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16. Abstract
This project aims to demonstrate the feasibility of externally bonded FRP reinforcement for the flexural strengthening of a damaged prestressed concrete (PC) girder. The exterior PC girder of Bridge A10062, located at the interchange of Interstates 44 and 270 in St. Louis County, Missouri, USA, was impact-damaged by an overheight truck. Removal of the loose concrete showed that two prestressing tendons were fractured due to the impact. This resulted in approximately 10% reduction in flexural moment capacity. In this case study, it was decided to use carbon FRP laminates to restore the original structural capacity of the girder. It was demonstrated CFRP bonded reinforcement could be an effective repair technique in terms of installation as well as design.

The demonstration consists of design and field construction. The project leads to a bridge strengthening protocol for consideration by MoDOT.

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STRENGTHENING OF AN IMPACTED PC GIRDER
ON BRIDGE A10062,
ST LOUIS COUNTY, MO

PREPARED FOR THE
MISSOURI DEPARTMENT OF TRANSPORTATION

IN COOPERATION WITH THE
UNIVERSITY TRANSPORTATION CENTER AT UMR

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EXECUTIVE SUMMARY

The exterior prestressed concrete (PC) girder of Bridge A10062, located at the interchange of Interstates 44 and 270 in St. Louis County, Missouri, USA, was impact-damaged by an overheight truck. Removal of the loose concrete showed that two prestressing tendons were fractured due to the impact. This resulted in approximately 10% reduction in flexural moment capacity. There has been limited research on the repair of PC bridge girders damaged by vehicular impact. Due to the repetitive nature of highway loading, repair methods such as internal strand splices and external post-tensioning were found to be questionable because they could not restore the ultimate strength to the damaged member. In this case study, it was decided to use carbon FRP (CFRP) laminates to restore the original structural capacity of the girder. It was demonstrated that CFRP bonded reinforcement could be an effective repair technique in terms of installation as well as design. If the present trend in growing availability of FRP materials and design information were to continue, a sharp increase in FRP application could be forecast.

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1. INTRODUCTION

1.1. BACKGROUND

There has been relatively limited research on the damage assessment and repair of prestressed concrete (PC) bridge girders subjected to vehicular impact. From a National Cooperative Highway Research Program (NCHRP) perspective, two publications (Shanafelt et al. 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 experience in the United States, 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 et al. 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 my 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). In this case study, it was decided to use carbon FRP (CFRP) laminates installed by manual lay-up to restore the original structural capacity of the girder.

1.2. FRP COMPOSITES

FRP material systems, composed of fibers embedded in a polymeric matrix, exhibit several properties, which make them suitable for their use as structural reinforcing elements (Nanni et al. 1993). FRP composites are characterized by excellent tensile strength in the direction of the fibers and by negligible strength in the direction transverse to the fibers. This illustrates the anisotropic nature of these materials. FRP composites do not exhibit yielding, but instead are elastic up to failure. They are also characterized by a range of low to high modulus of elasticity in tension and low compressive properties. FRP composites are corrosion resistant and are expected to perform better than other construction materials in terms of weathering behavior.

The FRP matrix consists of a polymer, or resin, used as a binder for the reinforcing fibers. The matrix has two main functions: to enable the load to be transferred among fibers and, to protect the fibers from environmental effects. In a composite material, the fibers have the role of the load-bearing constituent. Fibers give the composite high tensile strength and rigidity along their longitudinal direction. Several types of fibers have been developed for use in FRP composites. For structural applications, research and development has been conducted using carbon, aramid and glass fibers. In the order listed, these fibers exhibit an ultimate strain range of 1 to 4%, with no yielding occurring prior to failure. The ultimate strength range is approximately 830 to 480 ksi (5,700 to 3,300 MPa) and elastic moduli range from 10,000 to 39,000 ksi (70 to 270 GPa). Carbon fibers are the strongest, stiffest, and most durable.

FRP composites are used in the construction industry in various forms and systems:

- Sheets of fibers are thin, flexible fabric-like materials. The sheets can either be dry and have the resin applied to them in place, or pre-impregnated "prepreg" with uncured B-stage resin, which requires special storage and handling.
- Laminates are formed from sheets by stacking one or more layers of the sheet and resin to consolidate them into the desired thickness. By adjusting the orientation and stacking sequence of the layers, a variety of physical properties can be achieved.
- Unidirectional sheets having fibers that are all aligned in a common direction.
- Multidirectional sheets are similar to unidirectional sheets except that fibers running in multiple directions are woven together. The fibers used can be of a variety of materials (carbon and aramid, for example) to create hybrid FRP laminates.

As a point of reference, the thickness of an installed ply (which includes fibers and adhesive) is in range of 0.039 to 0.118 in (1 to 3 mm). The process followed for the field installation of externally bonded FRP reinforcement consists of the basic following steps: concrete surface preparation (e.g., cleaning, crack sealing, rust-proofing existing steel reinforcement, smoothing, etc.), primer coat application, resin (undercoat) application, adhesion of the sheet(s), resin application, curing, and finish coat application.

Other cured systems include FRP grids (2D and 3D) and FRP reinforcing bars for concrete. High-strength FRP rods can be used for prestressing concrete (either in new construction or in external post-tensioning). Several tendon/anchor systems for concrete prestressing are available worldwide (Nanni 1993).

1.3. EXTERNALLY BONDED REPAIR

Structural retrofit work has come to the forefront of industry practice in response to the problem of aging infrastructure and buildings worldwide. This problem, coupled with revisions in structural codes to better withstand natural phenomena, creates the need for structural retrofit technologies. Some important characteristics of repair work are: labor cost, shut-down costs, material costs, scheduling constraints, long-term durability, difficulty in selection of repair method, and evaluation of effectiveness.

An effective method for upgrading RC members (prestressed and non-prestressed) is plate bonding. This method originates from the strengthening of steel beams by means of adding steel plates. It began in South Africa and France, where steel plates bonded with epoxy resins were used for strengthening of concrete members (L'Hermite et. al 1967), and was followed by more than 10 years of research until it became an accepted field practice. Experiments have investigated the influence of factors such as plate thickness, type of adhesive and anchoring conditions (Swamy et al. 1987). Roberts et al. (1989) published a theoretical study of the behavior of RC beams bonded with steel plates that has become a landmark paper. This study was aimed at developing a simple analytical model capable of predicting the effect of a steel plate on the distribution of strain and stress in the RC beam.

In Germany and Switzerland during the mid 1980's, replacement of steel with FRP plates began to be viewed as a promising improvement in externally bonded repair (Meier et al. 1987 and 1991, Rostasy et al. 1992). Kaiser (1989) load tested carbon FRP composites and showed the validity of the strain compatibility method (i.e., classical approach for RC sections) in the analysis of repaired members. In the United States, Ritchie et al. (1991) and Saadatmanesh et al. (1991) studied the static behavior of RC beams with externally bonded glass FRP plates and developed analytical methods also based on strain compatibility. Later, Triantafillou et al. (1992) added concepts of fracture mechanics to this classical method. Berset (1992) investigated the use of externally bonded composites to strengthen RC beams in shear. In Saudi Arabia, Sharif et al. (1994), using both Roberts' theory and strain compatibility, developed a theoretical algorithm for predicting the flexural strength and the plate separation load of repaired beams.

For bridge structures subjected to cyclic loading, fatigue becomes an important issue that needs to be addressed by the designer. The fatigue behavior of FRP as a stand-alone material has been under investigations for almost 40 years in the context of aerospace, marine and mechanical applications (Broutman, 1974). Over this period of time, fatigue data have been generated for a variety of composite materials under axial and flexural fatigue loading. More recently, research has been carried out on the fatigue behavior of FRP for infrastructure applications (Demers, 1998). In the past decade a remarkable amount of research has focused on the static behavior of RC beams strengthened with externally bonded FRP laminates. However, little has been done on the fatigue performance of RC beams strengthened with externally bonded FRP sheets. The available literature includes publications by Shahawy et al. (1998), Nishizaki et al. (1997) and Demers (1998).

Of all countries, Japan has seen the largest number of field applications using bonded FRP composites. Two large manufacturing industries (Tonen and Mitsubishi Chemical) have aggressively pursued this technology. A joint venture of Mitsubishi Chemical and Obayashi Corporation (one of the largest Japanese contracting companies) was the first partnership to propose and execute column and chimney repair by FRP wrapping. Japanese manufacturer's literature (Tonen 1994, Mitsubishi Chemical 1994) also proposes the adoption of the working stress design method based on the classical flexural theory. The primary assumption remains that of perfect bond between FRP and concrete (and between concrete and steel). Allowable stress for the FRP sheets is set at one-third of the ultimate tensile capacity. This means that the allowable strain in the FRP, even in the case of low-elongation fibers, is larger than five times the strain at yield of conventional Grade 60 steel.

The advantages of FRP versus steel for the reinforcement of concrete structures include lower installation costs, improved corrosion resistance, on-site flexibility of use, and small changes in member size after repair. An additional advantage in terms of industry acceptance is due to the fact that building code enforcement for repair-type application is not as stringent as for new construction. Widespread implementation in structural repair is ultimately contingent upon availability of codes and familiarity of owners, engineers, and contractors with the performance of the new materials and technology.

1.4. PREVIOUS APPLICATION IN DAMAGE REPAIR

Figure 1.1 illustrates the effect of a vehicular impact on the four girders of the bridge overpass on highway Appia near Terracina, Rome (Nanni 1997). This is a short bridge, 34.48 ft (10.5 m) in span, made of four prestressed concrete girders having cross sectional dimensions of 3.28 by 4.92 ft (1.0 by 1.5 m). The conventional reinforcement (prestressing tendons and reinforcing bars) is clearly visible in the photograph after the loose concrete was removed.

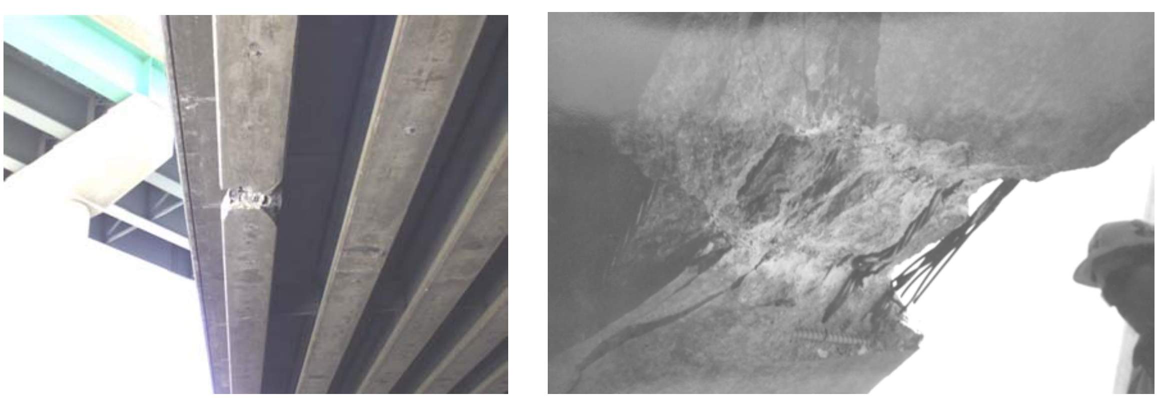
Figure 1.1. Girder Damage due to Vehicular Impact

The concrete cross section was restored with non-shrink mortar and, after surface preparation, CFRP sheets were adhered as shown in Figure 1.2. The objective of the CFRP strengthening was to make up for the loss of prestress. For each beam, three sheets, 1.08 ft (0.33 m) wide and 9.84 ft (3.0 m) long, were bonded to the soffit (0º fiber direction), and four strips, 0.52 ft (0.16 m) wide and 9.84 ft (3.0 m) long, were wrapped around the three sides (90º fiber direction). The total amount of CFRP material used was approximately 215.27 ft^2 (20 m^2).

Figure 1.2. Pattern of CFRP Strips

1.5. OBJECTIVE

Bridge A10062 is located at the interchange of the Interstates 44 and 270 in St. Louis County, Missouri, USA. Elevation and plan details are shown in Appendix A. The bridge has a relatively low roadway clearance of 14.ft-8 in. (4.47 m), it was impact-damaged by an overheight truck in one of its exterior PC girders. Removal of the loose concrete showed that two prestressing tendons were fractured due to the impact (see Figure 1.3).



(a) Overall View of the Damage (b) Fractured Prestressing Tendons

Figure 1.3. Fractured Tendons after the Removal of the Damaged Concrete

2. MATERIAL PROPERTIES

2.1. PC GIRDER AND DECK

The damaged girder was prestressed by 20 low-relaxation 7-wire steel strands with a tensile strength of 270ksi (1862 Mpa). It was assumed that a portion of the bridge deck with dimensions of 8.5 x 48.8 in. (21.6 x 122 cm) provided composite action with the girder. The cross section of the damaged girder and prestressing details are shown in Fig. 2.1. Material properties used in the analysis are shown in Table 2.1.

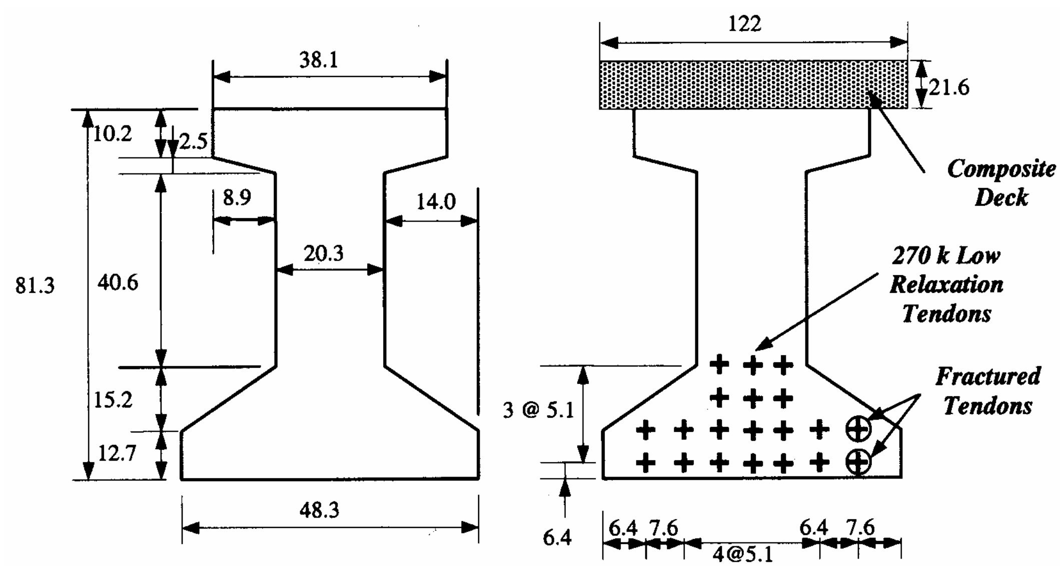


Figure 2.1. Girder Dimensions and Prestressing Details (dimensions in cm / 1in.=2.54 cm)

Table 2.1. Material Properties

Prestressing Tendons	Strand Type	Low Relaxation
	Strand Tensile Strength, ksi (MPa)	270 (1,862)
	Nominal Diameter, in. (mm)	0.5 (12.7)
	Strand Area, in^2 (mm^2)	0.15 (98.71)
	Modulus of Elasticity, ksi (GPa)	2,800 (19.31)
Concrete	Existing Concrete Deck, psi (MPa)	5,000 (34.5)
	PC Girder, psi (MPa)	8,000 (56.0)

2.2. FRP LAMINATE

A commercially available FRP strengthening system was selected for its high strength and excellent performance under sustained and cyclic loading. Primer, putty, CFRP sheets, and impregnating resin (i.e., saturant) were provided by Master Builders Technologies of Cleveland, OH (MBT 1998). In this system, carbon fibers are initially dry, unidirectionally oriented, and supported by a paper backing for ease of installation by manual lay-up. According to manufacturer's literature, the FRP tensile strength is 620 ksi (4,275 MPa), the modulus of elasticity is 33,000 ksi (4.8 GPa), and the design thickness is 0.0065 in. (0.165 mm). Note that, tensile strength and elastic modulus of the saturant is neglected in computing the strength of the system. Therefore, FRP laminate properties are calculated and reported (see Table 2.2) using the net fiber area. In tension, the CFRP laminate has a linear elastic behavior up to failure.

Table 2.2. Properties of Carbon FRP (MBT 1998)

Ultimate Strength, ksi (MPa), f_{pu}	620 (4,275)
Design Strength, ksi (MPa), f_{fe}	550 (3,792)
Tensile Modulus, ksi (GPa), E_f	33000 (4.8)
Thickness, in. (mm), t_f	0.0065 (0.165)
Ultimate Strain, %, ε_{fu}	1.7

3. CFRP DESIGN CALCULATIONS

3.1. PRELIMINARY DESIGN

The ultimate limit state analysis calculates the capacity of the section by combining force equilibrium, strain compatibility, and the constitutive laws for the case of an RC section of the materials at failure. As an example the stress and strain distributions at ultimate are shown in Figure 3.1. The non-linear stress strain behavior of concrete may be replaced for computational ease by a rectangular stress block with dimensions $\gamma f'_c$ by $\beta_1 c$.

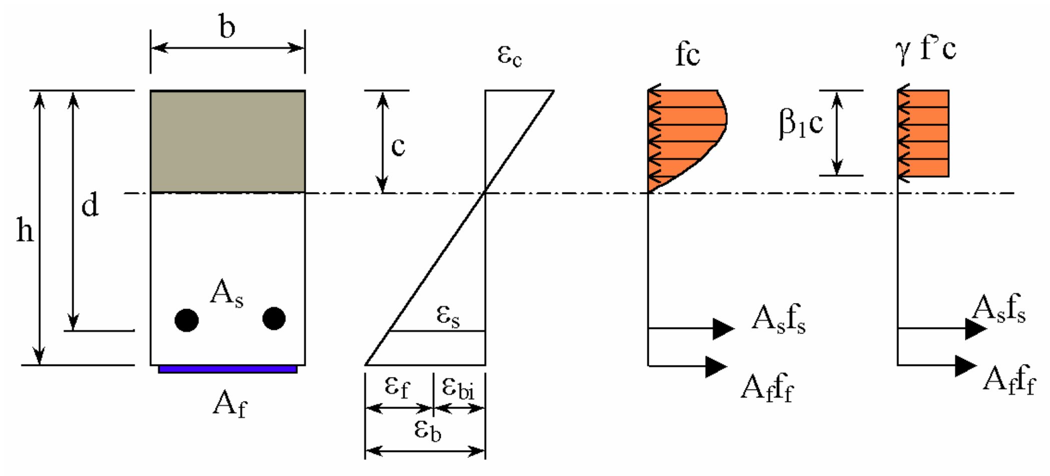


Figure 3.1. Strain and Stress Distribution in a RC Section at Ultimate

It should be noted that the Whitney stress block employed by American Concrete Institute (ACI) - Committee 318 is not valid when the concrete strain falls below 0.003 in/in (mm/mm). In this instance, the two most common representations of the stress-strain curve of concrete are the Modified Hognestad and Todeschini approximations. The Todeschini approximation (Todeschini et al. 1964) is the easiest to use and is readily adaptable to computer applications (Mac Gregor 1997).

The general equation for the nominal moment capacity of a RC section strengthened with FRP flexural reinforcement is given in Equation 3.1.

$$M_n = A_s f_s\left(d - \frac{\beta_1 c}{2}\right) + A_f f_f\left(h - \frac{\beta_1 c}{2}\right) \tag{3.1}$$

The term f_s indicates that the reinforcing steel is not necessarily at its yield stress. Addition of FRP to the beam may result in over-reinforcement for moment capacity thus the concrete may crush before the steel yields. There is discussion within the technical community and in particular within Committee 440 of the ACI to arrive to a scientifically based expression of the reduction factor to be applied to the ultimate strength of FRP. The current thinking is that the material properties reported by manufacturers should be considered as initial properties that do not consider long-term exposure to environmental conditions. Because long-term exposure to various types of environments can reduce the tensile properties and creep rupture and fatigue endurance of FRP laminates, the material properties used in design equations should be reduced based on the environmental exposure condition. The modulus of elasticity is unaffected by environmental conditions.

The stresses in each of the materials will depend on the strain distribution and the governing failure mode. Because of the number of variables involved, there is no direct procedure for determining the strain distribution and failure mode. Instead, a trial and error procedure is necessary. This procedure involves first estimating the depth to the neutral axis, c, and determining the failure mode based on this estimate. The estimated depth to the neutral axis may be confirmed or modified based on strain compatibility, the constitutive laws of the materials, and internal force equilibrium. In most situations, a first estimate of c = 0.15d is reasonable.

3.2. DESIGN CRITERIA

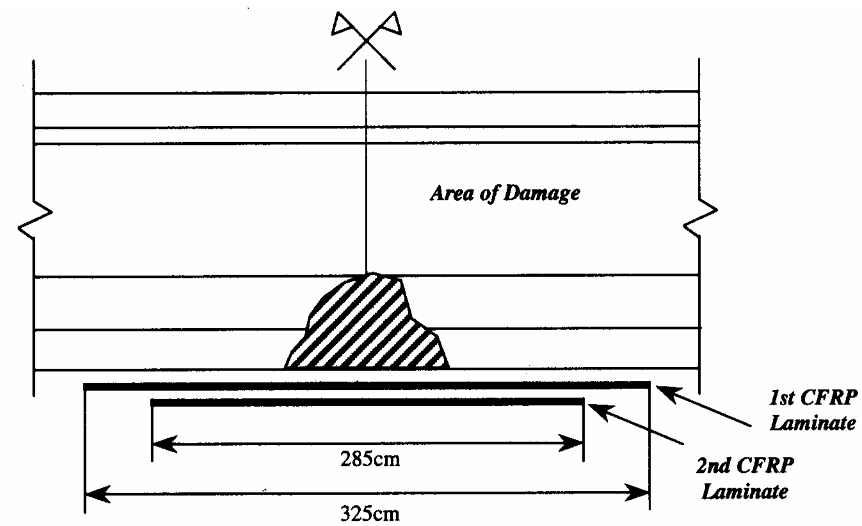
The nominal moment capacity of the PC girder plus concrete deck was determined by the conventional rectangular stress block approach. The stress in the tendons at ultimate was determined according to standard equations (PCI, 1999). The moment capacities were calculated using a computer program written by Masters Builders Technologies in collaboration with UMR. Since the program requires the use of a cross section with sides perpendicular to each other, the actual cross section was modified as shown in Appendix B. The computed factored moment capacity before damage was $\phi M_{n(original)}$ = 2096 ft-k (2,840 kN-m). As a result of the impact-damage, the capacity of the member was reduced to $\phi M_{n(damaged)}$ = 1,896 ft-k (2,569 kN-m). Thus, strengthening had to restore a loss of about 200 ft-k (271 kN-m) of moment capacity.

The parameters that affect the design of the strengthening of concrete flexural members have been investigated and used for many applications (Nanni et al. 1998). Included in the design protocol are the effects of initial strain, FRP/steel reinforcement ratios, material properties, steel reinforcing stress at working loads, deflections under working loads, and failure mechanisms. Pseudo-ductility can also addressed by considering the failure mechanism and the strain in the steel reinforcement at ultimate. The input requirements for the design of an FRP strengthened and/or stiffened concrete flexural member include existing concrete section, imposed loads (at installation and service), global geometry, and material properties. It is further assumed that the FRP laminate is externally bonded to the concrete surface when the concrete surface itself is subjected to a given level of strain and that perfect bond exists between FRP and concrete. The fundamental steps of the adopted design procedure are listed below:

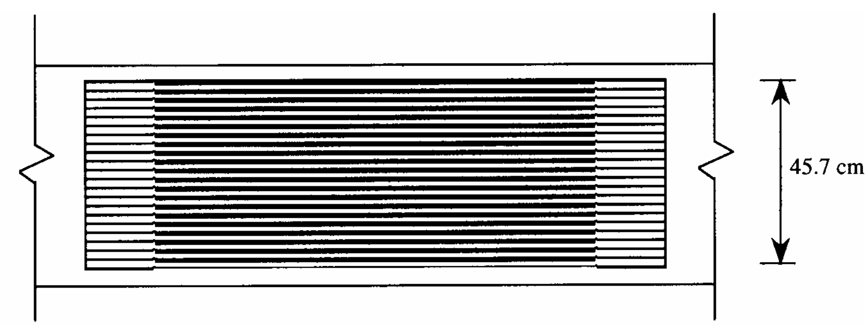
- Calculate critical section moment and curvature at yield of reinforcing steel
- Calculate tensile strain in existing member at the level where FRP is to be applied
- Calculate the area of FRP required to resist ultimate projected moment
- Check stress/strain at working loads
- Determine overall length of the FRP plies and laminate
- Check ductility of the system

- Check deflection under transitory loads

The rehabilitation of this impact–damaged girder called for concrete repair and application of CFRP laminates. The flexural strengthening consisted of two 18 in. (45.7cm) wide plies with lengths of 9 ft-4 in. and 10 ft-8 in. (285 and 325 cm), respectively, applied to the bottom of the girder with fibers aligned along its longitudinal axis. The double ply-laminate was centered over the damaged area (see Figure 3.2). Sixteen strips, 4 in. (10.2 cm) wide and spaced at 8 in. (20.4 cm) on centers, were then U-wrapped around the bulb of the girder over the previous installation (see Figure 3.3). The purpose of the U-wrap is to prevent the delamination of the FRP plies applied to the bottom surface of the girder. After repair, the factored capacity of the girder was computed to be $\phi M_{n(repaired)}$ = 2238.4 ft-k (3,033 kN-m), which is 7 % larger than the original capacity. Appendix B shows the supporting calculations for the flexural capacity of the original, damaged and repaired sections. The calculations are carried out in U.S customary units.

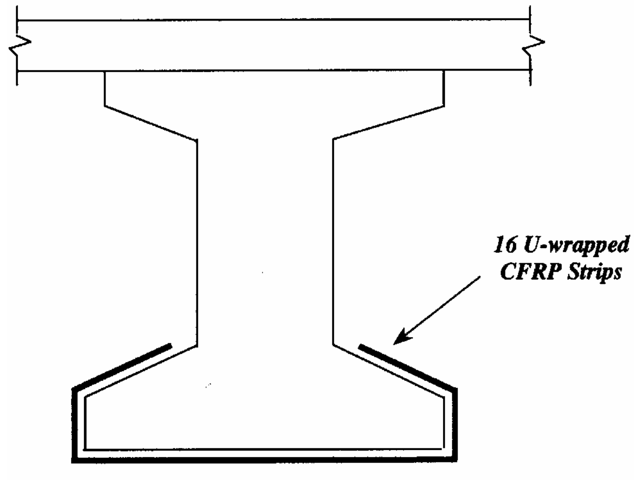


(a) Elevation View

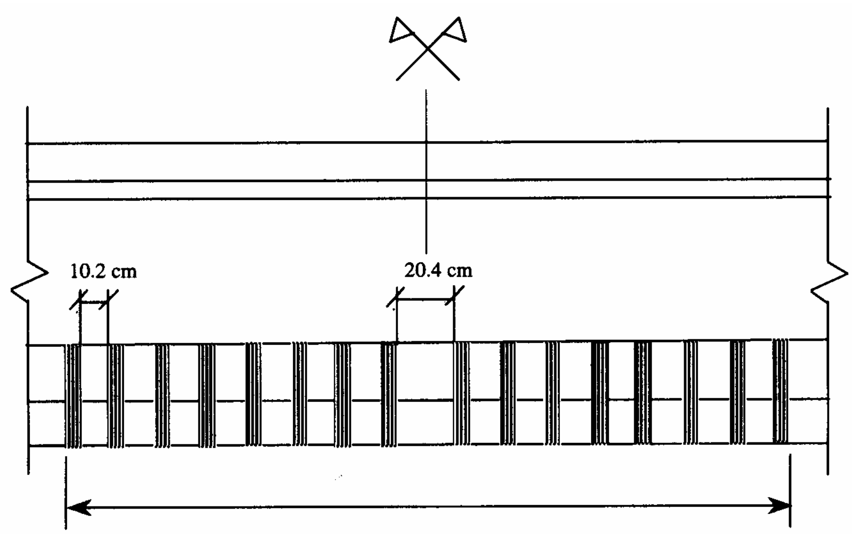


(b) Plan View

Figure 3.2. Two-Ply CFRP Laminate (1 in.=2.54cm)



(a) Elevation View



(b) Plan View

Figure 3.3. CFRP Strips U-Wrapped around the Girder Bulb (1 in.=2.54 cm)

4. INSTALLATION

Before carrying out the CFRP laminate installation, the damaged area of the girder was restored with a rapid setting, no-shrinkage, cementitious mortar. The sequential installation procedure was as follows:

Surface Preparation**:** the bottom edges of the girder were rounded for proper wrapping. Next, the concrete surface was sandblasted until the aggregate was exposed and the surface of the concrete was free of loose and unsound materials.

Application of primer**:** a layer of epoxy-based primer was applied to the prepared concrete surface using a short nap roller to penetrate the concrete pores and to provide an improved substrate for the saturant.

Application of putty**:** after the primer became tack-free, a thin layer of putty was applied using a trowel to level the concrete surface and to patch small holes.

Application of first layer of saturant: the first layer of saturant was rolled on the putty using a medium nap roller. The functions of the saturant are: to impregnate the dry fibers, to maintain the fibers in their intended orientation, to distribute stress to the fibers, and to protect the fibers from abrasion and environmental effects.

Application of fiber sheet: after the fiber sheet was measured and pre-cut, it was placed on the concrete surface and gently pressed into the saturant. Prior to removing the backing paper, a trowel was used to remove any air void. After the backing paper was removed, a ribbed roller was rolled in the fiber direction to facilitate impregnation by separating the fibers.

Application of second layer of saturant**:** a second layer of saturant was applied and worked into the fibers with a ribbed roller. After this, the second fiber sheet could be installed by repeating the described procedure.

Figure 4.1 illustrates the Missouri DOT crew at work. The installation of the FRP systems, including surface preparation, was performed in two hours.

(a) Overall View

(b) Surface Sandblasting

(c) Application of Putty

(d) Application of Saturant

(e) Manual Lay-up of CFRP Sheet

(f) Strips Manual Lay-up

Figure 4.1. Installation Crew at Work

5. DEFECT DETECTION AND REPAIR

5.1. DEFECT DETECTION

A routine inspection detected a blister in the repaired area. The blister dimensions were 33-in. (83 cm) long, 9.5-in. (24 cm) wide, and 0.75-in (1.9 cm) high. Its formation was caused by the run-off of excess saturant (see Figure 5.1). Excessive saturant was applied and cross-polymerization did not increase viscosity rapidly enough to prevent flow. No excess saturant should be present on the concrete surface after placement of a fiber sheet. It should be noted that the temperature and weather prior and after installation were within the acceptable ranges. The manufacturer recommends a temperature at installation of 40 ^{o}F (4.4 ^{o}C), and that the concrete surface was dry.

It was recommended to epoxy-inject the bubble at the time of painting of the FRP repair. UMR provided required material and assisted a MoDOT maintenance crew in this effort.

(a) Excess of Saturant (b) Blister in the repaired area

Figure 5.1. Installation Defect

5.3. DEFECT REPAIR

One lane of traffic on highway I-44 at the intersection of highway I-270 was closed for the injection of blister. A crew from MoDOT first cleaned the surface of FRP laminate where the blister was located. After marking the first injection point at the center of blister and 9 outlets around its edge (see Figure 5.4), the drilling of marked points was executed. No water and obstruction were found in the blister.

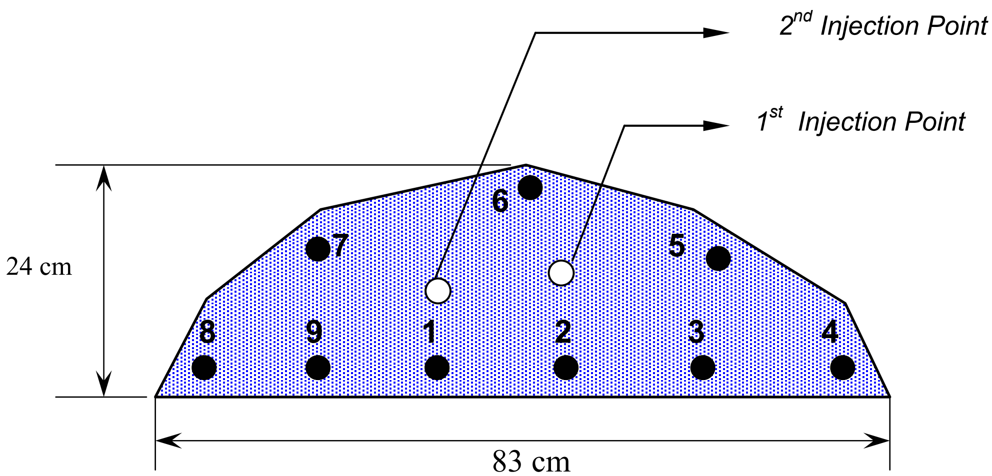


Figure 5.2. Outlet Layout (1 in. =2.54 cm)

Through the first injection point a general purpose gel epoxy adhesive was used to inject the blister. Once the epoxy started to flow through outlets 1 to 6, these were taped. To complete the blister injection, a second injection point was then determined and drilled, the epoxy in this case flew through outlets 7, 8, and 9. All holes were taped when the injection was completed. Then the surface of entire patch where FRP reinforcement was applied was cleaned up and tapes on the blister were removed when epoxy adhesive was set. The entire patch was then painted with two coats of a polymer-based coating. After the protective coating was set, three red lines were marked on the top coat at the edge of outlets 5, 6, and 5 as the reference of any further blister growth. Figure 5.3 illustrates the aforementioned process.

(a) Injection of Blister

(b) Coating

Figure 5.3. Repair of the Installation Defect

6. CONCLUSIONS

Traditional techniques used to repair concrete structures may be expensive, time consuming, and of limited effectiveness. Due to the inherent physico-mechanical properties of non-metallic composites and their ease of installation, it may be possible to develop new repair methods that are externally adhered to the concrete member. This paper describes a case study where an impact-damaged PC girder was upgraded using FRP laminates installed by manual lay-up. Although, the described strengthening technique offers an efficient option for the repair/retrofit of bridge girders, its successful and widespread implementation will ultimately depend on the engineers' materials and structural knowledge as well as contractors' quality installation. The widespread use of FRP laminates as external reinforcement for concrete also depends on the availability of national or international standards for design, testing, and inspection.

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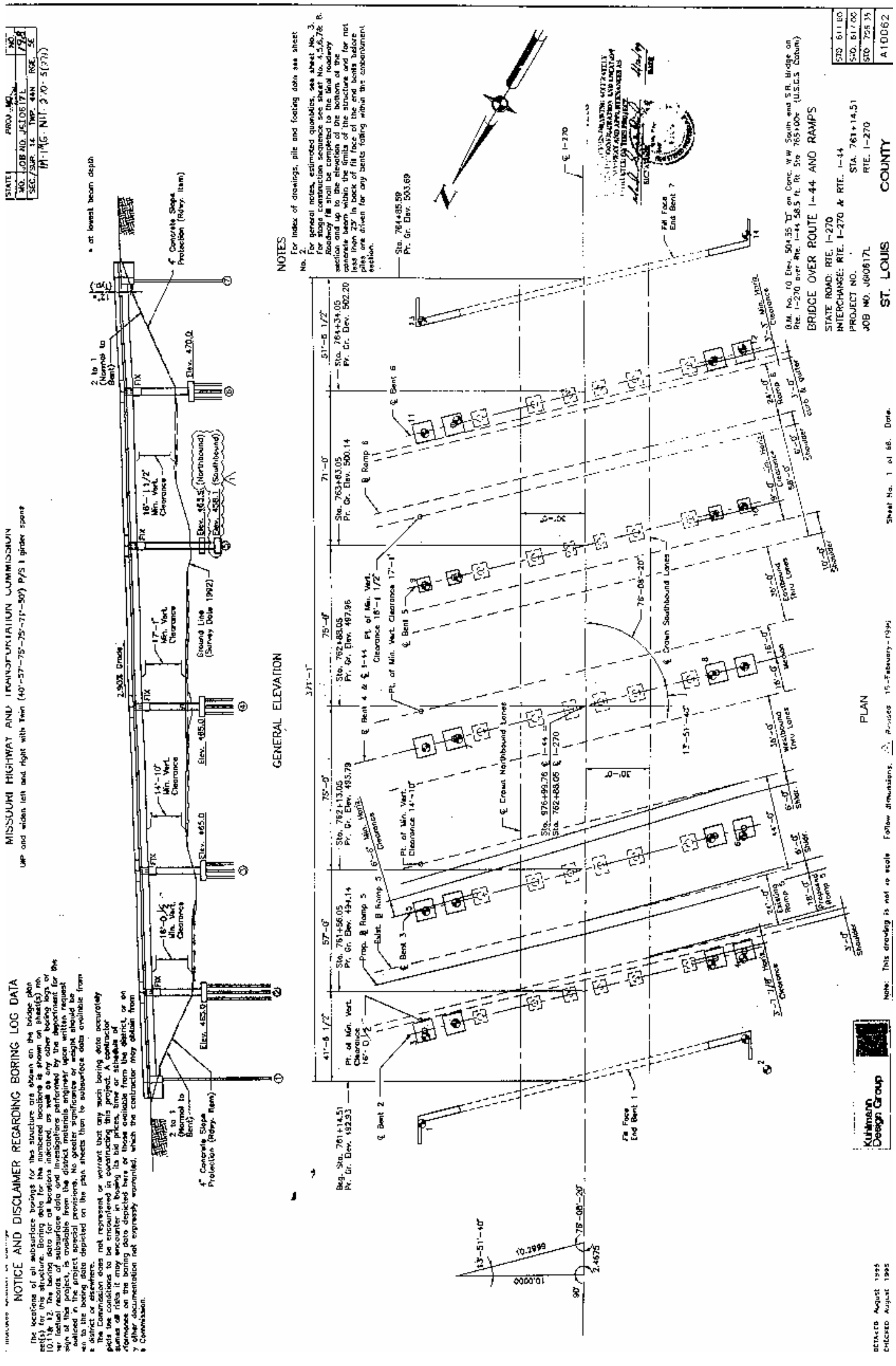
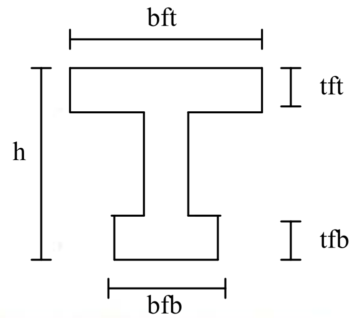
APPENDIX A: BRIDGE DRAWINGS



APPENDIX B: SUPPORTING CALCULATIONS



MBrace™ Flexural Strengthening Design

Project: Project

Condition: Condition

Designed by: Engineer
Company

Date: Date

Required Information about the Existing Structure

Section Dimensions

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

Reinforcement Layout

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

Load and Span Information

Mu := ▮	Factored moment to be resisted by the strengthened element [k-ft]
Ms := ▮	Service moment to be resisted by the strengthened element [k-ft]
Mip := ▮	Moment in place at the time of MBrace installation [k-ft]
ln := ▮	Clear span [ft]
lr := ▮	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:= ▮	Nominal compressive strength of the concrete [psi]
fy := ▮	Yield strength of the mild steel [ksi]
fpu := ▮	Ultimate strength of the prestressing steel [ksi]
fpy := ▮	Yield strength of the prestressing steel [ksi]
Ep := ▮	Modulus of elasticity of the prestressing steel [ksi]

Required MBrace Design Information

MBrace Material Selection

Fiber := ■ MBrace Fiber Reinforcement

1 -- MBrace CF 130 High Strength Carbon Fiber
2 -- MBrace AK 60 High Performance Aramid Fiber
3 -- MBrace EG 900 E-Glass Fiber (*not recommended for flexural strengthening*)

$ffu^* =$ ■	Ultimate tensile strength of the FRP [psi]
$\varepsilon fu^* =$ ■	Ultimate rupture strain of the FRP [in/in]
Ef = ■	Tensile modulus of elasticity of the FRP [psi]
tf = ■	Nominal design thickness of one ply of the FRP [in/ply]

Durability Reduction Factors

Ccr := ■	Creep rupture stress limit (Use 0.55 for Carbon, 0.30 for Aramid, and 0.20 for E-Glass)
Ce := ■	Reduction factor for environmental exposure

Layout of the MBrace Reinforcement

wf := ■	Width of the MBrace strip [in]
n := ■	Number of MBrace plies

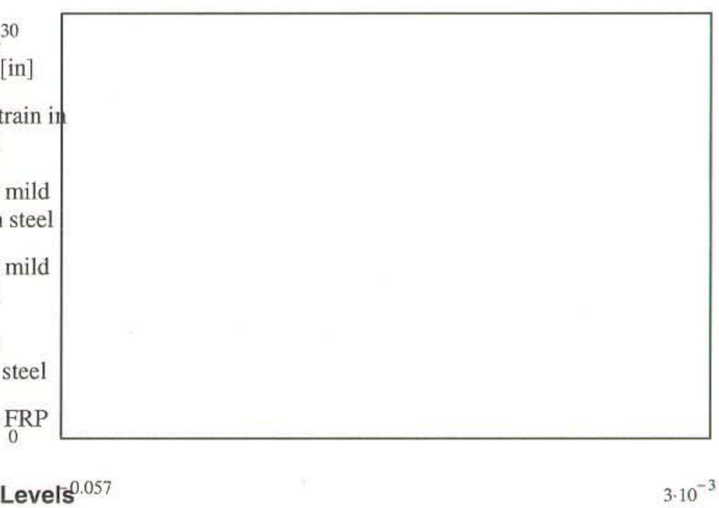
Results of the Flexural Strengthening Analysis

Design Ultimate Moment Capacity

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

Strain Distribution at Ultimate

cu = ■	Depth to the neutral axis [in]
εcu = ■	Maximum strain in the concrete
$\varepsilon s'(\varepsilon cu, cu) =$ ■	Strain in the mild compression steel
$\varepsilon s(\varepsilon cu, cu) =$ ■	Strain in the mild tension steel
$\varepsilon ps(\varepsilon cu, cu) =$ ■	Strain in the prestressing steel
$\varepsilon f(\varepsilon cu, cu) =$ ■	Strain in the FRP



Check of Stresses at Service Load Levels

fss = ■	Fss = ■	Mild tension steel stress at service [psi] vs service stress limit [psi]
ffs = ■	Ffs = ■	MBrace service stress [psi] vs creep rupture stress limit [psi]

Detailed Calculation of the Design Moment Capacity

Computation of Gross Section Properties

- Effective width of concrete in compression [in]

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

  $be = ▮$

- Cross sectional area [in^2]

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

  $Ac = ▮$

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

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

  $ct = ▮$

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

  $cb := h - ct$

  $cb = ▮$

- Gross moment of inertia [in^4]

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

  $Ig = ▮$

- Radius of gyration [in]

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

  $r = ▮$

Computation of Material Characteristics

- Modulus of elasticity for concrete [psi]

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

  $Ec = ▮$

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

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

  $\varepsilon'c = ▮$

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

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

  $\varepsilon sy = ▮$

Preliminary computations for FRP properties

- Design ultimate tensile strength [psi]

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

  $ffu = \blacksquare$

- Design rupture strain [in/in]

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

  $\varepsilon fu = \blacksquare$

- Bond dependent coefficient for flexure

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

  $\kappa m = \blacksquare$

Preliminary computations for prestressing steel properties

- Prestressing force [lbs]

  $Pe := Ap \cdot fpe$

  $Pe = \blacksquare$

- Eccentricity of prestressing force [in]

  $e := dp - ct$

  $e = 2.718$

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

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

  $\varepsilon p1 = \blacksquare$

- Bond reduction coefficient applied to unbonded tendons

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

  $\Omega b = \blacksquare$

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

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

  $c = \blacksquare$

- 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]

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

  $\varepsilon c2 = \blacksquare$

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

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

  $\varepsilon c3 = \blacksquare$

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

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

  $\varepsilon c = \blacksquare$

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

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

  $\varepsilon s' = \blacksquare$

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

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

  $\varepsilon s = \blacksquare$

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

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

$\varepsilon ps = ▮$

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

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

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

$\varepsilon bi = ▮$

$\varepsilon f = ▮$

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

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

$fs' = ▮$

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

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

$fs = ▮$

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

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

$fps = ▮$

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

$ff := Ef \cdot \varepsilon f$

$ff = ▮$

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

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

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

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

$Cc = ▮$

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

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

$\Sigma F = ▮$ O.K.

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

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

$\beta c = ▮$

- 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.

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

$\phi = ▮$

- Compute the design moment capacity [k-ft]

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

$\phi Mn = ▮$

Strengthening of Bridge A10062

MBrace™ Flexural Strengthening Design

Project: Bridge A10062

Condition:

Designed by: UMR

Date: December 1999

Required Information about the Existing Structure

Section Dimensions

$h := 40.5$	Total section height [in]
$bw := 8$	Width of web [in]
$bft := 48$	Width of top flange (zero for rectangular or inverted tee sections) [in]
$tft := 9.9$	Thickness of top flange (zero for rectangular or inverted tee sections) [in]
$bfb := 19$	Width of bottom flange (zero for rectangular or tee sections) [in]
$tfb := 8$	Thickness of bottom flange (zero for rectangular or tee sections) [in]

Reinforcement Layout

$As := 0$	Area of mild tension steel [in^2]
$d := 0$	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]
$Ap := 2.754$	Area of prestressing steel [in^2]
$dp := 36.3$	Depth to the prestressing steel centroid [in]
$fpe := 150$	Effective stress in the steel due to prestress [ksi]
$Bond := 1$	Type of tendon installation (Enter "1" for bonded, "0" for unbonded)

Material Property Specifications

$f'c := 5000$	Nominal compressive strength of the concrete [psi]
$fy := 0$	Yield strength of the mild steel [ksi]
$fpu := 270$	Ultimate strength of the prestressing steel [ksi]
$fpy := 250$	Yield strength of the prestressing steel [ksi]
$Ep := 28500$	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 AK 60 High Performance Aramid Fiber

3 -- MBrace EG 900 E-Glass Fiber (*not recommended for flexural strengthening*)

Flexural Capacity: Original Section

$wf := 0$	The total width of FRP
$c := \frac{ct}{2}$	Trial value of the neutral axis location

Horizontal Equilibrium to find the value of c:

Given $\quad Cc(wf,c) - Ap \cdot fps(wf,c) - As \cdot fs(wf,c) + As' \cdot fs'(wf,c) - tf \cdot wf \cdot ff(c) = 0 \qquad c := Find(c)$

Rotational Equilibrium to find the resistive moment:

$$Mn1 := \left[Ap \cdot fps(wf,c) \cdot (dp - yc(wf,c)) + As \cdot fs(wf,c) \cdot (ds - yc(wf,c)) \right] + As' \cdot fs'(wf,c) (yc(wf,c) - ds')$$

$$Mn := Mn1 + tf \cdot wf \cdot ff(c) \cdot (h - yc(wf,c))$$

Design Moment Capacity:

$$\phi Mn := \frac{0.9 \cdot Mn}{12000} \qquad \phi Mn = 2094.8 \text{ k-ft}$$

==================

Other quantities of interest:

$c = 4.851$	Actual depth to the neutral axis
$\varepsilon c(wf,c) = 3 \times 10^{-3}$	Maximum compressive strain level in the concrete at ultimate
$\varepsilon f(c) \cdot \frac{wf}{wf} = 0$	Strain level in the FRP at ultimate
$FailureMechanism(wf,c) = 1$	The governing mode of failure. A FailureMechanism value of "1"corresponds to concrete crushing A FailureMechanism value of "2" corresponds to FRP rupture

Flexural Capacity: Damaged Section

$wf := 0$ The total width of FRP

$c := \frac{ct}{2}$ Trial value of the neutral axis location

Horizontal Equilibrium to find the value of c:

Given $Cc(wf,c) - Ap \cdot fps(wf,c) - As \cdot fs(wf,c) + As' \cdot fs'(wf,c) - tf \cdot wf \cdot ff(c) = 0$ $c := Find(c)$

Rotational Equilibrium to find the resistive moment:

$$Mn1 := [Ap \cdot fps(wf,c) \cdot (dp - yc(wf,c)) + As \cdot fs(wf,c) \cdot (ds - yc(wf,c))] + As' \cdot fs'(wf,c) \cdot (yc(wf,c) - ds')$$

$$Mn := Mn1 + tf \cdot wf \cdot ff(c) \cdot (h - yc(wf,c))$$

Design Moment Capacity:

$\phi Mn := \frac{0.9 \cdot Mn}{12000}$ $\phi Mn = 1896.2$ k-ft

==================

Other quantities of interest:

$c = 4.366$	Actual depth to the neutral axis
$\varepsilon c(wf,c) = 3 \times 10^{-3}$	Maximum compressive strain level in the concrete at ultimate
$\varepsilon f(c) \cdot \frac{wf}{wf} = 0$	Strain level in the FRP at ultimate
$FailureMechanism(wf,c) = 1$	The governing mode of failure. A FailureMechanism value of "1"corresponds to concrete crushing A FailureMechanism value of "2" corresponds to FRP rupture

Flexural Capacity: Repaired Section

$wf := 36$ The total width of FRP

$c := \frac{ct}{2}$ Trial value of the neutral axis location

Horizontal Equilibrium to find the value of c:

Given $Cc(wf,c) - Ap \cdot fps(wf,c) - As \cdot fs(wf,c) + As' \cdot fs'(wf,c) - tf \cdot wf \cdot ff(c) = 0$ $c := Find(c)$

Rotational Equilibrium to find the resistive moment:

$$Mn1 := [Ap \cdot fps(wf,c) \cdot (dp - yc(wf,c)) + As \cdot fs(wf,c) \cdot (ds - yc(wf,c))] + As' \cdot fs'(wf,c) \cdot (yc(wf,c) - ds')$$

$$Mn := Mn1 + tf \cdot wf \cdot ff(c) \cdot (h - yc(wf,c))$$

Design Moment Capacity:

$\phi Mn := \frac{0.9 \cdot Mn}{12000}$ $\phi Mn = 2238.4$ k-ft

==================

Other quantities of interest:

$c = 5.347$ Actual depth to the neutral axis

$\varepsilon c(wf,c) = 2.573 \times 10^{-3}$ Maximum compressive strain level in the concrete at ultimate

$\varepsilon ps(wf,c) = 0$ Strain level in the tendons at ultimate

$\varepsilon f(c) \cdot \frac{wf}{wf} = 0.017$ Strain level in the FRP at ultimate

$FailureMechanism(wf,c) = 2$ The governing mode of failure.
A FailureMechanism value of "1"corresponds to concrete crushing
A FailureMechanism value of "2" corresponds to FRP rupture