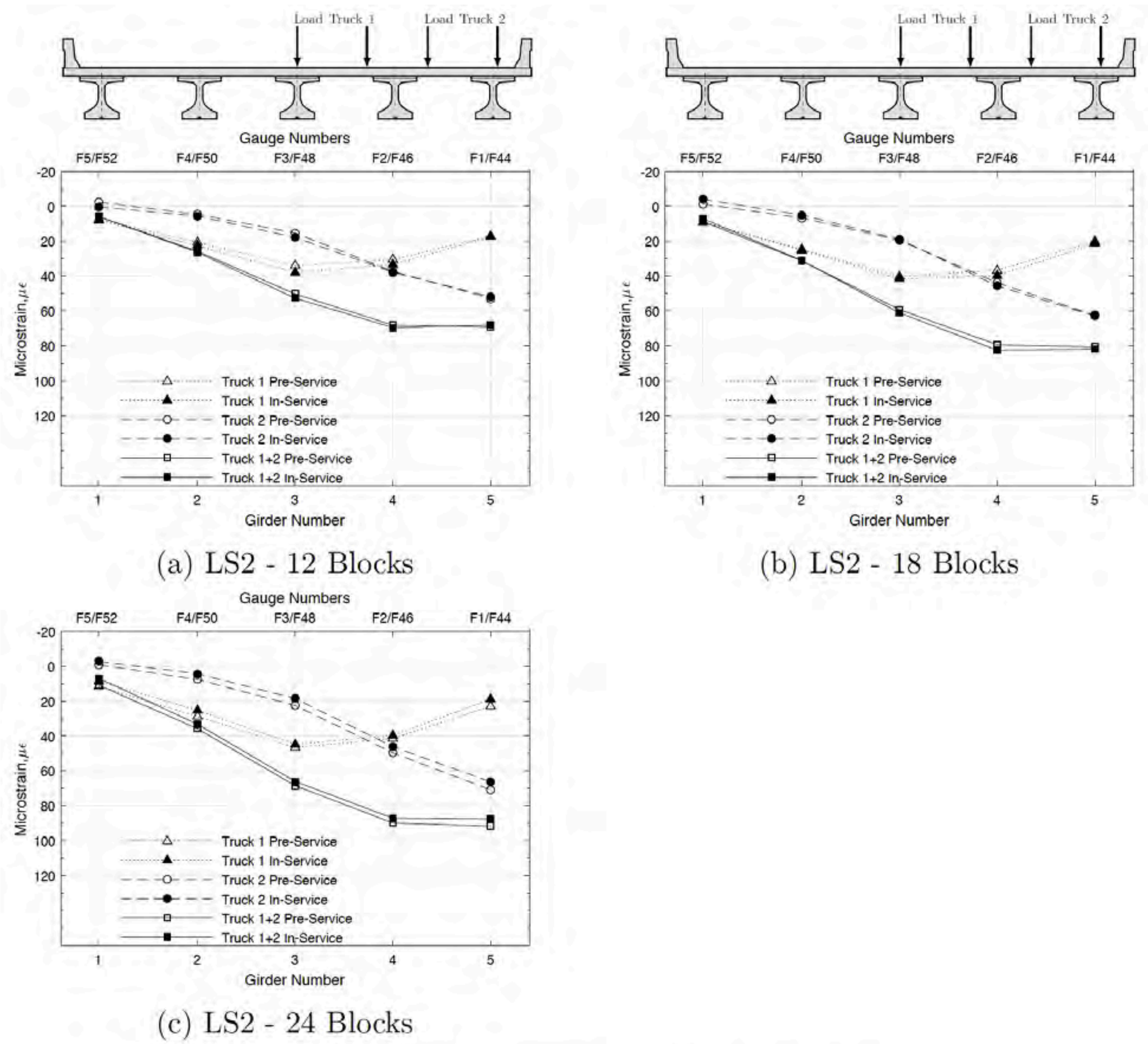


Fig. 5-18. Longitudinal Strain – Section B – LRWB – LS1A/1B

Load Position 1A/1B: Transverse strain results for HCWB and LRWB for load positions 1A and 1B at sections B and C are discussed in this section. Load positions 1A and 1B loaded the bridges symmetrically in the transverse direction. However, the results are not perfectly symmetrical.

Figure 5-21 shows transverse strain results at Section B for HCWB load position 1A/1B. As shown, the pre-service results are similar to in-service strain results, with the most change generally occurring in gauge B9, which is between FIB3 and FIB4. The trend also shows that B11 (between FIB2 and FIB3) had higher maximum strains for the pre-service test, whereas, gauge B9 recorded higher maximum strains for the in-service test. This could be attributed to a slight variation in the placement of the wheel loads, transversely.



(a) LS2 - 12 Blocks

(b) LS2 - 18 Blocks

(c) LS2 - 24 Blocks

Fig. 5-19. Longitudinal Strain – Section B – HCWB – LS2

Figure 5-22 shows transverse strain results at section C for HCWB load position 1A/1B. Here, the pre-service results show similar trends to in-service strain results. However, pre-service strains are slightly more than strains recorded for the in-service results. The trend shows that B10 (between FIB2 and FIB3) had maximum strains for both the pre-service and the in-service tests at section C for HCWB.

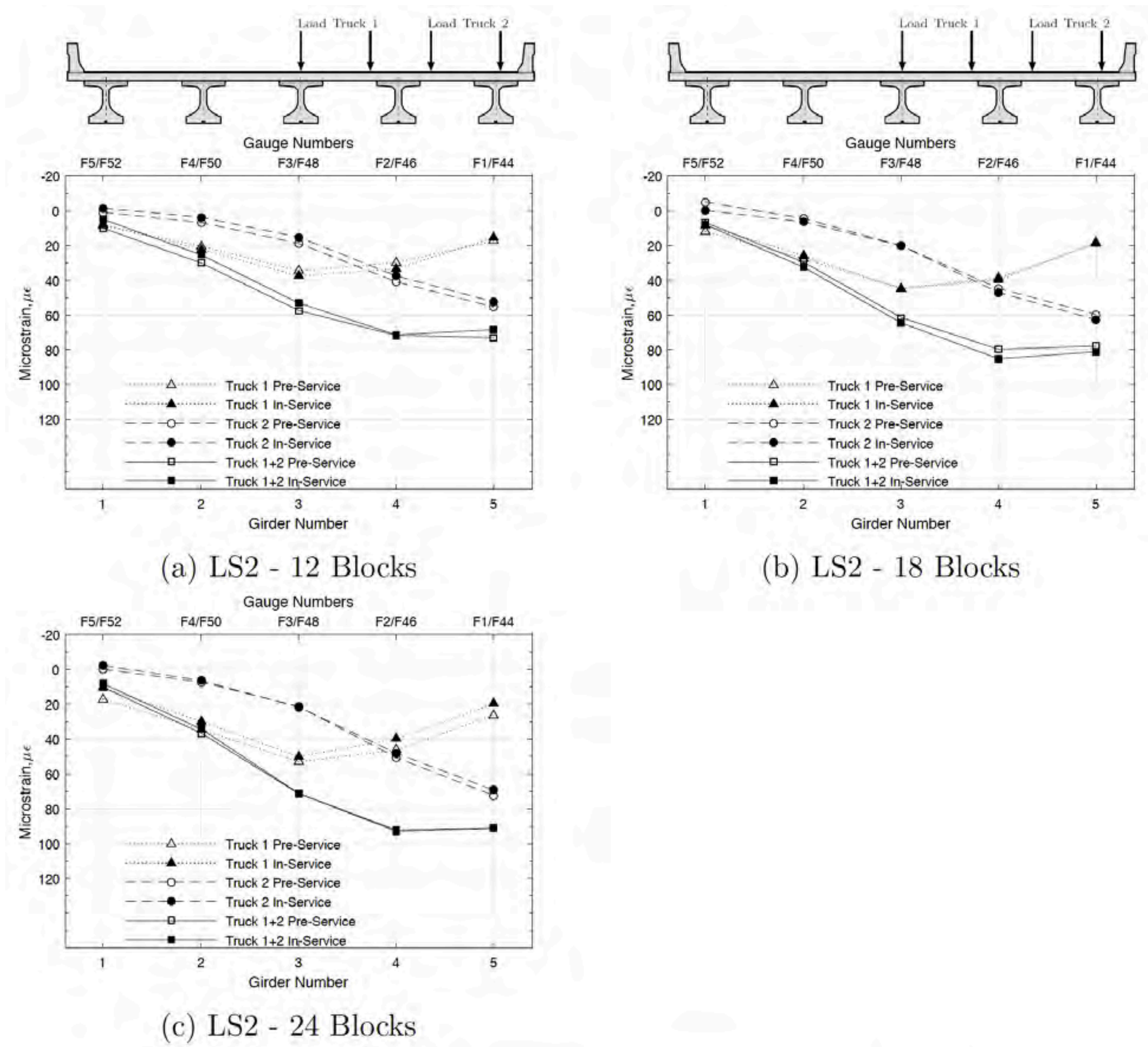


Fig. 5-20. Longitudinal Strain – Section B – LRWB – LS2

Figure 5-23 shows transverse strain results at Section B for LRWB load position 1A/1B. As shown, the pre-service results are similar to in-service strain results, although the 12 blocks load case exhibited the most change. The trend also shows that B11 (between FIB2 and FIB3) recorded the maximum strains for the pre-service and the in-service tests.

Figure 5-24 shows transverse strain results at section C for LRWB load position 1A/1B. Gauges at section C measured strains in the closure joints, as stated previously. Again, the pre-service results show similar trends to in-service strain results. The trend shows that

B10 (between FIB2 and FIB3) had the maximum strains for both the pre-service and the in-service tests. Pre-service strains generally exceeded in-service strains at this section.

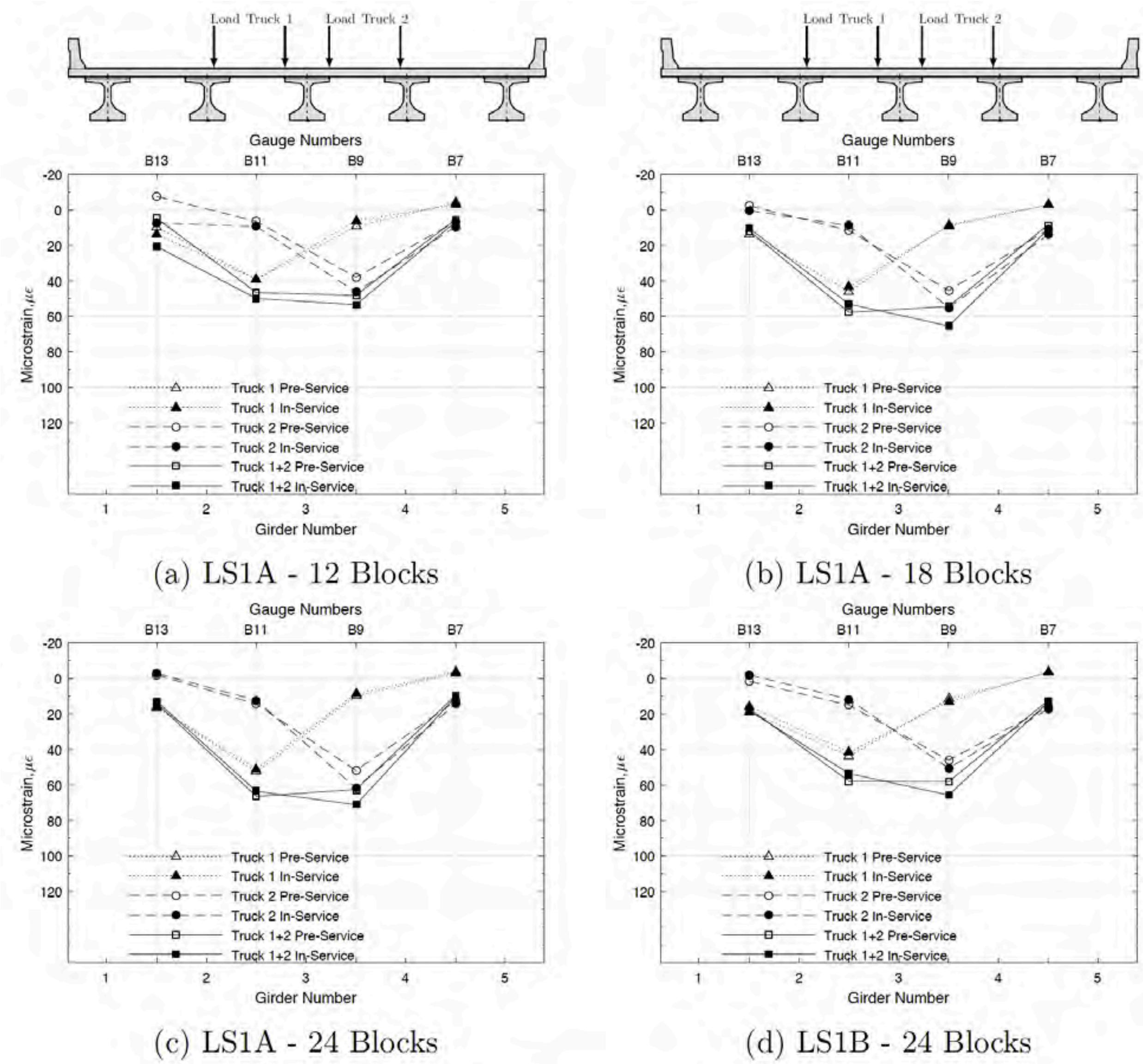


Fig. 5-21. Transverse Strain – Section B – HCWB – LS1A/1B

Load Position 3A/3B: Transverse strain results for HCWB and LRWB at load position 3A and 3B at sections B and C are discussed in this section.

Load positions 3A and 3B loaded the bridges closer to the south end, and they maximized strains between FIB3 and FIB4 - that is, gauge B9 at section B and gauge B8 at section C.

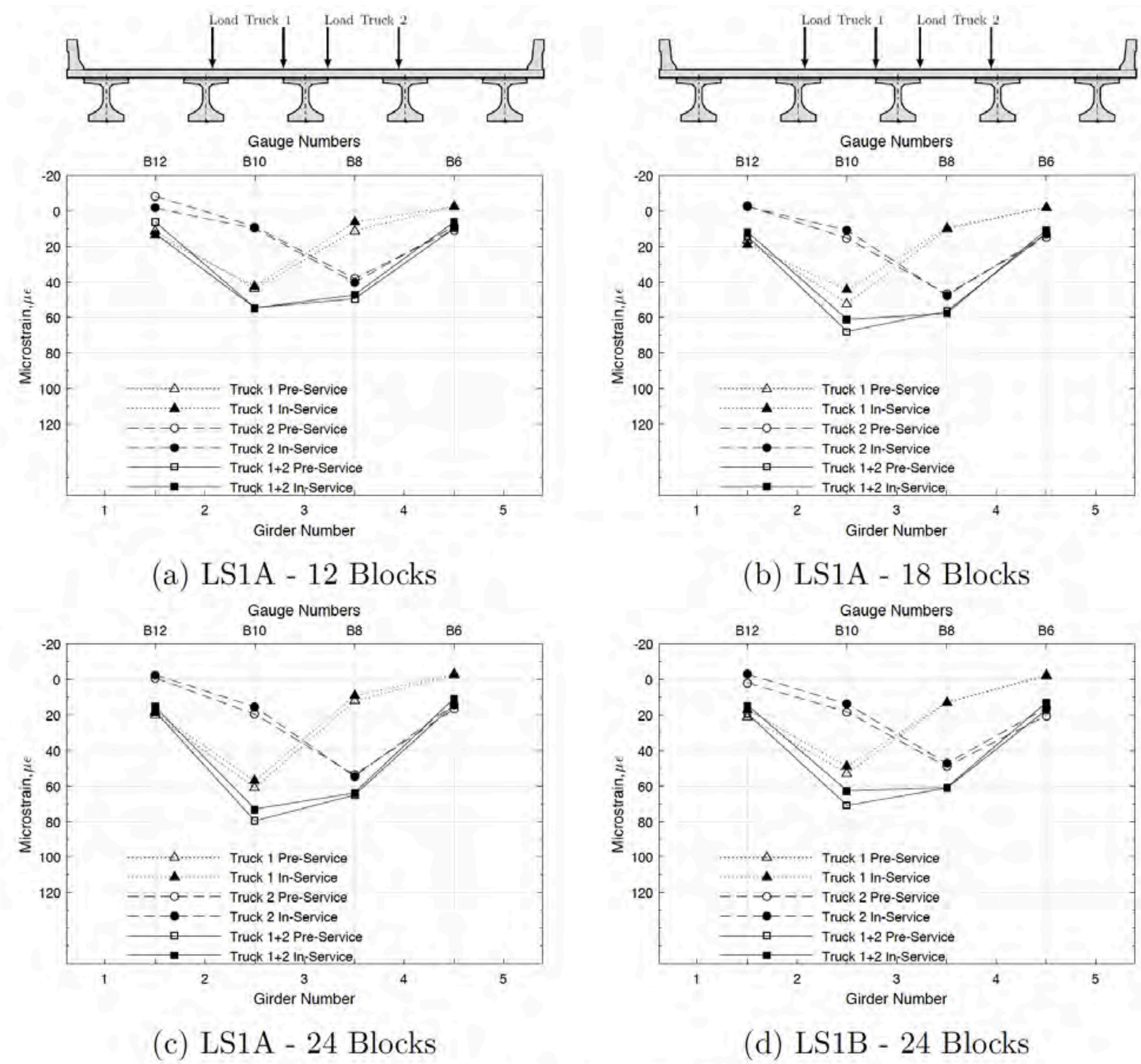


Fig. 5-22. Transverse Strain – Section C – HCWB – LS1A/1B

Figure 5-25 shows transverse strain results at Section B for HCWB, for load position 3A/3B. The trends for the pre-service and in-service strain results are similar. In-service strains slightly exceeded pre-service strains.

Figure 5-26 shows transverse strain results at section C for HCWB load position 3A/3B. Again, the pre-service results show similar trends to in-service strain results. Maximum strains were recorded at gauge B8 as expected. The pre-service strains are slightly more than the in-service strains.

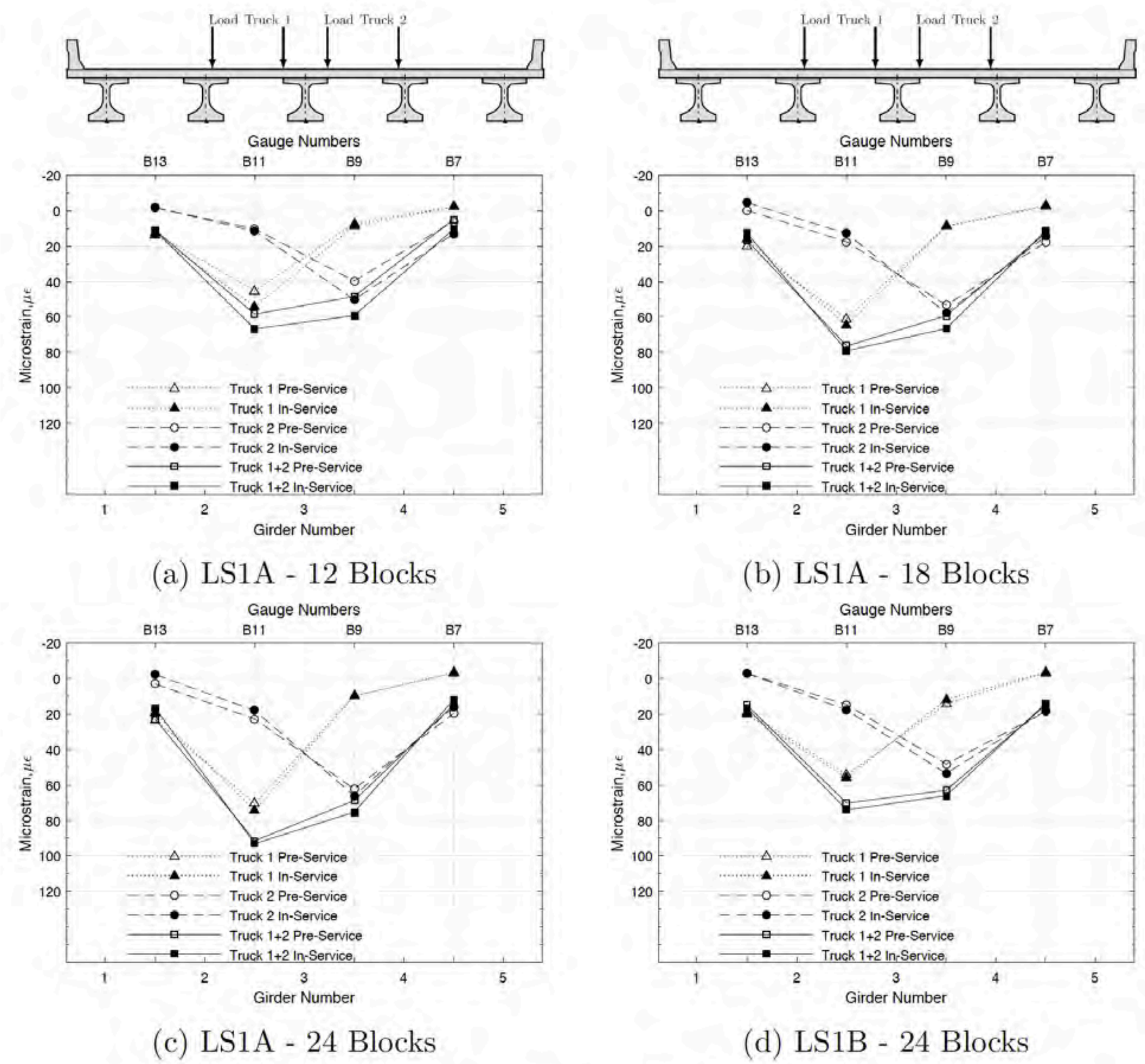
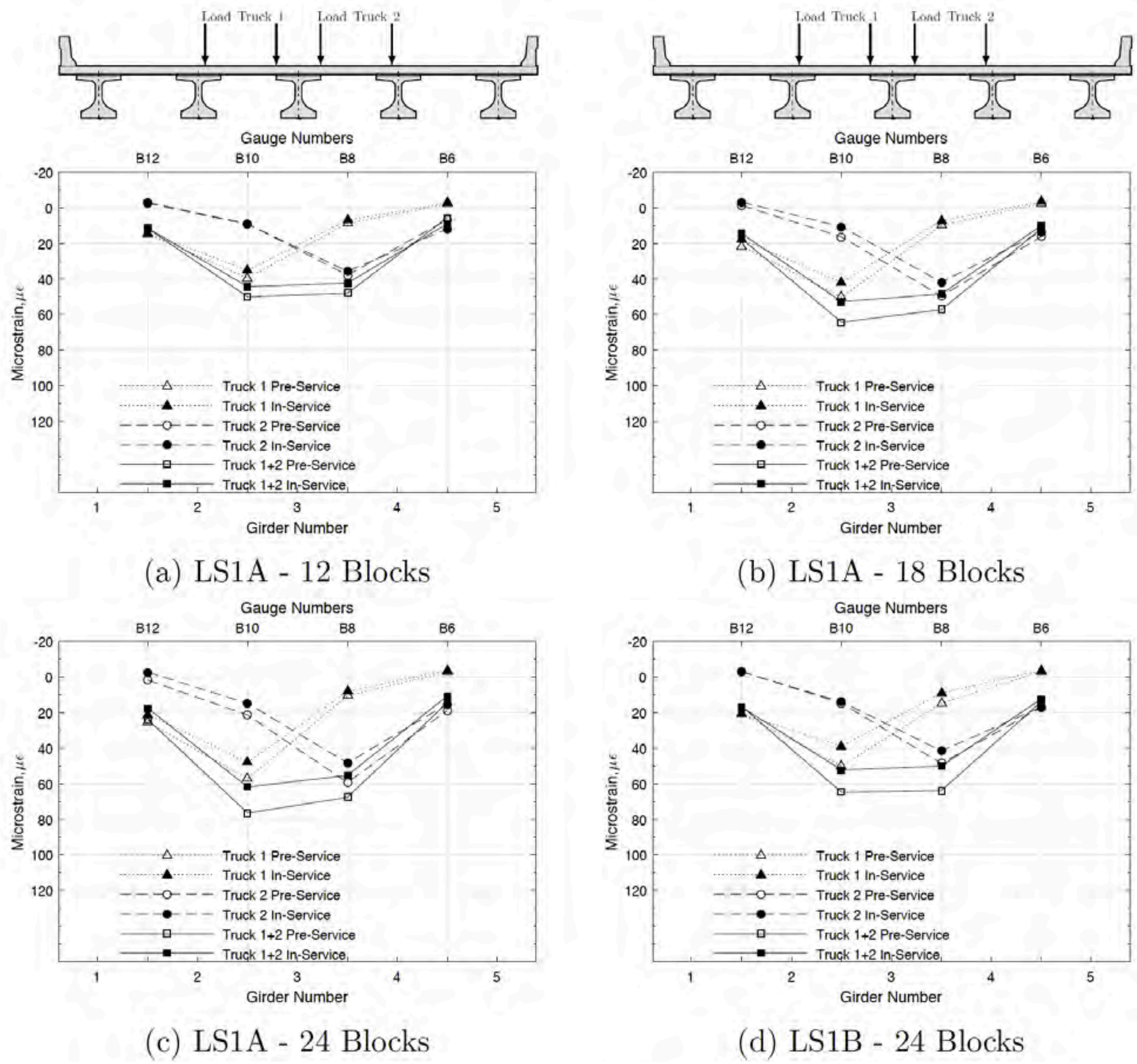


Fig. 5-23. Transverse Strain – Section B – LRWB – LS1A/1B

Figure 5-27 shows transverse strain results at Section B for LRWB load position 3A/3B. The trends for the pre-service and in-service strain results are similar for the 12 and 18 block cases. The 24 block case shows an unexpected trend for gauge B9 (between FIB3 and FIB4), especially in the pre-service results. This could be a result of gauge drift that was observed in B9. In-service strains generally slightly exceeded pre-service strains, except at gauge B9 for the 24 block cases as discussed.



(a) LS1A - 12 Blocks

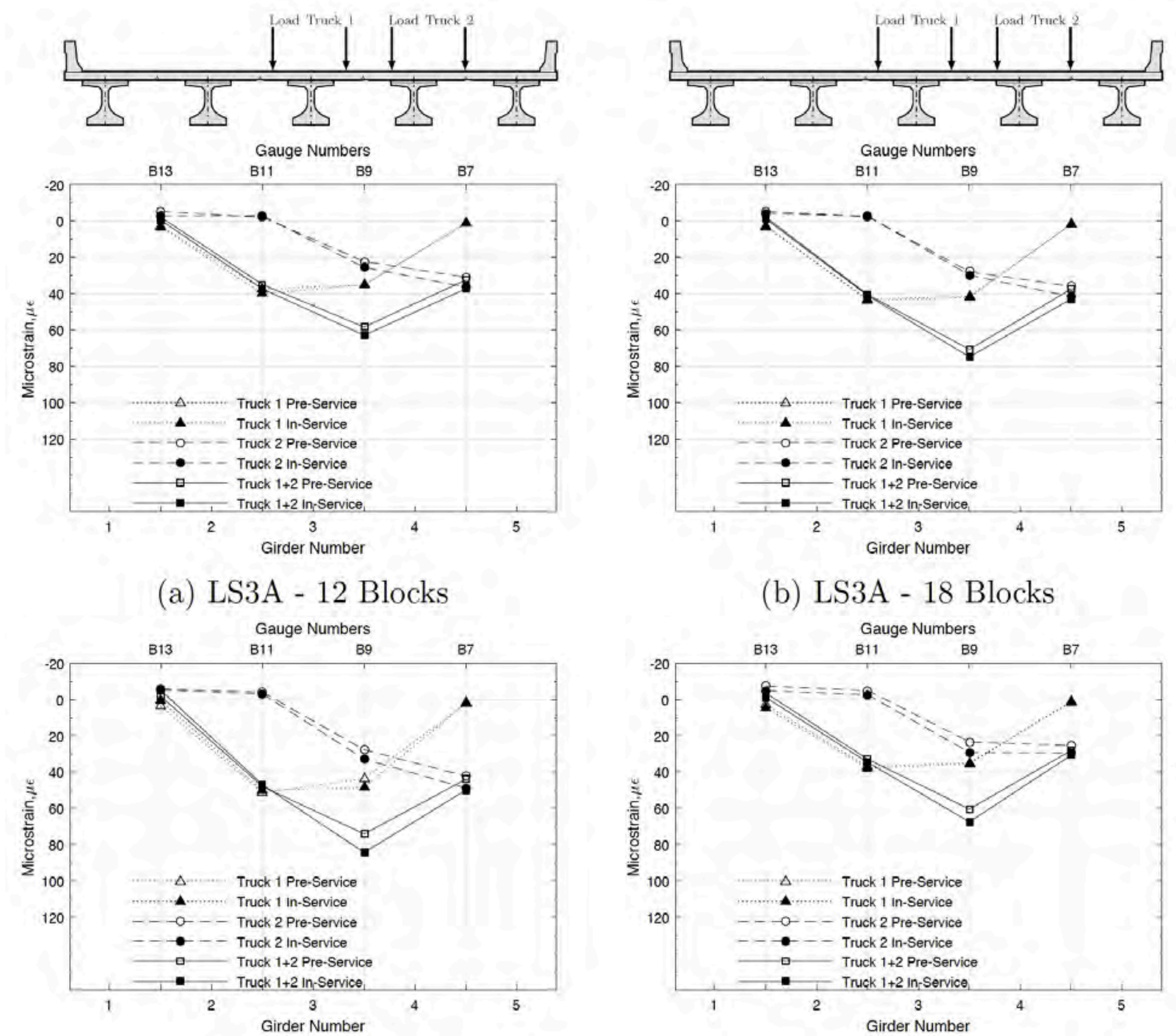
(b) LS1A - 18 Blocks

(c) LS1A - 24 Blocks

(d) LS1B - 24 Blocks

Fig. 5-24. Transverse Strain – Section C – LRWB – LS1A/1B

Figure 5-28 shows transverse strain results at section C for LRWB load position 3A/3B. The trends are similar for both the pre-service and the in-service tests. Maximum strain was recorded at gauge B8 as expected. The pre-service strains were more than the in-service strains.



(a) LS3A - 12 Blocks

(b) LS3A - 18 Blocks

(c) LS3A - 24 Blocks

(d) LS3B - 24 Blocks

Fig. 5-25. Transverse Strain – Section B – HCWB – LS3A/3B

5.4 Moment - Deflection Response

This section presents the theoretical moment vs. measured deflection responses of critical girders, for load positions 1/1A and 2. Deflections were measured at the bottom flange of the girders, at mid span. **Figures 5-29 to 5-31** compare the responses for the pre-service and in-service tests when both trucks were placed on each bridge. For load position 1/1A, FIB3 experienced the most deflection when two load trucks were placed. The responses of the first interior girder to the south, FIB4, were also analyzed. For load position 2, maximum deflection occurred at the exterior girder, FIB5.

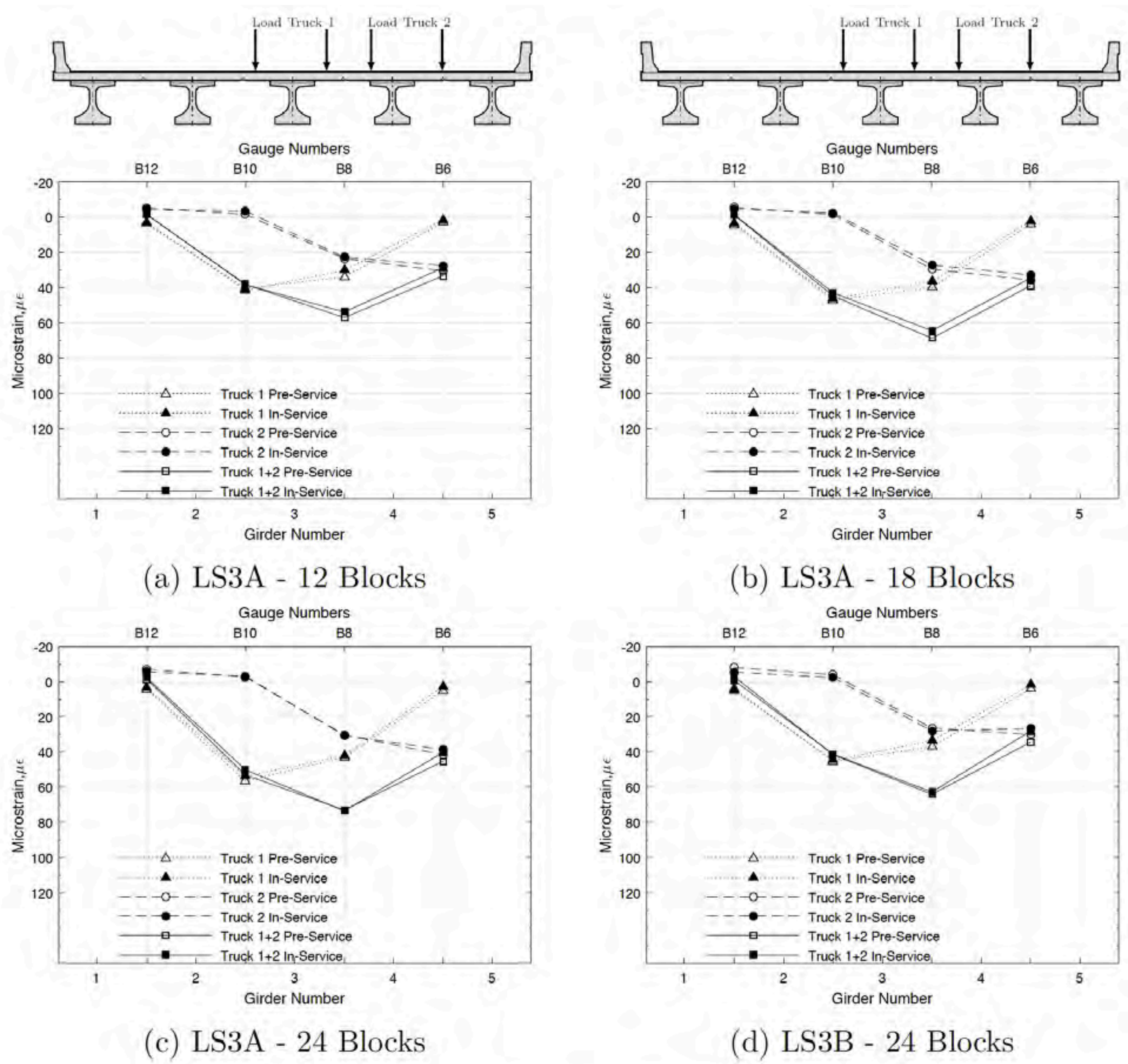
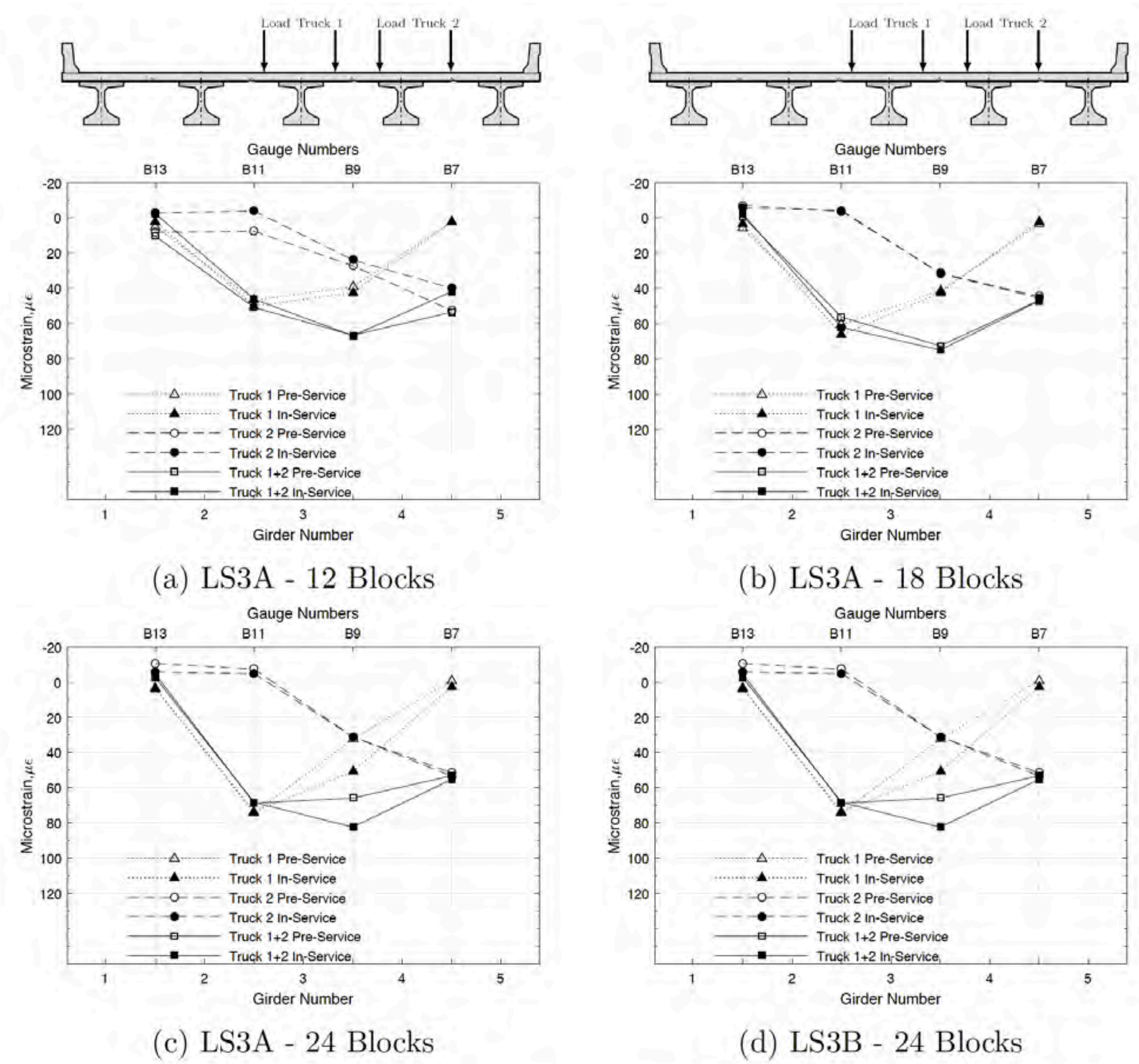


Fig. 5-26. Transverse Strain – Section C – HCWB – LS3A/3B

Figure 5-29 shows a linear moment - deflection relationship for FIB3 at load position 1/1A, for the tested span of each bridge. The response of the girder remained within the linear elastic limit. However, FIB3 on HCEB appears to have lost the most stiffness over time, compared to FIB3 on LRWB, HCWB, and LREB. The measured deflection on the girders increased by 18.1%, 16.8%, 13.8%, and 7.7%, respectively. Loss of stiffness is often attributed to cracking of members, in which case, one would expect HCEB and LREB bridge to lose more stiffness than HCWB and LRWB. (In the separate report, Report 4, Monitoring of Cracks in Precast Panels, where results of crack mapping of the precast deck panels are presented, it was concluded that the deck panels on the EB bridges cracked significantly more over time than the panels on the WB bridges.) However, LREB experienced the least stiffness loss of the four bridges. There also seems to be no correlation between stiffness loss and time in service: HCEB and LREB bridges were constructed before, and were in service longer, than the WB bridges. Nonetheless, the pre-service load tests were performed before the bridges were subjected to any traffic – i.e., the bridges were new – and therefore some "shakedown" was likely to occur once opened to traffic, which could cause an increase in deflections.



(a) LS3A - 12 Blocks (b) LS3A - 18 Blocks

(c) LS3A - 24 Blocks (d) LS3B - 24 Blocks

Fig. 5-27. Transverse Strain – Section B – LRWB – LS3A/3B

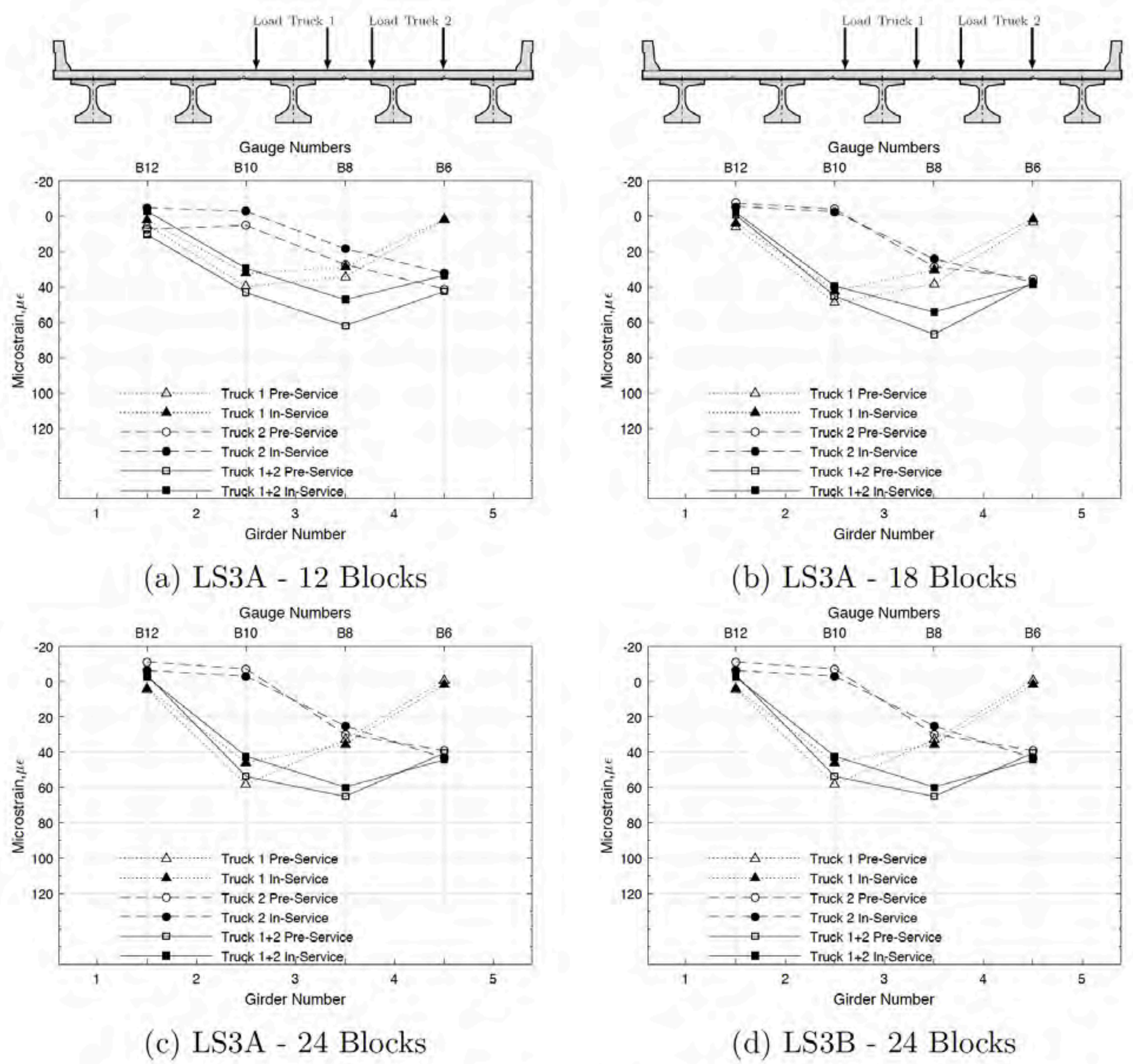


Fig. 5-28. Transverse Strain – Section C – LRWB – LS3A/3B

The response of FIB4 for load position 2 remained within the linear elastic limit (Figure 5-30) for each bridge. The change in deflection over time was less than 10% for all FIB girders in the tested spans, perhaps signifying a small change in stiffness.

Figure 5-31 shows a linear moment - deflection relationship for FIB5 for load position 2. Gauge readings for HCEB (pre-service) were omitted because gauge D11 was faulty. The change in deflection for HCWB, LREB, and LRWB was less than 5%.

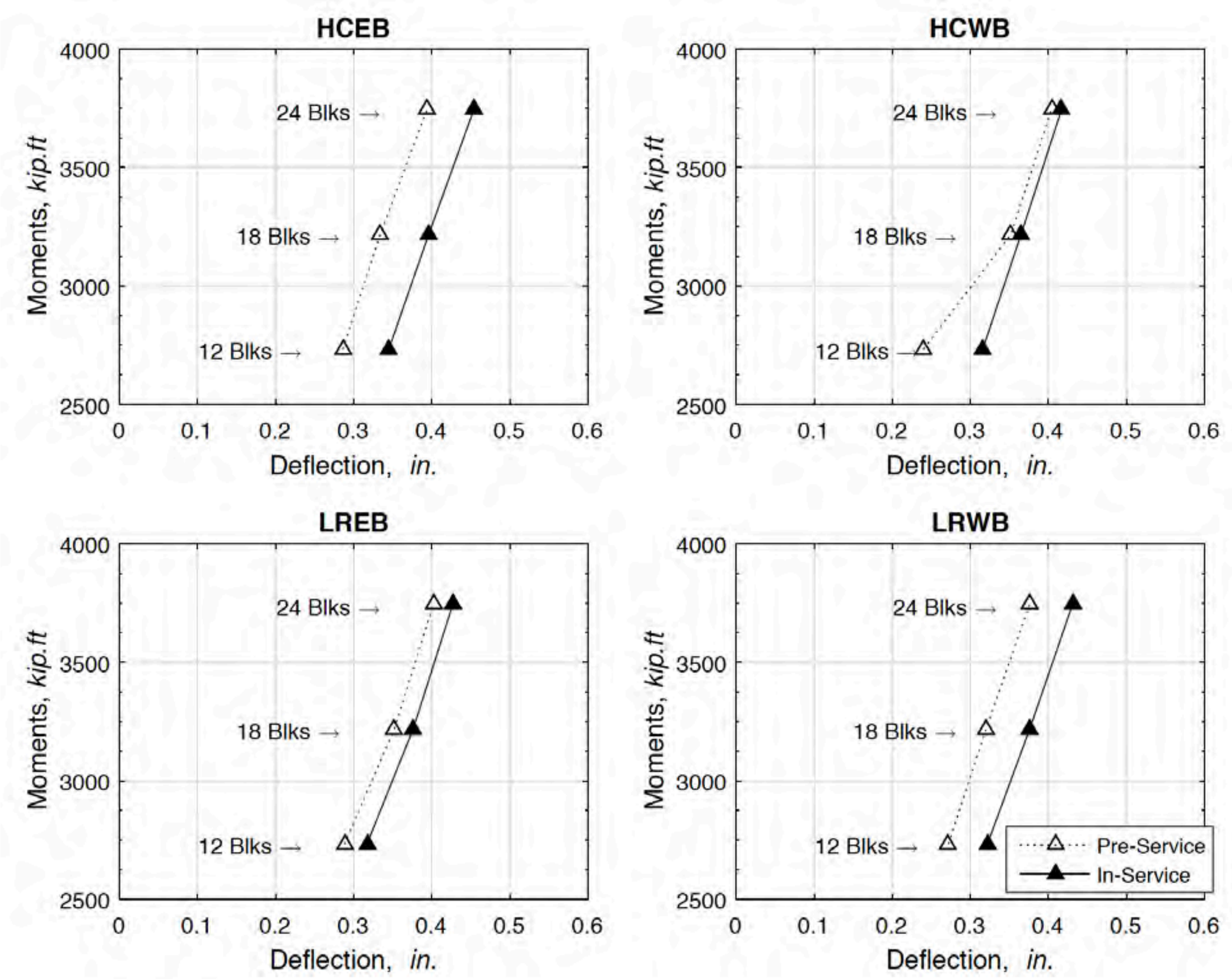


Fig. 5-29. Moment-Deflection Response – FIB3 – LS1/1A – Truck 1+2

5.5 Moment - Strain Response

This section presents the moment vs strain responses of critical girders measured at load positions 1/1A and 2. Strains were measured by gauge pairs at the bottom flange of each girder at section B. Average values from the gauge pairs were then used for the plots. From **Figures 5-32 to 5-34**, moment – strain responses were approximately linear. Figure 5-32 shows the response of FIB3 for load position 1/1A, Figure 5-33 shows the response of FIB4 for load position 2, and Figure 5-34 shows the response of FIB5 for load position 2 for each of the bridges. For HCWB, LREB, and LRWB the responses from the pre-service tests were relatively close to the responses from the in-service tests. However, HCEB shows more variation in the moment – strain response. The 18 block case for HCEB produced more strains than expected on FIB3, FIB4, and FIB5.

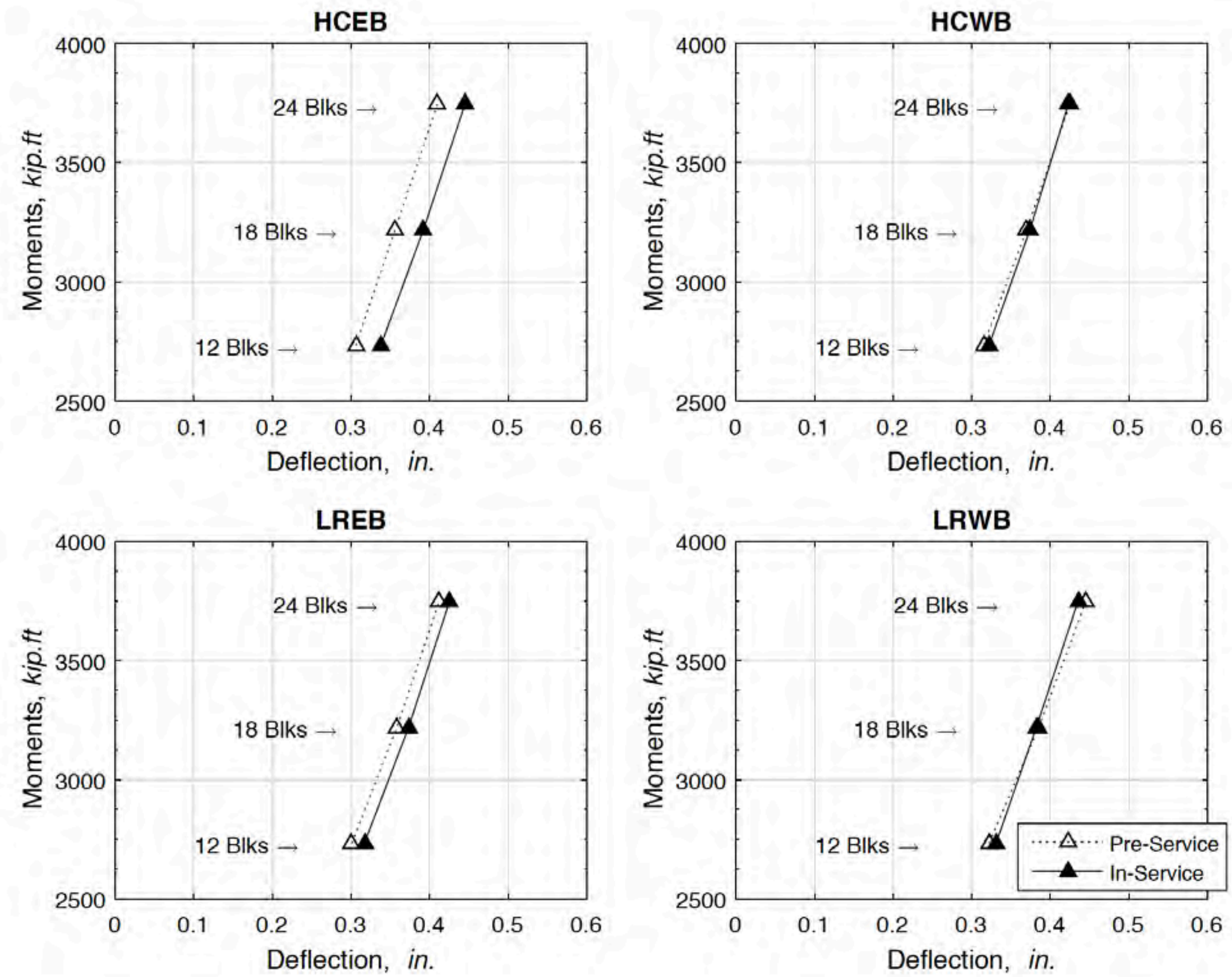


Fig. 5-30. Moment-Deflection Response – FIB4 – LS2 – Truck 1+2

5.6 Closure Joint and Deck Panel Behavior

The goal of load positions 1B and 3B was to equally load the panel (at section B) and the joint (at section C). At these load positions, data was obtained only for the 24 blocks load case. The plots presented in this section use the in-service transverse strains recorded at sections B and C, when both truck 1 and truck 2 were placed at load positions 1B and 3B. Analysis was done for HCWB and LRWB only, because a lot of transverse gauges malfunctioned on HCEB and LREB for the in-service test.

Figures 5-35 and 5-36 compare transverse strains in the panel and joint, for HCWB, for load positions 1B and 3B. The panel and closure joint responses to live load are similar. However, on LRWB, as shown in **Figures 5-37 and 5-38**, higher strains were observed in the panel than in the closure joint. This variation was as much as 29.5% for load position 1B and 22% at load position 3B. Some differences could be due to the precast panel and cast-in-place joint having different material properties – although, one would expect the panels to have lower strains than the joints because of the high strength of the concrete used in the panels. There could have been some slightly inconsistent sharing or transfer of load between the panels and joints, or there could have been a localized effect such that the strains were sensitive to the wheel load locations. The wheel loads were in relatively close proximity to the transverse strain gages, and the wheel loads may not have been exactly placed.

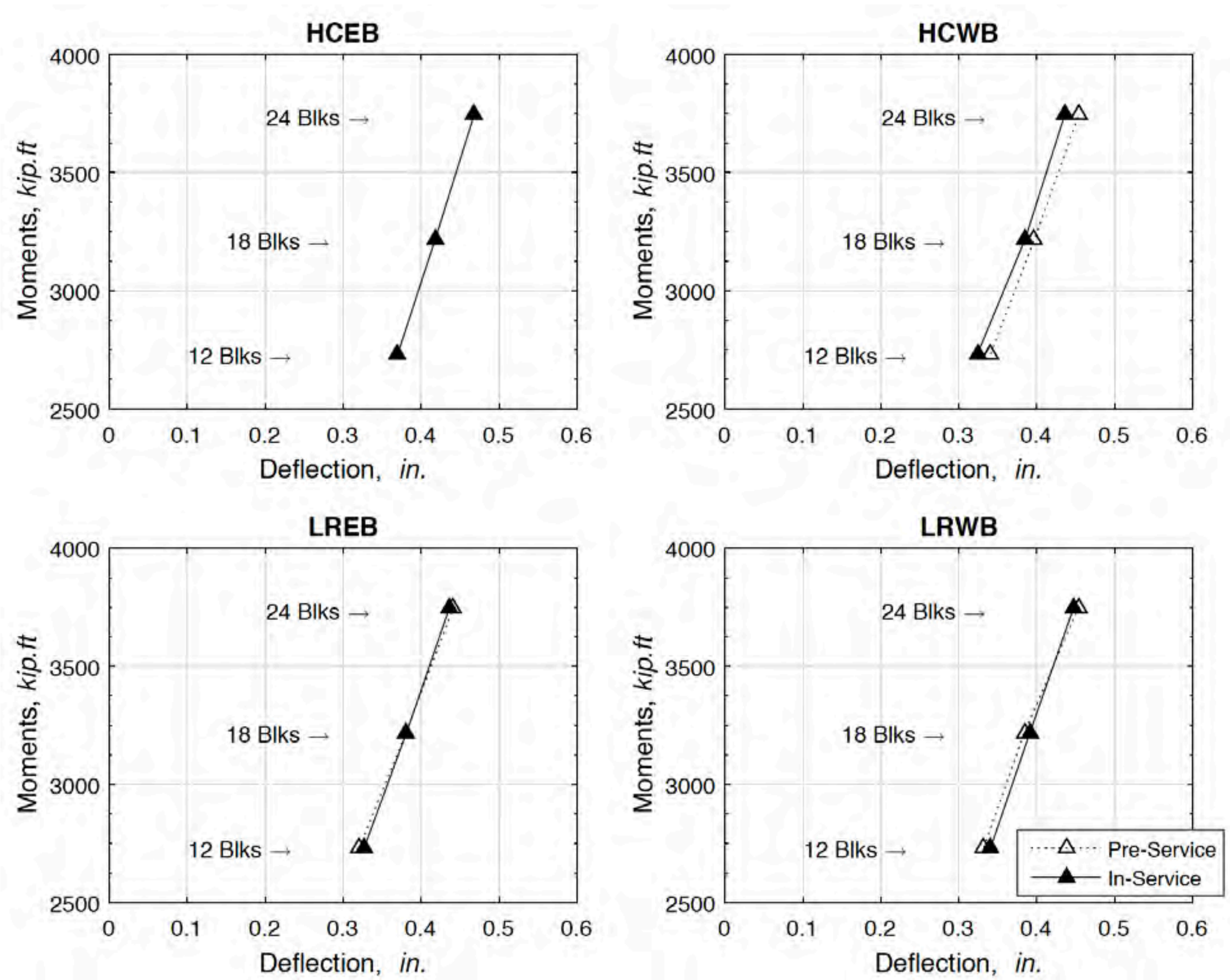


Fig. 5-31. Moment-Deflection Response – FIB5 – LS2 – Truck 1+2

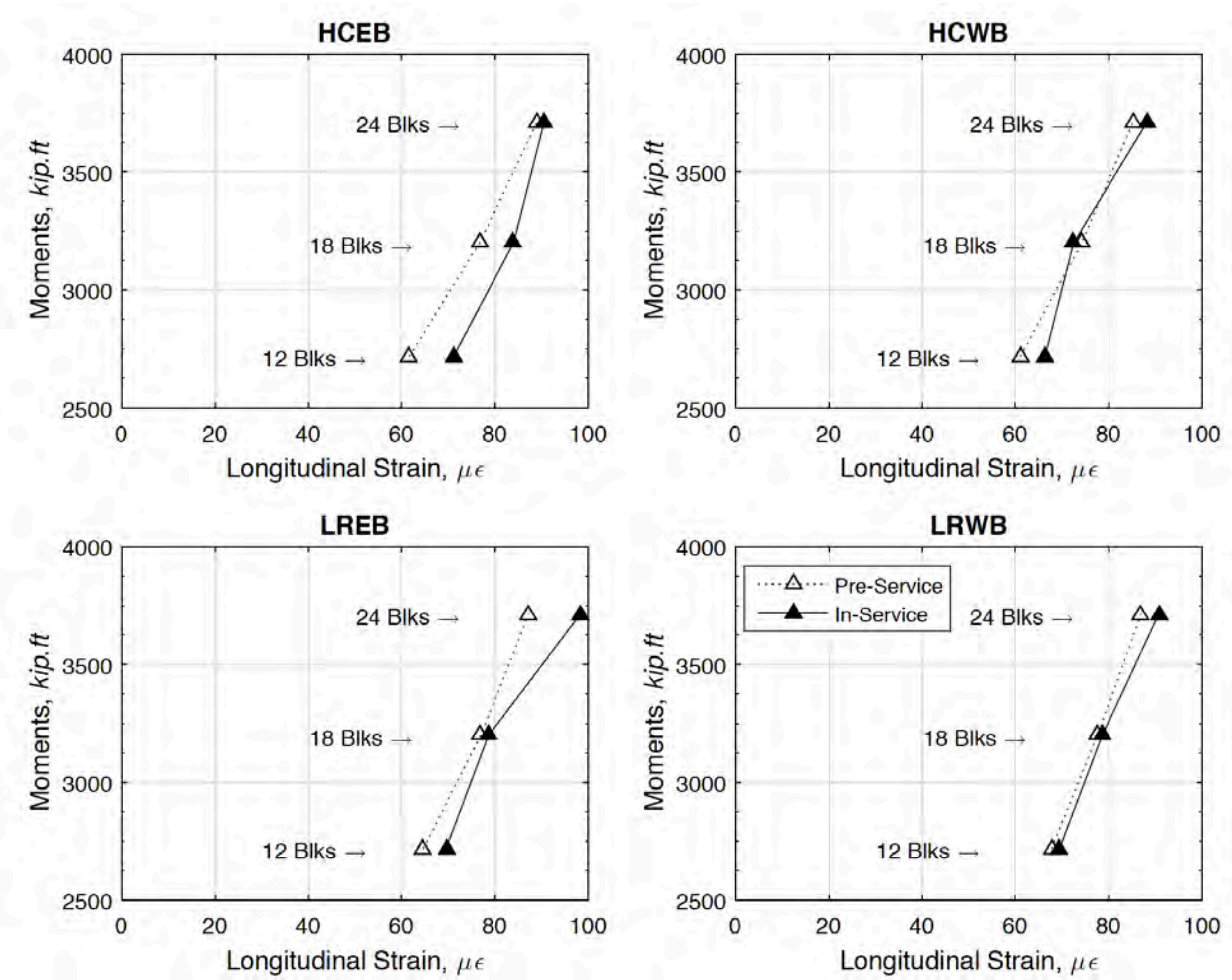


Fig. 5-32. Moment-Strain Response – FIB3 – LS1/1A – Truck 1+2

5.7 Precast Bent Cap Behavior

For the EB bent cap, the strains measured during the second load tests, after the bridges had been in service for a few months, were greater than those measured during the first load tests, before service. Also, the right face experienced greater strains than the left face – although, the difference between the two faces was not as significant as noted previously for the WB bent cap pre-service tests. For example, for Load Position 4, 24 blocks, with trucks 1 and 2 on the bridge, the pre-service EB bent cap bottom strain was 27 με. (average of left and right faces); the in-service strains on the left and right faces were larger: 33 and 100 με, respectively.

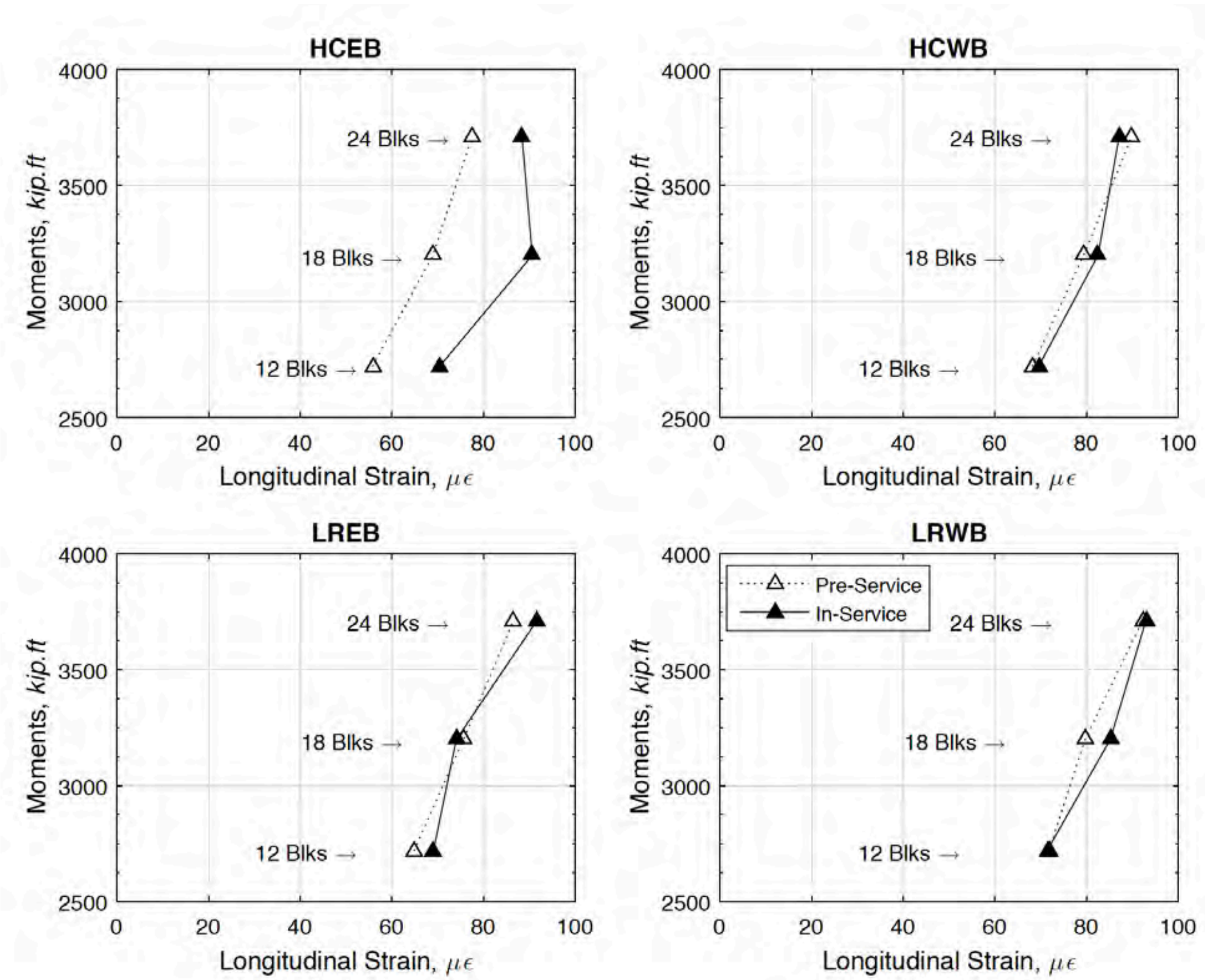


Fig. 5-33. Moment-Strain Response – FIB4 – LS2 – Truck 1+2

As just noted, the EB bent cap strains were larger after being in service than before being in service. However, the opposite was true for the WB bent cap. For example, for Load Position 4, 24 blocks, with trucks 1 and 2 on the bridge, the pre-service EB bent cap bottom strains were 35 and 270 με (average of 153 με) on left and right faces, respectively; the in-service strains on the left and right faces were smaller: 11 and 223 με (average of 117 με), respectively. These strains indicate that the section likely behaved as a cracked section. A rough upper-bound approximation of the theoretical strain would result in around 40 or 50 με, assuming a 20'-6" long simply-supported bent cap – conservatively ignoring any rotational stiffness provided by the columns – gross section properties, and half the weight of the trucks acting on FIB3 girder.

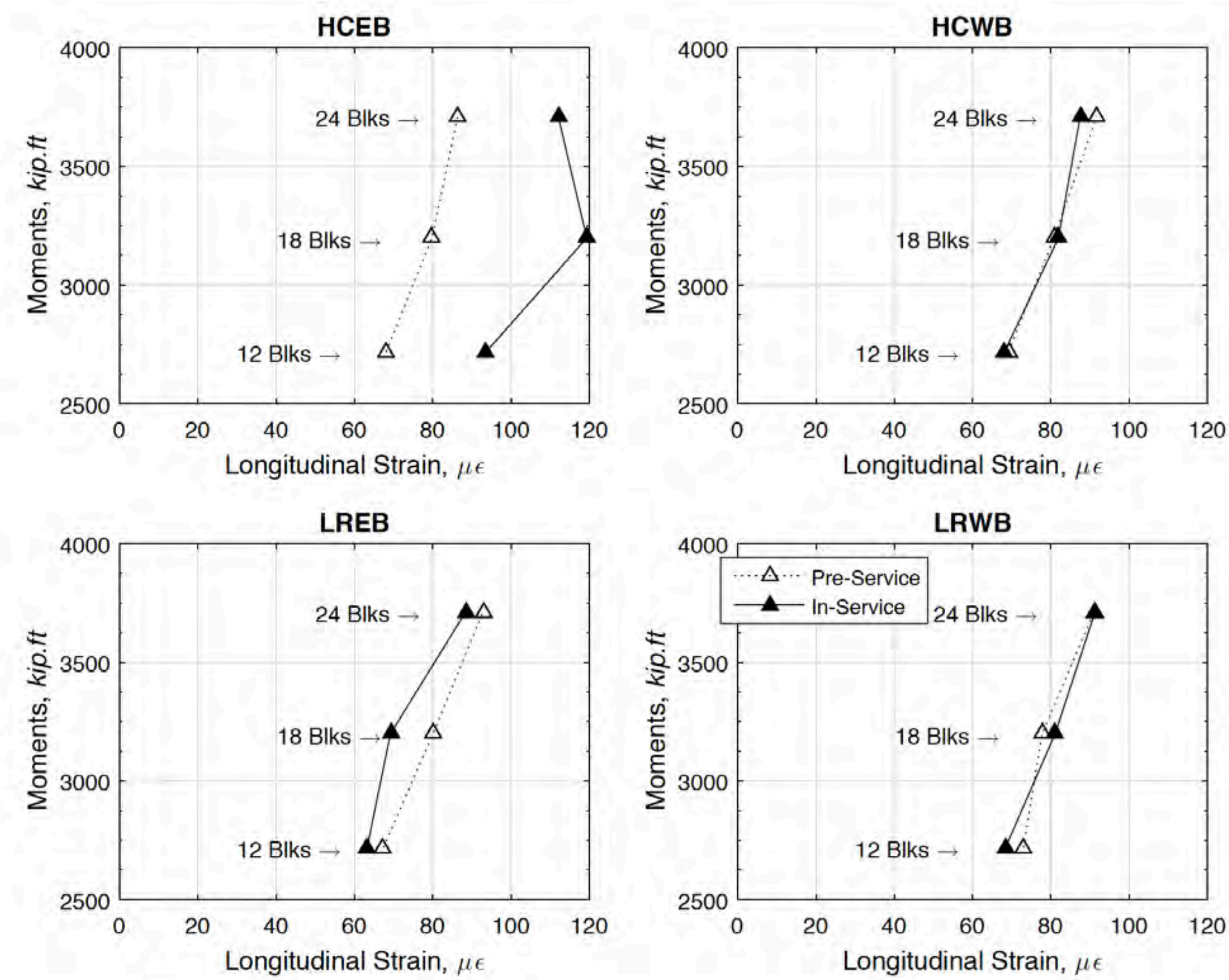


Fig. 5-34. Moment-Strain Response – FIB5 – LS2 – Truck 1+2

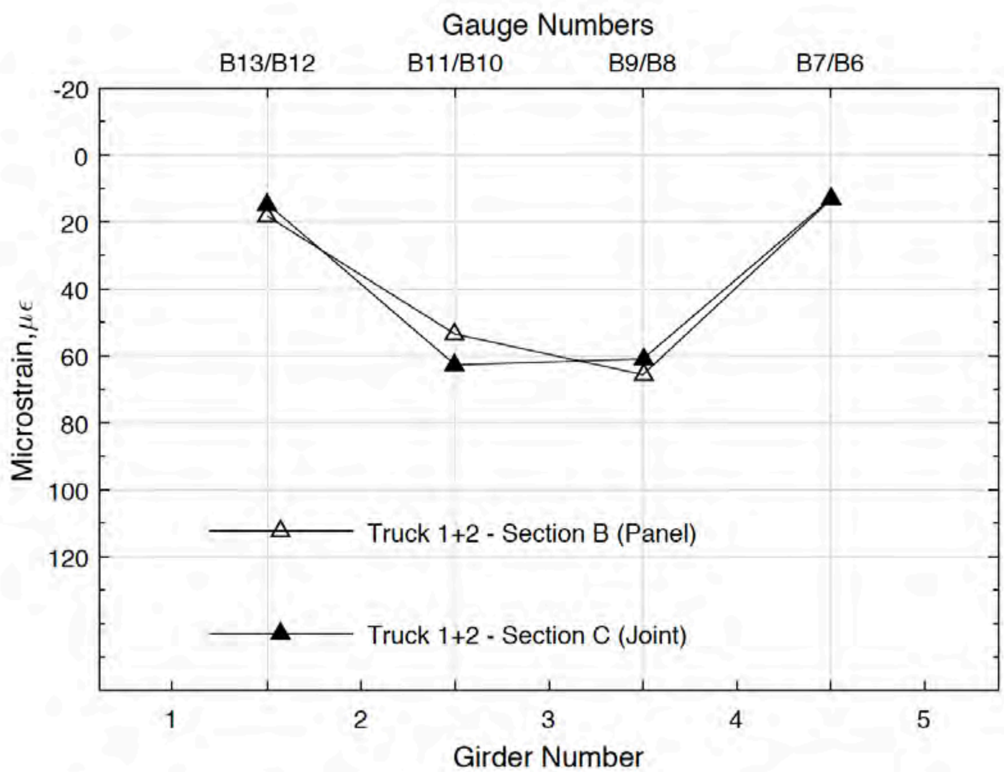


Fig. 5-35. Transverse Strain – HCWB – LS1B – 24 Blocks – Section B/C

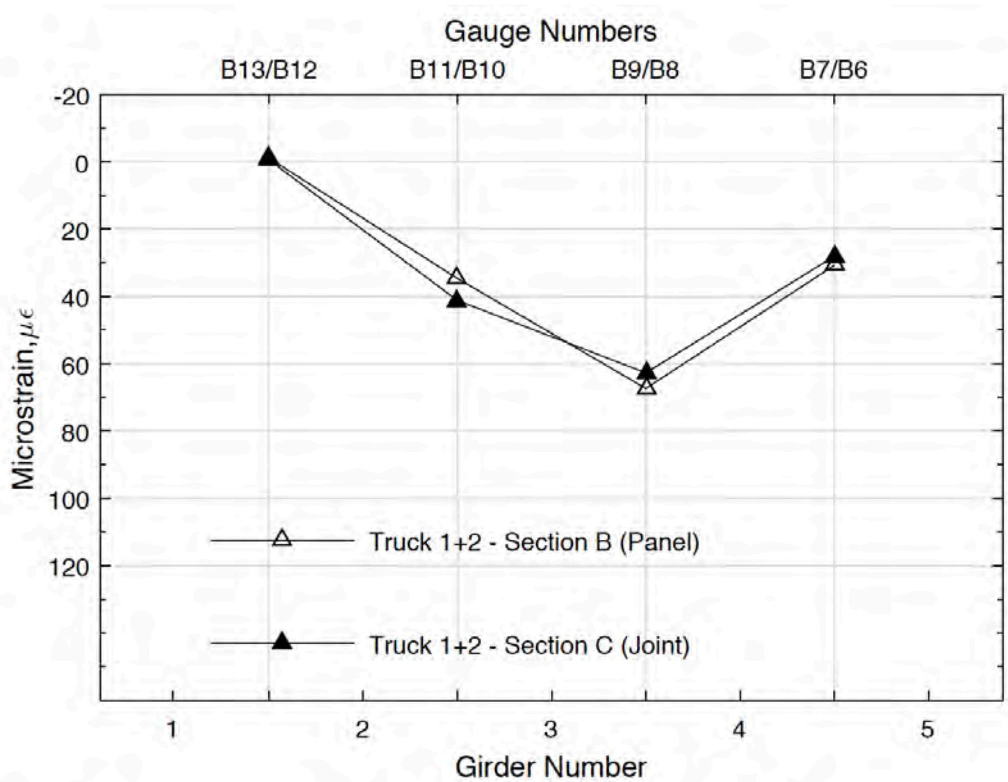


Fig. 5-36. Transverse Strain – HCWB – LS3B – 24 Blocks – Section B/C

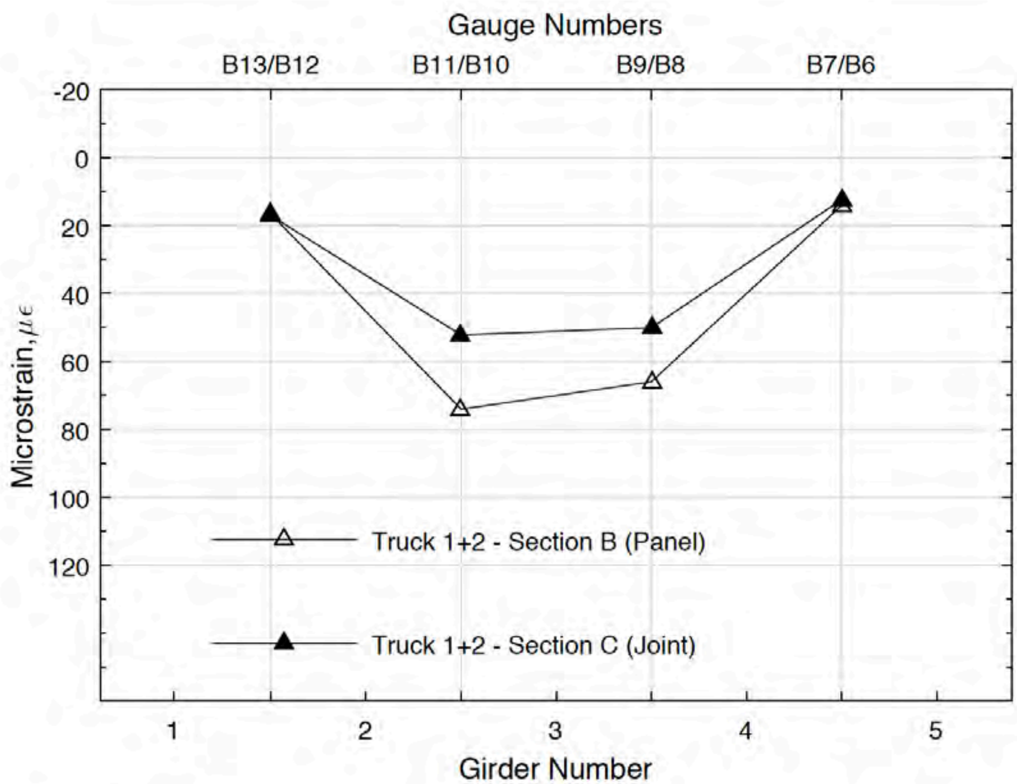


Fig. 5-37. Transverse Strain – LRWB – LS1B – 24 Blocks – Section B/C

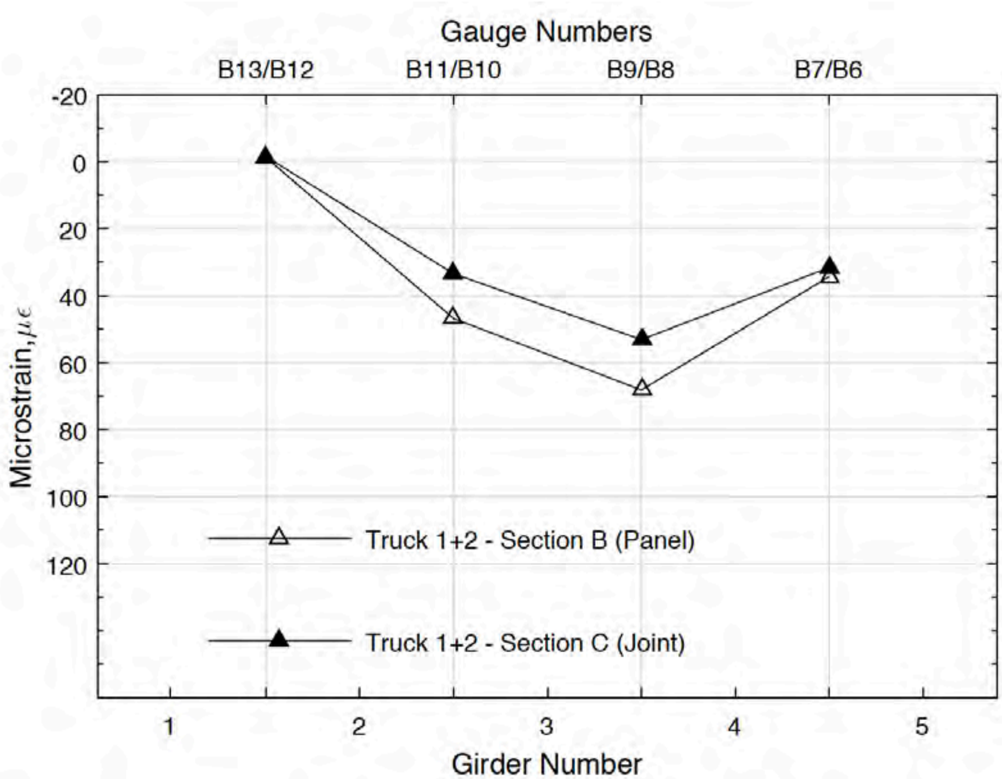


Fig. 5-38. Transverse Strain – LRWB – LS3B – 24 Blocks – Section B/C

CHAPTER 6
CONCLUSIONS

6.1 Discussion

This chapter discusses and summarizes the load test results and analyses performed on four (4) prestressed concrete Florida-I beam (FIB) bridges on U.S. Highway 90 in Gadsden County, Florida.

6.1.1 Discussion of Pre-service Test Results

For the pre-service tests, the distribution factors (DFs) calculated for both bridge types (prestressed panels on FIBs and non-prestressed panels on FIBs) showed very similar results when compared to each other. Also, the DFs were less than those predicted by AASHTO equations, except in the exterior girder. The AASHTO equations were represented very closely by the HCWB test before the barriers were constructed.

Compared to the AASHTO predictions, the DFs calculated from the before barrier test results, when both trucks were loaded, were 8% less for the interior girder and 1.7% greater for the exterior girder.

Using the moments calculated from the test strains, the distribution factors were less than the AASHTO predictions for one truck loaded on the bridge. The distribution factors for load position 2 were 0.345 for FIB4 and 0.651 for FIB5, 26% less for the interior girder and 19% less for the exterior girder, compared to AASHTO DF predictions. On the other hand, when both trucks were placed on the bridge, the DFs were 0.534 for FIB4 and 0.724 for FIB5, 21% less for the interior girder and 8% more the exterior girder, compared to AASHTO DF predictions. Barr et al. (2001) performed an extensive study on several bridges and showed that all DFs were less than the AASHTO predictions, which were up to 28% higher than observed in tests. Chen and Aswad (1996) found from finite element analyses that mid-span moments were 18 to 23% less in interior girders and 4 to 12% less in exterior girders, compared to AASHTO predictions.

Load tests were performed before and after the barriers were constructed on Hurricane Creek westbound bridge. The maximum DFs calculated from measured strains after the barriers were 0.726 for the exterior girder and 0.555 for the interior girder; before the barriers, the DFs were 0.683 for the exterior girder and 0.628 for the interior girder. This is a difference of 6% for the exterior girder and 12% for the interior girder. The DF before the barrier was higher in the interior girder than after the barrier. On the other hand, the longitudinal strain after the barriers were added decreased by 12% in the exterior girder compared to before the barrier. Although the strain was less, the difference in added stiffness with the barrier increased the DF in the exterior girder and decreased in the first interior girder. Eamon and Nowak (2002) showed that there was a 10 to 40% decrease in DF with the addition of secondary elements such as barriers, sidewalks, or diaphragms. The U.S. 90 bridge test percentages were not as high as these values, but Eamon and Nowak (2002) studied a vast array of bridges, including steel bridges.

Next, the cast-in-place transverse joints between the precast panels were analyzed to evaluate whether they behaved similarly to the panels. According to the data, both types of panels (non-prestressed and prestressed) behaved similarly, as did the joints. The higher transverse strains on the joints, as compared to those on the panels, could be attributed to lower modulus of elasticity of the cast-in-place joint concrete. (The compressive strength of the joint concrete was on average 6,000 psi, whereas the concrete panels were above 10,000 psi.) The HCWB and HCEB joint strains were higher than the panel strains, whereas LREB and LRWB joint strains were less than the panel's. The values that were compared were for 24 blocks when trucks 1 and 2 were loaded individually and together at various load positions.

To evaluate whether the deck panels behaved compositely with the girders, the strains on the cross section of the exterior girder, including the barrier cross section, were plotted versus their vertical location. It appeared that composite action was occurring, because the plots were linear, from the top of the barrier to the bottom of the FIB. For most of the load cases, the data trend showed an increase in strain for both compression and tension faces as more blocks were loaded onto the bridge. However, HCEB had a different trend, where the 12-block case resulted in increased strain, but once more blocks were loaded this behavior leveled out. It appeared that the mechanical connection was improved once more load was being transferred through the section.

6.1.2 Discussion of Pre-service vs. In-service Test Results

Generally, the bridges showed the same response to the test load. However, there was some variation between the pre-service test results and the in-service test results. The bridge response (deflection or strain) sometimes decreased but mostly increased somewhat. This could be due to slight changes in boundary conditions at the bearing pads over time, or a loss of stiffness in the members.

Composite action was achieved between the barriers, panel and girders in all bridges. This is evident in the plots of strain vs. bridge (superstructure) depth; the plots were approximately linear. The neutral axis for the composite section measured from the bottom flange of the girders was 31 in. for HCEB, 34 in. for LREB, 36 in. for HCWB, and 34 in. for LRWB.

To evaluate the panel and joint behavior using the in-service test results, transverse strains at load positions that ensured equal loading of both elements were analyzed. The analysis was done for the 24 blocks case. The HCWB joint had similar strains as the panels, whereas the LRWB joint had lower strains than the panels. This comparison could not be made for the eastbound bridges due to irregular transverse strains recorded for HCEB and LREB.

For the bent caps, the data was not consistent from EB to WB, for both the pre-service and in-service tests. For the WB bent cap, pre-service, the left face experienced much higher strains than the right face. This was also true for the in-service tests, but the strains were a little lower. For the EB bent cap, pre-service, the right and left faces had similar strains. For the in-service tests, however, the left face experienced much higher strains than the right face – although not as high as for the WB pre- and in-service tests. Overall, the response of the bent caps was erratic. However, the bottom strains (left and right faces) did increase mostly linearly with increasing load, for the 12-, 18-, and 24- block cases, and when comparing the sum of truck 1 and truck 2 load cases with the truck 1 + 2 load case. It seems that the bent caps were behaving as a cracked section.

6.2 Summary

Four (4) prestressed concrete Florida-I beam (FIB) bridges on U.S. Highway 90 in Gadsden County, Florida, were load tested. The only difference between the bridges was that the panels on the westbound bridges, Hurricane Creek (HCWB) and Little River (LRWB), were prestressed transverse to the bridge span, whereas the panels on the eastbound bridges, Hurricane Creek (HCEB) and Little River (LREB), were reinforced (non-prestressed).

The data was analyzed for the following: the similarities and differences between the two bridge types (non-prestressed and prestressed deck panels); a comparison between the transverse joint and the precast deck panel behavior; the composite action between the barriers, deck panels, and girders; the barriers' effect on the bridge response; and the change in deformational response before the bridges were placed in service (pre-service tests) to after they had been service for about 2 years (in-service tests).

Measured strains were used to calculate moments, so that distribution factors (DFs) could be calculated for each girder on each bridge span tested. These values were compared to the AASHTO DF predictions which were found to be mostly conservative.

The effect that the barrier has on live-load distribution was also evaluated through load testing on one of the bridges, both before and after the barriers were constructed. It appears that the barriers attract load to the exterior girder, but also help carry the load.

The deformational response of the bridges with prestressed deck panels was found to be similar to that of the bridges with non-prestressed deck panels. There appeared to be composite action between the barriers, deck panels, and girders. Based on strain measurements on the top of the barriers, on the slab, and on the girder, the barriers contributed to the overall stiffness of the cross section – particularly the exterior girders.

Comparing the pre-service tests to the in-service tests, the bridge response changed, but not drastically. Overall, the members carried the loads in the same overall pattern – from girder to girder, from joint to panel, etc. Some of the strains and deflections were larger for the in-service tests than for the pre-service tests.

The bent cap response was erratic, and the data was not repeatable from EB cap to WB cap, nor from pre-service to in-service tests.

The assessment of the bridge capacity was beyond the scope of the research.

6.3 Conclusions

Based on an evaluation of the load test data, the following conclusions were drawn:

- The bridge response remained linear with increasing load, throughout the test program.
- Deflections of all four (4) bridges were consistent with each other, with little difference between them, for the pre-service tests.
- Based on moments calculated from measured strains on both types of bridges (prestressed and non-prestressed deck panels), the bridge types behaved very similarly to each other.
- The distribution factors that were calculated from the test data showed that the AASHTO equations for distribution factors are conservative for interior and exterior girders when one lane is loaded.
- The amount of load carried by the exterior girder was increased with the addition of the barrier. The barrier "helped" the first interior girder and "hurt" the exterior girder with regard to the moment carried by each. However, the barriers also added stiffness to the exterior girder – effectively helping carry the load.
- Composite action was achieved between the girders, deck panels, and joints. This was evidenced by linearity in the measured strains throughout the structure depth.

- It appeared that the joints and panels acted as a unit, as evidenced by consistent patterns of strain distributions in the transverse direction. The data trends between the joints and panels were very similar, meaning they behaved in a similar manner and that the truck load was being transferred well between the panels and joints.
- Some of the transverse strain data was erratic. This could be attributed to inexact placement of the trucks. Or, it could be attributed to the fact that some of the non-prestressed panels had experienced cracking before being erected on the bridge, whereas the prestressed panels typically had very little cracking. The cracking occurred mostly in the direction longitudinal to the bridge, over the girders where there were shear pockets cast in the panels. It is possible that cracks in the panels caused anomalies in the load distribution or strain readings. This is worthy of further exploration in future load tests.
- Typically, the cast-in-place transverse joints had higher transverse strain values than the precast deck panels. This could be attributed to the lower concrete strength in the joints, which means the modulus of elasticity of the joints was less than for the panels.
- The response of the bridges from the pre-service tests to the in-service tests was different in some cases. This could be due to changes in stiffness or boundary conditions. The highest strain increase was noticed on HCEB FIB5.
- There is very little difference between the prestressed and non-prestressed panel behavior during loading. To evaluate the long-term behavior, more testing should be done after the bridges have been in service for a few years.
- Based on the load tests, precast panels are a viable option for use in Florida bridges. They can potentially save time during construction, and they perform well when loaded.
- The bent cap strains were erratic. Based on the mismatch between the strains on the right and left faces, the bent cap likely behaved as a cracked section. One face may be cracked more than the other. However, overall the bent caps responded well to load.

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APPENDIX A

Calculated Moments

Table A.1. Calculated Moments at Section B – HCEB

Load Position	Truck	Blocks	Load Step	FIB1 kip-ft	FIB2 kip-ft	FIB3 kip-ft	FIB4 kip-ft	FIB5 kip-ft	Σ M kip-ft
1	1	12	2	278.7	306.7	290.9	107.2	102.8	1086.2
		18	18	329.9	360.2	366.4	173.4	98.7	1328.6
		24	34	384.0	413.1	418.0	209.9	90.5	1515.5
	2	12	4	45.9	135.7	275.7	207.6	318.9	983.8
		18	20	66.6	190.7	366.5	328.1	287.5	1239.5
		24	36	107.8	231.8	442.2	424.0	337.0	1542.8
	1+2	12	3	339.4	463.4	590.5	380.9	362.6	2136.8
		18	19	404.2	553.3	733.8	507.7	359.1	2558.1
		24	35	502.8	654.2	852.2	604.8	418.4	3032.4
2	1	12	7	104.3	189.9	332.9	254.7	175.0	1056.8
		18	23	96.1	204.2	372.7	301.6	246.2	1220.7
		24	39	144.7	272.3	459.6	341.5	221.5	1439.6
	2	12	9	−41.7	13.8	129.1	255.4	629.9	986.4
		18	25	−16.9	43.1	185.3	355.8	707.8	1275.0
		24	41	−25.1	77.3	222.8	427.4	786.8	1489.2
	1+2	12	8	84.2	228.2	486.3	533.7	809.1	2141.5
		18	24	102.4	275.9	573.8	659.7	948.1	2559.9
		24	40	142.0	355.3	682.4	742.1	1026.6	2948.3

Table A.2. Calculated Moments at Section B – LREB

Load Position	Truck	Blocks	Load Step	FIB1 kip-ft	FIB2 kip-ft	FIB3 kip-ft	FIB4 kip-ft	FIB5 kip-ft	Σ M kip-ft
1A	1	12	2	298.0	323.6	301.0	145.9	75.7	1144.3
		18	38	317.0	373.4	356.4	188.9	90.2	1325.8
		24	74	352.9	415.6	413.3	211.9	90.1	1483.8
	2	12	4	82.9	186.9	324.5	323.8	264.2	1182.3
		18	40	69.5	200.7	356.9	354.5	294.0	1275.6
		24	76	91.3	222.8	410.0	429.5	362.0	1515.6
	1+2	12	3	365.0	488.4	603.3	455.2	317.2	2229.1
		18	39	396.7	565.5	718.1	541.0	374.4	2595.7
		24	75	436.9	625.2	813.8	625.3	440.6	2941.8
2	1	12	12	114.2	208.3	339.8	290.9	213.1	1166.3
		18	48	97.2	215.3	369.6	318.0	228.5	1228.6
		24	84	137.7	263.6	436.2	362.4	255.4	1455.2
	2	12	14	6.0	54.4	160.4	336.0	614.7	1171.6
		18	50	−34.9	42.9	171.2	382.8	722.8	1284.8
		24	86	−9.3	68.9	204.2	442.8	849.7	1556.2
	1+2	12	13	96.3	247.9	489.7	604.9	805.5	2244.2
		18	49	95.2	280.3	557.2	705.6	957.2	2595.5
		24	85	123.6	327.6	641.4	807.2	1109.5	3009.3

Table A.3. Calculated Moments at Section B – HCWB – Before Barrier

Load Position	Truck	Blocks	Load Step	FIB1 kip-ft	FIB2 kip-ft	FIB3 kip-ft	FIB4 kip-ft	FIB5 kip-ft	Σ M kip-ft
1A	1	12	2	249.0	343.0	292.6	162.4	45.6	1092.6
		18	13	282.7	400.0	383.6	217.0	68.6	1351.8
		24	24	308.4	436.1	388.8	212.3	65.7	1411.3
	2	12	4	26.4	131.7	269.4	340.7	222.5	990.7
		18	15	89.4	229.4	395.0	453.8	294.3	1462.0
		24	26	96.6	241.0	425.1	485.5	314.9	1563.2
	1+2	12	3	302.6	507.8	603.6	535.0	295.6	2244.6
		18	14	369.5	618.4	761.2	648.9	353.7	2751.9
		24	25	358.7	636.1	777.9	654.0	348.2	2775.0
2	1	12	7	39.8	148.0	295.9	307.5	186.2	977.5
		18	18	128.6	266.8	421.8	381.7	222.7	1421.6
		24	29	84.6	246.1	444.9	404.9	230.9	1411.4
	2	12	9	−61.1	−6.3	114.4	361.2	566.6	974.8
		18	20	−29.8	52.6	195.9	454.9	672.6	1346.1
		24	31	−67.8	8.2	173.2	489.6	738.9	1342.2
	1+2	12	8	21.3	192.5	459.4	697.5	770.1	2140.8
		18	19	61.4	275.6	577.1	797.7	866.2	2578.0
		24	30	54.8	291.5	625.7	884.7	962.7	2819.4

Table A.4. Calculated Moments at Section B – HCWB – After Barrier

Load Position	Truck	Blocks	Load Step	FIB1 kip-ft	FIB2 kip-ft	FIB3 kip-ft	FIB4 kip-ft	FIB5 kip-ft	Σ M kip-ft
1A	1	12	2	265.4	302.4	289.8	191.3	97.8	1146.7
		18	28	333.8	368.4	344.2	188.9	74.3	1309.6
		24	54	371.2	420.0	395.8	213.7	78.5	1479.3
	2	12	4	52.3	144.8	266.3	365.3	312.8	1141.5
		18	30	91.1	190.7	366.5	328.1	287.5	1264.0
		24	56	104.6	226.4	415.8	439.3	351.2	1537.4
	1+2	12	3	313.7	458.9	581.2	520.9	370.4	2245.2
		18	29	418.1	567.6	705.9	569.9	378.3	2639.8
		24	55	474.9	645.5	810.9	655.0	430.8	3017.0
2	1	12	12	91.8	185.4	323.1	290.2	206.4	1096.8
		18	38	115.1	219.4	383.2	348.0	243.4	1309.2
		24	64	141.5	257.6	443.4	393.0	279.5	1515.0
	2	12	14	−28.7	40.5	145.7	354.9	643.5	1155.9
		18	40	−15.7	59.4	185.1	416.8	761.6	1407.2
		24	66	−10.7	67.4	213.7	472.1	863.8	1606.3
	1+2	12	13	72.3	230.6	477.8	648.3	843.2	2272.2
		18	39	107.3	279.8	563.3	755.0	984.2	2689.7
		24	65	133.5	320.5	651.2	854.7	1117.7	3077.6

Table A.5. Calculated Moments at Section B – LRWB

Load Position	Truck	Blocks	Load Step	FIB1 kip-ft	FIB2 kip-ft	FIB3 kip-ft	FIB4 kip-ft	FIB5 kip-ft	Σ M kip-ft
1A	1	12	2	262.4	325.7	292.1	159.1	60.2	1099.5
		18	38	342.0	397.9	318.4	166.9	47.4	1272.6
		24	74	349.1	431.6	350.1	184.2	67.7	1382.7
	2	12	4	74.8	195.7	328.7	345.0	305.9	1250.0
		18	40	85.3	213.2	368.5	391.0	320.9	1378.8
		24	76	77.9	236.7	427.6	454.6	377.5	1574.2
	1+2	12	3	353.9	524.8	619.6	506.5	360.9	2365.8
		18	39	448.8	618.9	709.3	573.2	425.7	2776.0
		24	75	469.6	693.3	795.0	624.5	409.5	2991.9
2	1	12	12	101.9	193.3	316.2	271.9	219.4	1102.6
		18	48	144.6	261.2	408.8	353.5	238.8	1406.8
		24	84	207.5	327.5	485.2	423.2	328.4	1771.8
	2	12	14	11.1	64.5	172.3	373.7	695.8	1317.5
		18	50	−60.8	41.9	183.8	408.5	748.6	1322.0
		24	86	−1.6	68.3	194.7	463.5	908.9	1633.8
	1+2	12	13	122.6	285.1	525.0	656.3	916.2	2505.2
		18	49	81.6	280.3	561.7	728.7	976.0	2628.4
		24	85	124.1	347.1	652.8	843.2	1142.4	3109.6

APPENDIX B

Calculated Load Distribution Factors (Based on Moments)

Table B.1. Load Distribution Factors (Moment) with MPF – HCEB

Load Position	Truck	Blocks	Load Step	FIB1	FIB2	FIB3	FIB4	FIB5	Σ DF
1	1	12	2	0.308	0.339	0.321	0.118	0.114	1.2
		18	18	0.298	0.325	0.331	0.157	0.089	1.2
		24	34	0.304	0.327	0.331	0.166	0.072	1.2
	2	12	4	0.056	0.166	0.336	0.253	0.389	1.2
		18	20	0.064	0.185	0.355	0.318	0.278	1.2
		24	36	0.084	0.180	0.344	0.330	0.262	1.2
	1+2	12	3	0.318	0.434	0.553	0.357	0.339	2.0
		18	19	0.316	0.433	0.574	0.397	0.281	2.0
		24	35	0.332	0.431	0.562	0.399	0.276	2.0
2	1	12	7	0.118	0.216	0.378	0.289	0.199	1.2
		18	23	0.095	0.201	0.366	0.296	0.242	1.2
		24	39	0.121	0.227	0.383	0.285	0.185	1.2
	2	12	9	-0.051	0.017	0.157	0.311	0.766	1.2
		18	25	-0.016	0.041	0.174	0.335	0.666	1.2
		24	41	-0.020	0.062	0.179	0.344	0.634	1.2
	1+2	12	8	0.079	0.213	0.454	0.498	0.756	2.0
		18	24	0.080	0.216	0.448	0.515	0.741	2.0
		24	40	0.096	0.241	0.463	0.503	0.696	2.0

Table B.2. Load Distribution Factors (Moment) with MPF – LREB

Load Position	Truck	Blocks	Load Step	FIB1	FIB2	FIB3	FIB4	FIB5	Σ DF
1A	1	12	2	0.313	0.339	0.316	0.153	0.079	1.2
		18	38	0.287	0.338	0.323	0.171	0.082	1.2
		24	74	0.285	0.336	0.334	0.171	0.073	1.2
	2	12	4	0.084	0.190	0.329	0.329	0.268	1.2
		18	40	0.065	0.189	0.336	0.333	0.277	1.2
		24	76	0.072	0.176	0.325	0.340	0.287	1.2
	1+2	12	3	0.328	0.438	0.541	0.408	0.285	2.0
		18	39	0.306	0.436	0.553	0.417	0.289	2.0
		24	75	0.297	0.425	0.553	0.425	0.300	2.0
2	1	12	12	0.118	0.214	0.350	0.299	0.219	1.2
		18	48	0.095	0.210	0.361	0.311	0.223	1.2
		24	84	0.114	0.217	0.360	0.299	0.211	1.2
	2	12	14	0.006	0.056	0.164	0.344	0.630	1.2
		18	50	-0.033	0.040	0.160	0.358	0.675	1.2
		24	86	-0.007	0.053	0.157	0.341	0.655	1.2
	1+2	12	13	0.086	0.221	0.436	0.539	0.718	2.0
		18	49	0.073	0.216	0.429	0.544	0.738	2.0
		24	85	0.082	0.218	0.426	0.536	0.737	2.0

Table B.3. Load Distribution Factors (Moment) with MPF – HCWB – Before Barrier

Load Position	Truck	Blocks	Load Step	FIB1	FIB2	FIB3	FIB4	FIB5	Σ DF
1A	1	12	2	0.273	0.377	0.321	0.178	0.050	1.2
		18	13	0.251	0.355	0.341	0.193	0.061	1.2
		24	24	0.262	0.371	0.331	0.181	0.056	1.2
	2	12	4	0.032	0.160	0.326	0.413	0.269	1.2
		18	15	0.073	0.188	0.324	0.373	0.242	1.2
		24	26	0.074	0.185	0.326	0.373	0.242	1.2
	1+2	12	3	0.270	0.452	0.538	0.477	0.263	2.0
		18	14	0.269	0.449	0.553	0.472	0.257	2.0
		24	25	0.259	0.458	0.561	0.471	0.251	2.0
2	1	12	7	0.049	0.182	0.363	0.378	0.229	1.2
		18	18	0.109	0.225	0.356	0.322	0.188	1.2
		24	29	0.072	0.209	0.378	0.344	0.196	1.2
	2	12	9	-0.075	-0.008	0.141	0.445	0.697	1.2
		18	20	-0.027	0.047	0.175	0.405	0.600	1.2
		24	31	-0.061	0.007	0.155	0.438	0.661	1.2
	1+2	12	8	0.020	0.180	0.429	0.652	0.719	2.0
		18	19	0.048	0.214	0.448	0.619	0.672	2.0
		24	30	0.039	0.207	0.444	0.628	0.683	2.0

Table B.4. Load Distribution Factors (Moment) with MPF – HCWB – After Barrier

Load Position	Truck	Blocks	Load Step	FIB1	FIB2	FIB3	FIB4	FIB5	Σ DF
1A	1	12	2	0.278	0.316	0.303	0.200	0.102	1.2
		18	18	0.306	0.338	0.315	0.173	0.068	1.2
		24	34	0.301	0.341	0.321	0.173	0.064	1.2
	2	12	4	0.055	0.152	0.280	0.384	0.329	1.2
		18	20	0.081	0.174	0.320	0.346	0.278	1.2
		24	36	0.082	0.177	0.325	0.343	0.274	1.2
	1+2	12	3	0.279	0.409	0.518	0.464	0.330	2.0
		18	19	0.317	0.430	0.535	0.432	0.287	2.0
		24	35	0.315	0.428	0.538	0.434	0.286	2.0
2	1	12	7	0.100	0.203	0.353	0.317	0.226	1.2
		18	23	0.106	0.201	0.351	0.319	0.223	1.2
		24	39	0.112	0.204	0.351	0.311	0.221	1.2
	2	12	9	-0.030	0.042	0.151	0.368	0.668	1.2
		18	25	-0.013	0.051	0.158	0.355	0.649	1.2
		24	41	-0.008	0.050	0.160	0.353	0.645	1.2
	1+2	12	8	0.064	0.203	0.421	0.571	0.742	2.0
		18	24	0.080	0.208	0.419	0.561	0.732	2.0
		24	40	0.087	0.208	0.423	0.555	0.726	2.0

Table B.5. Load Distribution Factors (Moment) with MPF – LRWB

Load Position	Truck	Blocks	Load Step	FIB1	FIB2	FIB3	FIB4	FIB5	Σ DF
1A	1	12	2	0.286	0.355	0.319	0.174	0.066	1.2
		18	38	0.322	0.375	0.300	0.157	0.045	1.2
		24	74	0.303	0.375	0.304	0.160	0.059	1.2
	2	12	4	0.072	0.188	0.316	0.331	0.294	1.2
		18	40	0.074	0.186	0.321	0.340	0.279	1.2
		24	76	0.059	0.180	0.326	0.347	0.288	1.2
	1+2	12	3	0.299	0.444	0.524	0.428	0.305	2.0
		18	39	0.323	0.446	0.511	0.413	0.307	2.0
		24	75	0.314	0.463	0.531	0.417	0.274	2.0
2	1	12	12	0.111	0.210	0.344	0.296	0.239	1.2
		18	48	0.123	0.223	0.349	0.302	0.204	1.2
		24	84	0.141	0.222	0.329	0.287	0.222	1.2
	2	12	14	0.010	0.059	0.157	0.340	0.634	1.2
		18	50	-0.055	0.038	0.167	0.371	0.679	1.2
		24	86	-0.001	0.050	0.143	0.340	0.668	1.2
	1+2	12	13	0.098	0.228	0.419	0.524	0.731	2.0
		18	49	0.062	0.213	0.427	0.555	0.743	2.0
		24	85	0.080	0.223	0.420	0.542	0.735	2.0

APPENDIX C

Calculated Load Distribution Factors (Based on Deflections)

Table C.1. Load Distribution Factors (Deflection) with MPF – LREB

Load Position	Truck	Blocks	FIB1	FIB2	FIB3	FIB4	FIB5	ΣDF_Δ
1A	1	12	0.328	0.305	0.287	0.178	0.102	1.2
		18	0.311	0.301	0.297	0.185	0.105	1.2
		24	0.310	0.302	0.295	0.186	0.107	1.2
	2	12	0.095	0.175	0.296	0.314	0.320	1.2
		18	0.102	0.183	0.298	0.310	0.307	1.2
		24	0.098	0.180	0.289	0.311	0.322	1.2
	1+2	12	0.357	0.401	0.481	0.405	0.356	2.0
		18	0.342	0.404	0.492	0.411	0.351	2.0
		24	0.335	0.398	0.487	0.415	0.364	2.0
2	1	12	0.134	0.203	0.312	0.289	0.262	1.2
		18	0.136	0.209	0.312	0.285	0.258	1.2
		24	0.135	0.215	0.312	0.284	0.255	1.2
	2	12	-0.018	0.058	0.173	0.333	0.654	1.2
		18	-0.007	0.062	0.177	0.324	0.645	1.2
		24	-0.003	0.062	0.173	0.315	0.653	1.2
	1+2	12	0.104	0.224	0.399	0.506	0.768	2.0
		18	0.108	0.226	0.397	0.505	0.764	2.0
		24	0.108	0.225	0.396	0.503	0.769	2.0

Table C.2. Load Distribution Factors (Deflection) with MPF – HCWB – Before Barrier

Load Position	Truck	Blocks	FIB1	FIB2	FIB3	FIB4	FIB5	ΣDF_Δ
1A	1	12	0.292	0.359	0.301	0.180	0.067	1.2
		18	0.272	0.351	0.309	0.191	0.076	1.2
		24	0.282	0.358	0.308	0.182	0.069	1.2
	2	12	0.068	0.190	0.299	0.343	0.301	1.2
		18	0.077	0.198	0.302	0.337	0.286	1.2
		24	0.074	0.202	0.307	0.335	0.282	1.2
	1+2	12	0.295	0.456	0.502	0.438	0.307	2.0
		18	0.282	0.454	0.512	0.444	0.309	2.0
		24	0.293	0.464	0.513	0.435	0.295	2.0
2	1	12	0.095	0.220	0.319	0.316	0.249	1.2
		18	0.109	0.234	0.320	0.306	0.230	1.2
		24	0.104	0.233	0.322	0.309	0.232	1.2
	2	12	-0.051	0.045	0.184	0.396	0.626	1.2
		18	-0.049	0.052	0.190	0.396	0.611	1.2
		24	-0.048	0.046	0.188	0.398	0.616	1.2
	1+2	12	0.043	0.226	0.420	0.593	0.719	2.0
		18	0.056	0.241	0.426	0.586	0.692	2.0
		24	0.055	0.239	0.428	0.587	0.691	2.0

Table C.3. Load Distribution Factors (Deflection) with MPF – HCWB – After Barrier

Load Position	Truck	Blocks	FIB1	FIB2	FIB3	FIB4	FIB5	ΣDF_{Δ}
1A	1	12	0.338	0.293	0.283	0.176	0.110	1.2
		18	0.353	0.301	0.280	0.170	0.096	1.2
		24	0.348	0.306	0.279	0.170	0.097	1.2
	2	12	0.109	0.177	0.273	0.293	0.348	1.2
		18	0.113	0.186	0.282	0.292	0.327	1.2
		24	0.111	0.188	0.284	0.292	0.325	1.2
	1+2	12	0.363	0.393	0.468	0.396	0.380	2.0
		18	0.379	0.406	0.470	0.388	0.357	2.0
		24	0.377	0.404	0.472	0.389	0.357	2.0
2	1	12	0.144	0.198	0.301	0.278	0.279	1.2
		18	0.144	0.203	0.298	0.279	0.276	1.2
		24	0.146	0.205	0.301	0.277	0.272	1.2
	2	12	-0.017	0.058	0.170	0.311	0.678	1.2
		18	-0.013	0.062	0.168	0.309	0.673	1.2
		24	-0.017	0.062	0.168	0.310	0.676	1.2
	1+2	12	0.103	0.214	0.388	0.492	0.803	2.0
		18	0.106	0.219	0.386	0.493	0.795	2.0
		24	0.110	0.221	0.387	0.492	0.790	2.0

Table C.4. Load Distribution Factors (Deflection) with MPF – LRWB

Load Position	Truck	Blocks	FIB1	FIB2	FIB3	FIB4	FIB5	ΣDF_{Δ}
1A	1	12	0.329	0.294	0.272	0.194	0.111	1.2
		18	0.344	0.297	0.273	0.183	0.103	1.2
		24	0.340	0.301	0.277	0.186	0.095	1.2
	2	12	0.116	0.172	0.266	0.308	0.337	1.2
		18	0.106	0.170	0.274	0.311	0.339	1.2
		24	0.105	0.177	0.280	0.313	0.325	1.2
	1+2	12	0.351	0.383	0.451	0.429	0.386	2.0
		18	0.355	0.382	0.452	0.424	0.386	2.0
		24	0.356	0.393	0.460	0.425	0.367	2.0
2	1	12	0.128	0.190	0.284	0.305	0.293	1.2
		18	0.138	0.194	0.291	0.300	0.276	1.2
		24	0.140	0.199	0.288	0.296	0.278	1.2
	2	12	-0.026	0.050	0.154	0.338	0.685	1.2
		18	-0.009	0.065	0.166	0.333	0.645	1.2
		24	-0.004	0.064	0.160	0.326	0.654	1.2
	1+2	12	0.102	0.207	0.364	0.527	0.800	2.0
		18	0.111	0.211	0.370	0.530	0.779	2.0
		24	0.112	0.213	0.367	0.523	0.786	2.0