T.J. Wipf, F.W. Klaiber, B.M. Phares, M.E. Fagen

Investigation of Two Bridge Alternatives for Low Volume Roads - Phase II
Volume 1 of 2

Concept 1: Steel Beam Precast Units

July 2000

Sponsored by the
Iowa Department of Transportation
Highway Division and the
Iowa Highway Research Board

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Iowa DOT Project TR-410

Final

REPORT

IOWA STATE UNIVERSITY
OF SCIENCE AND TECHNOLOGY

Department of Civil and Construction Engineering

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ABSTRACT

This project continues the research sponsored by the Project Development Division of the Iowa DOT and the Iowa Highway Research Board which addressed numerous bridge problems on the Iowa secondary road system. It is a continuation (Phase 2) of Project HR-382 in which two replacement alternatives (Concept 1 - Steel Beam Precast Units and Concept 2 - modification of the Benton County Beam-in-Slab Bridge (BISB)) were investigated).

Work continued on both of the replacement alternatives in this study, the results of which are presented in two volumes. This volume (Volume 1) presents the results of Concept 1 - Steel Beam Precast Units, while the continued work on Concept 2 - Modification of the Beam-in-Slab Bridge is presented in Volume 2.

In previous research (HR-382), a precast unit bridge was developed through laboratory testing. The steel-beam precast unit bridge requires the fabrication of precast double-tee (PCDT) units, each consisting of two steel beams connected by a reinforced concrete deck. The weight of each PCDT unit is minimized by limiting the deck thickness to four inches which permits the units to be constructed off-site and then transported to the bridge site. The number of units required is a function of the width of bridge desired. Once the PCDT units are connected, a cast-in-place (CIP) reinforced concrete deck is cast over the PCDT units and the bridge railing attached. Since the steel beam PCDT unit bridge design is intended primarily for use on low-volume roads, used steel beams can be utilized for a significant cost savings.

This project involved three major tasks during the design/fabrication/construction and testing of the replacement bridge. The first task involved documenting the fabrication of the PCDT units through photographs, slides and a video. As part of this effort, a design methodology was developed that includes the development of standard plan sheets from computer templates. The second task involved transporting the completed units to the bridge site where final construction was completed by an independent contractor. The final task involved the service load testing of the bridge at different stages in the construction process and after completion of the construction. This process was also documented through slides and video.

Based upon the construction and service load testing, the steel-beam precast unit bridge was successfully shown to be a viable low volume road bridge alternative. The construction process utilized standard methods resulting in a simple system that can be completed with a limited staff. Results from the service load tests indicated adequate strength for all legal loads. An inspection of the bridge one year after its' construction revealed no change in the bridge's performance.

1. INTRODUCTION

1.1 Background

A report published in the November 1998 issue of "Better Roads" stated that 35% of all the city, county, and township bridges in the United States are considered structurally deficient or functionally obsolete [1]. A bridge is considered structurally deficient when the deck, superstructure, and/or substructure show signs of serious deterioration. Bridges that are functionally obsolete have roadway widths, vertical clearances, or load capacities that no longer meet current vehicular usage. Of the approximately 25,000 bridges in Iowa, approximately 6,600 are either structurally deficient or functionally obsolete. While some of these bridges can be strengthened and rehabilitated, many simply need to be replaced. However, due to the expense required to repair and update these structures and the fact that available funds are limited, the strengthening and rehabilitation alternative is frequently not the best option.

A recent study, HR-365 "Evaluation of Bridge Replacement Alternatives for the County Bridge System" [2], documented several replacement bridge alternatives currently being used on low volume roads in Iowa and surrounding states. A questionnaire determined that 69 percent of the surveyed counties have the ability and willingness to use their own work forces to design and construct relatively short span bridges (i.e., 40 ft or less) provided the construction procedures are relatively simple. The cost of a replacement bridge can be minimized through the utilization of a county workforce for its design and construction . The 1997 Iowa Code Section 309.40 Advertisement and letting [3] states:

> All contracts for road or bridge construction work and materials for which the engineer's estimate exceeds fifty thousand dollars, except surfacing materials obtained from local pits or quarries, shall be advertised and let at a public letting.

Most county budgets are limited, thus the number of bridges that can be replaced during a fiscal year is limited. The decision to replace a bridge is difficult enough for most counties. It is unfortunate that counties with their own bridge construction crew have to advertise a project within their own county due to the fifty thousand dollar limit.

Based upon the HR-365 study and the need for less expensive methods of design and construction, a "new" bridge replacement concept was developed. In HR-382, "Investigation of Two Bridge Alternatives for Low Volume Roads, Volume 1 of 2, Concept 1: Steel-Beam Precast Units", a new alternative - the steel-beam precast unit bridge, was investigated [4].

The steel-beam precast unit bridge requires the fabrication of precast double-tee (PCDT) units, each consisting of two steel beams connected by a reinforced concrete deck. The weight of each PCDT unit is minimized by limiting the deck thickness to 4 in. This allows the units to be constructed off-site (i.e. at a county maintenance shop) and then transported to the bridge site. The number of units required is a function of the width of desired bridge. Once the PCDT units are installed and connected, a reinforced cast-in-place (CIP) concrete deck is cast over the PCDT units and the bridge railing attached. Since the PCDT bridge is intended primarily for use on low-volume roads, new or used steel beams can be used.

A steel-beam precast unit laboratory model bridge was constructed and tested in the Iowa State University (ISU) Structures Engineering Laboratory. The overall model width was 21 ft with a span length of 32 ft. Specific details regarding the design and construction of this bridge can be found in the final report for HR-382, Investigation of Two Bridge Alternatives for Low Volume Roads, Volume 1 of 2, Concept 1: Steel-Beam Precast Units [4].

1.2 Objective

The objective of this project was to design, fabricate, construct and test a replacement bridge, document all of these tasks, and develop a design methodology. The first task involved documenting the fabrication of four precast steel-beam units at an off-site location. The bridge constructed from these units was based upon standard plans developed as part of this project and was designed based upon a methodology also developed as part of this task. The second task consisted of transporting the completed units to the bridge site where construction of the steel-beam precast unit bridge was completed by an independent contractor. The final task involved service load testing the bridge at different phases of construction and after completion.

1.3 Scope

Upon completion of the design and fabrication process, one PCDT unit was instrumented and tested during transportation to the bridge site. Service load tests were performed at several phases during and after the field construction process. The PCDT units were load tested with and without the CIP deck in-place. A third service load test was performed after the bridge/approach rails were in-place. The construction process was documented through photographs, slides and video. The design process involved the development of computer software that includes the design of the bridge and automatically generated construction drawings. All of these aids constitute an informal design/construction procedure developed to assist counties in designing and constructing this system using their own manpower.

1.4 Tasks

1.4.1 Literature Search

A review of literature on several precast bridge systems for low volume roads was conducted. The design performance and constructability of these bridge alternatives was also reviewed.

1.4.2 Design/Plan/Fabricate/Assemble the PCDT Units

After consideration of several factors, Black Hawk County, Iowa was selected as the location for constructing the field demonstration bridge. Based upon conditions at the selected site, a set of working drawings were prepared by the research team. The drawings were made from standard plan templates developed as part of this project. A design methodology also developed in this study was used to design the bridge superstructure. Quantity estimates and a project schedule were generated using these drawings. Fabrication and assembly of the four PCDT units were completed at the Black Hawk County maintenance facility.

1.4.3 Instrumentation and Transportation of PCDT Unit

Once the fabrication process was completed, the four PCDT units were transported to the bridge site. Construction loads were monitored by measuring and recording strains at critical locations on one of the PCDT units during the loading and transportation to the bridge site.

1.4.4 Instrumentation and Testing under Service Loads

The PCDT unit demonstration bridge was instrumented to measure strains and deflections at critical locations. Steel and concrete gages were used to measure strains and Celesco displacement transducers were used to measure vertical deflections. Service load tests were conducted prior to and after the placement of the CIP portion of the deck. The bridge/approach rails were not in-place for either of these tests. Once they were in-place, a third service load test was performed; results of these three tests are presented in this report.

1.4.5 Analysis of Testing Performed

The data from the lifting and transport tests as well as the service load tests are contained within this report. The data reduction was performed using computer spreadsheets and graphing software. Bridge behavior analyses were based upon the strain and deflection graphs generated using data from the field tests.

1.4.6 Summary and Conclusion

Based upon the design, fabrication and construction of the PCDT demonstration bridge, several design and constructability issues are presented in this report. Overall bridge behavior is presented in the graphs of the field test data and the theoretical data.

1.5 Methodology

The methods used in the research project are briefly described below.

The literature search was performed using the Structural Information Service in the Bridge Engineering Center, the Transportation Information Service at the Iowa Department of Transportation (DOT) and various computerized literature searches in the ISU Parks Library.

The field portion of this study followed traditional construction planning steps as follows:

Project Planning Phase:

- site selection
- design based upon site selected
- plans generated
- quantities estimated
- contract negotiations
- project planning (schedule, material bid analysis)

Fabrication:

- procurement of materials
- construction of PCDT units off-site

Instrumentation for Transport Test:

- installation of strain gages at critical locations on one PCDT unit
- testing during loading and transportation of the instrumented PCDT unit

Connection of PCDT Units:

- installation and connection of the PCDT units on-site
- placement of CIP reinforced concrete deck

Service Load Testing (without bridge rails):

- placement of strain gages at critical locations on all PCDT units
- service load test on connected PCDT units (CIP deck not in-place)
- service load test with CIP deck in-place (no bridge/approach rails)

Barrier Rails:

- bridge/approach rail placement
- service load test with bridge/approach rails in-place

<u>Data Analysis:</u>

The recorded data from loading and transporting one of the units, and the service load tests were reduced using Microsoft Excel (Version 7); Microcal Origin (Version 4.1) was used to generate the graphs. The bridge behavior was determined from design calculations, finite element analyses, the experimental data and the observed behavior.

1.6 Report Summary

The results of the design, fabrication, construction, and testing the PCDT unit demonstration bridge are summarized in this investigation. The literature review is presented in Chp. 2. Information on the design, construction, transportation, and assembly of the PCDT units are presented in Chp. 3 with the testing program performed in Chp. 4. The structural behavior results are presented in Chp. 5. The summary and conclusions of the project are presented in Chp. 6.

The Black Hawk County office was extremely satisfied with the experienced associated with the replacement bridge process. They submitted the overall project process to the Iowa Quality Initiative Structures Award Program within the Iowa DOT. To support their nomination, they discussed the ease with which the PCDT units were fabricated and with which the superstructure was constructed. They noted that the precast units were constructed to very close tolerances. The total construction time (including the placement of the guardrails and bridge rails) of the superstructure was 14 days.

Figure 1.1 shows the original replaced bridge as well as the replacement PCDT unit bridge. The original bridge was a 3 span I-beam bridge with overall dimensions of 58 ft in length and 20 ft in width. The replacement bridge was a single span bridge 64 ft long and 30 ft wide.

a. End view of original bridge

b. End view of replacement bridge during service load testing

Figure 1.1. Photographs of original and replacement bridges.

c. Side view of original span bridge

d. Side view of single span replacement bridge

Figure 1.1. Continued.

2. LITERATURE REVIEW

A literature search was performed to collect information on other prefabricated bridge systems which are used on low volume roads. Several methods were utilized in the literature search. The Structural Information Service in the ISU Bridge Engineering Center was searched first. The Transportation Research Information Service at the Iowa DOT was also searched for pertinent information. Several other computerized searches were conducted through the ISU Parks Library.

The literature reviewed in this report is not intended to be all inclusive on the topic of low volume road bridge alternatives. The literature focuses only on prefabricated systems used in short to medium span applications for low volume roads which are relevant to this project.

2.1 Classification of Bridges

In bridge engineering, it is common practice to classify bridges as short-span, medium-span, or long-span. Currently, no established criteria define the span ranges for these three classifications. In the absence of any established criteria, a common practice is to classify bridges by span lengths as follows:

Short-span bridges:	20 to 125 ft
Medium-span bridges:	125 to 400 ft
Long-span bridges:	Over 400 ft

Bridges with spans of 20 ft and under are classified as culverts [5]. This general classification of bridges automatically places certain types of bridges into each category. For example, precast concrete slabs and prestressed concrete beams would both be considered short-span bridges.

2.2 Constructability of Existing Prefabricated Bridge Systems

The term constructability, as used in this investigation, refers to the application of a disciplined and systematic optimization of the construction related aspects of the project during all phases of the project life. Consideration of constructability issues may reduce the project cost and schedule or improve the functionality of the finished product. However, the purpose of reviewing constructability is not to lower construction costs or in anyway modify the overall objectives of the project, nor is it an attempt to dictate a design that is the easiest to build. The purpose is to ensure that the impact of the design and construction details are recognized and taken into consideration during all phases of the project.

Recent trends in new bridge construction have shown a definite trend away from labor-intensive and time consuming field operations [6]. The use of prefabricated bridge elements works well on low-volume roads. The mass production of elements results in a more effective and efficient use of materials. In addition, the prefabrication process results in time and cost savings over more conventional procedures.

2.2.1 Slab Bridges

Reports indicate that more than 95 percent of all the bridges constructed within the United States during 1950-1990 are considered to be short-span bridges [7]. Many of these short-span bridges must traverse waterways or railways where the vertical clearance is limited. Many designers select the cast-in-place slab for use in these areas with critical clearance requirements. The slab bridge provides the largest possible span-to-depth ratio and does not require large equipment or complicated construction procedures [8]. However, constructability issues with the cast-in-place slab system limit its effective use.

While the cast-in-place slab system does not require large equipment and is relatively easy to construct, the slabs "have become too expensive and time consuming due to the extensive field formwork" [9]. In addition, the span length is limited to approximately 50 ft unless there are intermediate piers. The end result is slow field construction and very high labor costs.

2.2.2 Precast and Prestressed Concrete Low Volume Road Bridge Alternatives

The majority of prefabricated low volume bridge superstructures are composed of precast, prestressed concrete. "Precast, prestressed concrete is quite applicable as a construction material in low-volume bridges because of its ability to be prefabricated and its economic competitiveness in many regions of the county" [6].

GangaRao and Zelina [6] described the most common precast concrete bridge replacement alternatives for use on low volume roads. The various precast concrete alternatives are listed below:

- Precast concrete slabs
- Precast deck panels
- Precast I-beams
- Precast T-beams
- Precast box-beams

"The precast, prestressed concrete modules are an efficient and economical alternative that can be used to meet the heavy demand for new bridge construction".

2.2.2.1 Precast Concrete Slab

There are two general types of precast concrete slab systems, the solid and hollow core systems; both have been used in short-span bridge applications. The solid core system has the ability to span approximately 15-30 ft and the hollow core system can be used for spans of 25-50 feet. An advantage of using the precast slab systems is the rapid placement. Extensive research on the connection issues of the slab systems was performed by Martin and Osborn in 1983 [10]. This alternative is limited primarily by span length.

2.2.2.2 Precast Concrete Deck Panels

The use of concrete deck panels provides an easy and cost-effective method of constructing bridge decks for bridges. Typically, the panels are cast off-site at a precast, prestressed plant. "The modules are economically mass-produced, since the same formwork can be used repeatedly and quality control is improved with plant conditions" [11]. The units are trucked to the site and lifted into place and positioned on either concrete or steel beams. The panels span adjacent beams and act as the forms during placement of a cast-in-place concrete deck required for a full depth of deck. This eliminates the construction costs associated with installation and removal of formwork.

2.2.2.3 Precast I-Beams

Typical span lengths for precast, prestressed I-beams range from 40-140 feet. "These girders have stirrups projecting from their top flanges, to be embedded in the cast-in-place deck to develop composite action" [5]. The use of deck panels (see Sect 2.2.2.2) or stay-in-place metal forms are used during deck forming operations to reduce construction time and to improve safety during the placement of the deck.

2.2.2.4 Precast T-Beams

Various forms of precast, prestressed T-beams have been used in short span bridge applications. The single-T, double-T, and multiple-T sections can be used for span ranges between 20-80 feet. "The flanges of the T-beams may be cast as a full-thickness integral deck with the beams, or as the lower half of the deck to provide the formwork for a cast-in-place deck" [6].

The use of prestressed double-T beams in conjunction with the elimination of the cast-in-place portion of the deck was investigated by Shahawy [12]. By eliminating the cast-in-place portion, overall bridge construction time can be reduced with an associated reduction in labor costs. In the study performed by Shahawy, the precast double-T beams were tied together through V-joint edge flanges, which were filled with non-shrink grout, and then post-tensioned in the lateral direction to provide lateral resistance and load transfer.

The University of Nebraska-Lincoln developed a modification to the precast, prestressed T-beam. Analytical and experimental testing has shown that the Nebraska Inverted Tee (IT) system can span up to 85 feet with a structural depth of only 29 in. [9]. According to Kamel and Derrick [9], no other existing precast concrete or cast-in-place conventionally reinforced system has this capability. The IT system provides members which are relatively lightweight which aids in the handling and placement operations. In addition, the system requires no field formwork.

2.2.3 Steel Low Volume Road Bridge Alternatives

Typical steel beam short span bridge systems are composed of equally spaced rolled sections supporting a cast-in-place concrete deck. These structures can be designed using either composite or non-composite techniques. Non-composite structures are used primarily for spans less than 50 feet, while composite structures are used for spans over 50 feet.

Based on the literature search, there were no documented uses of steel-beam bridges with precast decks in short, medium, or long span applications.

2.3 Steel-Beam Precast Double-Tee Unit Bridge (PCDT)

A steel-beam precast (PC) experimental bridge, 32 ft in length, was constructed in the ISU Structures Engineering Laboratory (Iowa DOT Project HR-382 [4].) The bridge had three PC units which were 7 ft in width resulting in an overall bridge width of 21 ft. Each PC unit had a 4 in. thick reinforced concrete deck connected to two salvaged W21 x 62 steel beams which are on 3.5 ft centers. An overview of the model bridge is shown in Fig. 2.1. The reinforced concrete deck thickness limited the weight of each PC unit while at the same time provided sufficient strength so that the units could be moved without damaging them.

The steel-beam PC unit bridge system relies on composite action between the PC reinforced concrete deck and the steel beams as well as between the PC and the CIP deck [4]. Composite action between the PC reinforced concrete deck and the steel beams was accomplished with S3L 3/4 in. x 4 in. welded shear studs. The PC reinforced concrete deck was intentionally roughened ("grooves" placed in the wet PC concrete at 1 in. intervals to a

Figure 2.1. Photograph of laboratory PCDT unit bridge [4].

depth of approximately 1/4 in.) in the transverse direction to provide a shear transfer mechanism between the two layers of reinforced concrete.

To facilitate lifting the PC units, high strength steel threaded rods were attached to the top flanges of each steel beam. These were placed prior to placing the PC concrete and were of sufficient length so that they extended past the top surface of the PC concrete so that lifting brackets (four for each units) could be attached.

Based on the laboratory PC demonstration bridge testing program, Wipf et. al presented 11 conclusions [4]. Those pertinent to this part of the investigation follow:

1. The PCDT units result in a simple span bridge alternative for low volume roads that is relatively easy to construct.

2. The PC connector that was developed to connect adjacent PDCT units is relatively easy to fabricate and install.
3. The PCDT units are strong enough to resist the handling loads imposed on them during fabrication and transportation. When the PC concrete is given adequate time to cure, rough handling of the units during lifting, transporting, or placing should cause no distress.
4. The testing performed resulted in no interlayer delamination between the PC and CIP concrete.
5. The reinforced CIP deck improved the load distribution characteristics of the PCDT bridge.

The analytical and laboratory investigation performed by Wipf et. al [4] was the basis for the construction of the PCDT demonstration bridge presented in this report. Other systems described in the literature have all been field tested, and in many cases adopted into current practice.

Five major advantages of the PCDT system are as follows:

- System can be used in simply supported spans up to 85 ft
- Minimal field formwork is required
- Salvaged steel beams may be used
- Standard construction methods are used
- Construction can be performed with local agency bridge construction crews

3. STEEL-BEAM PRECAST DOUBLE-TEE (PCDT) UNITS

3.1 Overview

To assist the research team with this investigation, the Project Advisory Committee (PAC) organized in the previous related research was retained. Listed below are the members of the PAC:

Dennis Edgar - Fayette County (formerly Black Hawk County)
Bob L. Gumbert - Tama County
Wallace C. Mook - City of Bettendorf
Mark Nahra - Delaware County
Gerald D. Petermeier - Benton County
Jim Witt - Cerro Gordo and Winnebago Counties

With the aid of the PAC, the research team located a site for construction of the field demonstration bridge. Black Hawk County, IA was selected based upon their interest in participating in the project, availability of a bridge site, and an available, experienced bridge construction crew.

3.2 Design of PCDT Bridge

Based on previous work on the development of the PCDT bridge [4], a design methodology was developed. The methodology utilizes the Allowable Stress Design (ASD) procedure within the 1992 American Association of State and Highway Transportation Officials (AASHTO) Standard Specification [13]. The design procedure includes computer software to perform the design. However, design tables were also developed to be used in lieu of the computer software. Documentation of the design methodology is presented in Ref. 14.

Another design and construction aid was developed. This aid consists of a set of standard drawings that provide all necessary construction details of the PCDT bridge units and the complete bridge. The drawings are generated automatically with computer aided drafting (CAD) software that is provided. The user needs to input design and geometry information that is provided by either the design software previously mentioned, or by use of the design tables provided. The software calculates the design forces for the bridge girders and bridge deck using analysis assumptions based on the AASHTO design specifications. The design includes the selection of the girder and the bridge deck reinforcement. The shear connectors for composite action are also designed. In addition, the number of connectors required between adjacent PCDT units is determined. The design methodology is valid for bridges between 24 ft and 30 ft in width and for a range of single spans from 30 ft to 80 ft. The yield strength of the beams can be either 36 ksi or 50 ksi, the concrete deck reinforcement 60 ksi, and the compressive strength of the concrete in the deck must be at least 3.5 ksi. A detailed example of the design procedure is presented in Ref. 14. An example of the design methodology shown in Appendix A. It should be emphasized that the methodology shown in the appendix is a complete set of plans and design aids for the PCDT bridge. Together, they provide the bridge design and the construction drawings for use by bridge engineers.

The bridge replacement configuration presented in this report uses prefabricated precast double-tee (PCDT) units composed of two steel beams and a reinforced concrete composite deck. The design and associated plans were developed using the design and construction aids mentioned above. As shown in Fig. 3.1, each PCDT unit used for this

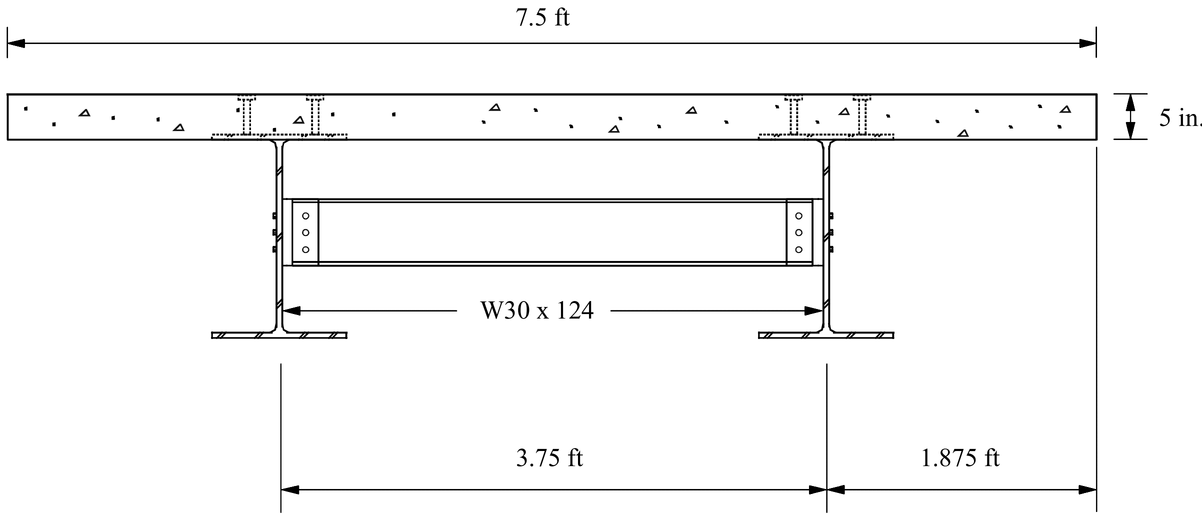


Figure 3.1. Cross section dimensions of PCDT units used in field demonstration bridge.

project had a center to center beam spacing of 3.75 ft, an overall width of 7.5 ft, and a 5 in. thick reinforced concrete composite deck. The design width of the demonstration bridge was 30 ft; thus, four PCDT units were required. Based upon the site selected for construction, the span length from center to center of abutment was 64 ft. The PCDT units were fabricated by the research team and the county bridge crew at the Black Hawk County Maintenance Facility in Waterloo, IA.

The steel-beam PCDT unit bridge was a replacement structure as the existing structure at the site selected needed to be replaced. Design and construction of the replacement substructure was completed by Black Hawk County.

Upon completion of the substructure and fabrication of the PCDT units, the PCDT units were transported to the site. Once the units were positioned on the abutments and connected, a CIP reinforced concrete deck was constructed over the PCDT units to provide the required full depth of concrete. Bridge and approach rails were installed after the CIP reinforced concrete deck had sufficiently cured.

Based upon the Iowa Code Section 309.40 Advertisement and letting, Black Hawk County advertised some of the project tasks for public bids. The project tasks that were contracted were the transportation, placement and connection of the PCDT units, placement of the CIP reinforced concrete deck and the bridge/approach rails. The low bid for these tasks was submitted by Cramer and Associates, Inc. located in Des Moines, Iowa.

3.3 Fabrication of Precast Double-Tee Units (PCDT)

3.3.1 Material Procurement

Based on the site selected for the construction of the steel-beam PCDT unit bridge, a set of working drawings were prepared (see Appendix B). Quantity estimates for the superstructure (excluding the structural concrete) were prepared from the working drawings. The quantities were calculated according to five major divisions: girders, PCDT connectors, diaphragms, PCDT reinforcement, and CIP reinforcement. A summary of the quantity estimate totals are shown in Table 3.1. Based on these quantities, several suppliers were contacted for price quotations. After reviewing all prices received, the superstructure materials (excluding the structural concrete) were purchased from Oden Enterprises, Inc., located in Wahoo, NE.

3.3.2 PCDT Connectors

The structural strength of the steel-beam PCDT unit bridge relies on the transfer of forces from unit to unit. The load transfer was accomplished by two mechanisms. First, the

Table 3.1. Summary of quantities.

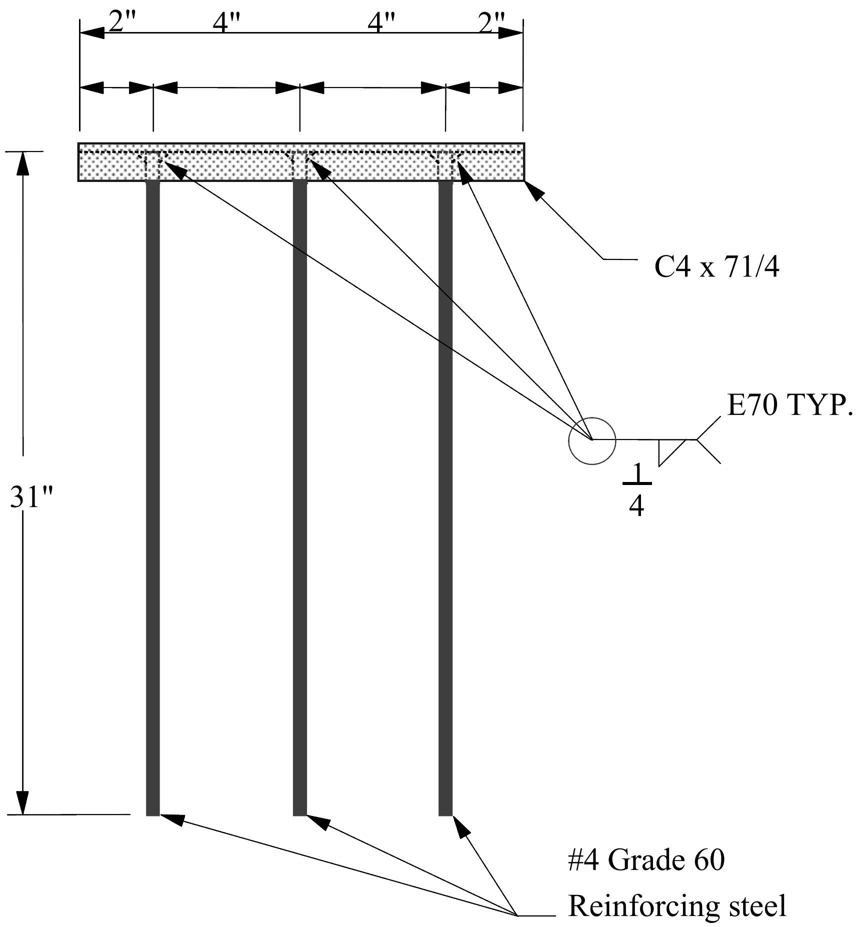
Items		Quantity
Beams	W30 x 124 - 65 ft	8
Flat Plate	3 in. x 3/8 in. - 20 ft	3
	3 in. x 3/8 in. - 20 ft	1
Channel	C4 x 7 1/4 - 20 ft	4
	C15 x 40 - 20 ft	5
Angle	3 in. x 5 in. x 1/2 in. - 20 ft	6
Reinforcement	#3 - 20 ft	270
	#4 - 20 ft	424
ASTM Type I Bolts	7/8 in. DIA. 2 in. Bolt	168
	7/8 in. DIA. 2 1/2 in. Bolt	24
	7/8 in. DIA. 2 3/4 in. Bolt	72
Nuts	7/8 in. Hex Nut	264
Washers	7/8 in. Flat Washer	528
Shear Studs	3/4 in. DIA. - 4 3/8 in.	848

Notes:
All Structural Steel A36
All Reinforcement Grade 60

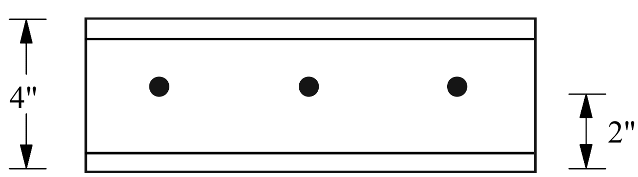
CIP reinforced concrete deck provides a continuous shear transfer medium. The second was through the PCDT unit connections [4].

The PCDT connection, shown in Fig. 3.2, consisted of a C4 x 7 1/4 channel with three Grade 60 - #4 reinforcing steel bars welded to the inside face of the channel. The length of the reinforcement was 31 in. with a center to center spacing of 4 in.

A design bridge width of 30 ft required the fabrication of four of the 7.5 ft wide PCDT units. The four units placed adjacent to each other resulted in three longitudinal joints that required the PC connectors; PC connector location for each joint is shown in Fig. 3.3. A total of 78 PC connectors were required in the superstructure.



a. Top View



b. Front View

Figure 3.2. Individual PCDT connection details.

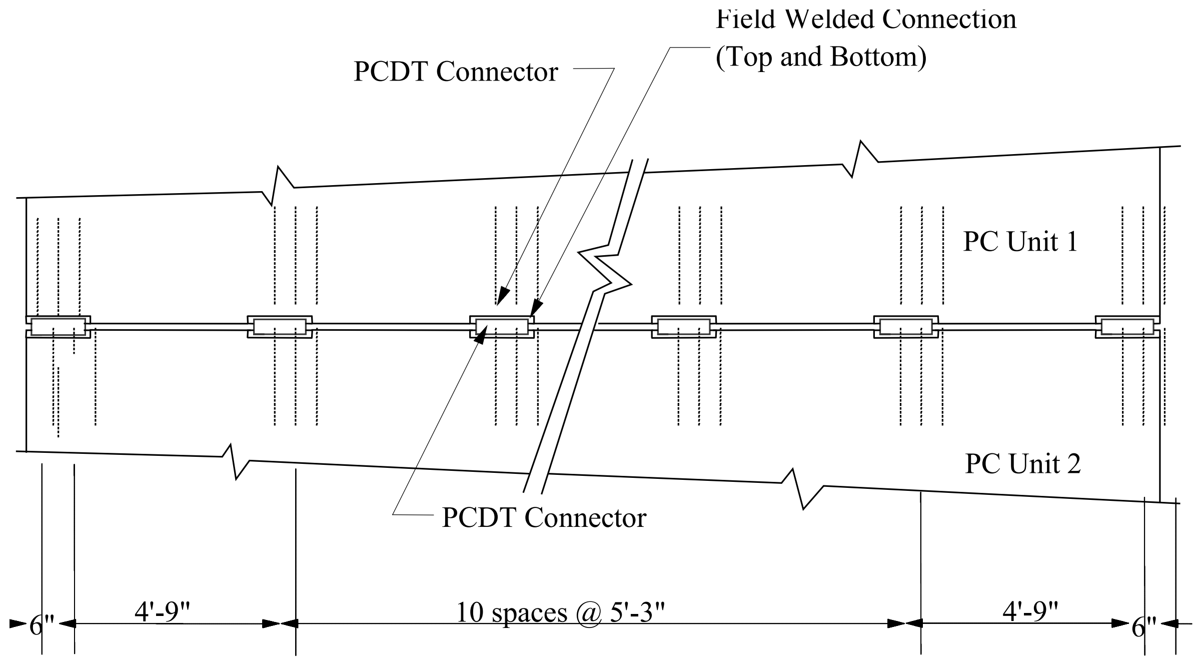


Figure 3.3. PCDT connector-plan view.

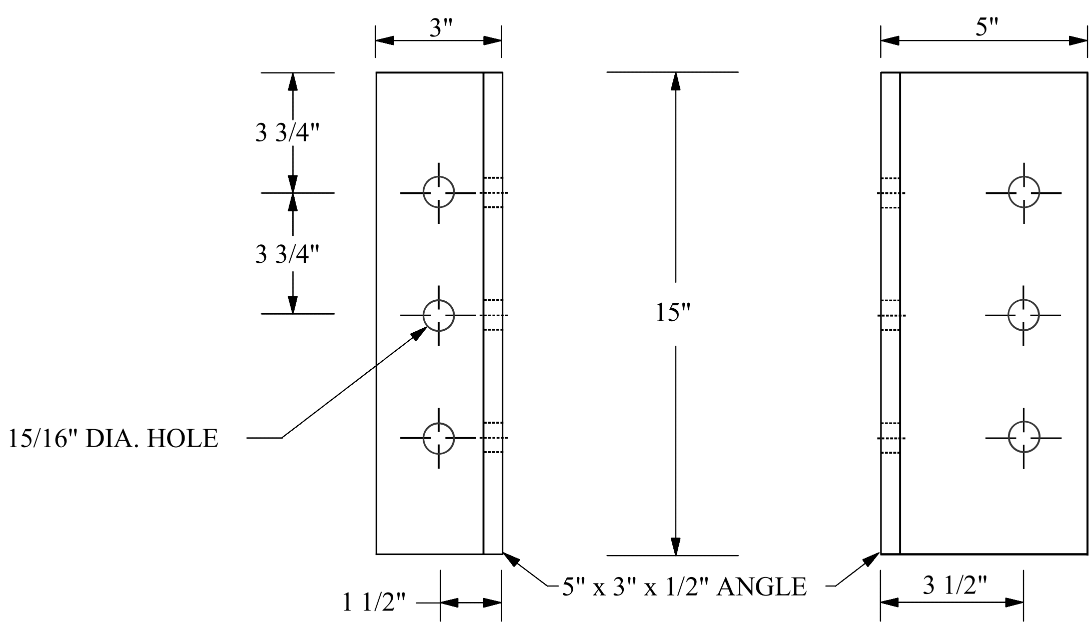


Figure 3.4. Details of diaphragm angles.

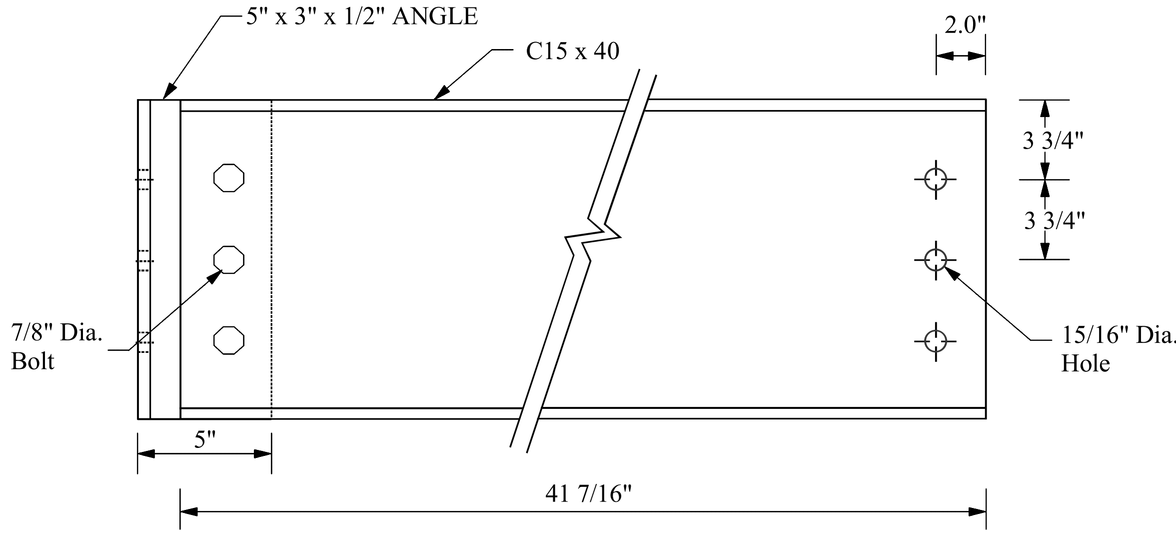


Figure 3.5. Details of diaphragm channels.

Fabrication of the PC connectors was performed at the ISU Structures Engineering Laboratory. The 78 - 12 in. long pieces of C4 x 7 1/4 channel and 234 - 31 in. long pieces of #4 reinforcement were cut in the laboratory. The #4 reinforcing bars were shop welded to the inside face of the C4 x 7 1/4 channel by a certified welder**.** All welds were performed using a MIG (metal inert gas) welder with a 75% argon - 25% CO_2 mixture and required one pass.

3.3.3 Diaphragm Details

Diaphragms were installed at the quarter points and at each end in the steel-beam PCDT units. The diaphragms were used to connect the individual beams of one PCDT unit and also to connect the beams of adjacent units. The diaphragms consisted of C15 x 40 channels field bolted to 5 in. x 3 in. x 1/2 in. - 15 in. long angles that were in turn bolted to the webs of the beams. The connection consisted of bolts that were tightened to a snug-tight position with an impact wrench; all bolts were 7/8 in. diameter ASTM Type I high strength bolts with washers. The channels were positioned at mid-height of the web of each beam.

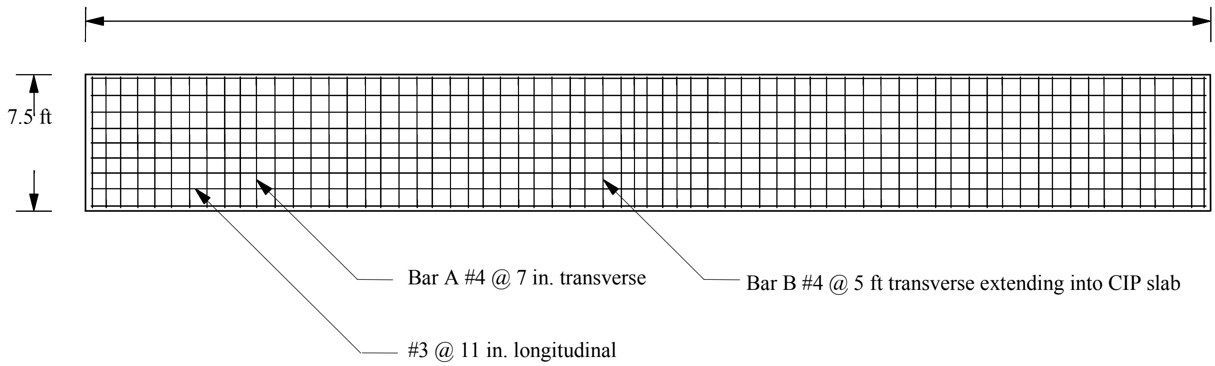
The details for the angles and channels are shown in Figs. 3.4 and 3.5, respectively. All bolt holes were drilled 15/16 in. in diameter (1/16 in. larger than the bolt).

3.3.4 PCDT Specimen

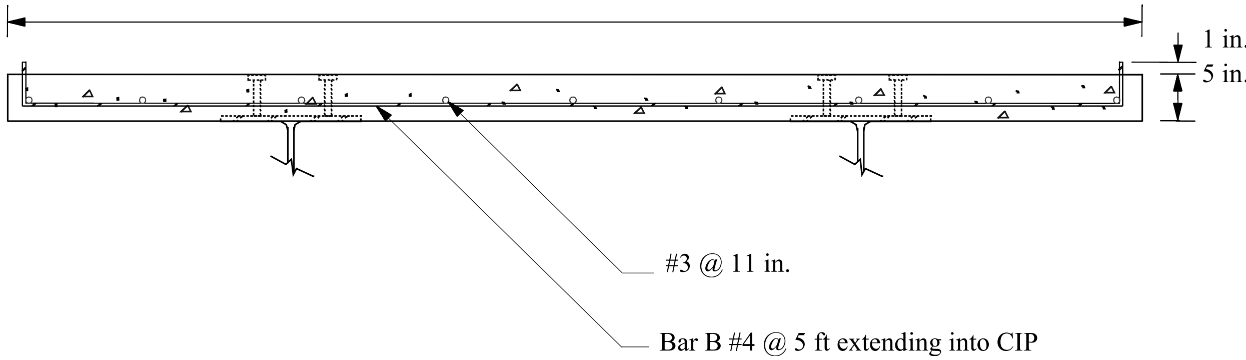
As shown in Fig. 3.1, the PCDT units constructed for the field demonstration bridge were 7.5 ft wide. The PCDT units had a 5 in. thick reinforced concrete composite deck and two W30 x 124 steel beams on 3.75 ft centers. The span length of the PCDT units was 64 ft from center-to-center of abutments (65 ft overall length).

The reinforcing steel used in the PCDT units is shown in Fig. 3.6. The design specified #3 reinforcement in the longitudinal direction on 11 in. centers and #4 reinforcement in the transverse direction on 7 in. centers. Transverse #4 bars spaced every 5 ft extended from the PCDT slab into the CIP slab to control differential shrinkage (see Fig. 3.6b). The reinforcement in the PCDT units was tied at all intersections. Individual 2 in. reinforcing bar chairs were placed under the transverse bars at a spacing of 3 ft in each direction.

Composite action between the PCDT reinforced concrete slab and the W30 x 124 steel beams was obtained with 3/4 in. dia. x 4 3/8 in. Nelson shear studs. The location of the studs, marked on the top flange surface with a steel punch, is shown in Fig. 3.7. The steel surface was then prepared by removing any surface rust with a grinding wheel. Once the surface was smooth and rust free, the shear studs were "shot" (welded) into place. To ensure that the welds achieved full penetration, two field bend tests per beam were performed. The



a. Plan view



b. End view

Figure. 3.6. Reinforcement for PCDT units.

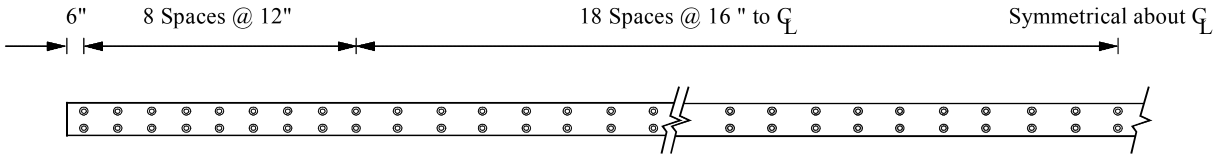


Figure 3.7. Location of shear studs for PCDT units, single beam shown.

Figure 3.8. Photograph of shear studs used in PCDT units.

bend test consisted of bending a random stud to a 45^{o} angle with respect to the beam flange; all welded studs tested passed this strength test. A photograph of the shear studs on the beams is shown in Fig. 3.8.

3.3.5 Modifications to PCDT Unit Bridge Design

The steel-beam precast unit bridge design does allow for some modifications at the discretion of the designer. However, all design changes must be reviewed by a registered professional engineer prior to construction.

Black Hawk County specified concrete end diaphragms for the PCDT unit field demonstration bridge in addition to the steel end diaphragms. Design changes included the removal of the steel diaphragms at each end that connected adjacent PCDT units. The PCDT reinforced concrete composite deck was formed so that approximately 12 in. of each steel beam was exposed at each end. This allowed for a monolithic concrete placement of the CIP deck and the end concrete diaphragms. The concrete diaphragms were reinforced with #4 continuous reinforcing bars and #4 hoops. Holes (15/16 in. dia.) were drilled through the web of each beam at the ends to accommodate the placement of the reinforcement in the concrete diaphragms. The details of the design changes are shown in Appendix B which were reproduced from Black Hawk County Project Number L-192 project drawings.

3.3.6 Construction of PCDT Units

The four PCDT units used in this field demonstration bridge were constructed over a three week period. While individual units may be constructed separately, Black Hawk County wished to construct all four units simultaneously for this project.

All the units were fully shored and fabricated using normal construction procedures. As shown in Fig. 3.9, the shoring consisted of seven plain concrete cast beams 12 in. x 12 in. x 32 ft in length with two W12 x 79-32 ft long beams placed on top. These temporary supports were located on approximately 11 ft intervals. The approximate 3 ft height of the temporary supports allowed for easy access to the underside of the PCDT units during construction.

The W30 x 124 beams utilized in the PCDT units were placed on the temporary supports and connected by the diaphragms. Steel plate shims were placed under the beams at the support locations where needed to level the structure.

Formwork used in constructing the PCDT units is shown in Fig. 3.10. The 2 in. x 6 in. headers were wedged between the top and bottom flanges of the beam using vertical 4 in. x 4 in. members. The 2 in. x 4 in. stringers were placed and nailed at each header. The 3/4 in. plywood used for the deck formwork was placed so that the bottom surface of the

Figure 3.9. Photograph of the temporary supports used during fabrication of PCDT units.

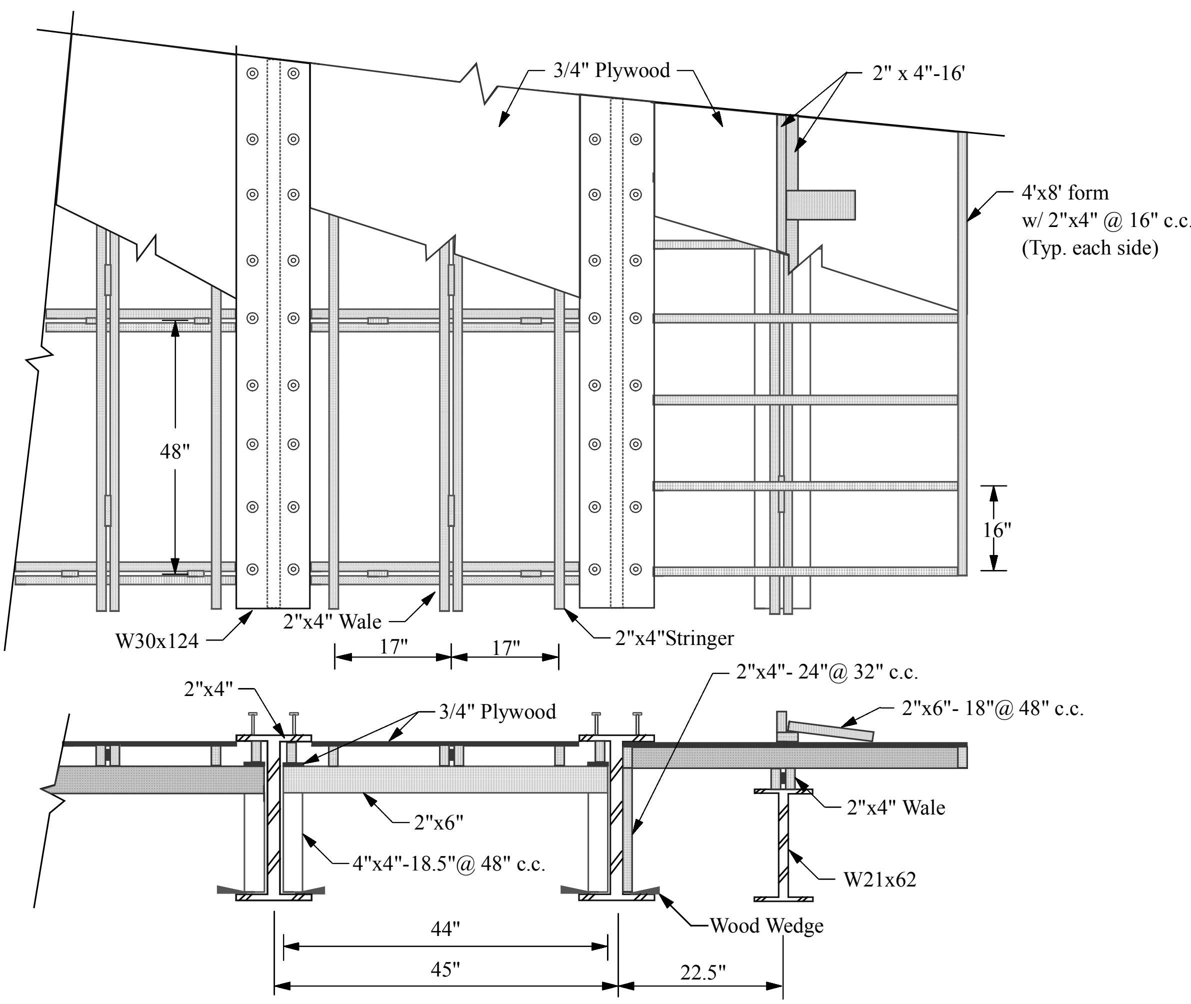


Figure 3.10. Formwork used to cast PCDT units.

concrete and the bottom surface of the top flange were at the same elevation; the plywood was nailed to the stringers at the ends and at the middle of each sheet.

The overhang for the exterior units was formed by utilizing W21 x 62 beams that were available at the county maintenance facility. These beams were placed on the temporary supports approximately 22.5 in. from the center of the outside W30 x 124 beams. Wall forms, 4 ft x 8 ft with 16 in. center to center stud spacing, were used as deck forms for the exterior flanges of the external units. A 2 in. x 4 in. wale was placed under the wall form (on top of the W21 x 62 beam) for strength and to provide a level surface. Support for the inside edge of the wall form was accomplished by wedging 2 in. x 4 in. studs on 32 in. centers between the flanges of the W30 x 124 beams. Photographs of some of the formwork are presented in Fig. 3.11.

Placement of concrete for each of the PCDT units was completed in one continuous pour. Units 1 and 3 (see Fig. 3.12) were cast in one day using the edge formwork shown in Fig. 3.11. Units 2 and 4 were cast one and two days later, respectively. The concrete edges of Units 1 and 3 provided the edge formwork during concrete placement operations of Units 2 and 4. A layer of 4-mil polyethylene film was placed between adjacent units in the longitudinal direction to provide a bond break. In addition, wood blockouts were placed on top of the PC connector channels to create a void in the concrete for placement of the weld plates later in the construction process (see Fig. 3.13). The concrete was transported from ready-mix trucks to the PCDT forms using a concrete bucket and mobile truck crane after which it was spread, and vibrated as necessary. Screeding of the top surface was performed to obtain the required deck thickness. Steel troweling followed the hand screed process to

a. PCDT deck formwork

b. PCDT edge formwork

Figure 3.11. Photographs of formwork used for PCDT units.

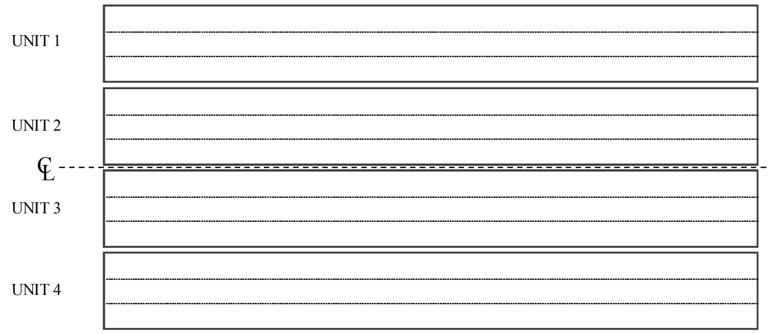


Figure 3.12. PCDT number designations.

Figure 3.13. Wood blockout covering PC connector.


Figure 3.14. Concrete placement operation for PCDT units.

provide a hard, dense surface free of defects. As previously noted, composite action between the PCDT and CIP concrete slabs was critical in the design. Therefore, the top surface of the PCDT slab was intentionally roughened in the transverse direction to provide the shear transfer mechanism between the two slabs. The "grooves" were scarified in the wet concrete using a bull float with lumber strips, approximately 1/4 in. wide on 1 in. centers nailed to the underside. The concrete placement process operation is shown in Fig. 3.14, with the final product shown in Fig. 3.15.

3.4 Transportation of the PCDT Units

The four PCDT Units were allowed to cure for 21 days prior to removal of the formwork. Once all the formwork was removed, the intermediate diaphragms connecting adjacent units were labeled and removed. Marks were also placed on the PCDT slab of the four units for alignment purposes in the field. After removing tack welds joining the PCDT connectors of adjacent units, placed during fabrication, all PCDT units were ready for transport to the bridge site.

The modification for concrete end diaphragms allowed the units to be lifted by connecting to the ends of the two steel beams in each unit; no special lifting devices were required. Two mobile cranes were used to pick each unit from the temporary supports and to place them on the hauling units (See Fig. 3.16). All rigging used in moving the units and transporting them to the bridge site was the decision of Cramer and Associates, Inc.

Two hauling units were used during the transportation of the PCDT units. The hauling units used to transport the PCDT units consisted of a standard tractor/trailer setup.

a. Placement of "grooves" for PCDT units.

b. Photograph of "grooves".

Figure 3.15. Intentionally roughened surface of PCDT units.

The flat bed trailers were both extended to the length of 60 ft and were limited to a capacity of 80,000 lbs. and 100,000 lbs. based upon the number of axles (20,000 lbs./axle). The units, weighing approximately 46,400 lbs. each, were placed as close to the front of the trailers as possible. However, as shown in Fig. 3.17, each PCDT unit cantilevered approximately 4 to 6 ft off the back of the trailer.

3.5 Assembly of PCDT Units

Upon arrival of the first PCDT unit (Unit 4) at the bridge site, the two intermediate diaphragms for connection to the adjacent unit (Unit 3) were reattached and allowed to cantilever from the unit; bolts connecting the diaphragms to the angle connections were not tightened. The lifting process at the bridge site was identical to that used at the county maintenance facility. Using the two on-site crawler cranes, Unit 4 was placed on the abutments. Upon arrival at the bridge site, the next unit (Unit 3) had the two intermediate diaphragms loosely attached (for the connection to Unit 2) and was lifted into place making sure that the diaphragm channels from Unit 4 were on the correct side of the diaphragm connection angle on Unit 3. This unit was correctly positioned with Unit 4 using the alignment marks that had been previously made on each unit. This process continued until all four units were properly positioned on the abutments after which the remaining diaphragm bolts were installed (connecting the first unit to the second unit, the second unit to the third, etc.). All bolts were tightened to a snug-tight position using an impact wrench.

Figure 3.16. Photograph of lifting one PCDT unit at county maintenance facility.

Figure 3.17. Photograph of one PCDT unit on flat bed.

The steel beams of each PCDT unit were welded to the top of the east abutment. Each side of each bottom flange required a 5/16 in. field weld approximately 6 in. in length. The field welding was performed by a certified welder using a shielded metal arc welding (SMAW) process.

Adjacent PCDT units were also joined using the PC connectors previously described in Section 3.2.2. The PCDT connection in Fig. 3.18 illustrates two adjacent PCDT units welded together. Tight construction tolerances necessitated the replacement of the 3 in. x 3/8 in. x 10 in. flat plates with 2 in. x 3/8 in. x 10 in. flat plates. The flat plates, 2 in. x 3/8 in. x 10 in., were field welded to the top and bottom flanges of the channel by a certified welder using a SMAW process.

Vertical misalignment of Units 3 and 4 along the longitudinal joint required "filler" plates under the flat connection weld plate. This vertical deviation between the two units ranged from 1/4 in. to 1/2 in.

3.6 Placement of CIP deck

Upon completion of welding the PCDT connectors, the units were ready for the CIP portion of the deck. The CIP deck was placed in two separate sequences. The longitudinal centerline of the bridge divided the two concrete placement operations (North and South). Formwork for lateral containment of the concrete on the outside edge of the exterior PCDT units (Units 1 and 4) was accomplished by attaching a 1 in. x 8 in. board with Tap-Con screws to the PCDT slab. The CIP deck thickness along each outside edge was 4 in. The formwork at the longitudinal centerline consisted of nominal 1 in. thick boards attached to vertical 2 in. x 4 in. boards at 12 in. intervals. The 2 in. x 4 in. vertical boards were attached to 3/4 in. nominal thickness plywood pieces used as bracing for the formwork (see Fig. 3.19). The CIP concrete depth at the bridge centerline was approximately 7 in. (2% slope). A 2 in. x 4 in. board was nailed onto the inside face of the longitudinal centerline

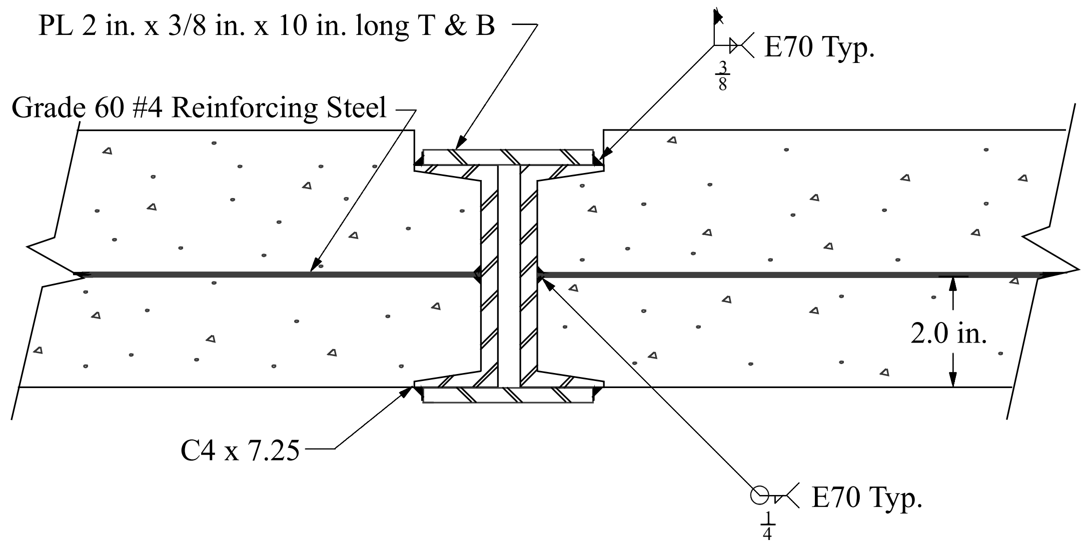


Figure 3.18. Side view of PCDT connection after adjacent units are welded together.

form to create a keyway for the next concrete placement (see Fig. 3.20). A keyway was also placed on the inside face of the formwork for each end diaphragm. Appropriate snap-tie spacing and bracing was utilized at the discretion of Cramer and Associates, Inc.

Once the formwork had been constructed, the reinforcing steel was placed. Two transverse #4 bars, 20 ft in length, were place on the longitudinal bars extending from the PCDT deck slab end to complete the bottom mat reinforcement. These bars had been intentionally left out during the fabrication of the PCDT deck so that there was less interference during the lifting of each unit.

The south half of the CIP deck steel was placed and tied at every reinforcing bar intersection and positioned onto individual chairs to provide the required top cover. The chairs were positioned under the longitudinal bars. The transverse bars were placed on top of the longitudinal bars and were continuous across the longitudinal centerline joint by inserting them through holes drilled in the formwork. Figure 3.21 shows the typical reinforcement spacing for the CIP deck.

The reinforcement for the end diaphragms was placed prior to the installation of the backwall form; see Fig. B.2 in Appendix B for details.

Figure 3.19. Formwork at bridge centerline for CIP deck placement.

Figure 3.20. Photograph of the keyway placed on longitudinal formwork.

Embedment rods for the bridge rail were placed and attached to the CIP deck reinforcement. The 1 in. diameter threaded rods had 34 in. of embedment and a 90^o bend with a 12 in. extension. Two threaded rods were required for each post and were spaced 5.5 in. apart (center to center). The spacing of the eight bridge rail posts consisted of 7 spaces at 8 ft - 4 in. The first bridge rail post was located approximately 3 ft - 4 in. from each end.

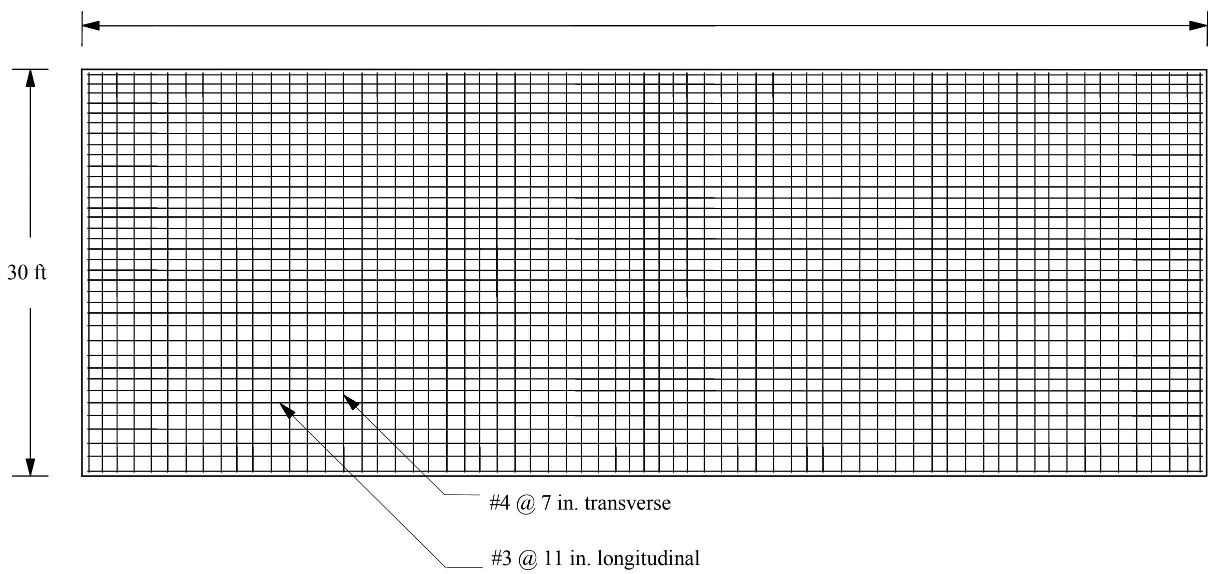


Figure 3.21. Reinforcement details for CIP deck.

Placement of concrete for each half of the CIP deck was completed in one continuous pour. The end diaphragms were placed monolithically with the CIP slab. As with the PCDT deck concrete, the ready-mix concrete was sampled and tested according to ASTM Standards. The concrete was placed and vibrated similar to the process used for the PCDT concrete, however the hand screed was replaced by a vibrating screed. A second strikeoff screed followed to ensure the top surface of the slab was at the correct elevation.

Bullfloating followed the strikeoff screed and was completed before any bleed water accumulated on the surface. Hand floating of the concrete along the edges was also completed prior to bleed water accumulation. A skid-resistant surface was placed in the concrete with a steel-wire tining rake, transverse to the direction of traffic, before the concrete cured.

A curing compound was applied to the CIP concrete surface after the skid-resistance texture was placed. The membrane forming compound aided in preventing the loss of moisture from the concrete slab. After the initial concrete set, the deck was covered with moist burlap and a 4-mil thick polyethylene film.

Approximately 21 days after the placement of the CIP deck, the bridge and approach rails were attached. This phase of construction was performed by Dave Gryp Construction, Inc., located in Victor, Iowa, under a subcontracted agreement with Cramer and Associates, Inc.

4. FIELD TESTING

4.1 Overview

A field testing program was utilized to verify the laboratory test results, the design methodology and load response of the precast bridge system [4]. The testing program consisted of instrumenting and monitoring one PCDT unit during its initial lift from the temporary supports, placement onto the hauling units, transportation to the bridge site, and lifting from the hauling unit and placement on the field abutments. Additionally, all four units were instrumented for service load testing of the assembled PCDT units with and without the CIP deck and with and without the bridge rails in place.

4.2 Testing of Construction Materials

The structure was constructed with a standard Iowa DOT C-4 concrete mix. The concrete was carefully monitored during placement to assure proper air entrainment and slump. Concrete cylinders (6 in. x 12 in.) were cast during concrete placement activities to monitor the compressive and split cylinder strengths. Standard modulus of rupture beams were also cast during concrete placement operations to determine the flexural tensile strength. All concrete testing operations were completed according to the applicable American Society of Testing Materials specifications. While all concrete specimens were made by representatives of Black Hawk County, they only tested the modulus of rupture beams. All other laboratory testing was performed at the ISU Structures Engineering Laboratory by the research team. All the concrete strength results are presented in Appendix C.

4.3 Instrumentation and Data Acquisition

A data acquisition system was used to collect and store deflection and strain gage data. Vertical deflection instrumentation consisted of Celesco displacement transducers which provided a voltage output through a precision potentiometer which was converted to a deflection by the data acquisition system.

Strain data were collected using gages purchased from Measurements Groups, Inc., Micro-Measurements Division. Steel and concrete strain gages were all oriented to measure longitudinal strains. Two different concrete gages were utilized in the testing program as it was necessary to use two different strain gage multiplexer units (120 and 350 ohm).

The PCDT units were instrumented at three sections: the 1/4 point, mid-span , and 3/4 point. The concrete strain gages were mounted on the top surface of the PCDT and CIP decks. The transverse location of the gages on the PCDT and CIP concrete deck varied from section to section. Steel strain gages were placed on the top surface of the lower flange and the bottom surface of the upper flange of the W30 x 124 beams. Typical locations of the strain gages are shown in Figure 4.1. The steel strain gages were placed on each side of the web and were centered within the half-width of the flange; the location of all gages used during each test is presented in the following sections.

4.4 Transport Testing

This phase of the testing program consisted of three individual tests: initial lifting of one PCDT unit (PCDT Unit 2) from the temporary supports onto the transport unit,

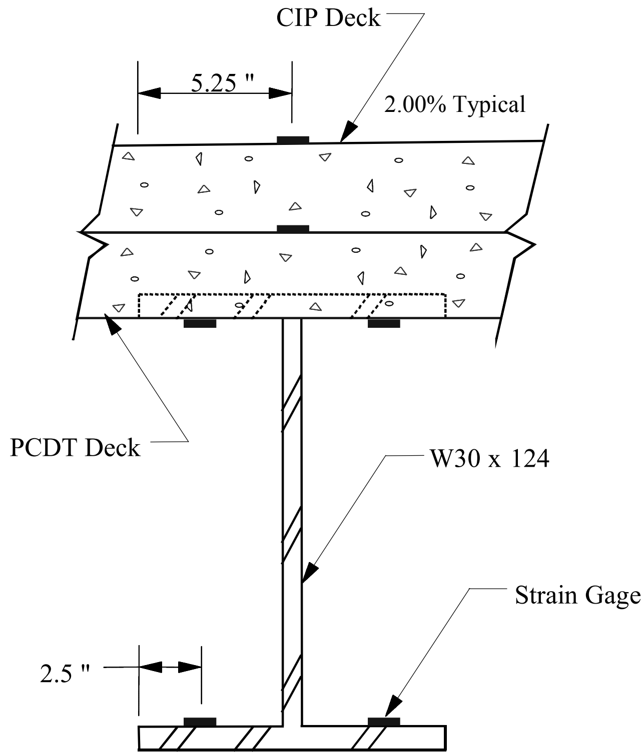


Figure 4.1. Typical location of strain gages.

transportation of the same PCDT unit to the bridge site, and lifting of the PCDT unit from the transport vehicle to the newly constructed bridge abutments.

Three locations on the PCDT unit were instrumented with concrete and steel strain gages; as previously noted, all gages were oriented to measure longitudinal strain. The location of the gages is shown in Fig. 4.2. Four concrete and eight steel gages were used these tests.

The first phase of the transport test involved measurement of the strains during the initial lift of PCDT Unit 2 from the temporary supports at the county maintenance facility; the test started prior to the hydraulic mobile truck cranes applying tension in the lifting

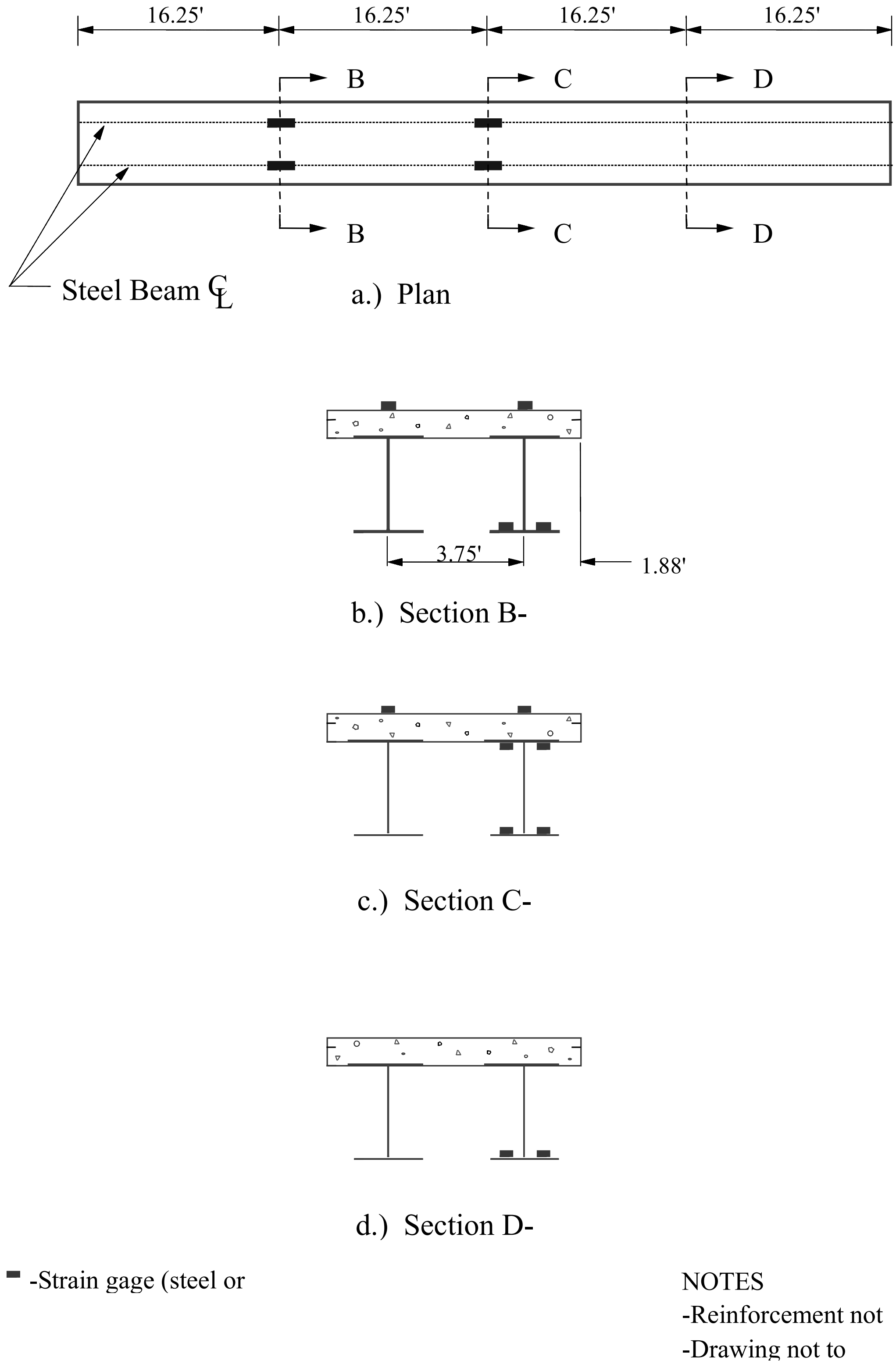


Figure 4.2. Location of strain gages used in transport test of PCDT Unit 2.

cables. As shown in Fig. 4.3, the data acquisition system was placed in a protective cargo box on top of the PCDT concrete deck and held securely in-place with tie-down straps. This test was stopped once the PCDT unit was properly positioned on the transport unit.

Figure 4.3. Data acquisition cargo box secured onto PCDT Unit 2.

The same strain gage setup was used to monitor the range of strains encountered in PCDT Unit 2 during transport to the bridge site. After the PCDT unit was properly chained to the transport bed and prior to movement of the vehicle, the second test was started. Once the hauling unit was at the bridge site, this portion of the testing was terminated.

PCDT Unit 2 was also monitored as it was lifted from the hauling unit and positioned on the abutments. Procedures identical to those used during the initial lift at the county maintenance facility were followed during this test. This test was started prior to the cranes applying tension to the lifting cables and was stopped once PCDT Unit 2 was positioned on the new abutments.

4.5 Service Load Tests without CIP Deck (First Service Load Test)

The initial service load test was performed once all four PCDT units were correctly positioned on the abutments and connected by the intermediate diaphragms and PCDT connectors (see Sec. 3.4). The average 28-day concrete strength of the PCDT units was 7,400 psi (52 days after concrete placement).

The four PCDT units were instrumented at three transverse sections with various combinations of concrete and steel strain gages. As previously noted, the gages were oriented to measure longitudinal strains. As can be seen in Fig. 4.4, the mid-span location was fully instrumented and the 1/4 and 3/4 sections were only partially instrumented due to symmetry. A total of ten concrete and 36 steel gages were used during this service load test. Gages used during the transport testing program were reused in the service load tests.

Displacement transducers were utilized to measure the vertical deflections during the service load tests. The deflection transducer wires were attached to the bottom flange of four beams at each quarter point and the bottom flange of all eight beams at the mid-span section (see Fig. 4.4). As may be seen in Fig. 4.5, the transducers were attached to the top of surveying tripods securely seated in the stream bed.

Service loading was provided through the use of an empty Black Hawk County rear tandem axle truck. Dimensions of the truck and axle weights are shown in Fig. 4.6. An empty truck was used in the tests of the PCDT assembly due to the relatively thin layer of reinforced concrete in the PCDT units (5 in.).

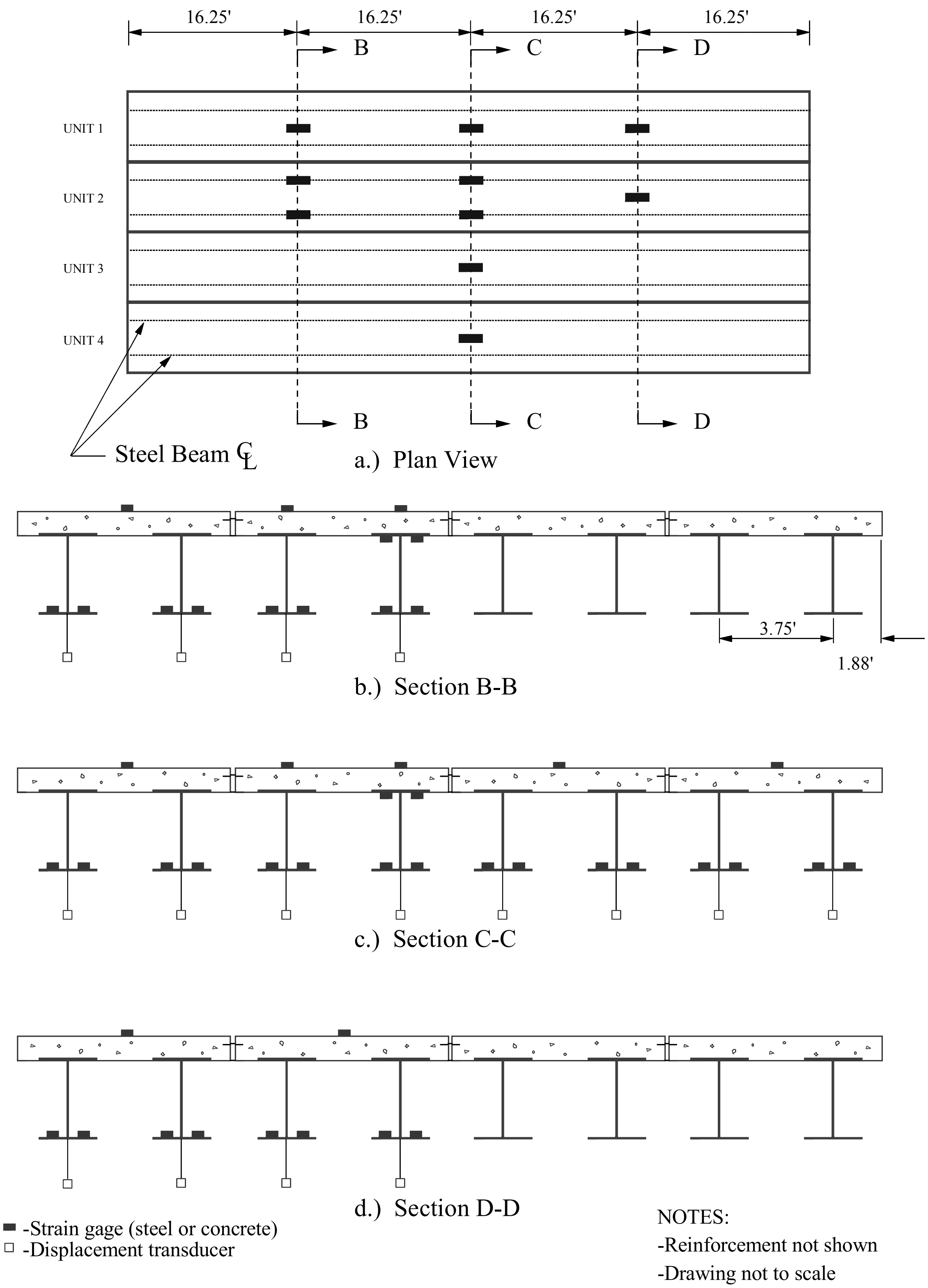


Figure 4.4. Location of strain gages used in first service load test.

Figure 4.5. Photograph of displacement transducers on tripods during service load tests.

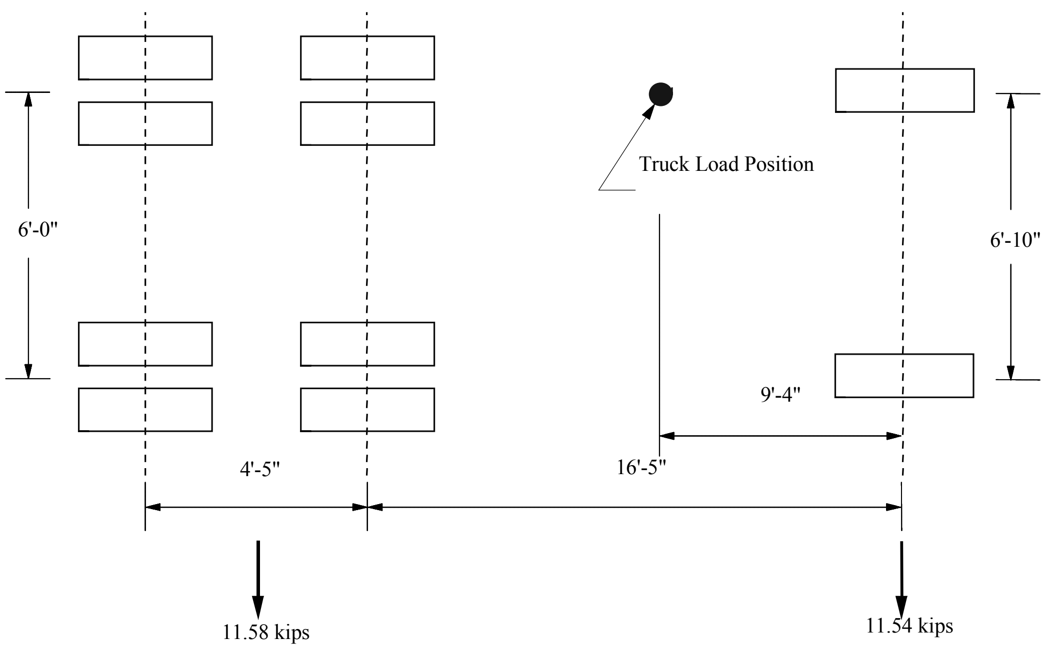


Fig. 4.6. Plan view of live load used for first service load test: Truck No. 23.

The first service load test was conducted by placing the unloaded truck at several pre-established locations on the bridge. The truck was positioned according to its center of gravity (9 ft - 4 in. from the front axle) and transversely based on the left wheel line. As shown in Fig. 4.7, in the service load tests, the truck crosses the bridge in 5 different lanes (3 while heading east and 2 while heading west). In each of the 5 lanes, the truck was positioned so that its center of gravity was at Section B, C, and D (i.e. the 1/4 point, mid-span, and 3/4 point). Thus, data were taken with the truck at 15 different locations on the bridge. Tests were identified by transverse section (B, C, or D) and longitudinal location (1, 2, 3, 4, or 5). Thus, C3 indicates the center of gravity of the truck was at Section C (mid-span) while the left wheel line was in Lane 3.

Prior to having the truck cross the bridge in one of the five lanes, all the strain gages and displacement transducers were initialized. The truck was then placed at the first position (B1) and all strain gage and displacement transducer measurements were read and recorded. Without re-zeroing the instruments, the truck was moved to the second and third positions (C1 and D1) and all instrument measurements were again read and recorded. The truck was then removed from the bridge and all instrument measurements were again read and recorded (i.e. final zero reading). This process was repeated for each of the remaining four lanes. A photograph taken during the first service load test is shown in Fig. 4.8.

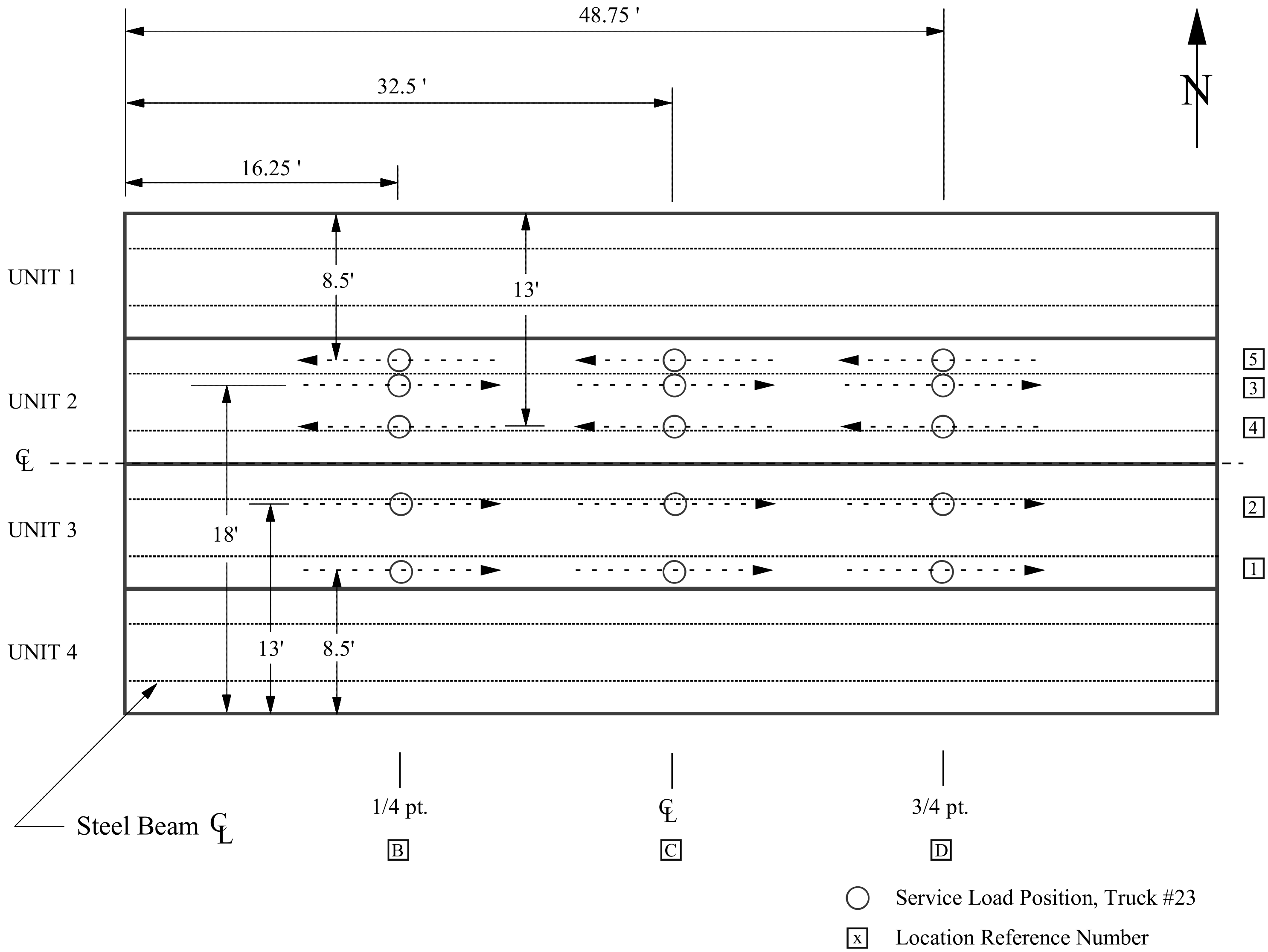


Figure 4.7. Location of live load in first service load test.

Fig. 4.8. First service load test, truck in position C3.

4.6 Service Load Test with CIP Deck (Second Service Load Test) and with Bridge Rail (Third Service Load Test)

The second service load test was performed once the CIP deck was placed and allowed to cure, however the bridge rail had not been installed. The third service load test was performed after the bridge rail was in place. The average 14-day and 28-day concrete compressive strength of the CIP deck was 5,500 psi and 6,150 psi, respectively. The second service load test was performed 23 days after concrete placement (CIP), and the third service load test was performed 8 months later.

The PCDT unit bridge was instrumented at the same three transverse sections (as in the first service load test) with concrete and steel strain gages. All the gages used during the first service load test were reused in this test. To evaluate the CIP deck, an additional eight concrete gages were installed for use in the second service load test. The location of all the gages used for both tests is shown in Fig. 4.9. Displacement transducers were utilized to measure the vertical deflections during both service load tests. The transducer locations

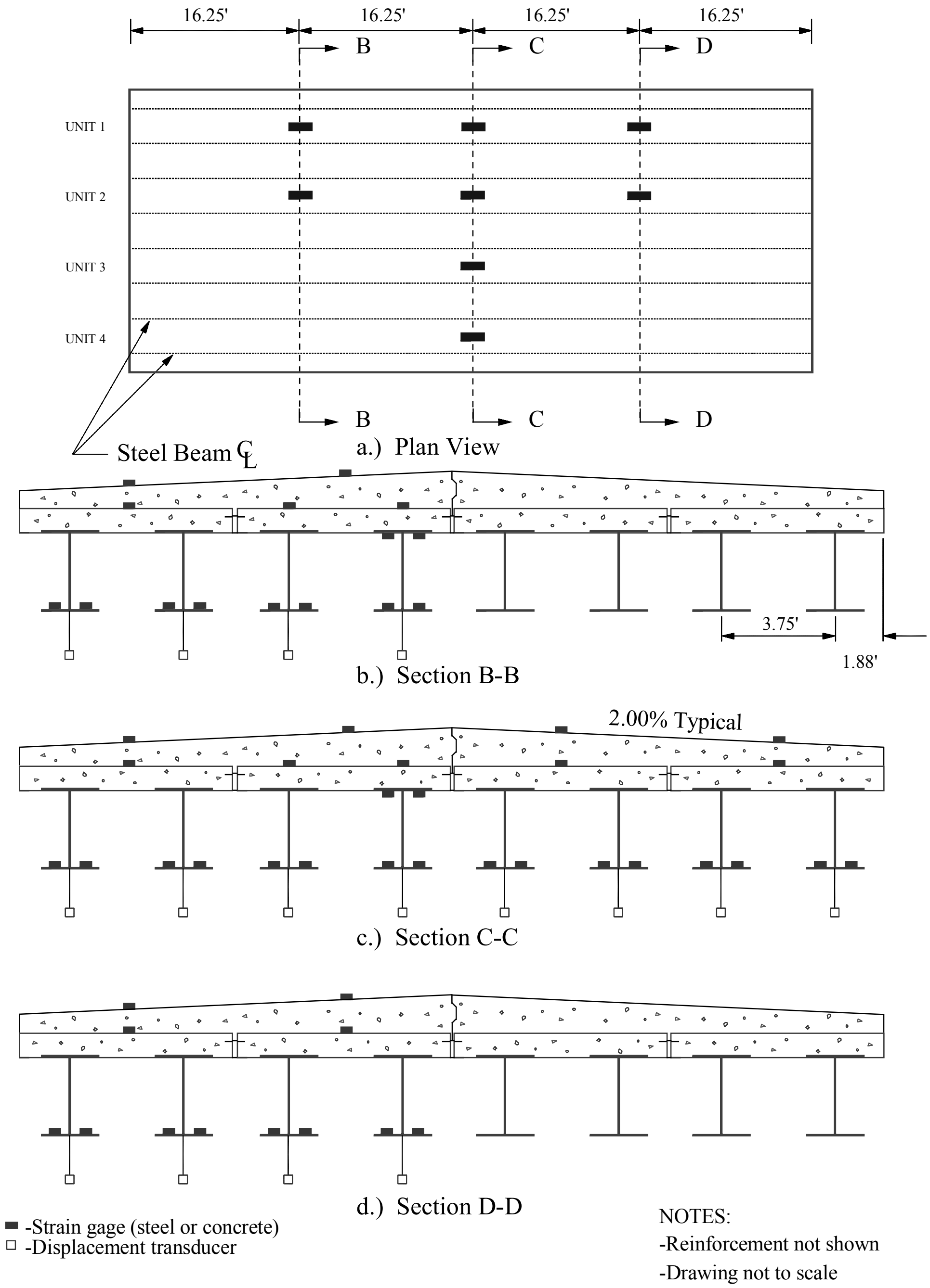


Figure 4.9. Location of strain gages used in second and third service load test.

were the same as the first service load test (see Fig. 4.4 and Fig. 4.9). In the third service load test, four additional strain gages were placed on the bridge rail at midspan (two each on each bridge rail). Several strain gages were also placed on the stringers near the abutments to detect end restraint.

The service load tests were again performed using Black Hawk County rear tandem axle trucks. The dimensions of the two trucks are shown in Fig. 4.10. The axle weights of the trucks are shown in Table 4.1. The trucks were fully loaded and were equipped with front end snow plows and rear sand spreaders.

The second and third service load tests were conducted by placing Truck No. 23 at several pre-established locations on the bridge. Unlike the first service load test, the truck was positioned according to the center of the rear tandem axles on the left wheel line. As shown in Fig. 4.11, the truck crossed the bridge in 5 different lanes and was stopped with the center of the left tandem axles at Sections B, C, and D. The designation used in the first service load tests to identify the numerous truck positions was also used in the second and third service load tests. Note that load position B5 was not used in the second service load test because the location of the data acquisition system and other monitoring equipment made it inaccessible.

In both the second and third service load tests, after completing the testing involving one truck, two additional tests were performed in which both tandem axle trucks were placed on the bridge simultaneously. A photograph, Fig. 4.12a, taken during the second service load test shows the two rear tandem axle trucks at Positions B1 and B5; Fig. 4.12b was taken during the third service load test and show the trucks at Positions C1 and C5. Figure 4.13

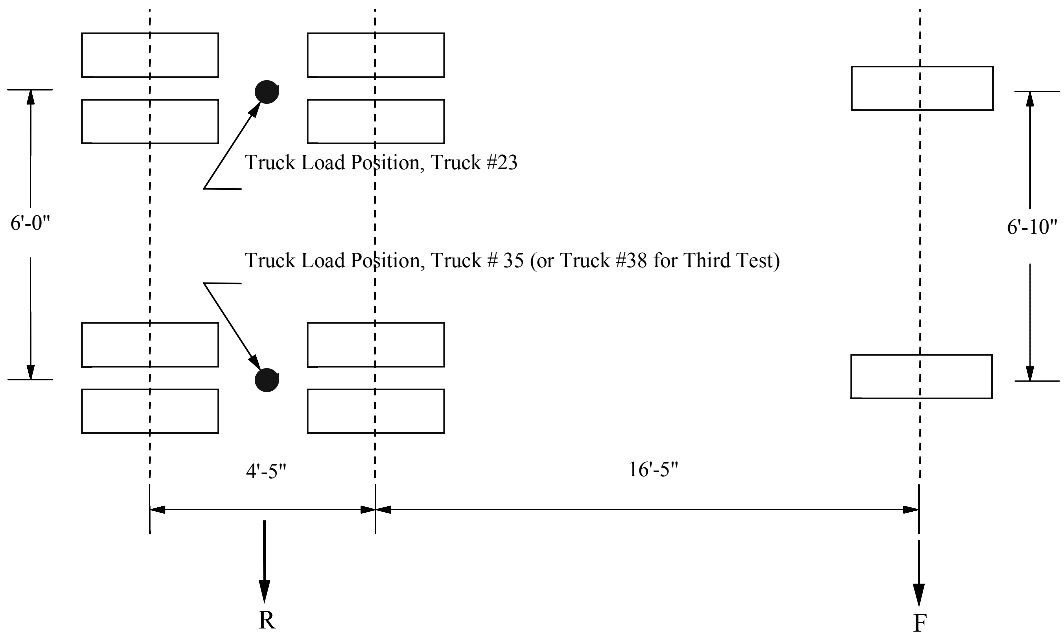


Figure 4.10. Plan view of live load used for second and third service load test.

Table 4.1. Live load used for second and third service load test.

Test	Truck	F (kips)	R (kips)	Total Weight (kips)
Second Test	No. 23	18.82	33.10	51.92
	No. 35	20.82	33.84	54.66
Third Test	No. 23	19.10	33.06	52.16
	No. 38	17.84	37.12	54.96

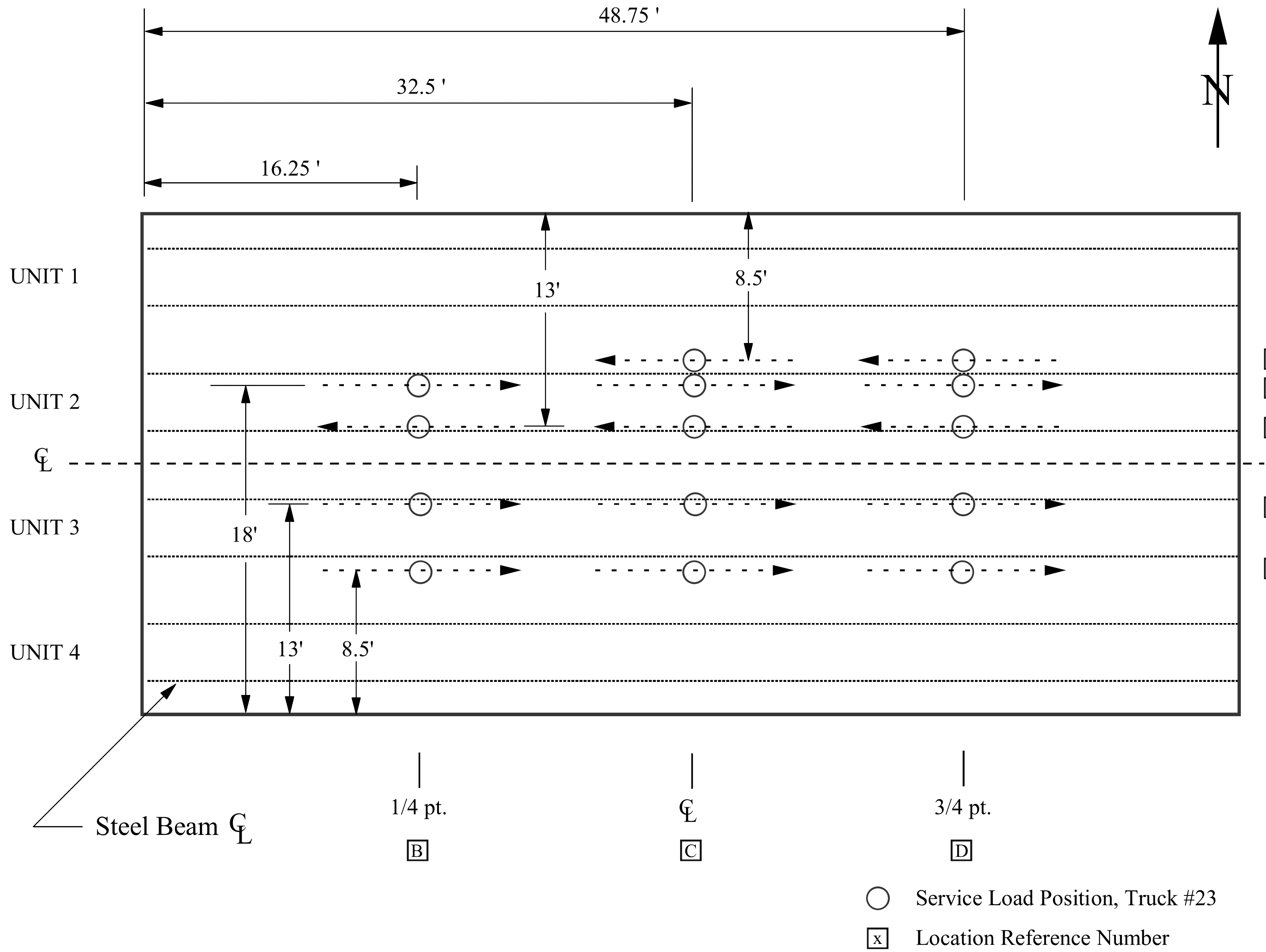


Figure 4.11. Location of live load in second and third service load test.

illustrates the positions of the two rear tandem axle trucks as they crossed the bridge in 2 different lanes headed east. As shown in this figure, Truck No. 23 was positioned using the center of the left rear tandem axles, while Truck No. 35 (or Truck No. 38 in the third test) was positioned using the center of the right rear tandem axles; data were only taken when both trucks were at the same transverse section, that is at Sections B, C, or D. Thus, data were taken with each of the two trucks at 6 different locations on the bridge. Tests were identified by transverse section (B, C, or D) and longitudinal location (1, 2, 4, or 5). Thus, B1 and B5 indicates that Truck No. 23 was at Section B, Lane 1 with Truck No. 35 (or Truck No. 38) at Section B, Lane 5.

a. Trucks in Position B1 and B5

b. Trucks in Position C1 and C5

Figure 4.12. Second and third service load tests.

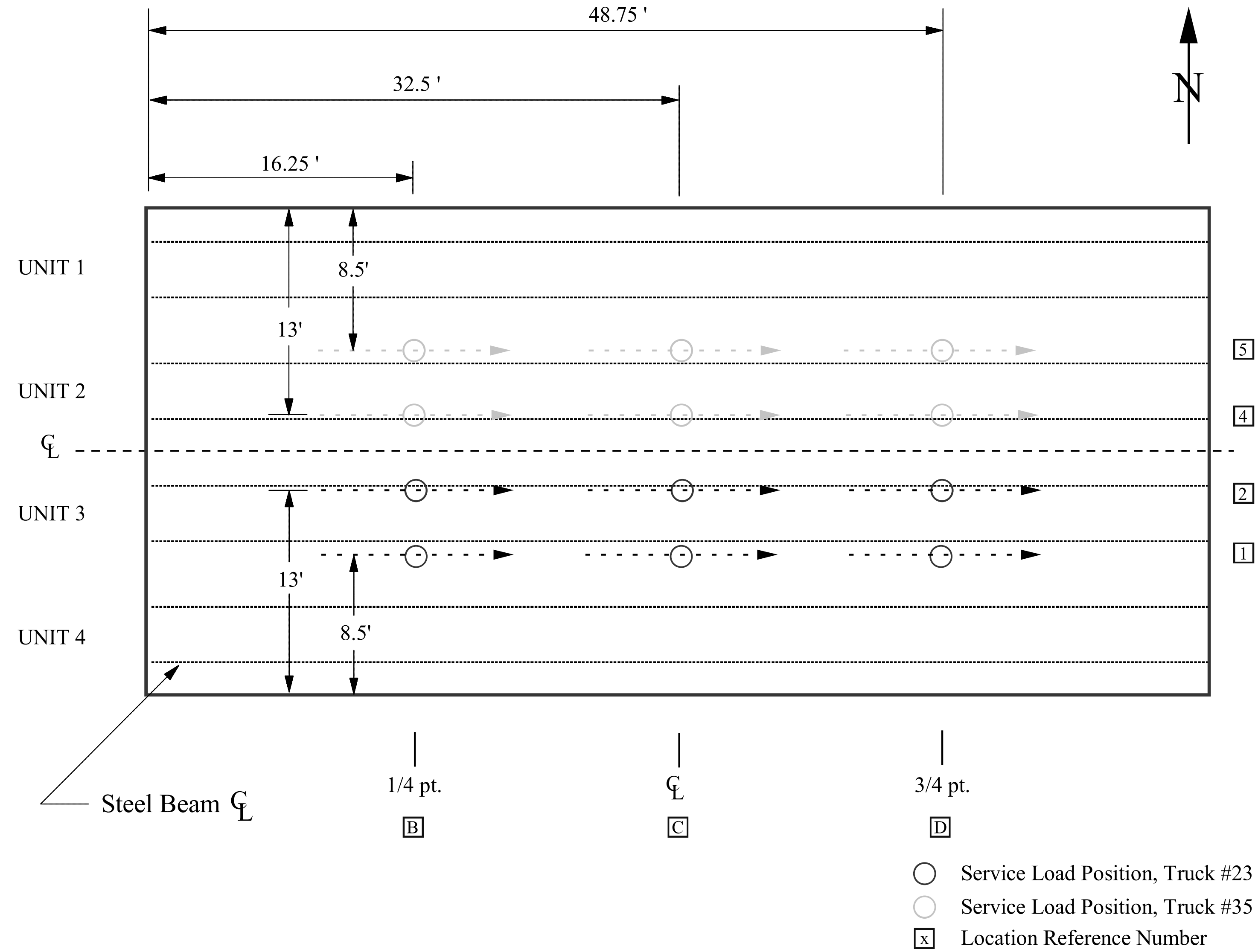


Figure 4.13. Location of live load in second and third service load test; two tandem axle trucks.

5. EXPERIMENTAL RESULTS

5.1 Construction Documentation

5.1.1 Video Documentation

Fabrication and construction of the PCDT Unit Bridge was documented using a video cassette recorder. Twelve hours of images were compiled. The final video was edited to approximately 20 minutes which shows all phases of fabrication and construction. The substructure construction was the responsibility of others and thus is not included in the video. BG Productions, Inc., located in Ames, IA assisted in the production of the video. (A copy of the video can be obtained by contacting the Iowa DOT Materials Office.)

5.1.2 Slide Documentation

The fabrication and construction process was also photographed using slide film. Over eight hundred slides were taken. Photographs from the slides which also illustrate the fabrication/construction process are presented in Appendix D. For convenience, the reader may review the video or the slides. The slides are an abbreviated version of the video. (Slides of the photographs in Appendix D can be obtained from the Research Engineer at the Iowa DOT Materials Office).

5.1.3 Data Analysis

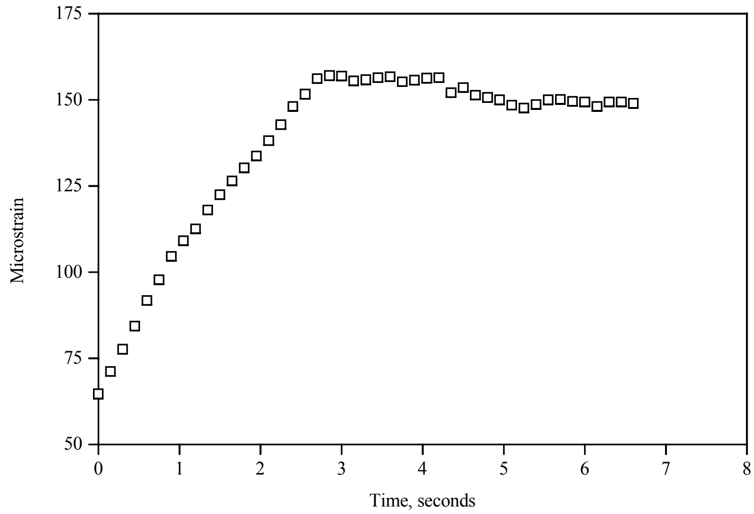
Data from the lifting and transport tests, and the various service load tests, were reduced using Microsoft Excel (Version 7). Microcal Origin (Version 4.1) was used to generate the graphs. The behavior of the completed bridge was evaluated by comparing the field test results with analytical results obtained from two model sources: 1) AASHTO design specifications and 2) beam theory analysis.

5.2 Transport Testing Results

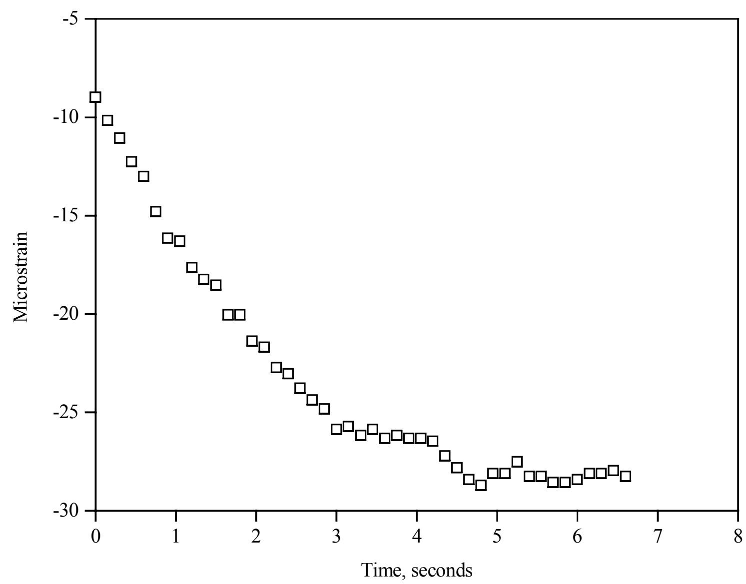
The first phase of transport testing involved lifting PCDT Unit 2 from the temporary supports and placing it on the transport vehicle. A data acquisition system program was written to record the data. The program had "trigger" values ($\pm$ 165 microstrains (μm) in the midspan bottom flanges) which started the recording process. Once these mid-span strains were greater than $\pm$ 165 μm, data would be recorded starting with the previous two seconds of data. If the strains dropped between $\pm$165 μm, the program would record data for two additional seconds and then pause until the "trigger" values were again reached.

In the first phase, there were approximately eight seconds of recorded data obtained during the approximate ten minute lifting operation. However, the exact position of Unit 2 during the data recording process is unknown. It is hypothesized that the data represents the time just before Unit 2 was lifted from the temporary support. Therefore, the last two seconds of data would represent the time frame of total suspension of the unit between the two hydraulic cranes. The data presented in Fig. 5.1 represents the average recorded strains in the two midspan bottom flange gages (Fig. 5.1a), two midspan top flange gages (Fig. 5.1b), and two PCDT slab gages (Fig. 5.1c); for the location of these gages see Fig. 4.2.

Based upon the lifting points and the section properties, analytical strains were calculated for comparison to the experimental strain values presented in Fig. 5.1. For clarity, not all data points are shown. Table 5.1 presents the analytical and maximum experimental

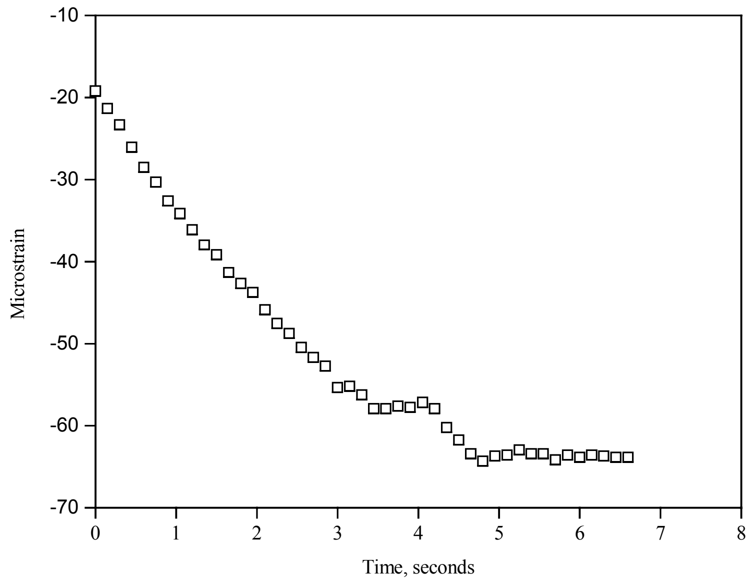


a. Bottom flange strains



b. Top flange strains

Figure 5.1. Lifting of PCDT Unit 2- test results at mid-span.



c. PCDT slab strains

Figure 5.1. Continued.

Table 5.1. Analytical strains at mid-span due to the lifting PCDT Unit 2.

Location	Analytical	Experimental
Slab	-71	-65
Top Flange	-40	-28
Bottom Flange	158	157

strain values. As shown in the table, the mid–span analytical and experimental strains were very comparable. Experimental strains measured at the 1/4 and 3/4 point of the span were significantly smaller and didn't agree with the analytical strains as well.

The second transport test involved monitoring the range of strains in PCDT Unit 2 during transportation to the bridge site. After the unit was properly chained to the transport bed and prior to movement of the vehicle, the gages were initialized and the test was started. Since the gages were initialized while in a pre-strained condition, the recorded strains were just the change in strain from the initial zero reference position. Therefore, the actual strains in the unit during transportation to the bridge site are not known; only the change in strains are known.

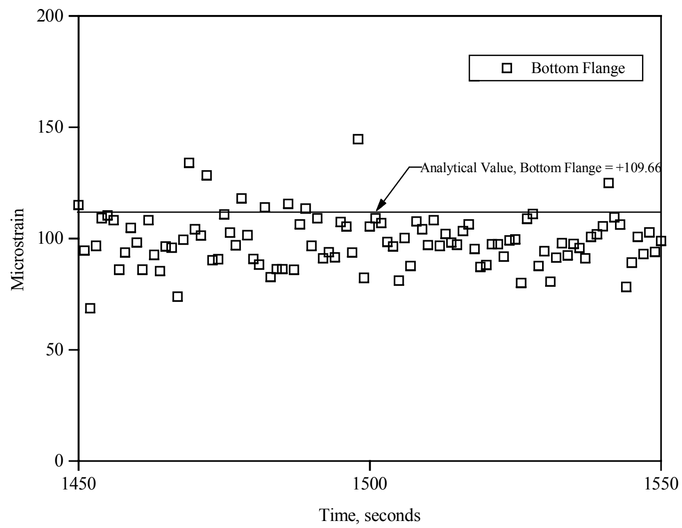
Thus, there were two ways to present the data. Data could be presented according to the initial zero; these plots were cluttered with data overlap and thus difficult to interpret. The second option was to calculate the analytical strains for the support conditions while on the hauling vehicle wasn't moving; these calculated strains then can be added to the recorded values. While this method does not provide the exact range of strains the unit experienced during transportation to the site, it does provide a close approximation.

A second data acquisition system program was written to record the data during the second transportation test. The program also contained "trigger" values ($\pm$ 10 μm in the midspan bottom flange) which started the recording process. Once these "trigger" values were read, data would be recorded starting with the prior two seconds of data. If the strains dropped between $\pm$10 μm, the program would record data for two additional seconds and then pause until the "trigger" values were again reached. During this test, the program was triggered almost immediately and recorded continuously for approximately forty minutes. Figure 5.2 presents a representative sample of the range of strains in PCDT Unit 2 during the transportation to the bridge site. The data presented in this figure are an average of the recorded strains in the two midspan bottom flange gages (Fig. 5.2a), the two midspan top flange and two midspan PCDT deck gages (Fig. 5.2b), and the analytical strains previously discussed. The analytical strains added to the midspan bottom flange strains, the top flange strains and strains on the top surface of the PC deck were +110 μm, -28 μm and -49 μm, respectively.

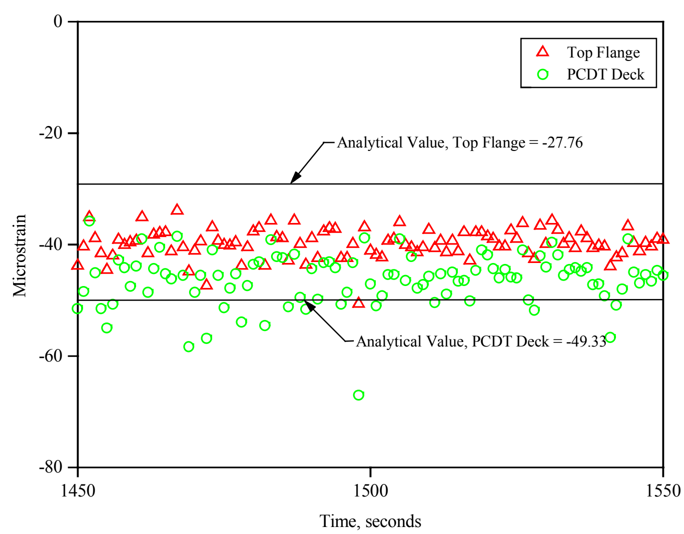
The third transport test involved monitoring PCDT Unit 2 as it was lifted from the transport vehicle onto the abutments at the bridge site. The data acquisition program written for the first phase was re-used during this test, with the mid-span bottom flange "trigger" strains changed to ± 75 μm. The strains at the mid-span bottom flange locations did not "trigger" the program; therefore, strains in the midspan bottom flange were less than ± 75 μm.

5.3 Service Load Tests

Deflections and strains at previously described locations caused by the truck loading (live load) were recorded in the three service load tests. As the concrete strains were very small in all tests, only the maximum recorded concrete strains in each service load test will be reported.



a. Bottom flange



b. Top flange and PCDT deck

Figure 5.2. Test results at mid-span for transportation of PCDT Unit 2.

As all strains and deflections measured during service load testing were quite small, only representative samples of the experimental results are presented in this chapter. The representative results include deflections and bottom flange strain data for each service load test. Even though all experimental data are small, one can observe the structural response and behavior of the bridge to the truck loading at the various positions. Based upon the location of the gages (see Figs. 4.4 and 4.9), data from Truck Positions C3 and C5 will be shown for each service load test involving one truck. Data from Truck Positions C1 and C5 and C2 and C4 are presented when two trucks were on the bridge in the second and third service load tests.

5.3.1. Service Load Test without CIP Deck (First Service Load Test)

The first service load test was performed when all four PCDT units were correctly positioned on the abutments, intermediate diaphragms connected and PCDT connectors welded. The five transverse positions of the truck on the bridge used in the first service load test are shown in Fig. 5.3. This figure corresponds to the live load locations shown previously in Fig. 4.7.

The maximum induced concrete strain during the first service load test was -24 μm (-1200 psi) on the top surface of the precast units when the truck was at position C1. The deflection and bottom flange strain data from Truck Position C3 are shown in Fig. 5.4. Displacement transducers at Beam Position 3 and 4 malfunctioned during this test; expected displacement points have been shown in this data plot. The maximum deflection recorded was 0.115 in. downward with a bottom flange strain value of 48 μm (1.4 ksi). Deflection

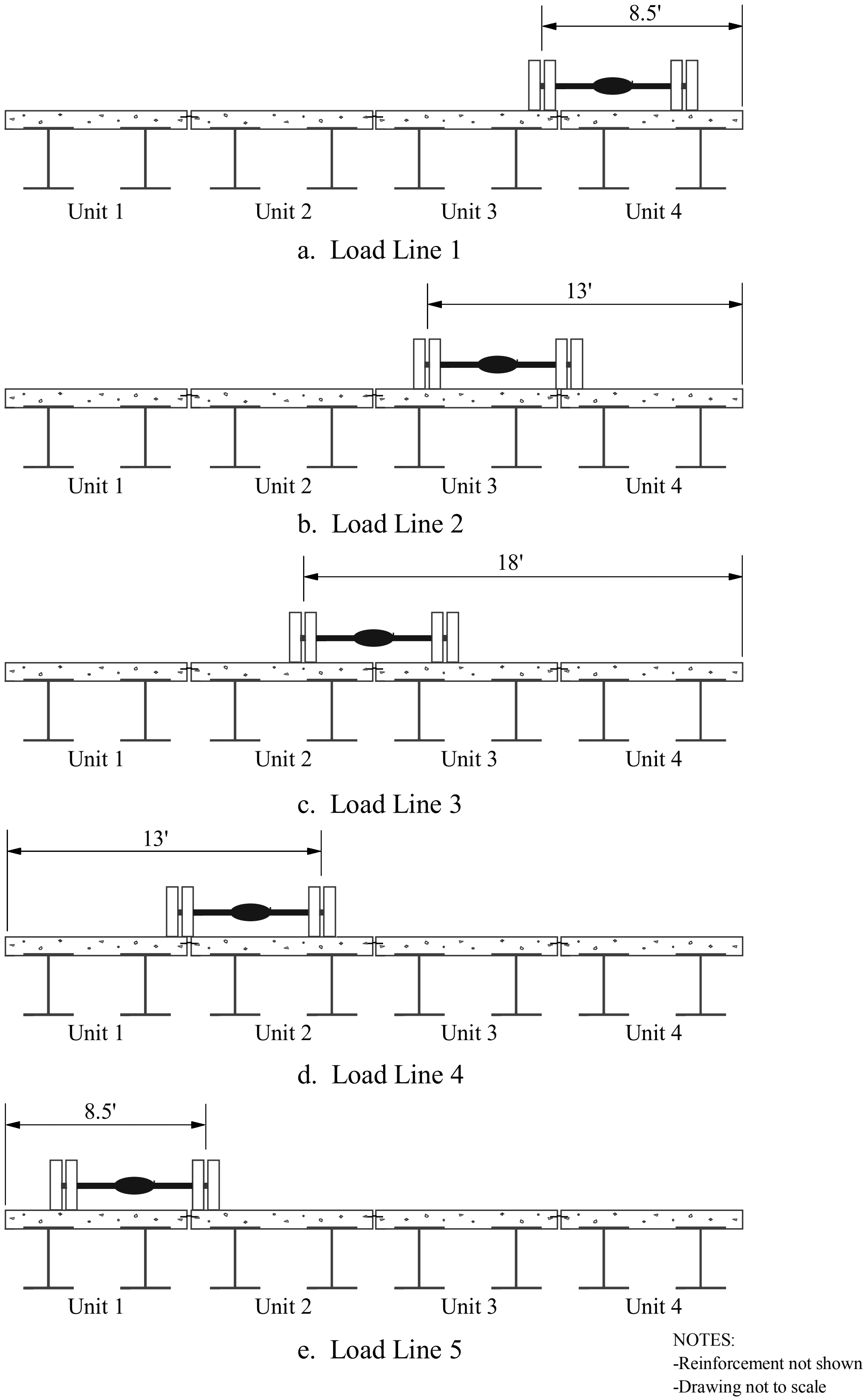
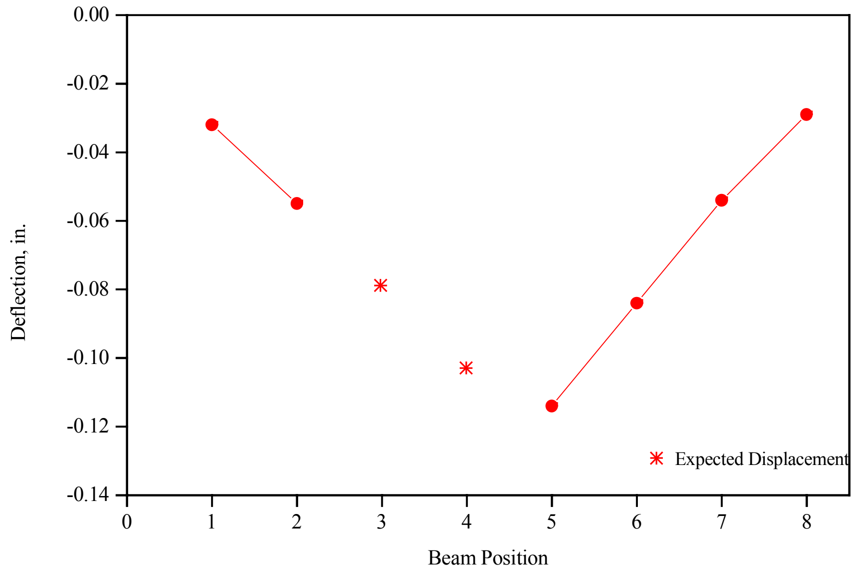
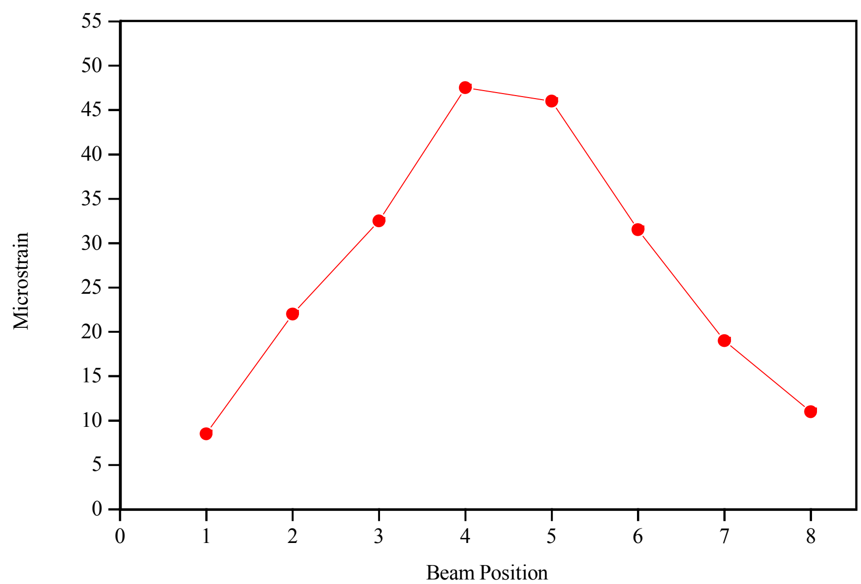


Figure 5.3. Truck load line locations, first service load test, Truck No. 23.

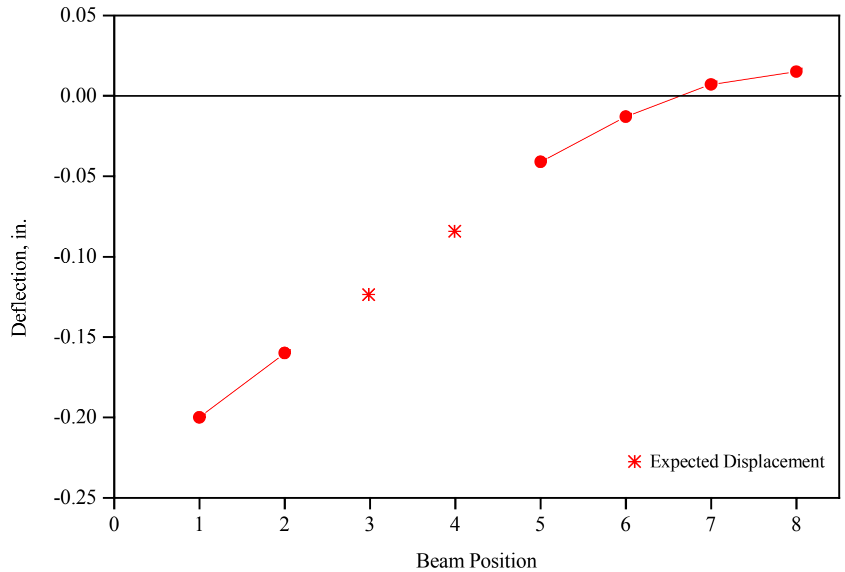


a. Deflection

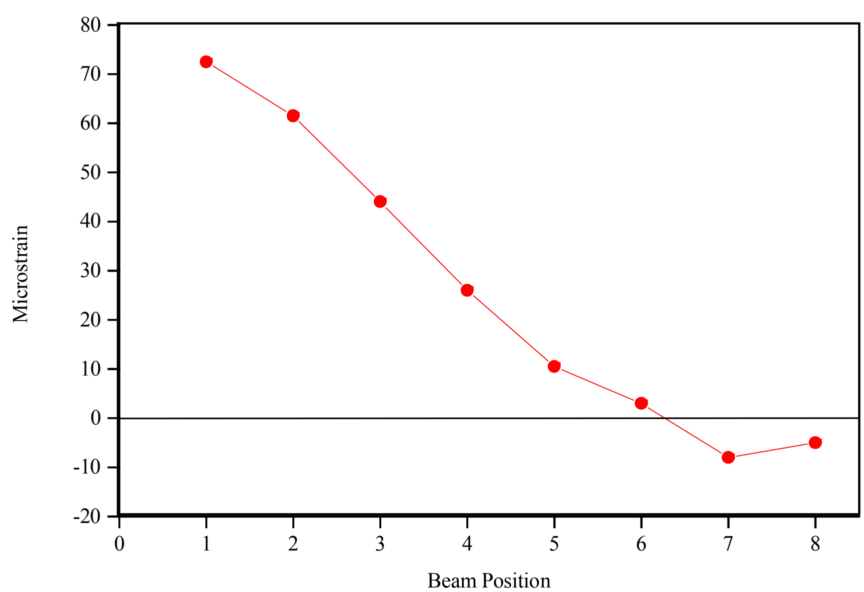


b. Bottom Flange Strains

Figure 5.4. Test results at mid-span, first service load test, truck at Position C3 .



a. Deflection



b. Bottom Flange Strains

Figure 5.5. Test results at mid-span, first service load test, truck at Position C5.

and bottom flange strain data occurring when the load was at Truck Position C5 are shown in Fig 5.5. The maximum deflection was 0.20 in. downward with a bottom flange strain reading of 73 μm (2.1 ksi). Note that Beam 8 experienced an upward movement of 0.02 in. and a compressive bottom flange strain of -9 μm (-0.3 ksi).

5.3.2. Service Load Test with CIP Deck (Second Service Load Test)

The second service load test was performed once the CIP deck was placed and cured without the bridge rail in-place. The transverse positions of the truck on the bridge during the second service load testing are shown in Fig. 5.6; note these transverse positions are the same as those used in the first service load test. This figure corresponds to the live load locations shown in Fig. 4.11.

The maximum induced live load concrete strains in the bridge during the second service load test was -22 μm (-110 psi) in the PC slab with the load at Truck Position C5 and -40 μm (-200 psi) in the CIP deck with the load at Truck Position C1. This means the induced stress at both locations was less than 200 psi. The displacement and bottom flange strain data for Truck Position C3 are shown in Fig. 5.7. The displacement transducer on Beam 2 malfunctioned in this test. The maximum recorded deflection with the load at Truck Position C3 was 0.11 in. downward with a maximum bottom flange strain value of 60 μm (1.7 ksi). Displacement and bottom flange strain data with the load at Truck Position C5 are shown in Fig. 5.8; in this test the displacement transducer on Beam 4 malfunctioned. The maximum deflection with the load at Truck Position C5 was 0.21 in. downward with a corresponding bottom flange strain value of 109 μm (3.2 ksi).

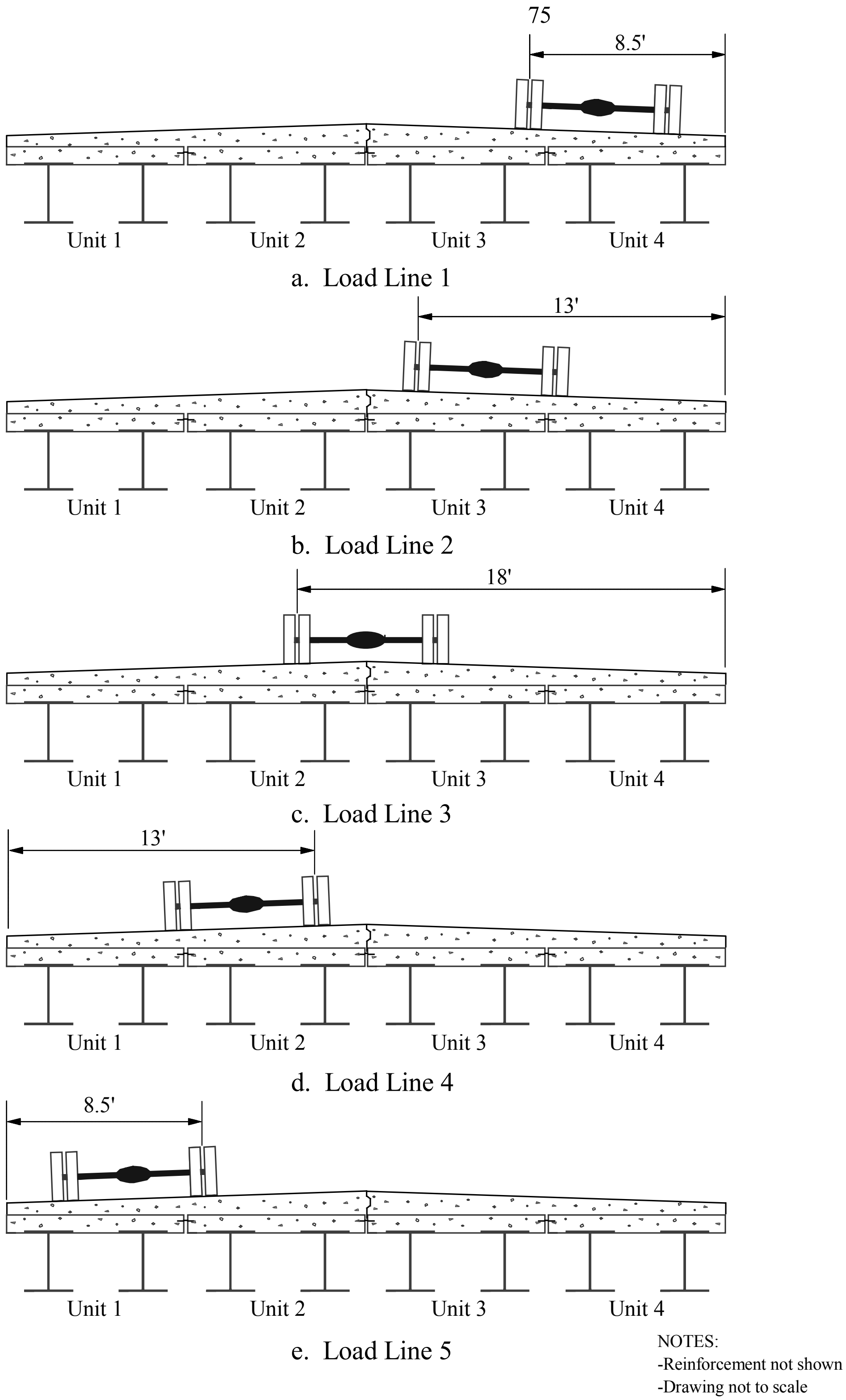
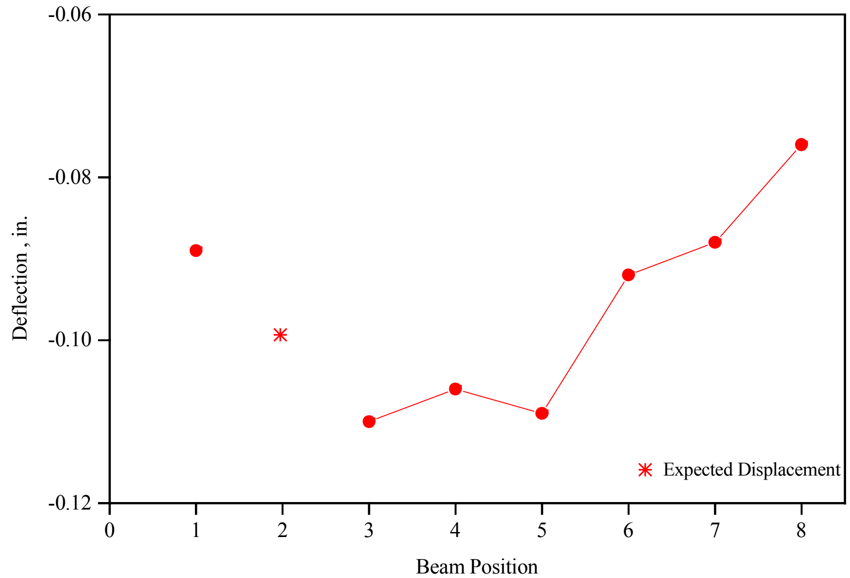
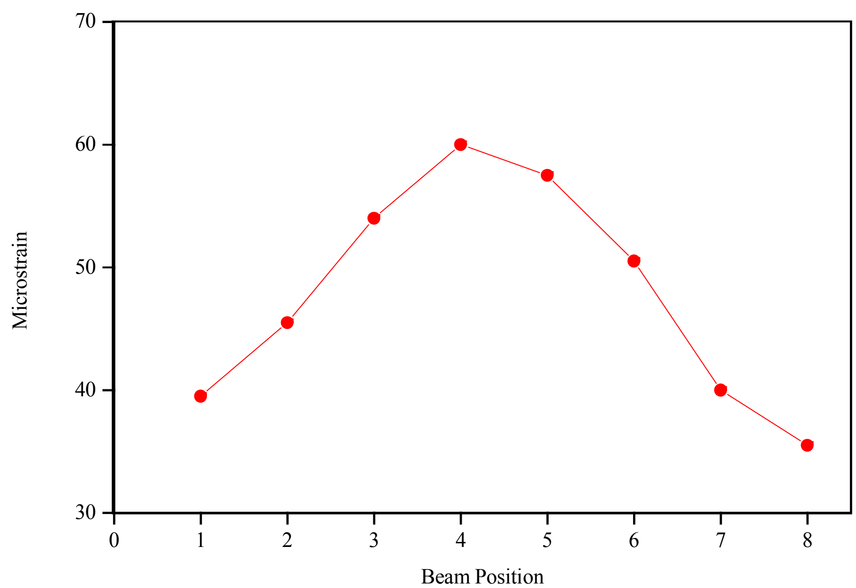


Figure 5.6. Truck load line locations, second and third service load test, Truck No. 23.

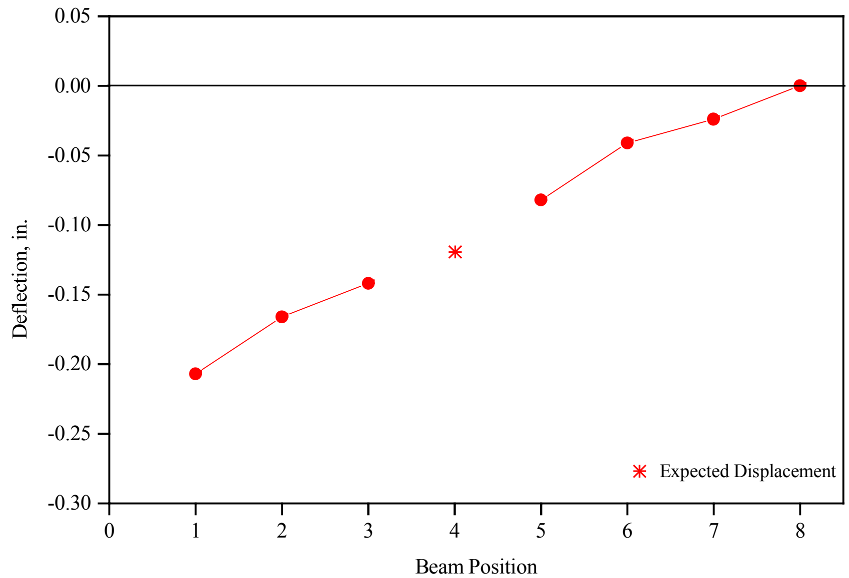


a. Deflection

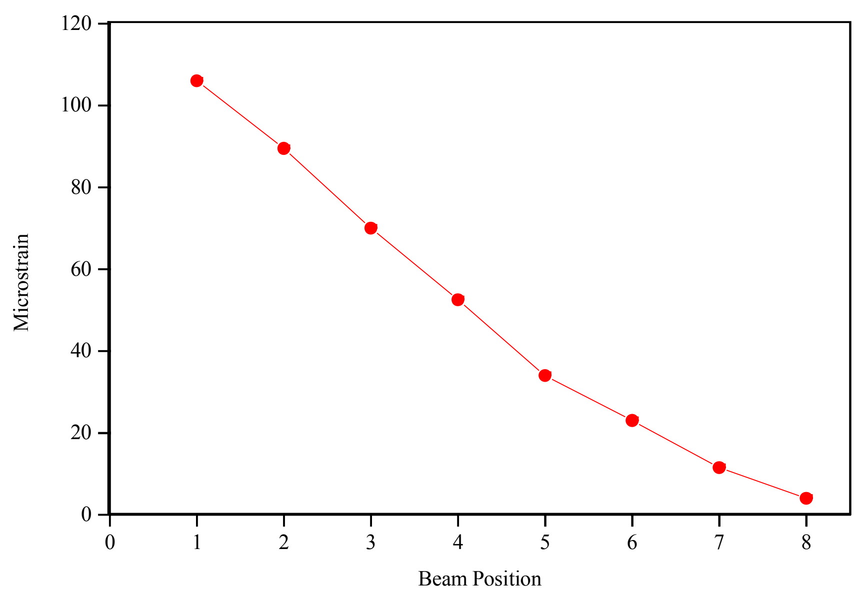


b. Bottom Flange Strains

Figure 5.7. Test results at mid-span, second service load test, truck at Position C3.



a. Deflection



b. Bottom Flange Strains

Figure 5.8. Test results at mid-span, second service load test, Truck at Position C5.

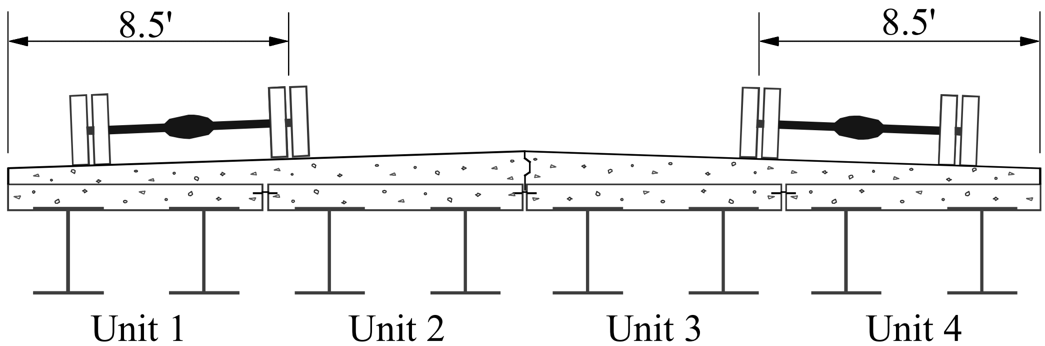
After completing the one truck test, two additional tests were performed in which two tandem axle trucks were placed on the bridge simultaneously. The transverse location of the trucks is shown in Fig. 5.9 which corresponds to the live load locations shown in Fig. 4.13.

The maximum induced live load concrete strains in the bridge during the load testing with two trucks was -23 µm (-115 psi) in the PC slab at Truck Position D1-D5 and -40 µm (-200 psi) in the CIP deck at Truck Positions C2 and C4. The displacement and bottom flange strains for Truck Positions C1 and C5 are shown in Fig. 5.10. Displacement transducer problems continued during this test with those at Positions 1, 2, 3, and 4 not working. Based on the data plots of Fig. 5.10, the maximum deflection was 0.21 in. downward with a maximum bottom flange strain value of 115 µm (3.3 ksi). Displacement and bottom flange strain data for Truck Positions C2 and C4 are shown in Fig. 5.11. The maximum recorded deflection was 0.22 in. downward with a bottom flange strain value of 109 µm (3.2 ksi).

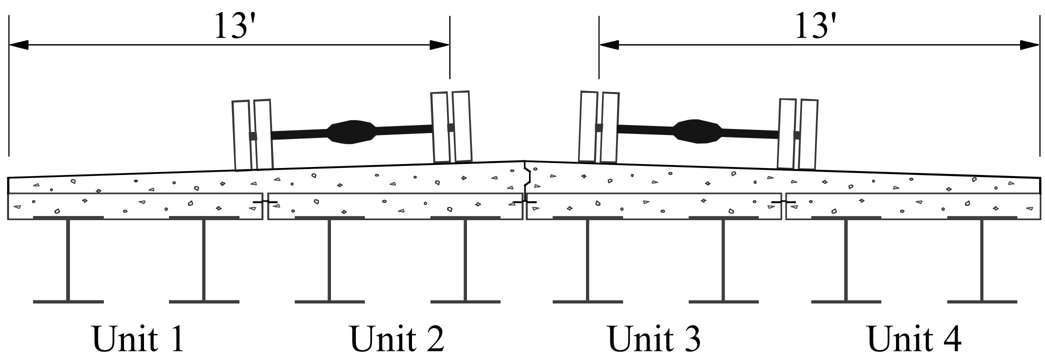
5.3.3 Service Load Test with CIP Deck and Bridge Rail (Third Service Load Test)

The third service load test was performed after the bridge rails (and approach guardrails) had been placed. The transverse load truck positions used in this test are the same as those used in Service Load Test 2 (see Fig. 5.6).

Figure 5.12 illustrates the experimental and design lateral live load distribution characteristics at midspan of the PCDT bridge in various states of construction under the loading conditions described previously. The lateral load distribution is shown as a percent



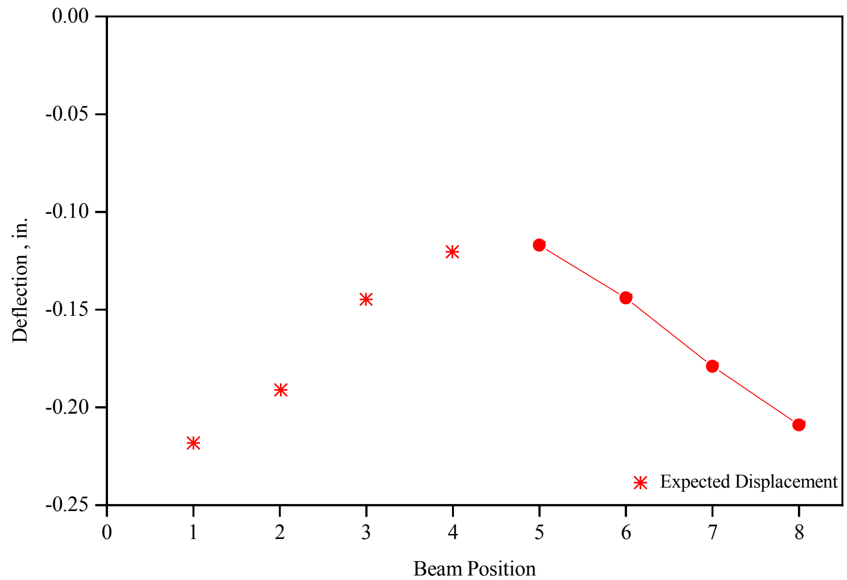
a. Load Line 1 and Load Line 5



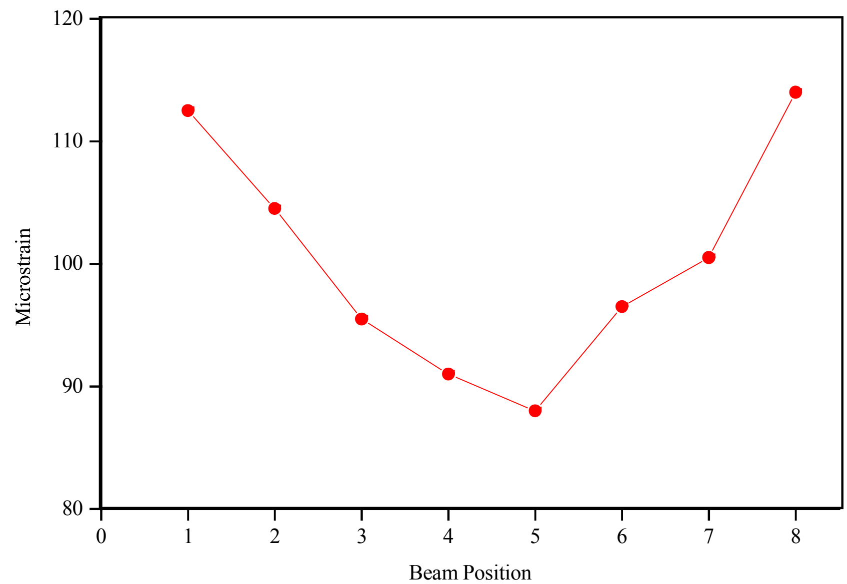
b. Load Line 2 and Load Line 4

NOTES:
-Reinforcement not shown
-Drawing not to scale

Figure 5.9. Truck load line locations, second and third service load test, Truck No. 23 and 35.

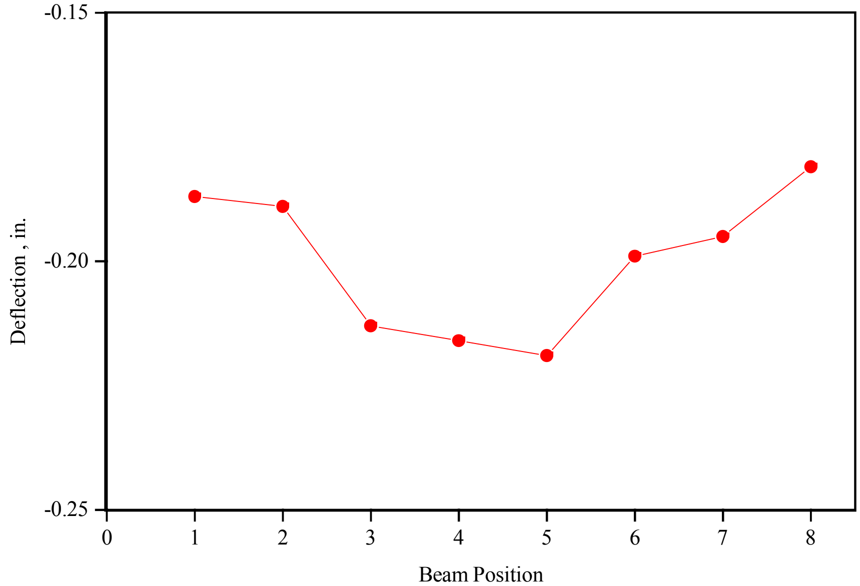


a. Deflection

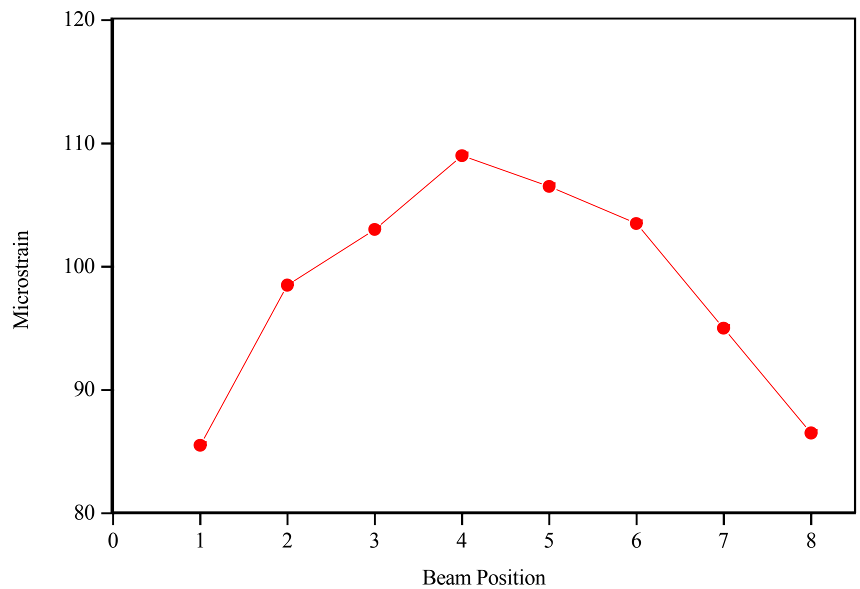


b. Bottom Flange Strains

Figure 5.10. Test results at mid-span, second service load test, trucks at Positions C1 and C5.

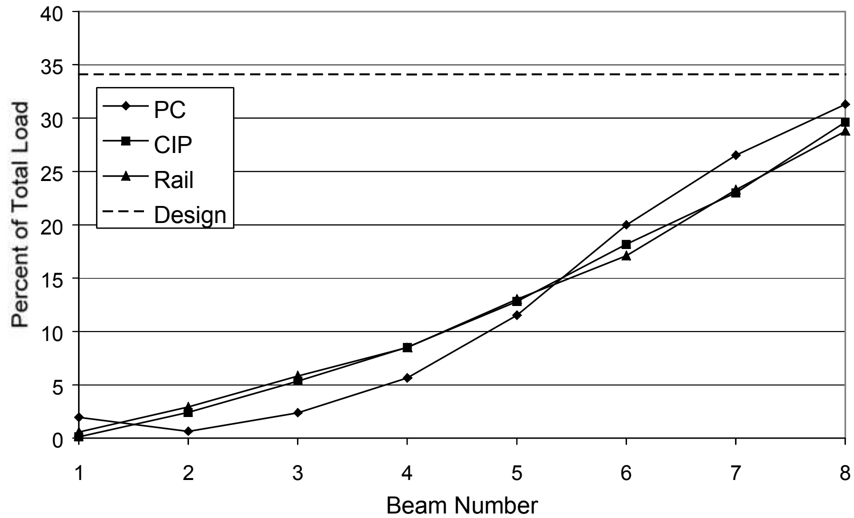


a. Deflection

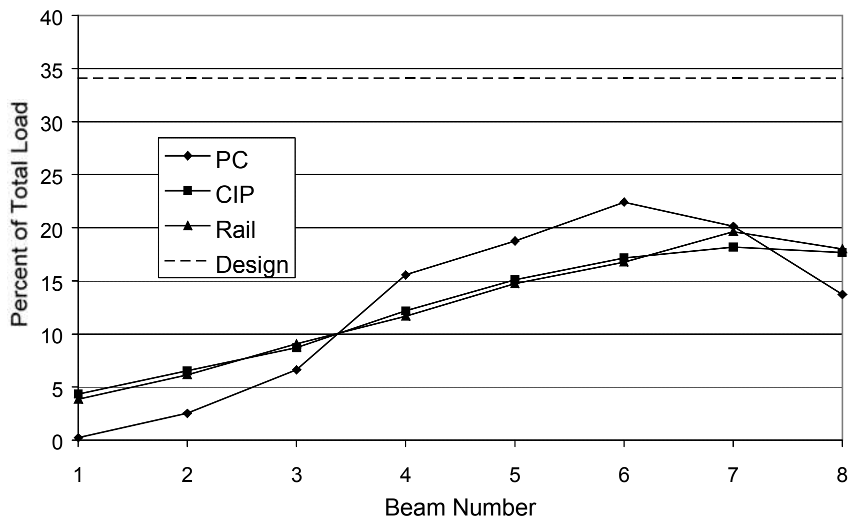


b. Bottom Flange Strains

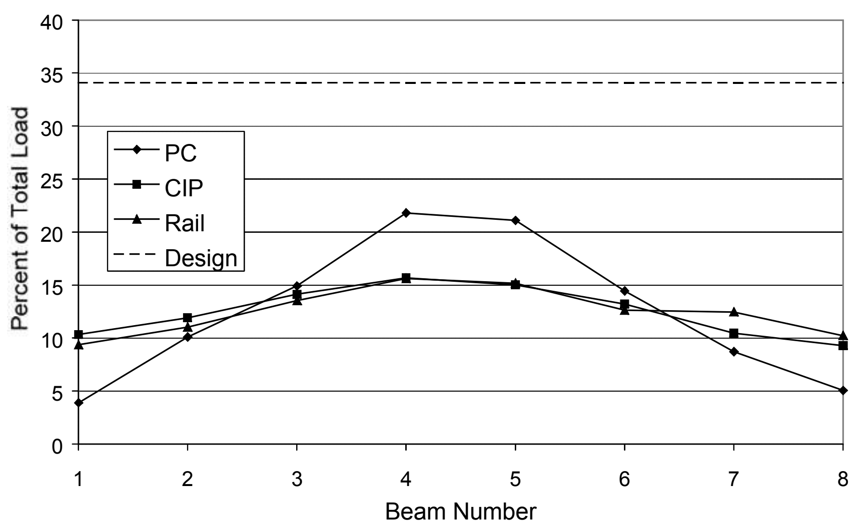
Figure 5.11. Test results at mid-span, second service load test, trucks at Positions C2 and C4.



a. Load line 1

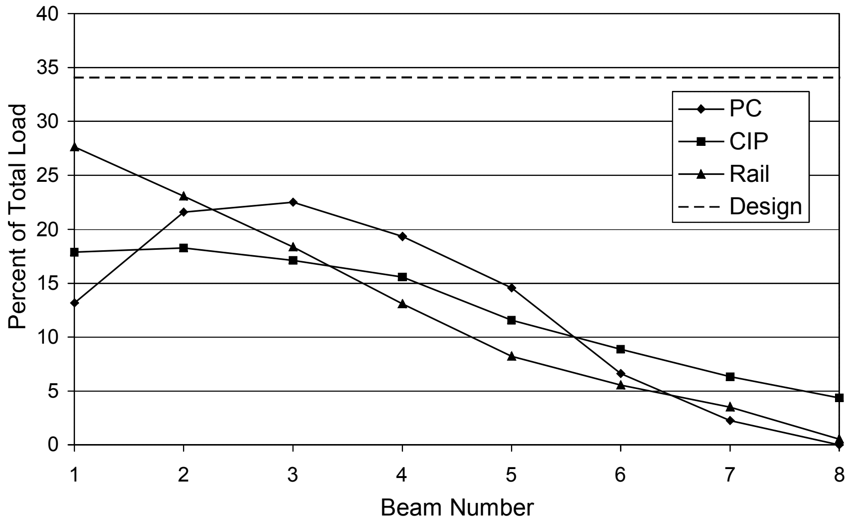


b. Load line 2

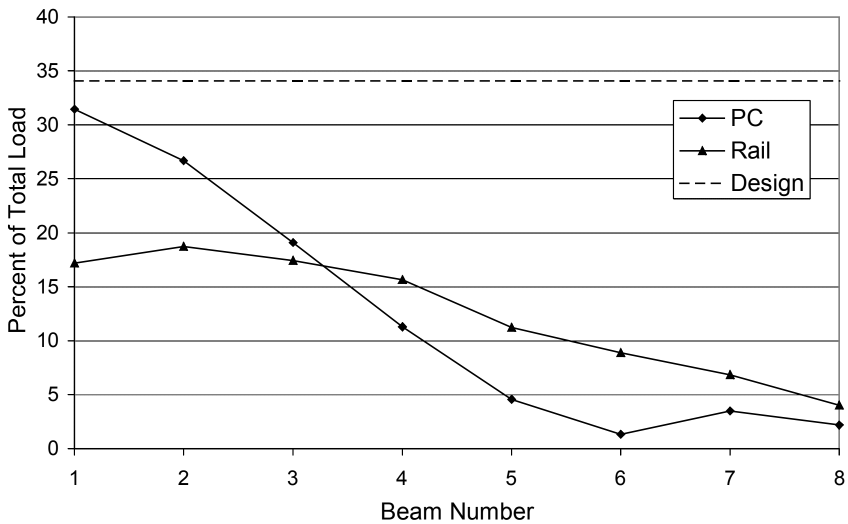


c. Load line 3

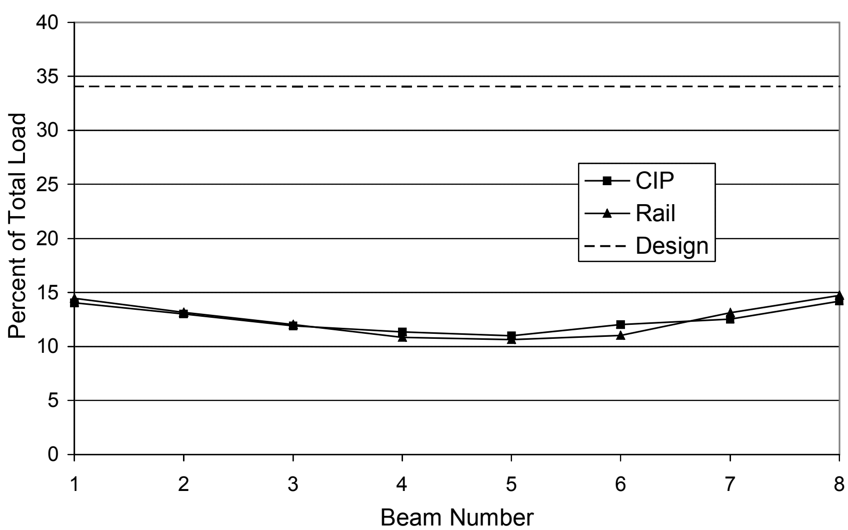
Figure 5.12. Lateral load distribution at midspan – load at Section C.



d. Load line 4

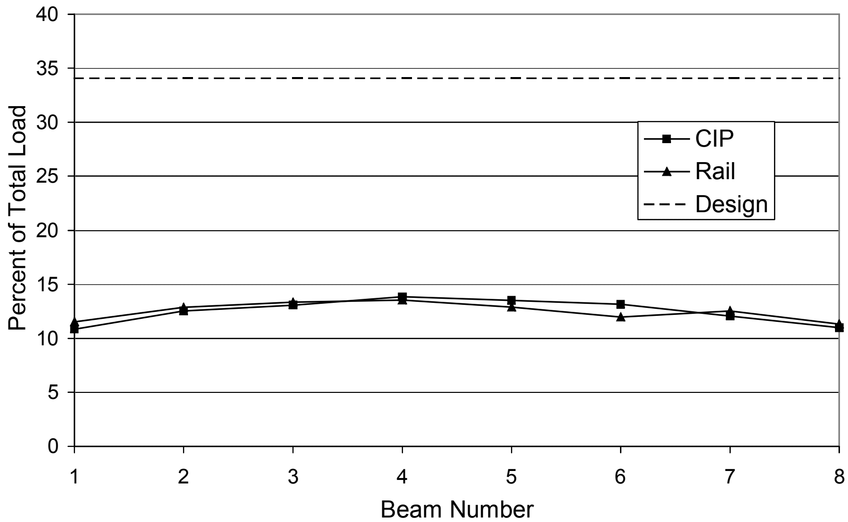


e. Load line 5



f. Load lines 1 and 5

Fig. 5.12. Continued.



g. Load lines 2 and 4

Figure 5.12. Continued.

of the total live load carried by each stringer. Since each stringer has the same material and section properties this percentage can be calculated using the following equation.

$$\text{Percent of Load}_i = \frac{\varepsilon_i}{\sum_{i=1}^{8} \varepsilon_i} \times 100 \qquad (1)$$

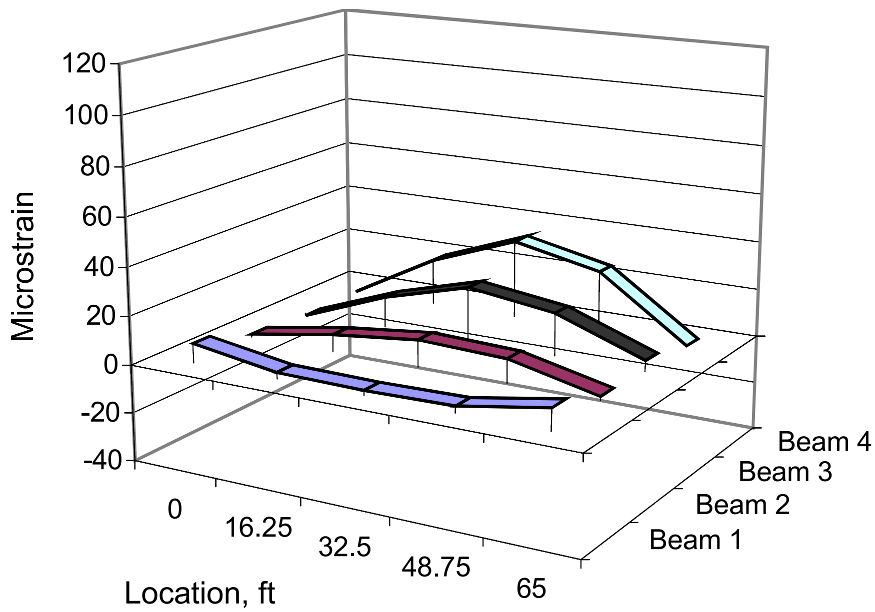
where:

ε_i = bottom flange, longitudinal strain in *i*th beam at midspan.

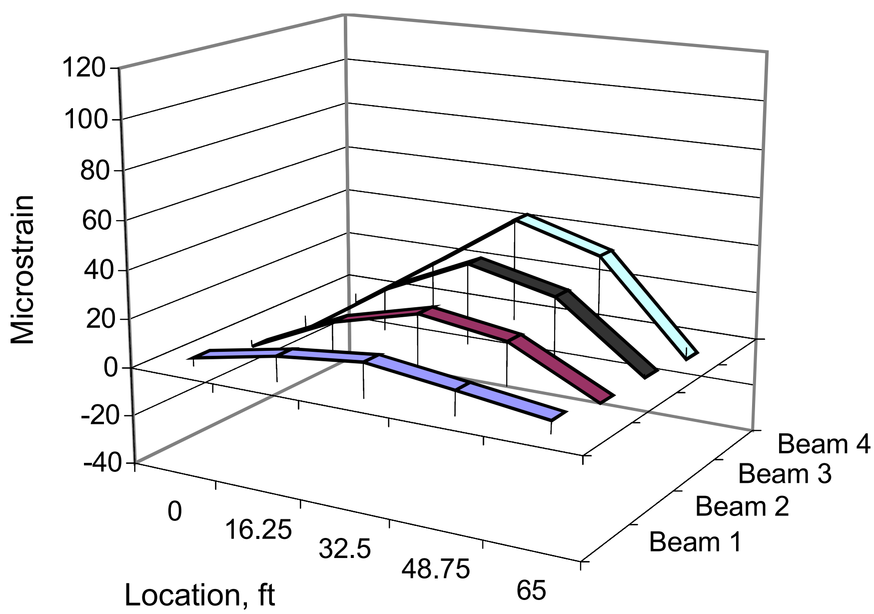
From these figures it can be seen that regardless of placement of the load, no individual stringer carried more than 35 percent of the total load. Additionally, in the completed bridge (i.e., CIP and bridge rail tests) all stringers carried less than 30 percent of the total live load. Also note in each of these figures, the design lateral load distribution factor of 34 percent is shown. This clearly illustrates that no stringer in the completed bridge exceeded the design value. For comparative purposes, if all beams were to carry equal amounts of the live load, since there are eight beams, the distribution would correspond to a 12.5 percent factor. It can also be seen from this figure that the lateral load distribution characteristics of the PCDT bridge is significantly influenced by the addition of the CIP concrete. This verifies findings in the first phase of this investigation [4].

Although the influence of the addition of the CIP concrete is clear in these figures, the influence of the PC connectors is not. If one were to consider the lateral load distribution in the PCDT before installation of the PC connectors, it seems logical that if the live load were placed completely on a single unit (e.g., line load 2) the load would be distributed approximately equally to the two stringers comprising that unit (i.e., 50 percent on each stringer). One can infer then, that the addition of the PC connectors reduced the maximum percent of load on a single stringer for line load 2 from an estimated 50 percent to approximately 23 percent. Note in the figure that many times the experimental load distribution in Beams 6 and 7 are inconsistent with the behavior at other beams. This is most likely due to difficulties in installing the PC connectors between Units 3 and 4.

Since the behavior of the completed bridge is of primary importance, the remaining discussion will focus exclusively on tests conducted after the guard rail had been installed. Figure 5.13 shows the longitudinal bottom flange strains along the length of Beams 1 through 4. From the various parts of this figure it can be seen that independent of the lateral or longitudinal position of the load, the longitudinal distribution of load is, in general, similar for each stringer. This indicates that the PC connector plus CIP concrete is effective in distributing the load laterally. Additionally, although the end supports were designed to act as simple supports, the bridge does exhibit some unintended rotational end restraint

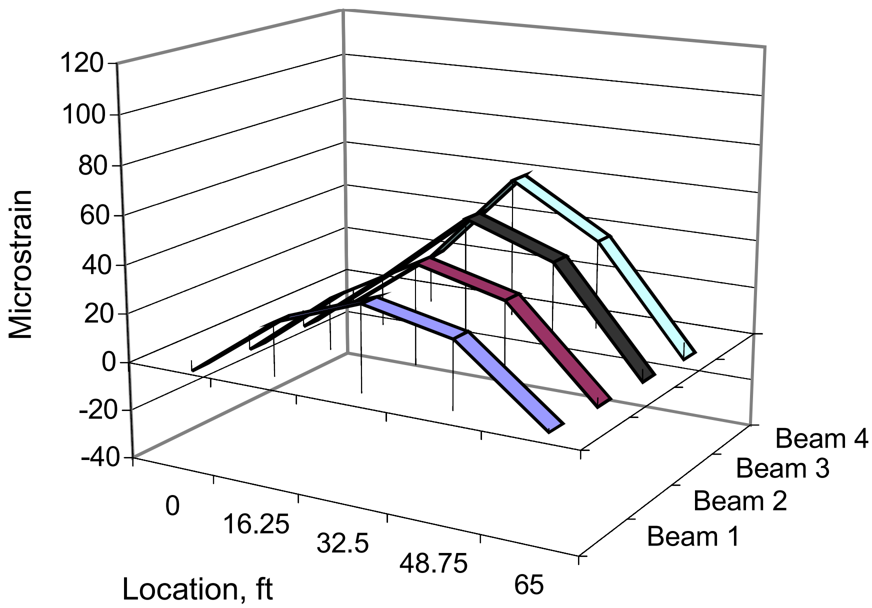


a. Load line 1

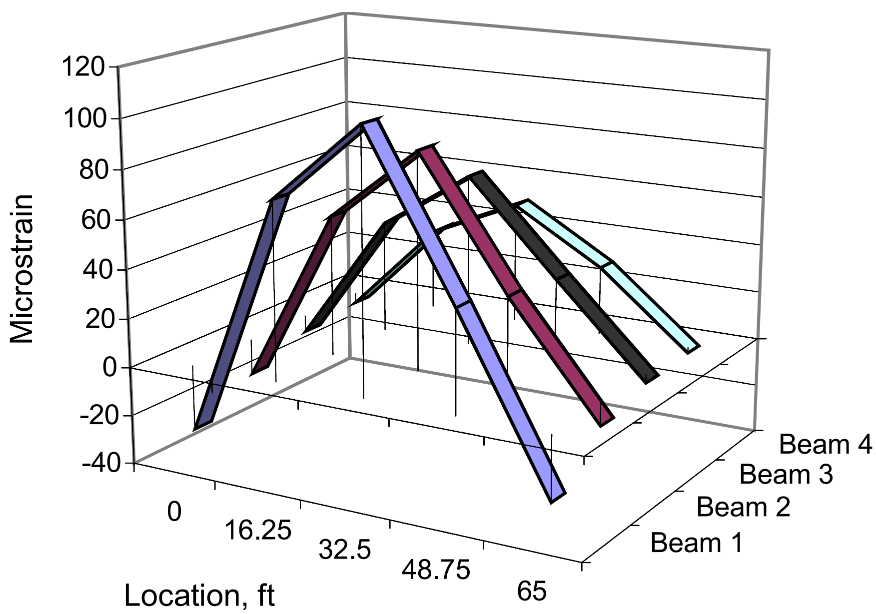


b. Load line 2

Figure 5.13. Two dimensional strain distribution – load at Section C.

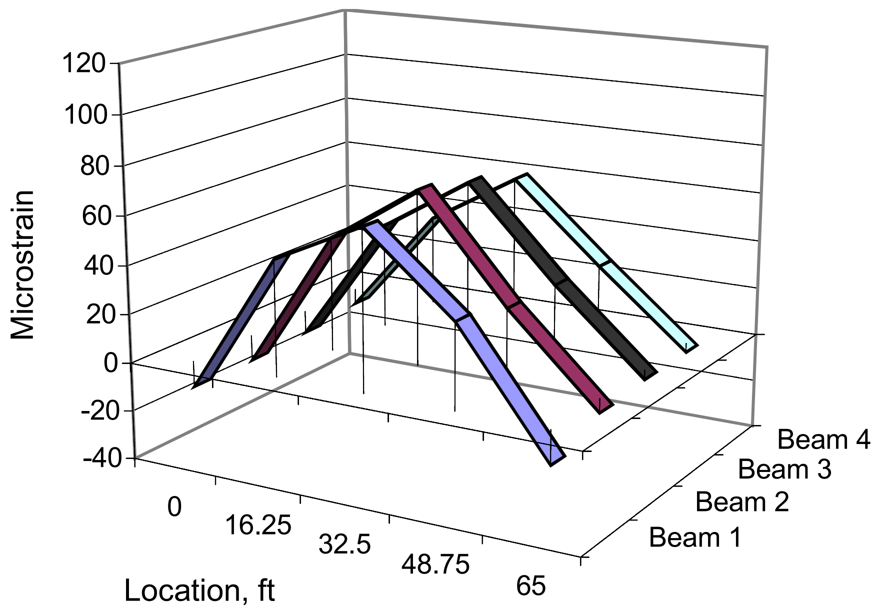


c. Load line 3

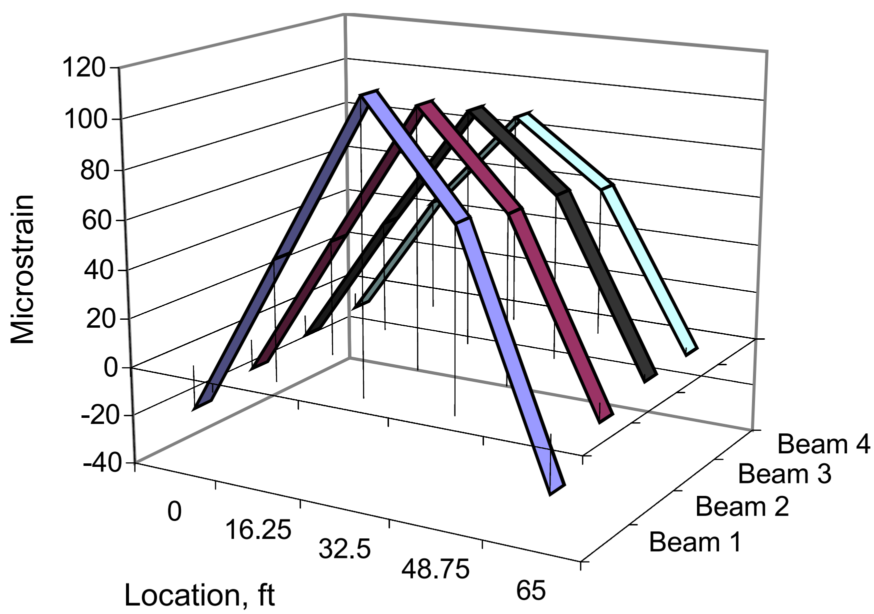


d. Load line 4

Figure 5.13. Continued.

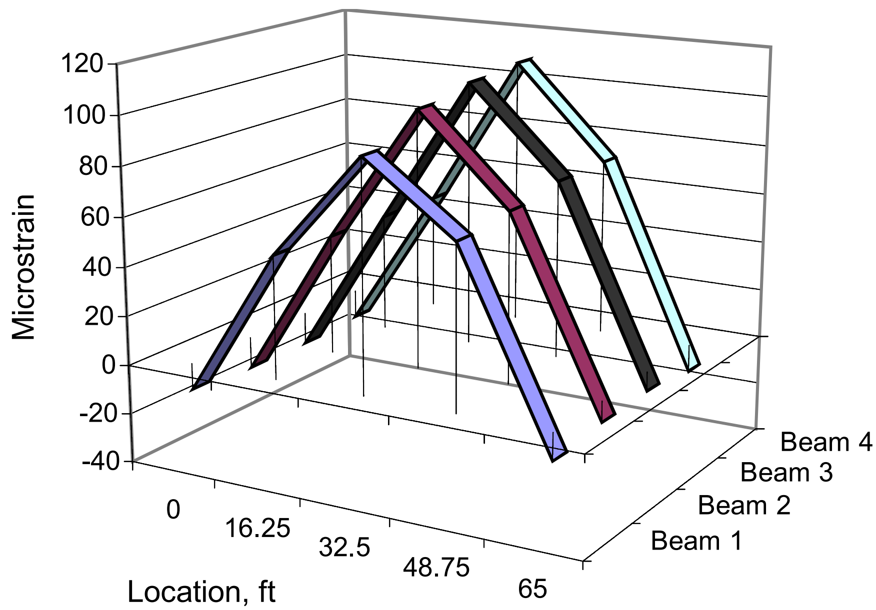


e. Load line 5



f. Load lines 1 and 5

Figure 5.13. Continued.



g. Load lines 2 and 4

Figure 5.13. Continued.

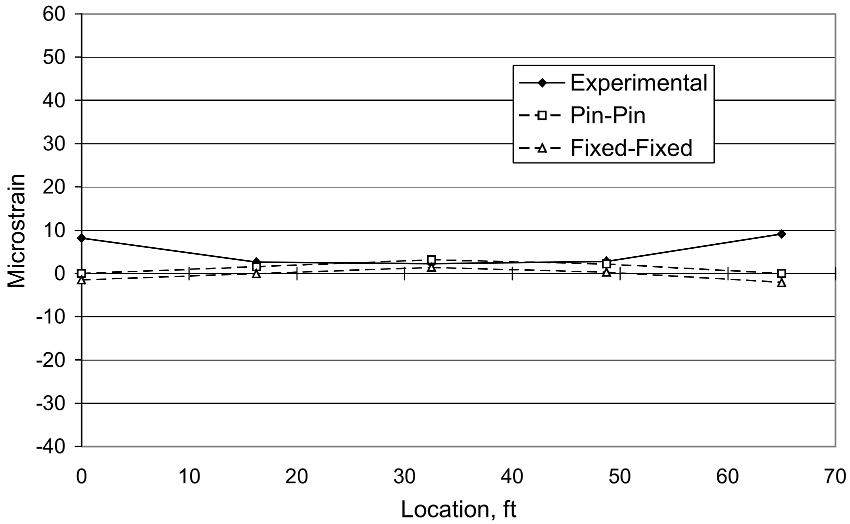
(indicated by the negative strain data). This end restraint is most likely the result of the type of end diaphragm that was utilized in the bridge.

Table 5.2 summarizes the measured strains in the south guardrail for the tests shown in Fig. 5.13. For reference, note that the south guardrail is nearest to Beam 8. These data indicate that the guardrail did resist a small amount of the live load.

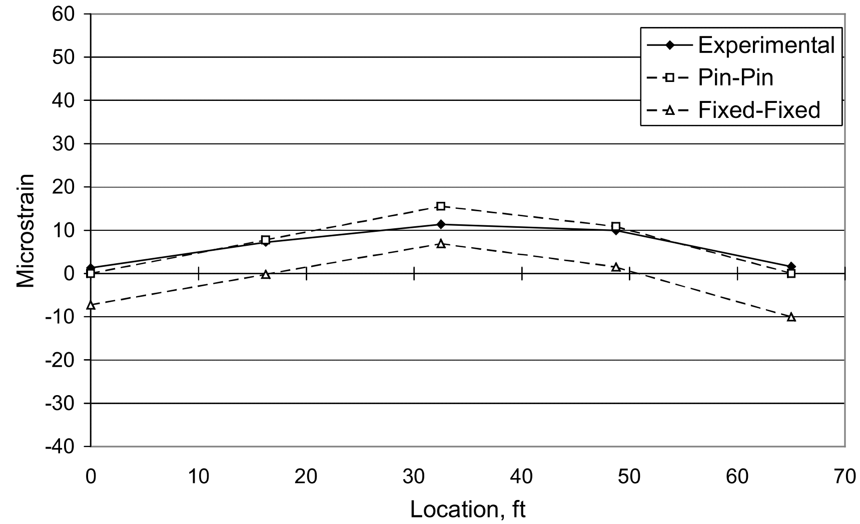
Table 5.2. South guardrail strain.

	South Guardrail Strain (μm strain)		
Load Line	Load at B	Load at C	Load at D
1	-43	-51	-23
2	-30	-33	-20
3	-17	-18	-10
4	5	6	16
5	0	2	9
1 and 5	-46	-47	-12
2 and 4	-32	-39	-29

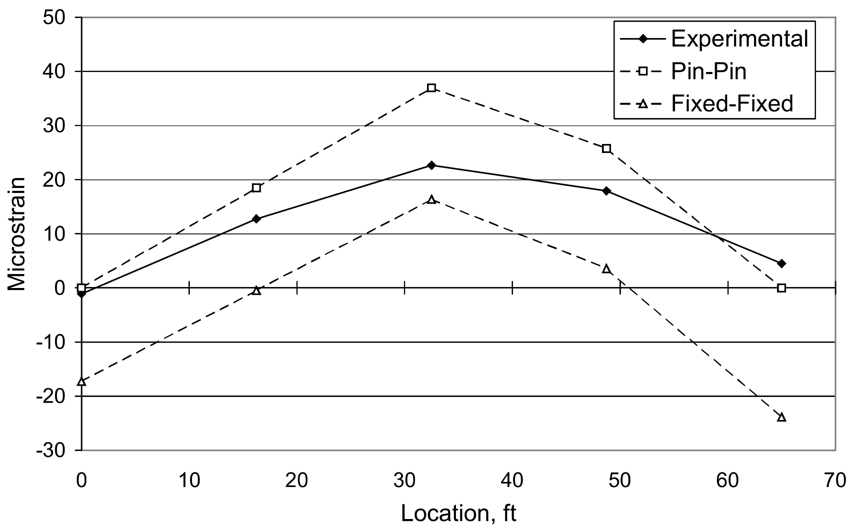
As was mentioned previously, the completed bridge exhibited some unintended end restraint that is most likely attributable to the end diaphragm details. To investigate the degree of end restraint resulting from the end diaphragms, analytical models were developed to predict the idealized behavior of the bridge. Although there are many different methods for developing analytical models, it was decided to use the experimentally determined lateral load distribution factors discussed previously in combination with classic beam theory to model the overall bridge behavior. In this model, the lateral load distribution data presented earlier were used to distribute a percentage of the truck point loads to each stringer. Then, using classic beam theory and elementary mechanics of materials, the bottom flange strains were predicted. Through this approach, the analytical models and experimental data can easily be compared. Figures 5.14 - 5.18 illustrate the results of this analysis. Each figure shows the experimental and analytical results along the length of Beams 1 through 4 for a particular load case. The analytical model results are indicated by the "pin-pin" condition, in which both ends of the stringers were assumed to be simply supported, and the "fixed-fixed" condition, in which both ends of the stringers were assumed to be fixed against rotation and translation. As shown, the behavior of the PCDT bridge generally lies between the two analytical solutions. This indicates that the PCDT bridge, as it was constructed for the demonstration project, is responding to live load with some unintended rotational end restraint. In addition, the degree of end restraint appears to be a function of the lateral position of the load. In fact, when the load is some distance from a particular stringer, that stringer tends to behave fairly closely to the "pin-pin" condition. However, as the load



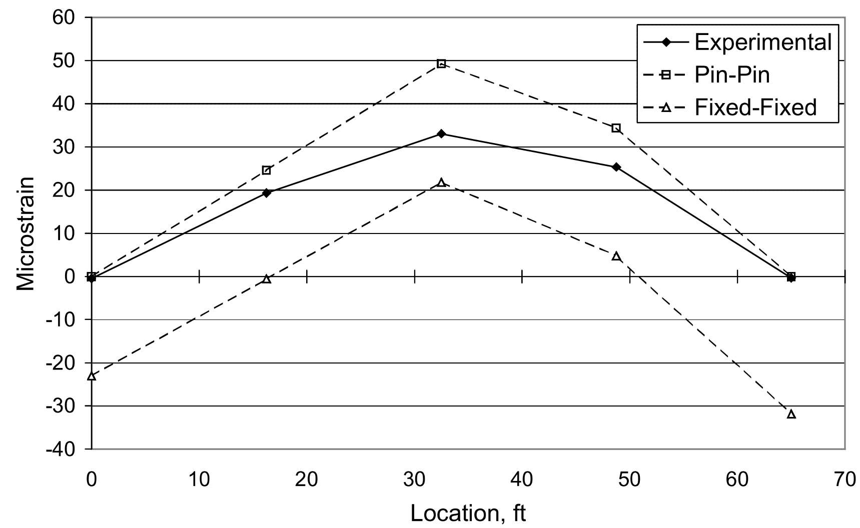
a. Beam 1



b. Beam 2

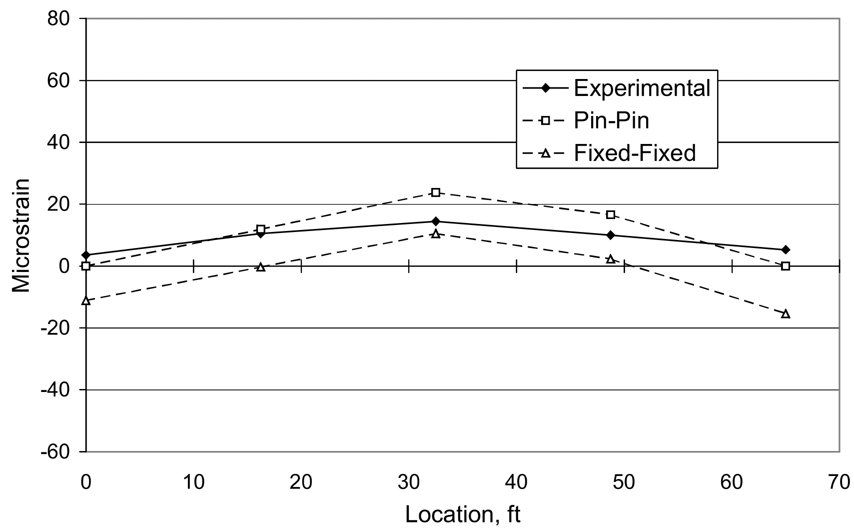


c. Beam 3

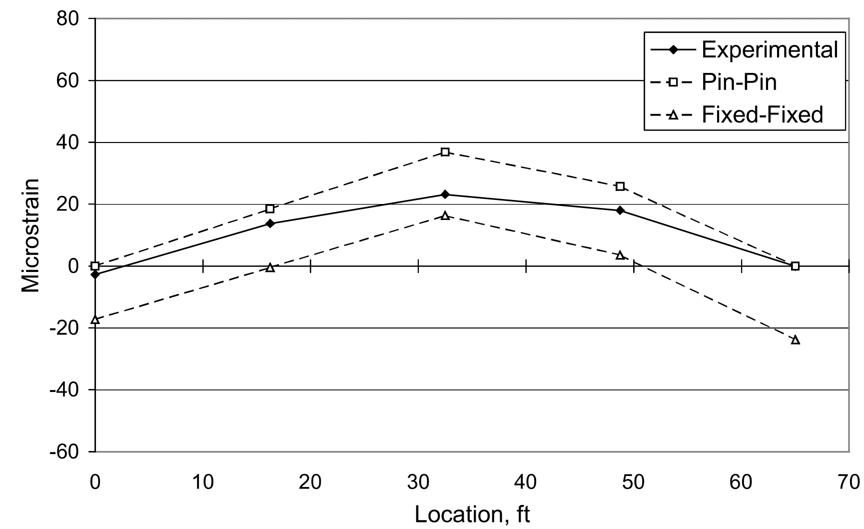


d. Beam 4

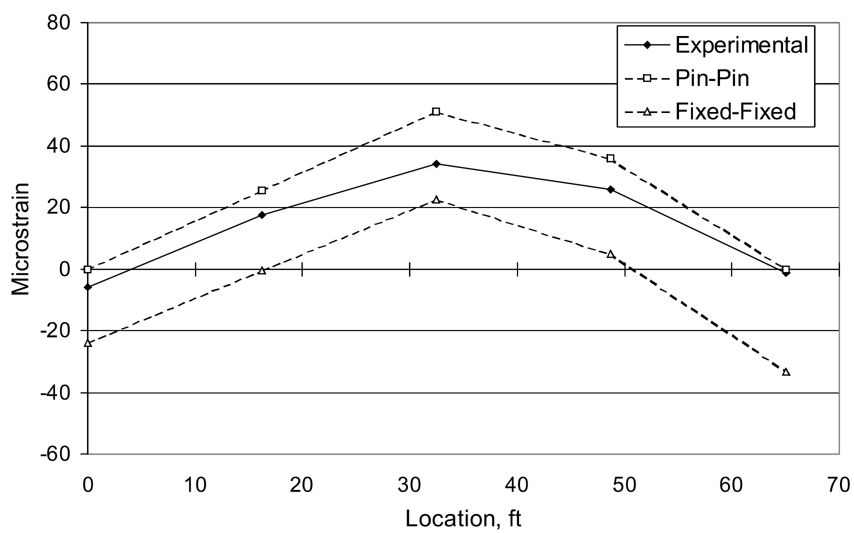
Figure 5.14. Experimental and analytical model comparison – load line 1, load at C



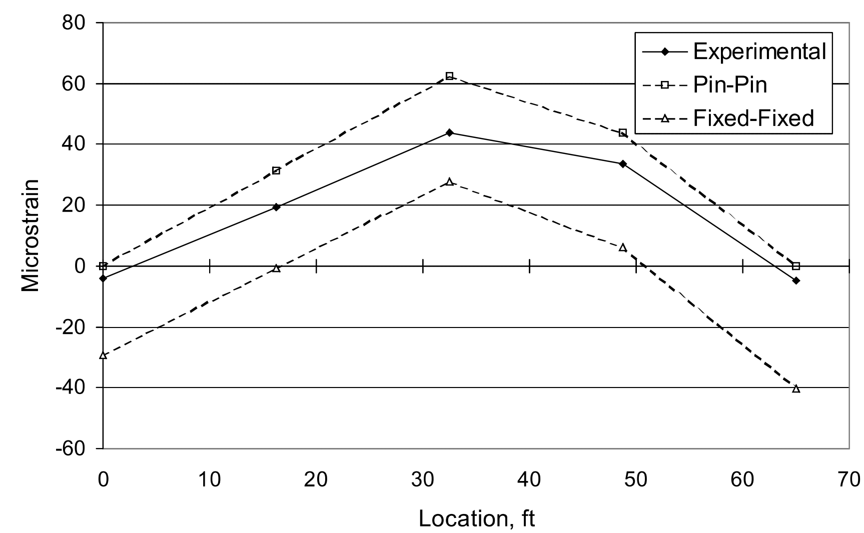
a. Beam 1



b. Beam 2

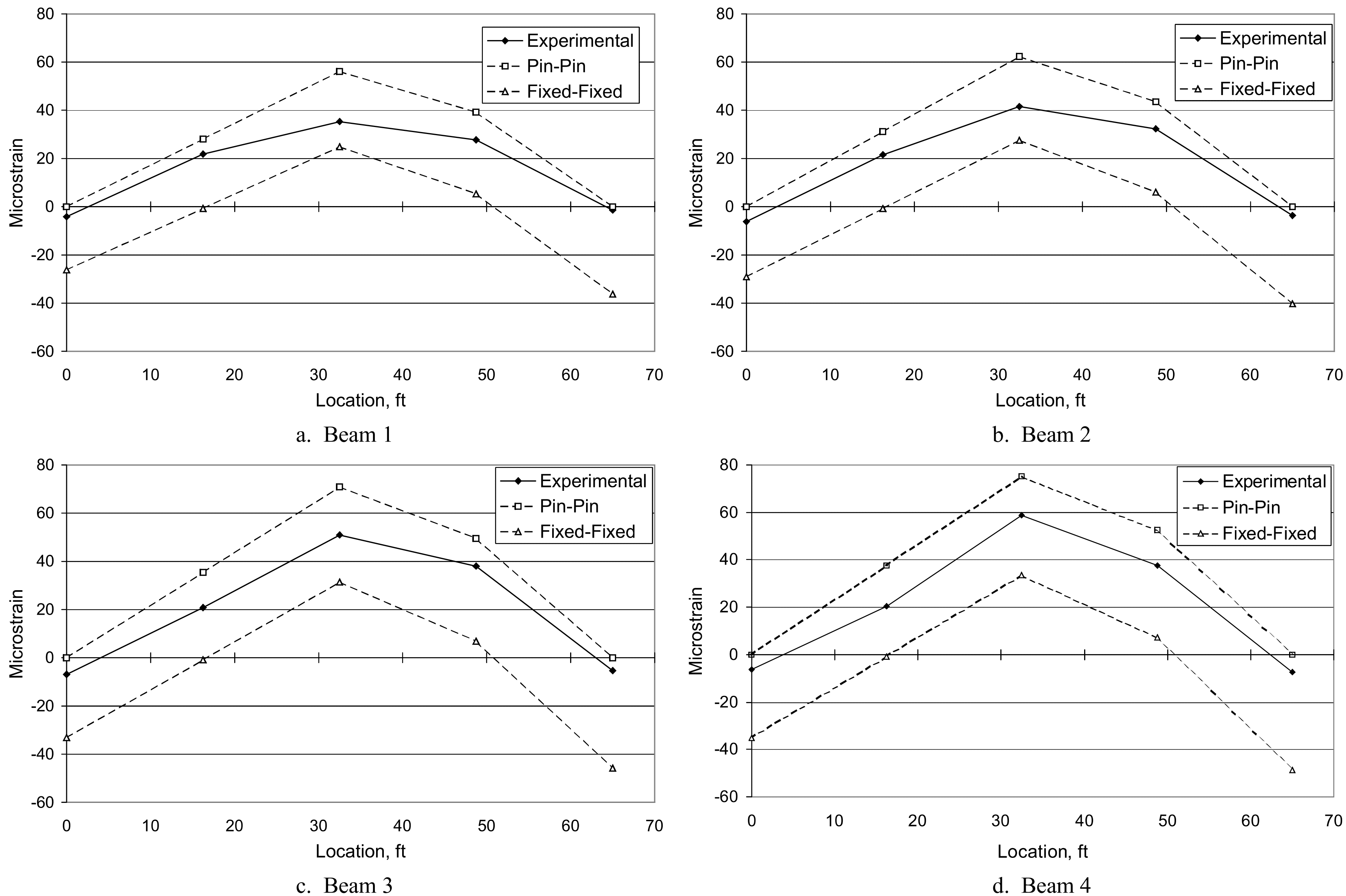


c. Beam 3



d. Beam 4

Figure 5.15. Experimental and analytical model comparison – load line 2, load at C.



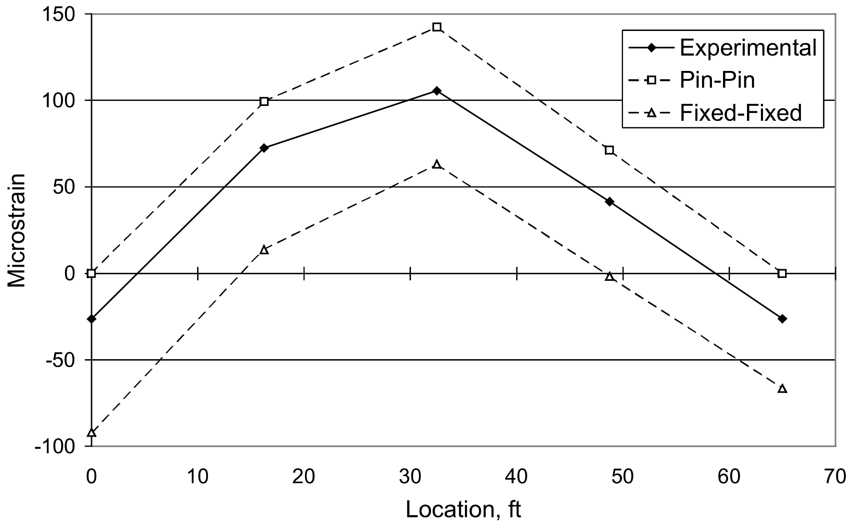
a. Beam 1

b. Beam 2

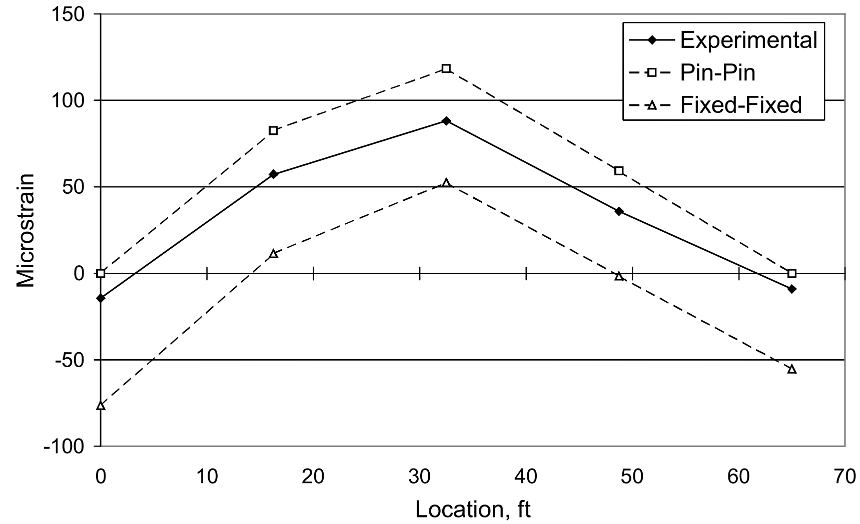
c. Beam 3

d. Beam 4

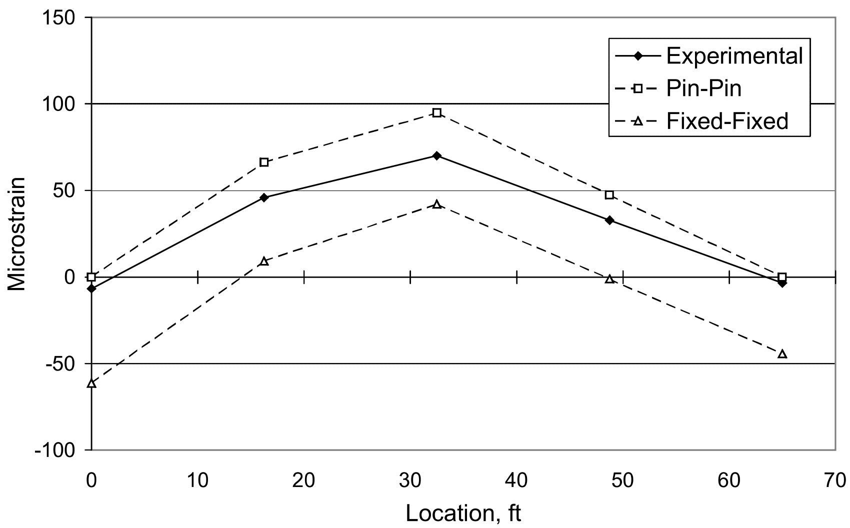
Figure 5.16. Experimental and analytical model comparison – load line 3, load at C.



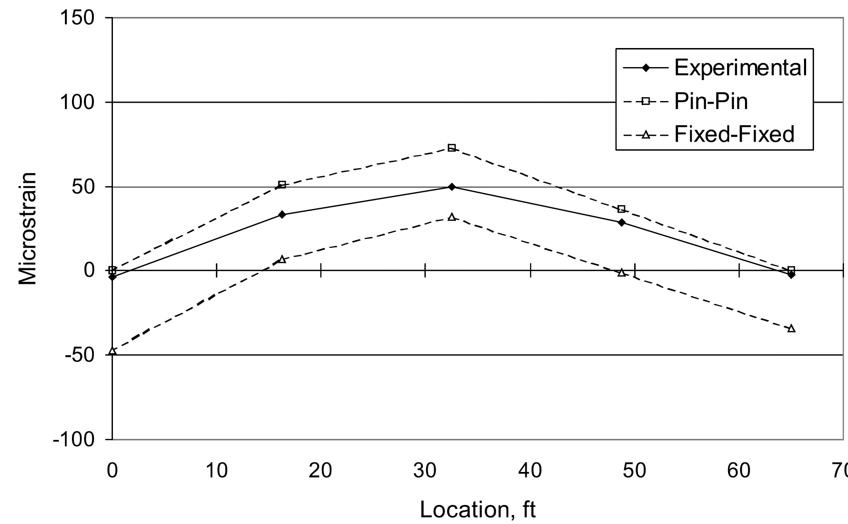
a. Beam 1



b. Beam 2

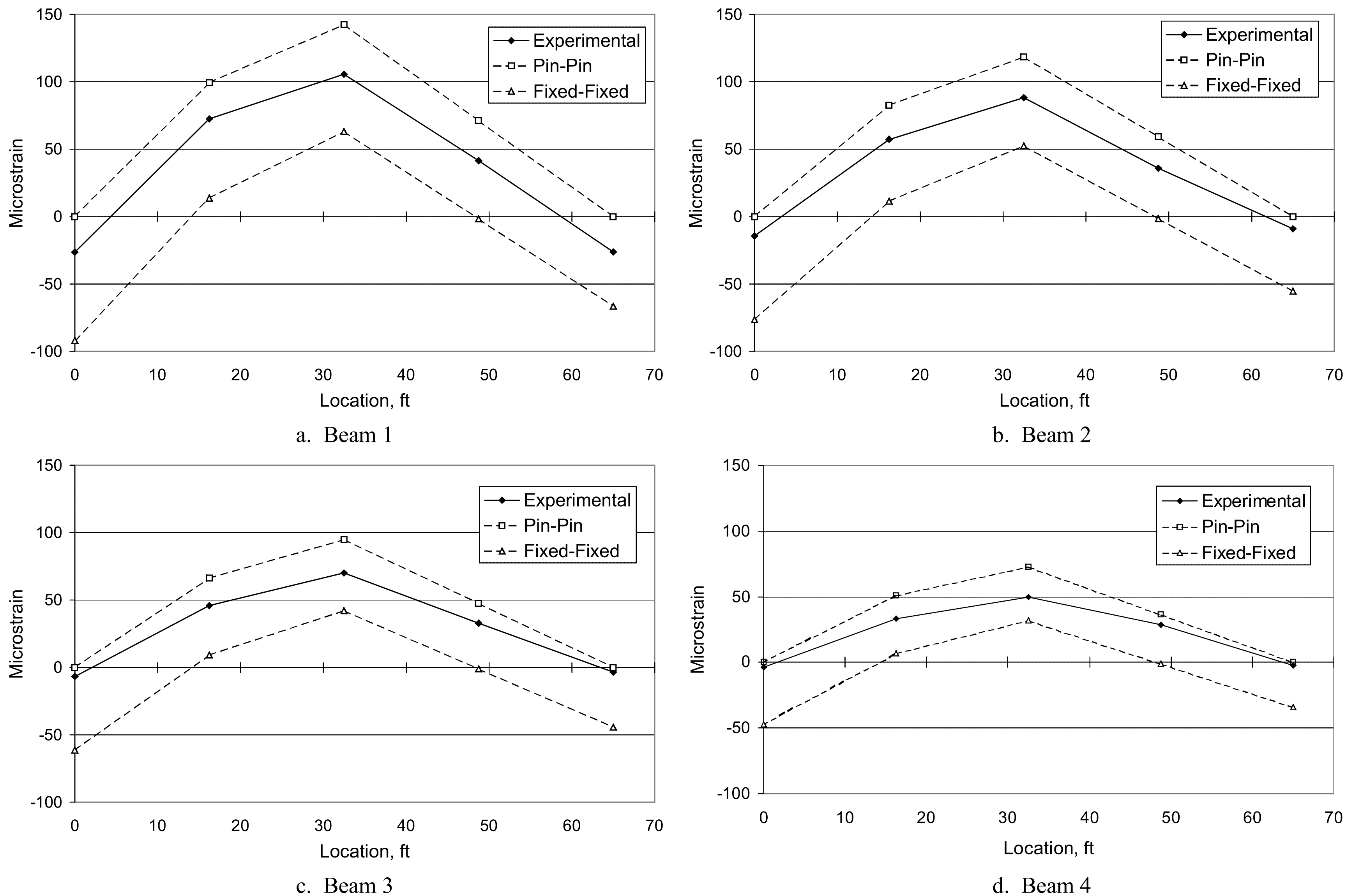


c. Beam 3



d. Beam 4

Figure 5.17. Experimental and analytical model comparison – load line 4, load at C.



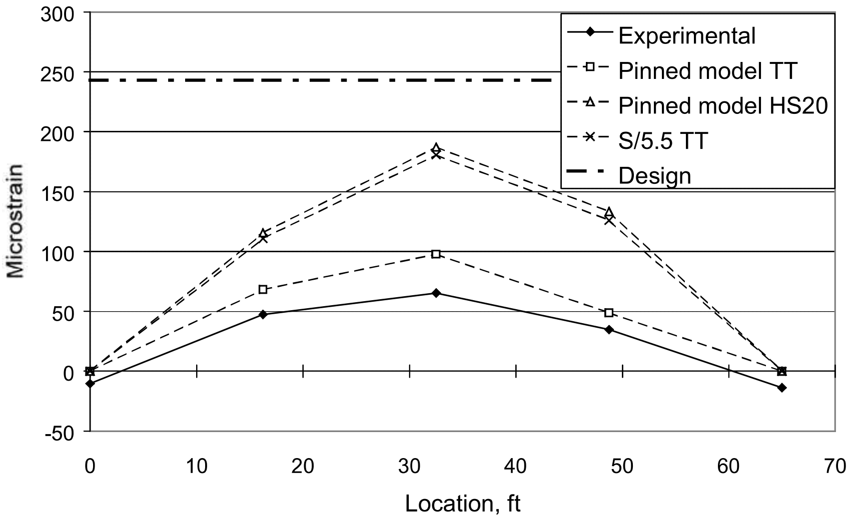
a. Beam 1 b. Beam 2

c. Beam 3 d. Beam 4

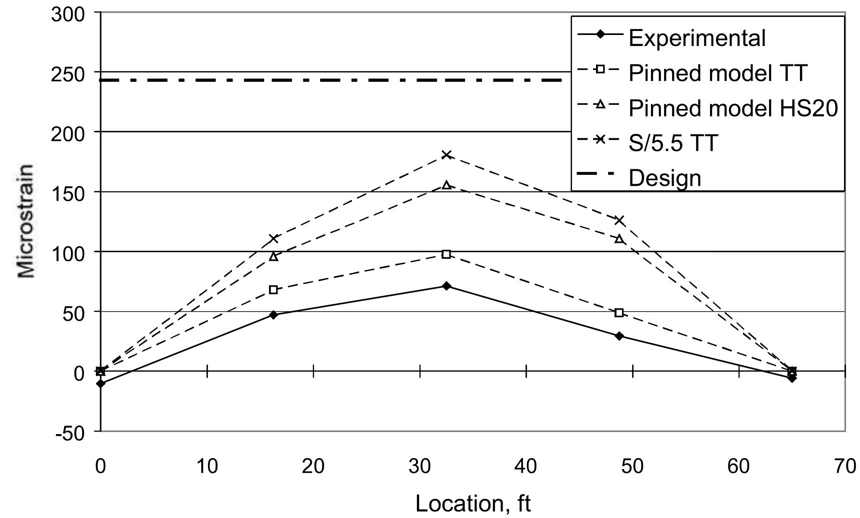
Figure 5.18. Experimental and analytical model comparison – load line 4, load at C.

gets closer to a particular stringer, the effects of the end restraint in that stringer are magnified and the stringer behavior shifts more towards the “fixed-fixed” condition. A comparison between selected design parameters, various analytical results, and the experimental results are presented in Fig. 5.19. The two design parameters shown are the AASHTO allowable live load design strain (i.e., the allowable design strain minus the design dead load and impact strains) and the actual AASHTO live load design strain (i.e., using AASHTO design specifications to determine the live load strain). For comparative purposes, three analytical results are also shown. The first utilizes the test truck (TT) with the AASHTO live load distribution factor (i.e., S/5.5). The second uses the "pin-pin" model previously described with the HS20 truck, and the third uses the "pin-pin" model with the TT. The final set of data shown in each figure is the experimentally determined bottom flange live load strains.

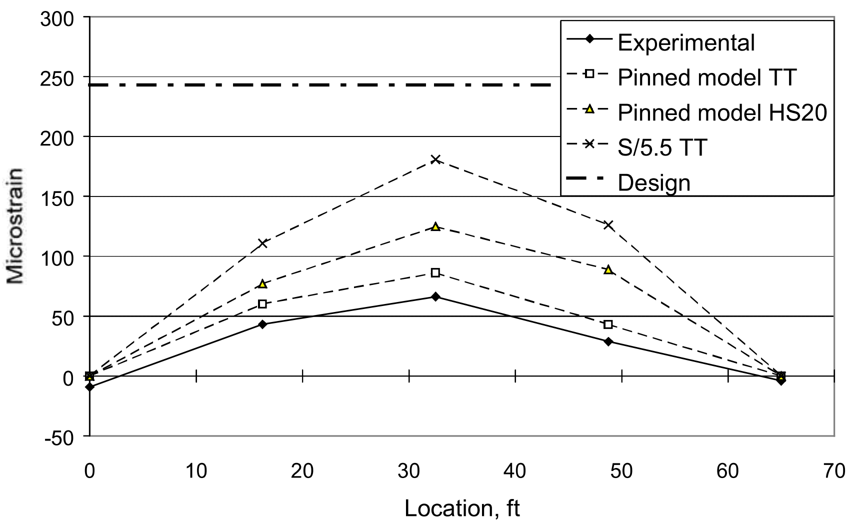
The difference between the analytical data using the TT and the HS20 truck is indicative of the relative size of the test truck in relation to loads typically used for design. Note also that the S/5.5 (TT) data set is the same for all stringers and all load cases. This results from the AASHTO provisions discounting differences in lateral load distribution characteristics for different stringers. Additionally, note the difference at mid-span between the S/5.5 (TT) data and the design data set. This difference represents the difference between the TT and the HS20 truck. Probably one of the most important pieces of information to note from these figures is that the experimental live load strains never exceed one-half of the design strains. There are three reasons for this. First, due to its lower total weight, the test truck generally induces less strain. Second, the lateral live load distribution is better in the



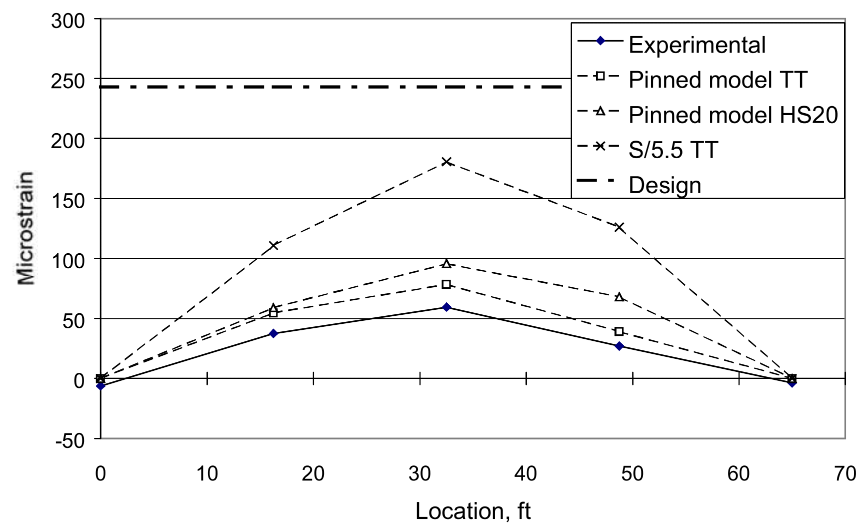
a. Beam 1



b. Beam 2



c. Beam 3



d. Beam 4

Figure 5.19. Comparison between design, analytical, and experimental results – load line 5 load at C.

PCDT bridge than what is prescribed in the AASHTO provisions. Finally, the unintended end restraint reduces the critical midspan response.

6. SUMMARY AND CONCLUSIONS

The study consisted of four major phases: literature review, documentation of the fabrication and construction, development of a design methodology, and service load testing. In the first phase, a literature review was completed focusing on the constructability of several prefabricated bridge systems used in short to medium span applications for low volume roads. Since none of the replacement systems were similar to the bridge system investigated in this study, the literature search provided minimal additional information.

In the second phase of the investigation, the fabrication and construction of the PCDT unit bridge system was documented through slides and a video. A design methodology (third phase) was developed that uses the AASHTO Standard Specification, and includes the capability to automatically generate the design and the construction drawings. The steel-beam PCDT unit bridge presented in this report was used as a replacement structure at the selected site in Black Hawk County, IA. The PCDT units were composed of two steel beams and a reinforced concrete deck. Each unit had a center to center beam spacing of 3.75 ft, an overall width of 7.5 ft, and a 5 in. thick reinforced concrete deck. The design width of the field demonstration bridge was 30 ft (thus, four PCDT units were required) with a span length of 64 ft center to center of abutments.

In the fourth phase, one of the units was monitored during its' lift and transport from the fabrication site to the bridge site. Strains were measured and recorded during the lifting of the unit from the temporary construction supports and placement on the hauling vehicle, during transportation to the bridge site, and during lifting from the hauling vehicle onto the abutments.

After the units were installed, three service load tests of the structure at different stages of construction were performed. The first service load test occurred after the four PCDT units were positioned on the abutments and connected by the intermediate diaphragms and PC connectors. The live load was provided using an unloaded tandem axle truck.

The second service load test was performed once the CIP deck was placed and cured but prior to the placement of the bridge guard rails. Two loaded tandem axle trucks were used for the live load in second and third service tests. After the bridge rails were installed, the third service load test was completed.

Although discussion of fabrication and construction has been limited to the processes used, the following list of suggestions will further assist in the fabrication/construction of the PCDT bridge system:

1. Each steel beam should be carefully inspected for natural camber and placed in a "camber up" position.
2. The channel size used in fabricating the PC connector should be the same height as the thickness of the PCDT concrete deck. This would eliminate the need for the wood blockouts used to create a void in the concrete for the later placement of the weld plates.
3. One may want to slot the bolt holes in the diaphragm channel sections. While slotted holes were not necessary in this demonstration project, they could aid in any misalignment problems in other PCDT bridges.
4. Fabrication of the PCDT units must be accomplished in a fully-shored condition. The spacing of the temporary supports must be calculated on an individual project basis however, the bottom flange fabrication stresses should be limited to 2 ksi.

5. If adequate space is available, fabrication of all units side by side will ensure the units will "fit" together at the bridge site. However, with proper quality control, the units can be fabricated one at a time to achieve the same "fit" at the bridge site.

Based on the laboratory tests, analytical modeling, and field tests completed in this investigation the following observations and conclusions can be made.

1. Used in combination, the PCDT units resulted in a simple-span bridge alternative for low-volume roads that is relatively easy to construct.
2. The PC connector used provides a connection with adequate strength to resist highway loads.
3. The PCDT units, with their relatively thin PC concrete deck, are strong enough to resist the handling stresses they will experience during construction and transportation.
4. The addition of the CIP concrete significantly improved the lateral load distribution characteristics of the bridge system.
5. The FEM developed in this investigation can accurately predict the behavior of the bridge system with various connector arrangements.
6. If a sufficient number of PC connectors are used, the behavior of the PCDT bridge is essentially the same as a typical steel stringer/concrete bridge in which the deck was placed in one pour.
7. A design methodology has been developed that allows easy design of the PCDT bridge superstructure through the use of a computer program, standard design tables, and a set of bridge plans (see Appendix A).

8. The lateral load distribution characteristics of the demonstration bridge are better than those prescribed by AASHTO design procedures.
9. The demonstration bridge exhibited some unintended end restraint. This end restraint is most likely used to the end diaphragm details used.
10. In the completed bridge, live load strains were detected in the bridge rails which indicated they were resisting a portion of the truck loads. This additional strength is not accounted for in the design, which results in a more conservative design.
11. Live load strains measured in the service load tests were less than 50 percent of the design values.

Overall, all phases of this project were successfully completed. Based upon the design, construction and service load testing, the steel-beam precast unit bridge was shown to be a viable low volume road bridge alternative. If salvaged beams are used, the initial bridge cost could be significantly reduced The construction process utilized standard methods resulting in a simple to use system that can be completed with a typical bridge construction crew. The service load tests verified the bridge had adequate capacity to support all legal loads and showed that the design methodology was conservative. In general, the measured values of strains and deflections were less than predicted values due to the end restraint which was caused by the addition of the concrete end diaphragm construction detail.

The final products of this study constitute a design/construction procedure that include the following for use by engineers: a design methodology that includes the complete bridge superstructure design and the associated construction plans, a set of 35 mm slides that describe the complete construction process of the PCDT units and the PCDT unit bridge, and a VHS video tape that provides information similar to the 35 mm slides. All of these are available from the Research Engineer in the Materials Office of the Iowa DOT.

7. ACKNOWLEDGEMENTS

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Robert L. Gumbert: County Engineer, Tama County (retired)
Mark J. Nahra: County Engineer, Delaware County
Gerald D. Petermeier: County Engineer, Benton County (retired)
Wallace C. Mook: Director of Public Works, Bettendorf
Jim Witt: County Engineer, Cerro Gordo and Winnebago Counties

Doug Wood, Manager of the ISU Structural Engineering Laboratory, also deserves special recognition for his involvement throughout the project; in particular with the field load testing. Special thanks are accorded to the following graduate and undergraduate civil engineering students for their assistance in various aspects of the project: Brent Phares, Erik Raker, Matt Thomas, John Rhodes, Thomas Threadgold, Dave Oxenford, Trevor Brown, and Christina Lundeen.

We would also like to thank Richard King, Black Hawk County Engineer, and Dennis Edgar, former assistant Black Hawk County Engineer, and the members of the Black Hawk County bridge crew for their hard work and dedication to this project: Daryl Ingamells, Merv Spears, Brian Davis and Dave Berry. Special thanks is also extended to Galen Webrand for his assistance during the project.

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APPENDIX A

DESIGN METHODOLOGY AND ASSOCIATED PLAN SHEETS

FOR

PCDT BRIDGE

The following pages are a complete set of plans and design aids for the PCDT bridge. Together, they represent the final product of this investigation and can be used by bridge engineers to produce a superstructure bridge design and the associated set of construction drawings. Note that these are only half-size versions. The CAD software that is available will produce full size plan sheets.

FOR PAGES 111-123, SEE THE *Word Templates* FOLDER ON THIS CD (separate from this document).

APPENDIX B

DRAWINGS FOR PCDT BRIDGE CONSTRUCTION

At

BLACK HAWK COUNTY NORTH OF SEC. 25

T-87N, R-14W

The following bridge design and associated drawing sheets were generated from the design methodology shown in Appendix A (excluding several modifications noted at the end of this Appendix).

FOR PAGES 127-136, SEE THE *CAD Templates* FOLDER ON THIS CD (separate from this document).

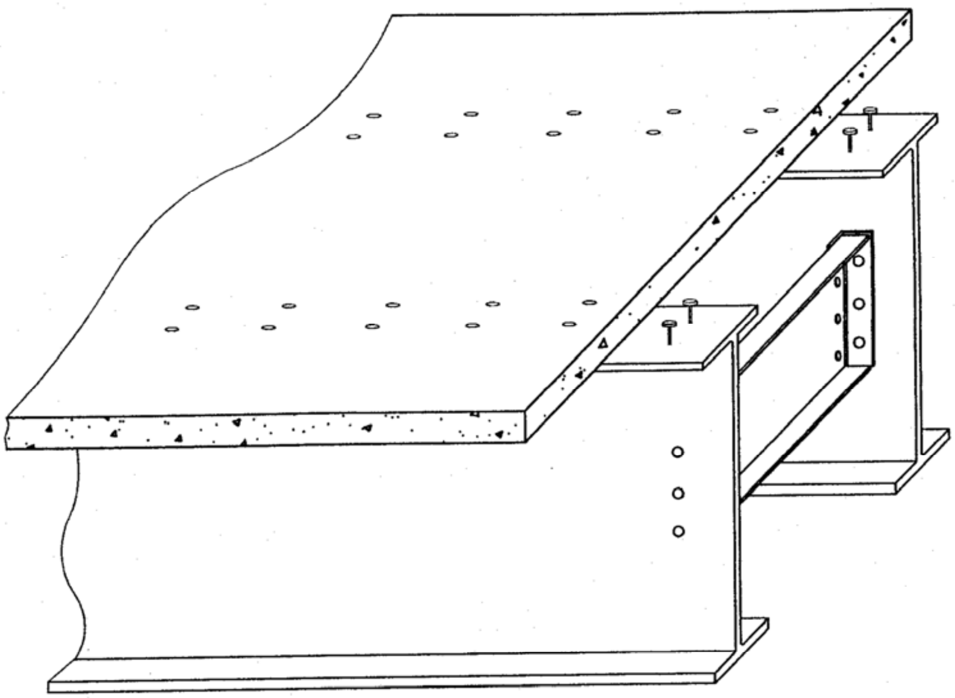

Figure B.1. Black Hawk County modification of the ends of the PCDT units.

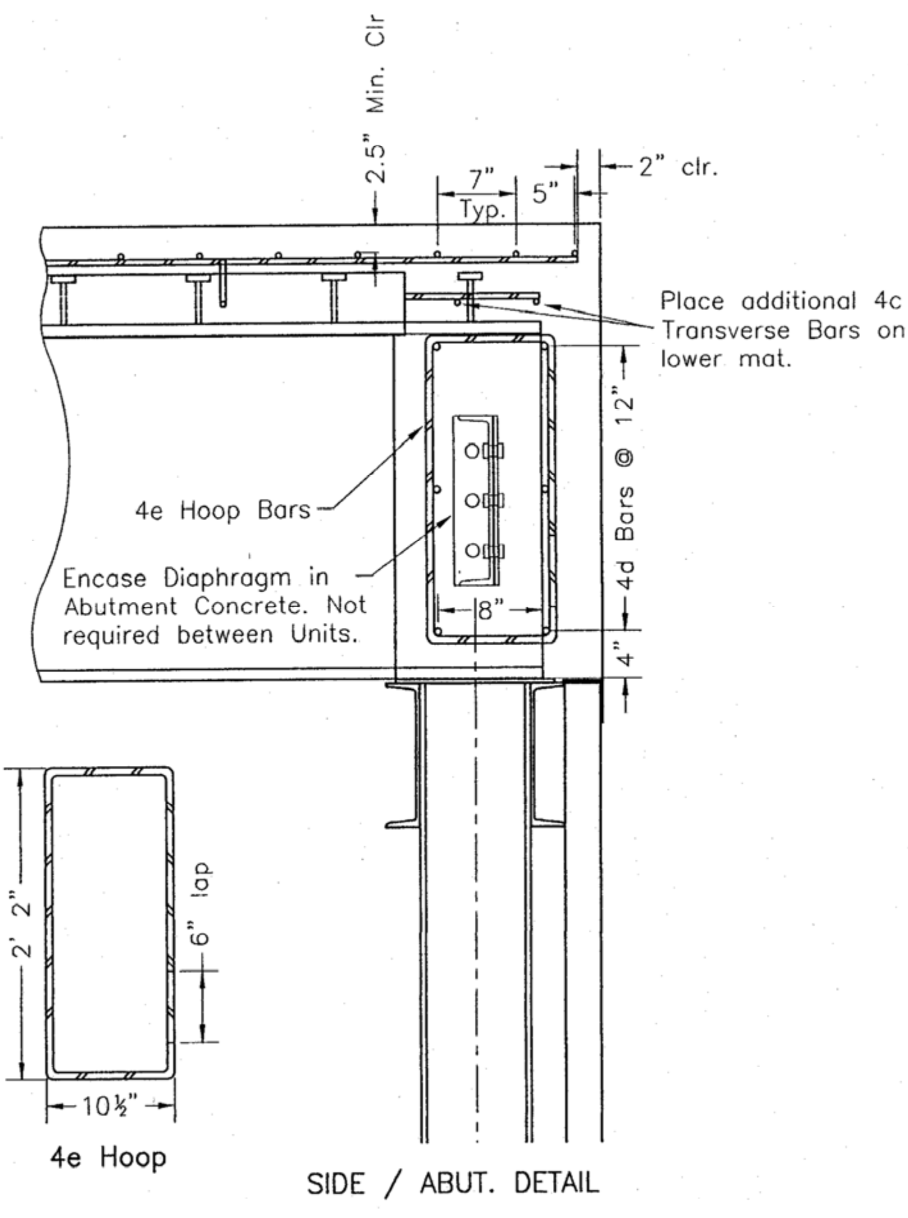

Figure B.2. Reinforcement details for concrete end diaphragms.

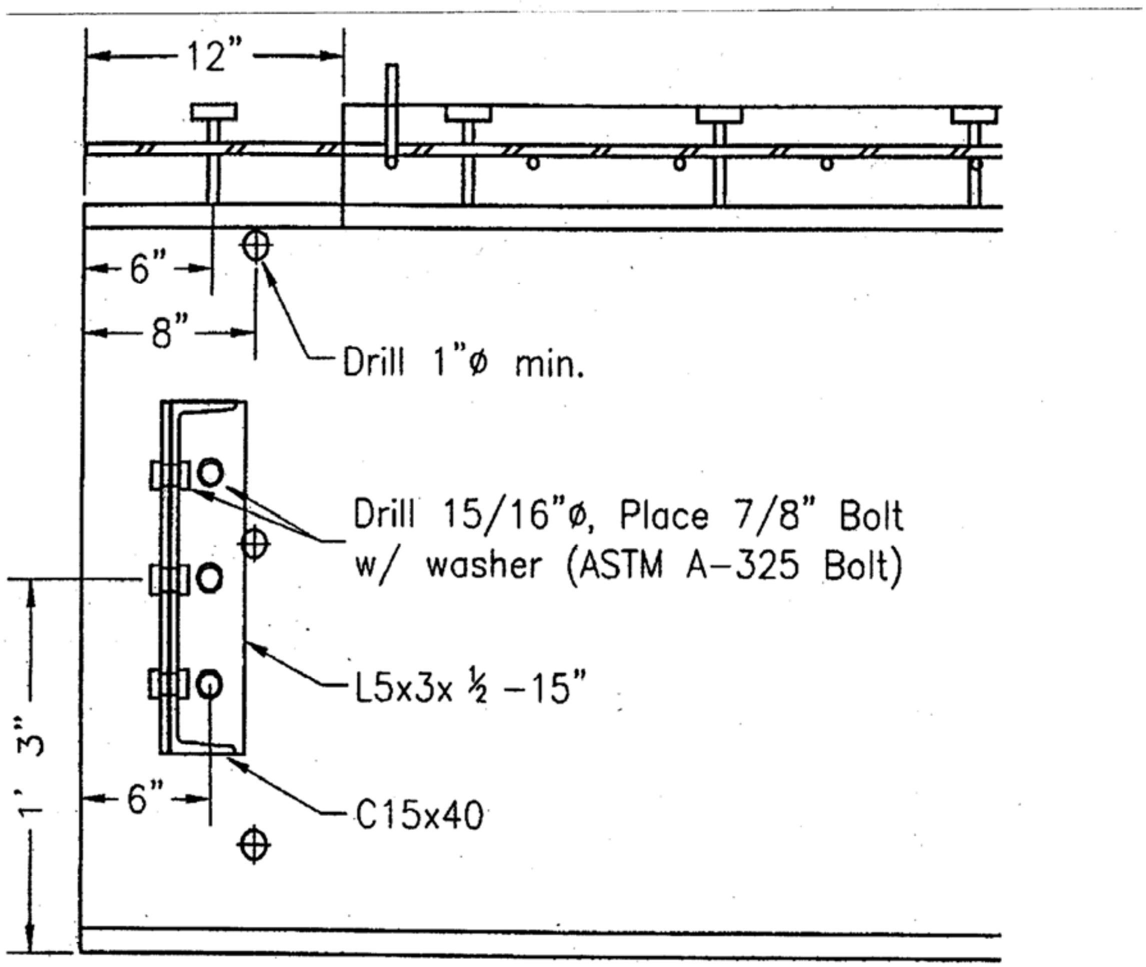

Figure B.3. Steel beam profile showing location of continuous transverse reinforcement.

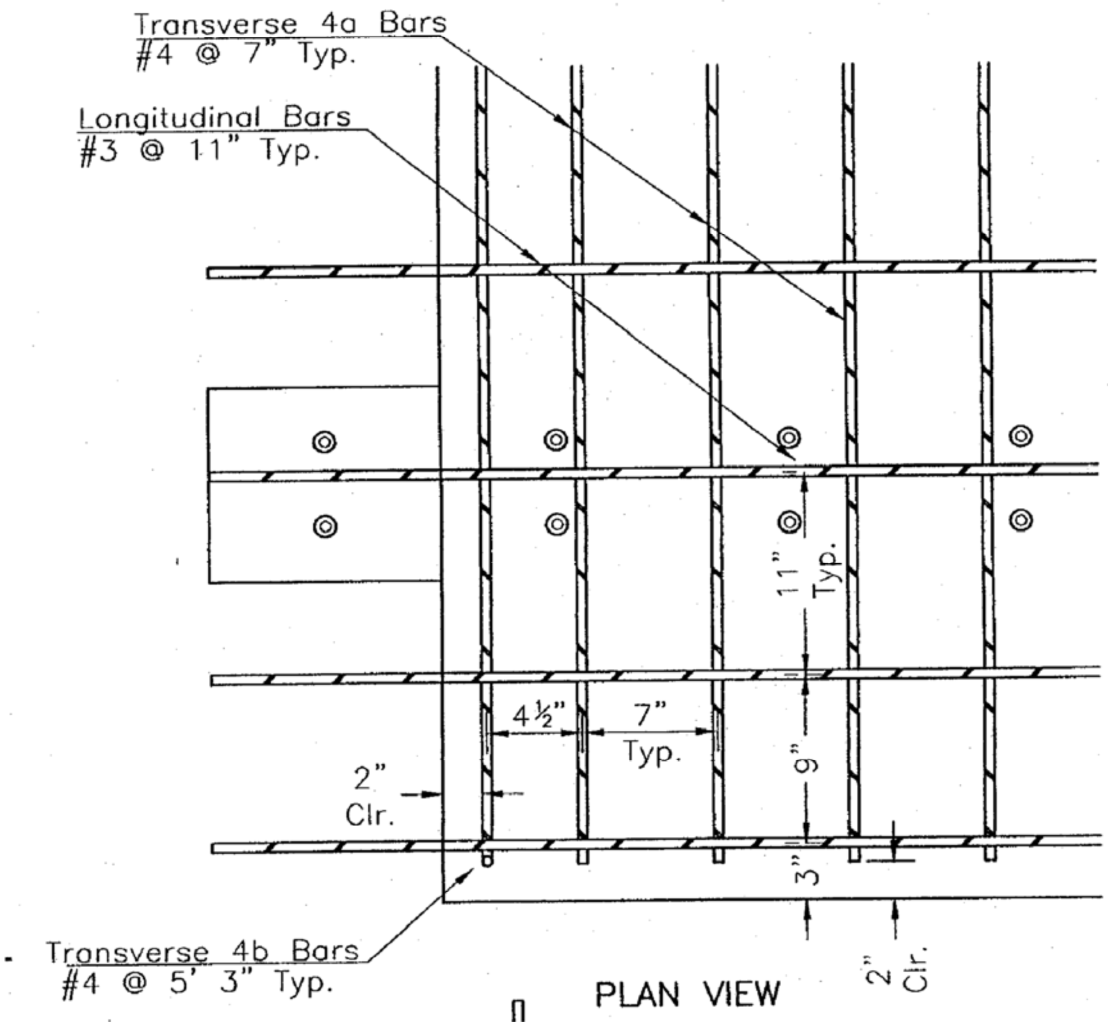

Figure B.4. Black Hawk County modified reinforcement spacing at PCDT ends.

APPENDIX C

CONCRETE TEST RESULTS

Table C.1. Compressive Strength Test Results, PCDT Units.

	Age (days)	Load (lbs.)	Strength (psi)	Avg. Strength (psi)
Unit 1				
	14	184,300	6,518	6,490
	14	182,700	6,462	
	29	203,100	7,183	7,280
	29	208,400	7,371	
	48	221,600	7,837	7,970
	48	229,000	8,099	
Unit 2				
	14	202,200	7,151	7,290
	14	210,100	7,431	
	28	232,700	8,230	8,240
	28	233,100	8,244	
	47	242,400	8,573	8,835
	47	257,100	9,093	
Unit 3				
	14	161,000	5,694	5,655
	14	158,800	5,616	
	29	184,500	6,525	6,680
	29	193,300	6,837	
	48	211,400	7,477	7,530
	48	214,500	7,586	
Unit 4				
	14	196,900	6,964	7,070
	14	202,900	7,176	
	28	213,700	7,558	7,690
	28	220,700	7,806	
	46	238,700	8,442	8,565
	46	245,500	8,683	

28-day design strength = 4,000 psi

Table C.2. Flexural Strength Test Results, PCDT Units.

	Age (days)	Load (lbs.)	Strength (psi)	Avg. Strength (psi)
Unit 1				
	28	5,900	704	630
	28	4,500	551	
Unit 2				
	28	7,150	865	815
	28	6,200	762	
Unit 3				
	28	4,000	4,93	560
	28	5,150	631	
Unit 4				
	28	6,700	821	760
	28	5,650	692	

28-day design strength = 474 psi

Table C.3. Compressive Strength Test Results, CIP Deck.

	Age (days)	Load (lbs.)	Strength (psi)	Avg. Strength (psi)
North Half				
	7	144,600	5,114	4,865
	7	133,100	4,707	
	7	135,100	4,778	
	14	168,000	5,942	5,670
	14	164,000	5,800	
	14	149,000	5,270	
	28	191,900	6,787	6,525
	28	187,700	6,638	
	28	174,000	6,154	
South Half				
	7	143,200	5,065	4,635
	7	131,200	4,640	
	7	118,800	4,201	
	14	149,600	5,291	5,330
	14	156,600	5,539	
	14	145,800	5,157	
	28	180,800	6,394	5,775
	28	156,100	5,521	
	28	152,800	5,404	

28-day design strength = 4,000 psi

Table C.4. Flexural Strength Test Results, CIP Deck.

	Age (days)	Load (lbs.)	Strength (psi)	Avg. Strength (psi)
North Half	7	5,650	700	700
	14	5,750	719	720
	28	6,750	835	800
South Half	7	5,500	683	685
	14	5,625	706	705
	28	5,400	671	670

28-day design strength = 474 psi

Table C.5. Split Cylinder Strength Test Results, CIP Deck.

	Age (days)	Load (lbs.)	Strength (psi)	Avg. Strength (psi)
North				
Half	28	59,400	525	480
	28	44,600	394	
	28	58,400	516	
South				
Half	28	46,300	409	490
	28	59,100	522	
	28	60,200	532	

28-day expected strength = 480 psi

APPENDIX D

SLIDES OF PCDT BRIDGE FABRICATION,CONSTRUCTION AND TESTING

Table D.1. Description of Slides.

Slide No.	Description
1	Title Slide: Construction of a demonstration bridge
2	End view (looking east) of original bridge
3	South profile of existing bridge
4	North profile of existing bridge
5	Title Slide: Fabrication of steel beams in the PCDT units
6	Top view of channel used in fabrication of precast connectors
7	Precast connector in-place in PCDT Unit
8	Diaphragm connection angle attached to beam web
9	Shear stud and ceramic ferral on beam flange
10	Placing shear studs
11	Shear stud in-place on top beam flange
12	Overview of all eight beams with shear studs
13	Title Slide: Fabrication of the PCDT Units
14	Construction of temporary supports
15	Placement of concrete in temporary supports
16	View of formwork and concrete in temporary supports
17	View of temporary supports and support beams
18	Placement of beams and diaphragms on the temporary supports
19	Positioning diaphragm for installation
20	Diaphragms connected to one beam
21	End-view of steel in PCDT unit
22	Diaphragms of one PCDT Unit in-place
23	Transverse view showing alignment of diaphragms
24	Formwork (headers) for placement of the PCDT concrete deck
25	Placement of headers
26	Overview of header placement
27	Placement of stringers and plywood in the formwork
28	Formwork for the exterior PCDT Units
29	Support of the exterior formwork
30	View of the exterior formwork from the underside
31	All the formwork in-place
32	Exterior edge formwork
33	PC connectors in-place
34	Wood blockout for void on top of the PC connectors
35	Reinforcement bar chairs supporting transverse deck reinforcement

Table D.1. (continued)

Slide No.	Description
36	View of all reinforcement in-place for one PCDT unit
37	End-view of one PDCT unit prior to concrete placement
38	Aerial view of all four PCDT units
39	Longitudinal bars extended through the end formwork
40	Placement of reinforcing steel in one PCDT unit
41	Placing the PC connectors in PCDT unit
42	Placing concrete in PCDT unit
43	Placement of concrete using concrete bucket
44	Finishing of PCDT deck concrete
45	Placing transverse "grooves" into the wet concrete
46	Modified bull float employed for placement of the "grooves"
47	View of finished PCDT unit surface
48	Wet burlap in-place
49	Overview of moist burlap in-place
50	View of the four finished PCDT units
51	Lifting the PCDT unit
52	Sideview of rigging system used
53	End view of rigging system used
54	PCDT unit being lifted
55	PCDT unit on hauling vehicle
56	PCDT unit being lifted off hauling vehicle at the bridge site
57	Title Slide: Field work
58	PCDT unit placed on the bridge abutments
59	Placing second PCDT unit on the bridge abutments
60	Top view of the PC connectors between two adjacent PCDT units
61	Top view of PC connector weld plate connecting adjacent PCDT units
62	End view of reinforcement in concrete end diaphragms
63	Top view of reinforcement in concrete end diaphragms
64	Forming the backwall of the concrete end diaphragms
65	Top view of the reinforcement in CIP deck
66	Centerline formwork used during CIP deck placement
67	Centerline keyway in-place
68	Placement of threaded rods for attaching guardrail system
69	Bridge system ready for placement of the south-half of the CIP deck
70	Placing south-half of the CIP deck
71	Vibrating screed used in placement of CIP concrete
72	Second screed used for finishing CIP concrete

Table D.1. (continued)

Slide No.	Description
73	Tining rake used to place the skid-resistance surface on the CIP deck
74	Title Slide: Finished bridge
75	End view of the bridge prior to placement of rails and approaches
76	Side view of the concrete end diaphragms
77	View from below showing all four PCDT units
78	Approach and bridge rail in-place
79	North profile of PCDT bridge
80	South profile of PCDT bridge
81	Title Slide: Service Load Testing
82	Deflection instrumentation in-place
83	Empty tandem axle truck providing loading on connected PCDT units
84	Empty tandem axle truck at edge of bridge
85	Fully loaded tandem truck for loading completed bridge
86	Fully loaded tandem axle truck on bridge (no bridge rails)
87	Two tandem axle trucks on completed bridge (no bridge rails)
88	Testing of completed bridge with bridge rails in place
89	Title Slide: For More Information
90	Title Slide: Research Sponsor

FOR PAGES 151-165, SEE *slideshowfinal.ppt* ON THIS CD (separate from this document).