Interim Report 930-466

DESIGN PROCEDURE FOR FRP STRENGTHENING OF WAR MEMORIAL BRIDGE

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Chapter 1: Introduction

1.1 Background

A recent inventory of the nation's infrastructure revealed that 27.6 percent of the bridges in the United States are structurally deficient or functionally obsolete (Jones Informational Media). The shortcomings of these bridges vary widely and may require the bridge to be load posted, repaired, or even replaced. The War Memorial Bridge in Macon County, Alabama, is just one example of a bridge that has been deemed structurally deficient.

The War Memorial Bridge crosses the Uphapee Creek just north of Tuskegee, Alabama, on Alabama Highway 81. Since the completion of this bridge in 1945, typical truck loads have increased to the point that they now exceed those for which it was designed. Overloading of the bridge causes excessive deflections, high stresses in the steel reinforcement, and may shorten the service life of the structure. In order to combat this problem, load postings were issued for two specific truck types.

In 1999, the Alabama Department of Transportation (ALDOT) proposed that the bridge be repaired so that the load posting could be removed. This repair involved increasing the positive moment capacity of the bridge so that the bridge could accommodate any truck of legal weight. Auburn University researchers suggested that the additional moment capacity be added by using fiber reinforced polymer (FRP) composites applied externally to the reinforced concrete girders.

ALDOT and Auburn University personnel performed bridge load tests and applied the external FRP composites to the bridge during the second half of 2001. Monitoring of the bridge's performance and behavior will continue and subsequent load tests will be performed at regular intervals. If successful, the addition of the FRP composites to the bridge girders will allow the load posting to be removed.

The use of FRP composites to strengthen reinforced concrete structures is a relatively new technology. There are still many questions regarding the behavior of FRP when bonded to concrete surfaces, so providing practical and conservative guidelines for the design of FRP strengthened structures is difficult. One method used to determine the amount of FRP necessary for strengthening is based on a draft report by Committee 440—Fiber Reinforced Polymer Reinforcement of the American Concrete Institute (ACI). The draft report gives design recommendations for reinforced concrete members externally reinforced with FRP and includes an entire chapter dedicated to the design of flexural members. The Committee 440 report should be officially published by ACI in the near future.

1.2 Scope

This report will focus on the design of the external FRP used for flexural strengthening of the War Memorial Bridge. Only the three-span continuous portion of the bridge was considered in this project. Results of the bridge analysis (presented in Chapter Two) show that only the positive moment capacity of the bridge is insufficient.

Because the shear capacity and negative moment capacity of the bridge were adequate at all points along the spans considered, they were not addressed.

Different modeling methods and criteria were used in order to complete multiple flexural designs. The required length of the external FRP composite was based on results of the bridge analysis performed by Auburn University and ALDOT personnel, ultimate strength design, and ACI 318-99 code provisions pertaining to the termination of flexural reinforcement. The design of the external FRP composite also included an analysis of the anchorage stresses at the concrete-FRP interface. Detailed discussion of each design procedure is presented in Chapter Three. All design results are presented in Chapter Four.

Chapter 2: Problem Overview

2.1 Bridge Information

The War Memorial Bridge is located in Macon County, Alabama, on Alabama Highway 81, about 0.3 miles north of Interstate 85. This bridge is constructed of reinforced concrete (RC) and was completed in 1945. The bridge accommodates two-way traffic in a north-south orientation. Multiple simply supported approach spans are located on both ends of the bridge. Between the simply supported approach spans, a three-span continuous section crosses Uphapee Creek.

The bridge is load posted for two types of trucks: the tri-axle dump truck and the concrete truck. Load postings limit these two truck types to a maximum gross weight of 28 tons. The short wheelbases of these two truck types cause large moments to occur in the bridge girders. Heavier trucks with longer wheelbases do not cause such critical moments, but may cause critical shear loads.

2.2 Bridge Geometry

The continuous portion of the War Memorial Bridge is three spans long and is symmetric about the center of the middle span. The end spans (Spans #9 and #11) are 48 feet long and the middle span (Span #10) is 65 feet long. Spans are measured relative to the girder ends. Each span consists of four reinforced concrete girders cast monolithically with a reinforced concrete slab. The depth of the girders varies from a minimum of 30.5 in to a maximum of 71 in over the interior supports.

The cross section of the bridge is symmetrical about its centerline. Girders are at an 88 in spacing, measured center-to-center on the girder webs. A cantilevered portion of the slab extends 28 in from the center of the exterior girders. The curb and guardrail were cast after the rest of the structure.

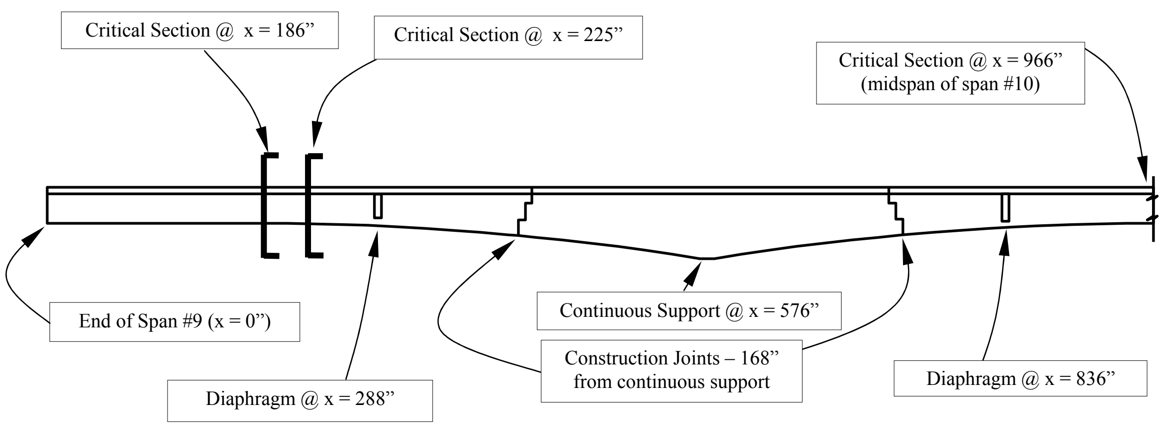

Figure 2.1 Elevation of a Typical Girder of the War Memorial Bridge

Because of the size of the bridge, multiple concrete castings were necessary. The bridge was cast in five stages, resulting in four construction joints located 168 in to either side of the interior supports. Diaphragms were cast monolithically with the girders and slab. There are two diaphragms in the center span at one-third points, and one in each end span at midspan. Diaphragms are 6 in shallower than the adjacent girders.

2.3 Analysis of the War Memorial Bridge

In order to determine the location of strength deficiencies, the capacity of the bridge girders had to be compared to the effects of factored loads along the three continuous spans. Using their standard bridge analysis and rating software, ALDOT personnel calculated the effects of factored loading on interior and exterior girders in the continuous region of the structure. Auburn University investigators used the information provided by ALDOT to construct strength demand curves. The design shear and moment capacities of both interior and exterior girders were determined by Auburn University and compared to the strength demand curves.

ALDOT personnel calculated the effects of factored loading on individual bridge girders with a computer program called PC-BARS. The analysis results were used to determine the maximum factored shears and moments for interior and exterior girders. For positive moment design, the critical loading was of a fully loaded tri-axle truck, which is currently prohibited from using the bridge. Maximum factored loads on the girders were obtained at distinct locations in the three-span continuous region of the structure. This was accomplished by varying the location of the loads being applied to the bridge.

The capacities of interior and exterior girders were calculated by Auburn University investigators. Shear and moment capacity were checked along the three spans. The effects of terminated reinforcement were accounted for in these checks. Plots of the nominal design shear and moment capacity of the girders were constructed so that they could be compared to the strength demand curves.

Results from the analysis of the girders' strength capacity and the PC-BARS program revealed that a strength deficiency did exist. The positive moment capacity of both interior and exterior girders was not sufficient in portions of all three spans. Negative moment capacity and shear capacity of the girders was found to be sufficient for all spans. In order to remove the load postings on the bridge, the positive moment capacity of the bridge needed to be increased.

Plots of the positive moment capacity and the positive moment demand of the interior and exterior bridge girders are shown in Figures 2.1 and 2.2. Moment capacities of the girders are shown for all of Span #9 (0–576 in) and half of Span #10 (576–966 in). Moment capacity of the RC members was based on actual member dimensions. Termination points of reinforcing bars exist within each member and are the cause for the abrupt changes in the moment capacity seen in both figures.

Regions where the moment demand exceeds the design moment capacity require strengthening. Critical locations were chosen within the deficient regions and were used in the external FRP design process. The critical locations were chosen to be at 186 in, 225 in, and 966 in. The cross section at each critical location was used to determine the amount of FRP needed for adequate flexural strengthening.

Preliminary calculations showed that the addition of external FRP composites to the girder soffits would increase the cracked-section moment of inertia, I_{cr}, by 4–5 percent. A result of the increased cracked-section moment of inertia of the strengthened portion of the structure would be the redistribution of moments throughout the structure. The amount of moment redistribution depends on how much external FRP composite material is used for strengthening, as well as where it is placed on individual girders. There are numerous difficulties inherent in precisely determining the distribution of moments in an indeterminate RC structure consisting of cracked, nonprismatic members (Nilson 1997). Given that the increase in stiffness of the strengthened portion of the structure would be limited to approximately five percent, the investigators estimated that the resulting increase in positive moment would be less than two percent. This slight increase was considered when selecting the quantity and extent of the FRP reinforcement.

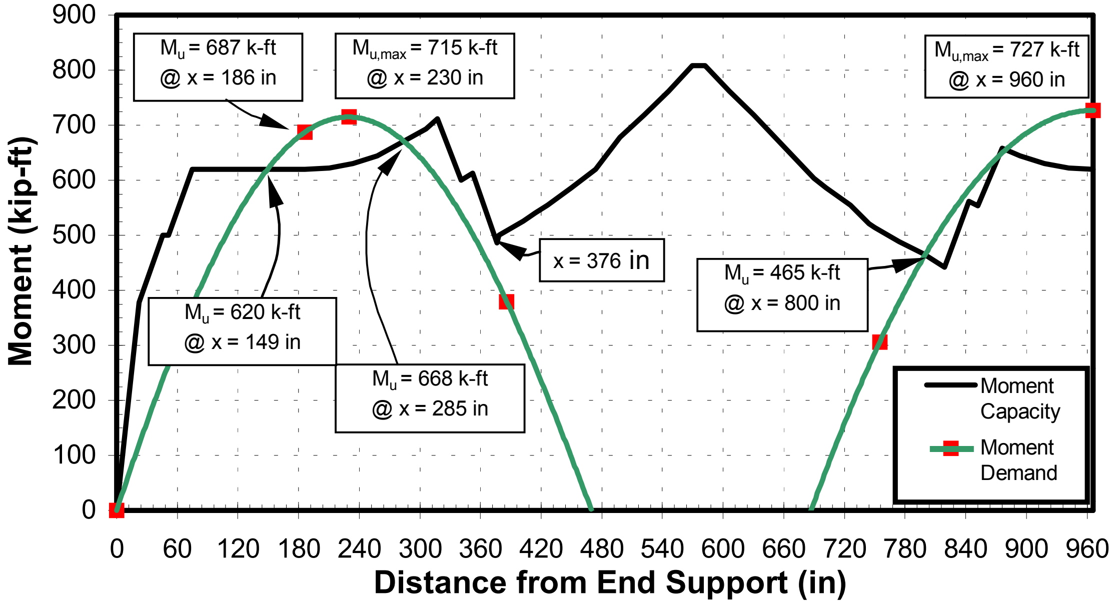

Figure 2.2 Design Moment Capacity and Moment Demand for an Interior Girder

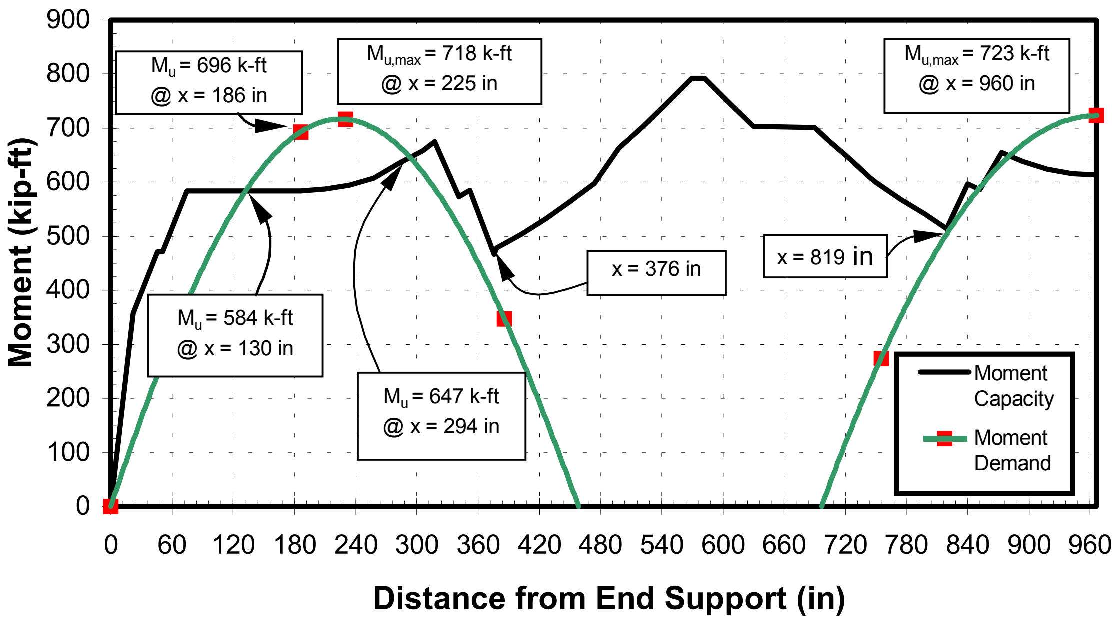

Figure 2.3 Design Moment Capacity and Moment Demand for an Exterior Girder

2.4 Member Geometry

Both the interior and exterior girders of the War Memorial Bridge are T-shaped reinforced concrete members. Deck slab and girder web were cast monolithically with steel reinforcement present in both the longitudinal and transverse directions. Because both the interior and exterior girders have a haunch, the depth of the members varies over a majority of the three continuous spans. A typical cross section of an exterior girder is shown in Figure 2.3. The cross section of an interior girder is symmetric about the girder centerline and has an effective flange width of 88 in.

Four layers of longitudinal reinforcing steel can be found in the cross section of the bridge girders. Two layers of steel are in the deck slab and act as compression reinforcement under service level loads. The other two rows of longitudinal steel are in the girder web. These bars act as the primary tension reinforcement and comprise a majority of the flexural steel in the member. Steel reinforcing bars in the web of the girders are square in shape and vary in size from 1 in square to 1-⅛ in square. Steel stirrups are spaced at 15 in on-center in regions of high positive and negative moment. All stirrups have a ½ in diameter.

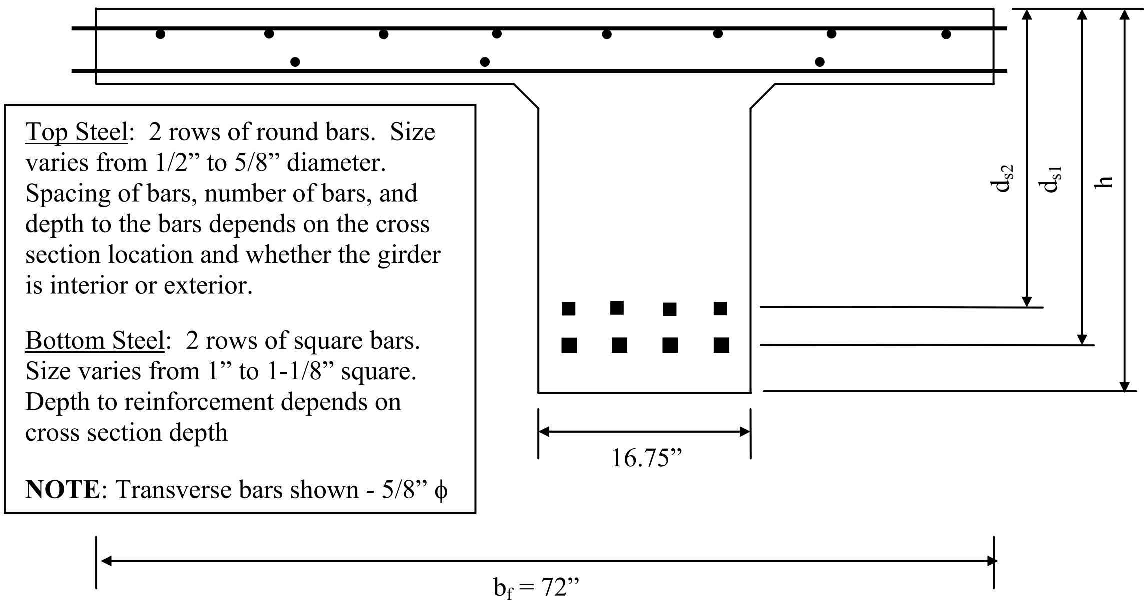

Figure 2.4 Typical Exterior Girder in the War Memorial Bridge

Longitudinal Steel Properties	**Concrete Properties**
Yield Strength, f_y = 33 ksi Elastic Modulus, E_s = 29000 ksi	Compressive Strength, f_c’ = 5000 psi Elastic Modulus, E_c = 4030 ksi Rupture Strength, f_r = 530 psi β_1 = 0.80

Table 2.1 Material Properties

Material properties of both the steel and concrete were provided by ALDOT. The yield strength of the reinforcing steel was determined to be 33 ksi (AASHTO Interim Revisions, 1994). This was a standard strength of reinforcing steel produced prior to 1954. Concrete strength was determined by testing sample cores that were taken from the bridge deck. A concrete compressive strength, f'_c, of 5000 psi was determined to be conservative for analysis and design purposes based on the results of the core tests. Material properties used for analysis purposes are listed in Table 2.1.

Chapter 3: Design Procedures

The flexural design of the FRP strengthened girders was completed using multiple variations of two distinct design procedures. The first design was done according to the recommendations in Chapter Nine of the ACI Committee 440 draft report, *Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures* (ACI Committee 440, 2000). The second design was performed with a computer spreadsheet that used a nonlinear stress-strain relationship for the compressive stress in the concrete. The methods used, their differences, and the variations associated with each design procedure will be discussed in the following sections.

3.1 ACI Committee 440 Flexural Design

The flexural design proposed in Chapter Nine of the ACI Committee 440 draft report is based on ultimate strength design. Force equilibrium, strain compatibility, and the constitutive material relationships of concrete, steel, and FRP are used to derive the equations needed to calculate the nominal amount of external FRP needed for flexural strengthening. The procedure requires the following assumptions to be true in order for the design to be valid (ACI Committee 440, 2000).

- All calculations are based on actual member dimensions, locations of reinforcing steel, and material properties of the existing member;

- Strains in the reinforcing steel and concrete are directly proportional to their distance from the neutral axis (plane sections remain plane);
- The maximum compression strain in the concrete is 0.003;
- Any tensile stresses in the concrete are neglected in design;
- The FRP composite behaves in a linear elastic manner until failure;
- Steel is linear elastic until it yields and perfectly plastic thereafter. Strain hardening is neglected;
- Perfect bond exists between the steel reinforcement and the concrete and between the concrete and the external FRP composite ("no slip" condition);
- The compressive stresses in the concrete at failure may be effectively represented by a rectangular stress distribution as defined in Section 10.2.7 of ACI 318-99.

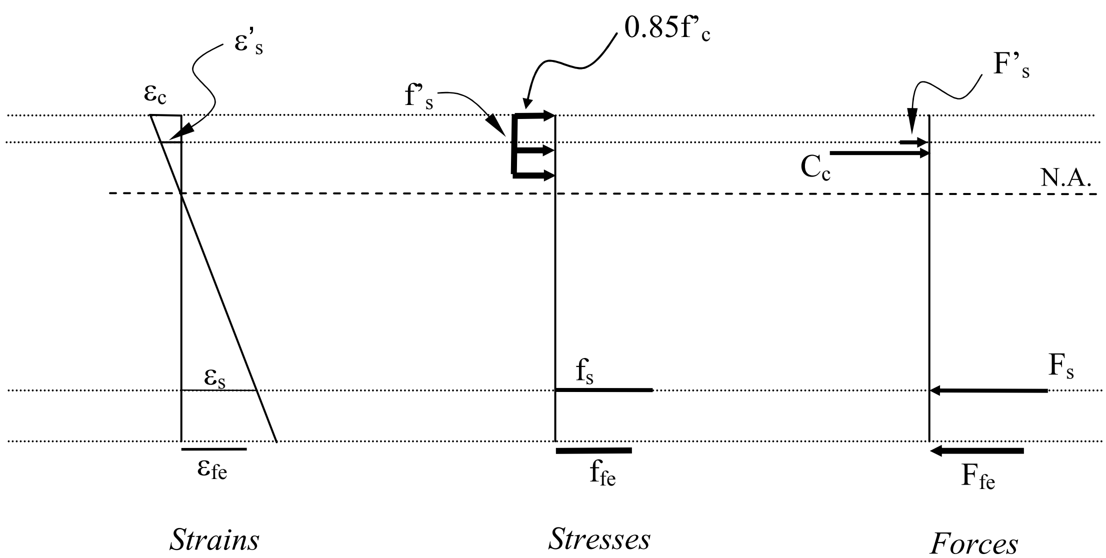

Figure 3.1 Strain, Stress, and Force diagram for a strengthened RC cross section

Figure 3.1 shows the strain, stress, and force diagrams for a typical FRP strengthened reinforced concrete section. A discontinuity in the strain profile exists at the bottom concrete fiber. The small difference in strain, ε_{bi}, is a result of dead loads present on the member when the FRP is applied. Once the FRP has been applied, strain compatibility between the FRP and the rest of the reinforced concrete section is assumed. The variation in strains between the bottom concrete fiber and the FRP composite should be accounted for in the design procedure (ACI Committee 440, 2000).

Equilibrium of the cross section is reached when the sum of cross-sectional forces is equal to zero (Equation 3.1). The terms for the force in the concrete, steel, and FRP composite are expanded in Equation 3.2. It is recommended that the Whitney stress block be used to represent the compressive stress in the concrete. This is the choice of the designer, though, and alternative stress distributions may be employed (ACI Committee 440, 2000).

$$\begin{gathered} \Sigma F = 0 \\ T = C \end{gathered} \quad (Equilibrium) \tag{3.1}$$

$$A_s f_y + A_f f_{fe} = 0.85 f'_c \, b \beta_1 c + A'_s \, f_y \tag{3.2}$$

The terms in Equation 3.2 are those commonly used in reinforced concrete design. All terms used can be found in the notation section of Appendix A. The term f_{fe} is the stress in the FRP composite and is defined as

$$f_{fe} = C_E \kappa_M f_{fu} \tag{3.3}$$

The term C_E is a reduction factor for the allowable FRP strain. It varies with the type of FRP being used and the level of exposure to environmental conditions. The κ_M term further reduces the amount of strain allowed in the FRP in an effort to preclude non-ductile failure modes, such as FRP debonding and delamination of the concrete cover at the level of the steel reinforcing bars. The κ_M term depends on the stiffness of the FRP composite and the amount of bonded area of FRP. It is defined in Equation 3.4.

$$\kappa_M = \begin{cases} 1 - \dfrac{nt_f E_f}{2{,}400{,}000} & \text{for} \quad nt_f E_f \le 1{,}200{,}000 \\ \dfrac{600{,}000}{nt_f E_f} & \phantom{\text{for}} \quad nt_f E_f > 1{,}200{,}000 \end{cases} \tag{3.4}$$

It should be noted that the use of Equation 3.4 to calculate κ_M is dependent on the use of the proper units for t_f and E_f.

Once force equilibrium has been satisfied, the nominal moment capacity of the strengthened section can be calculated by summing moments about the resultant of the compressive concrete force, C_c. Due to the use of Ψ, summing moments about a location other than the resultant of the compressive concrete force will change the calculated nominal moment capacity, M_n.

$$M_n = A_s f_y \left(d - \frac{\beta_1 c}{2} \right) + \Psi \, A_f f_{fe} \left(d_f - \frac{\beta_1 c}{2} \right) \tag{3.5}$$

External FRP composites used as tension reinforcement in reinforced concrete are not as dependable as interior steel reinforcement, so an additional reduction factor, Ψ, is used in Equation 3.5. ACI Committee 440 recommends that Ψ be taken as 0.85.

ACI Committee 440 suggests that the ACI 318-99 load and reduction factors be used to compute the required nominal moment capacity for members strengthened with externally applied FRP composites. Thus, the reduced nominal moment capacity, ϕM_n, must be greater than or equal to the factored ultimate moment.

$$\phi M_n \geq M_u \tag{3.6}$$

$$M_{n,req'd} = \frac{M_u}{\phi} \tag{3.7}$$

This requirement is consistent with the flexural design method used in ACI 318-99. The value of the strength reduction factor, ϕ, for reinforced concrete members strengthened with external FRP is consistent with Appendix B of ACI 318-99. The strain in the primary steel reinforcement at failure determines the magnitude of the flexural strength reduction factor. The ϕ factor for flexure ranges from 0.70 to 0.90.

3.1.1 ACI Committee 440 Design, Variation I

Calculation of the nominal amount of FRP needed for strengthening of the reinforced concrete girder was done with the aid of a simple computer spreadsheet. Given material properties and cross section geometry are used to calculate strains, stresses, and forces. Some changes were made to the equations given in the ACI Committee 440 draft report so that compression reinforcement could be included. To ensure the spreadsheet was performing the calculations correctly, moment-curvature calculations for an unstrengthened RC member were checked against the same calculations done by hand. A good correlation existed between the two sets of calculations for the unstrengthened RC bridge girder.

The following steps outline the procedure used in the spreadsheet.

1. Assume a thickness and width for the FRP composite. Most manufacturers will specify the thickness of their FRP composite products, so the width can be varied to increase or decrease the gross area of FRP.
2. Assume a neutral axis position, c.
3. Calculate κ_M, the allowable strain reduction factor for the FRP (Equation 3.4).
4. Calculate the strain in the FRP composite,

$$\varepsilon_{fe} = C_E \kappa_M \varepsilon_{fu} \tag{3.8}$$

5. Calculate the strain in the topmost concrete fiber,

$$\varepsilon_c = \frac{c}{d_f - c}\left(\varepsilon_{fe} + \varepsilon_{bi}\right) \tag{3.9}$$

6. Calculate the strain in the steel reinforcement,

$$\begin{aligned} \varepsilon_s &= \frac{d - c}{d_f - c}\left(\varepsilon_{fe} + \varepsilon_{bi}\right) \\ \varepsilon'_s &= \frac{d' - c}{d_f - c}\left(\varepsilon_{fe} + \varepsilon_{bi}\right) \end{aligned} \tag{3.10}$$

7. Calculate the stress in the steel reinforcement,

$$
\begin{aligned}
f_s &= E_s \varepsilon_s \le f_y \\
f'_s &= E_s \varepsilon'_s \le f_y
\end{aligned}
\tag{3.11}
$$

8. Calculate the stress in the FRP composite,

$$f_{fe} = E_f \varepsilon_{fe} \tag{3.12}$$

9. Calculate the force in the steel reinforcement and the FRP composite,

$$
\begin{aligned}
F_s &= A_s f_s \\
F'_s &= A'_s f'_s \\
F_f &= A_f f_{fe}
\end{aligned}
\tag{3.13}
$$

10. Calculate the force in the concrete,

$$C_c = 0.85 f'_c b \beta_1 c \tag{3.14}$$

11. Sum the forces on the cross section,

$$\Sigma F = F_s + F'_s + F_{fe} + C_c \tag{3.15}$$

12. If the sum of the forces is approximately zero ($|\Sigma F| < 0.001$ kips), then calculate the nominal moment capacity of the section,

$$M_n = A_s f_s \left(d - \frac{\beta_1 c}{2} \right) + A'_s f'_s \left(d' - \frac{\beta_1 c}{2} \right) + \Psi A_f f_{fe} \left(d_f - \frac{\beta_1 c}{2} \right) \tag{3.16}$$

If not, then return to Step 2.

13. Based on the amount of strain in the primary steel reinforcement, calculate the strength reduction factor,

$$\phi = \begin{cases} 0.70 & \varepsilon_s \le \varepsilon_y \\ 0.70 + 0.20\left(\dfrac{0.005 - \varepsilon_s}{0.005 - \varepsilon_y}\right) \quad \text{for} & \varepsilon_y < \varepsilon_s < 0.005 \\ 0.90 & \varepsilon_s \ge 0.005 \end{cases} \tag{3.17}$$

14. Compare ϕM_n to M_u. If $\phi M_n < M_u$, then the amount of FRP used must be increased in order to satisfy Equation 3.6. If ϕM_n is significantly greater than M_u, then the amount of FRP should be decreased so that the most efficient design is reached.

3.1.2 ACI Committee 440 Design, Variation II

As in the first variation of this design procedure, all calculations were done by a computer spreadsheet. Only the characterization of the compressive stress in the concrete was changed in this variation. The compressive concrete stresses were modeled as linear elastic. The resulting triangular stress distribution should produce more conservative estimates of the moment capacity of FRP strengthened RC members. The changes made in this variation of the design procedure are listed below.

10. Calculate the force in the concrete,

$$C_c = 0.5 E_c \varepsilon_c bc \tag{3.18}$$

12. If the sum of the forces converges within the given tolerance ($\Sigma F < 0.001$ K), then calculate the nominal moment capacity of the section,

$$M_n = A_s f_s \left(d - \frac{c}{3} \right) + A'_s \, f'_s \left(d' - \frac{c}{3} \right) + \Psi \; A_f f_{fe} \left(d_f - \frac{c}{3} \right) \tag{3.19}$$

If not, then return to Step 2.

3.2 Computer Spreadsheet Design

The second method used to design the FRP strengthened bridge girders used a nonlinear stress-strain relationship for the concrete in the member cross section. The computer spreadsheet design was used to accomplish two main goals:

- Determine the most efficient amount of external FRP composite required for adequate strengthening of the bridge girders;
- Check the validity of the design method proposed in Chapter Nine of the ACI Committee 440 draft report.

Some serviceability concerns related to the strengthening of RC members with external FRP composites were also examined with the use of the computer spreadsheet design.

3.2.1 Primary Design of FRP Strengthened RC Member

In order for the reinforced concrete girders of the War Memorial Bridge to be designed more efficiently, the modeling of compressive concrete stresses needed to be refined. The function used to model the nonlinear stress-strain behavior of the concrete was developed by Collins and Mitchell (1991). The use of the nonlinear stress-strain function resulted in a more accurate calculation of moment capacity and a better representation of the moment-curvature behavior of design cross sections.

Force equilibrium, strain compatibility, and constitutive material relationships had to be satisfied for the design to be valid. In order for this to be done, some assumptions were made about the cross section and its behavior.

- All calculations are based on actual dimensions, locations of reinforcing steel, and material properties of the existing member;
- The strains in the reinforcing steel and concrete are directly proportional to their distance from the neutral axis (plane sections remain plane);
- The maximum compression strain in the concrete is 0.003;
- FRP behaves in a linear elastic manner until failure;
- Steel is linear elastic until yield and perfectly plastic thereafter. Strain hardening is neglected;
- Perfect bond exists between the steel reinforcement and the concrete and between the concrete and the external FRP composite ("no slip" condition);
- The nonlinear behavior of concrete, as modeled by a function from Collins and Mitchell (1991), is used in the calculation of the moment capacity.

The spreadsheet divides the member cross section into distinct elements. Each element is defined by its material properties and position in the cross section, which are input by the user. The concrete of the section is divided into horizontal layers. These layers of concrete are made less thick in regions where compression is expected under service loading. Using thin layers of concrete in compressive regions results in a more accurate representation of the strain, stress, and force in the concrete. Representation of the reinforcing steel and the FRP composite depends greatly on the design goals. Multiple reinforcing bars or FRP composites can be represented as a single entity at a given position, or they can be input separately if stresses in particular components are important.

Determination of the nominal moment capacity of the RC member relies on equilibrium of the forces in the cross section. Force equilibrium is satisfied when the forces in the distinct material elements sum to zero for the entire cross section. Forces in individual elements depend on the material properties of the element, the element's position in the cross section, the location of the neutral axis, and the strain at the extreme compressive fiber. The following procedure outlines the method used by the computer spreadsheet to calculate the moment capacity of an RC member strengthened with external FRP composites.

1. All user-defined information must be input for each material element before any calculations are done. This includes material properties for all constitutive materials of the cross section and the geometry and location of individual material elements.
2. Input a given top fiber strain (ε_c). This strain is limited to 0.003.

3. Make a guess at the location of the neutral axis. Initial guess of the neutral axis should be larger than expected. This enables the solver function to converge on an answer more quickly and more accurately.
4. Strains, stresses, and forces will be calculated by the spreadsheet based on strain compatibility. If the forces in the cross section do not sum to zero, then the solver will choose a new value for the neutral axis location and repeat this step.
5. Once force equilibrium has been satisfied, the nominal moment capacity of the section is calculated by summing the moments of each element about the centroid of the compressive concrete force. Calculation of the moment capacity of the section includes the use of κ_M, ϕ (as defined in Appendix B of ACI 318-99), Ψ, and C_E.

Repetition of this procedure was done for top fiber strains between zero and the limiting value of 0.003. The resulting data was used to create a moment-curvature plot for the strengthened RC girder. A plot of the concrete stress over the depth of the member was also created from the data and compared to the concrete stress distributions used in the ACI Committee 440 design method.

3.2.2 Secondary Design of FRP Strengthened RC Member

The computer spreadsheet used for design of the RC girders externally strengthened with FRP was employed to do a similar design for the same critical sections of the bridge girders. The secondary computer spreadsheet design method uses alternative criteria for limiting the strain in the FRP composite. Other ideas used in the design proposed in the ACI Committee 440 draft report, such as C_E and Ψ, are used in calculation of the moment capacity of the section. Representation of the member cross section and calculation of strains, stresses, and forces are done in the same manner as described in the previous section.

Limiting the strain in the external FRP composite is an idea that stems from the results of many laboratory tests performed in recent years. Experiments done with RC members externally strengthened with FRP composites have shown that large strains in FRP composite material tend to produce non-ductile failure modes. Debonding of the FRP composite from the concrete surface and concrete cover rip-off failures along the level of the tension reinforcement are two common non-ductile failure modes.

In order to preclude these phenomena, the strain in the FRP composite should be limited in the design of the strengthened member. ACI Committee 440 does this by multiplying the maximum strain in the FRP, ε_{fu}, by a reduction factor, κ_M, which is dependent on the stiffness and bonded area of the FRP composite. The research of Sergio Breña (2000) at the University of Texas-Austin has shown that conservative designs of

FRP-strengthened RC beams are possible if an upper limit is placed on the strain in the FRP composite.

Based on laboratory tests of simply supported RC specimens strengthened with unidirectional external FRP composites and transverse FRP straps, Breña recommends an upper limit on the strain in the FRP to be 0.007. For members with similar geometries and strengthening schemes, the use of this design criterion should prevent the FRP composite from debonding from the concrete substrate. It was noted by Breña, though, that the amount of strain in the FRP composite at failure was highly dependent on the amount of FRP used and the distance from the critical section to the first flexural crack.

Because the girders of the War Memorial Bridge are not simply supported and the FRP strengthening scheme did not include transverse FRP straps, Breña's criterion was not directly applicable. Breña's philosophy on limiting the strain in the FRP in order to calculate the member design capacity may be a useful tool, though. Until more is known about the effects of FRP plate geometry on non-ductile failure modes, it may be best to utilize both the Breña and ACI Committee 440 philosophies on FRP strain limitation. In this manner, the more conservative of the two design approaches can be followed.

3.2.3 Design for Serviceability

Serviceability requirements often control the design of bridge members. Excessive deflections, cracking of the concrete, and corrosion of the steel reinforcement are just a few of the serviceability issues that must be considered. The service life of a bridge can be lengthened if care is taken to prevent such problems.

After the initial analysis of the War Memorial Bridge was performed, a serviceability problem became apparent. If the posting were removed, service level truck loads would cause stresses in the primary tensile reinforcement of the girders in excess of 85 percent of the steel yield stress. ACI Committee 440 recommends that the service level stresses in the primary steel reinforcement be reduced such that they are no more than 80 percent of the steel yield stress. Strengthening of the girders with external FRP composite should reduce the amount of tension carried by the primary tension reinforcement.

Calculations of the steel stresses were made by using strain compatibility and assuming cracked-section properties for the girder. The stress at the steel reinforcement centroid was calculated using Equation 3.20. The first term of the equation accounts for the stress due to dead loads on the unstrengthened cross section. The second term is a calculation of the steel stress under live loads acting on the strengthened cross section.

$$f_s = n_s \left(\frac{M_D (d-c)}{I_{cr}} + \frac{M_L (d-c)}{I'_{cr}} \right) \tag{3.20}$$

The amount of FRP composite applied to the beam was varied in order to analyze its effect on the stress level in the primary reinforcing steel. Discussion of the results of the serviceability design is included in Chapter Four.

3.3 *Additional Design Requirements*

Once the cross-sectional area of external FRP had been selected, it was necessary to determine how far the FRP composite would be extended from the critical design sections. After external FRP plate lengths had been chosen, it was necessary to ensure that the anchorage stresses in the FRP composite or the concrete would not be large enough to cause a non-ductile anchorage failure.

3.3.1 Plate Length of the External FRP Composite

Ultimate strength design requires that the nominal flexural capacity of a member, ϕM_n, be greater than or equal to the factored ultimate moment, M_u. With this in mind, the external FRP composite was needed for flexural strengthening on all girders where the unstrengthened design moment capacity was less than the factored ultimate moment. Plots of moment capacity versus moment demand are shown in Figures 2.1 and 2.2.

Section 12.10.3 of ACI 318-99 states that internal steel reinforcing bars that are terminated in a positive moment region must extend a distance equal to the effective depth of the member, d, past where they are needed for flexural strength. Using this design guideline, the external FRP composite plates were extended a distance equal to the depth of the beam, h, beyond the points at which they were needed for flexural resistance. Extension of the FRP composite plate accounts for the additional tensile stresses induced by the shear force in the member. For practical reasons, the composite plate lengths were later rounded up to the nearest foot.

3.3.2 Determination of Stresses in the FRP Composite Plate

When external FRP composites are applied to flexural members, large shear and peeling stresses have been shown to occur near the curtailment of the FRP composite plate. These stresses are dependent on the thickness of the composite plate, the stiffnesses of the composite plate and the adhesive used, and the distance from the plate curtailment to the support. Failure to account for the presence of such stresses may result in a premature failure of the member.

Experimental studies show that composite plate debonding and anchorage failures occur less often in simple span configurations if the composite plate is extended all the way to the support (Tedesco and El-Mihilmy, 2001; Sharif et. al., 1994; Arduini and Nanni, 1997). If this is not possible, then it is recommended that the FRP composite plate be extended as far as possible along the span, reducing the distance between the plate curtailment and the support. Studies also show that stiffer adhesives and stiffer FRP composites (larger elastic moduli or thicker plates) result in increased anchorage stresses at plate ends (Taljsten, 1997; Sharif et al, 1994; Rahimi and Hutchinson, 2001).

Extending the FRP composite to the supports on each span of the War Memorial Bridge was unnecessary because they are in a negative moment region. For a continuous structure, extending the external FRP composite plate to the inflection points would be analogous to extending the FRP plate to the supports on a simple span. In order to be sure that the design from Section 3.3.1 was acceptable for anchorage of the FRP plates, a check on the stresses in the FRP composite plate was performed.

The procedure proposed by Tedesco and El-Mihilmy (2001) was used to determine the shear and peeling stresses in the FRP composite under service level loading. As long as the calculated stresses were within tolerable limits, the design lengths from Section 3.3.1 were acceptable. If stresses were shown to be too high, the FRP composite plate would need to be extended further from the critical design section.

The following procedure is an outline of the method proposed by Tedesco and El-Mihilmy (2001) to calculate shear and peeling stresses in an FRP composite plate externally bonded to a concrete flexural member. Definitions of terms used in the equations can be found in the notation section of the Appendix. All coefficients have been calculated for pound and inch units.

1. Design the external FRP composite reinforcement for flexural strengthening. This can be done according to the design specifications given in Sections 3.1 and 3.2.
2. Calculate the cracked-section moment of inertia, I_{cr}. This should be done using a transformed cross section.

3. Calculate the coefficients α_f and Ψ_f.

$$\alpha_f = 1.411\sqrt{\frac{E_a t_f}{E_f}} \qquad (3.21)$$
$$\Psi_f = 1.35 - 12.5\left(\frac{L_f}{L}\right)$$

The factor α_f is a function of the mechanical properties of the adhesive and composite plate and accounts for the nonlinearities in the plate end region. The term Ψ_f is a calibration factor on the moment used in Equation 3.26 to calculate σ_x. This factor was obtained from statistical analysis done on the data gathered from tests on simply supported, FRP strengthened RC members. It was decided that assuming $\Psi_f = 1.0$ for this design was conservative because the girders in this study are not simply supported.

4. Calculate the elastic shear stress and normal bending stress, τ and σ_x.

$$\tau = \frac{V_o t_f (d_f - c)}{I_{cr}} \qquad (3.22)$$
$$\sigma_x = \frac{M_o \Psi_f^2 (d_f - c)}{I_{cr}}$$

5. Calculate τ_{max} and $\sigma_{z,max}$.

$$\tau_{\max} = \tau + \alpha_f \sigma_x \qquad (3.23)$$
$$\sigma_{z,\max} = 1.3\sqrt{\alpha_f}\,\tau_{\max}$$

6. Calculate the principal stresses, σ_1 and σ_2.

$$\sigma_1 = \frac{\sigma_{z,\max}}{2} + \sqrt{\left(\frac{\sigma_{z,\max}}{2}\right)^2 + \tau_{\max}^2}$$
$$\sigma_2 = \frac{\sigma_{z,\max}}{2} - \sqrt{\left(\frac{\sigma_{z,\max}}{2}\right)^2 + \tau_{\max}^2} \tag{3.24}$$

7. Compare the principal stresses with the appropriate stress limits. The stress limits will depend on the biaxial state of stress in the concrete at the concrete-FRP interface.

If both σ_1 and σ_2 are positive (tension-tension stress state),

$$f_{tu} = f_t = 6.4\sqrt{f'_c} \tag{3.25}$$

If σ_1 and σ_2 are of opposite signs (tension-compression stress state),

$$f_{tu} = f_t\left(1 + \frac{\sigma_2}{f'_c}\right) \tag{3.26}$$

A reduction factor of 0.70 was used to reduce the allowable tensile stress in the concrete at the bond interface, f_{tu}. A tensile failure in the concrete from excessive stresses at the bond interface will be non-ductile, so a reduction factor of 0.7 is consistent with other reduction factors in ACI 318-99.

The results of the design procedures in this section will be summarized in Chapter Four along with the results of the flexural design procedures described in Sections 3.1 and 3.2.

Chapter 4: Design Results

The procedures described in Chapter Three were used to design the FRP strengthened bridge girders of the War Memorial Bridge. Two different composites were given serious consideration and are the focus of this chapter. The two composites considered were the Tyfo UC composite manufactured by Fyfe Co. and the Fibercote composite. The choice of these composite materials was based on their material properties, the results of initial flexural designs, and some economic issues.

4.1 ACI Committee 440 Design Method

Both variations of the ACI Committee 440 design procedure were used to design the three critical sections of an interior and exterior girder. Design of the exterior girder was completed for both the Tyfo UC composite and the Fibercote composite. The two composites had different material properties and geometries. The Tyfo UC composite can be manufactured in 0.055 in or 0.075 in thick precured strips that are 2 in or 4 in wide. The Fibercote composite material is also precured and can be manufactured in various widths and thicknesses to accommodate specific needs. For the design of the bridge girders with the Fibercote composite, plate thicknesses of 0.05 in and 0.10 in were considered while the plate widths of 12 in and 14 in were considered. Properties of each composite product are listed in Table 4.1.

		Fibercote	Tyfo UC
Width	(in)	Specified by User	2" or 4" strips
Thickness	(in)	Specified by User	0.055" or 0.075" per sheet
Elastic Modulus	(ksi)	17500	22500
Tensile Strength	(ksi)	173	405
Maximum elongation	(percent)	0.99	1.80

Table 4.1 Composite material properties and geometry

Results from the two variations of the ACI Committee 440 design were obtained for a number of strengthening systems. The nominal moment capacity of the FRP strengthened member was compared to the required nominal moment to determine if the particular design of external FRP would be adequate. The strengthening systems used for each type of composite depended on the geometry and material properties of the FRP composite and the geometry of the bridge girder. For the Tyfo UC composite, a constant thickness of 0.055 in was used while the width of the composite sheet was varied. Design of the Fibercote composite was done by varying both the width and the thickness.

As expected, the nominal moment capacity calculated by variation I of the ACI Committee 440 design method was 1.8–2.3 percent larger than that calculated by variation II. The reason for the difference in the calculated moment capacity of a given cross section lies in the way the two variations model the compressive stress in the concrete. By using the smaller of the two calculated moment capacities, a slightly more conservative design was possible. A comparison of the results from variation I, variation

II, and the computer spreadsheet design in the next section will reveal just how conservative the ACI Committee 440 design truly is.

Tables 4.2 and 4.3 show the results obtained from the ACI Committee 440 design at the three critical sections of an exterior girder. According to the nominal design moment capacity calculated by variation II of the procedure, an 8 inch width of Tyfo UC composite was not adequate to strengthen all critical sections. A 10 in wide Tyfo UC composite plate will be necessary to adequately strengthen the critical section at 225 in. In order to account for the increased moments caused by increasing the stiffness of the girders, the 10 in wide Tyfo UC composite was chosen for strengthening for all necessary locations on the exterior girder.

An examination of the results from variation II of the design revealed that the Fibercote composite was unable to provide adequate flexural strengthening for all design cross sections. Results of both design variations showed that the Fibercote composite material was not able to provide an adequate increase in the flexural capacity at the 225 in critical section. Results from design variation I revealed that the Fibercote FRP composite was capable of strengthening the exterior girder at two of the three critical sections only if a 14 in wide, 0.05 in thick plate was used. This plate has nearly the same thickness as the Tyfo UC composite plate described earlier, but is almost 50 percent wider.

Critical Section	Variation	b_f	t_f (in)	M_n (kip-in)	$M_{n,req'd}$ (kip-in)
Critical Section 966”	***Variation I***	7.000	0.055	9640	9640
		8.000	0.055	9870	9640
		9.000	0.055	10100	9640
		10.000	0.055	10320	9640
	Variation II	7.000	0.055	9460	9640
		8.000	0.055	9680	9640
		9.000	0.055	9900	9640
		10.000	0.055	10130	9640

Critical Section	Variation	b_f	t_f (in)	M_n (kip-in)	$M_{n,req'd}$ (kip-in)
Critical Section 225”	***Variation I***	7.000	0.055	9370	9580
		8.000	0.055	9600	9580
		9.000	0.055	9830	9580
		10.000	0.055	10060	9580
	Variation II	7.000	0.055	9180	9580
		8.000	0.055	9410	9580
		9.000	0.055	9640	9580
		10.000	0.055	9870	9580

Critical Section	Variation	b_f	t_f (in)	M_n (kip-in)	$M_{n,req'd}$ (kip-in)
Critical Section 186”	***Variation I***	7.000	0.055	9260	9280
		8.000	0.055	9490	9280
		9.000	0.055	9720	9280
		10.000	0.055	9940	9280
	Variation II	7.000	0.055	9080	9280
		8.000	0.055	9300	9280
		9.000	0.055	9530	9280
		10.000	0.055	9750	9280

Table 4.2 ACI Committee 440 Design Results for an Exterior Girder Strengthened with Tyfo UC FRP Composite

		b_f	t_f (in)	M_n (kip-in)	$M_{n,req'd}$ (kip-in)
Critical Section 966”	***Variation I***	12.000	0.050	9430	9640
		14.000	0.050	9660	9640
		12.000	0.100	9530	9640
		14.000	0.100	9780	9640
	Variation II	12.000	0.050	9180	9640
		14.000	0.050	9410	9640
		12.000	0.100	9160	9640
		14.000	0.100	9400	9640

		b_f	t_f (in)	M_n (kip-in)	$M_{n,req'd}$ (kip-in)
Critical Section 225”	***Variation I***	12.000	0.050	9160	9580
		14.000	0.050	9390	9580
		12.000	0.100	9250	9580
		14.000	0.100	9510	9580
	Variation II	12.000	0.050	8910	9580
		14.000	0.050	9140	9580
		12.000	0.100	8890	9580
		14.000	0.100	9130	9580

		b_f	t_f (in)	M_n (kip-in)	$M_{n,req'd}$ (kip-in)
Critical Section 186”	***Variation I***	12.000	0.050	9050	9280
		14.000	0.050	9280	9280
		12.000	0.100	9150	9280
		14.000	0.100	9400	9280
	Variation II	12.000	0.050	8810	9280
		14.000	0.050	9040	9280
		12.000	0.100	8780	9280
		14.000	0.100	9030	9280

Table 4.3 ACI Committee 440 Design Results for an Exterior Girder Strengthened with Fibercote FRP Composite

Critical Section	Variation	b_f	t_f (in)	M_n (kip-in)	$M_{n,req'd}$ (kip-in)
966”	Variation I	7.000	0.055	9750	9700
		8.000	0.055	9990	9700
		9.000	0.055	10220	9700
		10.000	0.055	10460	9700
	Variation II	7.000	0.055	9540	9700
		8.000	0.055	9770	9700
		9.000	0.055	10000	9700
		10.000	0.055	10240	9700
225”	Variation I	7.000	0.055	9860	9530
		8.000	0.055	10100	9530
		9.000	0.055	10340	9530
		10.000	0.055	10580	9530
	Variation II	7.000	0.055	9640	9530
		8.000	0.055	9880	9530
		9.000	0.055	10120	9530
		10.000	0.055	10350	9530
186”	Variation I	7.000	0.055	9740	8890
		8.000	0.055	9980	8890
		9.000	0.055	10210	8890
		10.000	0.055	10450	8890
	Variation II	7.000	0.055	9530	8890
		8.000	0.055	9770	8890
		9.000	0.055	10000	8890
		10.000	0.055	10240	8890

Table 4.4 ACI Committee 440 Design Results for an Interior Girder Strengthened with Tyfo UC Composite

The Fibercote FRP composite material was used in the computer spreadsheet design to determine if a more refined model of the behavior of the strengthened cross section would produce different results. After the results of the design procedures were assessed, the Fibercote material was eliminated as a strengthening option. The Tyfo UC composite was used to complete the ACI Committee 440 design of the interior bridge girders.

Design results for the interior bridge girders are shown in Table 4.4. The required nominal moment capacity of the interior girders was slightly smaller than that of the exterior girders, as determined by the bridge analysis. An 8 inch width of Tyfo UC composite showed adequate strengthening of the interior girders. Calculated nominal moment capacity was 1–9 percent larger than the required nominal moment capacity at the three critical design sections.

4.2 Computer Spreadsheet Design

The computer spreadsheet was used to design strengthened interior and exterior girders at the three critical sections. A complete design was done for the Tyfo UC composite material. Design of the 225 in critical section on the exterior girder was done for the Fibercote material. The additional design with the Fibercote FRP composite was performed to verify that the results obtained from the ACI Committee 440 design procedure were reliable.

FRP composite plate widths were chosen based upon the results from the ACI Committee 440 design procedure. For design with the Tyfo UC composite, widths of 8 in, 10 in, and 12 in were used for both the interior and exterior girders. These widths were chosen because the Tyfo UC composite material comes in 2 in and 4 in wide strips. The thickness of the Tyfo UC composite was 0.055 in. A width of 14 in and thicknesses of 0.05 in and 0.10 in were used for design with the Fibercote composite.

4.2.1 Exterior Girder

Design with the Tyfo UC composite on the exterior girder revealed that the critical section at 225 in would control the design. The 8 in wide Tyfo UC composite plate did not provide an adequate amount of flexural strengthening at the 225 in critical section. The 10 in wide composite plate provided an adequate amount of flexural strengthening at all three critical sections. Nominal moment capacities of the strengthened cross sections calculated according to the ACI Committee 440 method were 3–5 percent larger than required for ultimate strength design. When moment redistribution in the strengthened girders was considered, the design moment capacity of the girders strengthened with the 10 in wide Tyfo UC composite was adequate at all critical sections.

Figures 4.1, 4.2, and 4.3 show the moment-curvature response of strengthened sections at the critical sections used for design. Figure 4.2 shows that the 8 in wide Tyfo UC composite plate was not adequate at the 225 in critical section. The 10 in and 12 in plate widths provided adequate flexural strengthening at all critical design sections. All designs provided an adequate amount of ductility before failure of the cross section occurred. The 10 in wide plate was chosen for economy and was used in further design procedures.

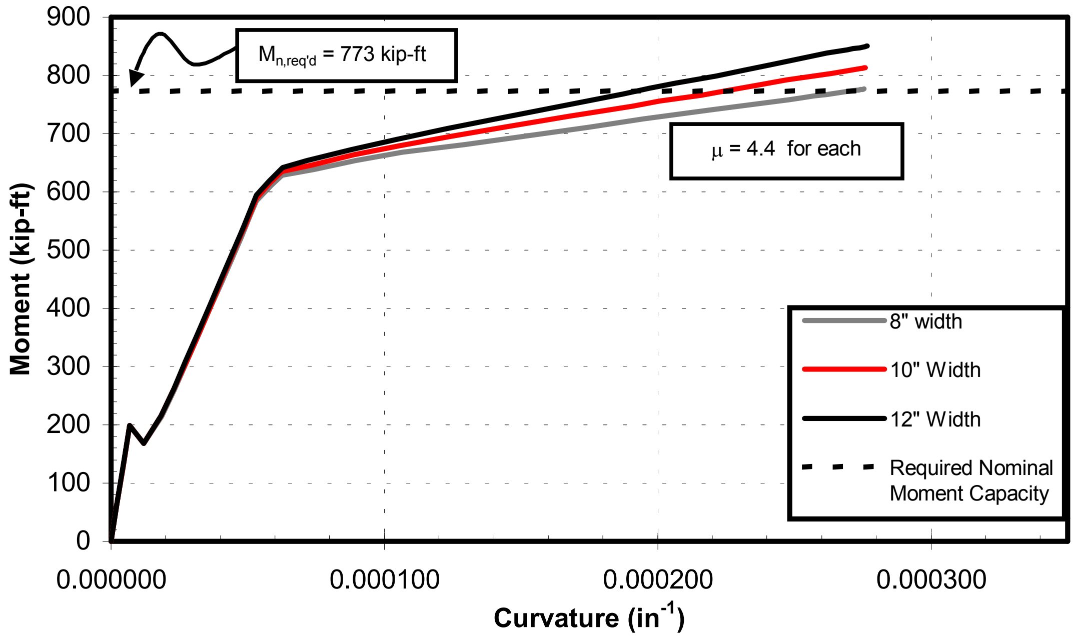

Figure 4.1 Moment vs. Curvature for Exterior Girder Cross Section at 186"

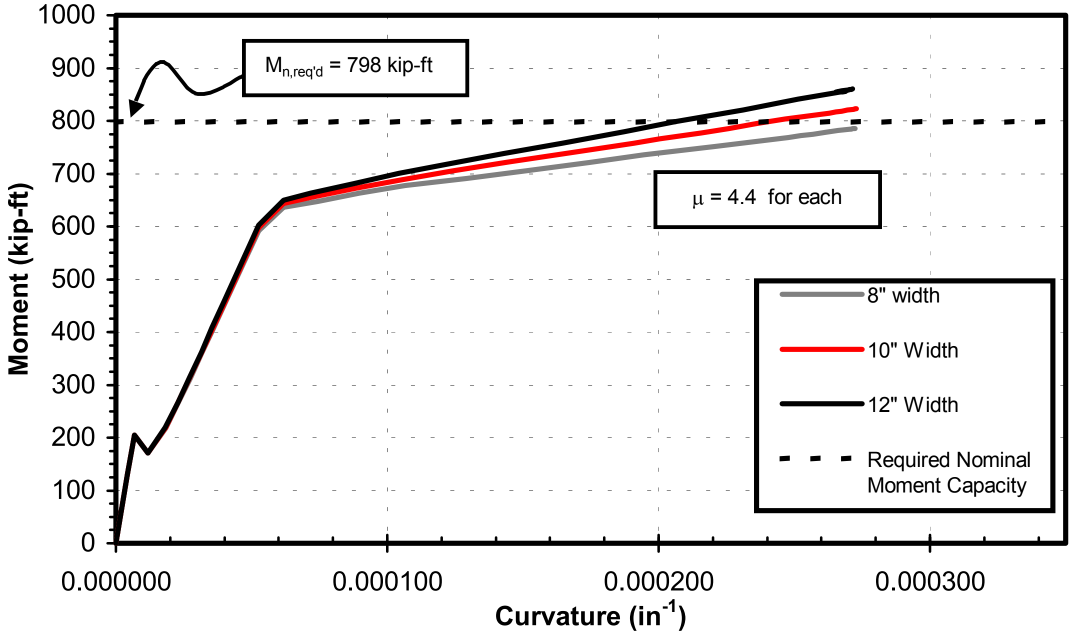

Figure 4.2 Moment vs. Curvature for Exterior Girder Cross Section at 225"

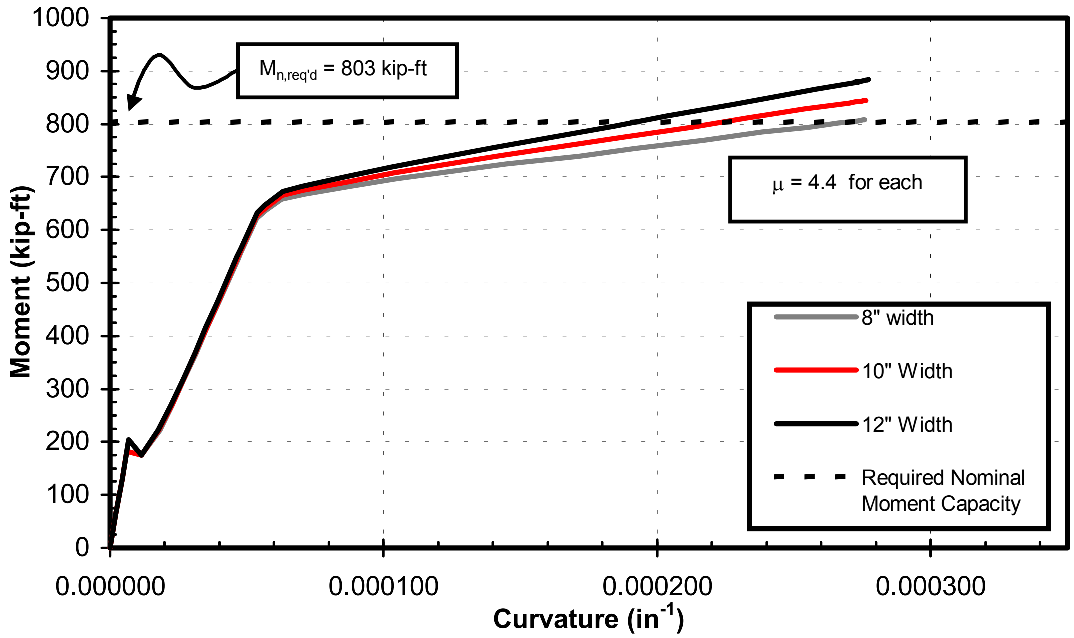

Figure 4.3 Moment vs. Curvature for Exterior Girder Cross Section at 966"

A design of the 225 in critical section was done with the Fibercote composite material. Results of the design with the Fibercote strengthened cross sections are shown in Tables 4.5 and 4.6. Calculation of the nominal moment capacity according to the ACI Committee 440 method revealed that the Fibercote composite material did not provide adequate flexural strengthening for either design thickness used. This verified the results obtained from the ACI Committee 440 design procedure.

The flexural capacity of the Fibercote strengthened cross sections was also calculated for two designs using alternative criteria for the allowable strain in the FRP. One of the designs used a constant value of $\kappa_M = 0.75$ while the other used a maximum allowable strain of 0.007 in the FRP composite. An upper limit on the strain in the FRP of 0.007 was recommended by Breña for a simply supported RC member strengthened with FRP and transverse FRP straps. Breña made no recommendations for limiting FRP strains to be used in design for continuous members or for members strengthened without the use of transverse FRP straps. Use of Breña's design criteria for an RC member other than that used in his research is not recommended. The use of Breña's limiting strain criteria is only done in this report as a comparison to the ACI Committee 440 criteria.

Results of the alternative design methods are in Tables 4.5 and 4.6. For a composite plate thickness of 0.10 in, both alternative design methods resulted in an adequately strengthened cross section. Although these designs made the use of the Fibercote material more promising, it was eliminated from consideration because it failed to meet design requirements made by the ACI Committee 440 draft report.

	ACI Committee 440 Method	κ_M = 0.75	Breña
b_f (in)	14.00	14.00	14.00
M_n (kip-ft)	769	816	795
$M_{n,req'd}$ (kip-ft)	798	798	798
ε_f	0.005315	0.00699	0.00628
μ	3.4	4.4	4.0

Table 4.5 Computer Spreadsheet Design Results for the 0.05" Thick Fibercote Reinforced Exterior Girder at 225"

	ACI Committee 440 Method	κ_M = 0.75	Breña
b_f (in)	14.00	14.00	14.00
M_n (kip-ft)	739	992	955
$M_{n,req'd}$ (kip-ft)	798	798	798
ε_f	0.002294	0.00699	0.00627
μ	1.6	4.3	3.9

Table 4.6 Computer Spreadsheet Design Results for the 0.10" Thick Fibercote Reinforced Exterior Girder at 225"

4.2.2 Interior Girder

Design of the interior bridge girders was done with the Tyfo UC composite material. The results of the computer spreadsheet design revealed that an 8 in wide plate would provide adequate flexural strength and ductility at the critical sections used for design. The nominal moment capacities calculated by the ACI Committee 440 method were 1.5–10 percent larger than the required moment capacity. When moment redistribution in the strengthened girders was considered, the design moment capacity of the girders strengthened with the 8 in wide Tyfo UC composite was adequate at all critical sections. Moment-curvature diagrams for the three critical sections are shown in Figures 4.4, 4.5, and 4.6.

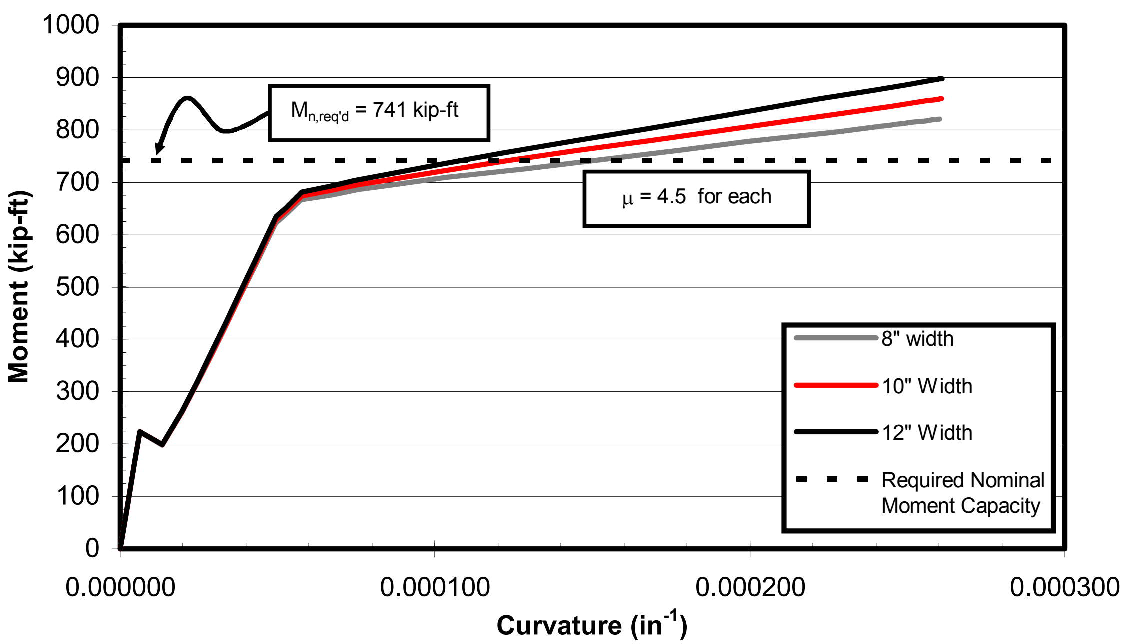

Figure 4.4 Moment vs. Curvature for Interior Girder Cross Section at 186”

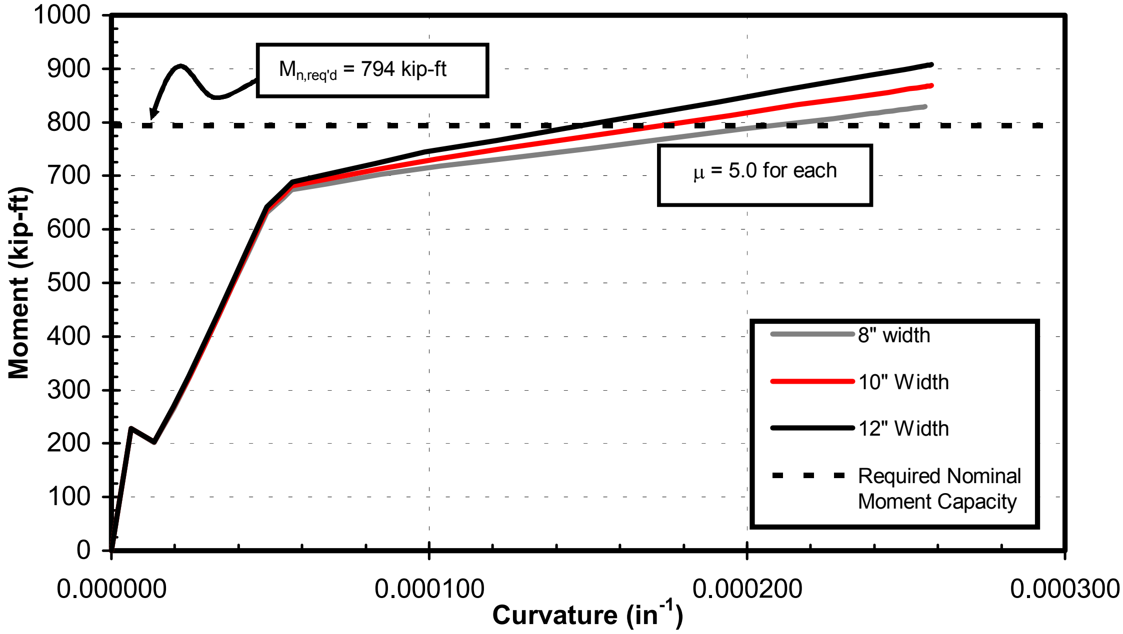

Figure 4.5 Moment vs. Curvature for Interior Girder Cross Section at 225"

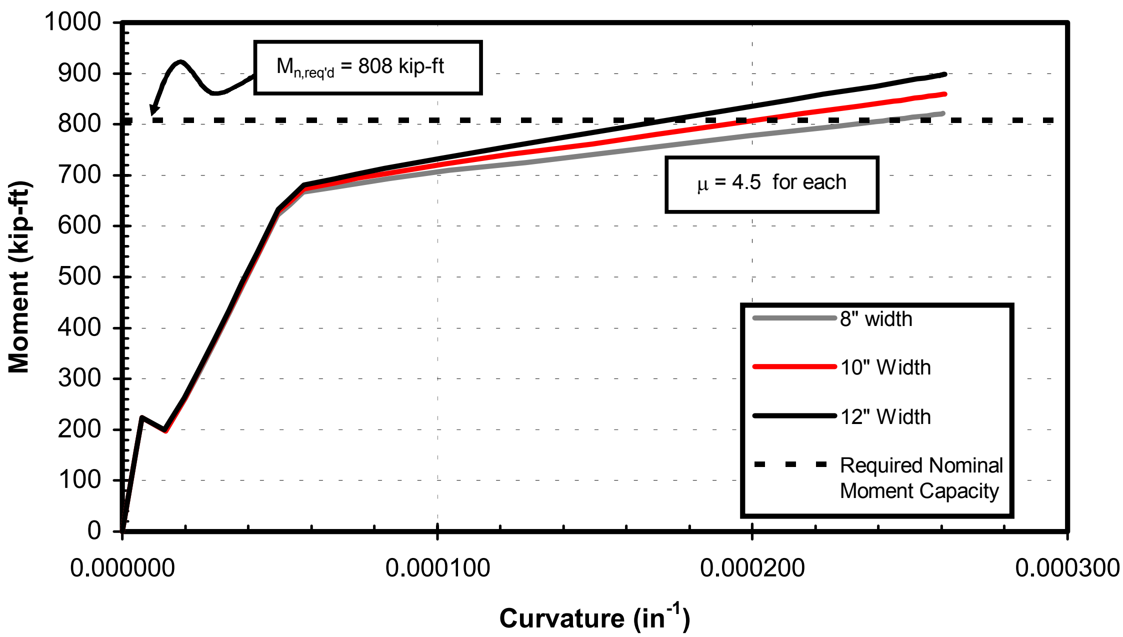

Figure 4.6 Moment vs. Curvature for Interior Girder Cross Section at 966"

4.3 Serviceability Design

Analysis of the bridge girders revealed that the stress in the primary tensile reinforcement was nearly 86 percent of the yield stress under service level loading. Current recommendations state that the stress in steel reinforcement should be less than 80 percent of the steel yield stress under service level loads (ACI Committee 440, 2000). Significant reductions in the stresses in the primary steel reinforcement were thought to be possible if enough external FRP composite was added to the girder.

In order to examine the stresses in the primary reinforcing steel, the amount of Tyfo UC composite material used to strengthen the exterior girder was varied. This procedure was carried out for the critical section at 966 in. Equation 3.20 was used to calculate the stress at the centroid of the primary reinforcing steel for various strengthened cross sections.

Area of FRP (in^2)	f_s (ksi)	% of Yield Stress
0.00	28.3	85.7
0.44	27.1	82.2
0.55	26.9	81.4
0.66	26.6	80.6
0.75	26.4	80.0
1.65	24.6	74.4
1.78	24.3	73.7
2.45	23.2	70.4

Table 4.7 Serviceability Design Results for the Exterior Girder at the 966" Critical Section

Table 4.7 shows the results of the serviceability design for the 966 in critical section of the exterior girder. Stresses at the centroid of the primary steel reinforcement for the unstrengthened cross section were nearly 86 percent of the steel yield stress. Use of the 10 in wide Tyfo UC composite plate (A_f = 0.55 in^2) reduced the stress at the reinforcement centroid to 81.4 percent of the yield stress at service level loading. This was a reduction of about five percent.

In order to reduce the steel stress to the level recommended by ACI Committee 440, a gross area of composite material equal to 0.75 in^2 was required. This would require a 0.055 in thick Tyfo UC composite plate to be 14 in wide, which is nearly as wide as the flat portion of the girder soffit. In order to provide the required gross area of composite material a thicker plate was needed. A 10 in width of the 0.075 in thick Tyfo UC composite would provide an area of 0.75 in^2. Use of the thicker plate resulted in the need for a revised flexural design to be performed.

The computer spreadsheet was used to perform the revised flexural design of the cross section at 966 in. The thicker composite plate resulted in a reduced value of κ_M used in the ACI Committee 440 design procedure. As κ_M decreases, so does the allowable strain in the external FRP composite material. The amount of strain in the composite plate at the ultimate moment capacity of the cross section was reduced by 37 percent. Because the FRP composite material underwent less strain, the force in the FRP composite was smaller at ultimate. The resulting moment capacity of the cross section strengthened with the 0.075 in thick Tyfo UC composite plate was four percent smaller than it had been. This did not satisfy ultimate strength design requirements. Use of the thicker Tyfo UC composite plate also reduced the ductility of the cross section by 37 percent. Based on these results, it was decided that the use of the 0.055 in thick Tyfo UC composite plate was acceptable for serviceability concerns.

4.4 Determination of Composite Plate Length

The length of the Tyfo UC composite used to strengthen the bridge girders was chosen based upon the plots of moment capacity versus moment demand found in Figures 2.1 and 2.2. To satisfy ultimate strength design, the composite plate had to be bonded to the girder at all locations where the moment demand exceeded the moment capacity of the member. Extension of the composite plates beyond the point at which they were no longer needed as tensile reinforcement was done to satisfy the provisions of Chapter 12 of ACI 318-99.

For Span #9 and Span #11, the moment demand exceeded the moment capacity from 149 in to 285 in for the interior girder. Moment demand was larger than moment capacity from 130 in to 294 in for the exterior girder. As stated earlier, the Tyfo UC composite must be bonded between these points to satisfy ultimate strength design. An additional length of composite, equal to the depth of the member, h, was added at both ends of the composite plate. Because the external FRP composite is applied to the girder soffit, the effective depth of the beam was taken as the total depth of the member, h, for the strengthened cross section. This procedure is consistent with the design of flexural steel reinforcement for reinforced concrete beams found in Section 12.10.3 of ACI 318-99.

For Span #10, ultimate strength demand required that the FRP composite be applied from 800 in to 966 in for the interior girder and from 819 in to 966 in for the exterior girder. A length of composite equal to the depth of the member, h, was added at both ends for proper anchorage. Once the total required length of each Tyfo UC composite plate had been calculated, this length was rounded to the nearest foot. This was done to make field measurements of the composite easier and to account for the possibility of misalignment during installation. The locations of the external FRP composite plates are shown in Figures 4.7 and 4.8.

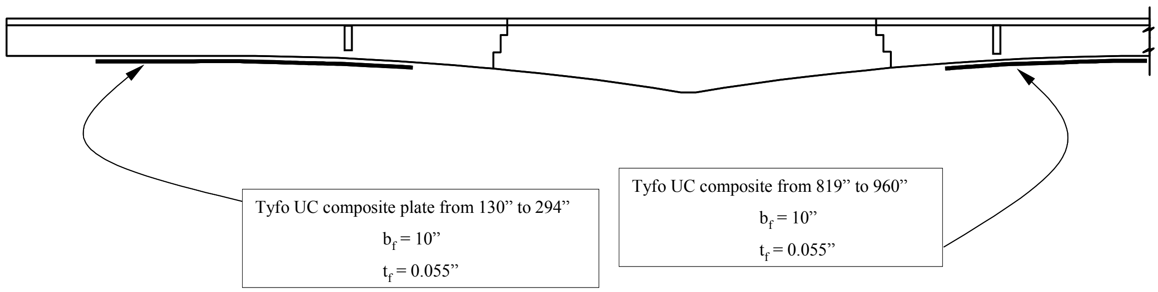

Figure 4.7 Location of External FRP Composite Plates for an Exterior Girder

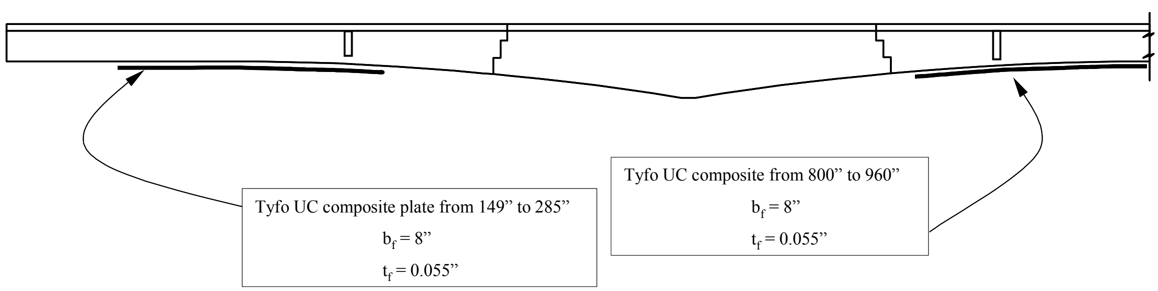

Figure 4.8 Location of External FRP Composite Plates for an Interior Girder

4.5 Anchorage Stresses at the Concrete-FRP Interface

Once the length of the FRP composite plates had been determined, a check of the shear and peeling stresses at the concrete-FRP interface was performed. This procedure was necessary to ensure that the ultimate loads used for design would not cause the FRP composite plates to debond from the concrete surface. Calculation of shear and peeling stresses at the concrete-FRP interface was performed according to the procedure proposed by Tedesco and El-Mihilmy (2001).

The ultimate factored moment at a given cross section, M_o, was obtained from the graphs shown in Figures 2.1 and 2.2. The shear force, V_o, that corresponds to this moment was not known. For simplicity, the ultimate factored shear force at the cross section was used. This procedure produced results that were at least as conservative as those produced by using M_o and V_o as specified by Tedesco and El-Mihilmy (2001).

Table 4.8 shows the results of the shear and peeling stress calculations for the interior and exterior girders. Results show that the biaxial state of stress in the concrete at the concrete-FRP interface was tension-compression at all locations checked. Tensile stresses were smaller than the maximum allowable tensile stress, ϕf_{tu}. Compressive stresses were far less than the concrete compressive strength, f'_c. These results verified that the cut-off locations chosen for the external FRP composite plate were acceptable.

Exterior Girder

X (in)	σ_1 (psi)	σ_2 (psi)	State of Stress	ϕf_{tu}
104	168.5	-132.4	Tension - Compression	306.5
325	164.6	-129.4	Tension - Compression	306.7
788	96.8	-76.1	Tension - Compression	310.0

Interior Girder

X (in)	σ_1 (psi)	σ_2 (psi)	State of Stress	ϕf_{tu}
120	181.2	-142.4	Tension - Compression	305.8
314	164.2	-129.0	Tension - Compression	306.7
770	95.9	-75.4	Tension - Compression	310.1

Table 4.8 Results from the Anchorage Stress Check

4.6 Comparison of Design Procedures

Results from the ACI Committee 440 design procedure were shown to be slightly unconservative when compared to the results from the computer spreadsheet design. The ACI Committee 440 design procedure overestimated the calculated moment capacity of the strengthened girders by one to two percent on average. The difference in calculated moment capacity of the strengthened cross section was due to the way the compressive stresses in the concrete were modeled.

The moment capacities of the strengthened cross sections calculated by the modified ACI Committee 440 design procedure (variation II) were conservative when compared to the computer spreadsheet design results. Moment capacities calculated by variation II of the ACI Committee 440 design procedure were up to one percent smaller than those calculated by the computer spreadsheet. As was the case with variation I of the ACI Committee 440 design procedure, the difference in calculated moment capacity was a result of the modeling method used for the compressive concrete stresses. Using a linear distribution of compressive stresses in the concrete at failure will always result in a conservative estimate of the amount of external FRP required for flexural strengthening. Comparison to a design that incorporates the nonlinear behavior of concrete may be done to ensure that the design is not overly conservative.

Chapter 5: Conclusions

The War Memorial Bridge is a multispan, continuous reinforced concrete bridge located in Macon County, Alabama. The live loads due to truck traffic have increased significantly since the design and construction of this bridge in 1945. The large live loads caused by the tri-axle dump truck and the concrete truck require that the bridge be load posted.

The Alabama Department of Transportation (ALDOT) wanted to strengthen the War Memorial Bridge so that the load postings could be removed. Removal of the load postings required that the bridge girders be strengthened in such a way that they met the requirements of ultimate strength design. Analysis of the structure revealed that only the positive moment capacity of the bridge girders required strengthening. The required additional positive moment capacity was provided by bonding external FRP composites to the girders.

5.1 Design Methods

A flexural design of the strengthened bridge girders was performed using the procedure outlined in Chapter Nine of *Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures* (ACI Committee 440, 2000). A computer spreadsheet was created to perform the required calculations in the procedure outlined in the ACI Committee 440 draft report. An alternative flexural design of the members was also done using a similar procedure. The alternative design procedure used a linear distribution of compressive concrete stresses at failure.

A supplemental flexural design procedure was performed with a computer spreadsheet program that accounted for the nonlinear stress-strain behavior of concrete under compressive loading. The computer spreadsheet program used the same equations as the ACI Committee 440 procedure for calculation of the maximum allowable stress in the composite and the moment capacity of the strengthened cross section. The results of the designs performed with the computer spreadsheet program were compared with the results obtained from the ACI Committee 440 procedure. The design procedure recommended in Chapter Nine of the ACI Committee 440 draft report was shown to be slightly unconservative.

The computer spreadsheet program was also used to perform additional flexural designs that used alternative criteria for limiting the strain in the FRP for design. These designs were done for the 225 in critical section of the exterior girder and the Fibercote composite material. Results of the designs based on the alternative criteria were compared to those obtained for the ACI Committee 440 procedure. Use of the Fibercote composite material was eliminated as a design possibility because of poor results from all flexural design procedures.

Serviceability of the bridge girders was another concern during design of the FRP strengthened RC members. An analysis was performed to determine how much external

FRP composite was necessary to reduce the stresses in the primary steel reinforcement to an acceptable level under service level live and dead loads.

The length of the external FRP composite plates was determined by considering both ultimate strength design and proper anchorage of the composite plates. Provisions regarding termination of flexural reinforcement from Chapter 12 of ACI 318-99 were used to determine how far the composite plates should be extended along the spans. Anchorage strength of the composite plates was then checked. The method proposed by Tedesco and El-Mihilmy (2001) was used to calculate the anchorage stresses in the concrete and external FRP composite at the concrete/FRP interface.

5.2 Results of the Design Procedures

Two external FRP composites were considered for strengthening the girders of the War Memorial Bridge. The two composite materials, the Tyfo UC composite and the Fibercote composite, had different material properties and geometries. After performing the flexural design procedures outlined in Chapter Three, the Fibercote composite material was eliminated as a choice for flexural strengthening. All subsequent design procedures were performed only for the Tyfo UC composite.

Results of the flexural design procedures revealed that the interior and exterior girders required different amounts of the Tyfo UC composite for flexural strengthening. Exterior girders were strengthened with a 10 in plate width on all spans. Interior girders were strengthened with an 8 in plate width. The length of the composite plate was determined from ultimate strength design requirements and from guidelines given in Chapter 12 of ACI 318-99.

A bonded length of composite equal to 240 inches was required for the exterior girders in Span #9 and Span #11, from 96 in to 336 in (measured from the end of the continuous beam). The exterior girders in Span #10 had composite plates extended 180 inches from midspan in both directions (from 786 in to 1146 in). Interior girders in the outer span required the composite plates to be bonded for 204 inches, from 116 in to 320 in (measured from the end of the continuous beam). The composite plates applied to the interior girders of the middle span were bonded for a total length of 408 inches (from 762 in to 1172 in).

An analysis of the anchorage stresses in the FRP composite and the concrete at the concrete/FRP interface was performed using the method proposed by Tedesco and El-Mihilmy (2001). Results showed that shear and peeling stresses were far below the allowable limits. These results show that the locations chosen as points of termination for the FRP composite were acceptable.

5.3 Recommendations

From the results obtained from the flexural design of the FRP strengthened bridge girders, some conclusions were drawn on the accuracy of the design methods used. The flexural design procedure recommended by ACI Committee 440 produced unconservative designs. Calculated moment capacities were up to two percent larger than those calculated by the computer spreadsheet program. This is a result of the possibly inaccurate assumption that the Whitney stress block is an accurate representation of the concrete stress distribution at failure.

An alternative design procedure that modeled the compressive stresses in the concrete at failure with a linear distribution was developed. This design procedure produced conservative results for all design cross sections considered. Use of the alternative model of the compressive concrete stresses requires some minor changes to be made to the equations given in Chapter Nine of the ACI Committee 440 draft report. The use of this procedure may also result in a design that is overly conservative.

In order for the most efficient design of the strengthened member to be made, the stress in the concrete at failure should be modeled as accurately as possible. The computer spreadsheet design accounted for the nonlinear stress-strain behavior of concrete under compressive loading. This design gave the most precise results for the FRP strengthened RC members. The moment capacities calculated with this design were about one percent larger than those calculated using a linear distribution of concrete stress. The use of a flexural design procedure that accounts for the nonlinear behavior of concrete is recommended for the design of FRP-strengthened RC members.

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Appendix

Notation

a = Effective depth of the rectangular (Whitney) compressive stress block (in)

A_f = Area of FRP reinforcement (in^2)

A_s = Area of steel reinforcement (in^2)

b = Effective width of concrete compression block (in)

b_f = Width of FRP composite plate (in)

c = Distance from the extreme concrete compressive fiber to the neutral axis (in)

C_C = Compressive force in the concrete (lbs)

C_E = Environmental reduction factor for FRP composite material

d = Distance from the extreme top concrete fiber to the primary steel reinforcement (in)

d' = Distance from the extreme top concrete fiber to the secondary steel reinforcement (in)

d_f = Depth from the extreme top concrete fiber to the external FRP composite (in)

E_a = Modulus of elasticity for the epoxy adhesive (psi)

E_c = Modulus of elasticity of the concrete (psi)

E_f = Tensile modulus of elasticity of the FRP composite (psi)

E_s = Modulus of elasticity for the steel reinforcement (psi)

f'_c = Compressive strength of concrete (psi)

f_{fe} = Stress in the FRP composite (psi)

f_{fu} = Ultimate rupture stress of the FRP composite (psi)

f_s = Stress in the primary steel reinforcement (psi)

f_s' = Stress in the primary steel reinforcement (psi)

f_t = Tensile strength of the concrete, as determined by a cylinder splitting test (psi)

f_{tu} = Stress limit used in the anchorage design (psi)

f_y = Yield stress of the steel reinforcement (psi)

F_{fe} = Tension force in the FRP composite (lbs)

F_s = Tension force in the steel reinforcement (lbs)

F_s' = Tension force in the steel reinforcement (lbs)

h = Total depth of the concrete cross-section (in)

I_{cr} = Moment of inertia for the unstrengthened cracked cross section (in^4)

I'_{cr} = Moment of inertia for the strengthened cracked cross section (in^4)

L = Span length (in)

L_f = Distance from the composite plate curtailment to the support (in)

M_n = Nominal moment capacity of the cross section (in-lbs)

M_o = Ultimate factored moment at the cross section where shear and peeling stresses in the composite plate are calculated (lbs)

M_u = Factored ultimate moment, based on factored dead and live loads (in-lbs)

n = Number of plies of FRP composite used for strengthening

n_s = Modular ratio of steel to concrete

t_f = Thickness of one FRP composite ply (in)

V_o = Shear force that corresponds to loading causing M_o used in the calculation of shear and peeling stresses (lbs)

α_f = A function of the mechanical properties of the adhesive and composite plate used in the calculation of shear and peeling stresses ($in^{0.5}$)

β_1 = Ratio of the depth of the Whitney stress block to the depth of the neutral axis (in)

ε_{bi} = Strain developed in the concrete surface on the bottom of the member at the time of FRP installation (in/in)

ε_c = Compressive strain in the concrete (in/in)

ε_{cu} = Maximum compressive strain in the concrete (in/in)

ε_{fe} = Effective strain level in the FRP composite (in/in)

ε_{fu} = Manufacturer specified rupture strain of FRP composite (in/in)

ε_s = Strain in the primary steel reinforcement (in/in)

ε_s' = Strain in the secondary steel reinforcement (in/in)

ε_y = Strain corresponding to the yield point of the reinforcing steel (in/in)

ϕ = Strength reduction factor

κ_M = Strain reduction factor for the FRP composite

μ = Sectional ductility ratio

σ_x = Elastic normal stress in the composite plate (psi)

$\sigma_{z,MAX}$ = Maximum out-of-plane normal stress in the composite plate (psi)

$\sigma_{1,2}$ = Principal stresses in the concrete at the concrete-FRP interface (psi)

τ = Elastic shear stress in the composite plate (psi)

τ_{MAX} = Maximum in-plane shear stress in the composite plate (psi)

Ψ = Strength reduction factor for the FRP composite

Ψ_f = A calibration factor for the moment, M_o, used when calculating σ_x

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