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IMPLEMENTATION OF A NON-METALLIC REINFORCED BRIDGE DECK

Volume 2: Thayer Road Bridge

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16. Abstract The primary maintenance problem with bridges in Indiana has been deterioration of the concrete deck which is often related to corrosion of the reinforcing steel. While a corrosion protection system consisting of epoxy-coated reinforcement in combination with 2-1/2 in. of Class C concrete cover has been used in Indiana, research and experience have demonstrated that this system can be compromised. As an alternative solution to the corrosion problem in reinforced concrete, fiber reinforced polymer (FRP) bars which are corrosion resistant can be provided as reinforcement. This research was divided into two phases directed towards the implementation of a nonmetallic reinforced bridge deck. The first phase evaluated the bond strength of fiber reinforced polymer reinforcement with the goal of developing a design expression for the calculation of development and splice lengths. Forty-six glass FRP, carbon FRP, and steel reinforced concrete beams with unconfined tension lap splices were tested. The second phase consisted of the design, construction, and performance evaluation of a glass FRP bar reinforced concrete bridge deck. Based on this study, design recommendations are provided for the calculation of development and splice lengths of both FRP and steel reinforcement. Furthermore, the behavior of the FRP reinforced bridge deck is assessed and compared with its design assumptions. The findings of this study provide design tools and behavioral data that will assist in the future development and deployment of this technology.					
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CHAPTER 1

INTRODUCTION

1.1 Introduction

Corrosion of steel reinforcement in bridge decks subjected to an aggressive environment ultimately causes deterioration of concrete and loss of serviceability of the deck. Bridge decks are susceptible to deterioration because deicing salts accelerate corrosion of the steel reinforcement due to the presence of chloride ions. The most common application for corrosion prevention is the use of epoxy coated reinforcement. Nevertheless, extensive premature corrosion of epoxy coated steel reinforcement has been found in bridges, indicating the shortcomings of this protection method (Ehsani, Saadatmanesh, and Tao (1996)). Recently, fiber reinforced polymer (FRP) bars have become an alternative solution for structures susceptible to corrosion problems.

The objective of this research is to evaluate the performance of a fiber reinforced polymer (FRP) bar reinforced concrete bridge deck. As a part of the study, the deck of a five span, steel girder bridge was instrumented during replacement of the deck. The data was collected and analyzed to provide information regarding the behavior of the FRP reinforced deck.

1.2 General Description of the Bridge

The bridge is located on Thayer Road over Interstate 65 (I-65) in Newton County, Indiana. The bridge consists of five spans. The existing bridge had concrete girders in the first and last spans while the middle spans were continuous steel girders. Expansion joints were provided at the end bents as well as at Piers 2 and 5 (Figure 1.1). Due to deterioration of the concrete deck, the deck was replaced. The deteriorated concrete due to corrosion of the top mat reinforcement in the existing deck is shown in Figure 1.2. In

addition to the deck replacement, the reinforced concrete girders in the first and last spans were replaced with steel girders, existing end bents were reconstructed, and the pier caps at Piers 2 and 5 were replaced.

Figure 1.1 Existing Bridge

Figure 1.2 Deteriorated Concrete

The rehabilitated bridge has five spans with continuous steel girders. Expansion joints were provided over the bents. A plan and elevation view of the bridge is provided in Figure 1.3. The bridge consists of span dimensions of 39.8 ft, 63.5 ft, 77.8 ft, 63.5 ft, and 40 ft for a total length of 284 ft. A typical cross-section of the bridge is shown in Figure 1.4. The total bridge width is 34.5 ft with a 31.5 ft clear roadway. As illustrated in the typical plan, the deck is supported by seven wide flange steel girders. Girders in Spans A and E are W36x135 while Spans B, C, and D are W36x150. The bridge has a 5° horizontal curve with a 7% vertical cross-slope. Therefore, the skew angle varies with a 8.7° angle at Bent 1 and a 21° angle at Bent 6. The top mat of the deck is reinforced with glass FRP bars while the bottom mat is reinforced with epoxy coated steel reinforcement (Fig 1.4). Permanent metal stay-in-place deck sections were used to form the bridge deck.

Figure 1.3 Plan and Elevation View

CHAPTER 2

BRIDGE DESIGN

2.1 Background

The sixteenth edition of the AASHTO Standard Specifications and the ACI Committee 440 Guide for the Design and Construction of Concrete Reinforced with FRP bars (440.1R-03, 2003) were used in the design of the Thayer Road Bridge. Shear design was based on the design equation developed by Tureyen and Frosch (2003). Design drawings and detailed design calculations for the instrumented bridge are included in Appendix A.

As described in the ACI Committee 440 report, the design philosophy for FRP reinforced concrete includes both strength and working stress approaches. The design recommendations are based on limit state design principles in which FRP reinforced concrete is designed based on its required strength then checked for creep rupture endurance and serviceability criteria. The approach for the flexural design of steel reinforced concrete and FRP bar reinforced concrete is different. Steel reinforced concrete sections are generally designed under-reinforced to ensure yielding of steel reinforcement before crushing of concrete because yielding provides both ductility and warning prior to member failure. However, if FRP reinforcement ruptures, failure of the member is sudden and brittle. Therefore, design procedures encourage failure of concrete prior to failure of the reinforcement along with an increase in the factor of safety. ACI Committee 440 suggests using a strength reduction factor of 0.7 for sections controlled by crushing of concrete and a reduction factor of 0.5 for sections controlled by FRP bar rupture. FRP reinforced concrete members have a relatively small stiffness after cracking due to their low modulus of elasticity; therefore, the lower stiffness produces higher deflections, crack widths, and stresses. For glass FRP bar reinforced concrete

specimens, serviceability as well as creep and fatigue rupture endurance may govern design because of the lower modulus of elasticity of the glass FRP bar.

2.2 Deck Design

For the Thayer Road Bridge, design forces were determined from a one way slab analysis using the equations in the AASHTO Standard Specifications. An HS20-44 truck with a 30% impact factor was used for the live load analysis. For dead loads, in addition to the actual loads, a 35 psf allowance for a future wearing surface (corresponds to 3 in. of asphalt) and a 15 psf allowance for the permanent metal deck forms were considered in the calculations. AASHTO Equation (3-15) which calculates the maximum moment of a simply supported section where the wheel load is applied at midspan was used to derive the distribution width which was computed as $8S/(S + 2)$. Although the AASHTO requirement allows the use of a continuity factor of 0.8 with Equation (3-15) for both positive and negative moments because the continuous slab is supported with seven girders, this factor was used only for the calculation of negative moments. Because of the experimental nature of the project and the lack of long-term data regarding the behavior of FRP reinforcement, the positive moment region was designed considering a simple span. In the event of failure of the negative moment reinforcement, the deck will maintain the capacity to carry the design vehicle loads.

Serviceability was also considered. Short-term live load deflections were limited to the girder spacing (span length) divided by 800. Crack widths were calculated using equations provided by Gergely and Lutz (1968), Kaar and Mattock (1963), and Frosch (1999). For the FRP reinforced section, the equations by Gergely and Lutz (1968), and Kaar and Mattock (1963) are modified by multiplying them by the modular ratio, E_f/E_s .

The 8 in. thick concrete slab is reinforced with both epoxy coated steel and FRP bars. Glass FRP bars are used in the top mat of the deck while epoxy coated reinforcement are provided in the bottom mat. INDOT Class C concrete ($f'_c = 4,000$ psi) was used for the design of the deck. The deck was designed considering a top clear cover of 2 in. and a bottom clear cover of 1 in. The ultimate tensile strength of the glass FRP bars was conservatively assumed as 80 ksi per ACI 440.1R-03. Since the material

properties from the manufacturers do not include the effect of long term exposure to the environment, an environmental factor of 0.7 according to ACI 440.1R-03, was used to determine the design tensile strength. A summary of the design forces acting on the 8 in. deck is provided in Table 2.1.

Table 2.1 Design Forces

	Ultimate	Service
Max. Negative Moment, M (kip·ft/ft)	9.1	4.4
Max. Positive Moment, M (kip·ft/ft)	11.4	5.5
Shear, V (kip/ft)	4.3	2.1

The width of the deck is 34.5 ft; therefore, reinforcement in the transverse direction were not spliced. However, splicing was required for the bars in the longitudinal direction. The required splice length for the steel bars was calculated using the AASHTO Standard Specifications. For the FRP reinforcement, the required splice length was calculated using ACI Committee 440 (440.1R-03, 2003) and Mosley (2000) design equations ignoring the environmental reduction factor. Based on the design equations, the required splice length was calculated as 32 in. At the time when the deck was designed, research on bond (Pay, 2005) was still in progress. However, test results showed that a bond specimen with a 36 in. spliced Pultrall FRP bars reached 50 ksi which was an indication that the 32 in. spliced bar will not reach it's ultimate design capacity of 80 ksi. Based on the design calculations, the maximum stress on the longitudinal bar was calculated as 18 ksi; therefore, a 32 in. splice length was considered to be adequate.

Based on the design forces, the reinforcing bars were selected for the deck. The provided reinforcing bars as designed are summarized below:

Reinforcement Perpendicular to Traffic: (No splicing required)

Top Bars: #6 Glass FRP reinforcing bars at a 6 in. spacing
Clear Cover: 2 in.

Bottom Bars: #5 steel reinforcing bars at a 8 in. spacing

Clear Cover: 1 in.

Reinforcement Parallel to Traffic: (Splicing required)

Top Bars: #5 Glass FRP reinforcing bars at a 6 in. spacing across the entire width of the deck.

Splice Length = 32 in.

Bottom Bars: #5 steel reinforcing bars at a 12 in. spacing across the entire width of the deck.

Splice Length = 24 in.

The computed crack widths for the deck in the transverse direction are provided in Table 2.2. The maximum crack width over the girder for negative moment region was 22 mils which was considered to be reasonable for the FRP reinforced deck.

Table 2.2 Crack Width Calculations (Transverse Direction)

	Between Girders (in.)	Over Girder (in.)
Gergely and Lutz (1968)	0.0047	0.022
Kaar and Mattock (1963)	0.0056	0.019
Frosch (1999)	0.0105	0.022

In addition to crack widths, maximum stresses in the reinforcing bars at service loads were calculated in both the transverse and longitudinal direction (Table 2.3). The service load forces considered included the HS20-44 truck with a 30% impact load as well as the dead loads. These calculations are provided in Appendix A, and the results are summarized in Table 2.3.

Table 2.3 Reinforcement Stresses for Service Loads

	FRP Bars	Steel Bars
Longitudinal (ksi)	2.1	9.8
Transverse (ksi)	11.0	25.0

Creep rupture of the FRP reinforcement under sustained stress was also checked. The sustained stress level in the FRP bar due to dead load was calculated as 1.5 ksi which is lower than the calculated allowable stress of 16 ksi.

Punching shear capacity of the deck was evaluated using the design equations provided in ACI 318-02. As the bottom mat of the deck consists of steel reinforcement, the equations provided in ACI 318 are applicable for the Thayer Road Bridge deck. The following section discusses the punching shear capacity evaluation of FRP-reinforced two-way slabs.

2.2.1 Punching Shear Capacity of FRP Reinforced Slabs

The ACI 440 report “Guide Design and Construction of Concrete Reinforced with FRP bars” (440.1R-03, 2003) could not address to the shear strength of the FRP bar reinforced two way slab due to the limited experience to-date. In 2006, ACI Committee 440 (440.1R-06, 2006) adapted a design equation which is the modified version of the one-way shear design model proposed by Tureyen and Frosch (2003). A statistical evaluation of test results shows that the modified Tureyen and Frosch (2003) design equation leads to conservative punching shear capacities for both FRP and steel reinforced concrete slabs (Ospina (2005)).

Experimental evidence has shown that the axial stiffness of the reinforcement and concrete strength significantly affects the punching shear capacity of two way slabs (Ahmed et al. 1993; Bank and Xi 1995; Matthys and Taerwe 2000; Ospina et al. 2003). According to Tureyen and Frosch, the nominal shear strength due to concrete contribution of reinforced sections subjected to shear can be estimated using the following equation:

$$V_c = 5\sqrt{f'_c}b_w c \quad (1)$$

$$c = kd$$

where:

b_w = width of the web, in.

c = cracked transformed section neutral axis depth, in.

$f_y = \text{specified yield strength of reinforcement, psi}$

Eq. (1) accounts for the axial stiffness of the reinforcement through the neutral axis depth c , which is the function of the flexural reinforcement ratio ρ , and the modular ratio n .

Equation (1) can be rewritten as Eq. (2) which is simply the ACI 318-05 one-way shear equation for steel-reinforced members modified by the factor $(5/2)k$.

$$V_c = \left(\frac{5}{2}k \right) 2\sqrt{f'_c}b_w d \quad (2)$$

According to ACI 318-05, shear stress due to ultimate loads in slabs subjected to bending in two directions is limited to no more than $4\sqrt{f'_c}$ for square concentrated loads or columns. Ignoring the column aspect ratio, the nominal shear strength provided by concrete can be calculated using Eq. (11-35) in ACI 318-05.

$$V_c = 4\sqrt{f'_c}b_o d \quad (3)$$

where:

$b_o = \text{the critical section perimeter, in.}$

Ospina (2005), recognizing the similarities between Eq (3) and (4), proposed the following equation to calculate the punching shear capacity of steel and FRP bar reinforced two way slabs.

$$V_c = 10\sqrt{f'_c}b_o kd \quad (4)$$

Eq. (4) can be rewritten as Eq. (5) which is the ACI 318-05 two way slab equation for steel reinforced members modified by the factor $(5/2)k$.

$$V_c = \left(\frac{5}{2}k \right) 4\sqrt{f'_c}b_o d \quad (5)$$

Ospina (2005) evaluated the performance of the proposed equation by comparing the results with the experimental results from punching shear tests on 138 steel reinforced

and 27 FRP bar reinforced slabs. The equation provides conservative results for both FRP and steel reinforced two-way slabs across the range of reinforcement type and ratios and concrete strength evaluated by Ospina (2005).

CHAPTER 3

BRIDGE CONSTRUCTION

3.1 Construction of the Deck

INDOT Class C concrete ($f'_c = 4,000$) was used for the deck. The top mat of the reinforcing bars in the deck consists of Glass Fiber Reinforced Polymer (GFRP) bars while the bottom mat bars consists of epoxy coated steel bars. Metal stay-in-place deck panels were used to form the bridge deck. Shear studs were provided along the steel girders in the positive moment region (Appendix A). The concrete pour started at 6:50 am and lasted approximately 4.5 hours. The west approach slab was poured on the day of the deck casting while the east approach slab was cast four days later. The casting schedule is provided in Table 3.1, and the completed bridge deck is shown in Figure 3.1. Details of the casting schedule are provided for the regions of the deck where instrumentation was provided.

Table 3.1 Casting Schedule

6/18/2004	Deck cast
6:50 AM	Casting started from east of the bridge at Bent 6
7:30 AM	Reached Pier 5
8:10 AM	Reached mid-span of Span D
8:35 AM	Reached Pier 4
9:00 AM	Reached mid-span of Span C
9:00 AM	Pump truck moved from East Side of the Bridge to West Side
10:00 AM	Pumping restarted
11:35 AM	Casting completed (Reached Bent 1)
12:25 PM	West approach cast
Note:	East approach slab not cast
6/22/2004	East approach slab cast
6/25/2004	Parapets were cast

Figure 3.1 Completed Bridge

3.1.1 Concrete

The deck was cast using INDOT Class C concrete. The mix design and source of the materials used in the mix are provided in Table 3.2.

The compressive strength of the concrete was estimated from tests of 6x12 in. cylinders. The cylinders were cured in the same manner as the deck. Load was applied using a 600 kip Forney testing machine at a rate of 35 psi/sec for the compressive tests. The compressive strength-gain curves are shown in Figure 3.2, and the compressive strengths are provided in Table 3.3.

Table 3.2 Mix Design per Cubic Yard (INDOT Class C)

Material	Weight (lb.)	Absolute Volume (ft ³)	Source	Standard
Cement (lbs)	658	3.35	Essroc Materials, IN	C150 Type 1
Fine Aggregate (lbs)	1231	7.35	IMI Kewanna, IN	Indot #23
Coarse Aggregate (lbs)	1771	10.16	Vulcan Materials, IN	Indot #8
Water (lbs)	273	4.38	-	Potable
Air Entraining Admix.(oz)	8.9	1.76	Daravair 1440	C260
Water Reducer (oz)	19.7	-	Daratard 17	C494 Type D
Fly Ash (lb)	None	0.00	-	
Slump	4 in.		-	
Air Content	6.5%		-	

Figure 3.2 Concrete Compressive Strength

Table 3.3 Concrete Compressive Strength

Age (days)	Concrete Strength (psi)				Test Date
	Specimen 1	Specimen 2	Specimen 3	Average	
0	0	0	0	0	6/18/2004
3	4809	4897	5118	4941	6/21/2004
7	5328	5376	5353	5352	6/25/2004
14	6047	5861	6095	6001	7/2/2004
21	6481	6916	6206	6534	7/9/2004
28	6131	6250	6831	6404	7/16/2004
56	-	7329	7103	7216	8/13/2004

3.1.2 FRP Reinforcement

No. 5 and No. 6 glass FRP bars from Pultrall Inc. were used in the top mat of the deck and are produced from E-Glass fibers and vinyl ester resin. The bars, commercially named as V-ROD® GFRP, are composed of 25% resin matrix and 75% glass fibers by volume with a surface deformation of a sand coating.

Tensile tests on representative coupons were performed for each reinforcement size to determine their mechanical properties. Coupons for FRP bars were tested considering the requirements of ACI 440 (ACI 440.3R-04). The ends of the bars were encased in a 1.5 in. Schedule 80 steel pipe to attach the coupon sample to the testing machine. Sikadur 33, a smooth-paste epoxy adhesive, was used to attach the bars to the steel pipe. Stoppers were provided at the ends of the pipe to center the bar inside the pipe. This type of gripping system is needed to ensure that failure does not occur at the gripped ends before reaching the ultimate tensile strength of the FRP bar. Three coupons were tested for each reinforcing bar size. Details of the test coupon are shown in Figure 3.3 while details for each FRP reinforcement size are provided in Table 3.4.

A 120 kip Baldwin universal testing machine was used to test the FRP coupons. Loads were measured directly from the test machine, and strains were measured using an extensometer with a 2 in. gage length. The extensometer was removed from the specimen at a load which corresponded to approximately 70 % of the manufacturer's reported tensile strength of the bar. The measured modulus of elasticity, E_r , and ultimate strength of the FRP bars are provided in Table 3.5. The bar stress was calculated by

dividing the measured load by the nominal bar cross-sectional area. The modulus of elasticity was computed from a straight line best-fit of the stress-strain curve. The rupture strain was not measured since the extensometer was detached prior to failure.

The #5 FRP bars slipped in the steel pipe; therefore, the ultimate strengths obtained from those specimens do not represent the actual ultimate strength. However, the modulus of elasticity of the FRP bars was obtained from an extensometer directly attached to the FRP bar and was not affected from slippage of the anchorage. The failure modes of the specimens are shown in Figure 3.4. Stress-strain curves for both #5 and #6 bars are plotted in Figure 3.5.

Figure 3.3 Test Coupon Details for FRP Reinforcement

Table 3.4 Test Coupon Details

Bar Type	Bar Size	Outside Diameter, d (in.)	Pipe Wall Thickness, t (in.)	Anchor Length, L_a (in.)	Pipe Length, L_p (in.)	Free Length, L (in.)
<i>Glass</i>	#5	1.90	0.2	15	17	25
	#6	1.90	0.2	18	20	40

Table 3.5 Properties of Reinforcing Bars

Bar Size	E_r (ksi)	u (ksi)	Surface Deformation
#5	6900	101*	Sand
#6	7200	111	Sand

* Slipped in the steel pipe

(a) #6 FRP Bar Failure

(b) #5 FRP Bar Failure
(Slipped in Anchorage Zone)

Figure 3.4 Failure of the Coupon Specimens

Figure 3.5 (a) Stress versus Strain Curves (#5 Bars)

Figure 3.5 (b) Stress versus Strain Curves (#6 Bars)

CHAPTER 4

INSTRUMENTATION

4.1 Instrumentation of the Deck and Steel Girders

To evaluate the behavior of the FRP reinforced deck, an instrumentation plan was developed and implemented. The goal was to use the measured strains to evaluate the performance of the FRP bars and compared the measured values with the design calculations. The instrumentation included uniaxial foil strain gages, embedded concrete gages, and thermocouples. Four wire, full bridge modules (4WFB350) were used to complete the full bridge circuit with uniaxial strain gages. Gage types used for the instrumentation are provided in Table 4.1. Details of the gage locations are given in Appendix B.

A data acquisition system, incorporating a Campbell Scientific Inc. CR10X, two AM 16/32 channel multiplexers, and two AM 416 multiplexers, was designed to measure and collect the data (Figure 4.1). The wiring scheme of the data acquisition system is shown in Figure 4.2. A 12 volt, 26 Amp-hours sealed battery and a 20 watt solar panel system were used to power the data acquisition system. The gage readings were recorded to the datalogger every ten minutes. The data was downloaded remotely from the datalogger to a computer through a wireless modem.

Table 4.1 Gage Types

	Reinforcing Bars	Steel Girders	Concrete
Brand	TML	TML	Micro Measurements
Type	FLA-6-350-11-5LT	FLA-6-350-11	EGP-5-350
Resistance	350 +/- 1.5 Ohms	350 +/- 1 Ohms	350 Ohms +/- 0.8%
Gage Factor	2.13 + or - 1 %	2.13 + or - 1 %	2.06 +/- 1%
Temp. Comp.	$6 \times 10^{-6} / ^\circ\text{F}$	$6 \times 10^{-6} / ^\circ\text{F}$	
Trans. Sensitivity	-0.3 %	-0.2 %	

Figure 4.1 Data Acquisition System

The reinforcing bars were instrumented in the deck in both the transverse and longitudinal directions. The locations were selected to allow investigation of the critical regions where maximum moments occur. The locations of the gages are tabulated in Table 4.2 and are labeled based on the girder and span designations shown in Figure 4.3. In the longitudinal direction, FRP and steel bars were instrumented over the piers and at mid-span. Gages provided at the midspan of Span D in the transverse direction and at Pier 5 in the longitudinal direction are shown schematically in Figures 4.4 and 4.5, respectively. Embedded concrete gages were placed between the FRP and steel bars at approximately the mid-height of the deck (Figure 4.6). In the transverse direction, FRP bars were instrumented over the steel girders while steel bars were instrumented between girder lines (Figure 4.7). Four strain gages and three thermocouples were attached to two steel girders over Pier 4 as shown in Figure 4.8. The strain gages were attached to the top and bottom flanges of the steel section.

In addition to dummy gages provided in the cabinet box, a concrete block was cast around strain gages attached to a FRP and steel bar, an embedded concrete gage, and a thermocouple wire. The concrete block has the same thickness as the deck (8 in.), and bars were placed with a minimum clear cover of 2 in. for FRP bars and 1 in. for steel bars. The concrete block was also placed inside the cabinet. Dummy gages were provided to ensure that measured drift did not occur and to evaluate thermal response of the gages. Especially for the FRP bars, variation in strain output with temperature is essential for proper temperature compensation. The temperature compensation of FRP and steel bars will be discussed in the data analysis section.

Table 4.2 Location of the Gages

Gage Designation	Channel	Location	Gage Type	Direction	Girder or Span No.
CFT2	1-1	Span C	Strain Gage on FRP bar	Transverse	2
CFT3	1-2	Span C	Strain Gage on FRP bar	Transverse	3
CFT4	1-3	Span C	Strain Gage on FRP bar	Transverse	4
CSTB	1-4	Span C	Strain Gage on Steel bar	Transverse	B
CSTC	1-5	Span C	Strain Gage on Steel bar	Transverse	C
CFL3	1-6	Span C	Strain Gage on FRP bar	Longitudinal	3
CSL3	1-7	Span C	Strain Gage on Steel bar	Longitudinal	3

Table 4.2 Location of the Gages (continued)

Gage Designation	Channel	Location	Gage Type	Direction	Girder or Span No.
DFT2	1-8	Span D	Strain Gage on FRP bar	Transverse	2
DFT3	1-9	Span D	Strain Gage on FRP bar	Transverse	3
DFT4	1-10	Span D	Strain Gage on FRP bar	Transverse	4
DSTB	1-11	Span D	Strain Gage on Steel bar	Transverse	B
DSTC	1-12	Span D	Strain Gage on Steel bar	Transverse	C
DFL3	1-13	Span D	Strain Gage on FRP bar	Longitudinal	3
DSL3	1-14	Span D	Strain Gage on Steel bar	Longitudinal	3
4FT2	1-15	Pier 4	Strain Gage on FRP bar	Transverse	2
4FT3	1-16	Pier 4	Strain Gage on FRP bar	Transverse	3
4FT4	2-1	Pier 4	Strain Gage on FRP bar	Transverse	4
4STB	2-2	Pier 4	Strain Gage on Steel bar	Transverse	B
4STC	2-3	Pier 4	Strain Gage on Steel bar	Transverse	C
4FL2	2-4	Pier 4	Strain Gage on FRP bar	Longitudinal	2
4FLB	2-5	Pier 4	Strain Gage on FRP bar	Longitudinal	B
4FL3	2-6	Pier 4	Strain Gage on FRP bar	Longitudinal	3
4SL2	2-7	Pier 4	Strain Gage on Steel bar	Longitudinal	2
4SLB	2-8	Pier 4	Strain Gage on Steel bar	Longitudinal	B
4SL3	2-9	Pier 4	Strain Gage on Steel bar	Longitudinal	3
4GL2t	2-10	Pier 4	Strain Gage on Girder	Longitudinal	2
4GL2b	2-11	Pier 4	Strain Gage on Girder	Longitudinal	2
4GL3t	2-12	Pier 4	Strain Gage on Girder	Longitudinal	3
4GL3b	2-13	Pier 4	Strain Gage on Girder	Longitudinal	3
5FL3	2-14	Pier 5	Strain Gage on FRP bar	Longitudinal	3
5SL3	2-15	Pier 5	Strain Gage on Steel bar	Longitudinal	3
DUMMY_FRP	2-16	Box	Strain Gage on FRP bar	BOX	BOX
CCL3	3-1	Span C	Concrete Gage	Longitudinal	3
DCL3	3-2	Span D	Concrete Gage	Longitudinal	3
4CL3	3-3	Pier 4	Concrete Gage	Longitudinal	3
5CL3	3-4	Pier 5	Concrete Gage	Longitudinal	3
DUMMY_STEEL	3-5	Box	Strain Gage on Steel bar	BOX	BOX
DUMMY_CONCRETE	3-6	Box	Concrete Gage	BOX	BOX
DUMMY_FRP_Block	3-7	Box	Strain Gage on FRP bar	BOX	BLOCK
DUMMY_Steel_Block	3-8	Box	Strain Gage on Steel bar	BOX	BLOCK
DUMMY_Conc._Block	3-9	Box	Concrete Gage	BOX	BLOCK
4TX3F	4-1	Pier 4	Thermocouple	None	3
4TX3S	4-2	Pier 4	Thermocouple	None	3
4TX3A	4-3	Pier 4	Thermocouple	None	3
4TX3t	4-4	Pier 4	Thermocouple	None	3
4TX3m	4-5	Pier 4	Thermocouple	None	3
4TX3b	4-6	Pier 4	Thermocouple	None	3
Ambient	4-7	Box	Thermocouple	BOX	BOX
Ambient_Block	4-8	Box	Thermocouple	BOX	BLOCK

Figure 4.3 Girder and Span Designations

Figure 4.4 Span D – Transverse Direction

Figure 4.5 Pier 5 - Longitudinal Direction

Figure 4.6 Gages in the Longitudinal Direction

Figure 4.7 Gages in the Transverse Direction

Figure 4.8 Gages on the Steel Girders

4.1.1 Strain Gage Installation

Strain gages were installed after the reinforcing bars were placed in the deck. The same installation procedure was used for both the steel and FRP bars. The deformation of the bars where strain gages were installed was ground with a grinder and subsequently hand polished with Grade 320 grit sand paper. The procedure outlined by Measurements Group Inc. (Micro Measurements, B-127-13) was followed for the installation of gages using M-Bond 200. Degreaser was sprayed to the ground surface to remove oils, greases, organic contaminants, and soluble chemical residues. The surface was then wet abraded to remove any loosely bonded adherents using M-Prep Conditioner A with a 400-grit sand paper. The surface was cleaned by applying Conditioner A and was scrubbed with a cotton tipped applicator until a clean tip was no longer discolored by scrubbing. The surface was dried by wiping through the cleaned area with a gauze sponge. The final step used in cleaning the surface was to bring the surface condition back to an optimum alkalinity of 7.0-7.5 pH by applying a neutralizer to the surface. The surface was scrubbed with a cotton tip applicator and dried by wiping the area with a gauze sponge. Finally, the strain gages were attached to the prepared surface using M-Bond 200. The strain gages were then covered with a coating of M-Coat D to prevent the gages from damage due to moisture and subsequently covered with M-Coat F rubber to prevent the gages from physical damage during construction. Finally, the rubber was sealed with silicone to provide additional m

CHAPTER 5

DATA ANALYSIS AND RESULTS

5.1 Introduction

To evaluate the behavior of the Thayer Road Bridge deck, the results obtained from the gages were analyzed. The field results were then compared with the reinforcement stresses used to design the deck. Strain and temperature measurements obtained from the gages are provided in Appendix C.

5.2 Data Analysis

Self temperature compensated foil strain gages with a coefficient of thermal expansion of $6.0 \times 10^{-6} / ^\circ\text{F}$ was used for both FRP and steel reinforcement. The coefficient of thermal expansion for Pultrall V-Rod® Glass FRP reinforcement is $3.5 \times 10^{-6} / ^\circ\text{F}$ (V-Rod® Technical Specifications). If a strain gage is employed on a material other than that is used in obtaining the gage manufacturer's thermal output data, a self temperature compensated mismatch occurs, and the thermal output of the gage will differ (Measurements Group, TN-504-1). Thermal output strain for the gage mounted on FRP bar can be calculated using the formula below.

$$\varepsilon = \varepsilon + \Delta\varepsilon \quad (5)$$

$$\Delta\varepsilon = (6.0 - 3.5) \times 10^{-6} \times \Delta T (^\circ\text{F}) = 2.5 \times 10^{-6} \times \Delta T (^\circ\text{F}) \quad (6)$$

Figure 5.1 Measured Temperature from 4TX3F

As shown in Figure 5.1, the temperature in the deck varied from -5 °F to 108 °F during the course of monitoring. As the construction temperature was 95 °F, the temperature deviates -100 °F to +13 °F from the temperature at the time of construction. Thermal output strain for the gage mounted on the FRP bar was calculated as -250 $\mu\epsilon$ to 33 $\mu\epsilon$ using Eq. (6). This compares well with the output measured by the dummy gage (Dummy_FRP) as shown in Figure 5.2. Considering that the modulus of elasticity of the FRP reinforcement is 7,200 ksi, the resulting stress change due to temperature was calculated as -1.8 ksi to 0.17 ksi. Because of the high magnitude of stresses developed, the strains and the resulting stresses presented in the report were temperature compensated for the gages attached to the FRP bars. It should be noted that the gages on the steel reinforcement are properly temperature compensated (Figure 5.3). The minimum and maximum strain gage readings are presented in Table 5.1 while the minimum and maximum temperature gage readings are provided in Table 5.2. Gages measured positive strains for tension and negative strains for compression.

Figure 5.2. Measured Strain from Gage Dummy FRP

Figure 5.3. Measured Strain from Gage Dummy Steel

Table 5.1 Strain Gage Readings

Gage Designation	Strain ($\mu\epsilon$) ⁺		Stress (ksi) ⁺	
	ϵ_{min}	ϵ_{max}	σ_{min}	σ_{max}
CFT2	-159	61	-1.14	0.44
CFT3	-233	127	-1.68	0.92
CFT4	-145	48	-1.04	0.35
CSTB	-54	303	-1.58	8.78
CSTC	-95	186	-2.75	5.38
CFL3	-119	23	-0.82	0.16
CSL3	-189	1948	-5.48§	56.49§
DFT2	-192	53	-1.38	0.38
DFT3	-158	56	-1.14	0.40
DFT4	-128	57	-0.92	0.41
DSTB	-77	106	-2.24	3.08
DSTC	-60	187	-1.75	5.44
DFL3	-275	62	-1.90	0.43
DSL3	-57	319	-1.66	9.26
4FT2	-130	73	-0.94	0.53
4FT3	-215	29	-1.55	0.21
4FT4	-2	148	-0.01	1.07
4STB	-206	183	-5.99	5.31
4STC	-15	259	-0.45	7.52
4FL2	*	*	*	*
4FLB	-222	27	-1.53	0.19
4FL3	-379	41	-2.62	0.28
4SL2	-318	197	-9.23	5.73
4SLB	-129	386	-3.73	11.19
4SL3	-101	113	-2.92	3.27
4GL2t	-32	370	-0.93	10.72
4GL2b	*	*	*	*
4GL3t	-32	314	-0.93	9.11
4GL3b	*	*	*	*
5FL3	-56	210	-0.39	1.45
5SL3	-200	297	-5.79	8.62
DUMMY_FRP	-29	299	-0.21	2.15
CCL3	23	233	0.11	1.11
DCL3	14	154	0.07	0.73
4CL3	12	99	0.06	0.47
5CL3	-29	139	-0.14	0.66
DUMMY_STEEL	-5	80	-0.14	2.32
DUMMY_CONCRETE	-451	247	-2.15	1.18
DUMMY_FRP_Block	-475	46	-3.42	0.33

Table 5.1 Strain Gage Readings (continued)

	Strain ($\mu\epsilon$)		Stress (ksi)	
Gage Designation	$\epsilon_{\min}$	$\epsilon_{\max}$	$\sigma_{\min}$	$\sigma_{\max}$
DUMMY_Steel_Block	-234	391	-6.77	11.35
DUMMY_Concrete_Block	*	*	*	*

* Gages malfunctioned

§ Reading is not consistent with the other gages

+ Positive for tension

Table 5.2 Temperature Gage Readings

Gage Designation	Min Temperature ($^{\circ}\text{F}$)	Max Temperature ($^{\circ}\text{F}$)
4TX3F	-5	108
4TX3S	-4	102
4TX3A	*	*
4TX3t	-3	102
4TX3m	-3	99
4TX3b	*	*
Ambient	3	116
Ambient Block	*	*

* Gages malfunctioned

The temperature in the deck was measured by thermocouples attached to the FRP and steel bars. The temperatures measured by both gages were almost identical over the 1.5 year period. The gages have been monitored over one and a half years. The lowest temperature (-5°F) was recorded on December 24, 2004 while the highest (108°F) was recorded on June 26, 2005 during this period. Of particular interest were the gages where maximum strain values were recorded. These gages for both the longitudinal and transverse reinforcement are presented in Table 5.3. The strain data for the gages attached to these bars where the maximum strains were observed are provided in Figures 5.4 through 5.7. The response from all gages are provided in Appendix C. It should be noted that gages attached to the FRP bars in the deck were temperature compensated.

Table 5.3 Gages with Maximum Reading

	FRP Bars	Steel Bars
Longitudinal	5FL3 ($210 \mu\epsilon$)	4SLB ($386 \mu\epsilon$)
Transverse	4FT4 ($148 \mu\epsilon$)	CSTB ($303 \mu\epsilon$)

Figure 5.4 Measured Strain from Gage 5FL3 (FRP, Longitudinal)

Figure 5.5 Measured Strain from Gage 4SLB (Steel, Longitudinal)

Figure 5.6 Measured Strain from Gage 4FT4 (FRP, Transverse)

Figure 5.7 Measured Strain from Gage CSTB (Steel, Transverse)

In the longitudinal direction, the maximum stresses in the FRP and steel bars were approximately 1.5 ksi and 11.2 ksi, respectively. In the transverse direction, the maximum stresses in the FRP and steel bars were approximately 1.1 ksi and 8.7 ksi, respectively. To compare the data obtained from the gages, the reinforcement stresses calculated during design for service loads are shown in Table 5.4.

Table 5.4 Calculated / Measured Reinforcement Stresses for Service Loads

	FRP Bars		Steel Bars	
	Calculated	Measured	Calculated	Measured
Longitudinal (ksi)	2.1	1.5	9.8	11.2
Transverse (ksi)	11.0	1.1	25.0	8.7

As shown in the table, gage readings and the stresses calculated for service loads agree well for the reinforcement in the longitudinal direction. These are stresses developed by negative moment over the pier. In the transverse direction, the stresses calculated from gage readings are significantly smaller than the stresses calculated for the service loads. Stresses for the service loads were determined from a one way slab analysis using the AASHTO Standard Specifications equations which yields conservative estimates in the transverse direction. The actual reinforcement stresses are significantly lower than those calculated.

CHAPTER 6

SUMMARY AND CONCLUSIONS

6.1 Introduction

The Thayer Road Bridge is the first bridge in Indiana to incorporate FRP reinforcement in its bridge deck. This first implementation incorporated FRP bars into the top mat of reinforcement, and the deck was designed using the sixteenth edition of the AASHTO Standard Specifications and the ACI Committee 440 Guide for the Design and Construction of Concrete Reinforced with FRP bars (440.1R-03, 2003). Instrumentation was incorporated in the deck to evaluate the performance of the FRP bars and compare the measured values with design calculations. Data was obtained from strain gages attached to the FRP and steel bars, embedded concrete gages, as well as temperature gages.

6.2 Conclusion

An evaluation was performed by comparing the field investigation results with design calculations to better understand the behavior of the FRP reinforced deck. Based on this comparison, it was found that the transverse reinforcement stresses were significantly lower than the stresses calculated for service loads indicating that one way slab analysis using the equations provided in the AASHTO Standard Specification yields conservative estimates for both FRP and steel reinforcement in the transverse direction. Stresses in the longitudinal bars; however, compare well with the calculated stresses for service loads. Overall, the stresses measured in the reinforcing bars were within the range considered in design. Based on performance to-date, it is expected that the FRP bar reinforced Thayer Road Bridge deck will continue to perform well structurally as well as provide an example of the durability that can be achieved using fiber-reinforced polymer reinforcement to eliminate corrosion.

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APPENDICES

Appendix A

Bridge Plans and Design Calculations

<table border="1" style="width: 100%; border-collapse: collapse;"> <tr> <td style="width: 50%;">PROJECT IM/65-8(154)242</td> <td style="width: 50%;">DESCRIPTION BRIDGE FILE</td> </tr> <tr> <td>CONTRACT 8-70502</td> <td>BRIDGE NO. I-65-242-4889A</td> </tr> </table>	PROJECT IM/65-8(154)242	DESCRIPTION BRIDGE FILE	CONTRACT 8-70502	BRIDGE NO. I-65-242-4889A	INDIANA DEPARTMENT OF TRANSPORTATION BRIDGE PLANS FOR SPANS OVER 20 FT. ON THAYER ROAD R.P. 231+78 PROJECT NO. IM/65-8(154)242 P.E. IM/65-8(154)242 CONST. BRIDGE RECONSTRUCTION TO STRUCTURE NO. I-65-242-4889, THAYER ROAD OVER I-65, 2.15 MILES NORTH OF STATE ROAD 10 IN SECTION 1, T. 32 N., R. 8 W. IN NEWTON COUNTY.	<table border="1" style="width: 100%; border-collapse: collapse;"> <tr> <th colspan="2">TRAFFIC DATA</th> </tr> <tr> <td>ANNUAL AADT</td> <td>2780 VEH.</td> </tr> <tr> <td>ADJUSTED AADT</td> <td>2600 V.P.D.</td> </tr> <tr> <td>ADJUSTED AADT</td> <td>5 V.P.H.</td> </tr> <tr> <td>DIRECTIONAL DISTRIBUTION</td> <td>50 %</td> </tr> <tr> <td>TRUCKS</td> <td>5 % S.H.V.</td> </tr> <tr> <td></td> <td>5 % S.A.A.T.</td> </tr> </table> <table border="1" style="width: 100%; border-collapse: collapse;"> <tr> <th colspan="2">DESIGN DATA</th> </tr> <tr> <td>DESIGN SPEED</td> <td>40 M.P.H.</td> </tr> <tr> <td>PROJECT DESIGN CREEPER</td> <td>OR</td> </tr> <tr> <td>FUNCTIONAL CLASSIFICATION</td> <td>LOCAL COLLECTOR</td> </tr> <tr> <td>RURAL / URBAN</td> <td>RURAL</td> </tr> <tr> <td>TERMINAL</td> <td>LEAVE</td> </tr> <tr> <td>ADDRESS CONTROL</td> <td>NO</td> </tr> </table>	TRAFFIC DATA		ANNUAL AADT	2780 VEH.	ADJUSTED AADT	2600 V.P.D.	ADJUSTED AADT	5 V.P.H.	DIRECTIONAL DISTRIBUTION	50 %	TRUCKS	5 % S.H.V.		5 % S.A.A.T.	DESIGN DATA		DESIGN SPEED	40 M.P.H.	PROJECT DESIGN CREEPER	OR	FUNCTIONAL CLASSIFICATION	LOCAL COLLECTOR	RURAL / URBAN	RURAL	TERMINAL	LEAVE	ADDRESS CONTROL	NO																																																																																						
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Figure A.1 Title and Index Sheet

Figure A.2 General Plan

Figure A.3 Structure Sections

Figure A.4 Floor Details (1)

Figure A.5 Floor Details (2)

Figure A.6 Floor Details (3)

Figure A.7 Framing Plan

Design of Bridge Deck Reinforced with Steel and FRP bars. (Thayer Road)

Properties of Slab

$$\beta_L := 1.67$$

$$t := 8 \quad \text{Thickness of the slab (in)}$$

$$f := 0 \quad \text{Bulb Tee Flange Width (ft)}$$

$$S := 5.6 - \frac{f}{2} \quad \text{Span (ft)}$$

$$n := 6 \quad \text{Number of spans}$$

$$c_t := 2 \quad \text{Clear Cover (TOP of the slab) (in)}$$

8.22 - AASHTO

$$c_b := 1 \quad \text{Clear Cover (BOTTOM of the slab) (in)}$$

$$b_w := 12 \quad \text{Width of the Slab (in)}$$

Properties of Reinforcement

Steel

$$f_{ys} := 60 \text{ ksi} \quad d_{sb} := \frac{5}{8} \text{ in} \quad \text{\#5 Steel}$$

$$E_s := 29000 \text{ ksi}$$

$$\epsilon_s := \frac{f_{ys}}{E_s} \quad \epsilon_s = 2.069 \times 10^{-3}$$

FRP Carbon

$$f_{cu} := \text{■}$$

$$E_{fc} := \text{■}$$

FRP Glass

$$f_{gu} := 80 \text{ ksi} \quad d_{fb} := \frac{6}{8} \text{ in} \quad \text{\#6 Glass FRP}$$

$$E_{fg} := 6000 \text{ ksi}$$

Properties of Concrete

$$f_c := 4000 \quad \text{Concrete Strength (psi)}$$

$$E_c := 57 \cdot \sqrt{f_c} \text{ ksi}$$

Loads

Dead Loads:

$$DL_1 := 150 \quad \text{Weight of concrete (lb/ft}^3\text{)}$$

$$DL_2 := 35 \quad \text{Wearing Surface (lb/ft}^2\text{)}$$

$$DL_3 := 15 \quad \text{Permanent Metal Deck Weight (lb/ft}^2\text{)}$$

Live Load:

$$LL := 16000 \quad \text{HS-20-44 Truck Loading (lb)}$$

$$w_{DL1} := DL_1 \cdot \frac{t - 1.5}{12} \cdot \frac{12}{12}$$

$$w_{DL2} := DL_2 \cdot \frac{12}{12}$$

$$w_{DL3} := DL_3 \cdot \frac{12}{12}$$

$$w_{DL} := w_{DL1} + w_{DL2} + w_{DL3} \quad w_{DL} = 131.25 \frac{\text{lb}}{\text{ft}} \text{ per ft.}$$

Live Load is distributed over a width of

$$L_s := \frac{8 \cdot S}{S + 2} \quad L_s = 5.895 \text{ (ft)} \quad \text{(derived from equation 3-15 AASHTO 16th Edition)}$$

Load Combinations:

$$\gamma := 1.3$$

$$\beta_D := 1.0$$

$$\beta_L := 1.67$$

Eq. 3-10 AASHTO 16th Edition

Calculation of Positive Moment (lb.ft/ft)

$$M_{PDL} := \frac{w_{DL} \cdot S^2}{8} \quad M_{PDL} = 514.5 \quad \# \text{ ft} \quad d := t - \frac{d_{sb}}{2} - c_b$$

$$M_{PLL} := \frac{LL \cdot S}{4 \cdot L_s} \quad M_{PLL} = 3.8 \times 10^3 \quad \# \text{ ft}$$

$$M_{PIL} := \begin{cases} I \leftarrow \frac{50}{S + 125} & M_{PIL} = 1.14 \times 10^3 \text{ ft} \\ M_{IL} \leftarrow M_{PLL} \cdot I \text{ if } I < 0.3 & IL := \frac{M_{PIL} \cdot 4 \cdot L_s}{S} \quad I := \frac{M_{PIL}}{M_{PLL}} \cdot 100 \\ M_{IL} \leftarrow M_{PLL} \cdot 0.3 \text{ otherwise} & IL = 4.8 \times 10^3 \quad I = 30 \% \\ M_{IL} & \end{cases}$$

Ultimate Loads

Service Loads for deflection calculations

$$M_{PUL} := 1.3 \left[(\beta_D) \cdot M_{PDL} + \beta_L \cdot (M_{PIL} + M_{PLL}) \right] \quad M_{PSL} := M_{PDL} + (M_{PIL} + M_{PLL})$$

$$M_{PUL} = 1.139 \times 10^4 \quad \# \text{ ft}$$

$$M_{PSL} = 5.455 \times 10^3 \quad \# \text{ ft}$$

$$\text{Steel} := \begin{cases} \text{for } A_{si} \in 0, 0.0001 \dots 3 \\ \left| \phi M_n \leftarrow \frac{0.9}{12} \left[A_{si} \cdot f_{ys} \cdot 1000 \cdot d \cdot \left(1 - 0.6 \cdot \frac{\frac{A_{si}}{d \cdot 12} \cdot f_{ys} \cdot 1000}{f_c} \right) \right] \right. \\ \text{if } |\phi M_n - M_{PUL}| < 10 \\ \left| A_s \leftarrow A_{si} \right. \\ \left| \text{break} \right. \\ A_s \\ f \leftarrow \frac{A_s}{\left(\frac{\pi \cdot d_{sb}^2}{4} \right)} \\ s \leftarrow \text{trunc} \left(\frac{12}{f} \right) + 0.5 \text{ if } \text{trunc} \left(\frac{12}{f} \right) + 0.5 < \frac{12}{f} \\ s \leftarrow \text{trunc} \left(\frac{12}{f} \right) \text{ otherwise} \\ s \leftarrow \min(1.5 \cdot t, 18) \text{ if } s > 1.5 \cdot t \vee s > 18 \\ A_{sc} \leftarrow \frac{12}{s} \cdot \frac{\pi \cdot d_{sb}^2}{4} \\ S_0 \leftarrow s \\ S_1 \leftarrow A_{sc} \\ S_2 \leftarrow \frac{A_{sc}}{d \cdot 12} \end{cases}$$

$$\begin{cases}
 S_3 \leftarrow A_s \\
 S_4 \leftarrow \frac{12}{f} \\
 S_5 \leftarrow \frac{A_s}{d \cdot 12} \\
 S_6 \leftarrow \frac{0.9}{12} \cdot A_{sc} \cdot f_{ys} \cdot 1000 \cdot d \cdot \left[1 - 0.6 \cdot \frac{\frac{A_{sc}}{d \cdot 12} \cdot f_{ys} \cdot 1000}{f_c} \right] \\
 S
 \end{cases}$$

Calculate ρ_b and M_b

$$\rho_b := \frac{0.85 \cdot \frac{d}{0.003 + \epsilon_s} \cdot 0.003 \cdot 0.85 \cdot f_c \cdot 12}{f_{ys} \cdot 1000 \cdot 12 \cdot d} \quad \rho_b = 0.029 \quad 100 \cdot \rho_b = 2.851 \%$$

$$M_b := f_{ys} \cdot 1000 \cdot \frac{12}{12} \cdot d \cdot \rho_b \cdot \left[(d) - 0.85 \cdot \frac{d}{0.003 + \epsilon_s} \cdot \frac{0.003}{2} \right]$$

$$M_b = 5.725 \times 10^4 \text{ # ft}$$

Results of the Analysis for Bottom Reinforcement

Required

Spacing of Bars

Area of Steel

Percent of Steel Reinforcement

$$\text{Steel}_4 = 9.299 \text{ in}$$

$$\text{Steel}_3 = 0.396 \text{ in}^2$$

$$\text{Steel}_5 \cdot 100 = 0.493 \%$$

Supplied

Spacing of Bars

Area of Steel

Percent of Steel Reinforcement

$$\text{Steel}_0 = 9 \text{ in}$$

$$\text{Steel}_1 = 0.409 \text{ in}^2$$

$$\text{Steel}_2 \cdot 100 = 0.51 \%$$

$$\phi M_n := \text{Steel}_6$$

$$0.75 \cdot \rho_b \cdot 100 = 2.138 \% \quad \rho \text{ should be less than 75\% of } \rho_b \quad \text{OK}$$

$$\phi M_n = 1.175 \times 10^4 \text{ # ft}$$

$$M_{PUL} = 1.139 \times 10^4 \text{ # ft}$$

$$M_{PUL} < \phi M_n \quad \text{OK}$$

Calculation of Negative Moment (Continuous Slab over Supports)

$$M_{DL} := \frac{w_{DL} \cdot S^2}{10} \quad M_{DL} = 411.6 \text{ # ft} \quad d := t - \frac{d_{fb}}{2} - c_t$$

$$M_{LL} := 0.8 \cdot \frac{LL \cdot S}{4 \cdot L_s} \quad M_{LL} = 3.04 \times 10^3 \text{ # ft}$$

$$M_{IL} := \begin{cases} I \leftarrow \frac{50}{S + 125} \\ M_{IL} \leftarrow M_{LL} \cdot I \text{ if } I < 0.3 \\ M_{IL} \leftarrow M_{LL} \cdot 0.3 \text{ otherwise} \\ M_{IL} \end{cases} \quad M_{IL} = 912 \text{ # ft}$$

Since it is continuous slab with 7 supports (six span) use 80% reduction for live and impact loading

Ultimate Loads

$$M_{UL} := 1.3 \cdot \left[(\beta_D) \cdot M_{DL} + \beta_L \cdot (M_{IL} + M_{LL}) \right]$$

$$M_{UL} = 9.115 \times 10^3 \text{ # ft}$$

Service Loads

$$M_{SL} := M_{DL} + (M_{IL} + M_{LL})$$

$$M_{SL} = 4.364 \times 10^3 \text{ # ft}$$

Use Environmental Reduction factors for FRP bars:

Glass

$$f_{fgu} := 0.7 f_{fg} \quad \epsilon_{fgu} := \frac{f_{fgu}}{E_{fg}}$$

Table 7.1 ACI 440

Calculate ρ_b and M_b

$$\begin{aligned} M &:= c \leftarrow \frac{t - c_t - \frac{d_{fb}}{2}}{0.003 + \epsilon_{fgu}} \cdot 0.003 \\ \epsilon_{st} &\leftarrow \left(c - c_b - \frac{d_{sb}}{2} \right) \cdot \frac{0.003}{c} \\ f_y &\leftarrow -f_{ys} \text{ if } \epsilon_{st} < -\epsilon_s \\ f_y &\leftarrow E_s \cdot \left[\left(c - c_b - \frac{d_{sb}}{2} \right) \cdot \frac{0.003}{c} \right] \text{ if } -\epsilon_s \leq \epsilon_{st} \leq \epsilon_s \\ f_y &\leftarrow f_{ys} \text{ if } \epsilon_{st} > \epsilon_s \\ \rho_b &\leftarrow \frac{(0.85 \cdot f_c \cdot 0.85 \cdot c \cdot 12) + f_y \cdot \text{Steel}_I \cdot 1000}{f_{fgu} \cdot 12 \cdot d \cdot 1000} \\ M_b &\leftarrow f_{fgu} \cdot 12 \cdot d \cdot 1000 \cdot \rho_b \cdot \left(\frac{t}{2} - c_t - \frac{d_{fb}}{2} \right) + f_y \cdot \text{Steel}_I \cdot 1000 \cdot \left(\frac{t}{2} - c_b - \frac{d_{sb}}{2} \right) + 0.85 \cdot c \cdot 0.85 \cdot f_c \cdot 12 \cdot \left(\frac{t}{2} - 0.85 \cdot \frac{c}{2} \right) \\ M_0 &\leftarrow \rho_b \\ M_1 &\leftarrow \frac{M_b}{12} \\ M & \end{aligned}$$

$$\rho_b := M_0$$

$$M_b := M_1$$

Balanced Reinforcement Percentage:

Balanced Moment

$$100 \cdot \rho_b = 1.294 \% \quad A_{fbal} := d \cdot 12 \cdot \rho_b$$

$$M_b = 2.046 \times 10^4 \text{ # ft}$$

$$A_{fbal} = 0.873 \text{ in}^2$$

Required FRP Reinforcement for the Slab considering Flexure:

$$\begin{aligned}
 \text{FRP} := & \left| \begin{aligned}
 & A_{fb} \leftarrow 0.1 \text{ if } (\rho_b \cdot d \cdot 12) < 0 \\
 & A_{fb} \leftarrow \rho_b \cdot d \cdot 12 \text{ otherwise} \\
 & \text{for } A_{fg} \in (A_{fb}), (A_{fb}) + 0.01 \dots 50 \cdot (A_{fb}) \\
 & \left| \begin{aligned}
 & c \leftarrow \frac{t - c_t - \frac{d_{fb}}{2}}{0.003 + \varepsilon_{fgu}} \cdot 0.003 \\
 & c_2 \leftarrow t \\
 & c_1 \leftarrow 0 \\
 & \text{for } i \in 1 \dots 30 \\
 & \left| \begin{aligned}
 & \varepsilon_{st} \leftarrow \left(c - c_b - \frac{d_{sb}}{2} \right) \cdot \frac{0.003}{c} \\
 & f_y \leftarrow -f_{ys} \text{ if } \varepsilon_{st} < -\varepsilon_s \\
 & f_y \leftarrow E_s \cdot \left[\left(c - c_b - \frac{d_{sb}}{2} \right) \cdot \frac{0.003}{c} \right] \text{ if } -\varepsilon_s \leq \varepsilon_{st} \leq \varepsilon_s \\
 & f_y \leftarrow f_{ys} \text{ if } \varepsilon_{st} > \varepsilon_s \\
 & F_s \leftarrow f_y \cdot \text{Steel}_1 \cdot 1000 \\
 & F_{fg} \leftarrow \frac{0.003}{c} \cdot \left(t - c_t - \frac{d_{fb}}{2} - c \right) \cdot E_{fg} \cdot A_{fg} \cdot 1000 \\
 & F_c \leftarrow 0.85 \cdot c \cdot 0.85 \cdot f_c \cdot 12 \\
 & \text{break if } |F_c + F_s - F_{fg}| < 10 \\
 & c_2 \leftarrow c \text{ if } F_c + F_s > F_{fg} \\
 & c_1 \leftarrow c \text{ otherwise} \\
 & c \leftarrow \frac{c_1 + c_2}{2}
 \end{aligned}
 \right. \\
 & M_n \leftarrow \left(\frac{t}{2} - c_t - \frac{d_{fb}}{2} \right) \cdot F_{fg} + F_c \cdot \left(\frac{t}{2} - 0.85 \cdot \frac{c}{2} \right) + \left(\frac{t}{2} - c_b - \frac{d_{sb}}{2} \right) \cdot F_s \\
 & \phi M_n \leftarrow 0.7 \cdot \frac{M_n}{12} \text{ if } A_{fg} > 1.4 \cdot (\rho_b \cdot d \cdot 12) \\
 & \phi M_n \leftarrow \frac{A_{fg}}{(\rho_b \cdot d \cdot 12) \cdot 2} \cdot \frac{M_n}{12} \text{ otherwise} \\
 & \text{break if } \phi M_n > M_{UL} \\
 & M_0 \leftarrow A_{fg} \\
 & M_1 \leftarrow \phi M_n \\
 & s \leftarrow \frac{12}{\frac{A_{fg}}{\left(\frac{\pi \cdot d_{fb}^2}{4} \right)}}
 \end{aligned}
 \right.
 \end{aligned}
 \end{aligned}$$

```

M2 ← s
M3 ← trunc(s) + 0.5 if trunc(s) + 0.5 < s
M3 ← trunc(s) otherwise
M4 ←  $\frac{12}{M_3} \frac{\pi \cdot d_{fb}^2}{4}$ 
M5 ←  $\frac{M_n}{12}$ 
M
Msup := Afg ← FRP4
c ←  $\frac{t - c_t - \frac{d_{fb}}{2}}{0.003 + \varepsilon_{fgu}} \cdot 0.003$ 
c2 ← t
c1 ← 0
for i ∈ 1 .. 30
    est ←  $\left( c - c_b - \frac{d_{sb}}{2} \right) \cdot \frac{0.003}{c}$ 
    fy ← -fys if est < -εs
    fy ← Es ·  $\left[ \left( c - c_b - \frac{d_{sb}}{2} \right) \cdot \frac{0.003}{c} \right]$  if -εs ≤ est ≤ εs
    fy ← fys if est > εs
    Fs ← fy · Steel1 · 1000
    Ffg ←  $\frac{0.003}{c} \cdot \left( t - c_t - \frac{d_{fb}}{2} - c \right) \cdot E_{fg} \cdot A_{fg} \cdot 1000$ 
    Fc ← 0.85 · c · 0.85 · fc · 12
    break if |Fc + Fs - Ffg| < 10
    c2 ← c if Fc + Fs > Ffg
    c1 ← c otherwise
    c ←  $\frac{c_1 + c_2}{2}$ 
Mn ←  $\left( \frac{t}{2} - c_t - \frac{d_{fb}}{2} \right) \cdot F_{fg} + F_c \cdot \left( \frac{t}{2} - 0.85 \cdot \frac{c}{2} \right) + \left( \frac{t}{2} - c_b - \frac{d_{sb}}{2} \right) \cdot F_s$ 
φMn ←  $0.7 \cdot \frac{M_n}{12}$  if Afg > 1.4 · (ρb · d · 12)
φMn ←  $\frac{A_{fg}}{(\rho_b \cdot d \cdot 12) \cdot 2} \cdot \frac{M_n}{12}$  otherwise
φMn

```

RequiredFRP Area (in^2)

$$A_{fb} := FRP_0$$

$$A_{fb} = 0.873 \text{ in}^2$$

$$\rho := \frac{A_{fb}}{b_w \left(t - c_t - \frac{d_{fb}}{2} \right)}$$

$$\rho = 0.013$$

$$\rho \cdot 100 = 1.294 \%$$

Calculated ϕM_n

$$\phi M_n := FRP_1$$

$$\phi M_n = 1.023 \times 10^4 \text{ # ft}$$

Spacing of Bars

$$s := FRP_2$$

$$s = 6.071 \text{ in}$$

$$\frac{\rho}{\rho_b} = 1$$

SuppliedFRP Area (in^2)

$$A_{fb} := FRP_4$$

$$A_{fb} = 0.884 \text{ in}^2$$

$$\rho := \frac{A_{fb}}{b_w \left(t - c_t - \frac{d_{fb}}{2} \right)}$$

Calculated ϕM_n

$$\phi M_n := M_{sup}$$

$$\phi M_n = 1.041 \times 10^4 \text{ # ft}$$

Spacing of Bars

$$s := FRP_3$$

$$s = 6 \text{ in}$$

$$M_{UL} = 9.115 \times 10^3 \text{ # ft}$$

$$\text{Since } M_{UL} < \phi M_n$$

OK

$$\rho = 0.013 \%$$

$$\rho \cdot 100 = 1.309 \%$$

$$\frac{\rho}{\rho_b} = 1.012$$

Serviceability Checks:*Crack Width and Deflection*Calculate M_{cr} and I_{cr} for positive and negative moment regions. E_c modulus of elasticity of concrete (ksi) E_s modulus of elasticity of steel (ksi) E_{fg} modulus elasticity of FRP Bar (ksi) t height of the specimen (in) c_t clear cover (TOP) (in) c_b clear cover (BOTTOM) (in.) b_w width of the specimen (12 in.) d_{sb} diameter of steel bars (in) d_{fb} diameter of FRP bars (in.)

```

CrackingMomentofInertia :=
  Ec ← 57 · √fc
  ns ←  $\frac{E_s}{E_c}$ 
  nf ←  $\frac{E_{fg}}{E_c}$ 
  nAs ← Steel1 · ns
  nAf ← Afb · nf
  for i ∈ 0, 0.0001 .. t
    if  $\left| nA_s \left( t - c_b - \frac{d_{sb}}{2} - i \right) - b_w \cdot \frac{i^2}{2} - nA_f \left( i - c_t - \frac{d_{fb}}{2} \right) \right| \leq 0.1$ 
      Icrp ←  $nA_s \left( t - c_b - \frac{d_{sb}}{2} - i \right)^2 + b_w \cdot \frac{i^3}{4} + \left( \frac{1}{12} \cdot b_w \cdot i^3 \right) + nA_f \left( i - c_t - \frac{d_{fb}}{2} \right)^2$ 
      cp ← i
    I0,0 ← Icrp
  I0,1 ← cp
  for i ∈ 0, 0.001 .. t
    if  $\left| nA_f \left( t - c_t - \frac{d_{fb}}{2} - i \right) - b_w \cdot \frac{i^2}{2} - nA_s \left( i - c_b - \frac{d_{sb}}{2} \right) \right| \leq 0.1$ 
      Icm ←  $nA_f \left( t - c_t - \frac{d_{fb}}{2} - i \right)^2 + b_w \cdot \frac{i^3}{4} + \left( \frac{1}{12} \cdot b_w \cdot i^3 \right) + nA_s \left( i - c_b - \frac{d_{sb}}{2} \right)^2$ 
      cn ← i
    I1,0 ← Icm
  I1,1 ← cn
  I

CrackingMomentofInertia =  $\begin{pmatrix} 102.178 & 1.706 \\ 35.584 & 1.11 \end{pmatrix}$ 

Icrp := CrackingMomentofInertia0,0      Cracking Moment of Inertia at Positive Moment Region
Icm := CrackingMomentofInertia1,0      Cracking Moment of Inertia at Negative Moment Region
ccrp := CrackingMomentofInertia0,1      c at cracking of positive moment (in)
ccm := CrackingMomentofInertia1,1      c at cracking of negative moment (in)

Icrp = 102.178    n4      Cracking Moment of Inertia at Positive Moment Region
Icm = 35.584    in4      Cracking Moment of Inertia at Negative Moment Region
ccrp = 1.706    in      c at cracking of positive moment (in)
ccm = 1.11    in      c at cracking of negative moment (in)

```

$$\begin{aligned}
\text{FlexuralCrackingMoment} &:= \begin{cases} E_c \leftarrow 57 \cdot \sqrt{f'_c} \\ n_s \leftarrow \frac{E_s}{E_c} \\ n_f \leftarrow \frac{E_{fg}}{E_c} \\ nA_s \leftarrow \text{Steel}_1 \cdot n_s \\ nA_f \leftarrow A_{fb} \cdot n_f \\ c \leftarrow \frac{\left(b_w \cdot t \cdot \frac{t}{2}\right) + nA_s \left(t - c_b - \frac{d_{sb}}{2}\right) + nA_f \left(c_t + \frac{d_{fb}}{2}\right)}{b_w \cdot t + nA_s + nA_f} \\ f_r \leftarrow 7.5 \cdot \sqrt{f'_c} \\ I_g \leftarrow \left(\frac{1}{12} \cdot b_w \cdot t^3\right) + b_w \cdot t \cdot \left(\frac{t}{2} - c\right)^2 + nA_f \left[(c) - c_t - \frac{d_{fb}}{2}\right] + nA_s \left[(t) - c - c_b - \frac{d_{sb}}{2}\right] \\ M_{crp} \leftarrow \frac{f_r \cdot I_g}{t - c} \\ M_{crn} \leftarrow \frac{f_r \cdot I_g}{c} \\ M_0 \leftarrow \frac{M_{crp}}{12} \\ M_1 \leftarrow \frac{M_{crn}}{12} \\ M_2 \leftarrow c \\ M_3 \leftarrow I_g \\ M \end{cases}
\end{aligned}$$

$$\text{FlexuralCrackingMoment} = \begin{pmatrix} 5.258 \times 10^3 \\ 5.092 \times 10^3 \\ 4.064 \\ 523.511 \end{pmatrix} \quad \begin{aligned} M_{\text{per}} &:= \text{FlexuralCrackingMoment}_0 \\ M_{\text{ncr}} &:= \text{FlexuralCrackingMoment}_1 \\ I_g &:= \text{FlexuralCrackingMoment}_3 \end{aligned}$$

$$M_{\text{per}} = 5.258 \times 10^3 \text{ \# ft}$$

$$M_{\text{ncr}} = 5.092 \times 10^3 \text{ \# ft}$$

$$I_g = 523.511 \text{ in}^4$$

Displacement Calculations (Assume Simply Supported Beam)

$$I_e := \left[\frac{M_{\text{per}}}{(M_{\text{PSL}})} \right]^3 \cdot I_g + \left[1 - \left[\frac{M_{\text{per}}}{(M_{\text{PSL}})} \right]^3 \right] \cdot I_{\text{crp}} \quad I_e = 479.506 \text{ in}^4$$

$$\text{Displacement} := \Delta \leftarrow \frac{5 \cdot M_{\text{PDL}} \cdot S^2 \cdot 12 \cdot 12}{48 \cdot 57000 \cdot \sqrt{f'_c} \cdot I_e} + \frac{(M_{\text{PLL}} + M_{\text{PIL}}) \cdot S^2 \cdot 12 \cdot 12}{12 \cdot (57000 \cdot \sqrt{f'_c} \cdot I_e)}$$

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$$\text{Displacement} = 0.015$$

Displacement Due to Service Loads (Dead+Live+Impact) (in)

$$\text{AllowableDisplacement} := \frac{S \cdot 12}{360}$$

Limit Short Term Displacement (in) ACI 9.5 (b)

$$\text{AllowableDisplacement} = 0.187 \text{ in}$$

$$\text{Displacement} := \Delta \leftarrow \frac{(M_{PLL} + M_{PIL}) \cdot S^2 \cdot 12 \cdot 12}{12 \cdot (57000 \cdot \sqrt{f_c} \cdot I_e)}$$

$$\text{Displacement} = 0.013$$

Displacement Due to Service Loads (Live+Impact) (in)

$$\text{AllowableDisplacement} := \frac{S \cdot 12}{800}$$

Limit Short Term Displacement (in) 8.9.3 AASHTO

$$\text{AllowableDisplacement} = 0.084 \text{ in}$$

OK

Crackwidth Calculation

For Positive Moment Region

$$\epsilon_1 := \frac{M_{PSL} \cdot \left(t - c_{crp} - c_b - \frac{d_{sb}}{2} \right) \cdot 12}{I_{crp}} \cdot \left(\frac{1}{57 \cdot \sqrt{f_c}} \right)$$

$$\epsilon_2 := \frac{M_{PSL} \cdot (t - c_{crp}) \cdot 12}{I_{crp}} \cdot \left(\frac{1}{57 \cdot \sqrt{f_c}} \right) \quad \beta := \frac{\epsilon_2}{\epsilon_1} \quad \beta = 1.264 \quad d_c := c_b + \frac{d_{sb}}{2}$$

$$A := \pi \cdot \left(c_b + \frac{d_{sb}}{2} \right)^2 \quad f_s := \frac{M_{PSL} \cdot \left(t - c_{crp} - c_b - \frac{d_{sb}}{2} \right) \cdot 12}{I_{crp}} \cdot \left(\frac{1}{57 \cdot \sqrt{f_c}} \right) \cdot E_s$$

$$f_s \cdot \frac{1}{1000} = 25.668 \text{ ksi}$$

$$w_b := 0.076 \cdot \beta \cdot \frac{f_s}{1000} \cdot \left(d_c \cdot A \right)^{\left(\frac{1}{3} \right)}$$

Gergely and Lutz:

$$w_b = 4.738 \text{ in (Mils)}$$

$$w_b := 0.115 \cdot \beta \cdot \frac{f_s}{1000} \cdot (A)^{\left(\frac{1}{4} \right)}$$

Kaar and Mattock:

$$w_b = 5.689 \text{ in (Mils)}$$

$$S_c := 2$$

Frosch:

$$w_c := S_c \cdot \frac{f_s}{E_s \cdot 1000} \cdot \beta \cdot \left[d_c^2 + \left(\frac{\text{Steel}_0}{2} \right)^2 \right]^{\frac{1}{2}}$$

$$w_c \cdot 1000 = 10.484 \text{ in (Mils)}$$

For Negative Moment Region

Gergely and Lutz:

$$e_1 := \frac{M_{SL} \cdot \left(t - c_{cm} - c_t - \frac{d_{fb}}{2} \right) \cdot 12}{I_{cm}} \cdot \left(\frac{1}{57 \cdot \sqrt{f_c}} \right)$$

$$e_2 := \frac{M_{SL} \cdot (t - c_{cm}) \cdot 12}{I_{cm}} \cdot \left(\frac{1}{57 \cdot \sqrt{f_c}} \right)$$

$$\beta := \frac{e_2}{e_1} \quad \beta = 1.526 \quad d_c := c_t + \frac{d_{fb}}{2}$$

$$A := \pi \cdot \left(c_t + \frac{d_{fb}}{2} \right)^2 \quad f_f := \frac{M_{SL} \cdot \left(t - c_{cm} - c_t - \frac{d_{fb}}{2} \right) \cdot 12}{I_{cm}} \cdot \left(\frac{1}{57 \cdot \sqrt{f_c}} \right) \cdot E_{fg}$$

$$\frac{f_f}{1000} = 11.058 \text{ ksi}$$

$$w_b := 0.076 \cdot \beta \cdot \frac{f_f}{1000} \cdot (d_c \cdot A)^{\left(\frac{1}{3} \right)} \cdot \frac{E_s}{E_{fg}} \quad \text{Gergely and Lutz:}$$

$$w_b = 21.562 \text{ in (Mils)}$$

$$w_b := 0.115 \cdot \beta \cdot \frac{f_f}{1000} \cdot (A)^{\left(\frac{1}{4} \right)} \cdot \frac{E_s}{E_{fg}} \quad \text{Kaar and Mattock:}$$

$$w_b = 19.244 \text{ in (Mils)}$$

$$S_c := 2$$

Frosch:

$$w_c := S_c \cdot \frac{f_f}{E_{fg} \cdot 1000} \cdot \beta \cdot \left[d_c^2 + \left(\frac{FRP_3}{2} \right)^2 \right]^{\frac{1}{2}}$$

$$w_c \cdot 1000 = 21.523 \text{ in (Mils)}$$

Shear Calculations: FRP

$$V_{DL} := \frac{w_{DL} \cdot S}{2} \cdot L_s \quad V_{DL} = 2.166 \times 10^3 \text{ lb}$$

$$V_{LL} := \frac{LL}{2} \quad V_{IL} := \frac{IL}{2} \quad V_{LL} = 8 \times 10^3 \text{ lb} \quad V_{IL} := \frac{IL}{2} \text{ lb}$$

$$V_u := 1.3 \left[(\beta_D) \cdot V_{DL} + \beta_L \cdot (V_{LL} + V_{IL}) \right]$$

$$V_u = 2.539 \times 10^4 \text{ lb}$$

$$V_n := 5 \cdot \sqrt{f_c} \cdot \text{CrackingMomentofInertia}_{1,1} \cdot L_s \cdot 12$$

$$V_n = 2.483 \times 10^4 \text{ lb}$$

$$\phi V_n := 0.85 \cdot V_n$$

$$\phi V_n = 2.111 \times 10^4 \text{ lb} \quad \text{Since } \phi V_n < V_u \text{ Shear is not OK based on FRP reinforcement}$$

Shear Calculations: Steel

$$V_n := 5 \cdot \sqrt{f_c} \cdot \text{CrackingMomentofInertia}_{0,1} \cdot L_s \cdot 12$$

$$V_n = 3.817 \times 10^4 \text{ lb}$$

$$\phi V_n := 0.85 \cdot V_n$$

$$\phi V_n = 3.245 \times 10^4 \text{ lb} \quad \text{Since } \phi V_n > V_u \text{ Shear is OK based on Steel reinforcement}$$

Creep Rupture Stress Limits:

$$f_{fs} := \frac{M_{DL} \cdot (t - d_{fb} - \text{CrackingMomentofInertia}_{1,1}) \cdot 12}{\text{CrackingMomentofInertia}_{1,0} \cdot 1000} \cdot \frac{E_{fg}}{E_c}$$

$$f_{fs} = 1.418 \text{ ksi}$$

$$f_s := \frac{0.20}{0.70} \cdot f_{fgu}$$

$$f_s = 16 \text{ ksi}$$

Calculate the Reinforcement Parallel to the Traffic

Method Proposed by Dr. Frosch

$$A_g := t \cdot b_w$$

Temperature Reinforcement:

$$A_s := A_g \cdot 0.0018 \quad \text{Top and Bottom Steel}$$

$$A_s := \begin{cases} 0.125 & \text{if } A_s < 0.125 \\ A_s & \text{otherwise} \end{cases}$$

$$A_s = 0.173$$

$$s_{\text{steel}} := \min \left(\frac{\frac{d_{sb}^2 \cdot \pi}{4}}{A_s}, 12, 3t, 18 \right) \quad \text{Use \#5 Bars}$$

$$s_{\text{steel}} = 18 \quad \text{Assume placed at top}$$

Distribution Reinforcement for bottom mat:

$$\text{Percent} := \min \left(\frac{220}{\sqrt{S}}, 67 \right)$$

$$\text{Percent} = 67$$

$$s_{\text{steel}} := \frac{\text{Steel}_0}{67} \cdot 100$$

$$s_{\text{steel}} = 13.433 \quad \text{in}$$

Calculate FRP spacing:

$$f_{fu} := 80 \text{ ksi}$$

$$A_f := \frac{\left(\frac{A_g \cdot 6 \cdot \sqrt{f_c}}{1000} \right) - \left(\frac{d_{sb}^2 \cdot \pi}{4} \cdot \frac{12}{18} \cdot f_{ys} \right)}{f_{fu}} \quad \text{Use \#5 Bars} \quad d_{fb} := \frac{5}{8}$$

$$A_f = 0.302 \quad \text{in}^2$$

$$s_{\text{FRP}} := \frac{12}{\left(\frac{A_f}{\frac{d_{fb}^2 \cdot \pi}{4}} \right)}$$

$$s_{\text{FRP}} = 12.192 \quad \text{in}$$

FRP Spacing based on ACI 440

$$\rho := 0.0018 \cdot \frac{60000}{f_{fu} \cdot 1000} \cdot \frac{E_s}{E_{fg}} \quad d_{fb} := \frac{5}{8}$$

$$\rho = 6.525 \times 10^{-3}$$

$$A_f := A_g \cdot \rho$$

$$s_{FRP} := \min \left(\frac{\frac{d_{fb}^2 \cdot \pi}{4}}{A_f}, 12, 3t, 12 \right)$$

$$s_{FRP} = 5.877 \quad \mathbf{6 \text{ in. spacing reasonable}}$$

Longitudinal Direction:

$$S_s := 12 \quad \text{in} \quad \text{spacing of steel bars at longitudinal direction} \quad \#5 \text{ Bars}$$

$$S_g := 6 \quad \text{in} \quad \text{spacing of glass bars at longitudinal direction} \quad \#5 \text{ Bars}$$

$$A_s := \frac{d_s^2 \cdot \pi}{4} \cdot \frac{12}{S_s} \quad A_g := \frac{d_g^2 \cdot \pi}{4} \cdot \frac{12}{S_g}$$

$$\% := \frac{A_s + A_g}{12.8} \cdot 100 \quad \% = 0.959 \quad \mathbf{of deck's cross section}$$

$$\begin{aligned} \text{FlexuralCrackingMoment} := & \quad E_c \leftarrow 57 \cdot \sqrt{f_c} \\ & n_s \leftarrow \frac{E_s}{E_c} \\ & n_f \leftarrow \frac{E_{fg}}{E_c} \\ & nA_s \leftarrow A_s \cdot n_s \\ & nA_g \leftarrow A_g \cdot n_f \\ & c \leftarrow \frac{\left(b_w \cdot t \cdot \frac{t}{2} \right) + nA_s \cdot \left(t - c_b - d_{sb} - \frac{d_s}{2} \right) + nA_g \cdot \left(c_t + d_{fb} + \frac{d_g}{2} \right)}{b_w \cdot t + nA_s + nA_g} \\ & f_r \leftarrow 7.5 \cdot \sqrt{f_c} \\ & I_g \leftarrow \left(\frac{1}{12} \cdot b_w \cdot t^3 \right) + b_w \cdot t \cdot \left(\frac{t}{2} - c \right)^2 + nA_g \cdot \left[(c) - c_t - \frac{d_{fb}}{2} \right]^2 + nA_s \cdot \left[(t) - c - c_b - \frac{d_{sb}}{2} \right]^2 \\ & M_{crp} \leftarrow \frac{f_r \cdot I_g}{t - c} \\ & M_{cm} \leftarrow \frac{f_r \cdot I_g}{c} \end{aligned}$$

$$\begin{aligned}
& \left| \begin{array}{l} M_0 \leftarrow \frac{M_{\text{crp}}}{12} \\ M_1 \leftarrow \frac{M_{\text{crm}}}{12} \\ M_2 \leftarrow c \\ M_3 \leftarrow I_g \\ M \end{array} \right. \\
& \text{FlexuralCrackingMoment} = \left(\begin{array}{l} 5.195 \times 10^3 \\ 5.092 \times 10^3 \\ 4.04 \\ 520.453 \end{array} \right) \quad \begin{array}{l} M_{\text{pcr}} := \text{FlexuralCrackingMoment}_0 \\ M_{\text{ncr}} := \text{FlexuralCrackingMoment}_1 \\ I_g := \text{FlexuralCrackingMoment}_3 \end{array} \\
& M_{\text{pcr}} = 5.195 \times 10^3 \text{ \# ft} \\
& M_{\text{ncr}} = 5.092 \times 10^3 \text{ \# ft} \\
& I_g = 520.453 \text{ in}^4 \\
& \text{CrackingMomentofInertia} := \left| \begin{array}{l} E_c \leftarrow 57 \cdot \sqrt{f_c} \\ n_s \leftarrow \frac{E_s}{E_c} \\ n_f \leftarrow \frac{E_{fg}}{E_c} \\ nA_s \leftarrow A_s \cdot n_s \\ nA_g \leftarrow A_g \cdot n_f \\ \text{for } i \in 0, 0.0001 \dots t \\ \quad \text{if } \left| nA_s \cdot \left(t - c_b - d_{sb} - \frac{d_s}{2} - i \right) - b_w \cdot \frac{i^2}{2} - nA_g \cdot \left(i - c_t - d_{fb} - \frac{d_g}{2} \right) \right| \leq 0.1 \\ \quad \quad \left| \begin{array}{l} I_{\text{crp}} \leftarrow nA_s \cdot \left(t - c_b - d_{sb} - \frac{d_s}{2} - i \right)^2 + b_w \cdot \frac{i^3}{4} + \left(\frac{1}{12} \cdot b_w \cdot i^3 \right) + nA_g \cdot \left(i - c_t - d_{fb} - \frac{d_g}{2} \right)^2 \\ cp \leftarrow i \end{array} \right. \\ I_{0,0} \leftarrow I_{\text{crp}} \\ I_{0,1} \leftarrow cp \\ \text{for } i \in 0, 0.001 \dots t \\ \quad \text{if } \left| nA_g \cdot \left(t - c_t - d_{fb} - \frac{d_g}{2} - i \right) - b_w \cdot \frac{i^2}{2} - nA_s \cdot \left(i - c_b - d_{sb} - \frac{d_s}{2} \right) \right| \leq 0.1 \\ \quad \quad \left| \begin{array}{l} I_{\text{crm}} \leftarrow nA_g \cdot \left(t - c_t - d_{fb} - \frac{d_g}{2} - i \right)^2 + b_w \cdot \frac{i^3}{4} + \left(\frac{1}{12} \cdot b_w \cdot i^3 \right) + nA_s \cdot \left(i - c_b - d_{sb} - \frac{d_s}{2} \right)^2 \\ cn \leftarrow i \end{array} \right. \\ I_{1,0} \leftarrow I_{\text{crm}} \\ I_{1,1} \leftarrow cn \\ I \end{array} \right.
\end{aligned}$$

$$\text{CrackingMomentofInertia} = \begin{pmatrix} 66.957 & 1.468 \\ 23.01 & 1.035 \end{pmatrix}$$

$$I_{crp} := \text{CrackingMomentofInertia}_{0,0} \quad \text{Cracking Moment of Inertia at Positive Moment Region}$$

$$I_{cm} := \text{CrackingMomentofInertia}_{1,0} \quad \text{Cracking Moment of Inertia at Negative Moment Region}$$

$$c_{crp} := \text{CrackingMomentofInertia}_{0,1} \quad c \text{ at cracking of positive moment (in)}$$

$$c_{cm} := \text{CrackingMomentofInertia}_{1,1} \quad c \text{ at cracking of negative moment (in)}$$

$$I_{crp} = 66.957 \quad \text{in}^4 \quad \text{Cracking Moment of Inertia at Positive Moment Region}$$

$$I_{cm} = 23.01 \quad \text{in}^4 \quad \text{Cracking Moment of Inertia at Negative Moment Region}$$

$$c_{crp} = 1.468 \quad \text{in} \quad c \text{ at cracking of positive moment (in)}$$

$$c_{cm} = 1.035 \quad \text{in} \quad c \text{ at cracking of negative moment (in)}$$

$$f_t := \frac{M_{ncr} \left(t - c_{cm} - c_t - d_{fb} - \frac{d_g}{2} \right) \cdot 12}{I_{cm}} \cdot \left(\frac{1}{57 \cdot \sqrt{f_c}} \right) \cdot E_{fg}$$

$$\frac{f_t}{1000} = 17.8 \quad \text{ksi}$$

For Negative Moment Region Due to Cracking Moment

Gergely and Lutz:

$$\epsilon_1 := \frac{M_{ncr} \left(t - c_{cm} - c_t - d_{fb} - \frac{d_g}{2} \right) \cdot 12}{I_{cm}} \cdot \left(\frac{1}{57 \cdot \sqrt{f_c}} \right)$$

$$\epsilon_2 := \frac{M_{ncr} (t - c_{cm}) \cdot 12}{I_{cm}} \cdot \left(\frac{1}{57 \cdot \sqrt{f_c}} \right)$$

$$\beta := \frac{\epsilon_2}{\epsilon_1} \quad \beta = 1.729 \quad d_c := c_t + d_{fb} + d_g$$

$$A := \pi \cdot (c_t + d_{fb} + d_g)^2 \quad f_t := \frac{M_{ncr} \left(t - c_{cm} - c_t - d_{fb} - \frac{d_g}{2} \right) \cdot 12}{I_{cm}} \cdot \left(\frac{1}{57 \cdot \sqrt{f_c}} \right) \cdot E_{fg}$$

$$w_b := 0.076 \cdot \beta \cdot \frac{f_f}{1000} \cdot (d_c \cdot A)^{\left(\frac{1}{3}\right)} \cdot \frac{E_s}{E_{fg}} \quad \text{Gergely and Lutz:}$$

$$w_b = 53.823 \quad \text{in (Mils)}$$

$$w_b := 0.115 \cdot \beta \cdot \frac{f_f}{1000} \cdot (A)^{\left(\frac{1}{4}\right)} \cdot \frac{E_s}{E_{fg}} \quad \text{Kaar and Mattock:}$$

$$w_b = 41.066 \quad \text{in (Mils)}$$

$$S_c := 2$$

Frosch:

$$w_c := S_c \cdot \frac{f_f}{E_{fg} \cdot 1000} \cdot \beta \cdot \left[d_c^2 + \left(\frac{FRP_3}{2} \right)^2 \right]^{\frac{1}{2}}$$

$$w_c \cdot 1000 = 45.384 \quad \text{in (Mils)}$$

Steel and FRP stresses at the time of cracking (Axial Tension)

$$\epsilon := \frac{6 \cdot \sqrt{f_c} \cdot 12.8}{A_s \cdot E_s + A_g \cdot E_{fg}}$$

$$\epsilon = 2.896 \times 10^{-3}$$

Steel Stress

$$f_s := \epsilon \cdot E_s \quad f_s = 83.988 \quad \text{ksi}$$

Glass Stress

$$f_g := \epsilon \cdot E_{fg} \quad f_g = 17.377 \quad \text{ksi}$$

Steel and FRP stresses at the negative moment region. Assume composite section behavior.

$$\begin{array}{llll} b_{eff} := 5.6 & A_s := b_{eff} \cdot A_s & \text{ft} & c_b = 1 \\ E_{fg} = 6 \times 10^3 \quad \text{ksi} & A_s = 1.718 & & c_t = 2 \\ E_s = 2.9 \times 10^4 \quad \text{ksi} & A_g := b_{eff} \cdot A_g & n := \frac{E_{fg}}{E_s} & n = 0.207 \\ & A_g = 3.436 & & h_{steelgirder} := 30.31 \quad \text{in} \\ I_{steelgirder} := 5770 \quad \text{in}^4 & A_{steelgirder} := 38.9 \quad \text{in}^2 & & \end{array}$$

Calculate centroid of the section measured from bottom:

$$y := \frac{A_{\text{steelgirder}} \cdot \frac{h_{\text{steelgirder}}}{2} + n \cdot A_g \cdot \left[(h_{\text{steelgirder}}) + t - c_t - d_{fb} - \frac{d_g}{2} \right] + A_s \cdot \left[(h_{\text{steelgirder}}) + c_b + d_{sb} + \frac{d_s}{2} \right]}{A_{\text{steelgirder}} + n \cdot A_g + A_s}$$

$$y = 16.213$$

$$y_1 := \left(h_{\text{steelgirder}} + t - y - c_t - d_{fb} - \frac{d_g}{2} \right)^2$$

$$y_2 := \left(h_{\text{steelgirder}} + c_b + d_{sb} + \frac{d_s}{2} - y \right)^2$$

$$y_3 := \left(y - \frac{h_{\text{steelgirder}}}{2} \right)^2$$

$$I_{tr} := n \cdot A_g \cdot y_1 + A_s \cdot y_2 + I_{\text{steelgirder}} + A_{\text{steelgirder}} \cdot y_3$$

$$I_{tr} = 6.516 \times 10^3 \text{ in}^4$$

Calculate Stresses at FRP reinforcement

$$M := 280.5 \text{ Kip-ft}$$

$$c := h_{\text{steelgirder}} + t - y - c_t - d_{fb} - \frac{d_g}{2}$$

$$\sigma_{frp} := \frac{n \cdot M \cdot c \cdot 12}{I_{tr}}$$

$$\sigma_{frp} = 2.048 \text{ ksi}$$

Calculate Stresses at Steel reinforcement

$$c := h_{\text{steelgirder}} + c_b + d_{sb} + \frac{d_s}{2} - y$$

$$\sigma_{\text{steel}} := \frac{M \cdot c \cdot 12}{I_{tr}}$$

$$\sigma_{\text{steel}} = 9.897 \text{ ksi}$$

Calculate the punching shear capacity

Wheel Contact Area: 10 in. X 20 in.

$$h_2 := 20 \text{ in.}$$

$$h_1 := 10 \text{ in.}$$

$$\beta_c := \frac{h_2}{h_1}$$

$$\alpha_s := 20$$

$$d_1 := t - 1 - \frac{d_{sb}}{2}$$

$$d_2 := t - 1 - d_{sb} - \frac{d_{sb}}{2}$$

$$d_{ave} := \frac{d_1 + d_2}{2}$$

$$d_{ave} = 6.375 \text{ in.}$$

$$b_0 := (h_2 + d_{ave}) \cdot 2 + (h_1 + d_{ave}) \cdot 2$$

$$b_0 = 85.5 \text{ in.}$$

$$V_{c1} := \left(2 + \frac{4}{\beta_c} \right) \cdot \sqrt{f'_c} \cdot b_0 \cdot d_{ave}$$

$$V_{c2} := \left(2 + \frac{\alpha_s \cdot d_{ave}}{b_0} \right) \cdot \sqrt{f'_c} \cdot b_0 \cdot d_{ave}$$

$$V_{c3} := 4 \cdot \sqrt{f'_c} \cdot b_0 \cdot d_{ave}$$

$$V := \frac{\min(V_{c1}, V_{c2}, V_{c3})}{1000}$$

$$V = 120.352 \text{ kips} \quad \phi V := 0.85 \cdot V$$

$$\phi V = 102.299 \text{ kips}$$

$$V_u := 1.3 \cdot \left[(\beta_D) \cdot V_{DL} + \beta_L \cdot (V_{LL} + V_{IL}) \right] \cdot \frac{2}{1000}$$

$$V_u = 50.789 \text{ kips}$$

$$\phi V_n > V_u \text{ Deck is OK for punching shear}$$

Calculate the Splice Length

FRP Bars

Based on 11.3 ACI 440

$$d_b := \frac{5}{8} \text{ in} \quad f_{fu} := 80000 \text{ psi}$$

$$l_{bf} := \frac{d_b \cdot f_{fu}}{2700} \cdot \frac{1}{12} \quad l_{bf} = 1.543 \text{ ft}$$

Modification factor:

Class A Splice = 1.3

Class B Splice = 1.6

No modification factor for concrete cover (Since concrete cover is larger than $2d_b$)

$$L_d := 1.6 \cdot l_{bf}$$

$$L_d = 2.469 \text{ ft} \quad \textbf{Class B Splice}$$

Mosley (2000)

$$l_d := \frac{80000 - 0.31 \cdot 2}{(6.6 \cdot 4000^5)} \cdot \tan\left(\frac{\pi}{180} \cdot 50\right) \cdot \frac{1}{12}$$

$$l_d = 2.163 \text{ ft}$$

Steel Bars

Based on AASHTO

$$l_d := \begin{cases} l_{d1} \leftarrow 0.04 \cdot 0.31 \cdot \frac{60000}{4000^5} \\ l_{d2} \leftarrow 0.0004 \cdot \frac{5}{8} \cdot 60000 \\ 1 \leftarrow \frac{\max(l_{d1}, l_{d2})}{12} \end{cases}$$

$$l_d = 1.25 \text{ ft} \quad (\text{Without Modification Factor})$$

$$l_d := l_d \cdot 1.5 \cdot 0.8 \cdot 1.3$$

$$l_d = 1.95 \text{ ft} \quad (\text{With Modification Factors})$$

Appendix B

Instrumentation Layout

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Figure B.1 Bridge Layout

Figure B.2 Transverse Direction

Figure B.3 Longitudinal Direction

Figure B.4 Span C - Transverse Direction

Gages are located at mid-span C

Figure B.5 Span C- Longitudinal Direction

Figure B.6 Span D - Transverse Direction

Figure B.7 Span D- Longitudinal Direction

Figure B.9 Pier 4 - Transverse Direction

Table B.1 Summary of the Gages

Gages	Deck	Girders	Box
Strain Gages	27	4	4
Embedded Concrete	4		2
Temperature Gages	2	4	2

Appendix C

Gage Readings

Figure C.1 Measured Strain from Gage CFT2

Figure C.2 Measured Strain from Gage CSTB

Figure C.3 Measured Strain from Gage CFT3

Figure C.4 Measured Strain from Gage CSTC

Figure C.5 Measured Strain from Gage CFT4

Figure C.6 Measured Strain from Gage CFL3

Figure C.7 Measured Strain from Gage CCL3

Figure C.8 Measured Strain from Gage CSL3

Figure C.9 Measured Strain from Gage DFT2

Figure C.10 Measured Strain from Gage DSTB

Figure C.11 Measured Strain from Gage DFT3

Figure C.12 Measured Strain from Gage DSTC

Figure C.13 Measured Strain from Gage DFT4

Figure C.14 Measured Strain from Gage DFL3

Figure C.15 Measured Strain from Gage DCL3

Figure C.16 Measured Strain from Gage DSL3

Figure C.17 Measured Strain from Gage 4FL2

Figure C.18 Measured Strain from Gage 4SL2

Figure C.19 Measured Strain from Gage 4GL2t

Figure C.20 Measured Strain from Gage 4GL2b

Figure C.21 Measured Strain from Gage 4FLB

Figure C.22 Measured Strain from Gage 4SLB

Figure C.23 Measured Strain from Gage 4FL3

Figure C.24 Measured Strain from Gage 4CL3

Figure C.25 Measured Strain from Gage 4SL3

Figure C.26 Measured Strain from Gage 4GL3t

Figure C 27 Measured Strain from Gage 4GL3b

Figure C.28 Measured Strain from Gage 4FT2

Figure C.29 Measured Strain from Gage 4STB

Figure C.30 Measured Strain from Gage 4FT3

Figure C.31 Measured Strain from Gage 4STC

Figure C.32 Measured Strain from Gage 4FT4

Figure C.33 Measured Strain from Gage 5FL3

Figure C.34 Measured Strain from Gage 5CL3

Figure C.35 Measured Strain from Gage 5SL3

Figure C.36 Measured Strain from Gage Dummy FRP

Figure C.37 Measured Strain from Gage Dummy Steel

Figure C.38 Measured Strain from Gage Dummy Concrete

Figure C.39 Measured Strain from Gage Dummy FRP Block

Figure C.40 Measured Strain from Gage Dummy Steel Block

Figure C.41 Measured Strain from Gage Dummy Concrete Block

Figure C.42 Measured Temperature from 4TX3F

Figure C.43 Measured Temperature from 4TX3S

Figure C.44 Measured Temperature from 4TX3A

Figure C.45 Measured Temperature from 4TX3t

Figure C.46 Measured Temperature from 4TX3m

Figure C.47 Measured Temperature from 4TX3b

Figure C.48 Measured Temperature from Ambient Box

Figure C.49 Measured Temperature from Ambient Block