TESTING OF FULL-SIZE REINFORCED CONCRETE BEAMS STRENGTHENED WITH FRP COMPOSITES: EXPERIMENTAL RESULTS AND DESIGN METHODS VERIFICATION

Final Report

SPR 387
TESTING OF FULL-SIZE REINFORCED CONCRETE BEAMS STRENGTHENED WITH FRP COMPOSITES: EXPERIMENTAL RESULTS AND DESIGN METHODS VERIFICATION

Final Report

SPR 387

by

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and

Federal Highway Administration
400 Seventh Street SW
Washington, DC 20590

June 2000
In 1997, a load rating of an historic reinforced concrete bridge in Oregon, Horsetail Creek Bridge, indicated substandard shear and moment capacities of the beams. As a result, the Bridge was strengthened with fiber reinforced polymer composites as a means of increasing load-carrying capacity while maintaining the historic appearance. Because composites were a relatively new construction material in infrastructure projects, subsequent tests were conducted to verify the design used on the Bridge. Four full-size beams were constructed to match the dimensions and strength capacity of the Bridge crossbeams as closely as possible. One of these beams was used as the control, while the other three beams were strengthened with various composite configurations including the same configuration used on the Bridge crossbeams. The beams were loaded in third point bending to determine their capacity. The beam strengthened with the same composite design used on the Bridge could not be broken with loading equipment used. Based on the maximum loads applied, the Bridge beams have at least a 50% increase in shear and a 99% increase in moment capacity over the unstrengthened condition. Design calculations show the Bridge beams now exceed the required shear and moment capacities.
# SI* (MODERN METRIC) CONVERSION FACTORS

## APPROXIMATE CONVERSIONS TO SI UNITS

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**NOTE:** Volumes greater than 1000 L shall be shown in \(\text{m}^3\).

## APPROXIMATE CONVERSIONS FROM SI UNITS

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* SI is the symbol for the International System of Measurement

(4-7-94 jbp)
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1.0 INTRODUCTION

1.1 SIGNIFICANCE OF THE RESEARCH

Nearly 40 percent of the bridges in the United States and Canada are structurally deficient (Cooper 1991, FHWA 1993, Rizkalla & Labossiere 1999, FHWA 2000). Structural elements composed of concrete and reinforcing steel are frequently rated as inadequate due to load conditions beyond the capacity of the original designs. In addition, degradation such as corrosion and fatigue has reduced the capacity of many structures. External post-tensioning, addition of steel plating and total replacement have been the traditional methods used to meet the need for increased load capacity.

In recent years, fiber reinforced polymers (FRP) have been used to increase the capacity of reinforced concrete structural elements. Fiber reinforced polymers are typically comprised of high strength fibers (e.g. aramid, carbon, glass) impregnated with an epoxy, polyester, or vinyl ester resin (often termed the matrix). As this study showed, the addition of these materials can dramatically change the load capacity as well as the failure mechanism of reinforced concrete beams.

Experimental studies have been conducted using FRP reinforcing on both beams and columns. Field application of FRP is common, but a complete understanding of the behavior of reinforced concrete (RC) beams retrofitted with FRP is still lacking. This study investigated the bending behavior by way of strain and deflection of full-size beams in more detail than any previously known study.

1.2 HORSETAIL CREEK BRIDGE

The Oregon Department of Transportation (ODOT) is currently undertaking an ongoing effort to load rate all state and local agency owned bridges. Bridge evaluation is required by the Federal Highway Administration, which partially funds state and local bridge construction projects.

The load rating process involves careful inspection and rating of each structural element in a bridge according to prescribed methods. The lowest rated bridge member determines the rating for the bridge. If the bridge is determined deficient, the bridge owner is required to either retrofit, replace, or post the bridge.

Horsetail Creek Bridge, shown in Figures 1.1 and 1.2, is located east of Portland, Oregon along the Historic Columbia River Highway. It was designed and constructed by K.S. Billner and opened to traffic in 1914. The structure is an 18.3 m (60 ft) long simple 3-span reinforced concrete slab-beam-column structure. The length and width of each span is 6.1 m (20 ft). A photograph of the original bridge is shown in Appendix A.
The Horsetail Creek Bridge beams were constructed without shear reinforcement (required by current standards and knowledge of RC beam behavior). Shear reinforcement inhibits the development of diagonal tension cracks (shear cracks). Once formed, these cracks can propagate quickly and result in a sudden failure before full flexural capacity of the beam is achieved. For this reason, a minimal amount of reinforcement (usually steel stirrups) must be provided (ACI 318-99). Adequate spacing in high shear regions enables the reinforcement to effectively mitigate diagonal tension cracking.

Load rating of Horsetail Creek Bridge identified flexural and shear Rating Factors of RF = 0.5 and RF = 0.06, respectively (CH2M HILL, 1997). An RF value less than 1 indicates a deficient structure. The exceptionally low rating factor for shear was due to the lack of shear stirrups, which required the load-rating engineer to use only the concrete section to resist the induced shear forces. The details of the load rating, including selected calculations, are presented in Appendix B. It should be noted that visual inspection revealed minimal signs of distress or environmental degradation. Only a few locations of exposed steel under the bridge railing and curb were visible.
As a consequence of the load rating, the Bridge was strengthened to an HS20 truck loading capacity using glass and carbon FRP. Of the strengthening options considered, FRP provided the required strength improvement and maintained the historic appearance of the Bridge.

1.3 PURPOSE OF THE STUDY

This study examined the increased load capacity as the result of FRP added to inadequate RC beams. In addition, this study investigated the bending behavior of reinforced concrete beams retrofitted with FRP by examining deflection and strain as a function of load. Laboratory testing was conducted on full-size beams that closely represented the Horsetail Creek Bridge beams in order to accomplish the following:

- To verify that the retrofit scheme used to strengthen the Horsetail Creek Bridge was sufficient for the traffic loads; and

- To provide experimental data to validate finite element models being developed in another research project.

A secondary objective was to evaluate the effectiveness of a fiber optic strain sensing system for monitoring strain in FRP strengthened beams. Under a separate study, fiber optic strain sensors were installed on Horsetail Creek Bridge to monitor static, dynamic and long-term load response. This project was part of a continuing effort to use fiber optic sensors for structural health monitoring.
2.0 TEST SETUP

2.1 BEAM CONSTRUCTION AND PROPERTIES

Four full-scale beams with similar geometry and rebar placement as the Horsetail Creek Bridge crossbeams were constructed in the Oregon State University laboratories. Figure 2.1 shows the beam dimensions and the location of the rebar. There were three main flexural steel bars extending the full length and two bars that bent up to reinforce negative moment regions of the beam. Smaller diameter bars were positioned near the compression face of the beam.

Figure 2.1: Position of steel reinforcement in all beams. Dimensions and rebar sizes are in mm.
The beams were designed to match the strength rather than the serviceability of the Horsetail Creek Bridge beams. For load rating purposes, AASHTO specifies the concrete strength of a bridge constructed before 1959 to be 2500 psi (17.2 MPa) and the steel yield stress to be 33,000 psi (228 MPa) (AASHTO, 1994). Concrete and steel are not readily available at these low strength levels. In an effort to construct beams with similar ultimate strength as the Horsetail Creek Bridge beams, reinforcement bars with smaller cross-sectional areas, Table 2.1, were used to account for the higher yield strength of today’s steel. Design calculations for the beams are provided in Appendix B.

Table 2.1: Steel reinforcement details

<table>
<thead>
<tr>
<th>Standard Bar Size</th>
<th>Metric Bar Size</th>
<th>Steel Area</th>
<th>Location of Reinforcement</th>
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<tbody>
<tr>
<td>#5</td>
<td>#16</td>
<td>0.31 in²</td>
<td>Straight and bent steel above elastic neutral axis. Derived from bridge deck reinforcement</td>
</tr>
<tr>
<td>#6</td>
<td>#19</td>
<td>0.44 in²</td>
<td>Bent reinforcement used for positive and negative moment reinforcement.</td>
</tr>
<tr>
<td>#7</td>
<td>#22</td>
<td>0.60 in²</td>
<td>Straight positive moment reinforcement bars present in all bridge beams.</td>
</tr>
</tbody>
</table>

The four beams were cast and cured separately under similar conditions. Type I ready-mix concrete with nominal 28-day strength of 3000 psi (20.7 MPa) and 6 in (152 mm) slump was used. The beams were cast in the same form to ensure the dimensions were as similar as possible. Each beam was cured in a moist condition until removed from the form 7-14 days after pouring. Ambient conditions during casting and curing did not vary significantly from beam to beam.

After curing, three of the four full-size beams were strengthened with FRP. A description of each beam is given in Table 2.2, and the FRP configurations are shown in Figure 2.2. The Control, Flexure-Only, Shear-Only, and Shear and Flexure beams will be referred to as the Control Beam, F-Only Beam, S-Only Beam, and S&F Beam in this report. Table 2.3 shows the material properties used for analysis, which are based on established design values.

Table 2.2: Experimental beam description

<table>
<thead>
<tr>
<th>Beam</th>
<th>Description</th>
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<tbody>
<tr>
<td>Control</td>
<td>Reinforced concrete beam with no shear stirrups and no FRP reinforcement</td>
</tr>
<tr>
<td>Flexure-only</td>
<td>Control beam with added flexural carbon FRP reinforcement</td>
</tr>
<tr>
<td>Shear-only</td>
<td>Control beam with added shear glass FRP reinforcement</td>
</tr>
<tr>
<td>Shear &amp; Flexure</td>
<td>Control beam with added shear and flexural reinforcement</td>
</tr>
</tbody>
</table>

1See also Figure 2.2.
Figure 2.2: FRP-strengthened experimental beams. The flexural and shear FRP composites were wrapped continuously around the bottom of the beam. All dimensions in mm.

Table 2.3: Design material properties

<table>
<thead>
<tr>
<th>Material</th>
<th>Limiting Stress</th>
<th>Limiting Strain</th>
<th>Limit State</th>
<th>Elastic Modulus</th>
</tr>
</thead>
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<tr>
<td>Concrete (Compression)</td>
<td>3000 psi (20.7 MPa)</td>
<td>0.003</td>
<td>Crushing</td>
<td>3120 ksi (21.5 GPa)</td>
</tr>
<tr>
<td>Steel Reinforcement</td>
<td>60 ksi (414 MPa)</td>
<td>0.002</td>
<td>Yielding</td>
<td>29,000 (200 GPa)</td>
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<tr>
<td>Glass FRP</td>
<td>60 ksi (414 MPa)</td>
<td>0.02</td>
<td>Rupture</td>
<td>3000 ksi (20.7 GPa)</td>
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<tr>
<td>Carbon FRP</td>
<td>110 ksi (760 MPa)</td>
<td>0.012</td>
<td>Rupture</td>
<td>9000 ksi (62 GPa)</td>
</tr>
</tbody>
</table>

1Design elastic modulus from $E_c=57,000(f'_c)^{1/2}$. 
2.1.1 Concrete Modulus Determination

Efforts were made to accurately determine the actual elastic moduli of the beams so that a correct estimation of beam stiffness could be made. A correlation was made between pulse velocity and compressive elastic modulus (ASTM 1983, 1994). From this work, it was determined that each beam possessed a slightly different elastic modulus, as shown in Table 2.4. The elastic moduli calculated from cylinder strengths were too high in comparison to the elastic moduli determined from design 28-day strength and the pulse velocity measurements.

Table 2.4: Elastic modulus results from pulse velocity correlation

<table>
<thead>
<tr>
<th>Beam</th>
<th>Average Measured Pulse Velocity (km/s)</th>
<th>Elastic Modulus from Correlation</th>
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<tr>
<td>Control</td>
<td>3.72</td>
<td>2,810,000 psi (19.3 GPa)</td>
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<tr>
<td>Flexure-only</td>
<td>3.53</td>
<td>2,550,000 psi (17.6 GPa)</td>
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<tr>
<td>Shear-only</td>
<td>3.60</td>
<td>2,630,000 psi (18.2 GPa)</td>
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<tr>
<td>Shear &amp; Flexure</td>
<td>3.48</td>
<td>2,480,000 psi (17.1 GPa)</td>
</tr>
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</table>

1 Correlation between ASTM C 469 and ASTM C 597 was conducted.

2.2 TESTING AND DATA COLLECTION

Details about data acquisition and the equipment used are found in Appendices C & D. A summary of the testing and data acquisition methods is presented below.

2.2.1 Beam Loading

All beams were tested in third-point bending as shown in Figure 2.3. No restraint was provided against rotation along any axis. Supports did not provide any fixity aside from friction due to normal forces. Thus, the beams could be analyzed as simply-supported beams. All beams spanned 18 ft (5.49 m) with a shear-span of 6 ft (1.83 m).

A 600 kip (2670 kN), internal-frame, hydraulic press with a load cell was used to load the beams. This machine was designed to compress test specimens by transferring all forces into its own frame. For beams that spanned beyond the frame of the machine (the situation for the beams in this project), the maximum applied force was limited to 160 kip (712 kN). This constraint was not known until after the project was initiated.

2.2.2 Data Collection

Deflection data were collected from three locations using direct current displacement transducers (DCDTs) as shown in Figure 2.3. A dial gauge was placed in the same longitudinal location as DCDT 2 to verify midspan deflection.
Resistance strain gauges with a 2.36 in (60 mm) gauge length were placed at select sites throughout the beam. Strain data were collected at the midspan section and two sections in the shear zone as shown in Figure 2.4. Other important strains were collected as needed. Gauges were placed on the concrete surface, on the FRP surface, or inside the beam on the steel. Fiber optic gauges were installed only on the three FRP-reinforced beams in the positions shown in Figure 2.5. The fiber optic gauges were monitored by Blue Road Research\textsuperscript{1} during the tests.

In order to ensure data collection systems were properly responding to applied loads, three cycles up to 15 kip (67 kN) were made. The load cycling helped to identify “noisy” and inadequate data collection channels in addition to providing more data for finite element models being developed under a separate project.

\textsuperscript{1} 2555 NE 205\textsuperscript{th} Avenue, Fairview, Oregon 97024. See: www.bluerr.com
Cracking was documented during the testing. Only the Control Beam and to a lesser degree, the F-Only Beam, provided a good map of the cracks because the S-Only and S&F Beams were wrapped with FRP laminates on the sides. Appendix C gives a complete description of visible cracking patterns. For this experimental study, crack widths were not measured.

Figure 2.5: Locations of fiber optic strain gauges. Dimensions in mm.
3.0 EXPERIMENTAL RESULTS

3.1 SUMMARY OF LOAD AND DEFLECTION

The Control, F-Only and S-Only beams were loaded to failure. The failure modes are shown in Table 3.1. The S&F Beam was loaded to 160 kip (712 kN), the capacity of the testing equipment, and held for several minutes without failing. The S&F Beam was reloaded to 160 kip (712 kN) with the load points positioned 2 ft (51 mm) apart to increase the applied moment and again held at this load for several minutes. There was no indication of imminent failure.

Table 3.1: Beam failure modes

<table>
<thead>
<tr>
<th>Beam</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>Control</td>
<td>Diagonal tension crack (shear failure)</td>
</tr>
<tr>
<td>Flexure-only</td>
<td>Diagonal tension crack (shear failure)</td>
</tr>
<tr>
<td>Shear-only</td>
<td>Yielding of tension steel followed by crushing of compression concrete after extended deflections</td>
</tr>
<tr>
<td>Shear &amp; Flexure</td>
<td>No failure observed. Believed to be yielding of tension steel followed by crushing of the concrete. FRP rupture might occur after significant deflections due to failure of the concrete</td>
</tr>
</tbody>
</table>

A summary of the capacity and deflection results is presented in Table 3.2. A load of 15 kip (67 kN) was selected for comparing deflection, and hence stiffness, before first significant cracking. First significant cracking is indicated by the sudden change in slope at approximately 20 kip. Stiffness after first significant cracking was calculated from the slope of the load-deflection curve after cracking.

Figures 3.1 to 3.4 show the load vs. deflection plots for the four beams. Midspan deflection for the S-Only Beam went beyond the range of the DCDT2. Consequently, part of the plot is shown as an extrapolated line. For all plots used in this study, the applied moment at the midspan in kip-ft is always three times the applied load in kip based on the relationship $M=PL/3$ where $P$ is $\frac{1}{2}$ the total applied load and $L$ is the span length. The applied moment in kN-m is 0.914 times the load in kN. The applied shear is $1/2$ the applied load.
### Table 3.2: Summary of load and deflection

<table>
<thead>
<tr>
<th>Item</th>
<th>Control</th>
<th>Flexure-Only</th>
<th>Shear-Only</th>
<th>Shear &amp; Flexure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Midspan Deflection at 15 kip (67 kN)</td>
<td>0.0465 in (1.18 mm)</td>
<td>0.0480 in (1.22 mm)</td>
<td>0.0489 in (1.24 mm)</td>
<td>0.0435 in (1.10 mm)</td>
</tr>
<tr>
<td>Stiffness After First Significant Cracking¹</td>
<td>115 kip/in (20.1 kN/mm)</td>
<td>139 kip/in (24.3 kN/m)</td>
<td>134 kip/in (23.5 kN/m)</td>
<td>150 kip/in (26.3 kN/m)</td>
</tr>
<tr>
<td>Midspan Deflection at Steel Yield²</td>
<td>Did Not Yield</td>
<td>Did Not Yield</td>
<td>0.896 in (23 mm)</td>
<td>Did Not Yield</td>
</tr>
<tr>
<td>Maximum Observed Deflection</td>
<td>0.963 in (24.5 mm)</td>
<td>1.193 in (30.3 mm)</td>
<td>1.390 in (35 mm)³</td>
<td>1.000 in (25 mm)</td>
</tr>
<tr>
<td>Midspan Deflection at Failure</td>
<td>0.963 in (24.5 mm)</td>
<td>1.193 in (30.3 mm)</td>
<td>2.00 in (51 mm)³</td>
<td>Did Not Fail⁴</td>
</tr>
<tr>
<td>Load at First Significant Cracking¹</td>
<td>17.6 kip (78.3 kN)</td>
<td>21.7 kip (96.5 kN)</td>
<td>19.7 kip (87.6 kN)</td>
<td>21.6 kip (96.1 kN)</td>
</tr>
<tr>
<td>Load at Failure</td>
<td>107 kip (476 kN)</td>
<td>155 kip (689 kN)</td>
<td>155 kip (689 kN)</td>
<td>Did Not Fail⁴</td>
</tr>
<tr>
<td>Applied Moment at Yield²</td>
<td>Did Not Yield</td>
<td>Did Not Yield</td>
<td>360 kip-ft (488 kN-m)</td>
<td>Did Not Yield</td>
</tr>
<tr>
<td>Maximum Applied Moment⁴</td>
<td>321 kip-ft (435 kN-m)</td>
<td>465 kip-ft (630 kN-m)</td>
<td>465 kip-ft (630 kN-m)</td>
<td>480 kip-ft (651 kN-m)⁵</td>
</tr>
<tr>
<td>Maximum Applied Shear</td>
<td>53.5 kip (234 kN)</td>
<td>77.5 kip (345 kN)</td>
<td>77.5 kip (345 kN)</td>
<td>80.0 kip (356 kN)</td>
</tr>
</tbody>
</table>

¹ First significant cracking is indicated by the first slope change of the load-deflection plot.
² Primary tension reinforcement only yielded in the S-Only Beam.
³ Extrapolated.
⁴ S&F Beam was not loaded to failure due to equipment limitations.
⁵ A second loading of the S&F Beam achieved a total applied moment of 640 kip-ft (868 kN-m).
Figure 3.1: Load vs. deflection for the Control Beam

Figure 3.2: Load vs. deflection for the Flexure-Only Beam
Figure 3.3: Load vs. deflection for the Shear-Only Beam

Figure 3.4: Load vs. deflection for the S&F Beam (beam did not fail)
3.2 STRAIN DATA

Appendix C presents the load vs. strain data. Figures 3.5 to 3.8 provide midspan strain as a function of load for the four beams. The steel yielding in the S-Only Beam is indicated in Figure 3.7. Figure 3.8 shows the strain in the tension steel reinforcement of the S&F Beam had just exceeded the design limit strain of 0.002. Consequently, the anticipated failure mode for the S&F Beam was flexural failure characterized by steel yielding followed by concrete crushing.
Figure 3.7: S-Only Beam load vs. strain at midspan

Figure 3.8: S&F Beam load vs. strain at midspan
4.0 INTERPRETATION AND DISCUSSION OF EXPERIMENTAL RESULTS

4.1 GAINS OVER THE CONTROL BEAM

Table 4.1 and Figure 4.1 compare the experimental results of the four beams. Important observations include the following:

• The F-Only and S-Only beams had the same increase in load, 45% greater than the Control Beam, but failed in different modes.

• The first load test of the S&F Beam revealed at least a 50% increase in load and moment capacity. The second load test showed that the S&F Beam had at least 99% greater moment capacity than the Control Beam.

• Post cracking stiffness increased up to 30% with the FRP strengthening.

• The addition of FRP in shear and flexure both independently and as a combined system allowed for greater deflections at failure.

• All reinforced beams cracked at higher loads than the unstrengthened Control Beam.

Post cracking stiffness was increased as a result of FRP application. The flexural CFRP produced the greatest effect; however, the addition of GFRP for shear reinforcement also increased the stiffness of the beam nearly as much as the CFRP. If the CFRP wrapped part way up the sides were not present, the GFRP may have provided the larger effect. If the CFRP on the sides were not present, the stiffness increase due to the two composite systems may have been additive to give the stiffness of the S&F Beam. This effect was not investigated. However, it is believed that the stiffness increase was the result of the FRP reducing the width of cracks in the concrete.

It is important to realize that the Control Beam failed in shear before reaching its flexural capacity. Consequently, the capacity increases observed in the FRP-strengthened beams would not have been as significant if the Control Beam had been deficient in only flexure. However, the S&F Beam showed increased capacity compared to the S-Only Beam, a beam with adequate shear strength. This agrees with results from other researchers (GangaRao and Vijay 1998, Rostasy, et. al. 1992, Ritchie, et. al. 1991, Saadatmenesh and Ehsani 1991) that FRP is effective in strengthening flexurally deficient beams.

The deflection and strain at failure increased in the FRP-strengthened beams. Again, this occurred because the Control Beam had inadequate flexural and shear reinforcement initially. If
designed improperly, the addition of CFRP for flexure may increase the stiffness and decrease the deflection.

Table 4.1: Comparison of the strengthened beams to the Control Beam

<table>
<thead>
<tr>
<th>Item</th>
<th>Control Beam Data</th>
<th>Percent Gain Over Control Beam1</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>F-Only</td>
</tr>
<tr>
<td>Midspan Deflection at 15 kip (67 kN)</td>
<td>0.0465 in (1.18 mm)</td>
<td>3.2%</td>
</tr>
<tr>
<td>Post Cracking Stiffness</td>
<td>115 kip/in (20.1 kN/mm)</td>
<td>21%</td>
</tr>
<tr>
<td>Maximum Observed Deflection</td>
<td>0.963 in (24.5 mm)</td>
<td>24%</td>
</tr>
<tr>
<td>Midspan Deflection at Failure</td>
<td>0.963 in (24.5 mm)</td>
<td>24%</td>
</tr>
<tr>
<td>Load at Failure</td>
<td>107 kip (476 kN)</td>
<td>45%</td>
</tr>
<tr>
<td>Load at First Significant Cracking</td>
<td>17.6 kip (78.3 kN)</td>
<td>23%</td>
</tr>
<tr>
<td>Maximum Applied Shear</td>
<td>53.5 kip (238 kN)</td>
<td>45%</td>
</tr>
<tr>
<td>Maximum Applied Moment</td>
<td>321 kip-ft (435 kN-m)</td>
<td>45%</td>
</tr>
</tbody>
</table>

1 0% means equivalent to Control Beam results. Negative means lower than Control Beam results.
2 Based on extrapolated deflection value.
3 Based on the maximum applied load. Beam did not fail.
4 Second load test of the S&F Beam reached a total applied moment of 640 kip-ft or 99% higher than the Control Beam.

Figure 4.1: Load-deflection comparison of all experimental beams
4.2 MEETING THE TRUCK TRAFFIC LOADS

4.2.1 Moment Demand

Values from the load rating calculations performed by CH2M HILL and TAMS Consultants (CH2M HILL, 1997) are given in Table 4.2. These values are used in the following analysis for calculating the required capacity of the Horsetail Creek Bridge crossbeams.

The total factored load to be resisted by the applied live and dead loads is

\[ M_u = \gamma_D M_{DL} + 1.3 \gamma_L (1+I) M_{LL} \]  \hspace{1cm} [4-1]

where

\[ \gamma_D = 1.2, \]
\[ \gamma_L = 1.3 \]
\[ I = 0.10 \]

such that: \[ M_u = 1.2 \times (82.3 + 25.0) + 1.3 \times 1.10 \times 225, \] or \[ M_u = 451 \text{ kip-ft (611 kN-m)} \]

To determine the required capacity of the fully reinforced element, the moment is divided by the strength reduction factor \( \phi = 0.85 \) such that,

\[ M_n = M_u / \phi = 451 / 0.85 \]
\[ M_n = 531 \text{ kip-ft (720 kN-m)} \]

Thus, the fully-reinforced, full-size beam should have supported at least a total applied moment of 531 kip-ft (720 kN-m). In third-point loading, this moment was not achievable with the given testing equipment. The maximum applied third-point moment was 480 kip-ft (651 kN-m).

To confirm that the beam was adequate to reach this moment capacity and to potentially fail the beam, the S&F Beam was reloaded with the load points closer to the beam midspan. This loading produced a moment of 640 ft-kip (868 kN-m). According to the conservative design method adopted for the Bridge and shown in Appendix E, the S&F Beam moment capacity was 590 kip-ft (887 kN-m).
Table 4.2: Calculations from load rating (LRFD)

<table>
<thead>
<tr>
<th>Item</th>
<th>Quantity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moment @ midspan from bridge dead load</td>
<td>82.3 ft-kip (112 kN-m)</td>
</tr>
<tr>
<td>Moment @ midspan from wearing surface dead load</td>
<td>25.0 ft-kip (33.9 kN-m)</td>
</tr>
<tr>
<td>Maximum live load moment @ midspan from an HS20 truck</td>
<td>225 ft-kip (305 kN-m)</td>
</tr>
<tr>
<td>Shear @ critical section from bridge dead load</td>
<td>14.4 kip (64.1 kN)</td>
</tr>
<tr>
<td>Shear @ critical section from wearing surface dead load</td>
<td>4.50 kip (20.0 kN)</td>
</tr>
<tr>
<td>Live load shear @ critical section from HS20 truck</td>
<td>46.5 kip (207 kN)</td>
</tr>
</tbody>
</table>

4.2.2 Shear Demand

Similar calculations to those provided in the above discussion show the total factored shear force to be,

\[ V_u = \gamma_D V_{DL} + 1.3 \gamma_L (1+I) V_{LL} \quad \text{[4-2]} \]

where

\[ \gamma_D = 1.2, \]
\[ \gamma_L = 1.3 \text{ and} \]
\[ I = 0.10 \]

such that: \( V_u = 1.2*(14.4+4.50) + 1.3*1.10*46.5 \), or \( V_u = 83.1 \text{ kip (370 kN)} \)

To determine the required capacity of the Horsetail Creek Bridge crossbeam, the required strength is divided by the reduction factor \( \phi = 0.85 \) such that,

\[ V_n = V_u / \phi = 83.1 / 0.85 \]
\[ V_n = 97.8 \text{ kip (435 kN)} \]

The maximum shear force near the supports achieved during testing was \( \frac{1}{2} \) of 160 kip or 80 kip (356 kN). The actual capacity of the beam was not verified in shear, although conservative calculations based on the design method outlined in Appendix E showed the capacity to be 107 kip (476 kN).
The required, pre-strengthened, and post-strengthened bridge capacities based on calculations are shown in Table 4.3. Testing of the S&F Beam verified that the strengthened Horsetail Creek Bridge beams have at least the required moment capacity. Since the S&F Beam test had to be stopped before reaching the required shear load level of 98 kip (436kN), the moment capacity of the Horsetail Creek Bridge beams was not verified. However, conservative design calculations indicate the shear capacity of the Horsetail Creek Bridge beams should be 107 kip (476 kN). It should be noted that the small differences in S&F Beam design values given above and the Horsetail Creek Bridge design values shown in the table are due to the difference in concrete properties (Table E-2) used in the calculations.

Table 4.3: Capacities of the full-size beams and the Horsetail Creek Bridge crossbeams. The values shown for the full-size beams are measured values. The values for Horsetail Creek Bridge are calculated values.

<table>
<thead>
<tr>
<th>Failure Mode</th>
<th>Control Shear</th>
<th>F-Only Shear</th>
<th>S-Only Flexure</th>
<th>S&amp;F1 Expect flexure</th>
<th>Horsetail Creek Bridge Required2</th>
<th>Before Strengthening3</th>
<th>After Strengthening4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear Capacity, kip (kN)</td>
<td>54 (240)</td>
<td>78 (347)</td>
<td>N/A</td>
<td>&gt;80 (356)</td>
<td>98 (436)</td>
<td>34 (151)</td>
<td>107 (476)</td>
</tr>
<tr>
<td>Moment Capacity, kip-ft (kN-m)</td>
<td>N/A</td>
<td>N/A</td>
<td>465 (630)</td>
<td>&gt;640 (868)</td>
<td>531 (720)</td>
<td>341 (462)</td>
<td>569 (771)</td>
</tr>
</tbody>
</table>

1Beam did not fail. Values shown are based on maximum levels applied during the test.
2Based on Load and Resistance Factor Method.
3Based on Ultimate Strength Design method.
4Based on design method outlined in Appendix E.
5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 CONCLUSIONS

- The unstrengthened Horsetail Creek Bridge crossbeams would have failed in shear at approximately 53 kip (236 kN) shearing force. The beams were substantially deficient in shear based on conventional calculations that showed the dead and live load shear acting on the bridge was 65.4 kip (291 kN).

- The strengthened Horsetail Creek Bridge crossbeams, which are retrofitted with both the GFRP for shear and CFRP for flexure, have at least 50% more static load shear capacity over the unstrengthened beams. The test had to be stopped at an applied shear of 80 kip (356kN) due to equipment limitations before reaching the 98 kip (436 kN) level required by traffic loads.

- The strengthened Horsetail Creek Bridge crossbeams have at least 99% more static load moment capacity than the unstrengthened beams. The fully reinforced beam exceeded the demand of 531 kip-ft (720 kN-m) by sustaining up to 640 kip-ft (868 kN-m) applied moment.

- The strengthened Horsetail Creek Bridge crossbeams are 30% stiffer than the unstrengthened beams.

- Horsetail Creek Bridge crossbeams retrofitted with only the flexural CFRP would still result in diagonal tension failure albeit at a more substantial load of 155 kip (689 kN). The CFRP was wrapped up the sides a sufficient amount to provide resistance across the diagonal tension crack. In addition, the increased stiffness provided by the CFRP decreased the deformation and offset cracking by reducing strain in the beam. However, this load increase should not be relied upon in design.

- Horsetail Creek Bridge crossbeams retrofitted only with the GFRP for shear would fail in flexure at the midspan at 155 kip (689 kN). Yielding of the main flexural steel would initiate prior to crushing of the concrete

- The addition of GFRP for shear was sufficient to offset the lack of stirrups and cause conventional RC beam failure by steel yielding at the midspan. This allowed ultimate deflections to be 200% higher than the shear deficient Control Beam, which failed due to a diagonal tension crack.

- Load at first significant crack was increased, primarily due to the added stiffness of the flexural CFRP, by approximately 23%. The added stiffness reduced the deflections, which in turn reduced the strains and stresses in the cross section for a given load.
5.2 RECOMMENDATIONS

The S&F Beam should be loaded to failure to determine the capacity and verify the failure mode of the strengthened Horsetail Creek Bridge crossbeams.
6.0 REFERENCES


ACI. 1995. Building Code Requirements for Structural Concrete: ACI 318-95. American Concrete Institute, Committee 318.


APPENDIX A: BRIDGE DRAWINGS AND PHOTOS
Figure A-1: Bridge location

Figure A-2: Horsetail Creek Bridge
Figure A-3: Bridge during retrofit

Figure A-4: Typical formwork w/ steel

Figure A-5: Strain gauge application

Figure A-6: Replicated steel reinforcing
A-5

Figure A-13: GFRP application to test beams

Figure A-14: CFRP application to test beam

Figure A-15: In situ epoxy mixing

Figure A-16: CFRP application to test beam

Figure A-17: Second coat of epoxy/CFRP

Figure A-18: Completed carbon reinforcement
Figure A-19: Testing of Control beam

Figure A-20: Early cracking of Control beam

Figure A-21: Control beam shear sections

Figure A-22: Shear failure of Control beam

Figure A-23: Shear failure through sections

Figure A-24: Completed shear failure
Figure A-25: Crack pattern around failure

Figure A-28: Flexure-only beam testing

Figure A-26: Support point: rotation at failure

Figure A-29: Failure of Flexure-only beam

Figure A-27: Load point: crushing at failure

Figure A-30: Diagonal tension failure
Figure A-31: Failure similar to Control beam

Figure A-32: CFRP transverse rupture

Figure A-33: Fiber optic

Figure A-34: Cracking at failure

Figure A-35: Flexure-only beam at failure
Figure A-36: Load point at failure

Figure A-37: Debonding of CFRP at support

Figure A-38: Fiber optic comparison gauges

Figure A-39: Overall view of Shear-only test
Figure A-40: Loading of Shear-only beam

Figure A-41: Failure of Shear-only

Figure A-42: Concrete crushing at midspan

Figure A-43: Shear-only beam support point

Figure A-44: S&F deflection under high load
Figure A-45: Visible S&F beam deflections

Figure A-46: S&F beam shear sections

Figure A-47: Maximum loading of S&F beam

Figure A-48: Increase moment max. loading
APPENDIX B: CALCULATIONS FOR LOAD RATING AND DESIGN OF EXPERIMENTAL BEAMS

Load Rating Calculations

For any structural element resisting forces on a bridge the Rating Factor (RF) is defined as

\[
RF = \frac{\phi R_n - \gamma_{DL}(D L)}{(D F)\gamma_{DL}(LL)(1+I)}
\]  

[B-1]

This equation originates from the American Association of State and Highway Transportation Officials, Manual for Condition Evaluation of Bridges (AASHTO, 1994) and Guide Specifications for Strength Evaluation of Existing Steel and Concrete Bridges (AASHTO, 1989). These specifications establish the way in which state and local agency bridges are evaluated. For reinforced concrete beams, Load and Resistance Factor Design (LRFD) is used, as is apparent in equation [B-1]. A description of variables is given in Table B-1. According to this method, if the RF for a specific element is less than 1.0, then the capacity of that element is considered inadequate for the conditions.

CH2M HILL in conjunction with TAMS Consultants (CH2M HILL, 1997) performed a load rating for Horsetail Falls Bridge. The analysis is shown below.

Table B-1: Load Equation Rating Variables

<table>
<thead>
<tr>
<th>Variable</th>
<th>Description</th>
<th>Horsetail Shear Load Rating Value</th>
<th>Horsetail Moment Load Rating Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>R_n</td>
<td>Nominal capacity of the structural member (e.g. shear or moment capacity)</td>
<td>V_n = 31.2 kip (V_n = 37.2 kip)†</td>
<td>M_n = 341 ft-kip</td>
</tr>
<tr>
<td>φ</td>
<td>Strength reduction factor.</td>
<td>0.85</td>
<td>0.90</td>
</tr>
<tr>
<td>γ_{DL}, γ_{LL}</td>
<td>Dead and live load factors, respectively.</td>
<td>1.20, 1.30</td>
<td>1.20, 1.30</td>
</tr>
<tr>
<td>D L, L L</td>
<td>Maximum dead and live load effects as calculated from the analysis.</td>
<td>See calcs.</td>
<td>See calcs.</td>
</tr>
<tr>
<td>I</td>
<td>Impact factor to account for uncertainty in dynamic loading.</td>
<td>0.10</td>
<td>0.10</td>
</tr>
<tr>
<td>DF</td>
<td>Distribution factor which accounts for wheel distribution per lane. These are essentially influence ordinates.</td>
<td>Single = 0.767, Multiple = 1.033</td>
<td>Single = 3.50, Multiple = 5.00</td>
</tr>
</tbody>
</table>

† Nominal shear capacity using the more detailed shear capacity equations. Not used here.

Load Rating of Horsetail for Flexural Capacity

The applied load configuration used in the load rating of HCB crossbeams is shown in Fig. B-1. An HS20 legal load truck was used in the analysis (32-kip axle).

The nominal moment capacity of the HCB crossbeams based on conventional reinforced concrete beam theory with tension reinforcement only is
\[ M_n = A_s f_y (d - a/2) \quad \text{[B-2]} \]

where \[ a = (A_s f_y)/(0.85 f_c' b) \quad \text{[B-3]} \]

This is a close approximation provided the beam is ductile (steel yields before crushing of the concrete). The properties for the crossbeams were assumed according to unknown material properties for bridges built before 1959 and in lieu of testing (AASHTO, 1989, Ch. 6). Using information from the original plans:

\[ a = (5.00 \text{in}^2 \times 33,000 \text{psi})/(0.85 \times 2500 \text{psi} \times 12 \text{in}) \]

\[ a = 6.471 \text{-in} \]

\[ M_n = (5.00 \text{in}^2 \times 33,000 \text{psi}) \times (28.0 \text{in} - (6.471 \text{in}/2)) \]

\[ M_n = 4,086,200 \text{ lb-in} = 341 \text{ kip-ft} \]

![Figure B-1: Truck position to induce maximum positive-moment influence in crossbeams (simply supported)](image)

From Figure B-1, the total positive moment influence from the two trucks positioned on the crossbeam is calculated by the influence ordinates. For this arrangement, the total influence is the sum of 0.7917, 3.792, 4.208, 1.208 and divided by two since there are two lanes. Thus the distribution factor is 5.0. Using the moment capacity, the calculated live and dead loads (CH2M Hill, 1997) and the load rating equation [B-1], the rating factor for positive moment is

\[ RF_{\text{flexure}} = \frac{(0.85 \times 341 \text{ ft - kip}) - (1.20 \times 107 \text{ ft - kip})}{(5.0) \times (1.30 \times 45.0 \text{ ft - kip}) \times (1.10)} \]

or

\[ RF_{\text{flexure}} = \frac{(0.9 \times 341 \text{ ft - kip}) - (1.20 \times 107 \text{ ft - kip})}{(1.033) \times (1.10) \times (1.30) \times (240.66 \text{ ft - kip})} \]

\[ RF_{\text{flexure}} = 0.50 \]
Load Rating of Horsetail for Shear Capacity

The live load distribution used in the load rating of HCB crossbeams for shear is shown in Fig. B-2. Again, an HS20 legal load truck was used in the analysis. The vehicle was positioned to induce the maximum shear on the beams.

![Figure B-2: Truck positioning to induced maximum shear influence in crossbeams (simply supported)](image)

The nominal shear capacity of HCB crossbeams, using the gross concrete section (typical AASHTO or ACI, see ACI 318/95, Eq. 11-3) is,

\[ V_c = 2.0(\sqrt{f'_c})(b_wd) \]  

The equation assumes that the steel reinforcement did not contribute to shear strength. This assumption was based on the fact that there were no shear stirrups in the beams. A more detailed calculation can be performed, but it is not presented here. For the Horsetail Creek Bridge crossbeams,

\[ V_c = (2.0)(\sqrt{2500\text{psi}})(12\text{in}*26.0\text{in}) = 31,200\text{lb} \]

\[ V_c = 31.2 \text{ kip} \]

In accordance with Figure B-2, the applied shear load due to live loads \( V_{\text{appl}} \) is as follows:

\[ V_{\text{appl}} = (22.5 \text{ kips})*(0.92 +0.62+0.42+0.12) = 46.80 \text{ kips}, \]

where: 0.92; 0.62; 0.42 and 0.12 are the shear influence line ordinates under each of the design vehicle axes.

Using this shear capacity, the load rating equation [B-1], the predetermined dead loads (CH2M HILL, 1997) and the associated factors in Table B-1, the rating factor for shear is

\[ RF_{\text{shear}} = \frac{(0.85)*(31.2 \text{ kip}) - (1.20)*(18.9 \text{ kip})}{(1.033)*(1.30)*(46.8 \text{ kip})*(1.10)} \]
RF_{shear} = 0.06

Since the rating factor is much lower than one, the deficient member requires immediate attention. Such a low rating suggests that the crossbeams should have shown significant distress. However, this is mainly a result of the ultra conservative load rating evaluation procedure adopted by AASHTO and ACI, which does not necessarily represent the real load capacity. Horsetail Creek Bridge did not show any visible signs of structural distress.

**Development of Similar beams for testing**

**Rationale**

The mechanical properties of the steel reinforcement used in Horsetail Creek Bridge beams were unknown. It is believed that the steel with which the bridge was constructed has a yield stress of approximately 33 ksi. For bridges constructed before 1959, AASHTO suggests using 33 ksi for the yield strength of the steel reinforcement if the steel cannot be tested (AASHTO, 1994). Current construction methods typically require steel with 60 ksi yield strength. Acquiring steel with yield strength less than 60 ksi is quite difficult. To achieve a 33 ksi yield strength, a special order of steel would have been required, which would have been too expensive for this study. Thus, a reevaluation of the beam strength and serviceability criteria was necessary.

**Structural Issues**

Regarding reinforced concrete design, there are two important issues of concern: strength and serviceability.

**Strength**

There are two design philosophies governing the way a member is designed for safety: Load and Resistance Factor Design (LRFD) and Allowable Stress Design (ASD). LRFD emphasizes on adequate prediction of the member strength and factoring the loads along with the predicted strength. This is the predominant design method used for reinforced concrete. ASD uses more strength-of-materials (mechanics) relationships to calculate the stress developed in a member than does LRFD. Prescribed limits of stress are established and the designer must ensure that these stresses are not achieved. LRFD is not currently utilized in the design of FRP strengthened RC beams. However, due to the more realistic and less conservative predictions of the method, LRFD concepts were adopted and adapted to develop design criteria for this study. For development of the full-scale beams, strength criteria were considered important.

**Serviceability**

Serviceability refers to the day-to-day performance of the structural member and must be assured at service load levels, not at ultimate strength. Prescribed limits are established, such as maximum permissible crack widths and deflections. Due to the nature of the conducted experiments, serviceability was not a major concern in designing the full-scale beams.
Horsetail Creek Bridge Beams Prior to Strengthening

There are two types of primary bending elements in HCB: crossbeams (orthogonal to traffic) and longitudinal beams (parallel to traffic). Prior to strengthening, the only structural difference between the two beam types was that the crossbeams had one more 1 in² flexural rebar than the longitudinal beams. Consequently, the crossbeams had a slightly higher capacity in bending. Load rating showed the crossbeams had a lower shear rating factor. For this reason, the experimental beams were designed after the crossbeams. The beam dimensions and steel reinforcement positions for the crossbeams are shown in Figures 2-1 and 2-2. There were no shear steel stirrups, which are now required by current standards.

Matching Moment Capacity

The critical section for any flexural loading of the beam is likely to be near the midspan. In an effort to keep the full-size beams as close to the original as possible, the number of steel reinforcement bars and locations, the estimated concrete strength, and the beam dimensions remained the same. The only parameter that was changed in order to match capacity with the original beam was the cross-sectional area of the flexural steel reinforcement. The calculations for determining the required steel cross-sectional area are given below. These calculations neglect the 5/8-in square bars near the top, which were found have little affect.

The moment capacity, \( M_n \), was approximately

\[
M_n = A_s f_y (d-a/2)
\]  \[B-5\]

where “a” is the equivalent rectangular Whitney stress block (Whitney, 1956). This condition is only true, provided the steel yields before the concrete crushes at the top compression fibers. The balance steel ratio, \( \rho_b \), is the ratio where simultaneous yielding of the steel and crushing of the concrete occurs (Nilson, 1997). For the pre-strengthened HCB beams,

\[
\rho_b = 0.85 \beta_1 \frac{f''_c}{f_y} \frac{87,000}{87,000 + f_y}
\]  \[B-6\]

If the steel ratio of the beam is lower than this value, yielding of the tension steel precedes crushing of the concrete. Hence,

\[
\rho_b = 0.85 (0.85) \frac{2500}{33,000} \frac{87,000}{33,000 + 33,000} = 0.0397
\]

\[
\rho = A_s / bd = 5.00 \text{ in}^2 / (12 \text{ in} * 28 \text{ in}) = 0.0149
\]

---


2 Midspan refers to the section at the geometric center between two support points, that is \( \frac{1}{2} \) the span length.
Since the steel ratio was below the balanced ratio, the steel yields first. For the pre-strengthened HCB beams, the equivalent stress block “a” was approximated by

\[ a = \frac{A_s f_y}{0.85 f_e' b} \]  \hspace{1cm} [B-7]

\[ a = \frac{(5.00 \text{ in}^2)(33 \text{ ksi})}{(0.85)(2.5 \text{ ksi})(12 \text{ in})} = 6.471 \text{ in} \]

Then, from equation [B-2]

\[ M_n = (5.00 \text{ in}^2)(33 \text{ ksi})(28-6.471/2) = 4086 \text{ kip-in} \]

\[ M_n = 341 \text{ kip-ft} \]

Since the geometry of the beam was to be retained as closely as possible, the area of steel was reduced to offset the increased yield strength. To do this, the tension force developed in the steel reinforcement was matched, such that

\[ A_s f_y = (5 \text{ in}^2)(33 \text{ ksi}) = 165 \text{ kip} \]  \hspace{1cm} [B-8]

Since the full-size beams were to be made using steel with \( f_y = 60 \text{ ksi} \) then,

\[ A_{s,\text{new}} = 165 \text{ kip}/60 \text{ ksi} = 2.75 \text{ in}^2 \]

Since steel reinforcing is fabricated in specific sizes, a reasonable combination of five bars in the same location was needed. Two #6 rebar and three #7 rebar provided a steel area of 2.68 in\(^2\). Using this combination of reinforcement, the new moment capacity was calculated by,

\[ a = \frac{(2.68 \text{ in}^2)(60 \text{ ksi})}{(0.85)(2.5 \text{ ksi})(12 \text{ in})} = 6.306 \text{ in} \]

\[ M_n = (2.68 \text{ in}^2)(60 \text{ ksi})(27.75-6.306/2) = 3955 \text{ kip-in} \]

\[ M_n = 330 \text{ kip-ft} \]
References


American Concrete Institute (ACI), 1995. Building Code and Commentary for Structural Concrete Design, ACI, Place of Publication.


## Notation

### Table B-2: Appendix B Notation

<table>
<thead>
<tr>
<th>Variable</th>
<th>Description</th>
<th>US Standard Units†</th>
<th>Metric Units†</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>Equivalent Whitney stress block depth converted from the depth to the neutral axis</td>
<td>in</td>
<td>mm</td>
</tr>
<tr>
<td>$A_s$</td>
<td>Area of primary tension reinforcing steel</td>
<td>in$^2$</td>
<td>mm$^2$</td>
</tr>
<tr>
<td>$A_{s,new}$</td>
<td>New area of primary tension reinforcing steel, converted for new tension reinforcement</td>
<td>in$^2$</td>
<td>mm$^2$</td>
</tr>
<tr>
<td>b</td>
<td>Compression flange/block width</td>
<td>in</td>
<td>mm</td>
</tr>
<tr>
<td>$b_w$</td>
<td>Web width of the beam</td>
<td>in</td>
<td>mm</td>
</tr>
<tr>
<td>d</td>
<td>Structural depth of the primary steel reinforcing from the top compression fibers in the beam</td>
<td>in</td>
<td>mm</td>
</tr>
<tr>
<td>DF</td>
<td>Distribution factor which accounts for wheel distribution per lane. In this analysis, these are influence ordinates.</td>
<td>~</td>
<td>~</td>
</tr>
<tr>
<td>DL, LL</td>
<td>Maximum applied dead and live load, respectively</td>
<td>Varies</td>
<td>Varies</td>
</tr>
<tr>
<td>$f_c'$</td>
<td>28-day specified compressive strength of the concrete</td>
<td>psi</td>
<td>kPa</td>
</tr>
<tr>
<td>$f_y$</td>
<td>Steel reinforcing yield stress</td>
<td>ksi</td>
<td>MPa</td>
</tr>
<tr>
<td>I</td>
<td>Impact factor</td>
<td>~</td>
<td>~</td>
</tr>
<tr>
<td>$M_n$</td>
<td>Nominal moment capacity</td>
<td>kip-ft</td>
<td>kN-m</td>
</tr>
<tr>
<td>RF</td>
<td>Rating factor of the structural element</td>
<td>~</td>
<td>~</td>
</tr>
<tr>
<td>RF$_{flexure}$</td>
<td>Rating factor in flexure</td>
<td>~</td>
<td>~</td>
</tr>
<tr>
<td>$R_n$</td>
<td>General nominal structural capacity</td>
<td>~</td>
<td>~</td>
</tr>
<tr>
<td>$V_c$</td>
<td>Shear capacity of the concrete section</td>
<td>kip</td>
<td>kN</td>
</tr>
<tr>
<td>$V_n$</td>
<td>Nominal shear capacity</td>
<td>kip</td>
<td>kN</td>
</tr>
<tr>
<td>$V_s$</td>
<td>Shear capacity of the steel stirrups</td>
<td>kip</td>
<td>kN</td>
</tr>
<tr>
<td>$\beta_1$</td>
<td>~</td>
<td>~</td>
<td>~</td>
</tr>
<tr>
<td>$\phi$</td>
<td>Strength reduction factor</td>
<td>~</td>
<td>~</td>
</tr>
<tr>
<td>$\gamma_{DL}, \gamma_{LL}$</td>
<td>Dead and live load factors, respectively</td>
<td>~</td>
<td>~</td>
</tr>
<tr>
<td>$\rho_b$</td>
<td>Balance steel ratio where simultaneous crushing of the concrete would occur with yielding of the tension steel.</td>
<td>~</td>
<td>~</td>
</tr>
</tbody>
</table>

† Typical units presented. The use of “~” implies the variable has no units.
APPENDIX C: EXPERIMENTAL DATA
APPENDIX C: EXPERIMENTAL DATA

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Figure C-1: Control Beam deflection characteristics
Figure C-2: Control Beam strain at 1067 mm from beam end

Figure C-3: Control Beam strain at 1500 mm from beam end
Figure C-4: Control Beam compressive strain comparison

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Figure C-6: Control Beam evidence of shear crack formation

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Figure C-8: Flexure-Only Beam deflection characteristics

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Figure C-11: Flexure-Only Beam compressive strain comparison
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Figure C-13: Flexure-Only Beam early tensile strain comparison
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Figure C-16: Flexure-Only Beam failure by shear crack formation

Figure C-16: Shear-Only Beam deflection characteristics
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Figure C-20: Shear-Only Beam tensile strain comparison
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Figure C-23: S&F Beam deflection characteristics

Figure C-24: S&F Beam strain 1067 mm from beam end
Figure C-25: S&F Beam strain 1500 mm from beam end

Figure C-26: S&F Beam compressive strain comparison
Figure C-27: S&F Beam tensile strain comparison

Table C-1: Resistance strain gauge identification

<table>
<thead>
<tr>
<th>Gauge I.D.</th>
<th>Coordinate Location</th>
<th>Gauge Description / Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>1STL1</td>
<td>1067 106 667</td>
<td>Bent #19 rebar on C-D face of beam at the 1067 mm section and located in a horizontal orientation</td>
</tr>
<tr>
<td>1STL2</td>
<td>1067 127 508</td>
<td>Straight #16 rebar at the 1067 mm section</td>
</tr>
<tr>
<td>1STL3</td>
<td>1500 106 384</td>
<td>Bent #19 rebar on C-D face of beam at the 1500 mm section, located at 45 degree orientation and at midheight</td>
</tr>
<tr>
<td>1STL4</td>
<td>1500 127 508</td>
<td>Straight #16 rebar at the 1500 mm section</td>
</tr>
<tr>
<td>1CON5</td>
<td>1067 0 384</td>
<td>Midheight of 1067 mm section on the C-D face</td>
</tr>
<tr>
<td>1CON6</td>
<td>1500 0 384</td>
<td>Midheight of 1500 mm section on the C-D face</td>
</tr>
<tr>
<td>1STL7</td>
<td>1500 49 51</td>
<td>#22 rebar at the 1500 mm section closest to C-D face</td>
</tr>
<tr>
<td>1STL8</td>
<td>1067 49 51</td>
<td>#22 rebar at the 1067 mm section closest to C-D face</td>
</tr>
<tr>
<td>1CON9</td>
<td>1500 152 0</td>
<td>Beam bottom at the 1500 mm section</td>
</tr>
<tr>
<td>1CON10</td>
<td>1067 152 0</td>
<td>Beam bottom at the 1067 mm section</td>
</tr>
<tr>
<td>1STL11</td>
<td>3048 127 508</td>
<td>Straight #16 rebar at the midspan section</td>
</tr>
<tr>
<td>1CON13</td>
<td>3048 152 768</td>
<td>Beam top at the midspan section</td>
</tr>
<tr>
<td>Gauge I.D.</td>
<td>Coordinate Location</td>
<td>Gauge Description / Notes</td>
</tr>
<tr>
<td>-----------</td>
<td>---------------------</td>
<td>--------------------------</td>
</tr>
<tr>
<td>1CON14</td>
<td>3048 0 384</td>
<td>Midheight at the midspan section</td>
</tr>
<tr>
<td>1CON15</td>
<td>1500 152 768</td>
<td>Beam top at the 1500 mm section</td>
</tr>
<tr>
<td>1CON16</td>
<td>1067 152 768</td>
<td>Beam top at the 1067 mm section</td>
</tr>
<tr>
<td>1CON17</td>
<td>3048 152 0</td>
<td>Beam bottom at the midspan section</td>
</tr>
<tr>
<td>1STL18</td>
<td>3048 49 51</td>
<td>#22 rebar at the midspan section closest to C-D face</td>
</tr>
<tr>
<td>2CON1</td>
<td>1067 0 384</td>
<td>Beam midheight at the 1067 mm section</td>
</tr>
<tr>
<td>2FRP2</td>
<td>1067 152 -t_FRP</td>
<td>Beam bottom at the 1067 mm section</td>
</tr>
<tr>
<td>2FRP3</td>
<td>1500 152 -t_FRP</td>
<td>Beam bottom at the 1500 mm section</td>
</tr>
<tr>
<td>2STL5</td>
<td>1500 106 384</td>
<td>Bent #19 rebar on the A-D end at the midheight of the 1500 mm section (closest to C-D face, oriented at 45 degrees)</td>
</tr>
<tr>
<td>2STL7</td>
<td>3048 127 508</td>
<td>Straight #16 rebar at the midspan section</td>
</tr>
<tr>
<td>2FRP8</td>
<td>3048 -t_FRP 102-t_FRP</td>
<td>Side of beam at the midspan section located 102 mm from the bottom face of the CFRP surface</td>
</tr>
<tr>
<td>2STL9</td>
<td>4597 106 384</td>
<td>Bent #19 rebar at the midheight 4597 mm from the B-C end (closest to C-D face, oriented at 45 degrees)</td>
</tr>
<tr>
<td>2FRP10</td>
<td>3048 152 -t_FRP</td>
<td>Beam bottom at the midspan section</td>
</tr>
<tr>
<td>2STL11</td>
<td>3048 152 51</td>
<td>#22 rebar at the midspan section (center bar of the three #22 rebars)</td>
</tr>
<tr>
<td>2CON12</td>
<td>3048 0 384</td>
<td>Beam midheight at the midspan section</td>
</tr>
<tr>
<td>2CON13</td>
<td>3048 152 768</td>
<td>Beam top at the midspan section</td>
</tr>
<tr>
<td>2SHRMID14</td>
<td></td>
<td>Middle gauge of 3 used for comparison to fiber optic shear gauges, see details</td>
</tr>
<tr>
<td>2SHRLOW15</td>
<td></td>
<td>Low gauge of 3 used for comparison to fiber optic shear gauges, see details</td>
</tr>
<tr>
<td>2SHRHIG16</td>
<td></td>
<td>High gauge of 3 used for comparison to fiber optic shear gauges, see details</td>
</tr>
<tr>
<td>2CON17</td>
<td>1500 152 768</td>
<td>Beam top at the 1500 mm section</td>
</tr>
<tr>
<td>2CON18</td>
<td>1067 152 768</td>
<td>Beam top at the 1067 mm section</td>
</tr>
<tr>
<td>2CON19</td>
<td>1500 0 384</td>
<td>Beam midheight at the 1500 mm section</td>
</tr>
<tr>
<td>3CON1</td>
<td>1500 152 768</td>
<td>Beam top at the 1500 mm section</td>
</tr>
<tr>
<td>3CON2</td>
<td>1067 152 768</td>
<td>Beam top at the 1067 mm section</td>
</tr>
<tr>
<td>3STL3</td>
<td>1500 106 384</td>
<td>Bent #19 rebar on the A-D end at the midheight of the 1500 mm section (closest to C-D face, oriented at 45 degrees)</td>
</tr>
<tr>
<td>3FRP4</td>
<td>2134 -t_FRP 692</td>
<td>Located 76 mm from the top surface of beam under the load point on the C-D face and D end, located on the FRP surface and oriented horizontal</td>
</tr>
<tr>
<td>Gauge I.D.¹</td>
<td>Coordinate Location²</td>
<td>Gauge Description / Notes</td>
</tr>
<tr>
<td>------------</td>
<td>----------------------</td>
<td>--------------------------</td>
</tr>
<tr>
<td>3FRP5</td>
<td>2845 -tFRP 692</td>
<td>Located 76 mm from the top surface of beam near midspan (203 mm from the midspan back toward A-D end, located on the FRP surface and oriented horizontal, located over an anchor)</td>
</tr>
<tr>
<td>3FRP6</td>
<td>610 305+tFRP 76-tFRP</td>
<td>Vertically oriented gauge (only one this beam) located 610 mm from the A-D end of the beam on the A-B face and 76 mm from the beam bottom of GFRP</td>
</tr>
<tr>
<td>3STL7</td>
<td>3048 152 51</td>
<td>#22 rebar at the midspan section (center bar of the three #22 rebars)</td>
</tr>
<tr>
<td>3STL8</td>
<td>3048 106 51</td>
<td>Bent #19 rebar at the midspan section (closest to C-D face) oriented in a horizontal fashion</td>
</tr>
<tr>
<td>3FRP9</td>
<td>2845 235 -tFRP</td>
<td>Beam bottom on FRP located 203 mm back from centerline toward A-D end and 70 mm from the A-B face</td>
</tr>
<tr>
<td>3CON10</td>
<td>3048 152 0</td>
<td>Beam bottom at the midspan section</td>
</tr>
<tr>
<td>3CON11</td>
<td>3048 0 384</td>
<td>Beam midheight at the midspan section</td>
</tr>
<tr>
<td>3STL12</td>
<td>3048 127 508</td>
<td>Straight #16 rebar at the midspan section</td>
</tr>
<tr>
<td>3CON13</td>
<td>3048 152 768</td>
<td>Beam top at the midspan section</td>
</tr>
<tr>
<td>3FRP14</td>
<td>1473 -tFRP 384</td>
<td>Beam midheight at the 1500 mm section (25 mm toward the A-D end since the joint did not allow placement)</td>
</tr>
<tr>
<td>3FRP15</td>
<td>1067 -tFRP 384</td>
<td>Beam midheight at the 1067 mm section</td>
</tr>
<tr>
<td>3FRP16</td>
<td>610 -tFRP 76-tFRP</td>
<td>Horizontally oriented gauge located 610 mm from the A-D end of the beam on the C-D face and 76 mm from the beam bottom of GFRP, opposite of gauge 6</td>
</tr>
<tr>
<td>3FRP17</td>
<td>1500 152 -tFRP</td>
<td>Beam bottom at the 1500 mm section</td>
</tr>
<tr>
<td>3FRP18</td>
<td>1067 152 -tFRP</td>
<td>Beam bottom at the 1067 mm section</td>
</tr>
<tr>
<td>3FRP19</td>
<td>305 -tFRP 76-tFRP</td>
<td>Located directly above the support on the A-D end on the C-D face, oriented horizontally 75 mm from the support face, similar to gauge 4</td>
</tr>
<tr>
<td>4FRP1</td>
<td>1500 -tFRP 384</td>
<td>Beam midheight at the 1500 mm section (on a joint)</td>
</tr>
<tr>
<td>4FRP2</td>
<td>610 -tFRP 76-tFRP</td>
<td>Horizontally oriented gauge located 610 mm from the A-D end of the beam on the C-D face and 75 mm from the beam bottom of GFRP, opposite of gauge 6</td>
</tr>
<tr>
<td>4CON3</td>
<td>1500 152 768</td>
<td>Beam top at the 1500 mm section</td>
</tr>
<tr>
<td>4FRP4</td>
<td>1067 -tFRP 384</td>
<td>Beam midheight at the 1067 mm section</td>
</tr>
<tr>
<td>4FRP5</td>
<td>610 305+tFRP 3-tFRP</td>
<td>Vertically oriented gauge (only for this beam) located 610 mm from the A-D end of the beam on the A-B face and 75 mm from the beam bottom of GFRP</td>
</tr>
<tr>
<td>4CON6</td>
<td>1067 152 768</td>
<td>Beam top at the 1067 mm section</td>
</tr>
<tr>
<td>4CON7</td>
<td>3048 152 768</td>
<td>Beam top at the midspan section</td>
</tr>
<tr>
<td>Gauge I.D.¹</td>
<td>Coordinate Location²</td>
<td>X (mm)</td>
</tr>
<tr>
<td>------------</td>
<td>----------------------</td>
<td>--------</td>
</tr>
<tr>
<td>4STL8</td>
<td></td>
<td>3048</td>
</tr>
<tr>
<td>4STL9</td>
<td></td>
<td>3048</td>
</tr>
<tr>
<td>4STL10</td>
<td></td>
<td>3048</td>
</tr>
<tr>
<td>4STL11</td>
<td></td>
<td>4597</td>
</tr>
<tr>
<td>4CON12</td>
<td></td>
<td>3048</td>
</tr>
<tr>
<td>4CON13</td>
<td></td>
<td>3048</td>
</tr>
<tr>
<td>4FRP14</td>
<td></td>
<td>3048</td>
</tr>
<tr>
<td>4FRP15</td>
<td></td>
<td>3048</td>
</tr>
<tr>
<td>4FRP16</td>
<td></td>
<td>2134</td>
</tr>
<tr>
<td>4FRP17</td>
<td></td>
<td>3048</td>
</tr>
<tr>
<td>4FRP18</td>
<td></td>
<td>1067</td>
</tr>
<tr>
<td>4FRP19</td>
<td></td>
<td>1500</td>
</tr>
</tbody>
</table>

1. The first number in the gauge I.D. is the beam number (1=control, 2=Flexure-only, 3=Shear-only and 4 = Shear & Flexural FRP reinforced beam). The second part is the material that the gauge is applied to (e.g. STL = gauge on the steel reinforcing; CON=gauge applied to exterior concrete surface). The last number is the gauge number for that experimental beam.
2. Coordinates are measured from lower, right-hand corner in Figure C-28. X is distance along the beam span, Y is distance through the depth, and Z is the vertical distance.
3. The designation tFRP is the thickness of the FRP reinforcement at that location. It is shown subtracted from or added to some coordinates to correctly fix the location of the gauges relative to the surface of the concrete. The thickness of the reinforcement varies with position on the beam.
Figure C-28: Common resistance strain gauge locations (dimensions in mm)

Figure C-29: Gauge type comparison—S&F Beam
Figure C-30: Strain from flexural fiber optic strain gauges—S&F Beam

Figure C-31: Strain from shear fiber optic strain gauges embedded in concrete—S&F Beam
Figure C-32: Comparison of shear fiber optic strain gauges embedded in concrete and on top of the FRP reinforcement—S&F beam

Table C-2: Fiber optic strain gauge identification

<table>
<thead>
<tr>
<th>Gauge I.D.</th>
<th>Location</th>
<th>Gauge Description / Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>4FOSCON-02</td>
<td>0.70 m A-B A-D</td>
<td>Shear gauge embedded in concrete</td>
</tr>
<tr>
<td>4FOSCON-03</td>
<td>0.70 m C-D B-C</td>
<td>Shear gauge embedded in concrete</td>
</tr>
<tr>
<td>4FOSCON-04</td>
<td>0.70 m C-D A-D</td>
<td>Shear gauge embedded in concrete</td>
</tr>
<tr>
<td>4FOSCON-05</td>
<td>0.70 m A-B B-C</td>
<td>Shear gauge embedded in concrete</td>
</tr>
<tr>
<td>4FOSFRP-F5</td>
<td>0.70 m C-D B-C</td>
<td>Shear gauge over FRP reinforcement</td>
</tr>
<tr>
<td>4FOSFRP-12</td>
<td>0.70 m A-B A-D</td>
<td>Shear gauge over FRP reinforcement</td>
</tr>
<tr>
<td>4FOFCON-24</td>
<td>1.00 m Bottom B-C</td>
<td>Flexure gauge embedded in concrete</td>
</tr>
<tr>
<td>4FOFFRP-26</td>
<td>1.00 m Bottom A-D</td>
<td>Flexure gauge over FRP reinforcement</td>
</tr>
<tr>
<td>4FOFCON-27</td>
<td>1.00 m Bottom Midspan</td>
<td>Flexure gauge embedded in concrete</td>
</tr>
<tr>
<td>4FOFCON-28</td>
<td>1.00 m Bottom Midspan</td>
<td>Flexure gauge over FRP reinforcement</td>
</tr>
</tbody>
</table>

1. The first number in the gauge I.D. is the beam number (4 = Shear & Flexural FRP reinforced beam). The first three letters are the strain intention (e.g. FOF = gauge intended to collect flexural strain at the beam bottom; FOS = gauge intended to collect strain in the high shear region). The second three letters are the material which the gauge is applied to (e.g. FRP = gauge on the exterior of the FRP composite; CON = gauge embedded in concrete surface, mostly under the FRP). The last number is the gauge number for that experimental beam.

2. See Figure C-33.
Crack Patterns

Flexural cracks are located near the midspan and are oriented nearly perpendicular to the long axis of the beam. Shear cracks are diagonal cracks that appear in the shear span (i.e. between the load and support points). A more appropriate term is diagonal tension cracks or inclined cracking. These cracks are the result of combined bending and shear forces realigning the principal tension direction (recall Mohr’s circle of stress). The cracks only occur when bending forces are restrained and inclined cracking is unrestrained. A diagonal tension crack that propagates through the entire beam results in shear failure. Shear crack and diagonal tension crack are used interchangeably in the following discussion.

Cracking was thoroughly mapped in the Control and F-Only beams. The S-Only and S&F beam cracks were mostly concealed under the FRP reinforcement.

Control Beam Cracking

For this beam test, load was briefly held steady at selected load levels to document cracking. Cracking of the Control Beam followed expected behavior. Loading from zero to 15 kip (66.7 kN) did not produce any notable cracking. First cracks appeared around 18 kip (80 kN) near the midspan. These flexural cracks increased in length and quantity up to approximately 60 kips (267 kN) at which time the first evidence of shear cracks was visible. The critical shear cracks did not completely develop until near the ultimate load of 107 kip (476 kN). Critical shear cracks were fully visible on both ends of the beam. Ultimately, one crack propagated from the support to the load point at approximately a 45° angle resulting in failure. Figure C-34 shows cracking before ultimate load and the diagonal tension crack responsible for failure. Both high shear regions of the beam developed shear cracks, but the critical crack occurred on the A-D end of the Control beam.

Flexure-Only Beam Cracking

Flexure-Only Beam cracking patterns were similar to the Control Beam. This similarity was anticipated. As was observed with the Control Beam, a critical diagonal tension crack resulted in
beam collapse. This critical shear crack, shown in Fig. C-35, developed at a higher load (approx. 80 kip, 356 kN) than the Control Beam. A complete assessment of cracking is not possible since the CFRP covered the main portion of the beam where tension cracks developed. Visible cracks during the test were fewer in number and did not appear to propagate as high as the Control Beam test. The shear crack that developed in the F-only Beam was visibly wider than the crack in the Control Beam. This is likely due to the additional resistance provided by the CFRP allowing extended deflection beyond the formation of the diagonal tension crack. The failing crack formed on the B-C end of the beam.

**Shear-only Beam Cracking**

Very little evidence of cracking could be seen through the GFRP on the S-only beam. When cracking did affect the glass FRP composite, it was visible as a color change (whitening of the epoxy). This is shown in Fig. C-36. These tension cracks occurred just prior to ultimate load. Since the composite is unidirectional (vertical fibers only) these vertical cracks do little to reduce the vertical shear strength provided by the FRP (unless numerous cracks cause debonding). Ultimately, the concrete at the top-midspan crushed.

**Shear & Flexure Beam Cracking**

The only evidence of cracking on the fully strengthened beam was at the midspan. These tension cracks were slightly audible and visible at about 120 kip (534 kN). As the load approached the machine limit, these cracks only increased in length. Even during the second loading, cracking was only visible at the midspan section.

A comparison of the crack patterns is shown in Figure C-37. No effort was made to size the cracks for this study.

![Figure C-34: Control Beam cracking (a) and crack responsible for failure (b)](image-url)
Figure C-35: Diagonal tension crack responsible for failure of F-Only Beam

Figure C-36: S-Only Beam (a) and flexural cracks (darkened for contrast) in GFRP near ultimate load (b)
Figure C-37: Comparison of beam cracking (dimensions in mm)
Beam Failure Modes

Shear Failure of Control Beam

The Control Beam was deficient in shear as expected based on the load rating calculations shown in Appendix B. The Control Beam exhibited classical diagonal tension failure. It is possible that the designer/engineer of Horsetail Creek Bridge anticipated diagonal tension cracking, thus bending two of the five flexural bars through the high shear zone. It is more likely, however, that the bent steel was simply intended for negative moment reinforcement over the columns. The bent bars provided minimal reinforcement once the diagonal tension crack initiated.

Shear Failure of Flexure-Only Beam

The F-Only Beam failed in shear but at a higher load than the Control Beam. Since shear reinforcement was absent, the addition of CFRP for flexure was not expected to add shear strength. From a conventional design standpoint, horizontal structural components are not used to resist diagonal tension cracking. Diagonal tension cracks were visible at slightly elevated load levels over the Control Beam. However, since the CFRP was wrapped up the sides (see Figure 2-2), the CFRP was able to equilibrate forces across the diagonal tension cracks. In addition, the CFRP also increased the beams flexural rigidity reducing the strain for any given load in comparison to the Control Beam.

The CFRP fibers were able to maintain integrity of the beam in the presence of the shear crack. Since the fiber orientation was horizontal, the vertical strain component eventually reached a level that failed the matrix of the composite on the side of the beam. The shear cracks were then able to propagate completely through the beam.

It would be advantageous to apply a composite with strength in two principal directions to provide horizontal and vertical strength. The most effective resistance to diagonal tension cracks would be an FRP with its principal direction oriented orthogonal to the crack (aligned with the principal tension strains). The difficulty is predicting the beam response, since a composite with uniquely directional properties will be applied to a beam supposedly homogeneous and isotropic. To simplify the analysis considerably two separate systems might be applied to strengthen the beam. One system for flexure and one for shear (i.e. one horizontal and one vertical like the web and the flange in an I-beam). For construction simplicity, a single bi-directional system with orthogonal fibers could be used.

Flexural strengthening should not be used to increase the design shear capacity (although it was observed to). Predictability of this behavior is not reliable. If a moment deficiency exists, moment strengthening should be performed and vice versa for shear deficiency. For design, the F-Only Beam would have the same strength as the Control Beam and less than the S-Only Beam. Experimentally, it has an equivalent strength as the S-only beam. It was experimentally studied to examine the independent effect of flexural reinforcement with CFRP.
Flexural Failure of the Shear-Only Beam

The S-Only Beam showed the desired failure mode of a properly designed reinforced concrete beam. The GFRP reduced or eliminated the diagonal tension cracking and forced the beam into flexural failure at the midspan section. Figure 3-3 shows the main flexural steel yielded at approximately 120 kip (534 kN). The resulting reduction in flexural rigidity caused a rapid increase in deflection. Ultimately, the concrete crushed at the top midspan. A considerable amount of “ductility” was present in the S-Only Beam as indicated by the extended deflections occurring after the steel yielded. A good design must ensure that shear strength is always in excess of the flexural strength.

Failure of the Shear & Flexure Beam

The S&F Beam was loaded to and held at the capacity of the testing system, 160 kip (712kN). This loading configuration produced an applied moment of 480 kip-ft (651kN-m). A second loading was conducted with the load points closer together to produce an applied moment of 640 kip-ft (868 kN-m). Deflections were visible, but the beam did not exhibit signs of failure. The load was held for approximately 5 minutes with no indication of steel yielding, concrete crushing, or increased deflection.

Calculations indicate (see Appendix E) that the beam would be limited by crushing of the concrete. Concrete strains at the top-midspan location were approaching 0.0015 at the maximum applied load (refer to Figure C-26). Strains in the CFRP reinforcement at midspan were approaching 0.003 and strains in the main tension reinforcing steel were slightly in excess of 0.002. This is clear evidence to the projected failing sequence of the beam in which the steel yields, extended deflections result, the concrete crushes, and the FRP ruptures from substantial deflections.

Fiber Optic Strain Data

Much of the data collected from the fiber optic strain gauges were not useful for analyses required in this project. This was the result of two specific shortcomings: the 700mm and 1000mm gauge lengths were too long, and the +/-15 microstrain resolution of the instrumentation was not sensitive enough to discern lower strain levels.

One particular example of the gauge length problem is illustrated in Figures C-14, C-38, and C-39. A fiber optic sensor with a 27.6 in (700 mm) gauge length was situated in the shear zone. Three resistance gauges were positioned along the fiber optic sensor as shown. Analyses have shown that a strain gradient would exist at the end of the beam similar to the one shown in Figure C-39. Though the beam test had a shear crack and point loading, a strain gradient would have been present. Indeed, the strains shown in Figures C-14 and C-38 from the three resistance gauges qualitatively agree with Figure C-39. However, the results from the fiber optic strain sensor are an average over the long gauge length, which can not show the strain gradient.
Fiber optic strain sensors are expected to play an important role in structural monitoring. For the strain sensors based on Bragg gratings (the type used in this project), the gauge length is easily varied from about 20mm to over 1500mm. In addition, recent advances in instrumentation have increased the sensitivity to less than one microstrain.
APPENDIX D: EQUIPMENT SPECIFICATIONS
APPENDIX D: EQUIPMENT SPECIFICATIONS

Sensor Equipment

Resistance Strain Gauges

Since strains were monitored on the internal steel reinforcement, on the concrete surface and on the surface of the FRP an appropriate gauge needed to be selected. To meet the needs of uniformity and economy, wire resistance gauges of 60-mm “active” length were chosen. These concrete-specific gauges were of sufficient length to integrate across aggregate non-uniformities on the beam surface. After adequate, but minimal preparation, these gauges were also easily applied to steel reinforcement (See Fig. A24, Appendix A).

Description: Wire resistance strain gauge with polyester backing; wire leads.
Manufacturer: Tokyo Sokki Kenkyujo Co., Ltd.
Distributor: (USA sales) Texas Measurements, Inc. 409-764-0442
Model: PL-60-11
Active Length: 60-mm
Resistance: 120-ohms
Gauge factor: 2.1

Displacement Transducers

Beam deflections were monitored at three points on the beam tension face (bottom). Linear Variable Differential Transformers (LVDT) powered with Direct-Current (DCDT) were chosen for ease of use and calibration. A style of DCDT with a loose core rather than a spring-loaded core was used to minimize the possibility of damage at beam failure.

Description: LVDT, DC powered (DCDT)
Manufacturer: Solartron Metrology
US offices: Buffalo NY 716-634-4452
Models used:

- DFG5 (serial no. 121316, nominal range +/- 5-mm)

Note: Type DFG5 = DCDT1
- DFG15 (serial no. 69784, nominal range +/- 15-mm)
  
  Note: Type DFG15 = DCDT2

- DFG15 (serial no. 72870, nominal range +/- 15-mm)

  Note: Type DFG15 = DCDT3

**Mechanical Dial Indicator**

Midspan deflections were measured using a dial indicator for all tests to confirm the results from DCDT measurements. Only one dial gauge was used, located at midspan (equivalent to DCDT2).

  **Description:** Mechanical dial indicator

  **Range:** 0-1 inch (small divisions 0.001 inch)

  **Manufacturer:** Varies

**Loading Machine**

A 600-kip, hydraulic Baldwin Test Machine with a load-indicating dial equipped with a peak-indicating needle was used. In addition to the indicating needle, an electronic pressure sensor with signal conditioning provided a load signal. This load signal was monitored during all tests. The sensor was calibrated using the load dial as a standard. The load dial had been calibrated and certified within a year of all testing.

**Fiber Optic Strain Sensing System**

**Strain Gauges**

The fiber optic strain sensors were based on Bragg gratings. Nominal gauge lengths were 700mm and 1000mm.

**Spectrum Analyzer**

Ando Corporation AQ-6330 Optical Spectrum Analyzer. The window was set for 220 data points per nanometer (nm) so the smallest resolvable wavelength difference was approximately 5 picometers (pm). At a wavelength of 1300nm, 1pm was equivalent to 1 microstrain. The long-term resolution based on the manufacturer’s specifications was +/-70 pm. Short term (within an hour) resolution was approximately +/-15 pm. The light sources used in conjunction with the spectrum analyzer were Optiphase, Inc. Broad Band Optical Source BBS-13-0150.
Demodulator
Blue Road Research Model BRR-3SA with a sensitivity of +/-150 microstrain over a dynamic range of +/-8000 microstrain. This demodulator had a built-in light source.

Data Acquisition
National Instruments, DAQCard-AI-16XE-50 with 16 analog inputs.

Pulse Velocity Tester
Manufacturer: CNS Farnell
Model: PUNDIT 6

Signal Conditioning and Data Acquisition Systems

Control Beam Test
Signal conditioning and data acquisition were achieved using a single hardware system manufactured by ADAC Corporation.

Signal Conditioning and Analog to Digital (A/D) Conversion
Description: Strain gauge bridge completion and preamplifiers were contained in a module with terminals for strain inputs, as well as for high-level signals. This module was connected via a 6-foot ribbon cable to a 12-bit A/D converter board, which resided in the Data Acquisition PC (personal computer).

Manufacturer: ADAC Corporation, Woburn, MA 01801, (781) 935-3200
Model No.: 4012BGEX (strain gauge amp and bridge completion); TB5525 & 5302EN (terminal board and enclosure) 5525MF (A/D board)

Personal Computer
486DX/66 IBM-compatible running Windows 3.11

Data Acquisition Software
LabTech Notebook for Windows, version 9.0 was used for data collection. This allowed real-time monitoring of signals and produced an ASCII record file in tabular form.
Flexure-Only Beam Test

Strain data from the Control Beam test was noisy. In addition, the ADAC strain measuring system was found to have insufficient strain zeroing capabilities. For these reasons, a more sophisticated strain measuring system by Hewlett Packard was obtained. This system was based on the HP 3852A scanning voltmeter, and HP Vee 5.0 software. The HP system had much wider zeroing capability, higher rejection of power-line-frequency noise and better strain resolution. For the Flexure-Only Beam test, the HP system was used to gather strains only; an entirely separate PC data system was used concurrently to log displacement and load data. A marker signal was applied, common to both systems, for synchronizing the systems. In addition, PC clocks were closely synchronized before testing.

A. Strain Monitoring

**Signal Conditioning and A/D Conversion**

*Description:* 5-½-digit integrating voltmeter with terminal board, bridge completion and amplifiers for strain gauges.

*Manufacturer:* Hewlett-Packard Corp., Palo Alto CA 94303, (800) 452-4844

*Model:* 3852A mainframe with 44701A integrating voltmeter, 44705A relay multiplexer, and 44717A relay multiplexer for 120-ohm strain gauges.

**Personal Computer**

486DX/66 IBM-compatible; Windows 95

**Data Acquisition Software**

HP Vee for Win 95, version 5.0

B. Load and DCDT monitoring

**Signal Conditioning and A/D Conversion**

*Description:* PC data acquisition system with on-board signal conditioning and 14-bit A/D conversion.

*Manufacturer:* Validyne Engineering, Northridge, CA (800) 423-5851

*Model:* UPC-607

**Personal Computer**

486DX/66 IBM-compatible
Data Collection Software

Validyne “EasySense” for DOS

Shear-Only Beam Test

The HP 3852A/HP Vee system was further developed so load and displacements could be monitored along with all strains. A single PC and data system was used. All data were written to a single data file. Four to six strains of interest were plotted in real time for monitoring during the test.

Shear & Flexure Beam Test

This beam used the same system as the Shear-Only Beam test.
APPENDIX E: DESIGN CALCULATIONS FOR FRP RETROFITTED REINFORCED CONCRETE MEMBERS
APPENDIX E: DESIGN CALCULATIONS FOR FRP RETROFITTED REINFORCED CONCRETE MEMBERS

The method used to design the FRP strengthening scheme for HCB is presented here as originally proposed (Kachlakiev, 1998). The calculations are based on the actual materials used in the construction of the bridge and the experimental beams.

**OSU Design Method**

As suggested in Chapter 2, a design process for shear and flexure was used to predict the strength of the experimental beams, hereafter referred to as the OSU Method. The following assumptions are necessary for this design process to be valid.

**Introduction and design philosophy**

The adopted design philosophy is outlined below. FRP composite materials are considered brittle, because they exhibit linear stress-strain diagrams to failure. It should be noted that concrete is also considered a brittle material and yet, reinforced concrete flexural members exhibit ductile behavior at failure. In regular reinforced concrete design, this ductile behavior is achieved through limiting the amount of steel reinforcement in the balanced area, and assuring that steel yields prior to concrete crushing. Thus, the yielding of steel reinforcement converts the behavior of an otherwise brittle system to a ductile system. There is no reason to expect that the addition of another brittle material (FRP) to the already ductile system will result in a brittle failure, as long as the total area of the reinforcement is restricted to 75% or less of the balanced section.

It is known that the load-deflection curve of an under-reinforced beam consist of two regions, i.e., linear-elastic prior to steel yielding followed by ideally-plastic response afterward. Theoretically, this behavior is similar to the behavior exhibited by a FRP strengthened beam. With increasing cross sectional area of FRP reinforcement, the nearly horizontal ideally-plastic portion of the curve increases in slope, eventually becoming identical to that of the linear-elastic portion. At this point, the ductile behavior of the system (concrete-steel-FRP) changes to brittle behavior. When areas of FRP and steel reinforcement are in the prescribed limits, a significant change in the behavior of the system occurs at yielding of steel. Under very little increase of load, the deflections become excessive, thus allowing for redistribution of moments to areas of redundancy, and warning of impending failure.

The parameters that affect the strengthening design of reinforced concrete beams include the following factors: a) the effects of initial strain; b) FRP/steel reinforcement ratios; c) material properties of concrete, steel reinforcement and FRP composites; d) stress of the steel reinforcement at working loads; e) deflections under working loads, f) failure mechanisms, and g) behavior of the strengthened beam under service loads. The restrictions include considering flexural behavior only (shear is not considered) and assuming a pre-cracked concrete section.
**General Assumptions**

1. Classical beam theory assumptions apply.
2. Elastic and homogeneous concrete material.
3. Elastic-perfectly plastic steel reinforcement.
4. Linear elastic behavior of FRP materials up to failure.
5. Strength provided by components is the summation of parts (e.g. total shear strength is the strength of the concrete plus steel plus FRP).

**Assumptions Specific to Flexure**

F1. Plane-cross sections remain plane during bending.
F2. Five failure modes are possible.
F3. Shear strength is in excess of flexural strength.

**Assumptions Specific to Shear**

S1. Limiting FRP-concrete bond stress is 200 psi.
S2. Limiting FRP-concrete interfacial strain is 0.004.
Flexural Design Input Requirements

The following two tables give the needed information to begin the strengthening design.

Table E-1: FRP strengthening input requirements for the actual bridge and experimental beams

<table>
<thead>
<tr>
<th>Input Variable</th>
<th>US Standard Value</th>
<th>Metric Value</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Horsetail</td>
<td>Exper.</td>
</tr>
<tr>
<td>Existing Concrete Section</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Area of tensile steel</td>
<td>$A_s = n^2$</td>
<td>5.00 in$^2$</td>
</tr>
<tr>
<td>Area of compressive steel</td>
<td>$A_s' = n^2$</td>
<td>0.00 in$^2$</td>
</tr>
<tr>
<td>Depth of tensile steel</td>
<td>$d = 28.00 \text{ in}$</td>
<td>27.75</td>
</tr>
<tr>
<td>Depth of compressive steel</td>
<td>$d' = 0.0$</td>
<td>0.0</td>
</tr>
<tr>
<td>Width of reinforced concrete section</td>
<td>$b = 12.0 \text{ in}$</td>
<td>12.0</td>
</tr>
<tr>
<td>Height of reinforced concrete section</td>
<td>$h = 30.00 \text{ in}$</td>
<td>30.25</td>
</tr>
<tr>
<td>Concrete clear cover</td>
<td>$c_c = 1.5 \text{ in}$</td>
<td>2.063</td>
</tr>
<tr>
<td>Concrete clear cover + diameter/2</td>
<td>$c_{cs} = 2.0 \text{ in}$</td>
<td>2.50</td>
</tr>
<tr>
<td>Beam Geometry</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Beam clear span length</td>
<td>$L_{cr} = 240 \text{ in}$</td>
<td>216 in</td>
</tr>
</tbody>
</table>

‡ Small steel reinforcing bars do exist above the elastic neutral axis, but were found to be near the neutral axis after cracking and near ultimate and are disregarded for design. If compressive reinforcement is available and placed to increase the resisting C-force, it should be included in design.

Table E-2: Design material properties for the actual bridge beams and experimental beams

<table>
<thead>
<tr>
<th>Input Variable</th>
<th>US Standard Value</th>
<th>Metric Value</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Horsetail</td>
<td>Exper.</td>
</tr>
<tr>
<td>Concrete</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Compressive strength</td>
<td>$f_{c'} = 2500 \text{ psi}$</td>
<td>3000</td>
</tr>
<tr>
<td>Elastic modulus</td>
<td>$E_c = 2850 \text{ ksi}$</td>
<td>3120</td>
</tr>
<tr>
<td>Ultimate strain (crushing)</td>
<td>$\epsilon_{cu} = 0.003$</td>
<td>$0.003$</td>
</tr>
<tr>
<td>Steel Reinforcing</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Yield strength</td>
<td>$f_y = 33 \text{ ksi}$</td>
<td>60</td>
</tr>
<tr>
<td>Elastic Modulus</td>
<td>$E_s = 29,000 \text{ ksi}$</td>
<td>29,000</td>
</tr>
<tr>
<td>Strain at yield</td>
<td>$\epsilon_y = f_y/E_s = 0.0011$</td>
<td>$0.0021$</td>
</tr>
<tr>
<td>CFRP Reinforcement</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Reinforcement type</td>
<td>CARBON FABRIC (epoxy saturated, composite properties)</td>
<td></td>
</tr>
<tr>
<td>Tensile strength</td>
<td>$f_{tu} = 110 \text{ ksi}$</td>
<td>758 MPa</td>
</tr>
<tr>
<td>Elastic modulus</td>
<td>$E_f = 9,000 \text{ ksi}$</td>
<td>62.0 GPa</td>
</tr>
<tr>
<td>Ultimate strain</td>
<td>$\epsilon_{tu} = 0.0122$</td>
<td>0.0122</td>
</tr>
<tr>
<td>FRP thickness per ply</td>
<td>$t_f = 0.041 \text{ in}$</td>
<td>1.04 mm</td>
</tr>
<tr>
<td>GFRP Reinforcement</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Reinforcement type</td>
<td>GLASS FABRIC (epoxy saturated, composite properties)</td>
<td></td>
</tr>
<tr>
<td>Tensile strength</td>
<td>$f_{tu} = 60 \text{ ksi}$</td>
<td>414 MPa</td>
</tr>
<tr>
<td>Elastic modulus</td>
<td>$E_f = 3,000 \text{ ksi}$</td>
<td>20.7 GPa</td>
</tr>
<tr>
<td>Ultimate strain</td>
<td>$\epsilon_{tu} = 0.02$</td>
<td>0.02</td>
</tr>
<tr>
<td>FRP thickness per ply</td>
<td>$t_f = 0.051 \text{ in}$</td>
<td>1.30 mm</td>
</tr>
</tbody>
</table>

† Design properties based on manufacturer literature. The same material was used for the experimental beams as was used to retrofit the actual bridge.

‡ $E_c = 57000 \left( f_{c'} \right)^{0.5}$
Loads and Existing Section Capacity

The information provided in this section is arranged to accommodate the load rating procedures as prescribed by ODOT and AASHTO (1989, 1994). In general, for flexural strengthening, a pre-retrofit moment capacity and required moment capacity will need to be provided. Accepted reinforced concrete theory should be used to calculate the existing section capacity.

<table>
<thead>
<tr>
<th>Input Variable</th>
<th>US Standard Value</th>
<th>Metric Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Live load moment</td>
<td>$M_{LL} =$ 241 ft-kip</td>
<td>326 kN-m</td>
</tr>
<tr>
<td>Dead load moment</td>
<td>$M_{DL} =$ 107 ft-kip</td>
<td>145 kN-m</td>
</tr>
<tr>
<td>Total unfactored moment</td>
<td>$M_{WL} =$ 355 ft-kip</td>
<td>481 kN-m</td>
</tr>
<tr>
<td>Existing section capacity</td>
<td>$M_{n,exist} =$ 341 ft-kip</td>
<td>462 kN-m</td>
</tr>
</tbody>
</table>

† Not including impact factor.

Rating Factor

Conventional load rating requires the calculation of a rating factor. If the rating factor is below 1.0, the structural member is considered inadequate for the required load and accepted factors. Load rating of the HCB is presented in Appendix B. It was determined for flexure to be RF = 0.50, for HCB beams.

Design Procedure and Assumptions

Comments for the following calculations will be provided to clarify the procedure. A systematic process is given resulting in recommended FRP strengthening scheme.

Moment and Curvature at Steel Yield

Calculate the moment and curvature at yield for the unstrengthened section.

$$\rho_s = \frac{A_s}{bd} = \frac{(5.00 \text{ in}^2)/(12 \text{ in})(28 \text{ in})}{0.0149} = 0.0149 \quad \text{[E-1]}$$

$$n_s = \frac{E_s}{E_c} = \frac{(29,000 \text{ ksi})/(2850 \text{ ksi})}{10.18} \quad \text{[E-2]}$$

$$k = \left(\rho_s n_s\right)^2 + 2\left(\rho_s n_s\right) = 0.419 \quad \text{[E-3]}$$

$$c_y = (k)(d) = (0.419)(28.0 \text{ in}) = 11.74 \text{ in} \quad \text{[E-4]}$$

These commonly used equations assume elastic material behavior (particularly for the concrete). The equations are only valid for a singly reinforced concrete beam (i.e. neglecting the presence of any compression steel).

Concrete strain $\varepsilon_{cy}$ at steel yield:
\[ \varepsilon_{cy} = \left( \frac{c_y}{d-c_y} \right) \left( \frac{f_y}{E_s} \right) = \left( \frac{11.74 \text{in}}{28.00 \text{in} - 11.74 \text{in}} \right) \left( \frac{33 \text{ksi}}{29,000 \text{ksi}} \right) \]  
\[ \varepsilon_{cy} = 0.00082 \text{ in/in} \]

Note that \( \varepsilon_{cy} \) is limited to \( \frac{1}{2} \varepsilon_{cu} = 0.003/2 = 0.0015 \), in order to preserve the validity of the linear approximation.

Moment \( M_y \) at yielding of the steel reinforcement:

\[ M_y = (A_s f_y)(d-c_y/3) \]  
\[ M_y = (5.00 \text{in}^2 \times 33 \text{ksi})(28.00\text{in}-11.74\text{in}/3)/12 \]
\[ M_y = 331 \text{ft-kip} \]

This equation is only valid for a singly reinforced section. The corresponding curvature at yielding of the steel reinforcement is calculated by

\[ \phi_y = \varepsilon_{cy}/c_y = (0.00082)/(11.74\text{in}) = 6.98 \times 10^{-5} \text{in}^{-1} \]  
\[ \phi_y \]

**Strain in the Beam at the Level of FRP Laminate**

Assume that the dead load plus an additional 10% of the live load will be acting on the beam at the time of retrofit. Calculate the applied moment

\[ M_{retrofit} = 0.1 \times (248\text{ft-kip}) + (107\text{ft-kip}) = 132 \text{ ft-kip} \]  
\[ \phi_{retrofit} = (M_{ret}/M_y)(\phi_y) = (132/331)(6.98 \times 10^{-5} \text{in}^{-1}) \]
\[ \phi_{retrofit} = 2.79 \times 10^{-5} \text{in}^{-1} \]

Assuming a linear slope of the moment curvature diagram.

Assume that the depth of the neutral axis equals that at yield when the beam is retrofit. This allows calculation of strain in the beam at the level of FRP laminate at the time of retrofit.

\[ \varepsilon_{b,retrofit} = (h-c_y)\phi_{retrofit} \]
\[ \varepsilon_{b,retrofit} = (30.0\text{in}-11.74-\text{in}) \times 2.79 \times 10^{-5} \text{in}^{-1} = 0.00051 \text{in/in} \]

**Area of the FRP Required to Resist the Ultimate Projected Moment**

This calculation relies on the load rating procedure to find the required capacity.
The rating factor for flexure was determined to be 0.5 for the existing HCB beams. Naturally, the FRP strengthened section should have a resistance factor of at least 1.0. The required moment capacity is then back calculated.

\[
M_{\text{required}} = \frac{(RF = 1.0) * (1.033) * (1 + 0.10) * 1.3 * 241 + 1.2 * 107}{0.9}
\]

\[
M_{\text{required}} = 538 \text{ ft-kip}
\]

The required resistance provided by the FRP is the current moment shortfall.

\[
M_{\text{required}}^{\text{FRP}} = M_{\text{required}} - M_{\text{existing}} = 538 - 341 = 197 \text{ ft-kip}
\]

**Determination of the Failure Mode**

To estimate the failure mode begin by calculating the depth of the neutral axis at the balanced condition.

\[
c_{\text{bal}} = \frac{h \cdot \varepsilon_{cu}}{(\varepsilon_{cu} + \varepsilon_{f} + \varepsilon_{b, \text{retrofit}})}
\]

\[
c_{\text{bal}} = \frac{30.0 \text{ in} * 0.003}{(0.003 + 0.0122 + 0.00051)} = 5.72 \text{ in}
\]

The use of the total section depth “h” as opposed to adding the distance to the centroid of the FRP composite is likely appropriate, since surface preparation will likely remove some material and the number of layers is yet unknown. Now, calculate the maximum area of tensile steel reinforcement to allow FRP rupture prior to concrete crushing.

\[
A_{s, \text{max}} = \left[\frac{0.85 \cdot f_{c} \cdot \beta \cdot c_{\text{bal}} \cdot b}{f_{y}}\right]
\]

\[
A_{s, \text{max}} \geq \left[\frac{0.85 \cdot 2500 \text{ psi} \cdot 0.85 \cdot 5.72 \text{ in} \cdot 12 \text{ in}}{33,000 \text{ psi}}\right] = 3.76 \text{ in}^{2}
\]

\[
A_{s, \text{max}} = 3.76 \text{ in}^{2} < A_{s, \text{provided}} = 5.00 \text{ in}^{2}
\]
This shows that crushing of the concrete will precluded rupture of the FRP laminate. Regardless of the selected FRP reinforcing, crushing of the concrete will control. This behavior will be common and depends largely on the geometry of the section.

**Required FRP Area**

For internal couple equilibrium, where $C = T$, 

$$A_s f_y + A_f [(h-c)/c] * (0.003) - \varepsilon_{b, \text{retrofit}}] * E_f = (0.85) * f_y * b * \beta * c \quad [E-16]$$

$$5 * 33,000 + A_f \{[(30-c)/c] * (0.003) - 0.00051\} * (9,000,000) = 0.85 * (2500) * 12 * (0.85) * c$$

$$A_f = (c^2 - 7.6125 * c) / (37.37 - 1.457 * c) \quad [E-17]$$

Equation for nominal moment capacity:

$$M_n = A_s * f_y * (d - \beta * c / 2) + A_f * f_y * (h - \beta * c / 2) \quad [E-18]$$

$$M_n = 5.0 * 33,000 * (28.0 - 0.85 * c / 2) +$$

$$+ A_f * [(30 - c) / c] * (0.003) - 0.00051] * (9,000) * (30.0 - 0.85 * c / 2) = 6,456,000 \text{ in - lb}$$

$$A_f = (26.1768 * c + c^2) / (346.509 - 18.4227 * c + 0.1914 * c^2) \quad [E-19]$$

Solving equations [E-17] and [E-19] simultaneously eliminates the unknown area of FRP.

Determine the neutral axis by reduction of the above. This results in,

$$c = 12.7331 \text{ in}$$

The required area of FRP reinforcing is then back calculated (equation [E-17] or [E-19])

$$A_{f, \text{required}} = 3.4647 \text{ in}^2$$

The required width of FRP reinforcement is,
\[ b_{f_p} = \frac{A_{f,\text{required}}}{t_f} = \frac{3.4647 \text{ in}^2}{0.041 \text{ in}^2 / \text{ply} - \text{in}} = 84.5 \text{ in} \]  

E-20

Since most FRP materials are manufactured to specific widths and thickness, select the most appropriate configuration. In this case, 24 inch widths are available and providing 4 layers at 12 inches wide will be adequate. Thus,

\[ A_{f,\text{provided}} = \frac{t_f}{b_{f,\text{total}}} = 4 * 0.041 \text{ in}^2 / \text{ply} - \text{in} * 24 \text{ in} = 3.936 \text{ in}^2 \]  

E-21

Using the selected FRP area, determine the position of the neutral axis at ultimate load.

\[ A_s \cdot f_y + A_f \cdot ((h-c_{\text{ult}})/c_{\text{ult}}) \cdot (0.003) - \varepsilon_{b,\text{retrofit}} \cdot E_f = (0.85) \cdot f_c \cdot b \cdot \beta \cdot c_{\text{ult}} \]

\[ A_f = (c_{\text{ult}}^2 - 7.6125 \cdot c_{\text{ult}}) / (37.37 - 1.457 \cdot c_{\text{ult}}) \]

\[ c_{\text{ult}} = 13.10 \text{ in} \]

Note that, for this method, adding more area of FRP reinforcing will result in a predicted lowering of the neutral axis (c increases). Check the failure mode by,

\[ \varepsilon_{c,\text{ult}} = \left[ \frac{c_{\text{ult}}}{h - c_{\text{ult}}} \right] \cdot (\varepsilon_{f_u} + \varepsilon_{b,\text{ret}}) \]

\[ \varepsilon_{c,\text{ult}} = 0.00969 >> 0.003 \]

which implies that crushing of the concrete controls.

The moment capacity after strengthening with FRP, according to equation [E-18] is,

\[ M_n = 6,825,804 \text{ in-lb} = 569 \text{ ft-kip} \]

Following load rating requirements, the rating factor after strengthening can be calculated from [E-11]

\[ RF_{\text{retrofit}} = \left[ \frac{0.85 \cdot 569 \text{ ft - kip} - 1.2 \cdot 107 \text{ ft - kip}}{5.0 \cdot (1 + 0.10) \cdot 1.3 \cdot 45 \text{ ft - kip}} \right] = 1.11 > 1.0 \]

The retrofitted beam then satisfies strength requirements.
System Ductility Requirements

Curvature
The curvature at ultimate load is,

\[ \psi_{ult} = \frac{\varepsilon_c}{c} = \frac{0.003}{13.1 \text{ in}} = 2.29 \times 10^{-4} \text{in}^{-1} \quad \text{[E-23]} \]

Or, checking against the FRP strains,

\[ \psi_{ult} = \frac{\varepsilon_{fu} + \varepsilon_{b,ret}}{h - c} = \frac{(0.0122 + 0.00051)}{(30.0 \text{ in} - 13.10 \text{ in})} = 7.52 \times 10^{-4} \text{in}^{-1} \quad \text{[E-24]} \]

Since the curvature using the FRP strain is tighter than using concrete crushing, concrete crushing controls the failure. A comparison of the moment and curvature at yield to ultimate is necessary to evaluate the ductility of the system. Consider the ratio

\[ \frac{M_n}{M_y} = \frac{569}{331} = 1.72 \quad \text{[E-25]} \]

\[ \frac{\psi_{ult}}{\psi_y} = \frac{2.29}{0.698} = 3.28 \quad \text{[E-26]} \]

NOTE:

- IF \( \frac{M_n}{M_y} \geq 1.3 \), THEN \( \frac{\psi_{ult}}{\psi_y} \) must be greater than 2.5
- IF \( \frac{M_n}{M_y} < 1.3 \), THEN \( \frac{\psi_{ult}}{\psi_y} \) must be greater than 2.0

In this case, both curvature requirements are satisfied.

Ductility Indices

For conventional design requirements, the ductility index \( \mu \) shall be greater than 2.0. When moment redistribution is relied upon, \( \mu \) shall be greater than or equal to 4.0. If seismic resistance of the system is essential for the design, \( \mu \) shall be greater than or equal to 3.0. The ductility index is defined by

\[ \mu = \frac{1}{2} \left[ \frac{E_{\text{total}}}{E_{\text{elastic}}} + 1 \right] = \left[ M_y \left( \frac{M_y \psi_{ult} + M_n \psi_{ult} - M_n \psi_y}{2M_n^2 \psi_y} \right) \right] + 0.5 \quad \text{[E-27]} \]

\[ \mu = 5.2 > 2.0 \]

Ductility for general design requirements are satisfied.

E-9
**Service Level Deflections**

Calculations of moment of inertia and hence deflections will be consistent with current methods. That is, the FRP will be considered in the same manner as steel, transforming the respective area using a modular ratio. These calculations must be performed, but are omitted here, since they do not provide new insight into FRP strengthening.

**Stresses and Strains Developed Under Working Loads**

It is necessary to check service level stresses and compare against allowable values. These calculations are only necessary if the working load moment is greater than 80% of the yield moment. From before,

\[
M_{WL} = 355 \text{ k-ft} > 0.8 \times M_y = 265 \text{ ft-kip} \quad \text{[E-28]}
\]

The following conditions must be satisfied:

- **Tensile Steel Reinforcing** – \( \varepsilon_s \leq 0.80 \varepsilon_y \)
- **Concrete in Compression** -- \( \sigma_c \leq 0.45 \sigma_c' \)
- **FRP composite** – \( \varepsilon_f \leq 0.30 \varepsilon_{fu} \)

If these limits are not satisfied then serviceability will govern the design and the previously calculated reinforcing will need to be changed.

**Elastic Stresses and Strains**

For these calculations, assume that the provided FRP area will be used in design (\( A_f = 3.936 \text{ in}^2 \)). Internal equilibrium is described by

\[
C = T \quad \text{[E-29]}
\]

Or,

\[
0.5c'b\varepsilon_c'E_{c,eff} = A_s\varepsilon_s + A_f\varepsilon_f \quad \text{[E-30]}
\]

The strain in the FRP is geometrically related by,

\[
\varepsilon_f = \left( \frac{h-c}{d-c} \right) \varepsilon_s - \varepsilon_{f,retrofit} \quad \text{[E-31]}
\]

In addition, the concrete strain is related by,

\[
\varepsilon_c = \left( \frac{c}{d-c} \right) \varepsilon_s \quad \text{[E-32]}
\]
Since linear elastic behavior of the concrete is desired, the limiting strain of 0.002 in the concrete will be required. The effective elastic modulus is then,

\[ E_{\text{eff}} = \frac{f'_c}{0.002} = \frac{2500 \text{ psi}}{0.002} = 1.25 \times 10^6 \text{ psi} \]  

\[ [E-33] \]

The strain in the steel as related to the depth to the neutral axis is,

\[ \epsilon_s = \frac{\epsilon_{b,\text{retrofit}}}{\left\{ \frac{(h - c^2 * b * E_{\text{eff}})}{(d - c)} + \frac{A_s * E_s}{A_f * E_f} \right\}} \]

\[ [E-34] \]

\[ \epsilon_s = \frac{0.00051}{\left\{ \frac{(30.0 - c^2 * 12.0 * 1.25 \times 10^6)}{(28 - c)} + \frac{5.0 * 29.0 \times 10^6}{0.984 * 9.0 \times 10^6} \right\}} \]

When simplified, equation [E-34] will give a quadratic relationship in the neutral axis location \( c \). In order to develop another equation to solve simultaneously with [E-34], equate the applied moment to the resisting couple created by the tension reinforcement,

\[ M_{WL} = A_s * E_s * \epsilon_s (d - c / 3) + A_f * E_f * \epsilon_f (h - c / 3) \]

\[ [E-35] \]

Where

\[ \epsilon_f = \epsilon_s \left[ \frac{h - c}{d - c} \right] - \epsilon_f,\text{ret} \]

\[ [E-36] \]

Combining equations [E-34] and [E-35] a solution for \( c \) should be achievable.

---

END OF FLEXURAL DESIGN

Shear Design Process

Designing an FRP reinforced beam for shear is different than flexure in that the strains of the FRP will be limited. Ultimate capacity (failure) calculations are not appropriate in this case, since strain limits the effectiveness of the concrete-FRP bond. For this reason, experimental studies have suggested that the strain in the FRP jacket be limited to a value of 0.004. Shear design is outlined as follows for the HCB beams and summarized for the experimental beams. For shear design, the use of the subscript “\( j \)” will designate the various properties of the FRP jacket. This notation will be useful in separating flexure and shear variables.
Concrete Shear Capacity

Typical reinforced concrete design will be used to calculate the concrete contribution to shear capacity. The shear capacity calculation here include the \( d = 28'' \) assumption. The actual bridge has a changing depth due to the roadway crowning. The calculations here are for comparison to experimental. The simplified capacity,

\[
V_c = 2.0\sqrt{f_c} \cdot (b)(d)
\]

\[
V_c = 2.0\sqrt{2500}(12)(28.0) = 33,600 \text{ lb} = 33.6 \text{ kips}
\]

Steel Shear Capacity

In the case of the Horsetail Creek Bridge and the experimental beams, no stirrups were provided. Thus,

\[
V_s = \frac{A_y f_y d}{s} = 0
\]

Shear Deficiency

Assuming that the total capacity is the sum of the constituent capacities, the required resistance of the FRP shear jacket is,

\[
\phi V_j = V_{\text{demand}} - \phi[V_s + V_c]
\]

\[
\phi V_j = (89.2 \text{ kip} - 0.85(33.6 \text{ kip})) = 60.6 \text{ kip}
\]

The FRP jacket must then resist a total force of 71.3 kips if a \( \Phi \)-factor of 0.85 is used. This is the resistance to be provided by the FRP at a limited strain of 0.005.

Require FRP Jacket Thickness

\[
t_j \geq \frac{\phi V_j}{2\varepsilon_j E_j D(\cot \theta)}
\]

Where \( \theta \) is the angle between the shear crack and the principal direction of the FRP jacket fibers (assumed unidirectional). Using the known values,

\[
t_j \geq \frac{\phi V_j}{2\varepsilon_j E_j D(\cot 45^\circ)} \text{ for 45\(^\circ\) shear crack} = \frac{71.3 \text{ kips}}{2(0.005)(3000 \text{ ksi})(12'')(\cot 45^\circ)} = 0.198 \text{ in}
\]
Required Number of Layers

\[ \text{#layers} = \frac{t_j}{t_j/\text{layer}} = \frac{0.198}{0.051} = 3.88 \approx 4 \text{ layers} \] \hspace{1cm} [E-41]

Check Concrete Bond Stress

The bond stress is empirically limited to 200 psi. Thus,

\[ \sigma_b = \frac{E_j t_j \varepsilon_j}{l_d} = \frac{(3000\text{ksi})(4 * 0.051)(0.005)}{12 \text{ in}} = 200 \text{ psi} \] \hspace{1cm} [E-42]

For this case, the bond is at its limit for the 12 in development length. Since the composite had more than 12 inches to develop bond, this requirement is satisfied.

Clearly, this method of design is very conservative. The required limitations are still in debate amongst the various researchers in FRP strengthening. The suggested 0.004 strain and 200 psi are conservative and this project was not able to suggest different values.

END OF SHEAR DESIGN

References


## Table E-4: Appendix E notation

<table>
<thead>
<tr>
<th>Variable</th>
<th>Description</th>
<th>US Standard Units†</th>
<th>Metric Units†</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A_f$</td>
<td>Area of FRP</td>
<td>in$^2$</td>
<td>mm$^2$</td>
</tr>
<tr>
<td>$A_{f,\text{required}}$</td>
<td>Required area of FRP to resist the load demand</td>
<td>in$^2$</td>
<td>mm$^2$</td>
</tr>
<tr>
<td>$A_s$</td>
<td>Area of steel</td>
<td>in$^2$</td>
<td>mm$^2$</td>
</tr>
<tr>
<td>$A_{s,\text{max}}$</td>
<td>Maximum area of steel such that concrete compression controls</td>
<td>in$^2$</td>
<td>mm$^2$</td>
</tr>
<tr>
<td>$A_{s,\text{provided}}$</td>
<td>Provided area of steel reinforcing</td>
<td>in$^2$</td>
<td>mm$^2$</td>
</tr>
<tr>
<td>$A_s$</td>
<td>Area of compression steel reinforcing</td>
<td>in$^2$</td>
<td>mm$^2$</td>
</tr>
<tr>
<td>$b$</td>
<td>Compression block width</td>
<td>in</td>
<td>mm</td>
</tr>
<tr>
<td>$b_{\text{total}}$</td>
<td>Total width of FRP to be provided</td>
<td>in</td>
<td>mm</td>
</tr>
<tr>
<td>$b_{\text{frp}}$</td>
<td>Width of FRP reinforcement</td>
<td>in</td>
<td>mm</td>
</tr>
<tr>
<td>$c$</td>
<td>Depth to the neutral axis from the top compression fiber</td>
<td>in</td>
<td>mm</td>
</tr>
<tr>
<td>$c_{\text{balanced}}$</td>
<td>Depth to the neutral axis at a balanced failure condition</td>
<td>in</td>
<td>mm</td>
</tr>
<tr>
<td>$c_{\text{clear}}$</td>
<td>Clear cover from concrete surface to near edge of steel</td>
<td>in</td>
<td>mm</td>
</tr>
<tr>
<td>$c_s$</td>
<td>Distance from the beam bottom to centroid of the steel</td>
<td>in</td>
<td>mm</td>
</tr>
<tr>
<td>$c_{\text{ultimate}}$</td>
<td>Depth to the neutral axis from the beam top at ultimate load</td>
<td>in</td>
<td>mm</td>
</tr>
<tr>
<td>$c_y$</td>
<td>Depth to the neutral axis from the beam top at steel yielding</td>
<td>in</td>
<td>mm</td>
</tr>
<tr>
<td>$d$</td>
<td>Structural steel depth from the beam top to the reinforcement</td>
<td>in</td>
<td>mm</td>
</tr>
<tr>
<td>$d_{\text{e}}$</td>
<td>Structural depth to steel centroid</td>
<td>in</td>
<td>mm</td>
</tr>
<tr>
<td>$DF$</td>
<td>Distribution factor account for lane distribution of live load</td>
<td>~</td>
<td>~</td>
</tr>
<tr>
<td>$DL$</td>
<td>Total unfactored dead load</td>
<td>varies</td>
<td>varies</td>
</tr>
<tr>
<td>$E_c$</td>
<td>Modulus of elasticity of the concrete</td>
<td>psi</td>
<td>MPa</td>
</tr>
<tr>
<td>$E_{c,\text{effective}}$</td>
<td>Straight-line approximation of concrete elastic modulus</td>
<td>psi</td>
<td>MPa</td>
</tr>
<tr>
<td>$E_{\text{elastic}}$</td>
<td>Elastic modulus</td>
<td>psi</td>
<td>MPa</td>
</tr>
<tr>
<td>$E_f$</td>
<td>Elastic modulus of the FRP reinforcement</td>
<td>ksi</td>
<td>GPa</td>
</tr>
<tr>
<td>$E_s$</td>
<td>Elastic modulus of the steel reinforcement</td>
<td>ksi</td>
<td>GPa</td>
</tr>
<tr>
<td>$E_{\text{total}}$</td>
<td>Total elastic modulus of the beam</td>
<td>psi</td>
<td>MPa</td>
</tr>
<tr>
<td>$f_c$</td>
<td>28-day specified concrete strength</td>
<td>psi</td>
<td>MPa</td>
</tr>
<tr>
<td>$f_{\text{tensile}}$</td>
<td>Todeschini stress-strain parameter</td>
<td>psi</td>
<td>MPa</td>
</tr>
<tr>
<td>$f_{\text{ult}}$</td>
<td>Ultimate stress (strength) of the FRP reinforcement</td>
<td>ksi</td>
<td>GPa</td>
</tr>
<tr>
<td>$h$</td>
<td>Total overall depth of the concrete beam</td>
<td>in</td>
<td>mm</td>
</tr>
<tr>
<td>$I$</td>
<td>Impact factor applied to live loads as specified by AASHTO</td>
<td>~</td>
<td>~</td>
</tr>
<tr>
<td>$j$</td>
<td>Internal moment arm parameter</td>
<td>~</td>
<td>~</td>
</tr>
<tr>
<td>$k$</td>
<td>Internal moment arm parameter</td>
<td>~</td>
<td>~</td>
</tr>
<tr>
<td>$L_{\text{clt}}$</td>
<td>Beam span to center line of supports</td>
<td>ft</td>
<td>m</td>
</tr>
<tr>
<td>$l_d$</td>
<td>Bond development length</td>
<td>in</td>
<td>mm</td>
</tr>
<tr>
<td>$LL$</td>
<td>Total unfactored applied live load</td>
<td>varies</td>
<td>varies</td>
</tr>
<tr>
<td>$M_{\text{DL}}$</td>
<td>Moment caused by the total unfactored dead loads</td>
<td>ft-kip</td>
<td>kN-m</td>
</tr>
<tr>
<td>$M_{\text{FRP,\text{required}}}$</td>
<td>Require moment resistance to be provided by the FRP</td>
<td>ft-kip</td>
<td>kN-m</td>
</tr>
<tr>
<td>Variable</td>
<td>Description</td>
<td>US Standard Units†</td>
<td>Metric Units†</td>
</tr>
<tr>
<td>----------</td>
<td>-------------</td>
<td>--------------------</td>
<td>---------------</td>
</tr>
<tr>
<td>M_{LL}</td>
<td>Moment caused by the total unfactored live loads</td>
<td>ft-kip</td>
<td>kN-m</td>
</tr>
<tr>
<td>M_{existing}</td>
<td>Existing section moment capacity, prior to strengthening</td>
<td>ft-kip</td>
<td>kN-m</td>
</tr>
<tr>
<td>M_{required}</td>
<td>Required moment capacity to resist the applied loads</td>
<td>ft-kip</td>
<td>kN-m</td>
</tr>
<tr>
<td>M_{retrofit}</td>
<td>Total unfactored moment applied at the time of retrofit</td>
<td>ft-kip</td>
<td>kN-m</td>
</tr>
<tr>
<td>M_{WL}</td>
<td>Working load moment (services level loads)</td>
<td>ft-kip</td>
<td>kN-m</td>
</tr>
<tr>
<td>M_y</td>
<td>Moment at which the primary tension steel reinforcing yields</td>
<td>ft-kip</td>
<td>kN-m</td>
</tr>
<tr>
<td>n_s</td>
<td>Ratio of steel elastic modulus over concrete elastic modulus</td>
<td>~</td>
<td>~</td>
</tr>
<tr>
<td>RF</td>
<td>Load rating factor as defined by AASHTO</td>
<td>~</td>
<td>~</td>
</tr>
<tr>
<td>RF_{existing}</td>
<td>Existing section rating factor</td>
<td>~</td>
<td>~</td>
</tr>
<tr>
<td>RF_{retrofit}</td>
<td>Rating factor after retrofit of the beam</td>
<td>~</td>
<td>~</td>
</tr>
<tr>
<td>R_n</td>
<td>Nominal resistance (any mode)</td>
<td>varies</td>
<td>varies</td>
</tr>
<tr>
<td>t_f</td>
<td>Thickness of the FRP reinforcement per layer</td>
<td>in</td>
<td>mm</td>
</tr>
<tr>
<td>t_j</td>
<td>Thickness of the FRP jacket for shear strengthening</td>
<td>in</td>
<td>mm</td>
</tr>
<tr>
<td>V_c</td>
<td>Shear strength of concrete section alone</td>
<td>kips</td>
<td>kN</td>
</tr>
<tr>
<td>V_{demand}</td>
<td>Factored applied loads shear force demand on the sections</td>
<td>kips</td>
<td>kN</td>
</tr>
<tr>
<td>β</td>
<td>Equivalent stress block parameter for location of C-force</td>
<td>~</td>
<td>~</td>
</tr>
<tr>
<td>ε_0</td>
<td>Todeschini stress-strain parameter</td>
<td>~</td>
<td>~</td>
</tr>
<tr>
<td>ε_{b,retrofit}</td>
<td>Strain at the time of retrofit at the level of FRP to be added</td>
<td>~</td>
<td>~</td>
</tr>
<tr>
<td>ε_c</td>
<td>Strain in the concrete top compression fiber</td>
<td>~</td>
<td>~</td>
</tr>
<tr>
<td>ε_{c,ultimate}</td>
<td>Strain in the concrete top compression fiber at ultimate load</td>
<td>~</td>
<td>~</td>
</tr>
<tr>
<td>ε_{cu}</td>
<td>Ultimate (crushing strain of the concrete), typically 0.003</td>
<td>~</td>
<td>~</td>
</tr>
<tr>
<td>ε_{cy}</td>
<td>Strain in the concrete top compression fiber at steel yielding</td>
<td>~</td>
<td>~</td>
</tr>
<tr>
<td>ε_{fu}</td>
<td>Ultimate (rupture) strain of the FRP reinforcement</td>
<td>~</td>
<td>~</td>
</tr>
<tr>
<td>ε_s</td>
<td>Strain in the steel reinforcing</td>
<td>~</td>
<td>~</td>
</tr>
<tr>
<td>ε_y</td>
<td>Yield strain of the steel reinforcement</td>
<td>~</td>
<td>~</td>
</tr>
<tr>
<td>φ</td>
<td>General strength reduction factor</td>
<td>~</td>
<td>~</td>
</tr>
<tr>
<td>γ_{DL}</td>
<td>Dead load factor</td>
<td>~</td>
<td>~</td>
</tr>
<tr>
<td>γ_{LL}</td>
<td>Live load factor</td>
<td>~</td>
<td>~</td>
</tr>
<tr>
<td>μ</td>
<td>Ductility index</td>
<td>~</td>
<td>~</td>
</tr>
<tr>
<td>ρ_s</td>
<td>Tension steel reinforcement ratio, typically ( \frac{A_s}{bd} )</td>
<td>~</td>
<td>~</td>
</tr>
<tr>
<td>σ_b</td>
<td>Bond stress limitation at FRP-concrete interface</td>
<td>psi</td>
<td>MPa</td>
</tr>
<tr>
<td>σ_c</td>
<td>Stress in the concrete</td>
<td>psi</td>
<td>MPa</td>
</tr>
<tr>
<td>ψ_{retrofit}</td>
<td>Curvature at the time of retrofit</td>
<td>in^{-1}</td>
<td>mm^{-1}</td>
</tr>
<tr>
<td>ψ_{ult}</td>
<td>Curvature at ultimate load</td>
<td>in^{-1}</td>
<td>mm^{-1}</td>
</tr>
<tr>
<td>ψ_{y}</td>
<td>Curvature at yielding of the primary steel tension reinforcement</td>
<td>in^{-1}</td>
<td>mm^{-1}</td>
</tr>
</tbody>
</table>

† Typical units given. The use of "~" implies variable has no units.