MoDOT

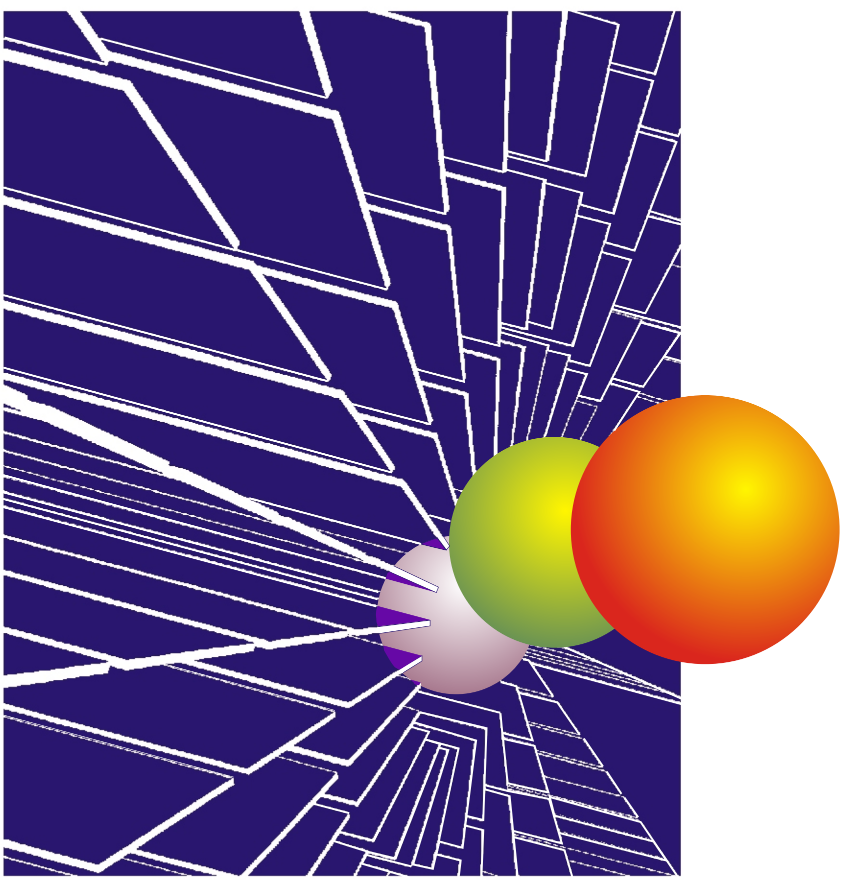
Research, Development and Technology

Co-Force America, Inc.

RDT 01-017

Repair and Strengthening of Impacted PC Girders on Bridge A4845 Jackson County, Missouri

RI 01-016



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16. Abstract

This project consisted of the repair and strengthening of 11 PC girders that were damaged by vehicular impact on Bridge A4845, Jackson County, MO. The repair/strengthening was achieved by externally bonded FRP reinforcement. Co-Force America, Inc. carried out the design for the repair/strengthening of the girders and provided quality control in the initial stages of the construction. Additional strips of FRP were installed by the University of Missouri-Rolla for long-term bond durability studies in the form of bond pull-off and torsion tests. The initial tests have shown good adhesion of the laminate to the concrete substrate. Follow up tests will be performed at one-year intervals for the next 5 years. A separate report will be submitted by UMR once all of the bond durability tests are completed.

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REPAIR AND STRENGTHENING OF IMPACTED PC GIRDERS ON BRIDGE A-4845

JACKSON COUNTY, MO

Final Report

Prepared for the

Missouri Department of Transportation (MODOT)

Project No. RI01-016

Submitted

November 2001

Executive Summary

Eleven PC girders of Bridge A-4845 located over Route 291 on Route 24 in Independence, MO, were impact-damaged by overheight vehicle/s causing concrete to spall and exposing rebar and prestressing strands. This project consisted of the repair and strengthening of the eleven girders.

After applying concrete patches to those sections that had been damaged, the strengthening was achieved by externally bonded FRP reinforcement using established methods and ensuring that the new ultimate flexural capacity be equal or higher than the original one. Co-Force America, Inc. (CFA) carried out the design for the strengthening of the girders and provided quality control in the initial stages of the construction. MoDOT personnel repaired and strengthened the girders without incidents or problems after they had been trained by CFA engineers on the proper installation methods.

Additional strips of FRP were installed by the University of Missouri-Rolla (UMR) for long-term bond durability studies in the form of bond pull-off and torsion tests. The initial tests have shown good adhesion of the laminate to the concrete substrate. Follow up tests will be performed at one-year intervals for the next five years. A separate report will be submitted by UMR once all of the bond durability tests are completed.

Acknowledgements

The authors would like to thank Anand Khataukar, graduate student, and Jason Cox, senior laboratory technician, at the University Transportation Center of the University of Missouri Rolla. They were responsible for applying the additional FRP laminates and conducting the in-situ bond tests.

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1 Introduction

1.1 Scope

Eleven prestressed concrete (PC) girders of Bridge A-4845 located over Route 291 on Route 24 in Independence, MO, were impact-damaged by overheight vehicles causing concrete to spall and exposing rebar and prestressing strands. The location of the bridge is shown in Appendix A.

Figure 1 – Impacted Girders on Bridge A4845

1.2 Background

There has been limited research on the repair of PC bridge girders damaged by vehicular impact. Traditional techniques used to repair concrete structures may be expensive, time consuming, and of limited effectiveness. On the other hand, FRP bonded reinforcement is an effective strengthening technique in terms of installation, design, and performance. In the event of FRP damage due to vehicular impact or other reasons, the external system can be removed and replaced with a new one.

From a National Cooperative Highway Research Program (NCHRP) perspective, two publications (Shanafelt and Horn 1980 and 1985) address this topic. Researchers at Iowa State University have recently published a comprehensive report (Klaiber et al. 1999). This document includes an extensive annotated bibliography as well as results from experiments conducted in the field and in the laboratory. With respect to US experience, in addition to Iowa, Departments of Transportation of other states such as Georgia (Aboutaha et al. 1997), Minnesota (Olson et al. 1992), and Texas (Zobel et al. 1997) have supported work in this area. Under the repetitive nature of highway loading, repair methods such as internal strand splices and external post-tensioning were found to be only partially satisfactory because they could not restore the ultimate strength to the damaged member (Olson et al. 1992; Zobel and Jirsa 1998). Strengthening of reinforced concrete (RC) and PC structures using externally bonded steel plates and composite laminates has proven to be an effective method for decreasing or restoring structural capacity (Dolan et al. 1999). Fiber reinforced polymer (FRP) composites come in the form of pre-cured laminates or fiber sheets to be installed by hand lay-up. The application of the latter offers several advantages such as ease of bonding to curved or irregular surfaces, lightweight, and the fact that fibers can be oriented along any direction. Strengthening of impact-damaged girders with FRP laminates has already been explored (Nanni 1997). A similar project involving the strengthening of a damaged bridge girder has been conducted by the authors. A truck impacted the four girders of the bridge overpass on highway Appia, near Rome (Nanni 1997). This is a short bridge, 10.5 m in span, made of four prestressed concrete girders having cross sectional dimensions of 1.0 by 1.5 m. The concrete cross section was restored with no-shrink mortar and, after surface preparation, CFRP sheets were adhered to the concrete (see Figure 2). Two Research projects (Bridge G270 see Mayo et al. 1999, and Bridge J857 see Alkhrdaji et al. 1999) and two highway bridge repair projects (A3050 and A10062, see Figure 2) using advanced composites technology have already been conducted under MoDOT sponsorship by researchers at UMR.

Bridge on Highway Appia Bridge A3050 Bridge A10062

Figure 2 - Examples of Accidental Damage Repair Using FRP Technology

In this project, it was decided to use carbon FRP (CFRP) laminates installed by manual lay-up to restore the original structural capacity of the girder. Co-Force America, Inc. (CFA) work involved the design of the CFRP strengthening, specifications for construction, and quality control of the installation. Quality control tests, performance monitoring, and bond-behavior investigation of the strengthening system were conducted by the University of Missouri-Rolla (UMR).

The project provided a new opportunity to use FRP technology for accidental damage repair. Data collected from this project will add to existing information and will be used to establish guidelines for repair specifications of highway bridges.

2 Design and Strengthening

The design for these girders was based on the premise that the loss of ultimate flexural capacity due to the damage to the prestressing tendons in the girders would be provided by the bonded CFRP. The MoDOT inspection crew stated that only three girders had been damaged enough to expose any tendons, and that the maximum number of tendons exposed in a single girder was two (see Figure 3a & 3b).

(a) (b)

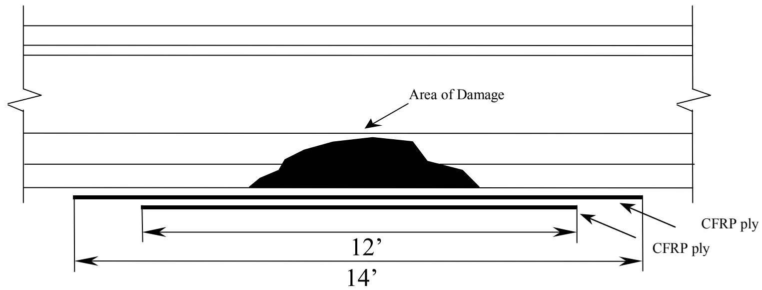
Figure 3 – Exposed Tendons on Bridge A4845

Girder details were based on the structural drawings provided by MoDOT. The girders are standard MoDOT Type 6 Prestressed I-girders with a total height of 4.5 ft, a top and bottom flange width of 2 ft, and a web width of 6.5 in. Each girder has a total of 24, 0.5 in. tendons with an assumed effective stress f_{pe} of 165 ksi. The concrete strength was assumed to be 4000 psi. The ultimate tensile strength of the fiber composite strengthening system is 550 ksi with a maximum elongation of 0.017 in./in. According to AASHTO, the strength reduction factor for prestressed girders is ϕ =1.0.

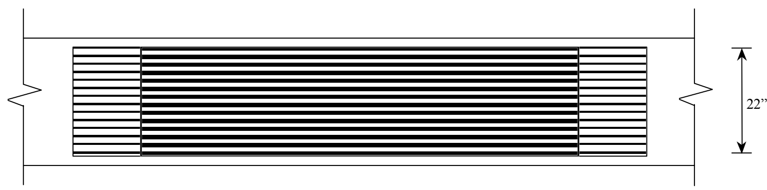
The factored moment capacity (ϕM_n) of an undamaged composite slab/girder is approximately 4408 kip-ft. Capacity calculations are achieved using an effective slab width of 96.5 in. and slab thickness of 7.5 in. and considering a 1-in haunch between the girders and the topping slab. In order to design the strengthening for the impacted girders two tendons were assumed to be damaged at 50% or one tendon totally broken. The capacity of a damaged girder is 4221 k-ft, making the capacity required for the strengthening to be 187 k-ft.

It was determined that two CFRP sheets be bonded to the soffitt of each girder. The moment capacity of a girder section after this strengthening is ϕM_n = 4407 k-ft. The CFRP will increase the moment capacity by approximately 4.4%. The bonded CFRP, in addition to its structural contribution, will help to prevent deterioration of the repaired areas. See Appendix B for the detailed strengthening design calculations.

The design called for the installation of two plies of 22 in. wide strips of CFRP to the soffitt of each beam. The first ply was 14 ft. in length and the second was 12 ft. in length. Figure 4 below shows a sketch of the longitudinal strip installation. In order to resist peeling of the longitudinal sheets a total of 12 strips of CFRP were wrapped around the bulb of the girders over the length of the repair. These strips are 6 in. wide and extend from the bottom of the web on one side of the bulb to the bottom of the web on the opposite side. The strips are spaced at 16 in. on center. Figure 5 below shows a sketch of the installation of the anchor strips.

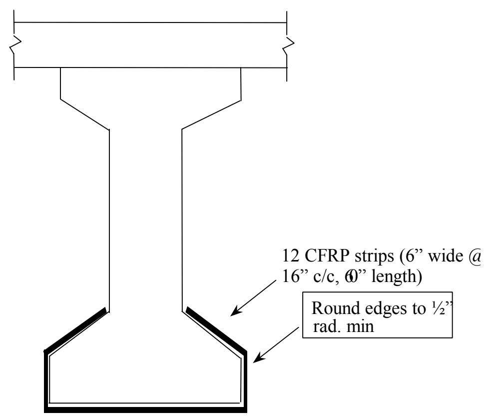


Side View

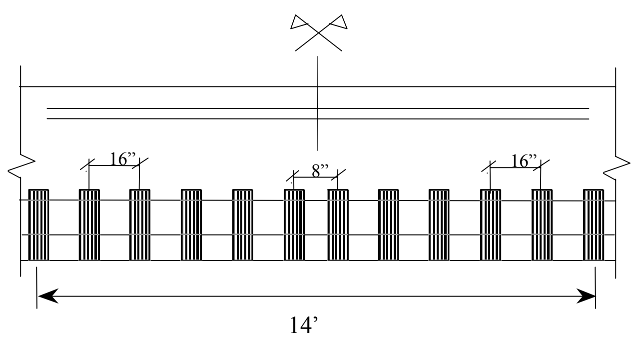


Bottom View

Figure 4 – Details of Longitudinal CFRP Sheets on Girder Soffitt



Cross Section



Side View

Figure 5 – Details of CFRP Anchor Strips

2.1 Strengthening Procedures

After the design had been completed and submitted the strengthening could be performed. In order to strengthen the impacted girders, they first had to be repaired. This consisted of applying concrete patches to those sections that had been damaged. Once the concrete had been repaired, the strengthening system could be applied.

A MoDOT bridge repair crew conducted the application of the CFRP strengthening as specified in the design. The MBrace Composite Strengthening System was the material used and was applied according to the manufacturer's recommendations (MBT 1998). A brief summary of the procedures for application of CFRP sheets is given below.

1. Surface Preparation: The surface of the structure is cleaned and prepared for installation through the use of sand blasting. This is to remove laitance and open the concrete pores.

2. Primer Application: A coat of MBrace Primer is applied to the concrete surface. The low-viscosity epoxy-based primer prepares the surface of the concrete for the application of the CFRP sheets.

3. Putty Application: A very thin coat of MBrace Putty is smoothed over the surface to fill in any small voids, cracks or uneven surfaces (See Figure 6)

4. Saturant: A layer of MBrace Saturant is applied next. This precedes the installation of the first layer of carbon fibers.

5. CFRP Sheets: The first ply of carbon fibers is then applied. The sheet is rolled into the saturant to insure good impregnation.

6. Saturant: The application of a second layer of saturant follows. The sheet is rolled thoroughly to ensure good penetration of the resin around the fibers

7. Additional plies can be installed until the required thickness of laminate is attained.

(a)

(b)

Figure 6 – MoDOT Personnel Applying MBrace Putty to a Damaged Girder

3 Long-Term Bond Durability Testing

The final aspect of this project is the continuing long-term bond durability monitoring being carried out by the University of Missouri-Rolla. This report outlines the concepts and procedures for the testing as well as the findings of the initial testing. As the testing is to proceed for a total of five years, the final report on the bond durability monitoring will be submitted by UMR at the time it is completed.

A team from the university installed five strips of CFRP on the bridge girder in a similar fashion to that adopted during the strengthening system installation. The five strips were installed at one end of the bridge away from the traffic lanes (see Figure 7) to avoid traffic disruption during testing. These strips will be used to perform a series of tests once every five years. The tests consist of bond pull-off and torsion tests and will help evaluate the performance of the installed strengthening over time.

Figure 7 – FRP Strips for Bond Durability Testing

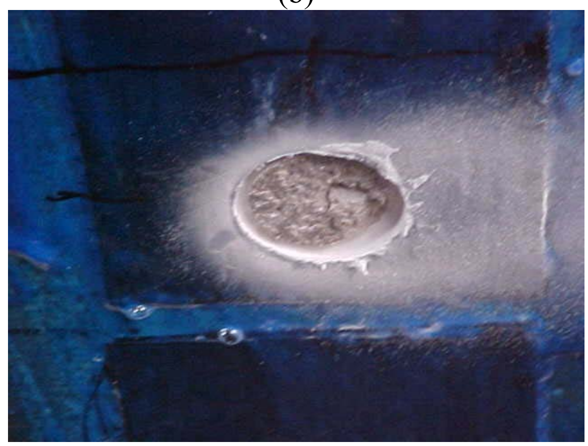
3.1 Pull-off Tests

The pull-off test is a simple and straightforward test to perform. A fixture of 1.625-in. diameter is adhered to the surface of the FRP with an epoxy adhesive (Figure 8a). After the epoxy is allowed to cure and the fixture is firmly attached to the FRP, the fixture is isolated from the surrounding FRP with a core drill (Figure 8b). The test apparatus is then attached to the fixture and aligned so that tension is applied perpendicular to the concrete surface (Figure 8c). A force is applied at a constant rate until the fixture detaches from the surface (Figure 8d). There are three possible failure modes for the pull-off test; 1) failure in the concrete substrate, 2) failure at the interface FRP laminate-concrete, or 3) failure in the epoxy adhesive compound (fixture delamination). Of the three failure modes, the third one is unacceptable in that it shows fixture installation problems. The second failure mode may be indicative of a bond durability problem. The first failure mode is the desirable one and should occur at a stress value larger than 200 psi. A full description of the pull-off test for concrete repair evaluation is given in the International Concrete Repair Institute's "1999 Concrete Repair Manual" (ICRI, 1999).

(a)

(b)

(c)



(d)

Figure 8 – Pull-off Test Procedures

The results of the pull-off tests conducted in August 2001 are given in Table 1. The load at which the specimen detached from the surface is given in the second column with the corresponding stress displayed in the third column. The ultimate stress (or bond tensile strength) is computed al the ultimate load divided by the fixture area. It is to be noted that according to ICRI recommendations, the minimum threshold is 200 psi; therefore, these results can be considered highly satisfactory. The results of the pull-off tests show that the failure was in the concrete indicating a strong bond between the FRP and the concrete surface. The approximate percentages of disk area as covered by concrete or exposed FRP is given in the fourth column of the table.

Table 1 – Results of the Initial Pull-off Tests

Pull-off Tests	Load (lb)	Stress (psi)	Disk Area Inspection
P1	1000	482	25% FRP, 75% concrete
P2	1200	579	30% FRP, 70% concrete
P3	1000	482	35% FRP, 65% concrete

Photos for the initial pull off tests are shown in Figure 9. The photos on the left display the locations of the pull-off test and the photos on the right display the sample discs.

(a) Location of P1

(b) P1 disc

(c) Location of P2

(d) P2 disc

(e) Location of P3

(f) P3 disc

Figure 9 – Pull-off Test Results

3.2 Torsion Tests

To perform the torsion tests, special disks are adhered to the FRP with epoxy. The disks are identical to the ones used for pull-off, with the exception of a grid of 4 grooves to improve bonding of the adhesive to the steel surface. Similar to the pull-off test, the FRP was cut along the perimeter of the disk using a core drill. Torsion was applied using a calibrated torque wrench until the fixture separated from the surface (Figure 10a & 10b).

(a)

(b)

Figure 10 – Apparatus for Torsion Testing

The shear stress that develops as the result of the applied torque has a triangular distribution with maximum at the external location and zero at the center of the circle. Due to this stress distribution, it is common practice to report the average shear stress equal to half the peak (Khataukar, 2001). The average shear stresses were calculated by taking the average of individual tests results with identical mode of failure.

Three samples were used for the initial torsion tests. The results of the tests are given in Table 2 below. The torque at which the disc separated from the surface is given in the second column. The third column displays the corresponding ultimate stress or bond shear strength. The amount of disk surface area covered by the respective materials is given in column 4.

Table 2 – Results of the Initial Torsion Testing

Torsion Tests	Torque (ft-lb)	τ_{max} (psi)	Disk Area Inspection
T1	75	534	60% FRP, 40% concrete
T2	95	676	50% FRP, 50% concrete
T3	100	712	45% FRP, 55% concrete

Figure 11 below displays the photos taken of the torsion test samples. The photos on the left show the locations of the torsion test specimens and the photos on the right show the torsion test discs.

(a) Location of T1

(b) T1 disc

(c) Location of T2

(d) T2 disc

(e) Location of T3

(f) T3 disc

Figure 11 – Torsion Test Results

The results of the torsion tests indicate that the bond is as strong as that indicated in the pull-off tests. This is consistent with results obtained in the laboratory as shown in Figure 12. In this figure, the data from A4845 bridge (hollow square) are compared with results obtained from eight series of specimens on which the FRP laminate was applied on a concrete substrate having temperature ranging from 10 to 120 ^{o}F (Khataukar, 2001).

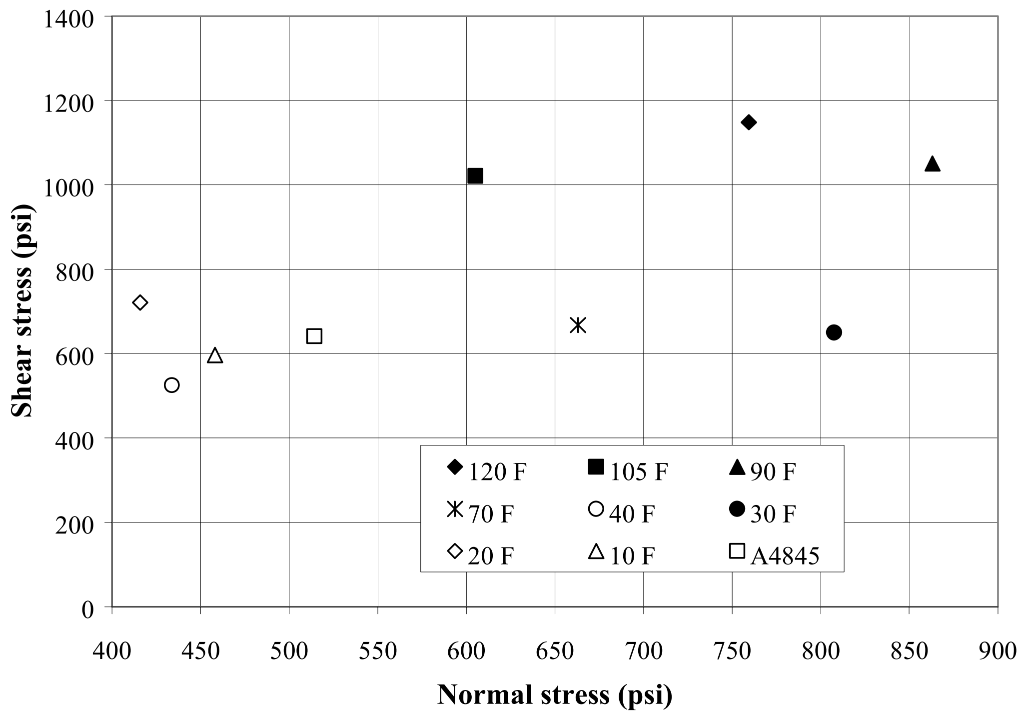


Figure 12 – Relationship Between Results Obtained by Pull-Off and Torsion Tests

4 Summary and Conclusion

Eleven PC girders of Bridge A-4845 located over Route 291 on Route 24 in Independence, MO, were impact-damaged by overheight vehicles causing concrete to spall and exposing rebar and prestressing strands. The damage was repaired and the girders strengthened with externally bonded carbon FRP laminates

The design of the strengthening was achieved using established methods and ensuring that the new ultimate flexural capacity be equal or higher than the original one . The girders were repaired and strengthened by MoDOT personnel without incidents or problems after they had been trained by Co-Force America, Inc. engineers on the proper installation methods.

The initial bond durability tests performed by UMR indicate that the strengthening system has achieved a good bond with the substrate and performance is as expected. Further bond durability testing by UMR in subsequent years will verify these findings.

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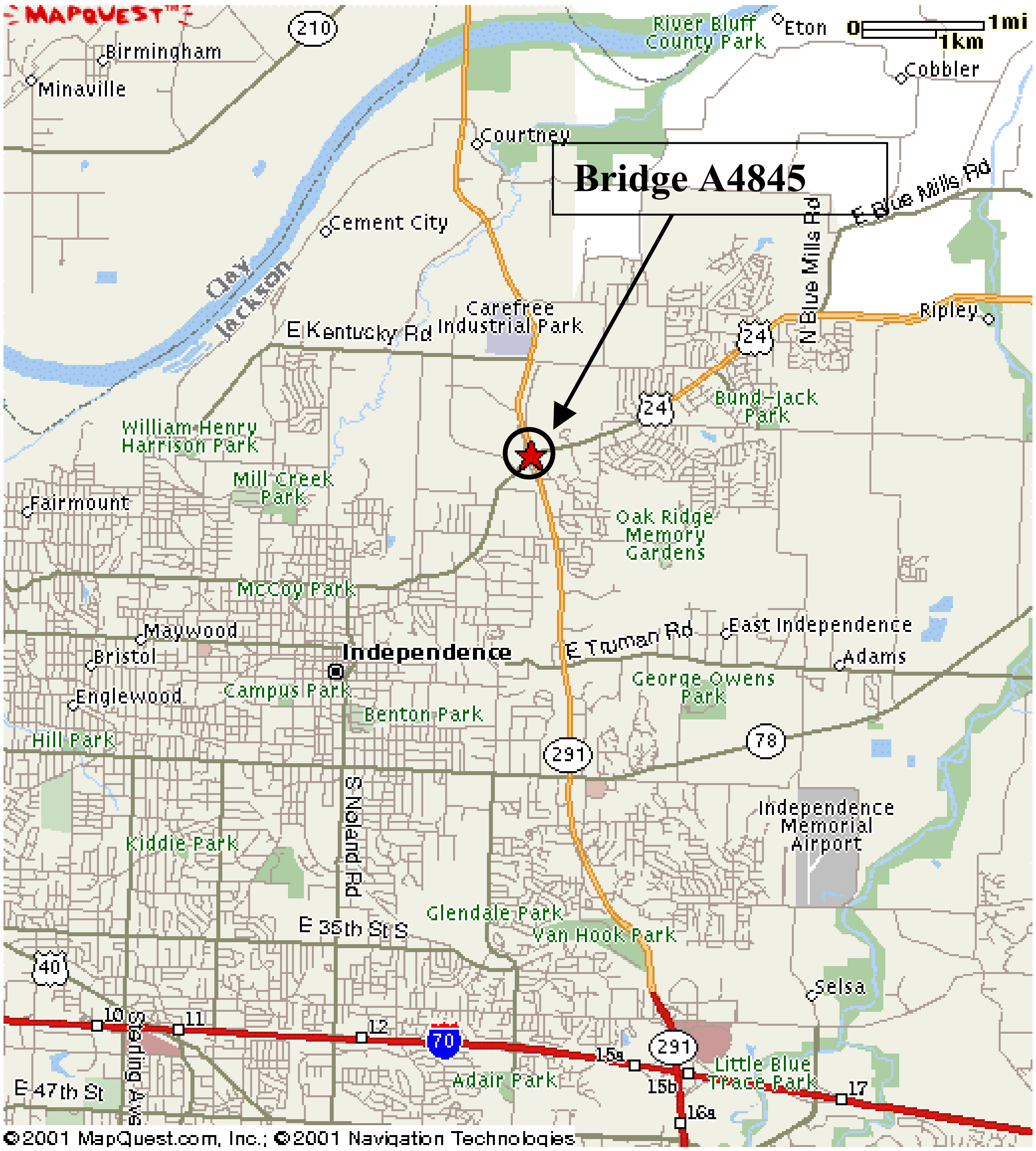
Appendix A: Location of Bridge A4845 in Jackson County, Missouri



Appendix B: Strengthening Design Calculations

Original Capacity

Required Information about the Existing Structure

Section Dimensions

$h := 62.5$	Total section height [in]
$bw := 6.5$	Width of web [in]
$bft := 96.5$	Width of top flange (zero for rectangular or inverted tee sections) [in]
$tft := 7.5$	Thickness of top flange (zero for rectangular or inverted tee sections) [in]
$bfb := 24$	Width of bottom flange (zero for rectangular or tee sections) [in]
$tfb := 6$	Thickness of bottom flange (zero for rectangular or tee sections) [in]

Reinforcement Layout

$As := 0$	Area of mild tension steel [in^2]
$d := 17.5$	Depth to the mild tension steel centroid [in]
$As' := 0$	Area of mild compression steel [in^2]
$d' := 0$	Depth to the mild compression steel centroid [in]
$Ap := .153 \cdot 24$	Area of prestressing steel [in ^{2}]
$dp := 58.83$	Depth to the prestressing steel centroid [in]
$fpe := 165$	Effective stress in the steel due to prestress [ksi]
$Bond := 1$	Type of tendon installation (Enter "1" for bonded, "0" for unbonded)

Load and Span Information

$Mu := 4408$	Factored moment to be resisted by the strengthened element [k-ft]
$Ms := 0.5 \cdot Mu$	Service moment to be resisted by the strengthened element [k-ft]
$Mip := 0.5 \cdot Ms$	Moment in place at the time of MBrace installation [k-ft]
$ln := 0$	Clear span [ft]
$lr := 1.0$	Ratio of loaded spans to total spans (e.g., 0.50 for alternate bay loading) This variable is used only if unbonded tendons are present

Material Property Specifications

$f'c := 4000$	Nominal compressive strength of the concrete [psi]
$fy := 60$	Yield strength of the mild steel [ksi]
$fpu := 270$	Ultimate strength of the prestressing steel [ksi]
$fpy := 243$	Yield strength of the prestressing steel [ksi]
$Ep := 28000$	Modulus of elasticity of the prestressing steel [ksi]

Design Ultimate Moment Capacity

$\phi Mn = 4408.4$ $\quad$ $Mu = 4408$ $\quad$ Design moment capacity [k-ft] vs moment demand [k-ft]

Beam’s Capacity after one tendon has been lost

Required Information about the Existing Structure

Section Dimensions

h := 62.5	Total section height [in]
bw := 6.5	Width of web [in]
bft := 96.5	Width of top flange (zero for rectangular or inverted tee sections) [in]
tft := 7.5	Thickness of top flange (zero for rectangular or inverted tee sections) [in]
bfb := 24	Width of bottom flange (zero for rectangular or tee sections) [in]
tfb := 6	Thickness of bottom flange (zero for rectangular or tee sections) [in]

Reinforcement Layout

As := 0	Area of mild tension steel [in^2]
d := 17.5	Depth to the mild tension steel centroid [in]
As' := 0	Area of mild compression steel [in^2]
d' := 0	Depth to the mild compression steel centroid [in]
Ap := .153·23	Area of prestressing steel [in ^{2}]
dp := 58.83	Depth to the prestressing steel centroid [in]
fpe := 165	Effective stress in the steel due to prestress [ksi]
Bond := 1	Type of tendon installation (Enter "1" for bonded, "0" for unbonded)

Load and Span Information

Mu := 4408	Factored moment to be resisted by the strengthened element [k-ft]
Ms := 0.5·Mu	Service moment to be resisted by the strengthened element [k-ft]
Mip := 0.5·Ms	Moment in place at the time of MBrace installation [k-ft]
ln := 0	Clear span [ft]
lr := 1.0	Ratio of loaded spans to total spans (e.g., 0.50 for alternate bay loading) This variable is used only if unbonded tendons are present

Material Property Specifications

f'c := 4000	Nominal compressive strength of the concrete [psi]
fy := 60	Yield strength of the mild steel [ksi]
fpu := 270	Ultimate strength of the prestressing steel [ksi]
fpy := 243	Yield strength of the prestressing steel [ksi]
Ep := 28000	Modulus of elasticity of the prestressing steel [ksi]

Design Ultimate Moment Capacity

ϕMn = 4221.1   Mu = 4408   Design moment capacity [k-ft] vs moment demand [k-ft]

Strengthening Design

Required Information about the Existing Structure

Section Dimensions

$h := 62.5$	Total section height [in]
$bw := 6.5$	Width of web [in]
$bft := 96.5$	Width of top flange (zero for rectangular or inverted tee sections) [in]
$tft := 7.5$	Thickness of top flange (zero for rectangular or inverted tee sections) [in]
$bfb := 24$	Width of bottom flange (zero for rectangular or tee sections) [in]
$tfb := 6$	Thickness of bottom flange (zero for rectangular or tee sections) [in]

Reinforcement Layout

$As := 0$	Area of mild tension steel [in^2]
$d := 17.5$	Depth to the mild tension steel centroid [in]
$As' := 0$	Area of mild compression steel [in^2]
$d' := 0$	Depth to the mild compression steel centroid [in]
$Ap := .153 \cdot 23$	Area of prestressing steel [in ^{2}]
$dp := 58.83$	Depth to the prestressing steel centroid [in]
$fpe := 165$	Effective stress in the steel due to prestress [ksi]
$Bond := 1$	Type of tendon installation (Enter "1" for bonded, "0" for unbonded)

Load and Span Information

$Mu := 4408$	Factored moment to be resisted by the strengthened element [k-ft]
$Ms := 0.5 \cdot Mu$	Service moment to be resisted by the strengthened element [k-ft]
$Mip := 0.5 \cdot Ms$	Moment in place at the time of MBrace installation [k-ft]
$ln := 0$	Clear span [ft]
$lr := 1.0$	Ratio of loaded spans to total spans (e.g., 0.50 for alternate bay loading) This variable is used only if unbonded tendons are present

Material Property Specifications

$f'c := 4000$	Nominal compressive strength of the concrete [psi]
$fy := 60$	Yield strength of the mild steel [ksi]
$fpu := 270$	Ultimate strength of the prestressing steel [ksi]
$fpy := 243$	Yield strength of the prestressing steel [ksi]
$Ep := 28000$	Modulus of elasticity of the prestressing steel [ksi]

Required MBrace Design Information

MBrace Material Selection

$Fiber := 1$

MBrace Fiber Reinforcement
1 -- MBrace CF 130 High Strength Carbon Fiber
2 -- MBrace CF 160 High Strength Carbon Fiber
3 -- MBrace AK 60 High Performance Aramid Fiber

Durability Reduction Factors

$Ccr := 0.55$ Creep rupture stress limit (Use 0.55 for Carbon, 0.30 for Aramid, and 0.20 for E-Glass

$Ce := 0.95$ Reduction factor to limit strain in the MBrace reinforcement

Layout of the MBrace Reinforcement

$wf := 22$ Width of the MBrace strip [in]

$n := 2$ Number of MBrace plies

Computation of Gross Section Properties

- Effective width of concrete in compression [in]

$$be := if(bft = 0, bw, bft)$$

$$be = 96.5$$

- Cross sectional area [in ^{2}]

$$Ac := bw \cdot h + (bft - bw) \cdot tft + (bfb - bw) \cdot tfb$$

$$Ac = 1.186 \times 10^3$$

- Distance from the top fiber to the centroid [in]

$$ct := \frac{0.5 \cdot bw \cdot h^2 + 0.5 \cdot (bft - bw) \cdot tft^2 + (bfb - bw) \cdot tfb \cdot (h - 0.5 \cdot tfb)}{Ac}$$

$$ct = 18.102$$

- Distance from the bottom fiber to the centroid [in]

$$cb := h - ct$$

$$cb = 44.398$$

- Gross moment of inertia [in ^{4}]

$$Ig := bw \cdot h \cdot \left[\frac{h^2}{12} + \left(\frac{h}{2} - ct\right)^2\right] + (bft - bw) \cdot tft \cdot \left[\frac{tft^2}{12} + \left(ct - \frac{tft}{2}\right)^2\right] + (bfb - bw) \cdot tfb \cdot \left[\frac{tfb^2}{12} + \left(cb - \frac{tfb}{2}\right)^2\right]$$

$$Ig = 5.249 \times 10^5$$

- Radius of gyration [in]

$$r := \sqrt{\frac{Ig}{Ac}}$$

$$r = 21.036$$

Computation of Material Characteristics

- Modulus of elasticity for concrete [psi]

$$Ec := 57000 \cdot \sqrt{f'c}$$

$$Ec = 3.605 \times 10^6$$

- Concrete strain corresponding to f'c [in/in]

$$\varepsilon'c := \frac{1.71 \cdot f'c}{Ec}$$

$$\varepsilon'c = 1.897 \times 10^{-3}$$

- Yield strain for the mild reinforcement [in/in]

$$\varepsilon sy := \frac{fy}{Es}$$

$$\varepsilon sy = 2.069 \times 10^{-3}$$

Preliminary computations for FRP properties

- Design ultimate tensile strength [psi]

$$ffu := Ce\cdot ffu^{*}$$

$$ffu = 522500$$

- Design rupture strain [in/in]

$$\varepsilon fu := Ce\cdot \varepsilon fu^{*}$$

$$\varepsilon fu = 0.016$$

- Bond dependent coefficient for flexure

$$\kappa m := \min\left[\left[1 \quad \begin{vmatrix} \dfrac{1}{60\cdot\varepsilon fu}\cdot\left(1-\dfrac{n\cdot Ef\cdot tf}{2000000}\right) & \text{if } n\cdot Ef\cdot tf < 1000000 \\ \dfrac{1}{60\cdot\varepsilon fu}\cdot\left(\dfrac{500000}{n\cdot Ef\cdot tf}\right) & \text{otherwise} \end{vmatrix}\right]\right]$$

$$\kappa m = 0.811$$

Preliminary computations for prestressing steel properties

- Prestressing force [lbs]

$$Pe := Ap\cdot fpe$$

$$Pe = 5.806\times 10^{5}$$

- Eccentricity of prestressing force [in]

$$e := dp - ct$$

$$e = 40.728$$

- Strain in the tendon at decompression [in/in]

$$\varepsilon p1 := \frac{Pe}{Ap\cdot Ep} + \frac{Pe}{Ac\cdot Ec}\cdot\left(1+\frac{e^{2}}{r^{2}}\right)$$

$$\varepsilon p1 = 6.538\times 10^{-3}$$

- Bond reduction coefficient applied to unbonded tendons

$$\Omega b := \text{if}\left[Bond = 0, \frac{3.0}{\left(\dfrac{ln}{dp}\right)}\cdot lr, 1.0\right]$$

$$\Omega b = 1$$

Moment capacity calculation based on the failure mode, strain compatibility, and equilibrium

- Find the depth to the neutral axis by trial and error [in]

$$c = 5.49$$

- Compute the strain in the concrete if failure is controlled by concrete crushing (Failure Mode 1) [in/in]

$$\varepsilon c1 := 0.003$$

- Compute the strain in the concrete if failure is controlled by tendon rupture (Failure Mode 2) [in/in]

$$\varepsilon c2 := \begin{array}{|l} 0.003 \text{ if } Ap = 0 \\ \text{otherwise} \\ \quad \begin{array}{|l} (0.03 - \varepsilon p1)\cdot\dfrac{c}{dp - c} \text{ if } Bond = 1 \\ \dfrac{1}{\Omega b}\cdot\left(\dfrac{0.94\,fpy}{Ep} - \varepsilon p1\right)\cdot\dfrac{c}{dp - c} \text{ if } Bond = 0 \end{array} \end{array}$$

$\varepsilon c2 = 0.0024$

- Compute the strain in the concrete if failure is controlled by FRP failure (Failure Mode 3) [in/in]

$$\varepsilon c3 := \begin{array}{|l} 0.003 \text{ if } Af = 0 \\ (\kappa m\cdot\varepsilon fu + \varepsilon bi)\cdot\dfrac{c}{h - c} \text{ otherwise} \end{array}$$

$\varepsilon c3 = 0.0013$

- Compute the strain in the concrete based on which mode of failure governs [in/in]

$$\varepsilon c := \min((\varepsilon c1 \;\; \varepsilon c2 \;\; \varepsilon c3))$$

$\varepsilon c = 0.0013$

- Compute the strain in the mild compression steel at ultimate [in/in]

$$\varepsilon s' := \begin{array}{|l} 0 \text{ if } As' = 0 \\ \varepsilon c\cdot\dfrac{c - d'}{c} \text{ otherwise} \end{array}$$

$\varepsilon s' = 0$

- Compute the strain in the mild tension steel at ultimate [in/in]

$$\varepsilon s := \begin{array}{|l} 0 \text{ if } As = 0 \\ \varepsilon c\cdot\dfrac{d - c}{c} \text{ otherwise} \end{array}$$

$\varepsilon s = 0$

- Compute the strain in the prestressing steel at ultimate [in/in]

$$\varepsilon ps := \begin{array}{|l} 0 \text{ if } Ap = 0 \\ \varepsilon p1 + \Omega b\cdot\varepsilon c\cdot\dfrac{dp - c}{c} \text{ otherwise} \end{array}$$

$\varepsilon ps = 0.019$

- Compute the strain in the FRP at ultimate [in/in]

$$\varepsilon f := \begin{array}{|l} 0 \text{ if } Af = 0 \\ \varepsilon c\cdot\dfrac{h - c}{c} - \varepsilon bi \text{ otherwise} \end{array}$$

Note: Based on M_{ip}, the initial strain in the substrate was computed to be:

$\varepsilon bi = -0.000088$

$\varepsilon f = 0.013$

- Compute the stress in the mild compression steel at ultimate for elastic/perfectly plastic behavior [psi]

$$fs' := \begin{cases} fy & \text{if } \varepsilon s' > \varepsilon sy \\ (-fy) & \text{if } \varepsilon s' < -\varepsilon sy \\ Es \cdot \varepsilon s' & \text{otherwise} \end{cases}$$

$fs' = 0$

- Compute the stress in the mild tension steel at ultimate for elastic/perfectly plastic behavior [psi]

$$fs := \begin{cases} fy & \text{if } \varepsilon s > \varepsilon sy \\ (-fy) & \text{if } \varepsilon s < -\varepsilon sy \\ Es \cdot \varepsilon s & \text{otherwise} \end{cases}$$

$fs = 0$

- Compute the stress in the prestressing steel at ultimate per PCI Design Aid 11.2.5 [psi]

$$fps := \begin{cases} \varepsilon ps \cdot Ep & \text{if } \varepsilon ps < 0.008 \\ (fpu - 2000) - \dfrac{\text{if}(fpu = 270000, 75, 58)}{\varepsilon ps - \text{if}(fpu = 270000, 0.0065, 0.006)} & \text{otherwise} \end{cases}$$

$fps = 261855$

- Compute the stress in the FRP at ultimate by Hooke's Law [psi]

$ff := Ef \cdot \varepsilon f$

$ff = 432025$

- Todeschini's equation defining the nonlinear compressive stress distribution in the concrete:

$$fc(y) := \frac{1.8 \cdot f'c \cdot \left(\dfrac{\varepsilon c \cdot y}{\varepsilon' c \cdot c}\right)}{1 + \left(\dfrac{\varepsilon c \cdot y}{\varepsilon' c \cdot c}\right)^2}$$

- Find the resultant compressive force from the compressive stress distribution in the concrete [lbs]

$$Cc := \int_0^c fc(y) \cdot be\, dy - \text{if}(bft = 0, 0, 1) \cdot \text{if}(c < tft, 0, 1) \cdot \int_0^{c-tft} fc(y) \cdot (be - bw)\, dy$$

$Cc = 1045026$

- Check internal force equilibrium by summing the internal force resultants. Revise "c" if the sum does not equal zero.

$\Sigma F := Cc + As' \cdot fs' - As \cdot fs - Ap \cdot fps - Af \cdot ff$

$\Sigma F = 0$ O.K.

- Locate the centroid of the compressive stress distribution in the concrete [in]

$$\beta c := 2 \cdot \left[c - \frac{\displaystyle\int_0^c fc(y) \cdot be \cdot y\, dy - \text{if}(bft = 0, 0, 1) \cdot \text{if}(c < tft, 0, 1) \cdot \int_0^{c-tft} fc(y) \cdot (be - bw) \cdot y\, dy}{Cc} \right]$$

$\beta c = 3.929$

- Additional reduction factor applied to the FRP contribution

  $\psi f := 0.85$

- Compute the strength reduction factor based on ductility per ACI 318-95 Section B.9.3.2.

$$\phi := \begin{cases} 0.90 \ \text{if} \ Ap \neq 0 \\ \text{otherwise} \\ \quad \begin{cases} 0.9 \ \text{if} \ \varepsilon s > 0.005 \\ 0.7 + \dfrac{(\varepsilon s - \varepsilon sy)}{0.025 - 5 \cdot \varepsilon sy} \ \text{if} \ \varepsilon sy \leq \varepsilon s \leq 0.005 \\ 0.7 \ \text{if} \ \varepsilon s < \varepsilon sy \end{cases} \end{cases}$$

$\phi = 0.9$

- Compute the design moment capacity [k-ft]

$$\phi Mn := \frac{\phi \cdot \left[As' \cdot fs' \cdot \left(\frac{\beta c}{2} - d' \right) + As \cdot fs \cdot \left(d - \frac{\beta c}{2} \right) + Ap \cdot fps \cdot \left(dp - \frac{\beta c}{2} \right) + \psi f \cdot Af \cdot ff \cdot \left(h - \frac{\beta c}{2} \right) \right]}{12000}$$

Design Ultimate Moment Capacity

$\phi Mn = 4406.8$ $\qquad$ $Mu = 4408$ $\qquad$ Design moment capacity [k-ft] vs moment demand [k-ft]