RETROFIT OF THE LOUISA-FORT GAY BRIDGE USING CFRP LAMINATES
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Retrofit of the Louisa-Fort Gay Bridge using CFRP Laminates

by

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in cooperation with

Transportation Cabinet
Commonwealth of Kentucky

and

Federal Highway Administration
U.S. Department of Transportation

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### Abstract

This report details the processes pertaining to the repair and strengthening of the Louisa-Fort Gay Bridge, Lawrence County, KY, using advanced composite materials. Site inspections revealed flexural cracks in the reinforced concrete girders of the continuous bridge structure. To determine the cause of these cracks, an evaluation was initiated based on vehicle classification and truck weight data. The results confirmed that certain reinforced concrete girders were stressed beyond the limits allowed by the AASHTO Code. A retrofit scheme using carbon fiber reinforced polymer (CFRP) composite was devised and the amount of CFRP laminates needed for flexural strengthening was determined from moment-curvature analyses. Retrofitting work began in September of 2003, and the project was completed in October of 2003. Crack gauges were installed at the affected areas to monitor the effectiveness of the retrofit. The bridge was inspected on a number of occasions and, as of September 28 2006, no movement in the crack gauges has been observed.
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EXECUTIVE SUMMARY

Routine inspection of the Louisa-Fort Gay Bridge revealed that flexural cracks have formed in the continuous reinforced concrete girders in the positive bending regions. In order to characterize the existing service load conditions of the bridge, vehicle classification and truck weight data were collected and analyzed. It was concluded that the trucks that were concurrently operating and traversing the bridge may have been carrying loads exceeding the legal weight limit. Further evaluation confirmed that most of the reinforced concrete girders were stressed beyond the allowable service limits specified by the American Association of State and Highway Transportation Official (AASHTO) Specification. In order to accommodate excessive service loads, as well as to extend the service life of the bridge, recommendations were to use carbon fiber reinforced polymer (CFRP) laminates to retrofit the reinforced concrete girders. A detailed moment-curvature analysis was carried out to establish the appropriate amount of laminates required for flexural strengthening. The retrofit was implemented in September of 2003 and the project was completed in October of 2003. Crack gauges were installed at critical crack locations to monitor the effect of the retrofit. As of September 28 2006, no crack movement has been observed.
ACKNOWLEDGMENTS

Financial support for this project was provided by the Federal Highway Administration and the Kentucky Transportation Cabinet under the Innovative Bridge Research and Construction Program – IBRC (Grant FRT118). The authors would like to thank Mr. David Steele with the Kentucky Transportation Cabinet for his guidance throughout this project. The authors also wish to express their gratitude to Mr. Scott Pabian and Mr. Abheetha Peiris for their contribution in completing this project.
1. INTRODUCTION

1.1 The Louisa-Fort Gay Bridge

The Louisa-Fort Gay Bridge is located in the small mining community of Lawrence County, Kentucky. The bridge was constructed in 1979. The bridge is a 12-span continuous bridge structure consists of composite concrete deck-steel girder spans and reinforced concrete middle spans that comprise an intersection for traffic coming from three different directions. A schematic plan view of the reinforced concrete spans (Spans 4-5-6-7) of the bridge is shown in Fig. 1.1.

![Schematic Plan View of Louisa-Fort Gay Bridge](image)

Fig. 1.1 – The continuous reinforced concrete middle spans of the Louisa-Fort Gay Bridge.

1.2 Cracks in Reinforced Concrete Girders

A routine inspection revealed cracks in the reinforced concrete girders in Spans 4, 6 and 7. These cracks were visible at ground elevation. Fig. 1.2 shows the view of the concrete spans and the estimated height of the superstructure from the ground elevation. Fig. 1.3 shows the cracks that had developed at the bottom web of a typical reinforced concrete girder of the bridge. These cracks formed primarily due to high flexural stresses that developed in the positive bending regions of the continuous structure.
Fig. 1.2 – The continuous reinforced concrete middle spans of the Louis-Fort Gay Bridge.

(a) Spans 5, 6 and 7.

(b) Spans 4 and 5

* approximate clear height of piers in feet

Fig. 1.2 – The continuous reinforced concrete middle spans of the Louis-Fort Gay Bridge.
Fig. 1.3 – Flexural cracks observed in Spans 4, 6 and 7 (see Fig. 1.2).
1.3 Objective

In this report, the strengthening and repair of the Louisa-Fort Gay Bridge is detailed. The objective of this project is to demonstrate that the use of high-strength carbon fiber reinforced polymer (CFRP) composite laminates can be an efficient and effective way of retrofitting the bridge elements by improving their respective strength and stiffness. It is also expected that such repair will extend the service life of the bridge.

1.4 Tasks

In order to achieve the objective of this study, the following tasks were carried out:

Task 1 Detailed Evaluation: A detailed evaluation was performed to determine a satisfactory repair using CFRP laminates. The appropriate amount of the CFRP laminates needed to strengthen the existing girders was quantified using the moment-curvature analysis. Details of the analysis are included in this report.

Task 2 Construction: The Construction phase of the project is presented herein. The repair, which involved crack repair, surface preparation, and CFRP laminate application, began in September 2003, and was completed in October 2003.

Task 3 Post-repair Monitoring: Crack gauges were installed at critical crack locations to monitor the effect of the retrofit. Inspections were scheduled on a regular basis and continued for a period of 3 years after the repair.
2. DETAILED EVALUATIONS

2.1 Truck Types

Vehicle classification and truck weight data were collected for the evaluation process. Fig. 2.1 shows the schematics of various truck types that were traversing the bridge. Fig. 2.1.c shows the legal truck weight of a typical Type 9 truck. Fig. 2.4.d shows the estimated weight that a Type 9 truck was actually carrying based on weigh-in-motion (WIM).

(a) AASHTO Truck HS20-44
[Total weight = 72 k]

(b) Type 6 Coal Truck
[Total weight = 70 k]

(c) Type 9 Coal Truck (Legal)
[Total weight = 126 k]

(d) Type 9 Coal Truck (WIM)
[Total weight = 225 k]

Fig. 2.1 – Truck type and loading that cross the Louisa-Fort Gay Bridge.
2.2 Finite Element Analysis

To investigate the effects of various truck types shown in previous section, analytical investigation was carried out. The process first involved the creation of a finite element model of reinforced concrete middle spans of the bridge. In this study, the reinforced concrete middle spans were constructed using SAP2000 (Wilson 2000). For live load analysis, one single truck, without any additional traffic or lane loads, was considered in the live load analysis.

The moments generated from the loading, particularly ones in the positive bending regions, were distributed to the respective girders based on the live load distribution factors computed for the parallel girders. Live load distribution factor computations are presented in Appendix A. The bending moments both at service and at ultimate conditions of respective girders were generated and tabulated in Appendix B.

2.3 Moment-Curvature Analysis

The flexural capacity of the reinforced concrete girders was determined from a moment-curvature analysis procedure. The moment-curvature ($M_n$-$\phi$) relation captures the basic load-deformation characteristics of a given girder under bending action. Different stress or load levels (i.e. allowable service stresses as defined in AASHTO or other code provisions) were identified once the moment-curvature characteristics of the reinforced concrete members were generated.

Typically, the moment-curvature relation of a reinforced concrete girder section can be derived based on stress equilibrium, strain compatibility, and material constitutive laws. The following assumptions were used in the formulation of the moment-curvature analysis:

- Strain distribution is linear throughout the beam section;
- Perfect bond exists between the concrete and any reinforcement (steel and externally bonded CFRP laminates);
- The tensile strength of concrete is ignored;
• Failure of the beam occurs when either the compressive strain in the concrete reaches 0.003 or the strain in the outermost layer of tension steel reaches its ultimate (e.g., $\varepsilon_u = 0.10$, typical for mild steel). For reinforced concrete girder strengthened with CFRP laminates, there is a possibility of tensile rupture ($\varepsilon_{ft} = \varepsilon_{fut} = 0.0168$, refer to Fig. C.1) of the laminates prior to concrete reaching 0.003. It should be noted that since $\varepsilon_{fut}$ is much smaller than $\varepsilon_u$, the rupture of the CFRP laminates would occur prior to the outermost tension steel reaching its ultimate. In this case, once the failure of the strengthened beam is initiated it would, in general, revert back to the behavior of un-strengthened beam (refer to Appendix B).

The schematic in Fig. 2.2 shows how the bending strength ($M_n$) and curvature ($\phi$) are computed for a typical beam section using an assumed linear strain distribution:

![Moment-curvature computation of typical beam section.](image)

Based on equilibrium conditions, strain compatibility, and material constitutive laws, the full range of moment-curvature ($M_n-\phi$) responses can be generated for a beam section (either the original or the strengthened section). In general, the process can be summarized as follows:

1. Select a small value for the concrete strain, $\varepsilon_c$, at the outmost concrete fiber in compression.
2. Assume the location of the neutral axis.
3. From linear strain distribution, determine the strains in all reinforcement (e.g. steel and FRP).
(4) Using concrete and reinforcement stress-strain relations, determine the stresses, (and consequently) the forces, of the concrete and reinforcement.

(5) Compute the resultant axial force of the section. Iterate steps 2 to 5 until the resultant axial force converges to zero (equilibrium of forces).

(6) Compute the moment or bending resistance \( M_n \) of the section and the corresponding curvature \( \phi \).

(7) Repeat steps 1 through 6 until \( \varepsilon_c \) reaches its pre-determined ultimate strain \( \varepsilon_{cu} \) in compression; herein the ultimate concrete compressive strain is the ACI maximum usable strain of 0.003, or the outermost layer of tension steel reaching its ultimate \( \varepsilon_s = \varepsilon_u \).

It should be noted that for the beams in Spans 4, 5, 6 and 7, failure in the un-strengthened beam resulted from \( \varepsilon_c = \varepsilon_{cu} \). For strengthened beams with CFRP laminates, the laminates reach \( \varepsilon_{fut} \) prior to \( \varepsilon_c \) reaching \( \varepsilon_{cu} \) or \( \varepsilon_s \) reaching \( \varepsilon_u \). As previously indicated, following the rupture of CFRP laminates, the beam behaves as an un-strengthened beam and failed when \( \varepsilon_c \) reached \( \varepsilon_{cu} \).

Unless all loads on a member, including self-weight, can be removed prior to the installation of an FRP system, the substrate to which the FRP is applied will be strained. The initial strain, \( \varepsilon_{fi} \), which exists when all possible loads are to be considered during the FRP application process were accounted for in this project. The initial strain level, due primarily to the self-weight of the structural members, can be determined from elastic analysis of the member and can be identified on the moment-curvature curve of the original member. Let \( \varepsilon_f \) be the strain at the FRP level (Fig. 2.5) of the strengthened beam section, defined as:

\[
\varepsilon_f = \varepsilon_{fi} + \varepsilon_{ff} \tag{2.1}
\]

As indicated previously, there is a possibility that tensile rupture of CFRP laminates could occur prior to concrete reaching its pre-defined limiting strain of 0.003. In that case, upon tensile rupture, the load-deformation characteristics of the strengthened
member would revert back to the behavior shown for an un-strengthened beam section. However, strengthening with CFRP laminates is such that it would ensure that the beam has ample strength to resist the anticipated overload as presented in the section to follow.

2.4 Design Recommendations

Finite element and moment-curvature analyses were carried out for each girder cross-section as the beams for Spans 4, 5, 6, and 7 have different design layout due to the difference in geometry and loading in the original design. The results indicate that many of the reinforced concrete girders, except for Span 5, were stressed beyond the specified allowable permitted by AASHTO Code to varying degrees [ranging from 2% to 16% (refer to results in Appendix B)].

The amount of CFRP laminates needed to strengthen the beams was determined via moment-curvature analyses. Fig. 2.3 shows the amount of CFRP laminates, produced by Sika Corporation, required for flexural strengthening. Moment-curvature analysis was necessary in determining the appropriate amount of material required, since the ultimate strength of the retrofitted sections was dictated by FRP rupture (refer to results in Appendix B).

![Fig. 2.3 – CFRP laminates required for beam strengthening.](image)

(No strengthening is required for girders in Span 5)
It should be noted that all of the girders in Span 5 were excluded from this retrofit procedure because these girders were determined to possess sufficient strength as to resist actual service loads. Girders 6 to 10 of Spans 4 and 6 were also excluded from strengthening for the same reason.
3. CONSTRUCTION PHASE

3.1 Introduction

The Louisa-Fort Gay retrofitting project began in September 2003 and was completed in October 2003. The sections to follow describe the work involved in the retrofitting process: (1) surface preparation, and (2) application of the CFRP laminates.

3.2 Surface Preparation

Surface preparation ensures the cleanliness and soundness of the affected areas where bond is critical. The affected areas of selected girders were ground and cleaned to remove all loose concrete particles, debris, and other contaminants that would have affected the bond between the laminates and the concrete substrate. In addition, concrete pull-out tests (ACI 503 1992) were conducted on the concrete substrate of each girder to ensure sufficient tensile strength. In accordance with the ACI code, all affected concrete surfaces were required to possess a minimum strength of 200 psi to ensure successful bonded application of laminates. The surface grinding process is shown in Fig. 3.1. It should be noted that the surface preparation process generally involved light-weight hand-tools (i.e. surface grinder, pressure blower, etc) and only required minimal labor (Fig. 3.1).

Fig. 3.1 – Preparing concrete substrate for CFRP laminates.
3.3 Application of CFRP Laminates

The application of CFRP laminates involved the following phases: (1) applying the mixed epoxy onto the concrete with a trowel or spatula to a specified thickness, (2) applying the mixed epoxy onto the CFRP laminates; (3) placing the CFRP laminates to the affected concrete surfaces; (4) pressing the CFRP laminates using a hard-roller until adhesive is forced out on both sides; and (5) removing excess adhesive. Clamps were used in various locations along the girders to secure the CFRP laminates while allowing the adhesive to properly cure. Figure 3.2 shows the process of a CFRP laminate being attached to a reinforced concrete girder. Completed retrofitting work of Spans 4 and 6 are shown in Fig. 3.3.

Fig. 3.2 – Application of CFRP laminate to RC girder.
Fig. 3.3 – Reinforced concrete beams retrofitted with CFRP laminates.

(a) Span 6 of the middle reinforced concrete span
(b) Span 4 of the middle reinforced concrete span
4. Post-Repair Monitoring

4.1 Crack Monitoring

The Louisa-Fort Gay Bridge was monitored for a period of three years, following the completion of the retrofit in October 2003 for crack propagation and movement. Avongard crack gauges (Fig. 4.1) were installed at the affected areas of the girders. A crack gauge consists of two overlapping plexiglass/acrylic plates as shown in Fig. 4.1. One plate consists of a black millimeter grid over a white background; the other plate is transparent, with red crosshairs centered over the grid. Any movement of the crack will cause the crosshairs to shift away from the origin of the grid. Since the completion of the retrofit in October 2003, no crack movement has been observed.

![The Avongard crack gauge.](image)

Fig. 4.1 – The Avongard crack gauge.
5. SUMMARY AND CONCLUSION

Routine inspection of the Louisa-Fort Gay Bridge revealed that flexural cracks have formed in the continuous reinforced concrete girders in the positive bending regions. In order to characterize the existing service load conditions of the bridge, vehicle classification and truck weight data were collected and analyzed. It was concluded that the trucks that were concurrently operating and traversing the bridge may have been carrying loads exceeding the legal weight limit. Further evaluation confirmed that most of the reinforced concrete girders were stressed beyond the allowable service limits specified by the American Association of State and Highway Transportation Official (AASHTO) Specification. In order to accommodate excessive service loads, as well as to extend the service life of the bridge, recommendations were to use carbon fiber reinforced polymer (CFRP) laminates to retrofit the reinforced concrete girders. A detailed moment-curvature analysis was carried out to establish the appropriate amount of laminates required for flexural strengthening. The retrofit was implemented in September of 2003 and the project was completed in October of 2003. Crack gauges were installed at critical crack locations to monitor the effect of the retrofit. As of September 28 2006, no crack movement has been observed.
References


APPENDIX A
Live Load Analysis
This section contains information related to the determination of live load distribution factors for parallel girders of the Louisa-Fort Gay Bridge.

**Interior Girders:**

Concrete T-beam

Average girder spacing \( S = 9.5 \text{ ft} \)  
(If \( S \) exceeds 10 ft, use footnote f.)

The distribution factor \( (DF) \) is:

\[
DF = \frac{S}{6}  
\]

(Bridge designed for two or more traffic lanes)

\[
= 1.583
\]

The impact factors \( (I) \) for different spans are:

\[
I = \frac{50}{(L + 125)}
\]

Where \( L \) is length, in feet, of the portion of the span that is loaded to produce the maximum stress in the member.

Then the live-load bending moment for the interior **T-girders 2 and 3** would be:

\[
M_{L+1} = \text{(live-load moment due to one truck)} \times 0.5 \times (DF) \times (1+ I)
\]

**Exterior Girders:**

The distribution factors are determined by the lever rule. The **lever rule** is a method of static analysis. It involves a distribution of load based on the assumption that each deck panel is simply supported over the girder, except at the exterior girder, which is continuous with the cantilever. Because the load distribution to any girder other than one directly next to the point of load application is neglected, the lever rule is a conservative method of analysis.
The live-load bending moment for the exterior girder 1 and interior girder 4 would be:

\[ M_{L+I} = (\text{live-load moment due to one truck}) \times 0.5 \times (DF) \times (1+f) \]
APPENDIX B

Moment-Curvature Analysis

This section contains the results of the moment-curvature analyses of the RC girders in Spans 4, 6, and 7.
Table B-1. Service loads and design loads of Section 11-11 for Girder 2 in Span 4.

<table>
<thead>
<tr>
<th>Load levels</th>
<th>Service moments (lb-in)</th>
<th>Design moments (lb-in)&lt;sup&gt;2&lt;/sup&gt;</th>
<th>&lt;sup&gt;3&lt;/sup&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>A. M&lt;sub&gt;DL&lt;/sub&gt;</td>
<td>14.7 x 10&lt;sup&gt;6&lt;/sup&gt;</td>
<td>19.1 x 10&lt;sup&gt;6&lt;/sup&gt;</td>
<td></td>
</tr>
<tr>
<td>B. M&lt;sub&gt;DL&lt;/sub&gt; &amp; M&lt;sub&gt;LL&lt;/sub&gt; (AASHTO)</td>
<td>24.1 x 10&lt;sup&gt;6&lt;/sup&gt;</td>
<td>39.5 x 10&lt;sup&gt;6&lt;/sup&gt;</td>
<td></td>
</tr>
<tr>
<td>C. M&lt;sub&gt;DL&lt;/sub&gt; &amp; M&lt;sub&gt;LL&lt;/sub&gt; (Type 6 – WIM)</td>
<td>24.3 x 10&lt;sup&gt;6&lt;/sup&gt;</td>
<td>40.0 x 10&lt;sup&gt;6&lt;/sup&gt;</td>
<td></td>
</tr>
<tr>
<td>D. M&lt;sub&gt;DL&lt;/sub&gt; &amp; M&lt;sub&gt;LL&lt;/sub&gt; (Type 9 – LEGAL)</td>
<td>27.0 x 10&lt;sup&gt;6&lt;/sup&gt;</td>
<td>45.7 x 10&lt;sup&gt;6&lt;/sup&gt;</td>
<td></td>
</tr>
<tr>
<td>E. M&lt;sub&gt;DL&lt;/sub&gt; &amp; M&lt;sub&gt;LL&lt;/sub&gt; (Type 9 – WIM)</td>
<td>36.6 x 10&lt;sup&gt;6&lt;/sup&gt;</td>
<td>66.6 x 10&lt;sup&gt;6&lt;/sup&gt;</td>
<td></td>
</tr>
</tbody>
</table>

<sup>1</sup> Refer to Fig B.1.a  
<sup>2</sup> Refer to Fig B.1.b  
<sup>3</sup> bold numbers indicate that the factored applied loads exceed the capacity of the original section.

Note: all sections are retrofitted with the same configurations (1-4 laminates) shown here.

Fig. B.1.a – Comparison of service moments with M<sub>n</sub>-φ<sub>n</sub>.

Fig. B.1.b – Comparison of design moments with φM<sub>n</sub>-φ<sub>n</sub>.

Number of strips required = 1
Table B-2. Service loads and design loads of Section 15-15 for Girder 3 in Span 6.

<table>
<thead>
<tr>
<th>Load levels</th>
<th>Service moments (lb-in)</th>
<th>Design moments (lb-in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A. $M_{DL}$</td>
<td>$6.4 \times 10^6$</td>
<td>$8.3 \times 10^6$</td>
</tr>
<tr>
<td>B. $M_{DL}$ &amp; $M_{LL}$ (AASHTO)</td>
<td>$12.5 \times 10^6$</td>
<td>$21.6 \times 10^6$</td>
</tr>
<tr>
<td>C. $M_{DL}$ &amp; $M_{LL}$ (Type 6 – WIM)</td>
<td>$12.8 \times 10^6$</td>
<td>$22.3 \times 10^6$</td>
</tr>
<tr>
<td>D. $M_{DL}$ &amp; $M_{LL}$ (Type 9 – LEGAL)</td>
<td>$13.8 \times 10^6$</td>
<td>$24.3 \times 10^6$</td>
</tr>
<tr>
<td>E. $M_{DL}$ &amp; $M_{LL}$ (Type 9 – WIM)</td>
<td>$19.6 \times 10^6$</td>
<td>$36.9 \times 10^6$</td>
</tr>
</tbody>
</table>

Refer to Fig. B.2.a

Fig. B.2.a – Comparison of service moments with $M_n\phi_n$.

Refer to Fig. B.2.b

Note: all sections are retrofitted with the same configurations (1-4 laminates) shown here.

Fig. 6.b – Comparison of design moments with $\phi M_n\phi_n$.

Number of strips required = 2
Table B-3. Service loads and design loads of Section 17-17 for Girder 2 in Span 7.

<table>
<thead>
<tr>
<th>Load levels</th>
<th>Service moments (lb-in)</th>
<th>Design moments (lb-in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A. MDL</td>
<td>8.3 x 10^6</td>
<td>10.8 x 10^6</td>
</tr>
<tr>
<td>B. MDL &amp; MLL (AASHTO)</td>
<td>15.7 x 10^6</td>
<td>26.8 x 10^6</td>
</tr>
<tr>
<td>C. MDL &amp; MLL (Type 6 – WIM)</td>
<td>15.9 x 10^6</td>
<td>27.4 x 10^6</td>
</tr>
<tr>
<td>D. MDL &amp; MLL (Type 9 – LEGAL)</td>
<td>17.1 x 10^6</td>
<td>30.0 x 10^6</td>
</tr>
<tr>
<td>E. MDL &amp; MLL (Type 9 – WIM)</td>
<td>24.1 x 10^6</td>
<td>45.0 x 10^6</td>
</tr>
</tbody>
</table>

1 Refer to Fig. B.3.a  
2 Refer to Fig. B.3.b  
3 bold numbers indicate that the factored applied loads exceed the capacity of the original section

Note: all sections are retrofitted with the same configurations (1-4 laminates) shown here.

Fig. B.3.a – Comparison of service moments with $M_n \phi_n$.

Number of strips required = 2

Fig. B.3.b – Comparison of design moments with $\phi M_n \phi_n$. 

B-4
Table B-4. Service loads and design loads of Section 18-18 for Girder 3 in Span 4.

<table>
<thead>
<tr>
<th>Load levels</th>
<th>Service moments (lb-in)(^1)</th>
<th>Design moments (lb-in)(^2,3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A. (\text{MDL})</td>
<td>(15.6 \times 10^6)</td>
<td>(20.3 \times 10^6)</td>
</tr>
<tr>
<td>B. (\text{MDL} &amp; \text{MLL (AASHTO)})</td>
<td>(24.9 \times 10^6)</td>
<td>(40.6 \times 10^6)</td>
</tr>
<tr>
<td>C. (\text{MDL} &amp; \text{MLL (Type 6 – WIM)})</td>
<td>(25.2 \times 10^6)</td>
<td>(41.1 \times 10^6)</td>
</tr>
<tr>
<td>D. (\text{MDL} &amp; \text{MLL (Type 9 – LEGAL)})</td>
<td>(25.6 \times 10^6)</td>
<td>(46.8 \times 10^6)</td>
</tr>
<tr>
<td>E. (\text{MDL} &amp; \text{MLL (Type 9 – WIM)})</td>
<td>(37.4 \times 10^6)</td>
<td>(67.7 \times 10^6)</td>
</tr>
</tbody>
</table>

\(^1\) Refer to Fig. B.4.a  \(^2\) Refer to Fig. B.4.b  \(^3\) bold numbers indicate that the factored applied loads exceed the capacity of the original section.

Note: all sections are retrofitted with the same configurations (1-4 laminates) shown here.

Fig. B.4.a – Comparison of service moments with \(M_n\phi_n\).

Number of strips required = 3

Fig. B.4.b – Comparison of design moments with \(\phi M_n\phi_n\).
Table B-5. Service loads and design loads of Section 19-19 for Girder 4 in Span 7.

<table>
<thead>
<tr>
<th>Load levels</th>
<th>Service moments (lb-in)</th>
<th>Design moments (lb-in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A. $M_{DL}$</td>
<td>$9.8 \times 10^6$</td>
<td>$12.7 \times 10^6$</td>
</tr>
<tr>
<td>B. $M_{DL}$ &amp; $M_{LL}$ (AASHTO)</td>
<td>$15.9 \times 10^6$</td>
<td>$27.1 \times 10^6$</td>
</tr>
<tr>
<td>C. $M_{DL}$ &amp; $M_{LL}$ (Type 6 – WIM)</td>
<td>$16.2 \times 10^6$</td>
<td>$27.7 \times 10^6$</td>
</tr>
<tr>
<td>D. $M_{DL}$ &amp; $M_{LL}$ (Type 9 – LEGAL)</td>
<td>$17.3 \times 10^6$</td>
<td>$30.3 \times 10^6$</td>
</tr>
<tr>
<td>E. $M_{DL}$ &amp; $M_{LL}$ (Type 9 – WIM)</td>
<td>$24.3 \times 10^6$</td>
<td>$45.3 \times 10^6$</td>
</tr>
</tbody>
</table>

1 Refer to Fig. B.5.a  2 Refer to Fig. B.5.b  3 Bold numbers indicate that the factored applied loads exceed the capacity of the original section

Note: all sections are retrofitted with the same configurations (1-4 laminates) shown here.

Fig. B.5.a – Comparison of service moments with $M_n \cdot \phi_n$.

Number of strips required = 4

Fig. B.5.b – Comparison of design moments with $\phi M_n \cdot \phi_n$. 
Table B-6. Service loads and design loads of Section 20-20 for Girder 4 in Span 4.

<table>
<thead>
<tr>
<th>Load levels</th>
<th>Service moments (lb-in)</th>
<th>Design moments (lb-in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A. $M_{DL}$</td>
<td>$16.9 \times 10^6$</td>
<td>$22.0 \times 10^6$</td>
</tr>
<tr>
<td>B. $M_{DL} &amp; M_{LL}$ (AASHTO)</td>
<td>$24.2 \times 10^6$</td>
<td>$37.8 \times 10^6$</td>
</tr>
<tr>
<td>C. $M_{DL} &amp; M_{LL}$ (Type 6 – WIM)</td>
<td>$23.9 \times 10^6$</td>
<td>$37.3 \times 10^6$</td>
</tr>
<tr>
<td>D. $M_{DL} &amp; M_{LL}$ (Type 9 – LEGAL)</td>
<td>$25.6 \times 10^6$</td>
<td>$40.9 \times 10^6$</td>
</tr>
<tr>
<td>E. $M_{DL} &amp; M_{LL}$ (Type 9 – WIM)</td>
<td>$32.5 \times 10^6$</td>
<td>$55.9 \times 10^6$</td>
</tr>
</tbody>
</table>

1 Refer to Fig. B.6.a  
2 Refer to Fig. B.6.b  
3 Bold numbers indicate that the factored applied loads exceed the capacity of the original section

Fig. B.6.a – Comparison of service moments with $M_n\cdot \phi_n$.

Fig. B.6.b – Comparison of design moments with $\phi M_n\cdot \phi_n$. 

Note: all sections are retrofitted with the same configurations (1-4 laminates) shown here.
APPENDIX C
Carbon Fiber Reinforced Polymer (CFRP) Laminate
This section contains information related to CFRP laminates used in the strengthening process.

CFRP laminates produced by Sika were used in the project. The following are the properties of Sika CarboDur® laminate, used for flexural strengthening:

![Graph showing physical and mechanical properties of CFRP laminate.](image)

Fig. C.1: Physical and mechanical properties of CFRP laminate used for flexural strengthening.
For more information or a complete publication list, contact us at:

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