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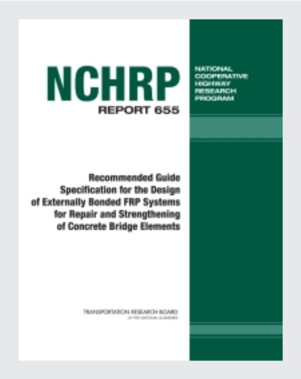

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ISBN 978-0-309-15485-7 | DOI 10.17226/14401

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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

NCHRP REPORT 655

Recommended Guide Specification for the Design of Externally Bonded FRP Systems for Repair and Strengthening of Concrete Bridge Elements

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Subscriber Categories
Bridges and Other Structures

Research sponsored by the American Association of State Highway and Transportation Officials
in cooperation with the Federal Highway Administration

TRANSPORTATION RESEARCH BOARD

WASHINGTON, D.C.
2010
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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Systematic, well-designed research provides the most effective approach to the solution of many problems facing highway administrators and engineers. Often, highway problems are of local interest and can best be studied by highway departments individually or in cooperation with their state universities and others. However, the accelerating growth of highway transportation develops increasingly complex problems of wide interest to highway authorities. These problems are best studied through a coordinated program of cooperative research.

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The needs for highway research are many, and the National Cooperative Highway Research Program can make significant contributions to the solution of highway transportation problems of mutual concern to many responsible groups. The program, however, is intended to complement rather than to substitute for or duplicate other highway research programs.

NCHRP REPORT 655

Project 10-73
ISSN 0077-5614
ISBN 978-0-309-15485-7
Library of Congress Control Number 2010930888

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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

are available from:

Transportation Research Board
Business Office
500 Fifth Street, NW
Washington, DC 20001

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http://www.national-academies.org/trb/bookstore

Printed in the United States of America

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FOREWORD

By Amir N. Hanna
Staff Officer
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This report presents a recommended guide specification for the design of externally bonded Fiber-Reinforced Polymer (FRP) systems for the repair and strengthening of concrete bridge elements. This guide specification addresses the design requirements for members subjected to different loading conditions (e.g., flexure, shear and torsion, and combined axial force and flexure). The guide specification is supplemented by design examples to illustrate its use for different FRP strengthening applications. The guide specification is presented in AASHTO LRFD format to facilitate use and incorporation into the *AASHTO LRFD Bridge Design Specification*. The material contained in the report should be of immediate interest to state engineers and others involved in the strengthening and repair of concrete structures using FRP composites.

Use of externally bonded FRP systems for the repair and strengthening of reinforced and prestressed concrete bridge structures has become accepted practice by some state highway agencies because of their technical and economic benefits. Such FRP systems are lightweight, exhibit high tensile strength, and are easy to install; these features facilitate handling and help expedite repair or construction, enhance long-term performance, and result in cost savings. In addition, research has shown that external bonding of FRP composites improves flexural behavior of concrete members and increases the capacity of concrete bents and columns.

In spite of their potential benefits, use of externally bonded FRP systems is hampered by the lack of nationally accepted design specifications for bridges. Thus, research was needed to review available information and develop a recommended guide specification for such repair and strengthening systems.

Under NCHRP Project 10-73, "Guide Specification for the Design of Externally Bonded FRP Systems for Repair and Strengthening of Concrete Bridge Elements," Georgia Institute of Technology conducted a review of relevant domestic and international information, identified and categorized the items necessary for developing a guide specification, and developed a reliability-based guide specification that employs Load and Resistance Factor Design (LRFD) methodology. The guide specification is accompanied by commentaries that are necessary for explaining the background, applicability, and limitations of the respective provisions. In addition, design examples are provided to illustrate use of the recommended guide specification for different strengthening requirements.

The recommended guide specification will be particularly useful to highway agencies because it will facilitate consideration of FRP systems among the options available for the repair and strengthening of concrete bridge elements and help select options that are expected to yield economic and other benefits. The incorporation of the recommended guide specification into the *AASHTO LRFD Bridge Design Specifications* will provide an easy access to the information needed for the design of externally bonded FRP systems for the repair and strengthening of concrete bridge elements.

CONTENTS

SUMMARY

Recommended Guide Specification for the Design of Externally Bonded FRP Systems for Repair and Strengthening of Concrete Bridge Elements

Fiber-reinforced polymer (FRP) composites now are being used to strengthen or to upgrade the load-carrying capacity of a wide range of bridge structures. These materials must offer technical and economical advantages in order to be successful in the highly competitive construction marketplace. While FRP composites increasingly are being used in combination with traditional construction materials for rehabilitation of existing structures, codes, and standards for structural condition assessment, evaluation and rehabilitation of bridge structures using composite materials do not exist as of yet. Design information for most FRP composite materials has been developed mainly by the composites industry.

This report summarizes the research conducted in NCHRP Project 10-73 to develop a recommended guide specification for the design of externally bonded FRP composite systems for repair and strengthening of reinforced and prestressed concrete highway bridge elements. This information will facilitate the use of FRP materials in strengthening reinforced concrete and prestressed bridge elements by providing bridge engineers with a rational basis for such use. The research produced a recommended *Guide Specification for the Design of Bonded FRP Reinforcement Systems for Repair and Strengthening of Concrete Bridge Elements.* This *Guide Specification* is presented in a format resembling that of the *AASHTO LRFD Bridge Design Specifications, 4th Edition (2007)* in order to facilitate their consideration and adoption by the AASHTO. The recommended *Guide Specification* is accompanied by commentaries that explain the background, applicability, and limitations of the provisions contained therein. Included also are step-by-step calculations in accordance with the recommended *Guide Specification* for six examples of commonly used FRP strengthening applications. These examples would serve as a tutorial on how to approach bridge strengthening projects in practice.

The report concludes with suggestions for implementing the guide specification and recommendations for further research to support future revisions and enhancements of the recommended guide specification.

CHAPTER 1

Introduction and Research Approach

1.1 Background

Because of the technical and economic benefits achieved by the use of externally bonded fiber-reinforced polymer (FRP) systems for the repair and strengthening of reinforced and prestressed concrete bridge structures, this method of rehabilitation of bridge structures has become accepted practice in many state highway agencies. Such FRP systems are lightweight, exhibit high tensile strength, and are easy to install; these features facilitate handling and help expedite repair or construction, enhance long-term performance, and result in cost savings. In addition, the external bonding of FRP composites improves flexural behavior of concrete members and increases the capacity of concrete bents and columns.

In spite of their potential benefits, the use of externally bonded FRP systems is hampered by the lack of nationally accepted design specifications for their use in the repair and strengthening of concrete bridge elements. NCHRP Project 10-73 was initiated to review available information and to develop a recommended guide specification for the design of externally bonded FRP systems. This specification will help highway agencies consider FRP systems among the options for the repair and strengthening of concrete bridge elements and select options that are expected to yield economic and other benefits.

1.2 Project Objective and Scope

The objective of this project was to develop a recommended guide specification for the design of externally bonded FRP composite systems for their use in the repair and strengthening of reinforced and prestressed concrete highway bridge elements. FRP composite systems covered in this project include thermoset polymers reinforced by carbon, glass, or aramid fibers. To achieve this project objective, the following tasks were carried out:

Task 1. Information relevant to the design of FRP systems used in repair and strengthening of concrete bridge elements was collected and reviewed. This information was assembled from published and unpublished reports, contacts with transportation agencies and industry organizations, and other domestic and foreign sources, including the American Concrete Institute "Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures" (ACI 2002) and similar publications.

Task 2. Based on the information gathered in Task 1, the items necessary for developing the guide specification were identified and categorized. These items addressed flexure, shear, axial loading, development length, detailing, and other design considerations in a manner similar to that provided in the *AASHTO LRFD Bridge Design Specifications.*

Task 3. Based on the information obtained in Tasks 1 and 2, a tentative outline of the proposed guide specification and a work plan for developing the specification along with a commentary and design examples were prepared. The plan described the proposed approach for incorporating appropriate resistance factors and other design criteria in the specification.

Task 4. The plan for developing the guide specification was executed. Based on the results of this work, the guide specification was developed.

Task 5. Using the specification developed in Task 4, a commentary and design examples to illustrate use of the specification were prepared.

Task 6. A final report that documents the entire research, including the recommended guide specification and commentary and the design examples was prepared.

1.3 Applicability of Results to Highway Practice

The research products resulting from this project provide a technically sound and documented basis for using FRP reinforcement in bridge rehabilitation and retrofit. The use of FRP reinforcement will have a significant impact on the economics of bridge maintenance and rehabilitation at state and national levels, and may permit them to upgrade the load-carrying capacity of bridge members through easy-to-install retrofits rather than replacement. The recommended guide specifications gives FRP manufacturers a consistent basis for reporting material properties while at the same time allows bridge design and maintenance engineers to use such material property data for conditions similar to those under which these properties are obtained. The guide specifications and commentary presented in Attachment A are formatted to facilitate consideration and adoption by AASHTO.

1.4 Report Organization

This report consists of four chapters and two attachments. Chapter 1 describes the objective and outlines the various tasks performed to accomplish the objective. Chapter 2 presents the findings of this study, and Chapter 3 addresses the analytical formulations and the experimental data that formed the basis upon which the proposed Guide Specifications were developed. Chapter 4 presents the conclusions and recommendations for further research. Attachment A presents recommended guide specifications and commentaries for the design of externally bonded FRP reinforcement systems for the repair and strengthening of concrete bridge elements. Attachment B contains step-by-step illustrative examples that serve as a tutorial on how to approach bridge strengthening projects in practice.

CHAPTER 2

Findings

The major research product of this project is a set of recommended guide specifications. The technical basis for these recommendations is described in this chapter.

2.1 Review of Current Practice

This task consisted of a review of relevant available FRP engineering practice, specifications, design guides, data, and research findings from both national and international sources. A brief review concerning the development of externally bonded plates for strengthening reinforced concrete bridge structures is provided below.

FRP composite materials provide effective and potentially economical solutions for rehabilitating and upgrading existing reinforced and prestressed concrete bridge structures that have suffered deterioration. Whether a bridge has been damaged due to overload or material deterioration or requires strengthening to resist increased future live loads due to traffic, wind or seismic forces, FRPs provide an efficient, cost-effective, and easy-to-construct means to reinforce concrete members. FRP composites may be designed to act as flexural, shear or confinement reinforcement. They may be placed in situ with less disruption of bridge utilization and other functions than is usual when rehabilitation involves the addition of steel reinforcement. (Meir and Betti 1997; Seible et al. 1997; Deaver et al. 2003; Hamelin et al. 2005; Triantafillou 2007)

The concept of using externally bonded FRP reinforcement to strengthen concrete structures was developed as an improvement to the use of externally bonded steel plates. Strengthening with externally bonded steel plates commenced in 1964 in Durban, South Africa, to address the problem of the accidental omission of steel reinforcing bars of a basement beam in an apartment complex (Dussek 1980). This innovative idea was developed further on a variety of construction projects involving bridges, parking garage decks, and office buildings in South Africa and Europe (Fleming and King 1967; Parkinson 1978). A review of research and development related to strengthening with externally bonded steel plate can be found in Eberline et al. (1988).

Realizing some difficulties associated with the handling and construction of the relatively heavy weight steel plates used for strengthening purposes, the Swiss Federal Laboratories for Materials Testing and Research Institute (EMPA) initiated an extensive research investigation in the early 1970s, the result of which suggested that lightweight carbon fiber reinforced composite plates could be used in lieu of heavy steel plates for externally strengthening reinforced concrete structures (Meier 1987; Kaiser 1989; Meier 1992; Meier et al.1993). The EMPA efforts led to the first field implementation of FRP rehabilitation for both bridge and building applications. The Ibach Bridge near Lucerne, Switzerland, and the City Hall of Gossau St. Gall in northeastern Switzerland both were strengthened in 1991 by bonding pultruded carbon fiber polymer plates to the exterior surfaces of the concrete structures. Details on some of these and other early applications are described by Meier et al. (1993).

The EMPA success in using carbon-fiber reinforced polymer composites for externally bonded repair and strengthening of reinforced concrete structures motivated researchers in North America, Europe, Asia, and Australia to further investigate the use of externally bonded FRP materials in structural rehabilitation. The results from these investigations led to a number of recommended design guides and specifications, including the following:

- ACI 440.2R-02 “Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures” (ACI 2002).
- ISIS Canada Design Manuals, 2001, “Strengthening Reinforced Concrete Structures with Externally-Bonded Fiber-Reinforced Polymers,” Winnipeg, Manitoba (ISIS 2001).
- *fib* technical report bulletin 14, “Externally bonded FRP reinforcement for RC structures,” published in Europe (fib 2001).

- CNR-DT 200, “Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Existing Structures,” Italian Advisory Committee on Technical Recommendations for Construction, Rome, Italy (CNR-DT 200 2006).
- Japan Society of Civil Engineers (JSCE), “Recommendations for Upgrading of Concrete Structures with Use of Continuous Fiber Sheets,” (JSCE 2001).
- The French Association of Civil Engineers, Title : Réparation et renforcement des structures en béton au moyen des matériaux composites (AFGC 2003).
- Avis Technique 3/01-345, “Élement de structure renforcés par un procédé collage de fibres de carbone,” Entriprise FREYSSINET, France (Freyssinet 2001).
- German Provisional Regulations, Allgemeine Bauaufsichtliche Zulassung, Nr. Z-36.12-65 vom 29, Deutsches Institut Für Bautechnik, Berlin (German Provisional 2003).
- Polish Standardization Proposal for Design Procedures of FR Strengthening (Gorski and Krzywon 2007)
- Caltrans Bridge Memo to Designers-MTD 20-4 (Caltrans 2007)
- GDOT Specification: Proposed Specifications-Polymeric Composite Materials for Rehabilitating Concrete Structures (Zureick 2002).
- *NCHRP Report 514: Bonded Repair and Retrofit of Concrete Structures Using FRP Composites—Recommended Construction Specifications and Process Control Manual* (Mirmiran et al. 2004).
- *NCHRP Report 609: Recommended Construction Specifications and Process Control Manual for Repair and Retrofit of Concrete Structures Using Bonded FRP Composites* (Mirmiran et al., 2008).

2.2 Development of Proposed Guide Specifications

The information gathered from national and international design guides as well as published and unpublished research reports and archival journal papers germane to the repair and strengthening of concrete structures was assembled. The essential elements of all available design guides were identified, selected, and categorized in a manner consistent with the *AASHTO LRFD Bridge Design Specification, 4th Edition* (AASHTO 2007), yielding the following five sections:

- Section 1: General Provisions,
- Section 2: Material Requirements,
- Section 3: Members under Flexure,
- Section 4: Members under Shear, and
- Section 5: Members under Combined Axial Force and Flexure.

Each section is further divided into subsections. The *Guide Specification* (Attachment A) was also organized into these five sections to facilitate its use by the professional bridge engineering community.

The recommendations contained in the *Guide Specifications* utilize the load combination requirements found in the *AASHTO LRFD Bridge Design Specifications.* The resistance criteria were developed using the same principles of structural reliability analysis on which the *AASHTO LRFD Bridge Design Specification* are based. Structural reliability analysis takes the uncertainties in concrete, steel, and FRP material strengths and stiffnesses into account using rational statistical models of these key engineering parameters. The criteria for checking safety and serviceability of structural members and components that have been strengthened with externally bonded FRP reinforcement are based on a target reliability index, β, of 3.5 (the target value assumed in the development of the *AASHTO LRFD Bridge Design Specification).* The factored resistance and factored loads used in these checks are consistent with those found in customary engineering practice to facilitate their use and to minimize the likelihood of misinterpretation.

2.3 Development of Reliability-Based Guide Specifications

2.3.1 Probability-Based Load and Resistance Factor Design

Design criteria for safety-related limit states in modern probability-based codes based on the notions of load and resistance factor design (LRFD) are represented by the following relationship:

$$\text{Required Strength}\left(Q_d\right) \leq \text{Design Strength}\left(R_d\right) \tag{2.1}$$

in which the required strength is determined from structural analysis using factored loads, and the design strength (or factored resistance) is determined using nominal material strengths and dimensions and partial resistance factors. The load and resistance factors account for uncertainties associated with the inherent randomness of the load and resistance variables, uncertainties arising from the use of approximate models to represent the mechanics of structural behavior, and consequences of failure, i.e., local vs. general or ductile vs. brittle. In the *AASHTO LRFD Bridge Design Specification,* these load and resistance factors are set in such a way that structural members, components, and systems are designed to achieve a target performance goal, which is expressed in terms of a reliability index, $\beta = 3.5$ (at the inventory load level). The design equation has the following form:

$$\sum \eta_i \gamma_i Q_i \leq \phi R_n = R_r \tag{2.2}$$

where

η_i = load modifier, a factor relating to ductility, redundancy, and operational importance;
γ_i = load factor;
Q_i = force effects;
R_n = nominal resistance;
R_r = factored resistance; and
ϕ = Resistance factor defined in the *AASHTO LRFD Bridge Design Specifications.*

2.3.2 Statistical Models for Structural Loads

Structural loads acting on bridges may be classified as permanent and transient in nature. Permanent loads remain on the bridge for an extended period of time and consist of the weight of the structure and permanently attached non-structural components. Transient loads include vehicular traffic, pedestrian loads, and environmental loads such as those caused by wind, ice floes, earthquakes, etc. The relative importance of the different loads in bridge performance depend on numerous factors, including type of construction, length of span, and the nature of the environmental exposure at the bridge site. For short- and medium-span girder bridges, the most important load combination is live load and dead load. Environmental effects are significant for long-span bridges. This project is focused on short- and medium-span bridges with spans ranging from 30 ft to 200 ft. Only dead load, live load (LL), and dynamic load (IL) were considered in the reliability analysis on which the recommendations are based.

Dead Load. Dead load is the weight of structural components and nonstructural attachments permanently connected to the bridge. The following are four components of dead load:

- DL_1 — weight of factory made elements,
- DL_2 — weight of cast in place concrete,
- DL_3 — weight of traffic wearing surface, and
- DL_4 — weight of miscellaneous nonstructural components.

Statistical parameters for each component of dead load have been developed in previous research (Nowak 1999) and are summarized in Table 2.1 The bias factors, λ, define the ratio of the mean to nominal dead load, enabling the statistics of dead load in situ to be determined for a variety of situations once the nominal value is identified from the design documentation. The coefficient of variation (COV), V, is defined as the ratio of standard deviation to mean value and is the fundamental measure of uncertainty in structural reliability analysis.

Table 2.1. Statistical parameters of dead load components.

Dead Load Component	Bias Factor λ	Coefficient of Variation, V
DL_1	1.03	0.08
DL_2	1.05	0.1
DL_3	1.00*	0.25
DL_4	1.05	0.10

*A 35-in. thick wearing surface is assumed.

Live Load. The live load is represented by the forces produced by vehicles moving on the bridge. The statistical models and parameters of live load effects (maximum moments or shears) have been developed previously (Nowak 1999; Eom and Nowak 2001). In these models, the static (truck weight) and dynamic (impact) components of the live load, LL and IL, are considered separately. The statistical parameters of the load effect were estimated based on data obtained from a truck survey (Agarwal and Wolkowicz 1976). The ratio of the mean maximum 75-year shear to AASHTO LRFD HL-93 design shear, λ_{LL}, varies from 1.28 to 1.22 depending on the span length, while coefficient of variation, V, is 0.12 for all spans. In the case of two-lane bridges, it was found that the maximum 75-year live load effect was caused by two trucks side by side (Nowak 1999). Based on numerous field tests (Kim and Nowak 1997; Eom and Nowak 2001), the mean dynamic load factor has been assumed to equal 0.1 with a coefficient of variation of 0.8.

Combination of Dead and Live Loads. This load combination consists of the three components of dead load, static live load, and dynamic load:

$$Q = DL_1 + DL_2 + DL_3 + LL + IL \tag{2.3}$$

The mean, μ_Q, and variance, σ_Q^2, of Q are:

$$\mu_Q = \mu_{DL1} + \mu_{DL2} + \mu_{DL3} + \mu_{LL+IL} \tag{2.4}$$

$$\sigma_Q^2 = \sigma_{DL1}^2 + \sigma_{DL2}^2 + \sigma_{DL3}^2 + \sigma_{LL+IL}^2 \tag{2.5}$$

where

μ_{DL1} and σ_{DL1} = mean and standard deviation of the dead load due to factory made (precast) elements,
μ_{DL2} and σ_{DL2} = mean and standard deviation of the dead load due to cast in place concrete,
μ_{DL3} and σ_{DL3} = mean and standard deviation of the dead load due to miscellaneous nonstructural components, and
μ_{LL+IL} and σ_{LL+IL} = mean and standard deviation of the live load with impact.

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The mean value, μ_{LL_P+IL}, of the combination of live load, LL, and dynamic load, IL, per girder is calculated as:

$$\mu_{LL\_P+IL} = 1.1 \cdot \lambda_{LL} \cdot \lambda_{GDF} \cdot LL \cdot \lambda_{LL\_P} \quad (2.6)$$

where

LL = the nominal HL-93 live load,
1.1 = the mean dynamic impact,
λ_{GDF} = the bias factor for the girder distribution factor, and
λ_{LL_P} = the live load analysis factor, which is assumed to equal 1.

The coefficient of variation, V_{LL_P}, and standard deviation, σ_{LL_P}, of the static part of the live load are:

$$V_{LL\_P} = \sqrt{V_{LL}^2 + V_P^2} \quad (2.7)$$

$$\sigma_{LL\_P} = V_{LL\_P} \cdot \mu_{LL\_P} \quad (2.8)$$

where V_{LL} is the coefficient of variation of the live load, and V_P is the coefficient of variation of the live load analysis factor equal to 0.12, and μ_{LL_P} is the mean value of the static part of the live load.

The standard deviation, σ_{LL_P+IL}, and the coefficient of variation, V_{LL_P+IL}, for the mean maximum load combination of live load and dynamic load are:

$$\sigma_{LL\_P+IL} = \sqrt{\sigma_{LL\_P}^2 + \sigma_{IL}^2} \quad (2.9)$$

$$V_{LL\_P+IL} = \frac{\sigma_{LL\_P+IL}}{\mu_{LL\_P+1L}} \quad (2.10)$$

2.3.3 Resistance Model

The resistance, R, is modeled as the product of four components:

$$R = R_n MFP \quad (2.11)$$

where

M = the variation of the material parameters,
F = variable reflecting the uncertainties in fabrication,
P = the analysis factor (theoretical model error), and
R_n = nominal resistance.

The mean value of resistance, μ_R, is:

$$\mu_R = R_n \mu_M \mu_F \mu_P \quad (2.12)$$

where

μ_M, μ_F, and μ_P are the mean values for M, F, and P, respectively.

As with the load models, it is convenient to express the resistance, R, in terms of the nominal resistance, R_n, and a bias factor, λ_R. The bias factor, λ_R, and the coefficient of variation, V_R, of the resistance, R, are:

$$\lambda_R = \lambda_M \lambda_F \lambda_P \quad (2.13)$$

$$V_R = \sqrt{(V_m)^2 + (V_F)^2 + (V_P)^2} \quad (2.14)$$

Where

λ_M, λ_F, and λ_P are the bias factors, and V_M, V_F, and V_P are the coefficients of variation of M, F, and P, respectively.

2.3.3.1 Material Strengths

Statistical parameters reflecting the variability due to material and fabrication uncertainties are those of concrete, reinforcing steel rebars, and FRP reinforcement. These statistical data should be representative of values that would be expected in a structure, and should reflect uncertainties due to inherent variability, modeling and prediction, and measurement. There are extensive databases that describe the probabilistic models obtained from previous probability-based code studies (e.g., Galambos, et al. 1982; MacGregor, et al. 1983; Bartlett and MacGregor 1996). These data are summarized in Table 2.2 for concrete and grade 60 reinforcement. The mean compressive strength of concrete reflects the difference between standard-cured and in situ conditions, and includes an allowance for aging.

For FRP reinforcement, the strength depends on the engineering characteristics of the fibers, matrix and adhesive systems and on the workmanship in fabrication and installation. In general, FRP composites used for strengthening reinforced concrete structures are made of aramid, carbon or glass fibers

Table 2.2. Statistical parameters of concrete and reinforcing steel properties.

Material Property	Mean/Nominal	COV	CDF
Rebar yield strength tension	1.12	0.10	Lognormal
Concrete compressive strength			
f_c = 4000 psi	1.00	0.18	Normal
f_c = 6000 psi	1.20	0.15	Normal

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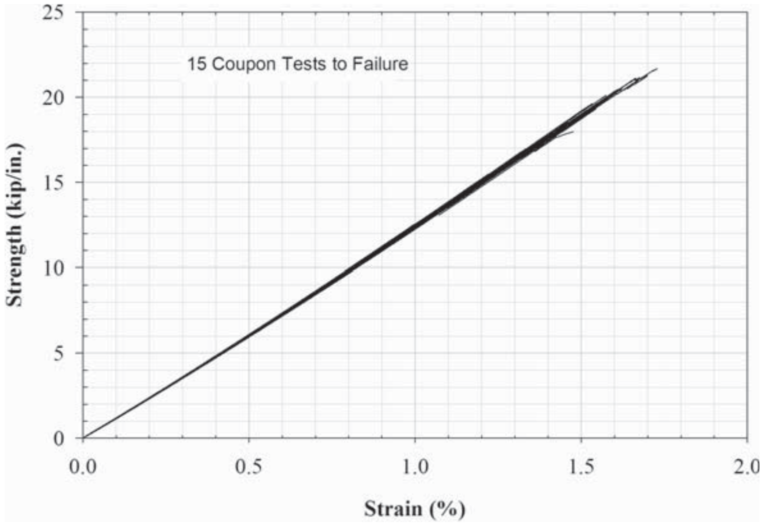

Figure 2.1. Load-strain relation for shop-manufactured composite system.

with a thermoset resin matrix to bind them together. Zureick and Kahn (2001) classified these systems in two groups:

- **Shop-manufactured composites.** Pre-manufactured composites in the form of plates, shells, or other shapes that are bonded in the field to the surface of the concrete member using structural adhesives. These composites are manufactured by a variety of techniques such as the pultrusion, filament winding, and resin transfer molding.
- **Field-manufactured composites.** Fibers in the form of tows or fabrics are impregnated in the field and placed on the surface of the structure requiring strengthening. Methods of impregnation have been done manually (hand lay-up), by a portable impregnator machine, or infusion under vacuum. The composite is bonded to the concrete and then left to cure under ambient or elevated temperature.

An advantage of the shop-manufactured composites over the field-manufactured composites is the ability to control the quality and uniformity in the composite reinforcing systems. Conversely, field-manufactured composites are better able to conform to non-uniform concrete surfaces. Figures 2.1 and 2.2 illustrate the scatter in material data for field-manufactured and shop-manufactured composites, respectively.

In this project, four single-layered and multilayered unidirectional carbon fiber-reinforcement systems evaluated for the strengthening of bridge pier caps in Georgia (Deaver et al. 2003) were examined.

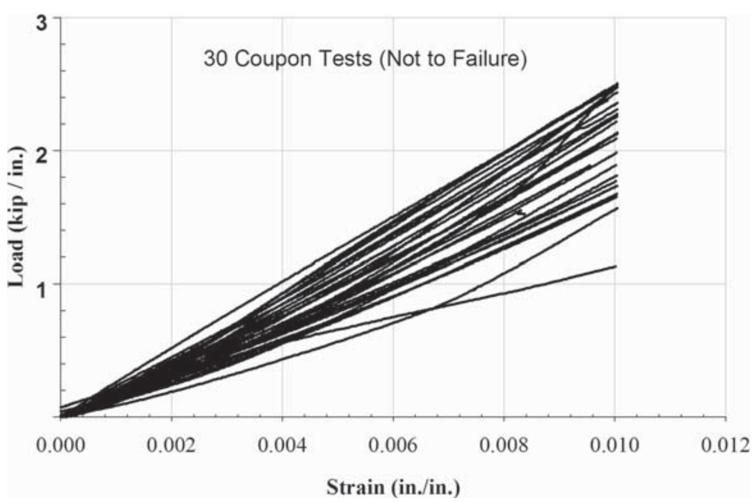

Figure 2.2. Load-strain relation for a field-manufactured composite system (not to failure).

Table 2.3. Statistical data for various FRP reinforcing systems in tension.

	Shop-Manufactured	Field-Manufactured		
	System 1	System 2	System 3	System 4
Sample Size	30	16	25	15
COV	4.3%	24.2	18.2	12.2
Bias	1.06	1.48	1.32	1.19

The strength of FRP in tension is described by a two-parameter Weibull distribution, defined by:

$$F(x) = 1 - e^{-\left(\frac{x}{u}\right)^{\alpha}}, x \geq 0 \tag{2.15}$$

where u and α are parameters of the distribution that can be related to the sample mean and sample coefficient of variation, as described subsequently.

The parameters u and α can be estimated from the sample mean, $\bar{x}$, and sample coefficient of variation, V (Zureick, Bennett, and Ellingwood 2006). As an approximation:

$$u = [1 + (3/8)V]\bar{x} \tag{2.16}$$

$$\alpha = \frac{1.2}{V} \tag{2.17}$$

Statistical data for four systems of FRP reinforcement are summarized in Table 2.3.

2.3.3.2 Modeling (or Analysis) Error

In addition to the uncertainties in resistance that arise from the uncertainties in material strength and fabrication, the statistics of resistance must include the effect of modeling uncertainties. The equations defining the limit states of interest invariably are based on idealizations of structural behavior. For example, in the Bernoulli-Navier hypothesis regarding beam behavior, strain hardening is neglected in steel reinforcement, structural materials are assumed to be homogeneous, etc. These factors are reflected in the mean and coefficient of variation in the parameter, P, in Equation 2.11. These statistical parameters describe the bias and variability that are not explained by the analytical model used to predict resistance. The mean and COV of P are determined by calculating the mean and coefficient of variation in the ratio of test-to-calculated strengths where the calculated strengths are determined from material strengths determined from companion specimen tests and known specimen geometry. When the structural mechanics of a limit state is well-understood and the design equation captures this understanding (beams in flexure usually fall into this category), μ_P normally is close to 1.0 and V_P is approximately 0.05. Conversely, when the mechanics is not well understood and the design equation is based on approximations of behavior (as with reinforced concrete beams in shear), μ_P typically is greater than 1.0 (because the approximate equations normally are selected to be conservative for design purposes) and V_P may range from 0.15 to 0.20 or more, representing a substantial contribution to V_R in Equation 2.14.

2.3.3.3 Resistance

A summary of the resistance statistics for typical reinforced concrete bridge girders without externally bonded FRP reinforcement is presented in Table 2.4, where the components of the statistics of the parameters in Equation 2.11 are also presented. These statistics were determined from previously published assessments of statistics in resistance of reinforced concrete structures (MacGregor et al. 1983; Bartlett and MacGregor 1996; Nowak 1999).

2.4 Reliability Basis for Proposed Resistance Criteria

2.4.1 Selection of Representative Structural Members

Representative bridge members were analyzed for purposes of developing reliability-based resistance factors. These are

1. Members under flexure:
   - Non-prestressed rectangular sections having overall dimensions of 12 in. × 24 in. with a wide range of reinforcement ratios.
   - AASHTO Type III prestressed girder.

Table 2.4. Statistical parameters of resistance.

Type of Structure	*FM*		*P*		*R*	
	λ_{FM}	V_{FM}	λ_P	V_P	λ_R	V_R
Reinforced Concrete						
Moment	1.12	0.12	1.02	0.06	1.14	0.13
Shear with steel	1.13	0.12	1.08	0.10	1.20	0.16
Shear without steel	1.17	0.14	1.20	0.10	1.40	0.17
Prestressed Concrete						
Moment	1.04	0.05	1.01	0.06	1.05	0.08
Shear with steel	1.07	0.10	1.08	0.10	1.15	0.14

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2. Members under shear:
   - Reinforced concrete rectangular sections, having the dimensions of 12 in. × 24 in. and 14 in. × 36 in. that are representative of a wide range of bridge girders.
3. Members under axial and under combined bending and axial loading:
   - A reinforced concrete circular section having a diameter of 36 in.
   - A reinforced concrete square section having the dimensions of 24 in. × 24 in.

2.4.2 Reliability Analysis Procedure

The starting point for developing probability-based design criteria is a description of the limit states of concern by a mathematical model, based on principles of structural mechanics and supported by experimental data. This model, denoted the limit state function, is given by:

$$G(\boldsymbol{X}) = G(X_1, X_2, \cdots X_m) = 0 \tag{2.18}$$

where $\boldsymbol{X} = (X_1, X_2, \cdots X_m)$ = vector of resistance and load variables that, in general, are random. The "failure" event is defined, by convention, such that the limit state is reached when $G(X)<0$. Thus, the limit state probability becomes:

$$P_f = \int_{\Omega} f_X(x_1, x_2, \cdots x_m)\, dx_1\, dx_2 \cdots dx_m \tag{2.19}$$

where $f_X(\mathrm{x})$ = joint probability density function of $\boldsymbol{X}$, and the domain of integration, Ω, is that region of $\mathbf{x}$ where $G(X)< 0$. An alternative measure of safety is the reliability index, β, defined by the relationship $P_f = \Phi(-\beta)$, in which $\Phi(-\beta)$ = standard normal probability distribution evaluated at $-\beta$ (Ellingwood 1994; Melchers 1999). The AASHTO *LRFD Bridge Design Specifications* (2007) were developed using a target reliability index equal to 3.5. Recent advances in computational reliability analysis have made it possible to determine P_f (or β) by Monte Carlo simulation, which facilitates analysis in situations in which the limit state function $g(\boldsymbol{X}) = 0$ cannot be expressed conveniently in closed form. The reliability assessments in subsequent sections of this paper are performed by simulation.

2.4.3 Selection of the Target Reliability Indices

The target reliability benchmarks for bridge structural members and components strengthened with externally bonded FRP reinforcement were selected through a comprehensive evaluation of selected representative bridge elements that were judged to be candidates for repair and/or strengthening. The starting point for this evaluation was the target reliability indices and LRFD criteria in the *AASHTO LRFD Bridge Design Specifications* (2007), as documented in *NCHRP Report 368* (Nowak 1999). For the design of new reinforced concrete or prestressed concrete girder bridges, the target reliability index is 3.5. For load rating existing reinforced concrete and prestressed concrete bridge girders using the LRFD method, the target reliability index, as specified in the *AASHTO Manual for Bridge Evaluation*, *1st Edition* (2008), is 2.5. From this starting point, the special characteristics of FRP materials as strengthening agents required careful consideration. The failure modes in FRP composite materials are brittle in nature; furthermore, as a relatively new rehabilitation technology, there is uncertainty in their performance in aggressive environments over an extended period of time. Thus the overriding considerations for the determination of the target reliability indices were the consequences of failure of a strengthened member and the cost of strengthening (specifically, how much does it cost to increase the reliability index?).

This comprehensive evaluation led to the conclusion that β should be 3.5 (or greater) for inventory loading and 2.5 (or greater) for operating/legal loads. The resistance criteria for strengthening concrete members with FRP reinforcement were developed for those reliability targets.

2.4.4 Development of Resistance Factors

Resistance criteria for structural members that have been strengthened with FRP reinforcement were developed in a form that is consistent with the load factors and bridge performance objectives found in the *LRFD Bridge Design Specifications, 4th Edition* (2007). These requirements have a reliability basis. The similarities of the criteria in the *Guide Specifications* for FRP composite systems with criteria currently used in steel or reinforced concrete bridge design and construction will facilitate use and minimize the likelihood of misinterpretation.

Equations defining the key limit states of flexure, shear, and combined axial force and bending and suitable resistance factors to provide the target reliabilities identified in 2.4.3 were developed. The equations for factored resistance for flexure, shear, and axial compression are:

For flexure:

$$M_r = 0.9\left\lfloor A_s f_s (d_s - k_2 c) + A_s' f_s' (k_2 c - d_s') \right\rfloor + \phi_{frp} T_{frp} (h - k_2 c) \tag{2.20}$$

where

A_s = area of nonprestressed tension reinforcement,
A_s' = area of compression reinforcement (in.^{2}),
f_s = stress in the steel tension reinforcement at development of nominal flexural resistance (ksi),

$f_s' =$ stress in the steel compression reinforcement at development of nominal flexural resistance (ksi),
$c =$ depth of the concrete compression zone (in.),
$d_s =$ distance from extreme compression surface to the centroid of nonprestressed tension reinforcement (in.),
$d_s' =$ distance from extreme compression fiber to the centroid of compression reinforcement.
$h =$ depth of section (in.),
$T_{frp} =$ tension force in the FRP reinforcement (kips),
$\phi_{frp} =$ resistance factor determined from reliability analysis, and
$k_2 =$ multiplier for locating resultant of the compression force in the concrete.

For shear:

$$V_r = \phi\left(V_c + V_s + V_p\right) + \phi_{frp} V_{frp} \tag{2.21}$$

Where

$V_c =$ the nominal shear strength provided by the concrete in accordance with Article 5.8.3.3 of the *AASHTO LRFD Bridge Design Specifications,*
$V_s =$ the nominal shear strength provided by the transverse steel reinforcement in accordance with Article 5.8.3.3 of the *AASHTO LRFD Bridge Design Specifications,*
$V_p =$ component of the effective prestressing force in the direction of applied shear as specified in Article 5.8.3.3 of the *AASHTO LRFD Bridge Design Specifications,*
$V_{frp} =$ the nominal shear strength provided by the externally bonded FRP reinforcement,
$\phi =$ 0.9, and
$\phi_{frp} =$ resistance factor determined from reliability analysis.

For axial compression:

$$P_r = \alpha\phi\left[0.85 f_{cc}'\left(A_g - A_{st}\right) + f_y A_{st}\right] \tag{2.22}$$

where

$\alpha =$ 0.85 for spiral reinforcement and 0.80 for tie reinforcement;
$\phi =$ resistance factor specified in Article 5.5.4.2 of the *AASHTO Bridge Design Specifications, 4th Edition*;
$A_g =$ gross area of section (in.^{2});
$A_{st} =$ total area of longitudinal reinforcement (in.^{2});
$f_y =$ specified yield strength of reinforcement (ksi); and
$f_{cc}' =$ compressive strength of the confined concrete determined according to Article 5.3.2.2.

The starting point for the factored resistance (or design strength) in all cases was the equations that are found in Section 5 of the *AASHTO Bridge Design Specifications* or *ACI Standard 318-05 (ACI 2005)*. The contribution of the FRP reinforcement to factored resistance is added to the existing term(s) for factored resistance. This format accomplishes two objectives: (1) In situations where only light FRP reinforcement is required, the design equation yields a factored resistance that is essentially the same as the factored resistance of the original structural member without FRP reinforcement and (2) Assigning a separate partial factor (ϕ_{frp}) to the FRP contribution facilitates achieving the reliability objectives summarized in Section 2.4.3 throughout a range of material strengths, beam geometries, and spans.

The reliability achieved in LRFD depends on both the nominal resistance and the resistance factor. As a manufactured product, the variability in strength of FRP reinforcement (Equation 2.15 and Table 2.3) must be reflected in the factored resistance. It has been customary in many structural engineering applications to identify the nominal strength with the 0.10-fractile (10 percent exclusion limit) of the strength distribution in the end use condition. Accordingly, the resistance criteria in these *Guide Specifications* are based on the assumption that the nominal ultimate tensile strength of the FRP is defined by the 10th percentile value of the two-parameter Weibull distribution as follows:

$$x_{0.10} = u\left(0.1054\right)^{1/\alpha} \tag{2.23}$$

in which the parameters are estimated from Equations 2.16 and 2.17. To ensure the level of structural performance envisioned in the reliability analysis, the nominal strength stipulated in the construction documents should be the 10th percentile value of strength.

The ϕ_{frp} factors found in Sections 3, 4 and 5 of the *Guide Specifications* were determined by iteration. A series of typical structural members (as identified in Section 2.4.1 above) were designed using the dead and live load requirements in the *AASHTO LRFD Bridge Design Specifications* and a range of tentative ϕ_{frp} factors (rounded to the nearest 0.05), and the reliability indices for these structural members were determined using the procedure in Section 2.4.2. Following the completion of this reliability assessment, the resistance factors proposed for the *Guide Specifications* were those that provided the closest fit to the target reliability index $\beta = 3.5$. This approach was also used in developing the AASHTO LRFD Code (Nowak 1999).

The applicability of the proposed *Guide Specification* is limited to structural members, components, and systems that can be shown to have a minimum strength prior to the application of externally bonded FRP reinforcement. This limit has been imposed to avoid a situation where the behavior of the strengthened member depends unduly on the performance of the FRP reinforcement. If the field application of the reinforcement is deficient or if the strengthened bridge is loaded accidentally beyond the level of the enhanced factored resistance, a sudden and potentially catastrophic failure of the strengthened component is likely to occur.

CHAPTER 3

Interpretation, Appraisal, and Application

3.1 General

This chapter presents the analytical formulations and the experimental data that form the basis upon which the proposed *Guide Specification* is based.

In recent years, there have been numerous publications aimed at demonstrating the effectiveness and benefit of externally bonded reinforcement for strengthening reinforced concrete structural members. However, the vast majority of tests described in these publications reported FRP material properties without sufficient information to enable independent verification of how the data were obtained. In this project only experimental test data, with sufficient documentation to permit their direct comparison with results determined from a technically sound structural analysis were considered in the reliability assessment on which the resistance factors in the proposed *Guide Specification* is based. This approach was followed throughout the entire project and resulted in small data sets.

Experimental data for various limit states governing the behavior of steel-reinforced concrete members with externally bonded FRP reinforcement are summarized in the following sections.

3.2 Flexural Strengthening

Experimental and analytical investigations of the behavior of reinforced concrete beams and slabs in flexure have shown that FRP strengthened reinforced concrete members may exhibit, in most cases, one of the following failure modes (Pelvris et al. 1995; Triantafillou 1998; Triantafillou and Antonopoulos 2000):

1. Crushing of concrete in compression (before or after yielding of the tension steel).
2. FRP reinforcement debonding at flexural crack locations.
3. FRP reinforcement end peeling.

3.2.1 Crushing of Concrete in Compression

Tests conducted on reinforced concrete members strengthened with externally bonded FRP composites yielded results that were consistent with prior tests of non-FRP reinforced beams when crushing of concrete in compression occurred first. The experimental data reported by Saadatmanesh and Ehsani (1991); Spadea et al. (1998); Ross et al. (1999); and Almusallam and Al-Salloum (2001) were examined within the context of customary assumptions associated with the analyses of reinforced concrete flexural members in accordance with the *AASHTO LRFD Bridge Design Specifications.* This examination yielded an average experimental to computed flexural strength ratio of 1.13 and a coefficient of variation of 11%. The experimental to calculated ratios are shown in Figure 3.1.

3.2.2 Debonding of FRP Reinforcement at Flexural Crack Locations

The limit state of FRP plate debonding in steel-reinforced concrete members that also have been reinforced externally with FRP plates occurs when the strain at the concrete/FRP plate interface reaches a limiting value on the order of one-half the ultimate tension strain of the composite materials determined from a standardized direct tension test (e.g., ASTM D3039), as illustrated in Figure 3.2. Laboratory test results (Meier and Kaiser1991; Saadatmanesh and Ehsani 1991; Arduini and Nanni 1997; Spadea et al. 1998; Swamy, et al. 1987; Zureick et al. 2002) have indicated that this limit state is reached when the strain in the FRP reinforcement is between 0.003 and 0.008, as shown in Figure 3.2. In the present analysis, which focuses on flexural strengthening, the strain at the limit state of FRP plate debonding ε_{frp} is set equal to 0.005; lesser values may be appropriate for other limit states involving more brittle failure modes. When the

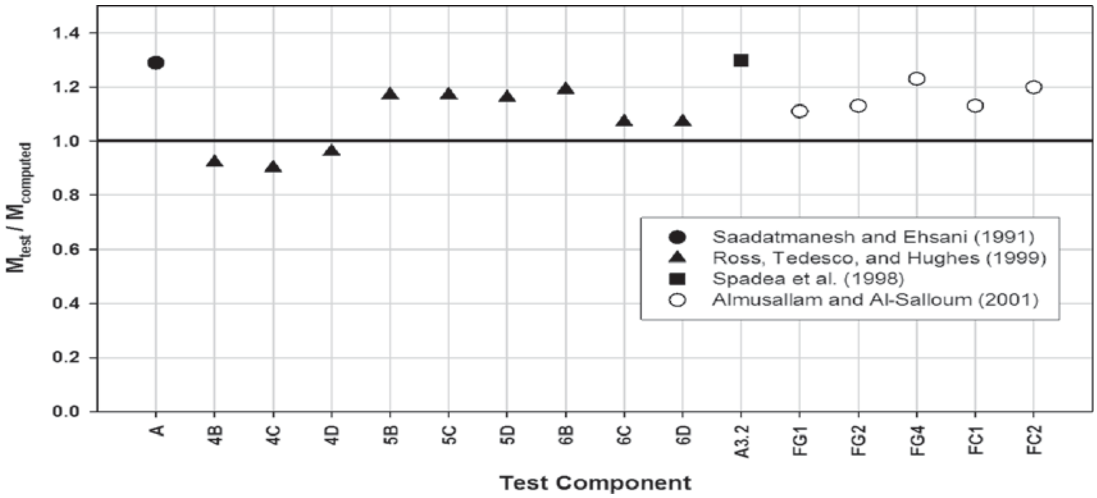

Figure 3.1. Ratios of test to computed flexural strength for crushing of concrete in compression.

strain in the FRP system is as low as that shown in the aforementioned experiments, the maximum compressive strain in the concrete compression zone invariably is below 0.003. This differs from the customary assumption made in flexural analysis of underreinforced concrete beams that the stress in the compression zone can be modeled by a uniform stress equal to $0.85f'_c$, and a more realistic stress distribution, such as that in Figure 3.3, is necessary for calculating the member flexural strength from the linear distribution of strain. For the development of the *Guide Specification* in this project, a nonlinear concrete model (Desayi and Krishnan, 1964; Todeschini, et al, (1964) was adopted. The stress-strain relationship for such a model is defined by the following equations:

$$f_c = \frac{2(0.9f'_c)(\varepsilon_c/\varepsilon_0)}{1+(\varepsilon_c/\varepsilon_0)^2} \tag{3.1}$$

where ε_c is the concrete strain, f'_c is the compression strength of the concrete, and ε_0 is the strain, corresponding to the maximum stress, computed from:

$$\varepsilon_0 = 1.71\frac{f'_c}{E_c} \tag{3.2}$$

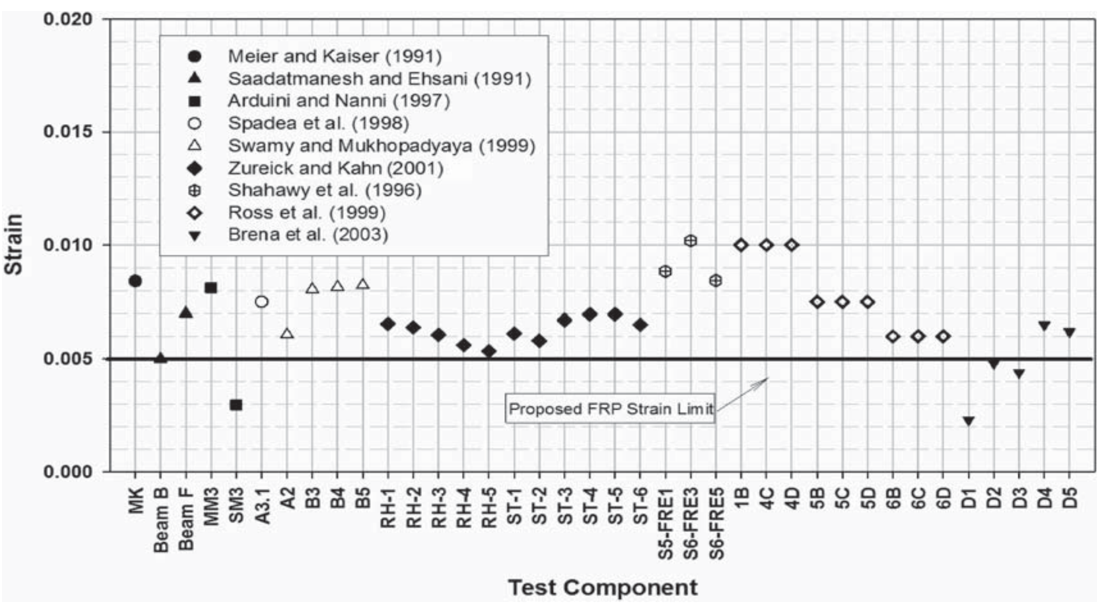

Figure 3.2. Ratios of test to computed flexural strength for the limit state of debonding of FRP reinforcement.

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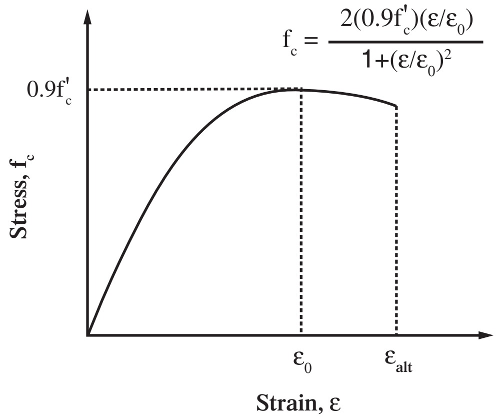

Figure 3.3. Stress-strain relationship for concrete.

where E_c is the modulus of elasticity for normal weight concrete. The compressive force in the concrete is obtained by integrating Equation 3.1. Alternatively, for a constant-width compression zone, the compressive force in the concrete can be approximated by an equivalent rectangular stress block having a depth "c" and an average stress of $\beta_2(0.9f'_c)$, in which β_2 is defined as:

$$\beta_2 = \frac{\ln\left[1+\left(\frac{\varepsilon_c}{\varepsilon_0}\right)^2\right]}{\left(\frac{\varepsilon_c}{\varepsilon_0}\right)} \qquad (3.3)$$

Thus the compression force in the concrete is:

$$C_c = bc\beta_2\left(0.9f'_c\right) \qquad (3.4)$$

The center of gravity of the compression zone is k_2c from the compression outer edge of the concrete section, where k_2 is given in the form:

$$k_2 = 1 - \frac{2\left[\left(\frac{\varepsilon_c}{\varepsilon_0}\right) - \arctan\left(\frac{\varepsilon_c}{\varepsilon_0}\right)\right]}{\beta_1\left(\frac{\varepsilon_c}{\varepsilon_0}\right)^2} \qquad (3.5)$$

The flexural strength of a rectangular beam or slab externally reinforced with an FRP reinforcement system can be determined from the conditions for equilibrium of forces and compatibility of strains on the cross section, as illustrated in Figure 3.4.

3.2.3 FRP Reinforcement End Peeling

The reinforcement end of an externally bonded reinforcement system, when subjected to combined shear and flexure, may separate in the form of debonding in three different modes: (1) critical diagonal crack debonding with concrete cover separation (Yao and Teng 2007) or without concrete cover separation (Oehlers and Seracino 2004); (2) concrete cover separation (Teng et al. 2002); and (3) plate end interfacial debonding (Teng et al. 2002).

Critical diagonal crack debonding may occur where the FRP end is located in a zone of high shear force and the amount of steel shear reinforcement is limited. In such a case a major diagonal shear crack forms and intersects the FRP and then propagates toward the end of the FRP reinforcement. This failure mode is suppressed if the shear strength of the strengthened member remains higher than the flexural strength.

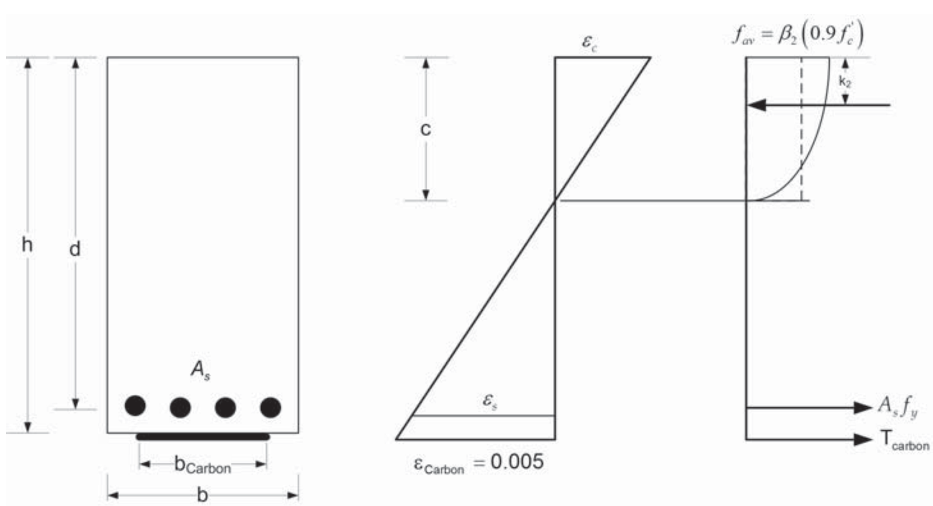

Figure 3.4. Strain and force diagrams for a reinforced concrete rectangular section.

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In beams with heavy steel shear reinforcement, multiple diagonal cracks of smaller widths dominate the behavior such that concrete cover separation will become the controlling debonding failure mode. Failure of the concrete cover is initiated by a crack near the FRP end due to the stress concentration. The crack then propagates to and then along the level of the longitudinal steel tension reinforcement. This mode of failure has been observed in tests on beams with externally bonded steel plates (Jones et al. 1988; Oehlers and Moran 1990) and FRP reinforcement (Malek et al. 1998; Lopez and Naaman 2003; Yao and Teng 2007). Plate-end interfacial debonding is also initiated by high interfacial shear and normal stresses near the end of the FRP that exceed the strength of the weakest element, generally the concrete. Debonding in this case propagates from the end of the FRP towards the middle, near the FRP-concrete interface. This failure mode is only likely to occur when the FRP is significantly narrower than the beam section.

In summary, provided that shear failure (in the form of a diagonal shear crack in the concrete) is suppressed (through shear strengthening, if needed), stress concentrations near the FRP reinforcement end may result in debonding through the concrete layer near the level of the longitudinal steel (or, rarely, near the FRP-concrete interface). A wide range of predictive models that include numerical, fracture mechanics, data-fitting, and strength of material-based methods have been developed to address this complex mode of failure (Yao 2004). However, the equations presented in the proposed *Guide Specification* are based on the approximate analysis of Roberts (1989), because of its simplicity for design purposes. Figure 3.5 shows evaluations of test results conducted on reinforced concrete beams reinforced with externally bonded FRP reinforcement (Yao and Teng 2007) and with externally bonded steel plates (Oehlers and Moran 1990; Swamy et al. 1987). The results of the peeling stress, f_{peel}, predicted by Robert's formula were normalized with respect to the interface shear transfer strength of $\tau_{int} = 0.065\sqrt{f_c'}$, in which f_c', is in ksi. For the vast majority of tests, the prediction is on the safe side.

3.3 Shear Strengthening

Shear strengthening of reinforced concrete members using FRP reinforcement may be provided by bonding the external reinforcement (typically in the form of sheets) with the principal fiber in the direction (insofar as practically possible) of maximum principal tensile stresses to maximize the effectiveness of FRP reinforcement. For the most common case in which the applied loads acting on a structural member are perpendicular to the member axis (e.g., beams under gravity loads or columns under seismic forces), the maximum principal stress trajectories in the shear-critical zones form an angle with the member axis which may be taken roughly equal to 45°. However, it is normally more practical to attach the external FRP reinforcement with the principal fiber direction perpendicular to the member axis.

Experimental and analytical investigations of the behavior of reinforced concrete members strengthened in shear have revealed the following failure modes:

1. Steel yielding followed by FRP debonding.
2. Steel yielding followed by FRP fracture.

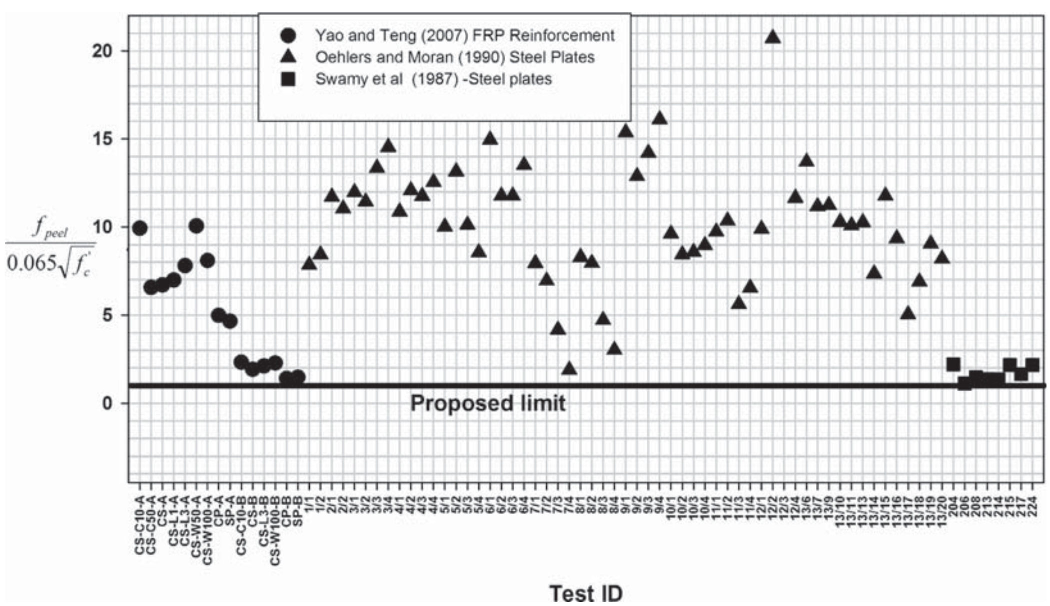

Figure 3.5. Normalized calculated peeling stress for externally bonded FRP reinforcement and for steel plates.

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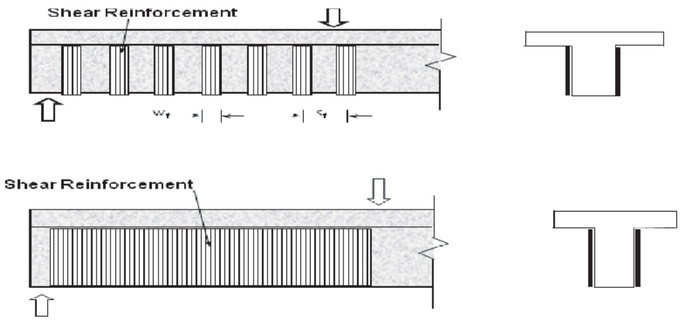

Figure 3.6. Side bonding.

3. FRP debonding before steel yielding.
4. Diagonal concrete crushing.

Depending on the amount of usable steel shear reinforcement in the structural element, FRP debonding can occur either before or after steel yielding. The third failure mode is highly unlikely to occur if proper detailing is provided.

Diagonal concrete crushing in the direction perpendicular to the tension field can be suppressed by limiting the total amount of steel and FRP reinforcement. Fracture of the FRP reinforcement is highly unlikely to occur because the strain when FRP debonds is substantially lower than that corresponding to the FRP fracture strength.

3.3.1 Reinforcing Schemes

Typical FRP strengthening schemes for beams and columns are summarized in the following paragraphs.

Side bonding is the least effective FRP shear reinforcement scheme due to premature debonding under shear loading and should be avoided if possible (Figure 3.6). Side bonding does not allow for the development of the shear-resisting mechanism based on a parallel chord truss model that was first proposed by Ritter (1899), due to the lack of tensile diagonals.

U-jacketing is the most common externally bonded shear strengthening method for reinforced concrete beams and girders (Figure 3.7). This system is prone to premature debonding of the FRP, which may reduce its effectiveness. However, the system is quite popular in practice due to its simplicity.

Jacketing combined with anchorage aims to increase the effectiveness of FRP by anchoring the fibers, preferably in the compression zone (Figure 3.8). Properly designed anchors may result in the fibers reaching their tensile capacity, permitting the jacket to behave as if it were completely wrapped.

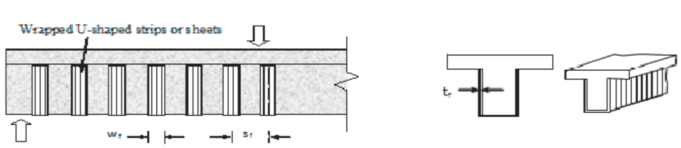

Figure 3.7. U-jacketing.

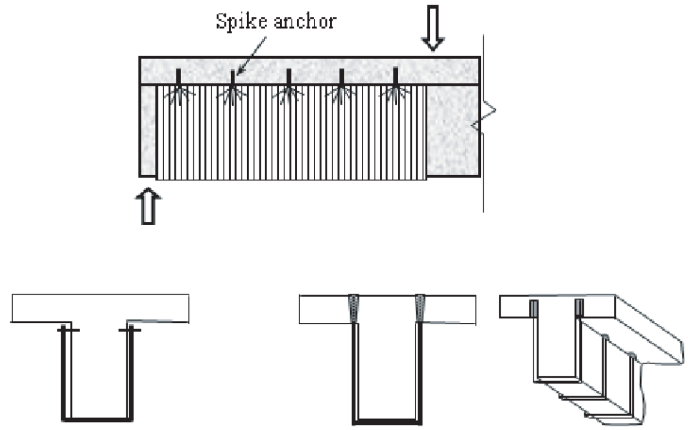

Figure 3.8. Jacketing with anchorages.

Complete wrapping ensures maximum straining of the fibers and is therefore the most desired reinforcing method, if practically possible (Figure 3.9).

Because of the lack of well-documented shear tests in which parameters relevant to the development of these guide specifications have been reported, only data related to U-jackets were examined. A total of 24 reinforced concrete test specimens having sufficient information necessary to calculate their nominal shear strengths were selected from the work of Deniaud and Cheng (2001), Deniaud and Cheng (2003), Taerwe et al. (1997), Norris et al. (1997), Leung et al. (2007), and Pellegrino and Modena (2006). The average value of the ratio of the experimental shear strength to that computed value using the equation proposed in the *Guide Specification* is 1.13 with a coefficient of variation of 28%. The data scatter is shown graphically in Figure 3.10.

3.4 Axially Loaded Members

3.4.1 Axially Loaded Compression Members

The most commonly used method of strengthening or upgrading the load-carrying capacity of reinforced concrete columns with FRP reinforcement is to wrap the reinforcement

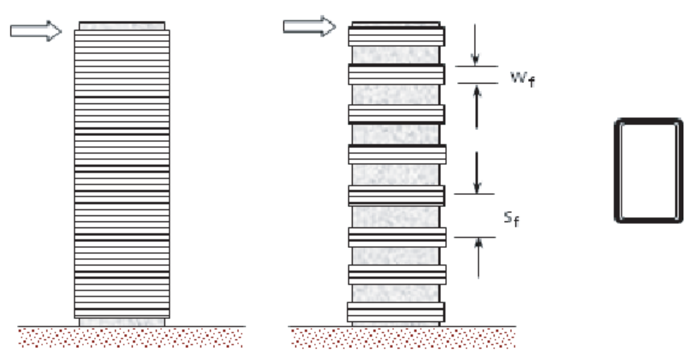

Figure 3.9. Complete wrapping reinforcement.

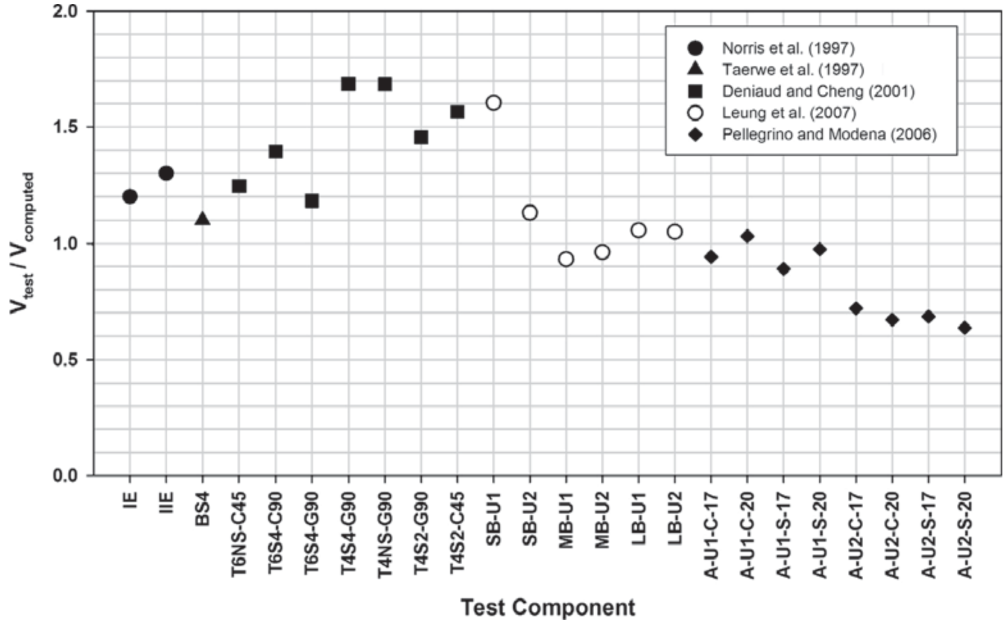

Figure 3.10. Scatter of computed strength of reinforced concrete beams with U-jacket FRP reinforcement.

around the section circumference, thus providing confinement that increases both the axial strength and ductility of the column. A review of available work related to strengthening of axially and eccentrically loaded columns with FRP reinforcement can be found in Teng et al. (2002). The axial compression strength of a column can be determined directly from Article 5.7.4.4 of the *AASHTO LRFD Bridge Design Specifications,* in which the compression strength of unconfined concrete is replaced by the compression strength of the confined concrete. A review of design guidelines for FRP reinforcement confining reinforced concrete columns of non-circular cross sections is found in Rocca et al. (2006).

In the recommended guide specification, lateral confinement pressure for reinforced concrete columns are based on the Canadian guidelines (ISIS 2001). The confinement model is simple enough for adoption in design and yields results that are consistent with the limited test data.

3.4.2 Strengthening Under Axial Loading and Flexure

The vast majority of research conducted on strengthening of axially loaded members has been limited to studying the effect of concrete confinement on the axial concentric compression strength of short reinforced concrete columns and piers, especially for seismic retrofitting that is necessitated by the inadequacy of transverse reinforcement. A comprehensive review of work conducted prior to 2001 was published by Triantafillou (2001). An examination of the 60 papers cited by Triantafillou (2001) shows clearly that there had been no systematic studies that address (1) FRP strengthening of reinforced concrete members under concentric tension and (2) FRP strengthening of reinforced concrete members subjected to combined axial loading and bending. The latter issue was recognized by researchers at the Laboratoire Central des Ponts et Chaussées in Paris, France, who examined two groups of 2.5 m columns of two different concrete strengths that had been externally strengthened with carbon fiber reinforcement and subjected to combined axial compression and bending (Quiertant et al. 2004; Quiertant and Toutlemonde, 2005).

3.5 Seismic Retrofitting with Externally Bonded FRP

Seismic retrofitting of existing reinforced concrete structural elements may be necessitated by the following:

(a) The inadequacy of transverse reinforcement, which may lead to brittle shear failure. This mechanism is associated with inclined cracking (diagonal tension), cover concrete spalling, and rupture or opening of the transverse reinforcement. The shear capacity of sub-standard elements (columns, shear walls, piers, exterior joints, etc.) can be enhanced by providing externally bonded FRPs with the fibers mainly in the hoop direction, in (preferably) closed-type jackets (Figure 3.11a,b).

(b) Poor confinement in flexural plastic hinge regions (column ends), where flexural cracking may be followed by cover-concrete crushing and spalling, buckling of the longitudinal reinforcement, or compressive crushing of the concrete. A ductile flexural plastic hinging at the column ends can be

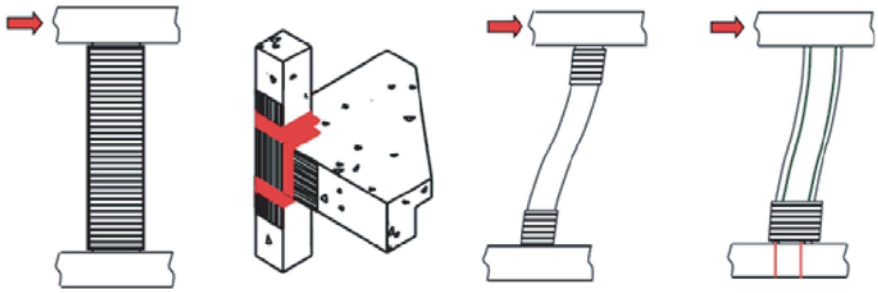

Figure 3.11. Seismic strengthening examples: (a) shear strengthening of RC column, (b) strengthening of beam-column joint, (c) local confinement in flexural plastic hinge regions, and (d) local confinement at lap splices.

achieved through added confinement in the form of FRP jackets with the fibers placed along the column perimeter (Figure 3.11c). This local confinement prevents spalling and delays crushing of concrete; also it delays or even eliminates buckling of longitudinal steel reinforcing bars.

(c) Poor detailing in lap splices at the lower ends of columns. The flexural strength of RC columns can only be developed and maintained when debonding of the reinforcement lap splice is prevented. Such debonding occurs once vertical cracks develop in the cover concrete and progresses with increased dilation and cover spalling. The associated rapid flexural strength degradation can be prevented or limited with increased lap confinement, again with fibers along the column perimeter (Figure 3.11d).

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CHAPTER 4

Conclusions, Implementation, and Recommendations for Further Research

4.1 Conclusions

This research developed a set of *Recommended Guide Specification for the Design of Externally Bonded FRP Reinforcement Systems for Repair and Strengthening of Concrete Bridge Elements.* The provisions contained in these specifications utilize the load combination requirements found in the *AASHTO LRFD Bridge Design Specifications 4th Edition* (2007) and employ the LRFD methodology. The load and resistance factors have been developed from structural reliability theory based on current probabilistic/statistical models of loads and structural performance and are designed to achieve the reliability levels that are already embedded in current AASHTO design and rating guidelines.

The recommended *Guide Specification* is accompanied by commentaries that are necessary for explaining the background, applicability, and limitations of the provisions contained therein. The guide specifications and commentary are presented in a format resembling that of the AASHTO *LRFD Bridge Design Specifications* in order to facilitate their consideration and adoption by AASHTO.

The *Guide Specification* consists of five sections. Section 1 addresses the scope, general requirements, and design basis of the proposed specifications. Section 2 defines the requirements for polymeric composite material systems intended for use for repair and strengthening of concrete bridge elements. The material provisions contained in Section 2 give a consistent basis for reporting material properties and the conditions under which these properties are obtained. Sections 3, 4, and 5 contain the recommended design provisions based on the limit states governing the response of a steel-reinforced concrete member subjected to flexure, shear, and combined axial loading and flexure, respectively.

In addition, step-by-step calculations in accordance with the *Recommended Guide Specification* are presented as examples of common FRP strengthening applications. These examples cover the five sections of the proposed *Guide Specification* and would serve as a tutorial of how to approach bridge strengthening projects in practice.

4.2 Implementation

The successful implementation of the *Guide Specification* developed in this research will require in-depth training for bridge design engineers to become fully acquainted with the fundamental principles, assumptions, limitations, and investigative procedures associated with the behavior and design of steel-reinforced concrete structural members reinforced with externally bonded FRP reinforcement. The training materials must emphasize the underlying basis for, and the details of, all relevant provisions of the *Guide Specification.* It is also recommended that the proposed *Guide Specification* be used for trial design by independent designers to identify potential improvements.

4.3 Recommendations for Further Research

Based on the findings and limitations of this research, further research on FRP strengthening of reinforced and prestressed concrete structural members is recommended. The following topics are proposed:

- **FRP Material Requirements.** Material requirements in the proposed guidelines stipulate that the FRP reinforcement be conditioned in four distinct environments and for a duration of 1,000 hours and then tested to ensure that the property of interest retains 85% of its original value. These requirements have been derived from the work of Steckel et al. (1999a, 1999b), and Hawkins et al. (1999), under the sponsorship of the California Department of Transportation (Caltrans), which called for property measurements after exposure intervals of 1,000 hours, 3,000 hours and 10,000 hours to allow estimates of degradation over the projected service life. It is suggested that research be conducted to (1) examine the reasonableness of test durations of up to 10,000 hours and (2) establish a standard practice of how to analyze and report the data resulting from any long-term tests.

- **Effect of Temperature on the Response of Reinforced Concrete Structural Members Externally Reinforced with FRP Reinforcement.** The proposed guide specifications require that the characteristic value of the glass transition temperature of the composite system and for the adhesive (when used) determined in accordance with ASTM D4065 shall be at least 40°F higher than the maximum design temperature, $T_{MaxDesign}$, defined in Section 3.12.2.2 of the *AASHTO LRFD Bridge Design Specification.* Research is needed to examine the validity of the requirement proposed in the Guidelines.
- **Experimental Studies Addressing FRP Reinforcement End Peeling.** It is suggested that the FRP reinforcement end peeling design equation be further validated under static and fatigue loading conditions and with exposure to various environmental conditions in order to establish the validity of the $0.065\sqrt{f_c'}$ limiting peel stress stipulated in the guidelines.
- **Shear Strengthening of Reinforced and Prestressed Concrete Girders with FRP Reinforcement.** Data concerning shear strengthening of concrete members are limited. Research on shear strengthening of bridge members with FRP reinforcement is needed. Such research should include (1) Complete wrapping of structural members, (2) The shear behavior of deep reinforced concrete beams fully wrapped with FRP reinforcement, and (3) Design guidelines and specifications related to the shear behavior of shallow and deep reinforced concrete beams reinforced with FRP reinforcement combined with mechanical anchorages. It is recommended that these studies include specifications for anchorage systems and design guidelines for anchorages for use in conjunction with the FRP reinforcement (Schuman and Karbhari 2004; Monti et al. 2004a, 2004b).
- **Experimental Behavior of Confined Rectangular Columns Under Axial Loading and Combined Axial Loading and Flexure.** Tests on eccentrically loaded compression slender and non-slender columns confined with FRP reinforcement are needed to address (1) the most appropriate confinement model for design purposes, (2) the effectiveness of the FRP reinforcement with respect to the cross section aspect ratio, and (3) experimental behavior of confined reinforced concrete columns under axial tension and flexure.

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ATTACHMENT A

Recommended Guide Specification for the Design of Bonded FRP Systems for Repair and Strengthening of Concrete Bridge Elements

This *Guide Specification* is the recommendation of the research team for NCHRP Project 10-73 that was conducted at Georgia Institute of Technology. The *Guide Specification* has not been approved by NCHRP or any AASHTO committee; nor has it been formally accepted for the AASHTO specifications.

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CONTENTS

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SECTION 1: GENERAL REQUIREMENTS

1.1 SCOPE

This *Guide Specification* is intended for the repair and strengthening of reinforced and prestressed highway bridge structures using externally-bonded fiber-reinforced polymeric (FRP) systems. This *Guide Specification* supplements the AASHTO *LRFD Bridge Design Specifications, 4th Edition (AASHTO 2007).* Except where specifically provided below, all provisions of the *LRFD Bridge Design Specifications* shall apply.

This *Guide Specification* states only the minimum requirements necessary to provide for public safety and are not intended to supplant proper training or the exercise of judgment by the Engineer of Record. The Owner or the Engineer of Record may require the structural design or the quality of materials and construction to exceed the minimum requirements.

This *Guide Specification* employs the Load and Resistance Factor Design (LRFD) methodology, in which the load and resistance factors have been developed from structural reliability theory based on current probabilistic/statistical models of loads and structural performance.

Seismic design shall be in accordance with either the provisions in the appropriate sections of the *LRFD Specifications* or the provisions in the AASHTO *Guide Specifications for LRFD Seismic Bridge Design*.

Except where specifically provided below, all provisions of the *LRFD Bridge Construction Specification* shall apply.

C1.1

Article 1.1 discusses the scope of the guide specifications, its applicability and limitations. This article is analogous to the opening articles, Articles X.1, of each of the sections of the AASHTO *LRFD Bridge Design Specifications, 4th Edition.*

The commentary is not intended to provide a complete historical background concerning the development of these or previous Specifications, nor is it intended to provide a detailed summary of the studies and research data reviewed in formulating the provisions of the Specification. However, references to North American and international guidelines (ACI 440.2R-02, 2000; ISIS Canada Design Manuals, 2001; *fib* technical report bulletin 14 , fib 2001; CNR-DT 200, 2006; JSCE, 2001; AFGC, 2003 ; and Avis Technique 3/01-345, 2001) as well as relevant research data dealing with externally bonded FRP reinforcement for reinforced and prestressed concrete structures are provided for those who wish to study the background material in depth.

NCHRP Report 609 presents recommended construction specifications concerning the use of externally bonded FRP reinforcement for strengthening concrete structures.

The commentary directs attention to other documents that provide suggestions for carrying out the requirements and intent of these Guide Specifications. However, the commentary and references herein are not part of these Guide Specifications.

1.2 DEFINITIONS

Definitions and terms related to the use of fiber-reinforced polymeric (FRP) materials in bridge engineering, rehabilitation and retrofit shall be as stipulated in ASTM D3878. Terms related to adhesives shall be as specified in ASTM D907.

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Definitions and terms related to highway bridge design shall be as stipulated in the AASHTO *LRFD Bridge Design Specifications, 4th Edition*.

1.3 SYMBOLS AND NOTATION

Variables used throughout the guide specifications as well as references to their usage are listed alphabetically below:

A_{frp} = effective area of FRP reinforcement for shear-friction (in^2)

A_g = gross area of column section (in^2)

A_h = area of one leg of the horizontal reinforcement (in^2)

A_s = area of nonprestressed tension reinforcement (in^2)

$A_s^{'}$ = area of compression reinforcement (in^2)

A_{st} = total area of longitudinal steel reinforcement (in^2)

A_{vf} = area of steel reinforcement required to develop strength in shear friction (in^2)

b = width of rectangular section (in)

b_{frp} = width of the FRP reinforcement (in.)

b_v = effective shear web width (in)

b_w = girder width (in)

C = clamping force across the crack face (kips)

c = depth of the concrete compression zone (in)

D_g = external diameter of circular column (in)

d_{frp} = effective FRP shear reinforcement depth (in)

d_s = distance from extreme compression surface to the centroid of nonprestressed tension reinforcement (in.)

d_v = effective shear depth (in)

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E_a = modulus of elasticity of adhesive (ksi)

E_c = modulus of elasticity of the concrete (ksi)

E_{frp} = modulus of the FRP reinforcement in the direction of structural action

f_c = stress in concrete at strain ε_c (ksi)

$f_c^{'}$ = 28 - day compression strength of the concrete (ksi)

$f_{cc}^{'}$ = compressive strength of confined concrete (ksi)

f_{frpu} = characteristic value of the tensile strength of FRP reinforcement (ksi)

f_{lfrp} = ultimate confinement pressure due to FRP strengthening (ksi)

f_{peel} = peel stress at the FRP reinforcement concrete interface (ksi)

f_s = stress in the steel tension reinforcement at development of nominal flexural resistance (ksi)

$f_s^{'}$ = stress in the steel compression reinforcement at development of nominal flexural resistance (ksi)

f_y = specified yield stress of steel reinforcement (ksi)

f_{yf} = yield strength of steel reinforcement for shear-friction (ksi)

G_a = characteristic value of the shear modulus of adhesive (ksi)

h = depth of section (in); overall thickness or depth of a member (in.)

I_T = moment of inertia of an equivalent FRP transformed section, neglecting any contribution of concrete in tension (in^4)

k_a = a coefficient that defines the effectiveness of the specific anchorage system

k_e = strength reduction factor applied for unexpected eccentricities

k_2 = multiplier for locating resultant of the compression force in the concrete

L_d = development length (in)

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l_u = unsupported length of compression member (in)

M_r = factored resistance of a steel-reinforced concrete rectangular section strengthened with FRP reinforcement externally bonded to the beam tension surface (kip-in)

M_u = factored moment at the reinforcement end-termination (kip-in)

N_b = FRP reinforcement strength per unit width at a tensile strain of 0.005 (kips/in)

N_{frp}^{e} = effective strength per unit width of the FRP reinforcement (kips/in)

$N_{frp,w}(r)$ = tensile strength of a closed (wrapped) jacket (kips/in)

N_s = FRP reinforcement strength per unit width at a tensile strain of 0.004 (kips/in)

N_{ut} = the characteristic value of the tension strength per unit width of the FRP reinforcement (kips/in)

P_r = factored axial load resistance (kips)

r = girder corner radius (in)

s_v = spacing of FRP reinforcement (in)

T_{frp} = tension force in the FRP reinforcement (kips)

T_r = the factored torsion strength of a concrete member strengthened with an externally bonded FRP system (kip-in)

t_a thickness of the adhesive layer (in)

t_{frp} thickness of the FRP reinforcement (in)

V_c = the nominal shear strength provided by the concrete (kips)

V_{frp} = the nominal shear strength provided by the externally bonded FRP reinforcement (kips)

V_{ni} = nominal shear-friction strength (kips)

V_p = component of the effective prestressing force in the direction of applied shear (kips)

V_r = factored shear strength of a concrete member strengthened with an externally bonded FRP system (kips)

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V_s = nominal shear strength provided by the transverse steel reinforcement (kips)

V_u = factored shear force at the reinforcement end-termination (kips)

w_{frp} = total width of FRP reinforcement (in)

y = distance from the extreme compression surface to the neutral axis of a transformed section, neglecting any contribution of concrete in tension (in)

α = angle between FRP reinforcement principal direction and the longitudinal axis of the member; angle between the shear-friction reinforcement and the shear plane (°)

α_1 = ratio of average stress in rectangular compression block to the specified concrete compressive strength

ε_c = strain in concrete

ε_{frp} = strain in FRP reinforcement

ε_{frp}^{ut} = characteristic value of the tensile failure strain of the FRP reinforcement

ε_o = the concrete strain corresponding to the maximum stress of the concrete stress-strain curve

μ = coefficient of friction

η = strain limitation coefficient that is less than unity

ν_a = Poisson's ratio of adhesive

τ_a = characteristic value of the limiting shear stress in the adhesive (ksi)

τ_{int} = interface shear transfer strength (ksi)

ϕ_{frp} = resistance factor for FRP component of resistance

1.4 DESIGN BASIS

1.4.1 Bridge elements strengthened with externally bonded reinforcement shall be designed to achieve the objectives of constructability, safety, and serviceability, with regard to issues of inspectability,

C1.4

The resistance criteria in this *Guide Specification* were developed using modern principles of structural reliability analysis, which are consistent with those on which the *AASHTO LRFD Bridge*

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economy and aesthetics, as stipulated in Article 2.5 of the *LRFD Bridge Specifications, 4th Edition*.

1.4.2 The provisions of this *Guide Specification* are limited to concretes with specified compressive strength f_c' not exceeding 8 ksi.

1.4.3 The characteristic value of the strength or failure strain in tension of FRP reinforcement used in bridge strengthening shall not exceed the 10th percentile value of the strength or failure strain, defined by a two-parameter Weibull distribution. The characteristic value shall be determined using a minimum of 10 samples. If the coefficient of variation in determined from this initial sample exceeds 15%, an additional 10 samples shall be tested, and the sample mean and sample coefficient of variation used to determine the parameter of interest shall be based on 20 samples. The test method shall be specified by the Engineer of Record.

1.4.4 The provisions of this *Guide Specification* shall apply to bridge elements for which the factored resistance satisfies the following requirement:

$$R_r \geq \eta_i \left[\left(DC + DW \right) + (LL + IM) \right]$$

R_r = factored resistance computed in accordance with Section 5 of the *LRFD Bridge Specifications, 4th Edition*.

η_i = load modifier specified in Article 1.3.2 of the *LRFD Bridge Specifications, 4th Edition*.

DC = force effects due to component and attachments

DW = force effects due to wearing surfaces and utilities

LL = force effects due to live loads

IM = force effects due to dynamic load allowance

Design Specifications are based. Structural reliability analysis takes the uncertainties in concrete, steel and FRP material strengths and stiffnesses into account using rational statistical models of these key engineering parameters. The criteria for checking safety and serviceability of structural members and components that have been strengthened with externally bonded FRP reinforcement are based on a target reliability index, β, equal to 3.5 under inventory loading, which was the target value assumed in the development of the *AASHTO LRFD Bridge Design Specifications*. The factored resistance and factored loads used in these checks are consistent with those found in customary engineering practice to facilitate their use in practice and to minimize the likelihood of misinterpretation.

The strength of FRP reinforcement depends on the engineering characteristics of the fibers and matrix and adhesive systems and on the workmanship in fabrication and installation. The resistance criteria in these Guide Specifications are based on the assumption that the nominal ultimate tensile strength of the FRP is the 10th percentile value of the two-parameter Weibull distribution that defines the uncertainty in the strength. Accordingly, to ensure the level of performance envisioned in these *Guide Specification*, the nominal strength stipulated in the construction documents should be the 10th percentile value of strength.

The two-parameter Weibull distribution is defined by

$$F(x) = 1 - e^{-\left(\frac{x}{u}\right)^{\alpha}}, \; x \geq 0$$

in which u and α are parameters of the distribution, which can be determined from the sample mean, $\overline{x}$, and sample coefficient of variation, COV. As an approximation,

$$u = \left[1 + (3/8)\; COV\right]\overline{x}$$

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$$\alpha = \frac{1.2}{COV}$$

The 10th percentile of the Weibull distribution then is estimated by,

$$x_{0.10} = u(0.1054)^{1/\alpha}$$

This *Guide Specification* are intended to apply only to bridge structural members and components that have a minimum strength prior to strengthening by externally bonded FRP reinforcement. If such a minimum cannot be shown by analysis or test to exist, the behavior of the strengthened member will depend almost entirely on the performance of the FRP reinforcement and if the field application of the FRP is deficient or if the bridge is accidentally overloaded, damage or failure may occur without warning. The limitation on strength prior to strengthening is intended to minimize the likelihood of occurrence of such damage or failure.

1.5 LIMIT STATES

Structural members shall satisfy Eq. 1.3.2.1-1 of the *LRFD Bridge Specifications, 4th Edition,* for each limit state, unless otherwise specified

The load factors, γ's, in Eq. 1.3.2.1-1 of the LRFD Specifications shall be as defined in LRFD Tables 3.4.1-1, 3.4.1-2 and 3.4.1-3. The resistance factors, φ's, are defined in Chapters 3, 4 and 5 of this *Guide Specification*.

C1.5

This *Guide Specification* applies to strength limit states I and II, serviceability limit states I, III and IV, Extreme Event limit states I and II, and the Fatigue limit state, as defined in Article 3.4 of the *LRFD Bridge Design Specifications.*

1.5.1 Service Limit States

Structural members shall satisfy LRFD Eq. 1.3.2.1-1 for the applicable combinations of factored force effects as specified at each of the following service limit states:

Service I - Load combination relating to the normal operational use of the bridge with a 55 mph wind and all loads taken at their nominal values.

C1.5.1

Service limit states customarily are defined by restrictions on stress, deformation, and crack width under regular service conditions. Compression in prestressed concrete components and tension in prestressed bent caps are investigated using the Service I load combination. The Service III limit-state load combination is used to investigate tensile stresses in prestressed concrete components.

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Service III - Load combination for longitudinal analysis relating to tension in prestressed concrete superstructures with the objective of crack control and to principal tension in the webs of segmental concrete girders

Service IV - Load combination relating only to tension in prestressed concrete columns with the objective of crack control.

The LRFD Service II limit state load combination is not applicable to concrete bridge structures reinforced with FRP systems as it is only applied to steel structures.

The live load specified in the *LRFD Specifications* reflects current exclusion weight limits mandated by various jurisdictions. Vehicles permitted under these limits were in service for many years prior to 1993. For longitudinal loading, there is no nationwide evidence that these vehicles have caused cracking in existing prestressed concrete components. The 0.80 factor on live load in the Service III combination reflects the fact that the event is expected to occur about once a year for bridges with two traffic lanes, less often for bridges with more than two traffic lanes, and about once a day for bridges with a single traffic lane. The Service I limit-state load combination should be used for checking tension related to transverse analysis of concrete segmental girders.

1.5.2 Strength Limit States

Structural members of a bridge shall satisfy LRFD Eq. 1.3.2.1-1 for the applicable combinations of factored extreme force effects, specified as follows:

Strength I - Basic load combination relating to the normal random vehicular use of the bridge without wind.

Strength II - Load combination relating to the use of the bridge by Owner-specified special design vehicles, evaluation permit vehicles, or both without wind.

Strength III - Load combination relating to the bridge exposed to wind velocity exceeding 55 mph.

Strength IV - Load combination relating to very high dead load to live load force effect ratios.

Strength V - Load combination relating to normal random vehicular use of the bridge with wind of 55 mph velocity.

C1.5.2

Design for strength limit states ensures that local and global strength and stability are provided to resist the specified load combinations that a bridge is expected to experience in its design life. The background for the load combination requirements in the LRFD Specifications is presented in Nowak (1993).

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1.5.3 Extreme-event Limit States

Structural members of a bridge shall satisfy LRFD Eq. 1.3.2.1-1 for the applicable combinations of factored extreme force effects as specified at each of the following:

Extreme-event I - Load combination including earthquake.

Extreme-event II - Load combination relating to ice load, collision by vessels and vehicles, and certain hydraulic events with a reduced live load other than that which is part of the vehicular collision load.

C.1.5.3

Consideration of extreme-event limit states is aimed at ensuring that the bridge structure survives a major earthquake or flood, collision from a vessel or heavy vehicle, or ice flow, or possible foundation scour.

1.5.4 Fatigue Limit State

Structural members, connections and components of a bridge shall satisfy LRFD Eq. 1.3.2.1-1 for the Fatigue I limit-state load combination, the load combination related to infinite load-induced fatigue life

C1.5.4

The fatigue limit states place restrictions on stress range resulting from a single design truck occurring at the number of expected stress range cycles. Concrete bridge structures are designed to provide a theoretically infinite fatigue design life.

The load factor for the Fatigue I load combination applied to a single design truck having the axle spacing specified in LRFD Article 3.6.1.4.1 reflects load levels found to be representative of the maximum stress range of the truck population. The Fatigue II limit-state load combination is not applicable to concrete bridge structures reinforced with FRP systems as it is not generally applicable to concrete components and connections.

1.6 LOADS AND LOAD COMBINATIONS

1.6.1 Loads

The loads defined in LRFD Article 3.3.2 and characterized in LRFD Article 3.6 through 3.15 shall be applied for designing reinforced concrete and prestressed highway bridge members strengthened with externally bonded FRP reinforcement

C1.6.1

The loads required for the design and evaluation of concrete bridge structures reinforced with FRP systems are classified in the *LRFD Specifications* as permanent and transient loads.

1.6.2 Load Combinations

The load combination requirements shall be determined in accordance with Article 3.4 of the AASHTO LRFD Bridge Design Specifications.

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1.7 EVALUATION OF EXISTING BRIDGE ELEMENTS

Bridge evaluations shall be performed using the evaluation criteria stipulated in the AASHTO *Manual for Bridge Evaluation, First Edition* (MBE, 2008). Eq. 6A.4.2.1-1 of the MBE shall be used in determining the load rating of each component and connection subjected to a single force effect (i.e., axial force, flexure, or shear).

The load rating shall be carried out at each applicable limit state with the lowest value determining the controlling rating factor. Limit states and load factors for load rating shall be selected from MBE Table 6A.4.2.2-1.Interaction of load effects (i.e., axial-bending interaction or shear-bending interaction), shall be considered, as provided in the MBE under the sections on resistance of structures, in developing the rating. .

C1.7

Bridge load ratings are performed for specific purposes, such as: NBI and BMS reporting, local planning and programming, determining load posting or bridge strengthening needs, and overload permit review. Live load models, evaluation criteria, and evaluation procedures are selected based upon the intended use of the load rating results. Live-load models used in evaluation are comprised of the design live load, legal loads, and permit loads.

Strength is the primary limit state for load rating; service and fatigue limit states are selectively applied in accordance with the provisions of the MBE. Live-load models for load rating include:

Design Load: HL-93 Design Load per LRFD Specifications

Legal Loads: AASHTO Legal loads, as specified in MBE Article 6A.4.4.2.1.1, and (2) The Notional Rating Load as specified in MBE Article 6A.4.4.2.1.2 or State legal loads.

Permit Load: Actual Permit Truck

Bridges that do not satisfy the HL-93 design load check should be evaluated for legal loads in accordance with the provisions of MBE Article 6A.4.4 to determine the need for load posting or strengthening. Legal loads for rating given in MBE Article 6A.4.4.2.1.1 that model routine commercial traffic are the same family of three AASHTO trucks (Type 3, Type 3S2, and Type 3-3) used in current and previous AASHTO evaluation Manuals. The single-unit legal load models given in MBE Article

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6A.4.4.2.1.2 represent the increasing presence of Formula B multi-axle specialized hauling vehicles in the traffic stream in many States.

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SECTION 2: MATERIALS REQUIREMENTS

2.1 SCOPE

This section defines the requirements for polymeric composite material systems intended for use for repair and strengthening of concrete bridge elements.

2.2 MATERIAL REQUIREMENTS

2.2.1 The contractor shall submit for approval evidence of acceptable quality control procedures followed in the manufacture of the composite reinforcement system. The quality control procedure shall include, but not be limited to, specifications for raw material procurement, the quality standards for the final product, in-process inspection and control procedures, test methods, sampling plans, criteria for acceptance or rejection, and record keeping standards.

2.2.2 The contractor shall furnish information describing the fiber, matrix, and adhesive systems intended for use as reinforcing materials that is sufficient to define their engineering properties. Descriptions of the fiber system shall include the fiber type, percent of fiber orientation in each direction, and fiber surface treatments. Where required by the Engineer of Record, the matrix and the adhesive shall be identified by their commercial names and the commercial names of each of their components, along with their weight fractions with respect to the resin system.

2.2.3 The contractor shall submit for approval test results that demonstrate that constituent materials and the composite system are in conformance with the physical and mechanical property values stipulated by the Engineer of Record. These tests shall be conducted by a testing laboratory approved by the Engineer of Record. For each property value, the batches from which test specimens were drawn shall be identified and the number of tested specimens from each batch, the mean value, the minimum value, the maximum value, and the coefficient of variation shall be reported. The minimum number of tested samples shall be 10.

2.2.4 When cured under conditions identical to those of the intended use, the composite material system as well as the adhesive system, if used, shall conform to the following requirements:

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2.2.4.1 The characteristic value of the glass transition temperature of the composite system, determined in accordance with ASTM D4065, shall be at least 40°F higher than the maximum design temperature, $T_{MaxDesign}$, defined in Section 3.12.2.2 of the *AASHTO LRFD Bridge Design Specifications.*

C 2.2.4.1 The glass transition temperature, T_g, is the approximate temperature value or temperature range at which the matrix changes from a glassy to a rubbery state. Above T_g, the composite softens and loses its mechanical properties, as illustrated in Figure C2.1 . In addition, it is to be noted that T_g decreases as the moisture content in the composite increases.

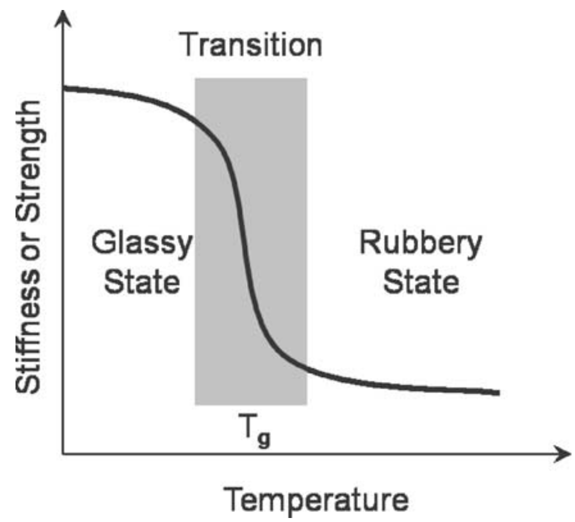

Figure C2.1 Effect of temperature on the properties of polymer composite materials (Zureick and Kahn, 2001)

2.2.4.2 The characteristic value of the tensile failure strain in the direction corresponding to the highest percentage of fibers shall not be less than 1%, when the tension test is conducted according to ASTM 3039.

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2.2.4.3 The average value and coefficient of variation of the moisture equilibrium content determined in accordance with ASTM D 5229/D 5229M shall not be greater than 2% and 10%, respectively. A minimum sample size of 10 shall be used in the calculation of these maximum values.

C2.2.4.3 The diffusion of moisture into organic polymers results in pronounced changes in mechanical, chemical, and thermophysical properties of practically all composite reinforcing systems. All organic matrix systems and organic reinforcing fibers absorb moisture to a certain degree. While both glass and carbon fibers are considered to be impervious to moisture absorption, aramid fibers absorb more moisture than many matrix systems. In all cases when moisture migrates through the matrix system and ultimately reaches the fiber-matrix interface, adhesion of the matrix system to the fibers become weak and the structural integrity of the composite system degrades.

2.2.4.4 After conditioning in the following environments, the characteristic value of the glass transition temperature determined in accordance with ASTM D4065 and the characteristic value of the tensile strain, determined in accordance with ASTM D3039, of the composite in the direction of interest shall retain 85% of the values established in Art. 2.2.4.1 and 2.2.4.2, respectively.

A. Water: Samples shall be immersed in distilled water having a temperature of 100 ± 3°F (38 ± 2°C) and tested after 1,000, hours of exposure.

B. Alternating ultraviolet light and condensation humidity: Samples shall be conditioned in an apparatus under Cycle 1 -UV exposure condition according to ASTM G154 Standard Practice. Samples shall be tested within two hours after removal from the apparatus.

C. *Alkali*: The sample shall be immersed in a saturated solution of calcium hydroxide (pH ~11) at ambient temperature of 73 ± 3°F (23 ± 2°C) for 1000 hours prior to testing. The pH level shall be monitored and the solution shall be maintained as needed.

D. *Freeze-thaw*: Composite samples shall be exposed to 100 repeated cycles of freezing and thawing in an apparatus meeting the requirements of ASTM C666.

C2.2.4.4. The physical and mechanical properties of FRP materials and of the concrete structure reinforced with an externally bonded reinforced system are sensitive to various environmental conditions that can affect one or more of the followings:

- Chemical and/or physical changes in the polymeric matrix.
- Loss of bond at the fiber/matrix interface and at the FRP-concrete interface.
- Strength and stiffness degradation of the reinforcing fibers.

The durability requirements in Article 2.2.4.4 are based on those developed for CALTRANS (Steckel et al., 1999a, 199b;; Hawkins et al., 1999) and for GDOT (Zureick, 2002).

Cycle No 1 UV exposure condition uses UVA-340 lamps that simulate direct solar radiation and have negligible UV output below 300nm, considered to be the “cut-on” wavelength for terrestrial sunlight.

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2.2.5 Where impact tolerance is stipulated by the Engineer, the stipulated impact tolerance shall be determined by ASTM D7136.

2.2.6 Adhesive: when adhesive material is used to bond the FRP reinforcement to the concrete surface, the following requirements shall be met:

2.2.6.1 After conditioning in the environments stipulated in Article 2.2.4.4 A-D, the characteristic value of the glass transition temperature of the adhesive material determined in accordance with ASTM D 4065, shall be at least 40°F higher than the maximum design temperature, $T_{MaxDesign}$, defined in Section 3.12.2.2 of AASHTO LRFD Bridge Design Specifications.

2.2.6.2 After conditioning in the environments stipulated in Article 2.2.4.4 A-D, the bond strength (ksi), determined by tests specified by the Engineer of Record, shall be at least $0.064\sqrt{f_c^{'}}$, where $f_c^{'}$ (ksi) is the specified compression strength of the concrete.

C.2.2.6.2

The bond strength limit of $0.064\sqrt{f_c^{'}}$ is based on tests conducted by Naaman (1999) on reinforced concrete beams strengthened with externally bonded FRP reinforcement

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SECTION 3: MEMBERS UNDER FLEXURE

3.1 GENERAL REQUIREMENTS

The factored resistance of structural members subjected to flexure shall equal or exceed the required strength calculated for the factored loads and forces in combinations stipulated by this *Guide Specification.*

Except where specifically provided below, all provisions of the AASHTO LRFD Bridge Design Specification (2007 edition), Article 5.7.3, shall apply.

3.2 DESIGN ASSUMPTIONS

The calculation of the strength of reinforced concrete members externally reinforced with FRP materials shall be based on the following assumptions:

- The distribution of strains over the depth of the member is linear and conditions of force equilibrium and strain compatibility are satisfied.
- Perfect bond exists between the reinforcing steel, FRP reinforcement and the concrete.
- The contribution of tension stresses in the concrete to flexural strength is neglected.
- The stress-strain behavior for FRP reinforcement is linear-elastic to the point of failure.
- The stress-strain behavior of steel reinforcement is bilinear, with elastic behavior up to yielding and perfectly plastic behavior thereafter.
- The maximum usable compression strain in the concrete is equal to 0.003.
- The maximum usable strain at the FRP/concrete interface is 0.005.

C3.2

The strength in flexure of a reinforced concrete member that has been additionally reinforced by an externally bonded FRP plate is derived from the classic Bernoulli-Navier hypothesis that plane sections remain plane and perpendicular to the neutral axis during flexure. The stresses on the section can be determined from the constitutive relations for the concrete, reinforcing steel and FRP reinforcement, and the flexural strength at any section is determined from requirements for axial force and moment equilibrium at that section.

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When concrete compressive strain is 0.003 under conditions of force equilibrium, it is permitted to model the distribution of concrete stress in compression as having a uniform stress of $0.85 f_c'$ over a depth $a = \beta_1 c$, in which c = depth to the neutral axis from the compression face of the beam and β_1 = stress block factor specified in Article 5.7.2.2 of the *AASHTO LRFD Bridge Design Specifications.*

When concrete compressive strain is less than 0.003 under conditions of force equilibrium, the concrete compression stress distribution shall be modeled as parabolic according to the following equation:

$$f_c = \frac{2\left(0.9 f_c'\right)\left(\varepsilon_c/\varepsilon_o\right)}{1+\left(\varepsilon_c/\varepsilon_o\right)^2} \qquad (3.2\text{-}1)$$

where $$\varepsilon_o = 1.71\frac{f_c'}{E_c} \qquad (3.2\text{-}2)$$

and

f_c = stress in concrete at strain ε_c (ksi)

ε_c = strain in concrete

f_c' = the 28 - day compression strength of the concrete (ksi)

ε_o = the concrete strain corresponding to the maximum stress of the concrete stress-strain curve

E_c = the modulus of elasticity of the concrete specified in Section 5.4.2.4 of the AASHTO *LRFD Bridge Design Specifications* (ksi)

When the compression strain in the extreme fiber of the concrete is less than 0.003 at force equilibrium, the Whitney compression stress block may no longer describe the compression resultant in the concrete accurately, and a more exact representation of the distribution of concrete stress in the compression zone is required. Eq (3.2-1), which provides this representation, was presented by Desayi and Krishnan, S. (1964) and by Todeschini et al. (1964).

3.3 FATIGUE LIMIT STATES

3.3.1 Subjected to the fatigue load combination specified in Article 3.4.1 of the AASHTO LRFD

C3.3

By limiting the maximum strain in the concrete to that specified in Eq. 3.3-1, the stress range in the

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Bridge Design Specifications, the maximum compression strain in the concrete, ε_c, the strain in the steel reinforcement, ε_s, and the strain in the FRP reinforcement, ε_{frp}, shall meet the following requirements

$$\varepsilon_c \leq 0.36 \frac{f_c^{'}}{E_c} \qquad (3.3\text{-}1)$$

$$\varepsilon_s \leq 0.8\varepsilon_y \qquad (3.3\text{-}2)$$

$$\varepsilon_{frp} \leq \eta\varepsilon_{frp}^{u} \qquad (3.3\text{-}3)$$

where

ε_{frp}^{u} = characteristic value of the tensile failure strain of the FRP reinforcement when tested in accordance with ASTM D3039

η = strain limitation coefficient that is less than unity. The Engineer of Record shall stipulate the value of η based on experimental data for the materials specified, and this value shall be provided in the contract documents. In the absence of such data, a value of η = 0.8, 0.5, and 0.3 shall be used for carbon, aramid, and glass fiber reinforcement, respectively.

concrete will be kept within $0.40 f_c^{'}$. Limiting the strain of the steel reinforcement under service load to 80% of the yield strain of the steel is equivalent to the recommendation of ACI Committee 440, where the stress in the reinforcing steel under service load is limited to 80% of the yield stress of the steel; this recommendation is based on the analytical work of El-Tawil et al. (2001).

Strain limits on the FRP reinforcement are placed to avoid creep-rupture of the reinforcement. Polymer composites reinforced with carbon fibers are less susceptible to creep rupture than those reinforced with glass or aramid fibers. The recommended strain reduction factors of 0.8, 0.5, and 0.3 are based on studies reported by Yamaguchi et al. (1997) and Malvar (1998) and are recommended for the design of externally bonded FRP reinforcement for reinforced concrete structures by fib Task Group 9.3 (fib, 2001), and by ACI 440 Committee (ACI 440.2R-02).

As the design is often governed by service limit state, FRP strains at Service I load combination are sufficiently low that creep rupture of the FRP is typically not of concern.

3.4 STRENGTH LIMIT STATES

3.4.1 Factored Flexural Resistance

3.4.1.1 *Rectangular Sections*

The factored resistance, M_r, of a steel-reinforced concrete rectangular section strengthened with FRP reinforcement externally bonded to the beam tension surface shall be taken as

$$M_r = 0.9\left[A_s f_s\left(d_s - k_2 c\right) + A_s^{'} f_s^{'}\left(k_2 c - d_s^{'}\right)\right] + \phi_{frp} T_{frp}\left(h - k_2 c\right) \qquad (3.4.1.1\text{-}1)$$

C3.4.1

The factored resistance is in line with the design strength determination in accordance with Article 5.7.3.2 of AASHTO LRFD Bridge Design Specifications, and is written so that the design strength for a reinforced concrete flexural member is simply augmented by the contribution of the externally bonded FRP reinforcement. This format ensures that when the FRP reinforcement is slight, the design strength approaches that of a flexural member without FRP reinforcement.

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where

$$T_{frp} = b_{frp} N_b \qquad (3.4.1.1\text{-}2)$$

$$k_2 = 1 - \frac{2\left[\left(\frac{\varepsilon_c}{\varepsilon_o}\right) - \arctan\left(\frac{\varepsilon_c}{\varepsilon_o}\right)\right]}{\beta_2 \left(\frac{\varepsilon_c}{\varepsilon_o}\right)^2} \qquad (3.4.1.1\text{-}3)$$

$$\beta_2 = \frac{Ln\left[1 + \left(\frac{\varepsilon_c}{\varepsilon_o}\right)^2\right]}{\left(\frac{\varepsilon_c}{\varepsilon_o}\right)} \qquad (3.4.1.1\text{-}4)$$

A_s = area of nonprestressed tension reinforcement

$A_s^{'}$ = area of compression reinforcement (in^2)

f_s = stress in the steel tension reinforcement at development of nominal flexural resistance (ksi)

$f_s^{'}$ = stress in the steel compression reinforcement at development of nominal flexural resistance (ksi)

c = depth of the concrete compression zone (in)

d_s = distance from extreme compression surface to the centroid of nonprestressed tension reinforcement (in)

h = depth of section (in)

T_{frp} = tension force in the FRP reinforcement (kips)

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ϕ_{frp} = resistance factor equal to 0.85

k_2 = multiplier for locating resultant of the compression force in the concrete

b_{frp} = width of the FRP reinforcement (in)

N_b = FRP reinforcement strength per unit width, corresponding to 0.5% strain in the FRP reinforcement when subjected to tension in accordance with ASTM D3039.

3.4.1.2 *Flanged Sections*

For flanged sections subjected to flexure about one axis where the neutral axis, based upon the stress distribution specified in Article 3.2, lies within the flange, the factored resistance, M_r, shall be computed in accordance with Article 3.4.1.1. When the neutral axis falls outside the flange , the factored flexural resistance shall be determined by an analysis based on the assumptions specified in Article 3.2.

C3.4.1.2

For most practical cases involving flanged sections strengthened externally with bonded FRP reinforcement to the tension surface, the depth of the neutral axis falls within the flange. When the neutral axis falls below the flange, the compression force exerted in the concrete is the sum of two components, one of which corresponds to the flange and one corresponds to the portion of the web under compression. Due to the nature of the assumed nonlinear stress-strain relationship of Eq. 3.2-1, the determination of the compressive force requires integration of the stress-strain function expressed in Eq. 3.2-1 over the area of the cross-section.

3.4.1.3 *Other Cross-Sections*

For cross-sections other than rectangular or flanged sections, the factored flexural resistance, M_r, shall be determined by an analysis based on the assumptions specified in Article 3.2.

3.4.1.4 *Prestressed Sections*

For rectangular and nonrectangular prestressed concrete sections subjected to flexure about one axis, the factored flexural resistance shall be determined by an analysis based on the assumptions specified in Article 3.2

3.4.2 Ductility requirements

The strain developed in the FRP reinforcement at the

C3.4.2

This provision ensures that the tension steel reinforcement yields before the point of incipient

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ultimate limit state defined by eq (3.4.1.1-1) shall be equal to or greater than 2.5 times the strain in the FRP reinforcement at the point where the steel tension reinforcement yields.

debonding of the externally bonded FRP reinforcement, thereby enabling the development of a ductile mode of flexural failure.

3.4.3 Detailing requirements

Flexural members shall be detailed to permit the development of the factored resistance defined by Eq (3.4-1).

C3.4.3

The externally bonded FRP reinforcement must be installed and detailed in such a manner that the assumptions in Section 3.2 are valid and the flexural capacity defined in eq (3.4-1) can be fully developed.

3.4.3.1 Development length

The tension development length ,L_d, shall be taken as

$$L_d \geq \frac{T_{frp}}{\tau_{\text{int}}\, b_{frp}} \qquad (3.4.3.1\text{-}1)$$

where $T_{FRP} =$ tensile force (kips) in the FRP reinforcement corresponding to an FRP strain of 0.005 and

$\tau_{\text{int}} = 0.065\sqrt{f_c'}$ is the interface shear transfer strength, (ksi).

C3.4.3.1

The minimum development length is required to allow the full tension strength of the FRP reinforcement to be developed in the region of maximum moment. The interface shear transfer strength limit of $\tau_{\text{int}} = 0.065\sqrt{f_c'}$ is based on the recommendation of Naaman and Lopez (1999) and Naaman et al. (1999) from tests conducted on uncracked and precracked reinforced concrete beams externally bonded with FRP reinforcement and subjected to bending with 300 freeze-thaw cycles. The limit also represents a lower bound for experimental data conducted on short-term direct tension tests of FRP reinforcement bonded to concrete surfaces (Haynes, 1997; Binzindavyi and Neale, 1999).

3.4.3.2. Reinforcement End Peeling

The peel stress at the end of externally bonded reinforcement shall meet the following requirement:

$$f_{peel} \leq 0.065\sqrt{f_c'} \qquad (3.4.3.2\text{-}1)$$

in which:

C3.4.3.2

The end of an externally bonded reinforcement system, when subjected to combined shear and flexure, may separate in the form of debonding in three different modes: critical diagonal crack debonding with (Yao and Teng 2007) or without (Oehlers and Seracino, 2004) concrete cover separation; concrete cover separation (Teng et al., 2002); and plate end interfacial debonding (Teng et al., 2002).

Critical diagonal crack debonding may occur where the FRP end is located in a zone of high shear force and the amount of steel shear reinforcement is

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$$f_{peel} = \tau_{av}\left[\left(\frac{3E_a}{E_{frp}}\right)\frac{t_{frp}}{t_a}\right]^{1/4} \qquad (3.4.3.2\text{-}2)$$

$$\tau_{av} = \left[V_u + \left(\frac{G_a}{E_{frp}t_{frp}t_a}\right)^{1/2} M_u\right]\frac{t_{frp}(h-y)}{I_T} \qquad (3.4.3.2\text{-}3)$$

and where:

h = overall thickness or depth of a member (in.)

y = distance from the extreme compression surface to the neutral axis of a transformed section, neglecting any contribution of concrete in tension (in).

I_T = moment of inertia of an equivalent FRP transformed section, neglecting any contribution of concrete in tension (in^4)

t_a = thickness of the adhesive layer (in)

t_{frp} = thickness of the FRP reinforcement (in)

$E_a = 2G_a(1+\nu_a)$ Young's modulus of adhesive (ksi)

G_a = characteristic value of the shear modulus of adhesive when tested in accordance with ASTM D5656 (ksi).

ν_a = Poisson's ratio of adhesive, taken as equal to 0.35

τ_a = characteristic value of the limiting shear stress in the adhesive (ksi), determined according to ASTM D 5656. In the absence of experimental data, a value of $\tau_a = 5\,ksi$ can be used.

V_u = factored shear force at the reinforcement end

limited. In such a case a major diagonal shear crack forms and intersects the FRP, and then propagates towards the end. This failure mode is suppressed if the shear strength of the strengthened member remains higher than the flexural strength.

In beams with heavy steel shear reinforcement, multiple diagonal cracks of smaller widths instead of a single major shear crack dominate the behavior, so concrete cover separation may take over as the controlling debonding failure mode. Failure of the concrete cover is initiated by a crack near the FRP end due to the stress concentration at that point. The crack then propagates to and then along the level of steel tension reinforcement. This mode of failure has been demonstrated experimentally for beams with externally bonded steel plates (Jones et al., 1988; Oehlers and Moran, 1990) and FRP reinforcement (Malek et al., 1998; Lopez and Naaman, 2003; Yao and Teng 2007).

Plate-end interfacial debonding is also initiated by high interfacial shear and normal stresses near the end of the FRP that exceed the strength of the weakest element, generally the concrete. Debonding in this case propagates from the end of the FRP towards the middle, near the FRP-concrete interface. Note that this failure mode is only likely to occur when the FRP is significantly narrower than the beam section.

In summary, provided that shear failure is suppressed (through shear strengthening, if needed), stress concentrations near the FRP reinforcement end may result in debonding through the concrete layer near the level of the longitudinal steel (or, rarely, near the FRP-concrete interface).

Although a wide range of predictive models that include numerical, fracture mechanics, data-fitting, and strength of material-based methods have been developed to address this complex mode of failure (Yao, 2004), the equations presented in 3.4.3.2 are based on the approximate analysis of Roberts (1989), due to its simplicity for design purposes.

At present, there is no standard test method for

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(kips)

M_u = factored moment at the reinforcement end (kip-in)

determining the peel strength between an FRP reinforcement system and a concrete surface. Until such a test method is developed, the ASTM Standard Test Method D 3167 is recommended for determining the peel strength within the adhesive layer. ASTM D3167 is used for determining the peel resistance of adhesive bonds between one rigid adherend and one flexible adherend. For cases in which the peeling occurs within the concrete layer, it is recommended that the peeling strength be limited to $0.065\sqrt{f_c^{'}}$.

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SECTION 4: MEMBERS UNDER SHEAR AND TORSION

4.1 GENERAL REQUIREMENTS

The factored shear and torsion resistance of structural members at all sections shall equal or exceed the required strength in shear or in torsion calculated for the factored loads and forces in combinations stipulated in Article 3.4 of the *AASHTO LRFD Bridge Design Specification, 4th Edition (2007).*

Except where specifically provided below, all provisions of the *AASHTO LRFD Bridge Design Specification, 4th Edition (2007), Article 5.8,* shall apply.

C4.1

The provisions for strengthening reinforced concrete structural members and components for shear and torsion using externally bonded FRP reinforcement have been developed with the assumption that all design requirements for shear and torsion in Article 5.8 of the *AASHTO LRFD Bridge Design Specification, 4th Edition (2007)* shall apply, *except* as specifically provided for in Section 4. Any duplication of provisions in these two documents is intended solely to facilitate the use and interpretation of provisions in Section 4.

4.2 STRENGTHENING SCHEMES

Reinforced concrete bridge elements shall be strengthened with externally bonded FRP reinforcement using one of the following methods:

- Side bonding
- U-jacketing
- U-jacketing combined with anchorage
- Complete wrapping

Transverse reinforcement shall be provided symmetrically on both sides of the element with spacing not to exceed the smaller value of 0.4 d_v or 12 inches, where d_v is the effective shear depth defined in Article 5.8.2.9 of *AASHTO LRFD Bridge Design Specifications*

C4.2

Typical FRP strengthening schemes for beams and columns are summarized as follows:

Side bonding (Fig. C4.2-1) is the least effective FRP shear reinforcement scheme due to premature debonding under shear loading and should be avoided if possible. Side bonding does not allow for the development of the shear-resisting mechanism based on a parallel chord truss model that was first proposed by Ritter (1899), due to the lack of tensile diagonals.

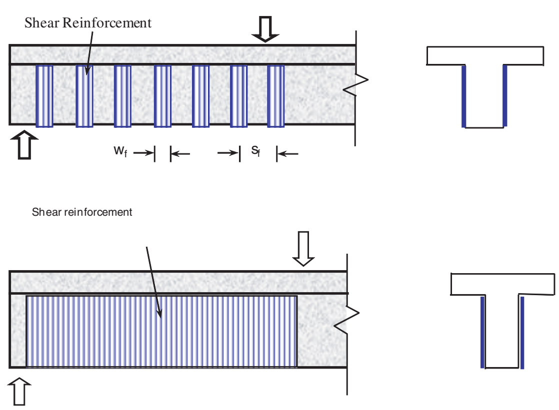

Figure C4.2-1 Side bonding

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U-jacketing (Fig. C4.2-2) is the most common externally bonded shear strengthening method for reinforced concrete beams and girders. The key drawback of this system is the possibility of premature debonding of the FRP, which may reduce its effectiveness. Despite this drawback, the system is quite popular in practice, due to its simplicity.

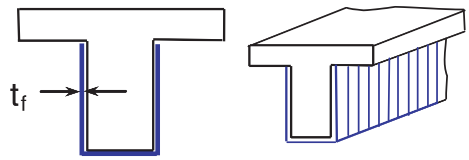

Figure C 4.2-2

- *Jacketing combined with anchorage* (Fig. C4.2-3) aims to increase the effectiveness of FRP by anchoring the fibers, preferably, in the compression zone. Properly designed anchors may result in the fibers reaching their tensile capacity, permitting the jacket to behave as if it were completely wrapped.

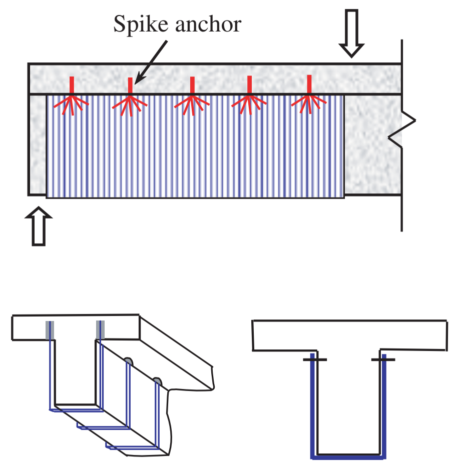

Figure C 4.2-3 Jacketing with anchorages

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4.3 STRENGTH IN SHEAR

The factored shear strength, V_r, of a concrete member strengthened with an externally bonded FRP system shall equal or exceed the required shear strength, V_u, determined from the effect of the factored loads.

C4.3

Shear strengthening of reinforced concrete members using FRP reinforcement may be provided by bonding the external reinforcement (typically in the form of sheets) with the principal fiber direction as parallel as practically possible to that of maximum principal tensile stresses, so that the effectiveness of FRP is maximized. For the most common case of structural members subjected to lateral loads, in which loads are perpendicular to the member axis (e.g. beams under gravity loads or columns under seismic forces), the maximum principal stress trajectories in the shear-critical zones form an angle with the member axis which may be taken roughly equal to 45^{o}. However, it is normally more practical to attach the external FRP reinforcement with the principal fiber direction perpendicular to the member axis.

Experimental and analytical investigations of the behavior of reinforced concrete members strengthened in shear have revealed the following failure modes:

1 - Steel yielding followed by FRP debonding

2 - Steel yielding followed by FRP fracture

3 - FRP debonding before steel yielding

4 - Diagonal concrete crushing

Depending on the amount of usable steel shear reinforcement in the structural element, FRP debonding can occur either before or after steel yielding. The third failure mode is, in fact, highly unlikely to occur if proper detailing is provided.

Diagonal concrete crushing in the direction perpendicular to the tension field can be suppressed by limiting the total amount of steel and FRP reinforcement. Note that fracture of the FRP reinforcement is highly unlikely to occur because the strain when FRP debonds is substantially lower than that corresponding to the FRP fracture strength.

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4.3.1 Factored Strength

The factored shear strength, V_r shall be defined as

$$V_r = \phi\left(V_c + V_s + V_p\right) + \phi_{frp} V_{frp} \quad (4.3.1\text{-}1)$$

in which:

V_c = the nominal shear strength provided by the concrete in accordance with Articles 5.8.3.3 of the AASHTO *LRFD Bridge Design Specifications*.

V_s = the nominal shear strength provided by the transverse steel reinforcement in accordance with Article 5.8.3.3 of the AASHTO *LRFD Bridge Design Specifications*;

V_p = component of the effective prestressing force in the direction of applied shear as specified in Article 5.8.3.3 of the AASHTO *LRFD Bridge Design Specifications*;

V_{frp} = the nominal shear strength provided by the externally bonded FRP system in accordance with Article 4.3;

$\phi = 0.9$

ϕ_{frp} is a resistance factor, defined as follows:

0.40 for side bonding shear reinforcement;

0.55 for U-jacketing;

0.60 for U-jacketing combined with anchorages;

0.65 for complete wrapping.

C4.3.1

The shear provisions in Article 4.3 draw upon the traditional ACI approach embodied by Chapter 11 of the *ACI Standard 318-05*, supplemented by the report of ACI Committee 440.2R-02 (ACI, 2002).

The contribution of the externally bonded FRP reinforcement to shear strength is based on fiber orientation and an assumed crack pattern following the formulation of Khalifa, et al (1998). Its contribution to member shear strength may be treated analogously to the treatment of internal steel, assuming that the FRP plate carries only normal stresses in the principal FRP material direction and that at the ultimate limit state in shear (concrete diagonal tension), the FRP develops an effective strain in the principal material direction of approximately 0.004. This limiting strain is conservative with respect to what tests have indicated (Sato et al., 1996; Araki et al., 1997; Triantafillou, 1998, Carolin, and Taljsten,2005; Chajes et al., 1995; Deniaud and Cheng, 2001;). Such a limiting strain value was also proposed by Priestley et al. (1996) to control circular bridge column dilation and was adopted by ACI Committee 440 (2002).

Statistical data to support the reliability-based determination of resistance factors were available only for U-jacketing. The resistance factor for that case was found to be 0.55; resistance factors for other methods of reinforcement were set by judgment.

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Exception: When structural members without shear stirrups are being evaluated for possible upgrading, V_s in eq (4.1) shall be zero and ϕ shall be 0.60.

Exception: For load combinations involving earthquake effects, ϕ and ϕ_{frp} in eq (4.1) shall be reduced by 20%.

4.3.2 Limitation on strength provided by concrete and steel

The sum of $V_c + V_s$ shall not exceed $0.25 f_c' b_v d_v$, in which b_v and d_v are effective web width and shear depth, defined in Article 5.8.2.9, in which f_c' is expressed in ksi units.

4.3.3 Regions requiring externally bonded shear reinforcement

Except for slabs, footings and culverts, shear reinforcement shall be provided where the required strength exceeds $0.5\phi\left(V_c + V_p\right)$ in which V_c, V_p, and ϕ are defined in Article 4.3.1, or where consideration of torsion is required by Eqs 5.8.2.1-3 or 5.8.6.3-1 of the *AASHTO LRFD Bridge Design Specification.* It is permitted to waive this minimum requirement if it can be demonstrated by test that the required shear strength can be developed when shear reinforcement is omitted. Such tests shall simulate the in-service effects of creep, shrinkage, temperature change and differential settlement.

C4.3.3

Shear reinforcement shall be provided in all reinforced concrete flexural members where there is a significant probability that diagonal cracking will occur.

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4.3.4 Strength provided by externally bonded FRP reinforcement

4.3.4.1 Nominal Strength

The contribution of the externally bonded FRP plate to the nominal shear strength shall be determined as follows:

a) For intermittent FRP reinforcement

$$V_{frp} = \frac{N^{e}_{frp} w_{frp} \left(\sin\alpha + \cos\alpha\right) d_{frp}}{s_v} \qquad (4.3.4.1\text{-}1)$$

b) For continuous FRP reinforcement

$$V_{frp} = N^{e}_{frp} \left(\sin\alpha + \cos\alpha\right) d_{frp} \qquad (4.3.4.1\text{-}2)$$

Where

w_{frp} = width of FRP reinforcement;

s_v = spacing of FRP reinforcement (measured parallel to the member axis);

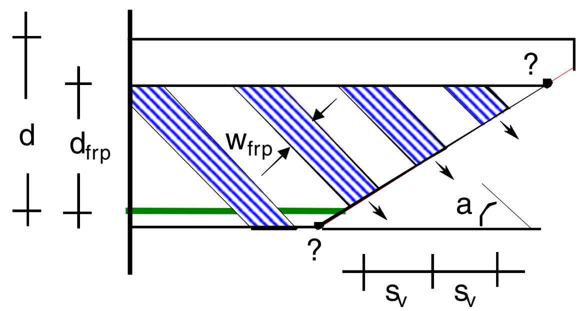
d_{frp} = Effective FRP shear reinforcement depth; and

N^{e}_{frp} = effective strength per unit width of the FRP reinforcement, determined in accordance with Article 4.3.4.2.

α = angle between FRP reinforcement principal direction and the longitudinal axis of the member.

C4.3.4.1

The contribution of the FRP to the shear strength of a member is based on an assumed crack pattern of 45° and the fiber orientation (angle between the principal fiber orientation and longitudinal axis of member in Fig. C4.3.4.1-1a. Eq (4.3.4.1-1) is analogous to the equation for shear appearing in Chapter 11 of *ACI Standard 318-05*. The effective strength per unit width in Eq (4.3.4.1-1) or Eq (4.3.4.1-2) may be taken equal to the mean FRP stress along the shear crack. The value of this stress at each location along the shear crack depends mainly on the strengthening scheme (complete wrapping, U-jacketing, anchored U-jacketing, and side bonding) and on the bond stress – slip relation at the FRP-concrete interface (Triantafillou 1998).



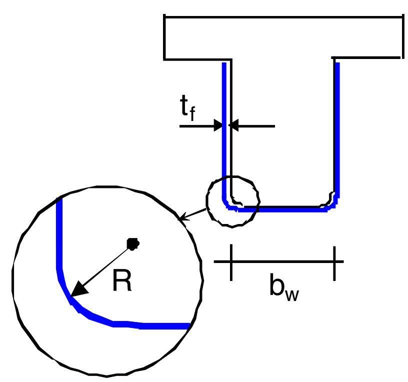


Figure C4.3.4.1

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4.3.4.2 Effective strength of FRP reinforcement

The effective strength of FRP reinforcement for each of the strengthening methods specified in Article 4.2 shall be determined as follows:

C4.3.4.2

Equations defining the effective strength of FRP shear reinforcement are based on the work of Priestley et al. (1996) and the work of Monti et al. (2004a, 2004b) simplified for design purposes. In such formulations the stress multiplied by thickness terms were replaced by the strength per unit width for consistency throughout these *Guide Specifications.*

4.3.4.2.1 For side bonding and U-jacketing without anchorage:

$$N_{frp}^{e} = N_s \qquad (4.3.4.2.1\text{-}1)$$

where

N_s = FRP tensile strength per 1-inch width corresponding to a tensile strain of 0.004

4.3.4.2.2 For U-jacketing combined with anchorage

$$N_{frp}^{e} = N_s + k_a \frac{1}{2}\left[N_{frp,w} - N_s\right] \qquad (4.3.4.2.2\text{-}1)$$

in which k_a is a coefficient that depends on the effectiveness of the specific anchorage system. If the anchorage system is engineered in accordance with Articles D.3 and D.4 of Appendix D in *ACI Standard 318-05* $k_a = 1$. Otherwise, k_a shall be taken as equal to zero. $N_{frp,w}$ is the tensile strength of a closed (wrapped) jacket applied to a member of radius at the corners of the cross section not less than ½ in., defined as:

$$N_{frp,w} = 0.5N_{ut} \geq N_s \qquad (4.3.4.2.2\text{-}2)$$

N_{ut} = nominal tensile strength of the FRP reinforcement;

N_s = Strength of FRP reinforcement corresponding to a strain of 0.004

The term k_a in eq 4.3.4.2.2-1 is a coefficient that defines the effectiveness of the specific anchorage system. In view of the limited available test data, on FRP reinforcement with mechanical anchorage systems, it is recommended that if the anchorage is engineered, the strength can be fully developed and $k_a = 1$; otherwise, its strength contribution is unknown.

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4.3.4.2.3 Complete wrapping (closed jackets)

$$N_{frp}^{e} = N_s + \frac{1}{2}\left[N_{frp,w} - N_s\right] \quad (4.3.4.2.3\text{-}1)$$

4.3.5 Maximum nominal shear strength provided by reinforcement

The nominal shear strength provided by all shear reinforcement (steel stirrups plus externally bonded FRP plate) shall not exceed:

$$V_s + V_{frp} \leq 8\sqrt{f_c^{'}}b_w d \qquad (4.3.5\text{-}1)$$

C4.3.5

The limitation on the total shear reinforcement that can be provided is based on the criterion given for steel alone in *ACI Standard 318-05*. The purpose of this limitation is to minimize the likelihood of sudden failure caused by yielding of the transverse steel or debonding of the FRP reinforcement.

4.4 STRENGTH IN TORSION

The factored torsion strength, T_r, of a concrete member strengthened with an externally bonded FRP system shall equal or exceed the required torsion strength, T_u, determined from the effect of the factored loads.

C4.4

Strengthening for increased torsional capacity may be required in conventional beams and columns, as well as in box girders and other structural members with hollow sections. The principles applied to strengthening in shear are also valid, for the most part, for the case of torsion.

The user of these *Guide Specifications* is cautioned that, in contrast to the provisions in Articles 4.2 and 4.3, supporting experimental data on the enhancement of the capacity of a member to withstand torsion by externally bonded FRP reinforcement does not exist. Accordingly, in situations where this limit state is considered, the Engineer of Record should consider the option of confirmatory testing.

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4.4.1 Factored strength in torsion

The factored torsion strength, T_r, shall be defined as:

$$T_r = \phi T_n + \phi_{frp} T_{frp} \qquad (4.4.1\text{-}1)$$

in which:

T_n = nominal strength in torsion specified in Article 5.8.3.6 of the *AASHTO LRFD Bridge Design*

Specifications;

T_{frp} = nominal shear strength provided by the externally bonded FRP system in accordance with Article 4.4.2;

$\phi = 0.90$

$\phi_{frp} = 0.65$

4.4.2 Nominal strength in torsion

Externally bonded FRP reinforcement used to strengthen members in torsion shall be completely wrapped, as defined in Article 4.2.The nominal strength in torsion shall be calculated as follows:

For intermittent FRP reinforcement,

$$T_{frp} = \frac{N^e_{frp} d_{frp} \alpha_t x_1 y_1}{s} \qquad (4.4.2\text{-}1)$$

For continuous FRP reinforcement

$$T_{frp} = N^e_{frp} d_{frp} \alpha_t x_1 y_1 \qquad (4.4.2\text{-}2)$$

in which

$\alpha_t = 0.66 + 0.33\left(y_1/x_1\right) \leq 1.5$

x_1 = lesser dimension of the member

y_1 = larger dimension of the member

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4.5 STRENGTH IN INTERFACE SHEAR TRANSFER – SHEAR FRICTION

The factored strength in shear-friction of a concrete member strengthened with an externally bonded FRP system shall equal or exceed the required shear strength, V_u, determined from the effect of the factored loads

C4.5

The provisions for interface shear transfer in Article 4.5 are presented for consistency with Section 5.8.4 of the *AASHTO LRFD Bridge Design Specifications.*

The user of these *Guide Specifications* is cautioned that, in contrast to the provisions in Articles 4.2 and 4.3, supporting experimental data on the enhancement of the capacity of a member to withstand shear friction by externally bonded FRP reinforcement does not exist. Accordingly, in situations where this limit state is considered, the Engineer of Record should consider the option of confirmatory testing.

4.5.1 Applicability

It is permitted to determine the factored strength by shear-friction when shear transfer occurs across a given plane, such as an existing or potential crack, an interface between dissimilar materials, an interface between two concretes cast at different times, or the interface between different elements of the cross section.

A crack shall be assumed to occur along the shear plane considered, and the required area of shear-friction reinforcement, A_{vf}, across the shear plane shall be calculated using 4.5.3.

All reinforcement provided to resist interface shear transfer shall be appropriately placed along the shear plane and shall be anchored to fully develop the required strength on both sides of the interface.

C4.5.1

In shear-friction analysis, it is presumed that a crack will form in an unfavorable location and that reinforcement must be provided across the crack to resist relative displacement along the crack. When shear acts along a crack, one crack face slips relative to the other. In reinforced concrete construction, the crack faces are irregular and this slip is accompanied by separation of the crack faces. The slip movement and irregularities on the crack face introduce tension in the reinforcement that crosses the crack, and causes a clamping force to be developed normal to the crack. The applied shear then is resisted by friction between the crack faces (including shearing of aggregate protruding on the crack faces) and, usually to a lesser extent, by “dowel” action of the reinforcement that crosses the crack. The effectiveness of the shear-friction mechanism in withstanding applied shear depends on assuming the correct location of the crack.

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4.5.2 Factored Strength for Shear-Friction

The factored strength for shear friction shall be

$$V_{ri} = \phi V_{ni} \qquad (4.5.2\text{-}1)$$

in which

V_{ni} = nominal shear-friction strength calculated in accordance with 4.5.3;

$\phi = 0.90$

C4.5.2

The format for factored strength for shear-friction in Eq (4.5.2-1) is different from the format for factored strength for shear strength in Eq (4.3.1-1) because the load-resisting mechanism of interface shear transfer is different from that represented by Article 4.3. The contribution of the FRP reinforcement is included in the clamping force that appears in the expression for V_{ni}, rather than additive to the factored shear strength. Similarly, the resistance factor for contribution of the FRP reinforcement is embedded in the clamping force.

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4.5.3 Nominal strength for shear-friction

4.5.3.1 - Where shear-friction reinforcement is perpendicular to the shear plane defined in Article 4.5.1, V_{ni} shall be computed by:

$$V_{ni} = C\mu \qquad (4.5.3.1\text{-}1)$$

in which

C = clamping force across the crack face, defined in Article 4.5.3.3;

μ = coefficient of friction defined in 4.5.3.4

4.5.3.2 – Where shear-friction reinforcement is inclined to the shear plane, such that the shear force produces tension in shear-friction reinforcement, V_{ni} shall be computed by:

$$V_{ni} = C\left(\mu \sin\alpha + \cos\alpha\right) \qquad (4.5.3.2\text{-}1)$$

in which α = angle between the shear-friction reinforcement and the shear plane.

C 4.5.3

The calculation of nominal strength for interface shear transfer in Article 4.5.3 is based on Section 11.7 of *ACI Standard 318-05* rather than Article 5.8.4.1 of the *AASHTO LRFD Bridge Design Specification.* The ACI approach is based on the assumption that the resistance to interface shear transfer is directly proportional to the net clamping force. This resistance is determined simply as $C\mu$, in which C is the clamping force normal to the shear plane and μ is the coefficient of friction. The assumption that all shear resistance is due to friction between the crack faces neglects the contribution of dowel action of the steel reinforcement crossing the crack and necessitates the use of artificially high values of μ so that the calculated strength will be consistent with test results.

The ACI approach is simpler than the AASHTO approach in ascribing the resistance to interface shear transfer entirely to the clamping force. Furthermore, Article 5.8.4.1 of the *AASHTO Specification* contains several experimental constants (c, K_1 and K_2) that would have to be revised to account for the presence of FRP shear reinforcement.

The clamping forces in Articles 4.5.3.2 and 4.5.3.3 have been modified to account for the presence of FRP reinforcement crossing the crack. To preserve the familiar format of the factored resistance, the resistance factor, ϕ_{frp}, is included in the expression for the clamping force

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4.5.3.3 – The clamping force, C, shall be determined as follows:

$$C = A_{vf} f_{yf} + \phi_{frp} A_{frp} E_{frp} \varepsilon_{frp} \quad (4.5.3.3\text{-}1)$$

In which

A_{vf} = area of steel reinforcement for shear-friction;

f_{yf} = yield strength of steel reinforcement for shear-friction;

A_{frp} = effective area of FRP reinforcement for shear-friction;

E_{frp} = effective modulus of FRP reinforcement for shear-friction;

ε_{frp} = strain in FRP reinforcement for shear-friction, and

$\phi_{frp} = 0.65$

The strain in the FRP reinforcement for shear-friction shall be taken as 0.004 unless test data are provided to support an alternative value.

4.5.3.4 – The coefficient of friction, μ, shall be determined as follows:

- $\mu = 1.4\lambda$ for concrete placed monolithically;
- $\mu = 1.0\lambda$ for concrete placed against hardened concrete intentionally roughened
- $\mu = 0.7\lambda$ for concrete anchored to structural steel by studs or other mechanical devices
- $\mu = 0.6\lambda$ for concrete placed by other methods than those above

in which

- $\lambda = 1.0$ for normal weight concrete
- $\lambda = 0.75$ for light weight concrete

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4.5.3.5 The nominal shear strength V_n shall not exceed the smaller of $0.2 f_c' A_c$ or $800 A_c$, where A_c is the area of the concrete section resisting shear transfer.

4.5.3.6 Net tension across the shear plane shall be resisted by additional reinforcement. The value of f_y used for design of shear-friction reinforcement shall not exceed 60 ksi. It is permitted to take permanent net compression across the shear plane as additive to the force in the shear-friction reinforcement, $A_{vf} f_y$, when calculating the required A_{vf}

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SECTION 5: MEMBERS UNDER COMBINED AXIAL FORCE AND FLEXURE

5.1 GENERAL REQUIREMENTS

The factored resistance of structural members subjected to axial forces and combined axial forces and flexure shall equal or exceed the required strength at all sections calculated for the factored loads and forces in combinations stipulated by these *Guide Specifications*.

Except where specifically provided below, all provisions of *Article 6.9 of* the *AASHTO LRFD Bridge Design Specifications, 4th Edition (2007),* shall apply.

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5.2 METHODS FOR STRENGTHENING WITH FRP REINFORCEMENT

5.2.1 Columns shall be strengthened with FRP reinforcement using the complete wrapping method specified in Article 4.2.

5.3 COLUMNS IN AXIAL COMPRESSION

5.3.1 General Requirements

The factored axial load resistance, P_r, for a confined column shall be taken as follows:

For members with spiral reinforcement

$$P_r = 0.85\phi\left[0.85 f'_{cc}\left(A_g - A_{st}\right) + f_y A_{st}\right] \tag{5.3.1-1}$$

For members with tie reinforcement

$$P_r = 0.80\phi\left[0.85 f'_{cc}\left(A_g - A_{st}\right) + f_y A_{st}\right] \tag{5.3.1-2}$$

C5.3.1

The design procedure for columns strengthened with FRP is the same as for reinforcement concrete columns without strengthening. However, the concrete compressive strength f_c' is substituted by the increased confined concrete compressive strength f_{cc}' as calculated according to Article 5.3.2.2.

The multipliers of 0.85 and 0.80 in Equations 5.3.1-1 and 5.3.1-2 reflect the effect of minimum accidental eccentricities of axial force (0.05h and 0.10h, respectively, for columns with spiral or tied reinforcement) which impart small end moments to columns. Columns with eccentricities greater than these values must be designed using the provisions of Section 5.5.to take these extra moments into account.

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where

ϕ = resistance factor specified in Article 5.5.4.2 of the *AASHTO Bridge Design Specifications, 4th Edition*

A_g = gross area of section (in^2)

A_g = total area of longitudinal reinforcement, (in^2).

f_y = specified yield strength of reinforcement (ksi)

f'_{cc} = compressive strength of the confined concrete determined according to Article 5.3.2.2.

Confined circular columns sustain ultimate axial strains that are far greater than those of non-confined columns. Any gain in strength due to strain hardening of the steel reinforcement is not accounted for in the above equation, thus providing additional safety. This gain is a function of the ultimate axial strains, unless buckling of the steel reinforcement initiates failure of the column.

5.3.2 Short Columns in Compression

Columns in compression shall be fully wrapped over the entire length.

C5.3.2

The provisions in Article 5.3.2 apply to short columns in which second-order effects are negligible and the limit state of instability can be ignored.

5.3.2.1 Limitations

Provisions in this section shall apply to circular columns in which the slenderness parameter l_u/D does not exceed 8 and to rectangular columns in which the aspect ratio, h/b does not exceed 1.1, the minimum radius of corners is one inch, and the slenderness parameter, l_u/b, does not exceed 9, where:

D = external diameter of the circular member

b = smaller dimension of the rectangular member

h = larger dimension of the rectangular member

C5.3.2.1

The limitations are similar to those in the Canadian guidelines for column strengthening (ISIS 2001). The limitation on column slenderness in this section ensures that the development of column strength not prevented by column instability.

5.3.2.2 Confinement in Columns

C5.3.2.2

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The compressive strength of the confined concrete, f'_{cc}, shall be determined from:

$$f'_{cc} = f'_c\left(1+\frac{2f_l}{f'_c}\right)$$

(5.3.2.2-1)

The confinement pressure due to FRP strengthening, f_l [ksi] for circular columns shall be determined as:

$$f_l = \phi_{frp}\frac{2N_{frp}}{D} \le \frac{f'_c}{2}\left(\frac{1}{k_e\phi}-1\right)$$

(5.3.2.2-2)

where

k_e is a strength reduction factor applied for unexpected eccentricities. It shall be taken as follows:

k_e =0.80 for tied columns, and

k_e = 0.85 for spiral columns.

N_{frp} = Strength per width of FRP reinforcement corresponding to a strain of 0.004.

ϕ_{frp} = 0.65,

The confinement pressure shall be greater or equal to 600 psi.

For rectangular columns, the diameter D in Eq (5.3.2.2-2) shall be replaced with the smaller dimension of the width and depth.

The bonding of FRP sheets, where the fiber orientation is perpendicular to the column axis to limit the circumferential strains in the column, constitutes confinement. Various confinement models have been developed over the years and comparisons among the most common models have been presented by Rocca et al. (2008). The expression for the compressive strength of confined concrete adopted in these guides is similar to that of ISIS Canada due to its simplicity. The stress-strain curve for concrete confined by FRP reinforcement can be considered to be bilinear, but differs from the situation where the confinement is provided by spiral reinforcement or steel jacketing. The secondary stiffness depends on the hoop stiffness of the confining reinforcement.

The maximum value of the confinement pressure specified in Eq 5.3.2.2-2 was established to limit the axial compression strains in overstrengthened columns. The minimum confinement pressure of 600 psi reflects the fact that the effectiveness of the confinement pressure depends upon a certain level of ductility. Relevant background related the maximum and minium values of confinement pressure in FRP reinforcement jackets in axially loaded columns is given by Thériault and Neale (2000).

When Equation 5.3.2.2-2 is applied to rectangular columns after replacing D with the smaller dimension of the rectangular section, the factored axial strength estimated from eqs. 5.3.1-1 or 5.3.1-2 errs on the conservative side. At present, this is justified owing to the limited properly documented available test data.

The gain in strength provided by the confinement of rectangular sections is very little compared to that attainable for circular sections. As a result, neither minimum nor maximum limits are specified for rectangular sections since the attainable confinement pressure, which relies on ductility development, is very limited for rectangular columns.

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5.3.3 Slender columns

Columns not meeting the limitations on slenderness in 5.3.2.1 shall be designated as slender and their design shall be based on forces and moments determined from rational analysis. Such an analysis shall take into account the influence of forces, deflections and foundation rotations, and duration of loads on member stiffness and on the development of moments, shears and axial forces.

C5.3.3

The provisions for short columns in Article 5.3.2 are adequate for the majority of rehabilitation projects because second-order structural actions leading to instability seldom would occur. There is only limited test data to support the development of column strength provisions in situations where this is not the case. In such situations, the required columnstrength should be determined by rational analysis, supplemented by confirmatory testing, where feasible.

5.4 COMBINED AXIAL COMPRESSION AND BENDING

5.4.1 General requirements

Members subjected to moment in combination with axial load shall be designed for the maximum moment that can accompany the axial load. The factored axial force at given eccentricity shall not exceed P_r given in Section 5.3.1. The maximum required moment, M_u, shall be magnified, as appropriate, for slenderness effects.

C5.4.1

The design procedure for the members strengthen with FRP is the sameas for reinforcement concrete members without strengthening. However, the concrete compressive strength $f_c^{'}$ is substituted by the increased confined concrete compressive strength $f_{cc}^{'}$ as calculated according to articles 5.3.2.2.

5.4.2 Design Basis

Design of columns subject to combinations of axial force and flexure shall be based on stress and strain compatibility. The maximum usable strain in the extreme concrete compression fiber shall be assumed to equal 0.003.

Externally bonded FRP reinforcement of columns strengthened to withstand end moments due to lateral load shall be reinforced over a distance from the column ends equal to the maximum column dimension or the distance over which the moment exceeds 75% of the maximum required moment, whichever distance is larger.

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The tensile strength of the FRP reinforcement in the longitudinal direction of the column shall be determined by rational analysis. However, the strength in the longitudinal direction shall not be less than 50% of the strength in the perimeter direction.

5.4.3. Limitations

The contribution of the FRP reinforcement to column capacity shall not be considered at eccentricity ratios greater than those corresponding to balanced strain conditions, at which tension reinforcement reaches the strain corresponding the steel yield strength and concrete in compression reaches an ultimate strain of 0.003 at any cross section.

C5.4.3

Provisions in Article 5.4 are limited to members subjected to combined axial loading and bending where failures occur by concrete crushing in compression rather than reinforcement yielding in tension. If the eccentricity of axial force present in the member is greater than 0.10h for the spirally reinforced columns or 0.05h for tied columns, strengthening requires externally bonded FRP reinforcement to withstand force in the longitudinal direction of the columnin addition to its perimeter.

5.5 AXIAL TENSION

5.5.1 Limitation

Members that are axially loaded in tension shall be reinforced symmetrically with respect to the column cross section principal axes.

5.5.2 General requirements

The factored axial load resistance, P_r, for an axially loaded member with externally bonded FRP reinforcement shall be

$$P_r = 0.9 A_s f_y + \phi_{frp} N_{frp} w_{frp}$$

In which

$\phi_{frp} = 0.5$

N_{frp} = tensile strength per unit width in the load direction at a strain value of 0.005.

w_{frp} = total length of FRP reinforcement along the cross section.

C5.5.2

FRP systems can be used to provide additional tensile strength to concrete members. The tension strength provided by the FRP is limited by the design tensile strength of the FRP and the ability to transfer stresses into the substrate through bond. The effective strain in the FRP can be determined based on the criteria given for shear strengthening.

For members completely wrapped by the FRP systems, loss of the aggregate interlock of concrete occurs at fiber strain less than the ultimate fiber strain. To preclude this mode of failure, the maximumdesign strain should be limited to 0.4%:

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$$\varepsilon_{fe} = 0.004 \le 0.75\varepsilon_{fu}$$

where

ε_{fe} is the effective strain level in FRP reinforcement attained at failure

ε_{fu} is the design rupture strain of FRP reinforcement

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ATTACHMENT B

Illustrative Examples

The following examples are presented to illustrate calculations associated with a number of commonly used FRP strengthening techniques in accordance with the recommended *Guide Specification* for the Design of Bonded FRP Reinforcement Systems for Repair and Strengthening of Concrete Bridge Elements. These examples should illustrate how to approach bridge strengthening projects in practice. These examples cover the five sections of the proposed *Guide Specification.*

Example 1: Calculation of the characteristic value of the strength of an FRP reinforcement system
Example 2: Flexural strengthening of a T-beam in an unstressed condition
Example 3: Flexural strengthening of a T-beam in a stressed condition
Example 4: Shear strengthening of a T-beam using U-jacket FRP reinforcement
Example 5: Shear strengthening of a rectangular beam using complete wrapping FRP reinforcing system
Example 6: Strengthening of an axially loaded circular column.

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Example 1

It is required to calculate the characteristic value of the strength of a field-manufactured FRP system to be bonded externally to strengthen an existing bridge member. Tensile tests were conducted on coupons in accordance with ASTM D3039. Results are given in Table B1. It is also required to establish the linear load-strain relationship for use in the design following the recommended Guide Specifications.

Table B1: Summary of test data

Coupon ID	Strength (kips/in) at 1% strain
1	2.00
2	2.17
3	2.01
4	2.10
5	1.87
6	2.14
7	2.08
8	2.12
9	2.10
10	2.09
11	2.10
12	2.14
13	2.12
14	2.21
15	1.95
16	2.22

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Step 1: Examine the data set to find out if there is any outlier. To do so, the Maximum Normed Residual outlined in ASTM D7290 Standard Practice will be used. To do so

1.1 Sort the data in ascending order as shown in Table B2.

Table B2- Strength data sorted in ascending order

Coupon ID	Strength (kips/in) at 1% strain
5	1.87
15	1.95
1	2.00
3	2.01
7	2.08
10	2.09
4	2.1
9	2.1
11	2.1
8	2.12
13	2.12
6	2.14
12	2.14
2	2.17
14	2.21
16	2.22

1.2 Calculate the arithmetic mean, $\overline{x}$, and the standard deviation, s, of the sample population from the equations

$$\overline{x} = \frac{\sum_{i=1}^{n} x_i}{n}$$

$$s = \sqrt{\frac{\sum_{i=1}^{n} \left(x_i - \overline{x}\right)^2}{n-1}}$$

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$$\bar{x} = \frac{1.87 + 1.95 + ... + 2.21 + 2.22}{16} = 2.09\ kips / in$$

$$s = \sqrt{\frac{\sum_{i=1}^{16} (x_i - 2.09)^2}{(16-1)}} = 0.092\ kips / in$$

Step 3

1.2 Calculate the Maximum Normed Residual (MNR) as follows:

For each data value, compute the term $\frac{|x_i - \bar{x}|}{s}$ representing the deviation from the sample mean divided by the sample standard deviation.

- For the first data value, coupon ID 5:

$$\frac{|x_i - \bar{x}|}{s} = \frac{|1.87 - 2.09|}{0.092} = 2.388$$

- For the second data value, coupon ID38:

$$\frac{|x_i - \bar{x}|}{s} = \frac{|1.95 - 2.09|}{0.092} = 1.52$$

- For the remaining data, computed $\frac{|x_i - \bar{x}|}{s}$ values are given in Table B3.
- Compute the critical value, C_r, from the equation:

$$C_r \approx \left(2 - \frac{8}{5\sqrt{n}}\right)^2 = \left(2 - \frac{8}{5\sqrt{16}}\right)^2 = 2.56$$

- The Maximum Normed Residual (MNR) is

$$MNR = \max\left(\frac{|x_i - \bar{x}|}{s}\right) = 2.39$$

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Because $MNR = 2.39 < C_r = 2.56$, the sample population is outlier-free, and one can proceed with the statistical evaluation.

Table B3 Strength deviation from the sample mean

Coupon ID	Strength (kips/in) at 1% strain	$\frac{\lvert x_i - \bar{x} \rvert}{s}$
5	1.87	2.39
15	1.95	1.52
1	2.00	0.98
3	2.01	0.87
7	2.08	0.11
10	2.09	0.00
4	2.1	0.11
9	2.1	0.11
11	2.1	0.11
8	2.12	0.33
13	2.12	0.33
6	2.14	0.54
12	2.14	0.54
2	2.17	0.87
14	2.21	1.30
16	2.22	1.41

Step 4

The sample size is greater than 10 and the coefficient of variation $COV = \frac{0.092}{2.09} = 0.044 < 0.15$, the composite material strength data meet the requirements of Article 1.4.3 of the *Guide Specifications*.

Step 5: Estimate the parameters of the two-parameter Weibull distribution from the following equations (See Commentary C1.4 of the Guide Specifications)

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$$u = \left[1 + (3/8)COV\right]\bar{x} = \left[1 + (3/8)(0.044)\right](2.09) = 1.02(2.09) = 2.13$$

$$\alpha = \frac{1.2}{COV} = \frac{1.2}{0.044} = 27.3$$

Step 6: Compute the characteristic value of the strength from the equation:

$$x_{0.10} = u\left(0.1054\right)^{1/\alpha} = (2.13)\left(0.1054\right)^{1/27.3} = 1.96\ kips/in$$

Step 7: Establish the FRP reinforcement strength-strain design relationship as shown in B-1.

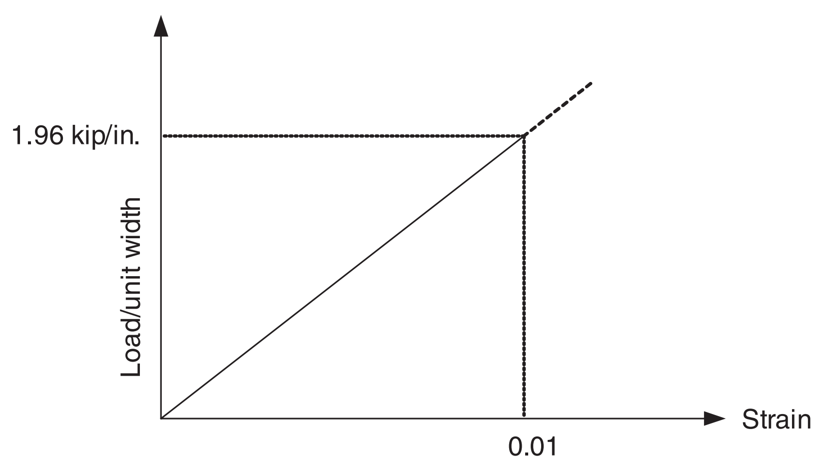

Figure B-1 Load-strain relationship of the FRP reinforcement

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Example 2

Flexural strengthening of a simply supported cast in-place reinforced concrete girder

This example illustrates the flexural strengthening of a reinforced concrete T-beam with an externally bonded carbon fiber-reinforced polymeric reinforcement system to accommodate higher loading.

Bridge Data

Span: 39 ft
Type: Cast in-place reinforced concrete
Year built: 1957
Location: State of Georgia
Concrete compression Strength: $f_c^{'} = 3.9\ ksi$ (from *in-situ* testing)
Reinforcing steel yield strength: $f_y = 40\ ksi$
Girder dimensions and Steel Reinforcement: See Figure B.2
FRP reinforcement: Shop-fabricated carbon fiber/Epoxy composite plates
Plate thickness, $t = 0.039''$
Glass Transition Temperature: $T_g = 165^o F$
Tensile strain in the FRP reinforcement at failure: $\varepsilon_{frp}^{tu} = 0.013$
Tensile strength in the FRP reinforcement at 1% strain: $P_{frp} = 9.3\ kips/in$
Shear modulus of the adhesive = 185 ksi

Structural Analysis Results under the New Loading

For Strength I Load Combination: $M_D = 239\ kip - ft$ and $M_{L+I} = 615\ kip - ft$.
For Fatigue Limit State: $M_{L+I} = 308\ kip - ft$

Special Notes

Hydraulic jacking procedure of the bridge will be used so that strengthening is carried out in an unstressed condition.

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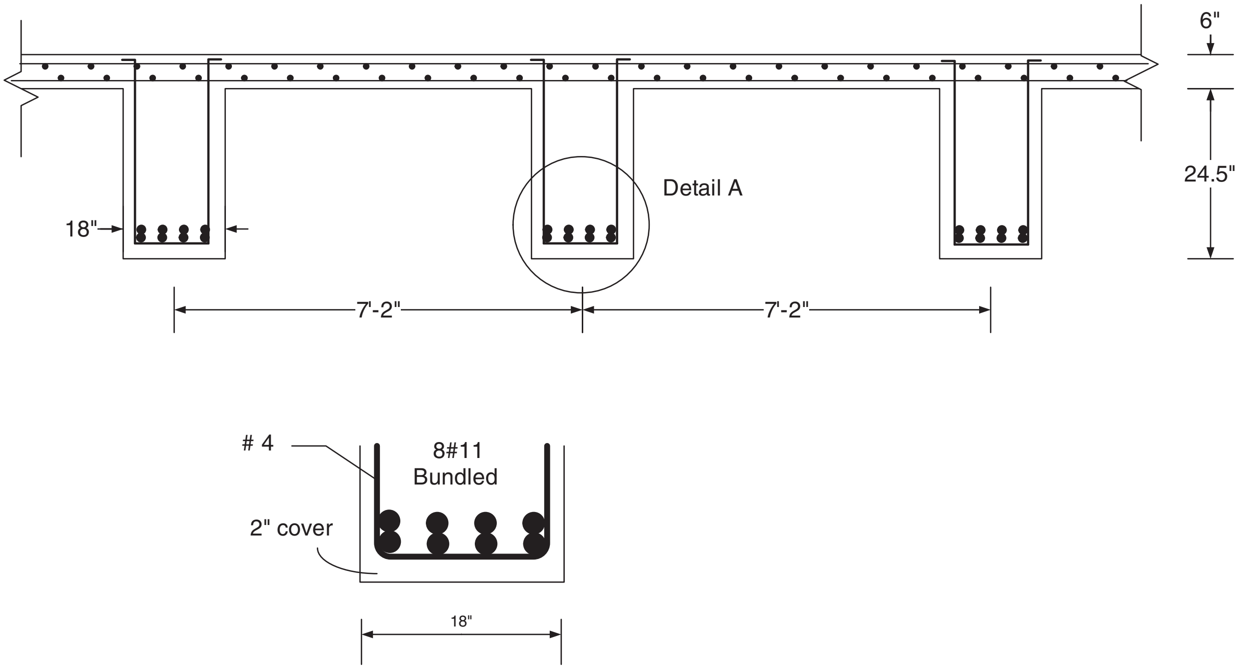

Figure B-2 Bridge cross section at mid-span

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SOLUTION

Step 1:

Determine if the FRP reinforcement material is in compliance with Section 2 of the Guide Specification and be sure that the glass transition temperature is higher than the maximum design temperature plus 40°F.

The maximum design temperature, $T_{MaxDesign}$, determined from Article 3.12.2.2 of AASHTO LRFD Bridge Design Specifications for the location of the bridge (State of Georgia)

$$T_{MaxDesign} = 110^{o} F$$

$T_{MaxDesign} + 40^{o} F = 110^{o} F + 40^{o} F = 150^{o} F < T_{g} = 165^{o} F$. Thus, Article 2.2.4.1 of the Guide Specification is satisfied.

Step 2: Establish the linear stress-strain relationship of the FRP reinforcement based on the design assumptions specified in Article 3.2 of the Guide and compute the tensile strength corresponding to a strain value of 0.005. Results are presented in Figure B.3

$$N_{b} = \frac{0.005}{0.01}(9.3) = 4.65\, kip / in.$$

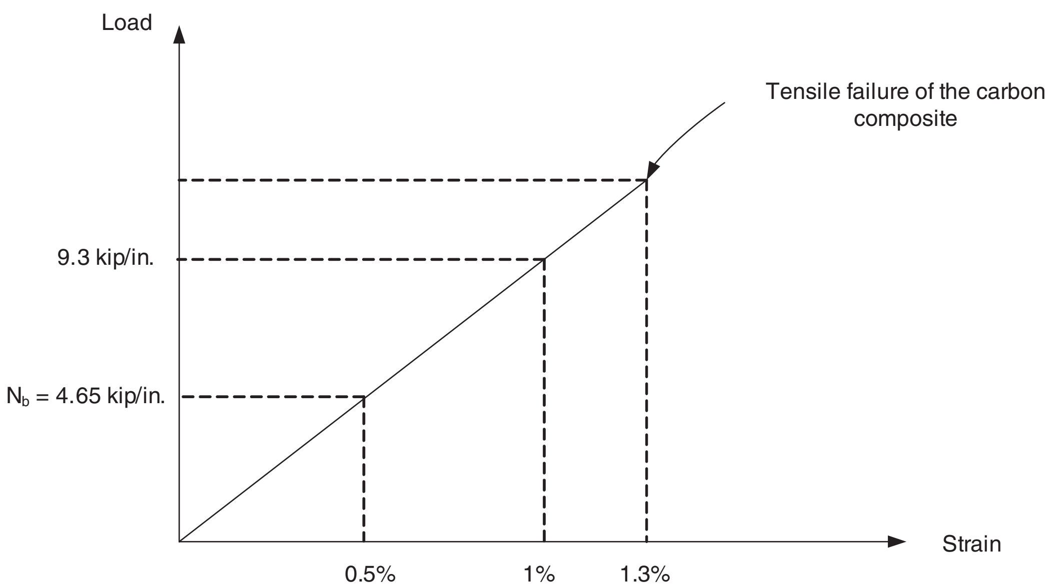

Figure B-3 FRP reinforcement stress-strain diagram for design purposes

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Step 3: Calculate the flexural strength of the T-beam

Effective depth

$$d = 30.5 - 2 - 0.5 - 1.41 = 26.59\ in.$$

Effective Flange Width

As per Article 4.6.2.6.1 of AASHTO LRFD Bridge Design Specifications, the effective flange width is taken as the minimum of

- One-quarter of the effective span length;
- Twelve times the average depth of the slab, plus the greater of web thickness or one-half the width of the top flange of the girder; or
- The average spacing of adjacent beams.

$$b_e = Minimum \begin{cases} \dfrac{l_e}{4} = \dfrac{(39)(12)}{4} = 117\ in. \\ 12t_s + b_w = 12(6) + 18 = 90\ in. \\ s = 86\ in. \end{cases}$$

$$b_e = 86\ in.$$

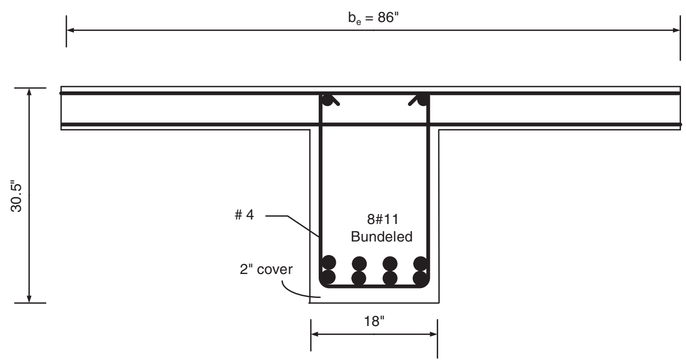

Figure B-4 Reinforced Concrete T-Beam

Assumptions:

- A rectangular stress block to represent the distribution of concrete compression stresses (Article 5.7.2.2 of AASHTO LRFD Bridge Design Specifications),
- No contribution of the steel in the compression zone to the flexural strength,
- The strain in the tension steel is greater than the yield strain, and
- The neutral axis is located in the flange of the section

Thus, the compression and tension forces are $C_c = 0.85 f_c^{'} b_e a$ and $T = A_s f_y$, respectively, as illustrated in Figure B-5.

From the condition of equilibrium of forces:

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$$0.85 f_c' b_e a = A_s f_y$$

Thus,

$$a = \frac{A_s f_y}{0.85 f_c' b_e} = \frac{12.48(40)}{0.85(3.9)(86)} = 1.75\ in.$$

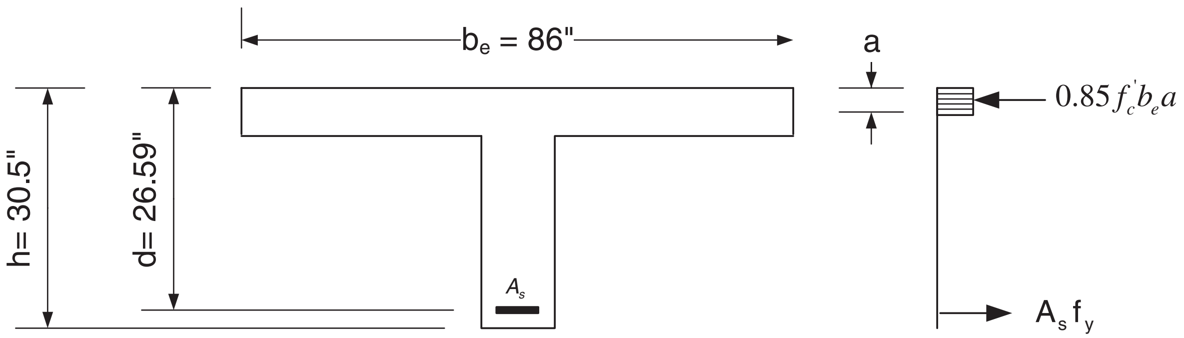

Figure B-5 Force equilibrium on a reinforced concrete T-beam

The depth of the neutral axis: $c = \frac{a}{\beta_1} = \frac{1.75}{0.85} = 2.06\ in.$

Since $c = 2.06\ in. < t_s = 6\ in.$, the assumption that depth of the neutral axis fall within the flange is appropriate.

Referring to Figure B-5, the strain in the tension steel can be computed as follows:

$$\frac{\varepsilon_s}{0.003} = \frac{d - c}{c}$$

$$\varepsilon_s = \frac{26.59 - 2.06}{2.06}(0.003) = 0.036$$

Since $\varepsilon_s = 0.036 > \frac{f_y}{E_s} = \frac{40}{29,000} = 0.00138$, the assumption that the tension steel yielded is correct.

The nominal flexural strength of the girder can then be computed from

$$M_n = A_s f_y \left(d - \frac{a}{2} \right) = (12.48)(40)\left(26.59 - \frac{1.75}{2} \right) = 12,837\ kip - in.$$

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$$\phi M_n = 0.9(12{,}837) = 11{,}553\ kip - in.$$

Check compliance with Article 1.4.4 of the proposed *Guide Specifications*

$$\phi M_n = 11{,}553\ kip - in. > M_D + M_{L+I} = 239 + 615 = 854\ kip - ft = 10{,}248\ kip - in.$$

Proceed with the design of an externally bonded FRP reinforcement system.

Step 4: Estimate the amount of FRP reinforcement required to accommodate the increase in flexural strength.

The factored moment for Strength I limit state is

$$M_u = 1.25 M_D + 1.75 M_{L+I} = 1.25(239) + 1.75(615) = 1{,}375\ kip - ft = 16{,}500\ kip - in.$$

For a preliminary estimate of the amount of FRP reinforcement necessary to resist 1,375 k-ft of moment, the following approximate design equation can be used:

$$T_{FRP} \approx \frac{M_u - \phi M_n^{unreinforced}}{h}$$

$$T_{frp} \approx \frac{(1{,}375 - 963)(12)}{30.5} = 162\ kips$$

$$T_{frp} = n N_b b_{frp}$$

Where n is the number of FRP reinforcement plates.

Use a reinforcement width of $b_{frp} = 14''$, the number of required layers is:

$$n = \frac{T_{frp}}{N_b b_{frp}} = \frac{162}{(4.65)(14)} = 2.5$$

Try 3 layers of the FRP reinforcement, for which $T_{frp} = 3(4.65)(14) = 195.3\ kips$

Step 5: Compute the factored flexural resistance of the strengthened T-beam

<u>Location of the neutral axis</u>

The depth of the neutral axis can be determined from both strain compatibility and force equilibrium conditions as follows:

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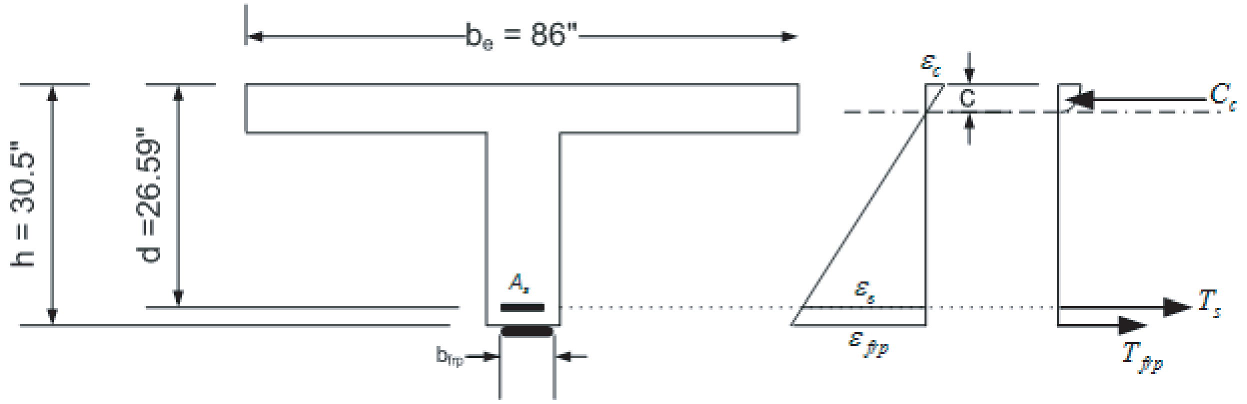

Figure B-6 Reinforced concrete T-beam externally strengthened with FRP reinforcement

Assume $c = 6\ in.$

$$\varepsilon_c = \frac{c}{h-c}\varepsilon_{FRP} = \frac{6}{30.5-6}(0.005) = 0.00122$$

$$E_c = 1{,}820\sqrt{f_c'} = 1{,}820\sqrt{3.9} = 3{,}594\ ksi$$

$$\varepsilon_o = 1.71\frac{f_c'}{E_c} = 1.71\frac{3.9}{3{,}594} = 0.00186$$

$$\frac{\varepsilon_c}{\varepsilon_o} = \frac{0.00122}{0.00186} = 0.66$$

$$\beta_2 = \frac{Ln\left[1+\left(\frac{\varepsilon_c}{\varepsilon_o}\right)^2\right]}{\left(\frac{\varepsilon_c}{\varepsilon_o}\right)} = \frac{Ln\left[1+(0.66)^2\right]}{(0.66)} = 0.548$$

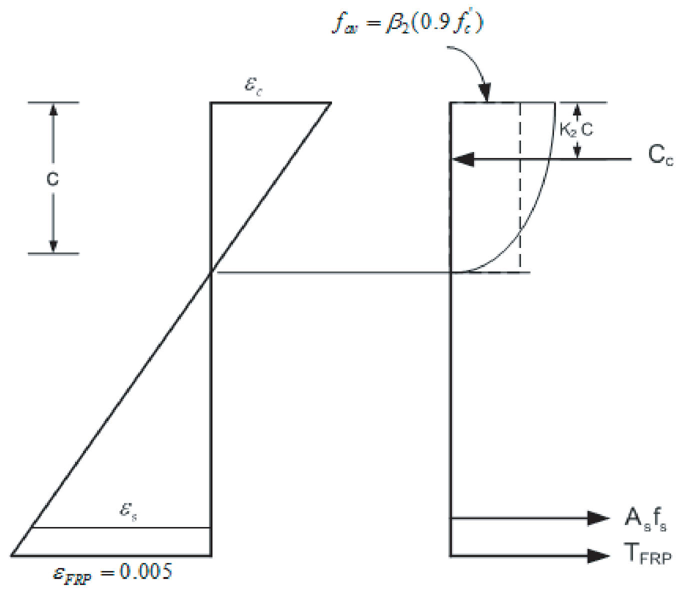

Figure B-7 Strain and stress diagrams for the reinforced concrete T-beam externally reinforced with bonded carbon fiber FRP reinforcement

<u>Compression force in the concrete:</u>

$$C_c = 0.9 f_c' \beta_2 c b_e = 0.9(3.9)(0.548)(6)(86) = 992.5\ kips$$

<u>Tension Force in the tension steel:</u>

Strain in the steel:

$$\varepsilon_s = \frac{d-c}{c}\varepsilon_c = \frac{26.59-6}{6}(0.00122) = 0.00418 > \varepsilon_y = \frac{f_y}{E} = \frac{40}{29{,}000} = 0.001379$$

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Thus,

$$T_s = A_s f_y = (12.48)(40) = 499.2\ kips$$

Tension Force in the FRP reinforcement:

$$T_{frp} = 3(4.65)(14) = 195.3\ kips$$

Total Tension Force

$$T = T_{frp} + T_s = 195.3 + 499.2 = 694.5\ kips$$

Clearly equilibrium of the forces is not satisfied $C_c - T = 992.5 - 694.5 = 298\ kips$, and the assumed depth for the neutral axis ($c = 6\ in.$) is incorrect. By trial and error, one can find that by assuming a depth of the neutral axis, $c = 4.96\ in.$, and repeating the above calculations, the following values are computed:

For $c = 4.97\ in.$

$\varepsilon_c = 0.00097$, $\varepsilon_s = 0.0042 > \varepsilon_y$, $\dfrac{\varepsilon_c}{\varepsilon_o} = 0.53$, $\beta_2 = 0.46$, $C_c = 695.2\ kips$, $T_s = 499.2\ kips$,
$T_{frp} = 195.3\ kips$, $T = T_{frp} + T_s = 195.3 + 499.2 = 694.5\ kips$, and
$C_c - T = 695.2 - 694.5 = 0.7\ kips$, close enough to zero.

The factored flexural resistance

$$M_r = 0.9\left[A_s f_s\left(d_s - k_2 c\right)\right] + \phi_{frp} T_{FRP}\left(h - k_2 c\right)$$

With $k_2 = 1 - \dfrac{2\left[\left(\dfrac{\varepsilon_c}{\varepsilon_o}\right) - \arctan\left(\dfrac{\varepsilon_c}{\varepsilon_o}\right)\right]}{\beta_2\left(\dfrac{\varepsilon_c}{\varepsilon_o}\right)^2} = 1 - \dfrac{2\left[(0.53) - \arctan(0.53)\right]}{0.46(0.53)^2} = 0.35$ and $\phi_{frp} = 0.85$

$$M_r = 0.9\left[(12.48)(40)\left(26.59 - (0.35)(4.97)\right)\right] + 0.85(195.3)\left[30.5 - (0.35)(4.97)\right] = 15{,}939\ kips - in.$$

$$M_r = 15{,}939\ kip - in. < 16{,}500\ kip - in.$$

Increase the width of the FRP reinforcement to $b_{frp} = 17\ in.$ and re-compute the flexural resistance M_r .
By doing so, we can find $c = 5.1\ in.$ and

$$M_r = 16{,}930\ kips - in > 16{,}500\ kip - in..$$

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Thus, AASHTO Strength I Load Combination limit is satisfied.

Step 6: Check ductility requirements (Article 3.4.2 of the Guide)

When reinforcing steel first yields at $\varepsilon_s = \varepsilon_y = \dfrac{f_y}{E_s} = \dfrac{40}{29,000} = 0.00138$. For such a case, the strain and stress diagrams are shown in Figure B-8.

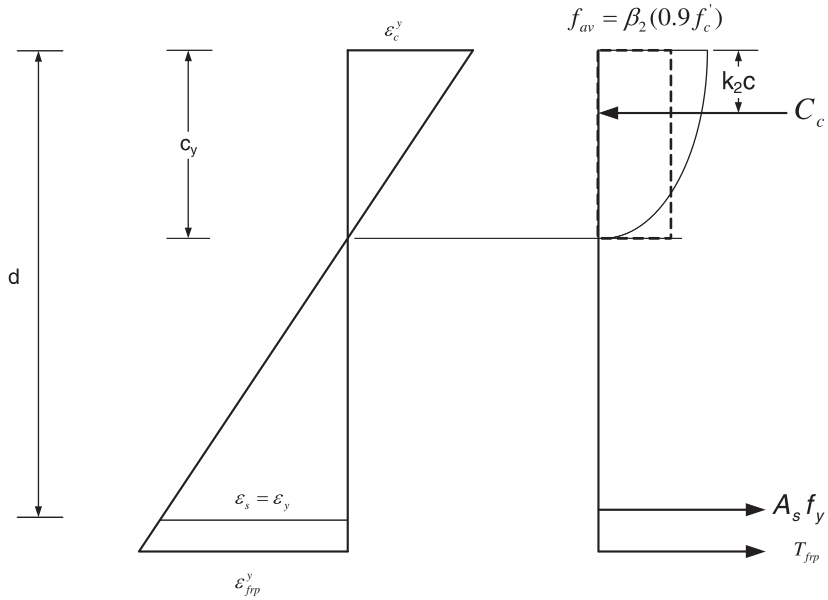

Figure B-8 Strain and stress distribution in the T-beam when tension steel reinforcement yield

By satisfying the conditions of force equilibrium and strain compatibility, the strain in the FRP reinforcement when the steel tensile reinforcement yields can be found numerically to be $\varepsilon_{frp}^y = 0.0016$. Thus, the ductility requirement of Article 3.4.2 of the guide specification is:

$$\frac{\varepsilon_{frp}^u}{\varepsilon_{frp}^y} = \frac{0.005}{0.0016} = 3.1 > 2.5 \textbf{. OK}$$

Step 7: Development length

$$L_d = \frac{T_{frp}}{\tau_{\text{int}} b_{frp}} = \frac{237.15}{0.065\sqrt{3.9}(17)} = 109 \ in. = 9.1 \ ft$$

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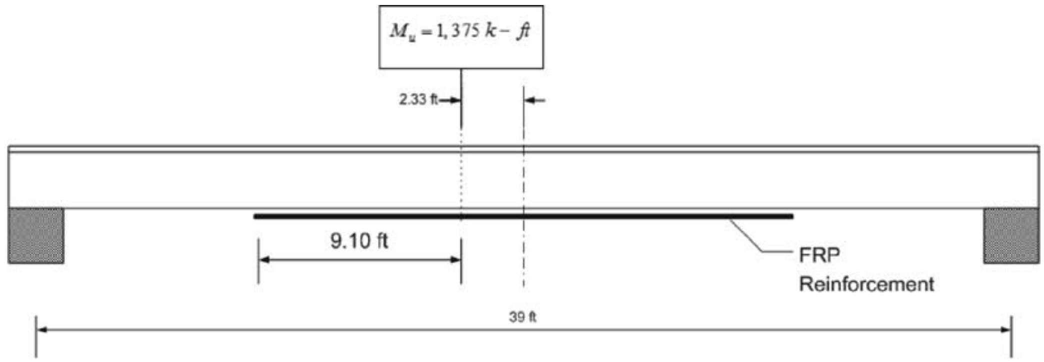
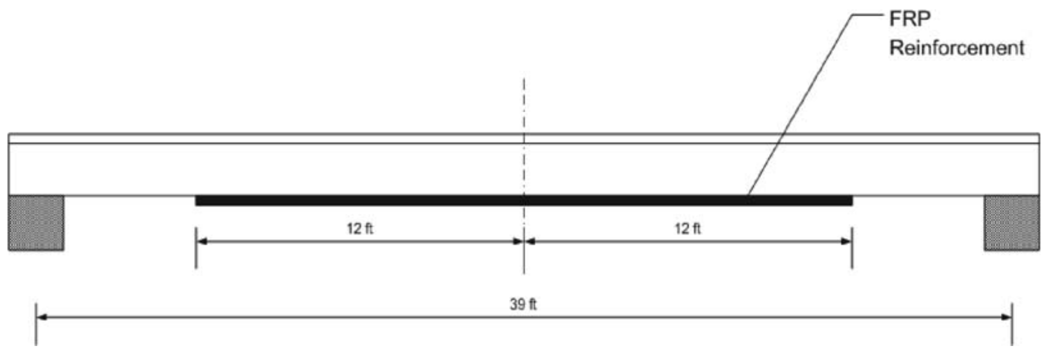


B-9 FRP Reinforcement location

Distance of FRP reinforcement end termination from the girder centerline = 9.10 + 2.33 =11.43 ft.
Use 12 ft and reinforce symmetrically as shown in Figure B-10.



B-10 FRP reinforcement Detail

Step 8: Check fatigue load combination limit state

For the fatigue load combination: $0.75M_{L+I} = 0.75(308) = 231\,kip-ft = 2{,}772\,kip-in.$

Determine the cracking moment: $M_{cr} = f_r \dfrac{I_g}{y_t}$ with $f_r = 0.24\sqrt{f_c^{'}} = 0.24\sqrt{3.9} = 0.474\ ksi$

Section Properties:

$I_g = 78{,}096\ in^4$

$y_t = 20.4\ in$

$$M_{cr} = (0.474)\frac{78{,}096}{20.4} = 1{,}815\ kip-in. < 2{,}772\ kip-in.$$

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Neglect the concrete part in tension and calculate the moment of inertia of an equivalent transformed FRP section:

From the FRP reinforcement load-strain data:

$$E_{frp} = \frac{f_{frp}}{\varepsilon_{frp}} = \frac{N_b / t_{frp}}{\varepsilon_{frp}} = \frac{4.65/(0.039)}{0.005} = 23{,}850\ ksi$$

Modular ratio for the concrete: $n_c = \dfrac{E_c}{E_{frp}} = \dfrac{3{,}594}{23{,}850} = 0.15$

Modular ratio of the steel: $n_s = \dfrac{E_s}{E_{frp}} = \dfrac{29{,}000}{23{,}850} = 1.2$

Based on the modular ratios for the concrete and for the steel, an equivalent FRP transformed section is constructed as shown below with the neutral axis assumed to lie in the flange.

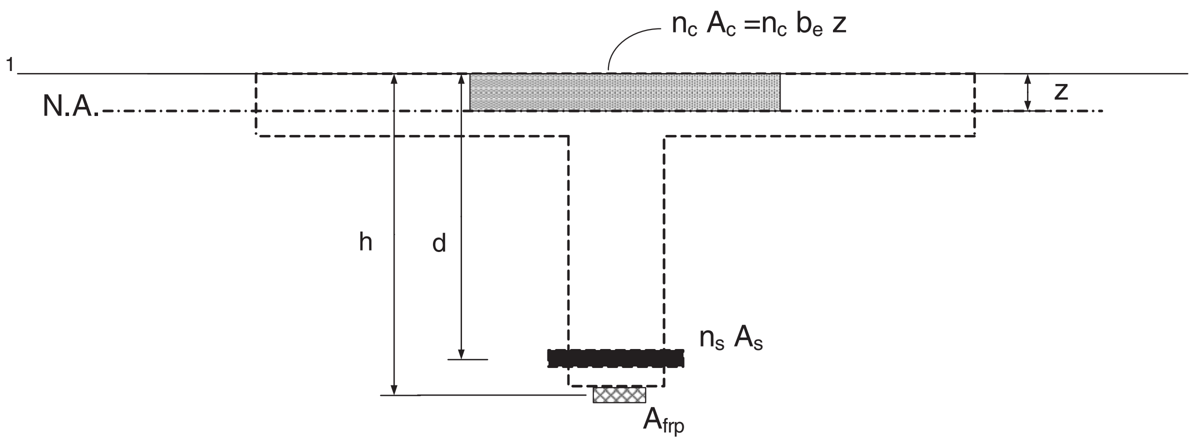

Figure B11-Equivalent FRP transformed section

By summing the moment of areas about reference line 1-1:

$$A_{frp}\left(h + \frac{t_{frp}}{2}\right) + n_s A_s d + n_c A_c \frac{z}{2} = \left(A_{frp} + n_s A_s + n_c A_c\right) z$$

$$A_{frp}\left(h + \frac{t_{frp}}{2}\right) + n_s A_s d + n_c b_e z \frac{z}{2} = \left(A_{frp} + n_s A_s + n_c b_e z\right) z$$

$$z^2 + \frac{2\left(A_{frp} + n_s A_s\right)}{n_c b_e} z - \frac{2\left[A_{frp}\left(h + \frac{t_{frp}}{2}\right) + n_s A_s d\right]}{n_c b_e} = 0$$

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$$\frac{2\left(A_{frp}+n_s A_s\right)}{n_c b_e}=\frac{2\left[(3)(17)(0.039)+1.2(12.48)\right]}{(0.15)(86)}=2.6\ in.$$

$$\frac{2\left(A_{frp}h+n_s A_s d\right)}{n_c b_e}=\frac{2\left[(3)(17)(0.039)\left(30.5+\frac{0.117}{2}\right)+(1.2)(12.48)(26.63)\right]}{(0.15)(86)}=71.22\ in.^2$$

The equation ($z^2+2.6z-71.22=0$) has the solutions of $z=7.24\ in.$ or $z=-9.84\ in.$ and only the positive solution $z=7.24\ in.$ is valid. Because $z=7.24\ in.>6\ in.$, the assumption that the neutral axis fall in the flange was incorrect.

Assume that the neutral axis is located at a distance $z>6\ in.$ By summing the moment of areas about reference line 1-1:

$$A_{frp}\left(h+\frac{nt_{frp}}{2}\right)+n_s A_s d+n_c(b_e-b_w)t_s\frac{t_s}{2}+n_c b_w z\frac{z}{2}=\left[A_{frp}+n_s A_s+n_c(b_e-b_w)t_s+n_c b_w z\right]z$$

$$z^2+\frac{2\left[\left(A_{frp}+n_s A_s+n_c(b_e-b_w)\right)t_s\right]}{n_c b_w}z-\frac{2\left[A_{frp}\left(h+\frac{nt_{frp}}{2}\right)+n_s A_s d+n_c(b_e-b_w)t_s\frac{t_s}{2}\right]}{n_c b_w}=0$$

By substituting all parameters into the above equation, the following equation is obtained

$$z^2+57.9z-476.4=0$$

Which has a positive solution $z=7.31\ in.$

The moment of inertia of the equivalent transformed FRP section can be computed to be $I_T=8,345\ in.^4$

Strain in the concrete, steel reinforcement, and FRP reinforcement, respectively, due to the fatigue load combination:

$$\varepsilon_c=\frac{M_f z}{I_T E_{frp}}=\frac{(231)(12)(7.31)}{(8,345)(23,850)}=0.00010<0.36\frac{f_c^{'}}{E_c}=0.36\frac{3.9}{3,595}=0.00039$$

$$\varepsilon_s=\frac{M_f(d-z)}{I_T E_{frp}}=\frac{(231)(12)(26.63-7.31)}{(8,345)(23,850)}=0.0003<0.8\varepsilon_y=0.8\frac{(40)}{(29,000)}=0.0011$$

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$$\varepsilon_{frp} = \frac{M_f (h + t_{frp} - z)}{I_T E_{frp}} = \frac{(231)(12)\left[30.50 + 3(0.039) - 7.31\right]}{(8,345)(23,850)} = 0.00032 < \eta \varepsilon_{frp}^{u} = 0.8(0.013) = 0.0104$$

Step 9: Check reinforcement end termination peeling

The reinforcement end terminates at a distance of 19.5-12 = 7.5 ft from each of the end supports.

It is required to calculate the moment and shear at 7.5 ft from the end support. From analysis, we will use the following combinations:

$$M_u = 1.25 M_D + 1.75 M_{L+I} = 503 \; kip - ft$$

$$V_u = 1.25 V_D + 1.75 V_{L+I} = 112 \; kips$$

Calculate the peel stress from the equation:

$$f_{peel} = \tau_{av} \left[\left(\frac{3E_a}{E_{FRP}} \right) \frac{t_{FRP}}{t_a} \right]^{1/4}$$

$$E_a = 2G_a \left(1 + \nu_a\right)$$

$$\tau_{av} = \left[V_u + \left(\frac{G_a}{E_{frp} t_{FRP} t_a} \right)^{1/2} M_u \right] \frac{t_{FRP} \left(h - z\right)}{I_T}$$

$$\tau_{av} = \left[112 + \left(\frac{185}{(23,850)(0.117)(0.125)} \right)^{1/2} (503)(12) \right] \frac{(0.117)(30.5 - 7.31)}{8,345} = 1.5 \; ksi$$

$$f_{peel} = (1.5) \left[\left(\frac{3(500)}{23,850} \right) \frac{0.117}{0.125} \right]^{1/4} = 0.740 \; ksi > 0.065\sqrt{3.9} = 0.128 \; ksi$$

Provide mechanical anchors at the FRP reinforcement ends.

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Example 3

Flexural strengthening of a simply supported cast in-place reinforced concrete girder

This example illustrates the flexural strengthening of a reinforced concrete T-beam with an externally bonded carbon fiber-reinforced polymeric reinforcement system to accommodate higher loading. This example is identical to Example 2 except that strengthening of the bridge will be carried out under the effect of the bridge dead load (stressed condition).

Bridge Data

Use the same data provided in Example 2.

SOLUTION

Step 1:

Determine if the FRP reinforcement material is in compliance with Section 2 of the Guide Specification and be sure that the glass transition temperature is higher than the maximum design temperature plus 40°F.
Based upon the location of the bridge (State of Georgia), the maximum design temperature determined from Article 3.12.2.2 of AASHTO LRFD Bridge Design Specifications is:

$$T_{MaxDesign} = 110^o F$$

Because $T_{MaxDesign} + 40^o F = 110^o F + 40^o F = 150^o F < T_g = 165^o F$, Article 2.2.4.1 of the Guide is satisfied.

Step 2: Determine the cracking moment for the T-beam

Determine the cracking moment:

$$M_{cr} = f_r \frac{I_g}{y_t}$$

$$f_r = 0.24\sqrt{f_c^{'}} = 0.24\sqrt{3.9} = 0.474\ ksi$$

$$I_g = 78{,}096\ in^4$$

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$$y_t = 20.4\ in$$

$$M_{cr} = (0.474)\frac{78,096}{20.4} = 1,815\ kip-in = 151\ kip-ft$$

Step 3: Determine the initial strain at the time of strengthening

Because $M_D = 239\ kip-ft > M_{cr} = 152.2\ kip-ft$, the cracked moment of inertia of the section will be used to compute the initial strain resulting from the dead load.

$$n_c = \frac{E_s}{E_c} = \frac{29,000}{3,594} = 8$$

Assume that the neutral axis lies inside the flange of the T-section. In such a case the depth of the neutral axis can be computed from:

$$y_N = \frac{nA_s}{b_e}\left(-1+\sqrt{1+\frac{2db_e}{nA_s}}\right) = \frac{(8)(12.48)}{86}\left(-1+\sqrt{1+\frac{2(26.59)(86)}{(8)(12.48)}}\right) = 6.79\ in. > t_s = 6\ in.$$

The assumption associated with the location of the neutral axis is not correct.

Assume that the neutral axis falls in the web at a distance y_N, from the top of the flange, which can be determined by considering a cracked transformed section as follows:

$$(86)(6)(3)+(18)(y_N-6)\left(6+\frac{y_N-6}{2}\right)+(8)(12.48)(26.59) = \left[(86)(6)+(18)(y_N-6)+(8)(12.48)\right]y_N$$

From which $y_N = 6.82\ in.$

$$I_{cr} = \frac{(86)(6)^3}{12}+(86)(6)(6.82-3)^2+(18)\frac{(6.82-6)^3}{3}+(8)(12.48)(26.59-6.82)^2 = 48,104\ in.^4$$

The initial tensile stress at the bottom concrete surface:

$$\sigma_{bo} = \frac{M_D(h-y_N)}{I_{cr}} = \frac{(239)(12)(30.5-6.82)}{48,104} = 1.41\ ksi$$

At the time of installing the externally bonded FRP reinforcement, the dead load initial strain at the bottom surface is:

$$\varepsilon_{bo} = \frac{\sigma_{bo}}{E_c} = \frac{1.41}{3,594} = 0.00039$$

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Step 4: Determine the maximum strain in the FRP reinforcement

With a maximum useable strain of 0.005 at the FRP reinforcement/concrete interface, the maximum strain in the FRP reinforcement that can be developed is:

$$\varepsilon = 0.005 - \varepsilon_{bo} = 0.005 - 0.00039 = 0.0046$$

The force per unit width in the FRP reinforcement corresponding to a strain of 0.0047 is:

$$N_{0.0047} = \frac{0.0046}{0.01}(9.3) = 4.28\ kip/in.$$

To estimate the amount of FRP reinforcement necessary to resist 1,375 k-ft of moment, the following approximate design equation can be used:

$$T_{frp} \approx \frac{M_u - \phi M_n^{unreinforced}}{h}$$

$$T_{frp} = \frac{(1{,}375 - 963)(12)}{30.5} = 162\ kips$$

$$T_{frp} = nN_b b_{frp}$$

Where n is the number of FRP reinforcement plates.

Use a reinforcement width of $b_{frp} = 17\ in.$, the number of required layers is:

$$n = \frac{T_{frp}}{N_{0.0047} b_{frp}} = \frac{162}{(4.28)(17)} = 2.23$$

Try 3 layers of the FRP reinforcement, for which $T_{frp} = 3(4.28)(17) = 218.3\ kips$

Step 5: Compute the factored flexural resistance of the strengthened T-beam

The computation procedure is similar to that of Example 2. By iteration, we find $c = 5.24\ in.$, and $M_r = 16{,}475\ kip-in. = 1{,}373\ kip-ft \approx 1{,}375\ kip-ft$

The remaining steps can be followed as presented in Example 2.

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Example 4

U-Jacket Shear strengthening of a reinforced concrete bridge

This example illustrates the shear strengthening of a reinforced concrete T-beam with an externally bonded carbon fiber-reinforced polymeric reinforcement U-jacket system to accommodate higher loading.

Bridge Data

Span: 39 ft
Type: Cast in-place reinforced concrete
Year built: 1957
Location: State of Georgia
Concrete compression Strength: $f_c^{'} = 3.9\ ksi$ (from *in-situ* testing)
Reinforcing steel yield strength: $f_y = 40\ ksi$
Girder dimensions and
Steel Reinforcement: See Figure B-12
FRP reinforcement: Shop-fabricated carbon fiber/Epoxy composite plates
Plate thickness, $t = 0.039''$
Glass Transition Temperature: $T_g = 165^o F$
Tensile strain in the FRP reinforcement at failure: $\varepsilon_{frp}^{tu} = 0.013$
Tensile strength in the FRP reinforcement at 1% strain:
$P_{frp} = 9.3\ kips / in$

$V_D = 24\ kips$, $V_{L+I} = 61\ kips$

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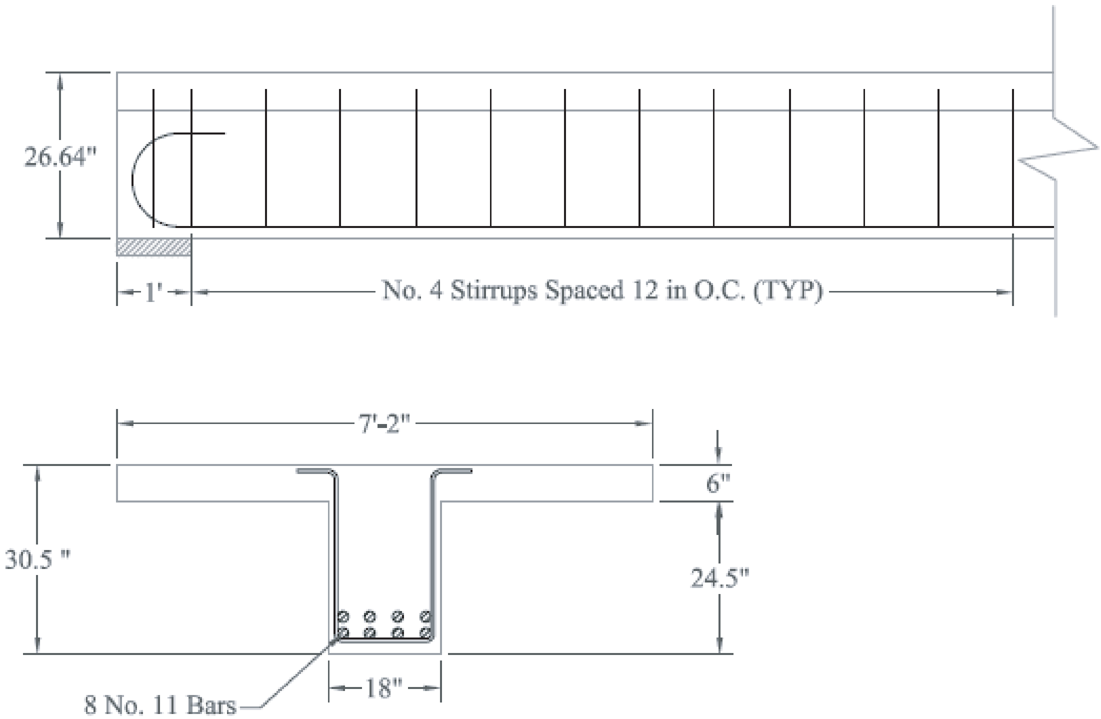

Figure B-12 Beam geometry and reinforcement

SOLUTION

Step 1: Calculate Nominal Shear Strength of Reinforced Concrete T-beam

In accordance with Article 5.8.2.9 of the AASHTO LRFD Bridge Design Specifications (2007)

The effective web width: $b_v = 18\ in.$

The effective shear depth: $d_v = Maximum\begin{cases} D_{T\&C_c} \\ 0.9d_e \\ 0.72h \end{cases}$

$D_{T\&C_c}$ = Distance between the resultants of the tensile and compressive forces due to flexure

d_e = Effective depth from extreme compression fiber to the centroid of the tensile force in the tensile reinforcement--Article 5.7.3.3.1 of AASHTO (2007)

h = Overall depth of the member

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From Example 2, $D_{T\&C_c} = 26.59 - \dfrac{1.75}{2} = 25.72\ in.,\ d_e = 26.59\ in.,\ h = 30.5\ in.$

$$d_v = Maximum\begin{cases} 25.72\ in. \\ 0.9d_e = 0.9(26.59) = 23.93\ in. \\ 0.72h = 0.72(30.5) = 21.96\ in. \end{cases}$$

$$d_v = 25.72\ in.$$

Check if the transverse reinforcement of the reinforced concrete girder meets the minimum transverse shear reinforcement specified in Article 5.8.2.5 of AASHTO (2007)

$$A_v \geq 0.0316\sqrt{f'_c}\frac{b_v S}{f_{yv}}$$

For 2#4 steel stirrups, $A_v = 0.4\ in.^2$

$$A_v = 0.4\ in^2 > 0.0316\sqrt{f'_c}\frac{b_v S}{f_{yv}} = 0.0316\sqrt{3.9}\frac{(18)(12)}{40} = 0.35 in.^2 \quad \text{O.K}$$

Nominal Shear Resistance—Article 5.8.3.3 of AASHTO (2007)

$$V_n = Minimum\begin{cases} V_c + V_s + V_p \\ 0.25 f'_c b_v d_v + V_p \end{cases}$$

Where $V_c = 0.0316\beta\sqrt{f'_c} b_v d_v$, $V_s = \dfrac{A_v f_y d_v (\cot\theta + \cot\alpha)\sin\alpha}{S}$, and $V_p = 0$ (non-prestressed girder)

Because the minimum transverse reinforcement requirement of Article 5.8.2.5 of AASHTO (2007) is met and the girder is neither prestressed nor axially loaded, the values of β and θ can be determined by the simplified procedures of Article 5.8.3.4.1 of the AASHTO LRFD Bridge Design Specifications (AASHTO, 2007). Therefore:

β= 2.0
θ= 45°

$$V_c = 0.0316(2)\sqrt{3.9}(18)(25.72) = 57.78\ kips$$

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$$V_s = \frac{(0.4)(40)(25.72)\left[\cot(45)+\cot(90)\right]\sin(90)}{12} = 34.3\ kips$$

$$V_n = Minimum \begin{cases} V_c + V_s + V_p = 57.78 + 34.3 + 0 = 92.08\ kips \\ 0.25 f'_c b_v d_v + V_p = 0.25(3.9)(18)(25.72) + 0 = 451.4\ kips \end{cases}$$

$$V_n = 92.08\ kips$$

Step 2: Estimate the amount of FRP reinforcement needed to increase the shear strength to 156 kips

$$V_u = 1.25 V_D + 1.75 V_{L+I} = 1.25(24) + 1.75(61) = 137\ kips$$

$$V_{frp} = \frac{V_u - \varphi V_n}{\varphi_{frp}} = \frac{137 - (0.9)(92.08)}{0.55} = 98.41\ kips$$

From the linear stress-strain relationship of the FRP reinforcement compute the tensile strength corresponding to a strain value of 0.004.

$$N_s = \frac{0.004}{0.01}(9.3) = 3.72\ kip/in.$$

For a U-jacket type of shear reinforcement without mechanical anchors:

$N^e_{frp} = N_s = 3.72\ kip/in.$, with $d_{frp} = 30.5 - 6 = 24.5\ in.$

If intermittent transverse shear reinforcement is used, FRP shear reinforcement shall be provided symmetrically on both sides of the member with spacing not to exceed the smaller value of 0.4 d_v or 12 inches (Article 4.3, Guide).

$$s_v = Minimum \begin{cases} 0.4 d_v = 0.4(25.56) = 10.2\ in.\ \ (Governs) \\ 12\ in. \end{cases}$$

Try 2-in wide FRP plates spaced at 10 inches apart.

$$V_{frp} = \frac{N^e_{frp} w_{frp} (\sin\alpha + \cos\alpha) d_{frp}}{s_v} = \frac{2(3.72)(2)\left[\sin(90) + \cos(90)\right](24.5)}{s_v} = \frac{364.6}{s_v}$$

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Table B4 presents the FRP reinforcement shear strength for different values of s_v

Table B4-FRP Reinforcement Shear Strength for various values of s_v

s_v (in.)	V_{frp} (kips)
10	36.5
4	91.2
3	121.5
2.75	132

Use 2-in wide FRP plates with 3 in. center-to-center. This will leave 1- in.- gaps between the plates to facilitate future inspection, using currently employed inspection techniques, of the bridge girder.

If continuous FRP shear reinforcement is provided, then

$$V_{frp} = N^e_{frp}\left(\sin\alpha + \cos\alpha\right)d_{frp} = 2(3.72)(1+0)(24.5) = 182.3\ kips\ ,$$

and future inspection will require thermography techniques.

In either case, intermittent or continuous reinforcement, the nominal shear strength provided by the externally bonded FRP shear reinforcement shall satisfy Article 4.3.5 stipulating,

$$V_s + V_{frp} \leq 8\sqrt{f_c^{'}}b_w d_{frp}$$

$$34 + V_{frp} \leq 8\sqrt{f_c^{'}}b_w d_{frp} = 8\sqrt{3.9}(18)(24.5) = 6{,}967\ kips$$

$$V_{frp} \leq 6{,}967 - 34 = 6{,}933\ kips$$

In both cases, the provision of Article 4.3.5 of the Guide is satisfied.

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Example 5

Shear strengthening Using Complete wrapping Reinforcing System

This example illustrates the shear strengthening of a reinforced concrete member with an externally bonded carbon fiber-reinforced polymeric reinforcement completely wrapped around the section that requires strengthening. The shear forces obtained from structural analysis are: $V_D = 80\ kips$ and $V_{L+I} = 100\ kips$

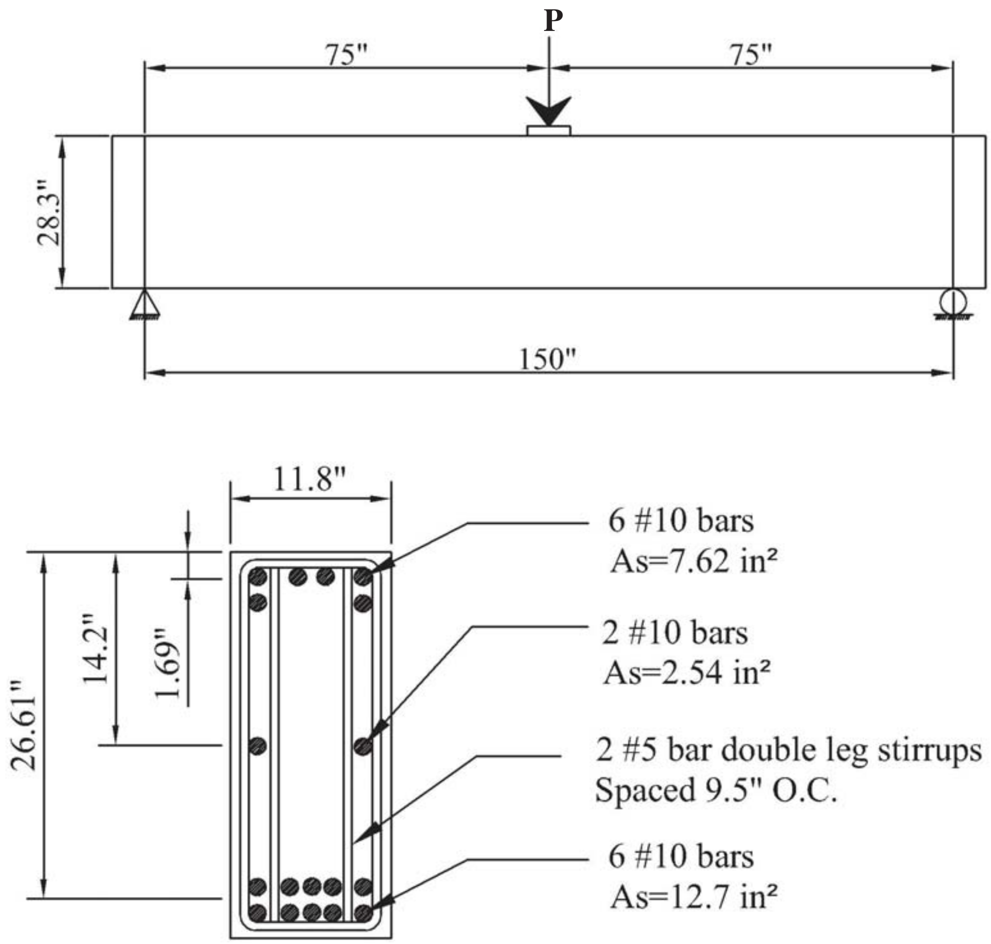

Figure B-13 Reinforcement details

Span:	150 in.
Type:	Reinforced concrete
Concrete compression Strength:	$f_c' = 3.9\ ksi$
Reinforcing steel yield strength:	$f_y = 60\ ksi$

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Steel Reinforcement: See Figure B-13.

FRP reinforcement: Field-fabricated carbon fiber/Epoxy composites

Tensile strain in the FRP reinforcement at failure:

$\varepsilon_{frp}^{tu} = 0.018$

Tensile strength in the FRP reinforcement is 0.94 kip/in. at a strain of 0.01.

SOLUTION

Step 1: Calculate nominal shear strength of the reinforced concrete member and Check compliance with Article 1.4.4 of the proposed *Guide Specifications*

In accordance with Article 5.8.2.9 of the AASHTO LRFD Bridge Design Specifications (2007)

The effective web width: $b_v = 11.8\ in.$

The effective shear depth: $d_v = Maximum \begin{cases} D_{T\&C_c} \\ 0.9d_e \\ 0.72h \end{cases}$

$D_{T\&C_c}$ = Distance between the resultants of the tensile and compressive forces due to flexure

d_e = Effective depth from extreme compression fiber to the centroid of the tensile force in the tensile reinforcement--Article 5.7.3.3.1 of AASHTO (2007)

h = Overall depth of the member

Calculation of $D_{T\&C_c}$:

Assume a rectangular concrete compression stress block for the section and establish the section force equilibrium by a trial-and-error procedure:

$$d_v = Maximum \begin{cases} 22.3\ in.\ from\ flexural\ analysis\ of\ the\ \text{section} \\ 0.9d_e = 0.9(26.61) = 23.9\ in. \\ 0.72h = 0.72(30.5) = 20.4\ in. \end{cases}$$

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$d_v = 23.9\ in.$

Check if the transverse reinforcement of the reinforced concrete girder meets the minimum transverse shear reinforcement specified in Article 5.8.2.5 of AASHTO (2007)

$$A_v \geq 0.0316\sqrt{f'_c}\,\frac{b_v S}{f_{yv}}$$

For 2 double leg #5 steel stirrups, $A_v = 1.24\ in.^2$

$$A_v = 1.24\ in.^2 > 0.0316\sqrt{f'_c}\,\frac{b_v S}{f_{yv}} = 0.0316\sqrt{3.9}\,\frac{(11.8)(9.5)}{60} = 0.11\ in.^2 \quad \text{O.K}$$

Nominal Shear Resistance—Article 5.8.3.3 of AASHTO (2007)

$$V_n = Minimum\begin{cases} V_c + V_s + V_p \\ 0.25 f'_c b_v d_v + V_p \end{cases}$$

Where $V_c = 0.0316\beta\sqrt{f'_c}\,b_v d_v$, $V_s = \dfrac{A_v f_y d_v(\cot\theta + \cot\alpha)\sin\alpha}{S}$, and $V_p = 0$ (non-prestressed girder)

Because the minimum transverse reinforcement requirement of Article 5.8.2.5 of AASHTO (2007) is met and the girder is neither prestressed nor axially loaded, the values of β and θ can be determined by the simplified procedures of Article 5.8.3.4.1 of the AASHTO LRFD Bridge Design Specifications (AASHTO, 2007). Therefore:

$\beta = 2.0$
$\theta = 45°$

$$V_c = 0.0316(2)\sqrt{3.9}(11.8)(23.9) = 35.2\ kips$$

$$V_s = \frac{(1.24)(60)(23.9)\left[\cot(45) + \cot(90)\right]\sin(90)}{9.5} = 187.2 kips$$

$$V_n = Minimum\begin{cases} V_c + V_s + V_p = 35.2 + 187.2 + 0 = 222.4\ kips \\ 0.25 f'_c b_v d_v + V_p = 0.25(3.9)(11.8)(23.9) + 0 = 275.0\ kips \end{cases}$$

$$V_n = 222.4\ kips$$

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$$V_r = \phi V_n = 0.9(222.4) = 200\ kips > V_D + V_{L+I} = 80 + 100 = 180\ kips$$

The member meets the requirement of Article 1.4.4. Thus, proceed with strengthening using externally bonded FRP shear reinforcement wrapped around the member.

Step 2: Estimate the amount of FRP reinforcement needed to increase the shear strength to 156 kips

$$V_u = 1.25V_D + 1.75V_{L+I} = 1.25(80) + 1.75(100) = 275\ kips$$

$$\left(V_{frp}\right)_{required} = \frac{V_u - V_r}{\varphi_{frp}} = \frac{275 - 200}{0.65} = 115\ kips$$

From the linear stress-strain relationship of the FRP reinforcement compute the tensile strength corresponding to a strain value of 0.004.

$$N_s = \frac{0.004}{0.01}(0.94) = 0.376\ kips$$

Check if $N_{frp,w} = 0.5N_{ut} \geq N_S$

$$N_{frp,w} = 0.5N_{ut} = (0.5)(1.69) = 0.85\ kip/in. \geq 0.376\ kip/in.$$

$$N_{frp}^{e} = N_s + \frac{1}{2}\left[N_{frp,w} - N_s\right] = 0.376 + \frac{1}{2}\left[0.85 - 0.376\right] = 0.613\ kip/in.$$

$$V_{frp} = N_{frp}^{e}(\sin\alpha + \cos\alpha)d_{frp} = 2\left(0.613\right)\left[\sin\left(90\right) + \cos(90)\right]\left(23.9\right) = 29.3\ kips$$

Number of required layers:

$$n = \frac{\left(V_{frp}\right)_{required}}{V_{frp}} = \frac{115}{29.3} = 3.9$$

Use 4 layers for which the provided $V_{frp} = 4(29.3) = 117.2\ kips$

The nominal shear strength provided by the externally bonded FRP shear reinforcement shall satisfy Article 4.3.5 stipulating,

$$V_s + V_{frp} \leq 8\sqrt{f_c^{'}}b_w d_{frp}$$

$$V_{frp} \leq 8\sqrt{f_c^{'}}b_w d_{frp} - V_s = 8\sqrt{3.9}\left(11.8\right)(23.9) - 187.2 = 4{,}268\ kips$$

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$$V_{frp} = 117.2\ kips \leq 4,268\ kips$$

The provision of Article 4.3.5 of the Guide is satisfied.

Calculate the factored shear resistance

$$V_r = \varphi\left(V_c + V_s\right) + \varphi_{frp} V_{frp} = 0.9\left(35.2 + 187.2\right) + 0.65(117.2) = 276\ kips \text{ O.K.}$$

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Example 6

Axial Strength of a confined circular column

It is required to strengthen the column shown below so that the axial compression strength is 4,000 kips.

Column Data

Column height: 24 ft
Column diameter: 42 inches
Compression Concrete Strength: 4 ksi
Vertical reinforcement: 16 #8
Type of transverse reinforcement: #3 ties
Tie spacing: 12 in.

FRP reinforcement:
Field-fabricated carbon fiber/Epoxy composites
Dry fabric weight: 9 oz/yd^2
Glass Transition Temperature: $T_g = 165^o F$
Thickness of a single layer after curing: $0.0397\ in.$
Tensile strength of a single layer FRP reinforcement at 1% strain: $P_{frp} = 3.8\ kips/in.$

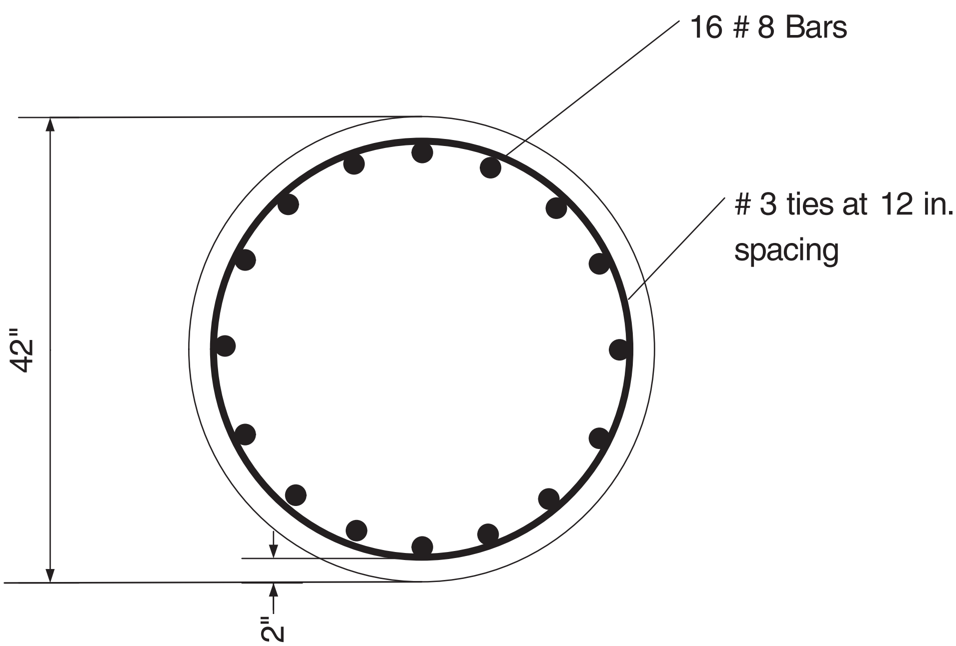

Figure B-14 Reinforcement details of a circular column

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SOLUTION

Step 1: Compute the axial strength of the column

$$P_n = 0.80\left[0.85 f_c^{'}\left(A_g - A_{st}\right) + f_y A_{st}\right]$$

$$P_r = \phi P_n = 0.75 P_n$$

$$A_g = \frac{\pi D^2}{4} = \frac{\pi (42)^2}{4} = 1385\ in.^2$$

$$A_{st} = 16(0.79) = 12.64\ in^2$$

$$P_n = 0.80\left[0.85(4)\left(1385 - 12.64\right) + (60)(12.64)\right] = 4,340\ kips$$

$$P_r = \varphi P_n = 0.75(4,340) = 3,255\ kips$$

Step 2: Compute the FRP reinforcement strength at a strain of 0.004.

$$N_{frpo} = \frac{0.004}{0.01}\left(3.8\right) = 1.52\ kip / in.$$

Step 3: Compute the required confined concrete strength

Using Eq. 5.3.1-1 of the Guide Specifications:

$$P_r = 0.80\phi\left[0.85 f'_{cc}\left(A_g - A_{st}\right) + f_y A_{st}\right] \geq P_u$$

From which

$$f_{cc}^{'} \geq \frac{\left(\frac{P_u}{0.80\phi} - f_y A_{st}\right)}{0.85\left(A_g - A_{st}\right)} = \frac{\left(\frac{4000}{(0.8)(0.75)} - (60)(12.46)\right)}{0.85\left(1385 - 12.46\right)} = 5.07\ ksi$$

$$f'_{cc} = f_c^{'}\left(1 + \frac{2 f_l}{f_c^{'}}\right) \geq 5.07\ ksi$$

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$$4\left(1+\frac{2f_l}{4}\right)\geq 5.07$$

$$f_l \geq 0.535\ ksi$$

As per Article 5.3.2.2, the confinement pressure shall be greater or equal to 600 psi but less than that specified in Eq. 5.3.3.3-2 as follows

$$f_l = 0.6\ ksi \leq \frac{f_c^{'}}{2}\left(\frac{1}{k_e\varphi}-1\right)=\frac{4}{2}\left(\frac{1}{(0.8)(0.75)}-1\right)=1.33\ ksi$$

O.K.

$$f_l = \phi_{frp}\frac{2N_{frp}}{D}$$

$$N_{frp}=\frac{f_l D}{2\varphi_{frp}}=\frac{(0.6)(42)}{2(0.65)}=19.38\ kip/in.$$

Required number of layers

$$n=\frac{N_{frp}}{N_{frpo}}=\frac{19.38}{1.52}=12.75$$

Use 13 layers that will have a thickness of $0.507\ in.$.

Accordingly, the column axial strength is computed as follows:

$$f_l = \phi_{frp}\frac{2N_{frp}}{D}=0.65\frac{2(13)(1.52)}{42}=0.611\ ksi \leq \frac{f_c^{'}}{2}\left(\frac{1}{k_e\phi}-1\right)=1.33\ ksi$$

$$f'_{cc}=f'_c\left(1+\frac{2f_l}{f_c^{'}}\right)=(4)\left(1+\frac{2(0.611)}{4}\right)=5.22\ ksi$$

$$P_n = 0.80\left[0.85f'_{cc}\left(A_g - A_{st}\right)+f_y A_{st}\right]=0.8\left[0.85(5.22)(1385-12.46)+(60)(12.46)\right]=5470\ kips$$

$$P_r=\phi P_n = 0.75(5{,}470)=4{,}102\ kips > 4{,}000\ kips$$

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Abbreviations and acronyms used without definitions in TRB publications:

AAAE	American Association of Airport Executives
AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
ACI–NA	Airports Council International–North America
ACRP	Airport Cooperative Research Program
ADA	Americans with Disabilities Act
APTA	American Public Transportation Association
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
ASTM	American Society for Testing and Materials
ATA	Air Transport Association
ATA	American Trucking Associations
CTAA	Community Transportation Association of America
CTBSSP	Commercial Truck and Bus Safety Synthesis Program
DHS	Department of Homeland Security
DOE	Department of Energy
EPA	Environmental Protection Agency
FAA	Federal Aviation Administration
FHWA	Federal Highway Administration
FMCSA	Federal Motor Carrier Safety Administration
FRA	Federal Railroad Administration
FTA	Federal Transit Administration
HMCRP	Hazardous Materials Cooperative Research Program
IEEE	Institute of Electrical and Electronics Engineers
ISTEA	Intermodal Surface Transportation Efficiency Act of 1991
ITE	Institute of Transportation Engineers
NASA	National Aeronautics and Space Administration
NASAO	National Association of State Aviation Officials
NCFRP	National Cooperative Freight Research Program
NCHRP	National Cooperative Highway Research Program
NHTSA	National Highway Traffic Safety Administration
NTSB	National Transportation Safety Board
PHMSA	Pipeline and Hazardous Materials Safety Administration
RITA	Research and Innovative Technology Administration
SAE	Society of Automotive Engineers
SAFETEA-LU	Safe, Accountable, Flexible, Efficient Transportation Equity Act: A Legacy for Users (2005)
TCRP	Transit Cooperative Research Program
TEA-21	Transportation Equity Act for the 21st Century (1998)
TRB	Transportation Research Board
TSA	Transportation Security Administration
U.S.DOT	United States Department of Transportation

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