



GUIDE SPECIFICATIONS FOR **DESIGN OF BONDED FRP SYSTEMS** FOR REPAIR AND STRENGTHENING OF CONCRETE BRIDGE ELEMENTS

March 2023 | Second Edition





American Association of State Highway and Transportation Officials
555 12th Street, N.W., Suite 1000
Washington, DC 20004
202-624-5800 phone/202-624-5806 fax
www.transportation.org

Cover photos: Top right courtesy of Matt Chynoweth, Michigan Department of Transportation; top left courtesy of Oregon Department of Transportation; bottom right courtesy of Michael Goff.

© 2023 by the American Association of State Highway and Transportation Officials. All rights reserved.
Duplication is a violation of applicable law.

ISBN: 978-1-56051-807-5

Pub Code: FRPS-2



© 2023 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS
555 12th Street, N.W., Suite 1000
Washington, D.C. 20004

**EXECUTIVE COMMITTEE
2022–2023**

OFFICERS:

PRESIDENT: Roger Millar, Washington*

VICE PRESIDENT: Craig Thompson, Wisconsin**

SECRETARY-TREASURER: Russell McMurry, Georgia

EXECUTIVE DIRECTOR: Jim Tymon, Washington, D. C.

REGIONAL REPRESENTATIVES:

REGION I: Nicole Majeski, Delaware
Everett Lott, District of Columbia

REGION II: Vacant
Lorie Tudor, Arkansas

REGION III: Nancy Daubenberger, Minnesota
Jack Marchbanks, Ohio

REGION IV: Vacant
Joel Jundt, South Dakota

IMMEDIATE PAST PRESIDENT: Shawn Wilson, Louisiana

***Elected at the 2022 Annual Meeting**

****Elected at the 1/5/2023 Special Virtual Meeting**

AASHTO Committee on Bridges and Structures, 2022

CARMEN SWANWICK, *Chair*

SCOT BECKER, *Vice Chair*

JOSEPH L. HARTMANN, *Federal Highway Administration, U.S. DOT Liaison*

PATRICIA J. BUSH, *AASHTO Liaison*

ALABAMA, Eric J. Christie, William “Tim” Colquett, Jeffrey Huner

ALASKA, Leslie Daughtery, Elmer E. Marx, Richard A. Pratt

ARIZONA, David B. Benton, David L. Eberhart, Pe-Shen Yang

ARKANSAS, Charles “Rick” Ellis, Andy Nannemar, Steve Peyton

CALIFORNIA, Erol Kaslan, Ruth Fernandes, Gedmund Setberg,

COLORADO, Michael Collins, Stephen Harelson, Jessica Martinez

CONNECTICUT, Andrew Cardinali, Timothy D. Fields, Bart Sweeney

DELAWARE, Jason Arndt, Jason Hastings, Craig A. Stevens

DISTRICT OF COLUMBIA, Donald L. Cooney, Konjit C. “Connie” Eskender

FLORIDA, Felix Padilla, William Potter

GEORGIA, Donn Digamon, Doug Franks, Steve Gaston

HAWAII, James Fu, Nicholas Groves

IDAHO, Matthew Farrar

ILLINOIS, Tim A. Armbrrecht, Ruben Boehler, Jayme Schiff

INDIANA, Andrew Fitzgerald, Jennifer Hart, Anne M. Rearick, Stephanie Wagner

IOWA, Jim Hauber, James S. Nelson, Michael Nop

KANSAS, Mark Hurt, Karen Peterson, Calvin Reed

KENTUCKY, Michael Carpenter, Royce Meredith, Carl Van Zee

LOUISIANA, Arthur D’Andrea, Chris Guidry, Zhengzheng “Jenny” Fu

MAINE, Jeffrey S. Folsom, Wayne Frankhauser, Michael Wight

MARYLAND, Maurice Agostino, Jesse Creel, Jeffrey L. Robert

MASSACHUSETTS, Alexander K. Bardow, Justin Slack, Matthew Weidele

MICHIGAN, Matthew Chynoweth, Rebecca Curtis, Richard E. Liptak

MINNESOTA, Arielle Ehrlich, Ed Lutgen, Kevin Western

MISSISSIPPI, Micah Dew, Bradnado Turnquest, Scott Westerfield

MISSOURI, David Hagemeye, Bryan Hartnagel, Darren Kemna

MONTANA, Stephanie Brandenberger, Nathan Haddick, Amanda Jackson

NEBRASKA, Ross Barron, Fouad Jaber, Kyle Zillig

NEVADA, David Chase, Jessen Mortensen

NEW HAMPSHIRE, Loretta Doughty, David L. Scott

NEW JERSEY, Xiaohua “Hannah” Cheng, Eric Yermack

NEW MEXICO, Shane Kuhlman, Jeff C. Vigil

NEW YORK, Brenda Crudele, James Flynn, Ernest Holmberg

NORTH CAROLINA, Brian Hanks, Scott Hidden, Girchuru Muchane

NORTH DAKOTA, Jon D. Ketterling, Jason R. Thorenson

OHIO, Alexander B.C. Dettloff, Timothy J. Keller, Jeffrey Syar

OKLAHOMA, Justin Hernandez, Walter Peters, Tim Tegeler

OREGON, Raymond Mabey, Albert Nako, Tanarat Potisuk

PENNSYLVANIA, Richard Runyen, Shane Szalankiewicz

PUERTO RICO, (Vacant)

RHODE ISLAND, Keith Gaulin, Mary Vittoria-Bertrand

SOUTH CAROLINA, Terry B. Koon, Hongfen Li, Jeff Sizemore

SOUTH DAKOTA, Steve Johnson, Dave Madden, Todd Thompson

TENNESSEE, Ted A. Kniazewycz

TEXAS, Graham Bettis, Bernie Carrasco, Jamie F. Farris

UTAH, Mark Daniels, Rebecca Nix, Cheryl Hersh Simmons

VERMONT, Carolyn Cota, Kristin M. Higgins, Bob Klinefelter, Jim Lacroix

VIRGINIA, Kendal R. Walus, Andrew M. Zickler

WASHINGTON, Tony M. Allen, Mark A. Gaines, Bijan Khaleghi

WEST VIRGINIA, Tracy Brown, Chad Robinson

WISCONSIN, Aaron Bonk, Josh Dietsche, William Oliva

WYOMING, Jeff R. Booher, Paul G. Cortez, Michael Menghini

Associate Members

CHESAPEAKE BAY BRIDGE AND TUNNEL DISTRICT, Timothy Holloway

MARYLAND TRANSPORTATION AUTHORITY, James Harkness

MULTNOMAH COUNTY TRANSPORTATION DIVISION, Jon Henrichsen

TRANSPORTATION RESEARCH BOARD, Ahmad Abu-Hawash

UNITED STATES COAST GUARD, Kamal Elnahal

U.S. ARMY CORPS OF ENGINEERS, Phillip Sauser

This page intentionally left blank.

SECTION 1: INTRODUCTION

TABLE OF CONTENTS

1.1—SCOPE 1-1

1.2—FRP Systems 1-1

1.3—SUBSTRATE REPAIR AND SURFACE PREPARATION 1-2

1.4—INSTALLATION OF FRP 1-3

 1.4.1—Wet Lay-Up FRP Systems 1-3

 1.4.2—Pre-Cured FRP Systems 1-4

 1.4.3—NSM FRP Systems 1-4

1.5—INSPECTION, EVALUATION, AND ACCEPTANCE 1-5

1.6—REPAIR OF DEFECTS 1-6

1.7—REFERENCES 1-7

This page intentionally left blank.

SECTION 1: INTRODUCTION

1.1—SCOPE

These Guide Specifications are intended for the repair and strengthening of reinforced and prestressed highway bridge structures using externally bonded fiber reinforced polymer (FRP) composite systems. These Guide Specifications supplement the *AASHTO LRFD Bridge Design Specifications*, Ninth Edition, 2020 (AASHTO LRFD). Except where specifically provided below, all provisions of AASHTO LRFD shall apply.

These Guide Specifications state 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.

These specifications employ 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 AASHTO LRFD or the provisions in the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*.

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

1.2—FRP SYSTEMS

In general, procedures for the installation of FRP systems are developed by the manufacturer and can vary between the available systems. Two optional types of externally bonded FRP systems are available. The first type consists of dry fiber-fabric sheets saturated with an epoxy resin. The second type of system consists of precured laminates (manufacturer fabricated), in the form of plates, shells, and rods.

The dry fiber-fabric sheets are commonly applied using a wet-layup process, where the fiber-fabric sheets are saturated using an impregnating resin on site. Prepreg systems, where the fiber-fabric sheets are saturated off-site but cured

C1.1

This Article discusses the scope of these Guide Specifications, their applicability, and their limitations. This Article is analogous to the opening articles, Articles X.1, of each of the Sections of AASHTO LRFD.

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 specifications. However, references to North American and international guidelines (ACI 440.2R, CSA S806-12, 2017; fib technical report bulletin 90, 2019; CNR-DT 200 R1, 2013; Technical Report No. 55, 2012; JSCE, 2001; and AFGC, 2003) 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.

C1.2

The wet-layup system is the more versatile of these systems because the fabric/cloth is flexible and can conform to most shapes in the field. Installation procedures may also vary depending on the type and condition of the structure to be strengthened. The application of FRP systems will not stop the ongoing corrosion of existing steel reinforcement. The cause of corrosion to internal steel reinforcement should be addressed and corrosion-related deterioration should be repaired prior to application of any FRP system.

NSM FRP systems are advantageous as they are less prone to delamination and environmental degradation, and allow for effective inspection

in place by heating or by other methods are also available. Pre-cured laminate plates and rod panels are bonded to the concrete surface with an adhesive resin. In near-surface mounted (NSM) FRP systems, small diameter pre-cured rods and plates/strips are installed and bonded into grooves on the concrete surface using adhesive resin. Pre-cured shells are applied around columns to provide confinement. Pre-cured systems comprising sheets and grids with multi-directional fiber reinforcement are also available.

1.3—SUBSTRATE REPAIR AND SURFACE PREPARATION

All repairs to the concrete substrate shall be carried out according to applicable specifications prior to FRP application. Externally bonded FRP strengthening shall not be applied on concrete suspected of containing corroded reinforcement. Corroded reinforcement shall be cleaned, and the surrounding contaminated concrete shall be removed (ICRI 310.1R-08 and ACI 440.2R). Concrete removal shall extend along the reinforcing steel until there is no further delamination, cracking, or corrosion and the reinforcing steel is well bonded to the surrounding concrete (ICRI 310.1R-08). Any other defective concrete within the FRP application area shall be removed according to ICRI 310.1R-08. Replacement of concrete shall be carried out with material conforming to ICRI 320.2R-18. The repair material shall have a compressive strength equal to or greater than the original concrete, but not less than 4.5 ksi and 5.5 ksi at 7 and 28 days, respectively. Cracks wider than 0.01 in. and with spacing less than 1.5 in. or cracks wider than $\frac{1}{32}$ in. shall be filled using epoxy injection (NCHRP Report 609).

The concrete surface shall be prepared to a minimum concrete surface profile (CSP) 3 as defined by the ICRI-surface-profile chips (ICRI 310.2R-13, NCHRP Report 609). Localized out-of-plane variations (suggested to be taken as any 6 in. by 6 in. section) throughout the concrete surface where the FRP system is applied, including form lines, shall not exceed $\frac{1}{32}$ in., or the tolerances recommended by the FRP system manufacturer, whichever is smaller. Holes and voids shall be filled with a compatible epoxy paste (thickened epoxy/putty). It is recommended that surface preparation be accomplished using abrasive or water-blasting techniques. All laitance, dust, dirt, oil, curing compound, existing coatings, and any other matter

of concrete surfaces compared to the wet-layup system. NSM FRP systems have a greater anchoring or bond capacity compared to the surface bonded FRP systems, and may be more suitable for cases in which the concrete surface requires significant surface preparation.

C1.3

The bond behavior of the FRP system is highly dependent on a sound concrete substrate and on the workmanship of the installation; either one or both can significantly influence the integrity of the FRP strengthening system. Proper preparation and profiling of the concrete substrate is necessary to achieve optimum bond strength. Improper surface preparation can lead to premature debonding or delamination.

that could interfere with the bond between the FRP system and concrete substrate shall be removed.

When fibers are wrapped around corners, the corners shall be rounded to a minimum $\frac{1}{2}$ -in. radius to prevent stress concentrations in the FRP system and voids between the FRP system and the concrete. Rough edges shall be smoothed by grinding or with putty prior to FRP application. The substrate repair, including any crack injections, and surface preparation shall be inspected by a licensed engineer or qualified inspector knowledgeable in FRP systems prior to FRP installation.

1.4—INSTALLATION OF FRP

Unless approved by the Engineer of Record, externally bonded FRP shall not be applied if rain, snow, or dew point condensation is expected. Primers, putty, saturating resin, or adhesives shall not be applied on cold, frozen, damp, or wet surfaces. Ambient and concrete surface temperature shall be between 50–90 degrees F unless specified by the manufacturer. Enclosures and external heat sources may be utilized to meet the required environmental conditions.

Mixing of all resin, primer, and putty shall be carried out at the manufacturer-specified temperature range, mix proportions, and mix durations. Any resin that exceeds its pot life or shows sign of increased viscosity shall not be used.

1.4.1—Wet Lay-Up FRP Systems

A primer coating shall be applied if required for the FRP system. The saturating resin shall ensure full impregnation of the fabric sheet. Manufacturer recommendations shall be followed for impregnating the fabric. The saturated fabric is placed on the concrete substrate and rolled in the direction of the fibers to remove entrapped air. An

C1.4

When concrete and atmospheric temperatures exceed 90 degrees F, difficulties may be experienced in application of the epoxy compound owing to acceleration of the reaction and hardening rates. If ambient temperatures above 90 degrees F are anticipated, work should be scheduled when the temperature is lower, such as in the early morning hours. If it is necessary to apply epoxy compounds at temperatures exceeding 90 degrees F, the work should be supervised by a person experienced in applying epoxy at high temperatures. Epoxy systems formulated for elevated temperature are available.

At low temperatures, difficulties may occur due to deceleration of the reaction rates. Manufacturer-recommended substrate temperatures or a minimum temperature of 50 degrees F and rising is recommended during installation. With manufacturer approval, specialized devices, such as warming blankets, may be used. The presence of frost or ice crystals may also be detrimental to the bond between the FRP and the concrete.

NCHRP Report 609 and ICRI 330.2-16 provide specifications for the installation of externally bonded FRP reinforcement for strengthening concrete structures.

C1.4.1

The primer may be applied by a clean roller or brush. When applied uniformly, the bond between concrete substrate and FRP system is improved. If a primer is not required, a first coat of saturating resin should be applied as an undercoat to all locations of the concrete surface where the FRP is to be applied. The rate of coverage for the primer and saturating

additional saturating resin coating shall be applied over the top of the fabric to ensure full saturation of the fibers. The fabric sheets shall be aligned with the fibers in the direction indicated on the design drawings or other contract documents. For multi-ply applications, the steps shall be repeated for each additional fabric sheet. The installed fibers shall be free of kinks, wrinkles, and waviness.

resin is listed in the system specifications or data sheet. Special equipment, such as saturators, may be used on large projects to evenly impregnate the fabric and to speed up the process.

A “dry” lay-up, where a first coat of saturating resin is applied on to the substrate and the fabric is installed without pre-impregnation (unsaturated) can be carried out when applying a single “thin” fiber sheet. Here, it is important to apply sufficient coating of saturating resin over the installed fabric to fully impregnate the fibers. The Engineer of Record should approve the use of dry lay-up prior to installation.

In multi-ply applications, each new ply should be supported by the previously-applied system and be applied prior to the gelation of the previous layer. If the previous layer has cured, new plies of fabric sheets may only be applied after surface preparation per manufacturer instructions. Too many plies applied in a single day could result in slippage or separation due to self-weight of the saturated fabric sheets (NCHRP Report 609). Fiber misalignment can affect strength of the FRP system. Anchors such as U-wraps and FRP spike anchors will provide anchorage against premature delamination of the FRP system.

1.4.2—Pre-Cured FRP Systems

For pre-cured FRP laminate plates, depending on manufacturer specifications, the adhesive resin shall be applied to the required thickness on the concrete substrate, the FRP laminate, or both. The laminate shall be pressed on to the concrete using a hard rubber roller or similar tool until any entrapped air is released and excess adhesive resin is forced out on both sides. The manufacturer-specified bond thickness shall be maintained during the installation of pre-cured FRP laminates.

For FRP rod panels, the adhesive resin is applied on to the concrete surface to the manufacturer-specified thickness. The rod panels are pressed into the adhesive resin until the FRP rods are covered by the resin. Excess resin is spread uniformly over the rod panels.

C1.4.2

The adhesive resin may be applied using a spatula or any other suitable method. Splicing of pre-cured FRP laminate plates using a splice plate is not recommended. FRP rod panels facilitate modular construction suitable for locations with limited access and can be spliced using a finger joint.

1.4.3—NSM FRP Systems

In NSM FRP, pre-cured FRP rods/bars or plates/strips are installed into pre-cut or ground grooves in the concrete. The grooves shall be cleaned prior to FRP installation to remove all laitance that would

C1.4.3

NSM FRP systems allow for strengthening in negative moment regions without exposure to mechanical or abrasion damage. Compared to other externally bonded FRP systems, NSM FRP

prevent sufficient bonding with the FRP. The groove shall be half-filled with adhesive resin for NSM FRP, and the FRP rod or strip is pressed into the groove to force the resin to flow around it and completely fill the space between the FRP and sides of the groove. The groove shall then be fully filled with additional adhesive resin and the surface is leveled.

1.5—INSPECTION, EVALUATION, AND ACCEPTANCE

FRP strengthening is performed on structurally deficient or damaged reinforced or prestressed concrete bridge elements. Before a strengthening procedure is implemented, the extent of deficiency and suitability of FRP strengthening shall be evaluated. Systems considered for FRP strengthening shall be inspected by a licensed engineer or qualified inspector knowledgeable in FRP systems and installation procedures. The following shall be recorded at the time of installation:

- date and time of installation;
- ambient temperature, relative humidity, and general weather observations and surface temperature of concrete;
- surface dryness, surface preparation methods, and resulting profile using the ICRI-surface-profile chips;
- qualitative description of surface cleanliness;
- type of auxiliary heat source, if applicable;
- widths of cracks not injected with epoxy;
- fiber-fabric sheets or pre-cured laminate batch number(s) and approximate locations in the structure;
- batch numbers, mixture ratios, mixing times, and qualitative descriptions of the appearance of all mixed resins, including primers, putties, saturants, adhesives, and coatings mixed for the day;
- observations of progress of cure of resins;
- conformance with installation procedures;
- location and size of any delaminations or air voids;
- general progress of work;

systems also have better durability under long-term environmental exposure.

C1.5

Evaluation of moisture content or outgassing of the concrete should be done by determining if moisture will collect at bond lines between old concrete and epoxy adhesive before epoxy has cured. This may be accomplished by taping an 18 in. × 18 in. polyethylene sheet to the concrete surface. If moisture collects on the underside of the polyethylene sheet before epoxy would cure, then allow concrete to dry sufficiently to prevent the possibility of a moisture barrier between old concrete and new epoxy (ASTM D4263).

During installation, sample cups of mixed resin should be prepared according to a predetermined sampling plan and retained for testing to determine level of curing in accordance with ASTM D2583. The relative cure of the resin can also be evaluated on the project site by physical observation of resin tackiness and hardness of work surfaces or hardness of retained resin samples.

For bond-critical applications, tension adhesion testing of cored samples should be conducted using the methods in ACI 503R or ASTM D7522/D7522M. The sampling frequency should be specified. Tension adhesion strengths should exceed 200 psi and exhibit failure of the concrete substrate (ACI 440.2R).

The necessary evaluation criteria for repair of existing concrete structures and post-repair evaluation criteria are well established in the following documents:

- ACI 201.1R: *Guide for Conducting a Visual Inspection of Concrete in Service*
- ACI 224.1R: *Causes, Evaluation, and Repair of Cracks in Concrete*
- ACI 364.1R: *Guide for Assessment of Concrete Structures Prior to Rehabilitation*

- level of curing of resin in accordance with ASTM D2582; and
- adhesion strength.
- ACI 440.2R: *Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures*
- ACI 503R: *Use of Epoxy Compounds with Concrete*
- ACI 546.3: *Guide to Materials Selection for Concrete Repair*
- ICRI 310.1R: *Guide for Surface Preparation for the Repair of Deteriorated Concrete Resulting from Reinforcing Steel Corrosion*
- ICRI 320.2R: *Guide for Selecting and Specifying Materials for Repairs of Concrete Surfaces*
- NCHRP Report 609: *Recommended Construction Specifications Process Control Manual for Repair and Retrofit of Concrete Structures Using Bonded FRP Composites*

Relevant specifications and guidelines provided by FRP manufacturers should also be carefully reviewed prior to the design of any strengthening system.

1.6—REPAIR OF DEFECTS

The repair of defects highlighted herein is applicable only to wet-layup FRP systems. Repair of other FRP systems shall be submitted for approval by a licensed engineer. Small defects no larger than $\frac{1}{4}$ in. diameter do not need corrective action unless they occur next to edges or when there are more than five such defects in an area of 10 ft². Small voids between $\frac{1}{4}$ in.– $1\frac{1}{4}$ in. in diameter shall be repaired in accordance with ICRI 330.2 by epoxy injection, provided the defect does not extend the entire thickness of the laminate in multi-ply applications.

Defects larger than $1\frac{1}{4}$ in. in diameter are considered large defects. An area of at least 1 in. beyond the defect shall be carefully removed and covered using an FRP patch of similar material. The substrate shall be prepared in accordance with Article 1.3 and the FRP material applied in accordance with Article 1.4.1, except the patch shall extend the lap length for the FRP material or a minimum of 6 in. on all sides of the removed area. These repairs are only applicable to defects in a single ply application. Unless repair is approved by the Engineer of Record, full replacement of FRP system shall be carried out if large defects are found in multi-ply FRP systems.

C1.6

Based on NCHRP Report 609, defects in FRP systems may include:

- voids between the concrete and the FRP system,
- delamination or voids within layers of the FRP system,
- broken or damaged edges of the FRP system,
- wrinkling or buckling of fibers and fiber tows,
- fracture of fibers, breakage in fabric,
- cracks, blisters, and peeling of the protective coating,
- resin-starved areas with non-uniform impregnation,
- under-cured or incompletely cured resin, and
- incorrect fiber orientation.

Large defects, such as debonding between layers of multi-ply FRP systems, may require full replacement. Depending on the size of the defect and whether the area is critical to structural integrity, it may be advisable to remove and re-apply the entire FRP system (NCHRP Report 609).

1.7—REFERENCES

- AASHTO. *AASHTO LRFD Bridge Design Specifications*, 9th Ed. LRFDDBS-9. American Association of State Highway and Transportation Officials, Washington, DC, 2020.
- AASHTO. *LRFD Bridge Construction Specifications*, 4th Ed., with 2020 and 2022 Interim Revisions. LRFDCONS-4. American Association of State Highway and Transportation Officials, Washington, DC, 2017.
- AASHTO. *AASHTO Guide Specifications for LRFD Seismic Bridge Design*, 2nd Ed., with 2012, 2014, 2015, and 2022 Interim Revisions. LRFDSEIS-2. American Association of State Highway and Transportation Officials, Washington, DC, 2011.
- ACI. *Causes, Evaluation, and Repair of Cracks in Concrete Structures*, 224.1, American Concrete Institute, Farmington Hills, MI, 2007.
- ACI. *Evaluation of Concrete Structures Prior to Rehabilitation*, 364.1R. American Concrete Institute, Farmington Hills, MI, 2019.
- ACI. *Guide for Conducting a Visual Inspection of Concrete in Service*, 201.1R, American Concrete Institute, Farmington Hills, MI, 2008.
- ACI. *Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures*, 440.2R. American Concrete Institute, Farmington Hills, MI, 2017.
- ACI. *Guide for Evaluation of Concrete Structures before Rehabilitation*, 364.1R, American Concrete Institute, Farmington Hills, MI, 2007.
- ACI. *Guide to Materials Selection for Concrete Repair*, 546.3, American Concrete Institute, Farmington Hills, MI, 2014.
- ACI. *Use of Epoxy Compounds with Concrete*, 503R (Reapproved 2008), American Concrete Institute, Farmington Hills, MI, 2008.
- AFGC. *Réparation et Renforcement des Structures en Béton au Moyen des Matériaux Composites*, Association Française de Génie Civil, Bagneux Cedex, France, 2003.
- ASTM. D2582, Standard Test Method for Puncture-Propagation Tear Resistance of Plastic Film and Thin Sheeting. ASTM International, West Conshohocken, PA.
- Canadian Standards Association (CSA). *Design and construction of building components with fibre-reinforced polymers*, CSA-S806-12(R2017), Mississauga, ON., Canada, 2017.
- Concrete Society. *Design guidance for strengthening concrete structures using fibre composite material*, Technical Report No. 55, Surrey, UK, 2012.
- Fédération Internationale du Béton (fib). *Externally bonded FRP reinforcement for RC structures*, Task Group 5.1, Bulletin No. 90, Technical Rep., Lausanne, Switzerland, 2019.
- ICRI. 310.1R, Guideline for Surface Preparation for the Repair of Deteriorated Concrete Resulting from Reinforcing Steel Corrosion. International Concrete Repair Institute, St. Paul, MN, 2008.
- ICRI. 310.2R, Selecting and Specifying Concrete Surface Preparation for Sealers, Coatings, Polymer Overlays, and Concrete Repair. International Concrete Repair Institute, St. Paul, MN, 2013.

ICRI. 320.2R, Guide for Selecting and Specifying Materials for Repair of Concrete Surfaces. International Concrete Repair Institute, St. Paul, MN, 2018.

ICRI. 330.2, Guide Specifications for Externally Bonded FRP Fabric Systems for Strengthening Concrete Structures. International Concrete Repair Institute, St. Paul, MN, 2016.

Italian National Research Council (CNR). CNR-DT 200 R1/2013, *Guide for the design and construction of externally bonded FRP systems for strengthening existing structures: materials, RC and PC structures, masonry structures*. Italian National Research Council, Rome, Italy, 2013.

JSCE. *Recommendations for Upgrading of Concrete Structures with Use of Continuous Fiber Sheets*, Japan Society of Civil Engineers, Tokyo, Japan, 2001.

Mirmiran, A., M. Shahawy, A. Nanni, V. Karbhari, B. Yalim, and A.S. Kalayci. *National Cooperative Highway Research Report 609: Recommended Construction Specifications and Process Control Manual For Repair and Retrofit of Concrete Structures Using Bonded FRP Composites*. National Cooperative Highway Research Program, Transportation Research Board, Washington, DC, 2008. <https://doi.org/10.17226/23116>.

SECTION 2: DEFINITIONS & NOTATIONS

TABLE OF CONTENTS

2.1—DEFINITIONS 2-1

2.2—NOTATIONS 2-1

2.3—REFERENCES 2-4

This page intentionally left blank.

SECTION 2: DEFINITIONS & NOTATIONS

2.1—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.

Definitions and terms related to highway bridge design shall be as stipulated in AASHTO LRFD.

2.2—NOTATIONS

Variables used throughout the Guide Specifications are listed alphabetically below:

A_c	=	area of section calculated using the gross composite concrete section properties of the girder and the deck and the deck-to-girder modular ratio (in. ²) (5.4.1)
A_f	=	area of FRP reinforcement (in. ²) (5.4.1) (5.5.2.3) (6.3.2)
A_{fs}	=	total cross-sectional area of the FRP spike anchors (in. ²) (5.5.3.2) (6.3.4)
A_{fUC}	=	total area of FRP U-wraps for continuous anchoring from the center of flexural FRP reinforcement toward one end (in. ²) (5.5.3.1)
A_{fUE}	=	total area of FRP U-wraps for anchoring one end of the flexural reinforcement (in. ²) (5.5.2.3) (5.5.3.1)
A_g	=	gross area of section (in. ²) (7.3.1)
A_{ps}	=	area of prestressed steel tension reinforcement (in. ²) (5.4.1)
A_s	=	area of non-prestressed steel tension reinforcement (in. ²) (5.4.1)
A'_s	=	area of steel compression reinforcement (in. ²) (5.4.1)
A_{st}	=	total area of longitudinal steel reinforcement (in. ²) (7.3.1)
a	=	depth of equivalent rectangular stress block (in.) (5.2.2)
b	=	width of the compression face of the member (in.) (5.4.1); smaller dimension of the rectangular member (in.) (7.3.2.1)
b_v	=	effective web width (in.) taken as the minimum web width within the effective depth, d_f (6.3.2)
b_w	=	web width (in.) (5.4.1)
c	=	distance from compression face to the neutral axis (in.) (5.2.2) (5.4.1)
c_e	=	clear distance between the groove edge and beam edge for NSM FRP rods or strips (in.) (5.5.5)
c_g	=	clear spacing between grooves for NSM FRP rods or strips (in.) (5.5.5)
D	=	external diameter of the circular member (in.) (7.3.2.1)
DC	=	force effects due to components and attachments (3.1.4)
D_e	=	equivalent confined circular core diameter (in.) (7.3.2.2)
DW	=	force effects due to wearing surfaces and utilities (3.1.4)
d_f	=	distance from compression face to the centroid of FRP reinforcement (in.) (5.4.1) (5.4.2.1) (6.3.2)
d_{fs}	=	spike anchor diameter (in.) (5.5.3.2) (6.3.4)

d_{fv}	=	effective depth of FRP measured from the top of FRP reinforcement to the centroid of the longitudinal reinforcement (in.) (6.3.2) (6.3.4) (6.4.2)
d_p	=	distance from compression face to the centroid of prestressed tension reinforcement (in.) (5.4.1) (5.4.2.1)
d_s	=	distance from compression face to the centroid of nonprestressed tension reinforcement (in.) (5.4.2.1)
d'_s	=	distance from compression face to the centroid of compression reinforcement (in.) (5.4.2.1)
d_r	=	diameter of NSM rod (in.) (5.5.5)
d_v	=	effective shear depth (in.) (5.5.2.3) (5.5.2.4) (6.3.3.3)
E_c	=	modulus of elasticity of the concrete (ksi) (5.4.1)
E_f	=	modulus of elasticity of FRP reinforcement (ksi) (5.2.1) (5.3.2) (5.5.2.3) (6.3.2) (6.4.2) (7.3.2.2) (7.5.2)
E_{fs}	=	modulus of elasticity of FRP spike anchor (ksi) (5.5.3.2) (6.3.4)
e	=	eccentricity of prestressing steel (in.) (5.4.1)
f_c	=	stress in concrete at strain, ϵ_c (ksi) (5.2.2) (5.3.1) (7.3.2.3)
f'_c	=	compressive strength of concrete for use in design (ksi) (3.1.2) (4.3.1) (5.2.1) (5.2.2) (5.3.1) (6.3.3.3) (7.3.1)
f'_{cc}	=	compressive strength of confined concrete (ksi) (7.3.1)
f_{fe}	=	effective stress in FRP reinforcement (ksi) (5.4.1) (5.4.2.1) (5.5.1) (5.5.5) (6.3.2)
f_{fus}	=	ultimate tensile strength of FRP spike anchor (ksi) (5.5.3.2) (6.3.4)
f_l	=	confinement pressure due to FRP strengthening (ksi) (7.3.2.2)
f_{ps}	=	average stress in prestressing steel at the time for which the nominal resistance of member is required (ksi) (5.4.1)
f_s	=	stress in the steel tension reinforcement at development of nominal flexural resistance (ksi) (5.4.1) (7.3.2.3)
f'_s	=	stress in the steel compression reinforcement at development of nominal flexural resistance (ksi) (5.4.1)
f_y	=	specified minimum yield strength of steel reinforcement (ksi) (5.3.1) (7.3.1) (7.3.2.3)
h	=	overall thickness or depth of a member (in.) (C5.4.1); larger dimension of the rectangular member (in.) (7.3.2.1)
h_f	=	compression flange depth (in.) (5.4.1)
h_{fs}	=	FRP spike anchor depth (in.) (5.5.3.2) (6.3.4)
h_g	=	groove depth for NSM FRP rods or strips (in.) (5.5.5)
h_s	=	height of NSM FRP strip (in.) (5.5.5)
IM	=	dynamic load allowance (3.1.4) (3.4)
k_e	=	strength reduction factor applied for unexpected eccentricities (7.3.2.2)
L_d	=	development length (in.) (5.5.1)
L_{dr}	=	development length for NSM FRP rods (in.) (5.5.5)
L_{ds}	=	development length for NSM FRP strips (in.) (5.5.5)
LL	=	force effects due to live loads (3.1.4)
l_{fs}	=	splay length of spike anchor (in.) (5.5.2.4) (5.5.3.2) (6.3.4)
l_u	=	unbraced length (in.) (7.3.2.1)
M_{cr}	=	cracking moment (kip-in) (5.5.2.1)
$M_{LL,r}$	=	live load moment during FRP system application and cure (kip-in.) (3.4)

M_n	=	nominal flexural resistance (kip-in.) (5.4.2)
M_{nf}	=	nominal FRP flexural resistance (kip-in.) (5.4.2)
$M_{n,r}$	=	nominal flexural resistance of member prior to application of FRP system (kip-in.) (3.4)
M_r	=	factored flexural resistance (kip-in.) (5.4.2) (5.4.2.1)
n	=	number of layers of FRP (5.2.1) (5.5.2.3) (6.3.2) (6.4.2) (7.3.2.2) (7.5.2)
n_{fs}	=	number of FRP spike anchors in each row along the width of the flexural FRP (5.5.3.2)
P_e	=	effective prestressing force (kip) (5.4.1)
P_r	=	factored axial load resistance (kip) (7.3.1) (7.4.1) (7.5.2)
p_{fs}	=	spacing between FRP spike anchors along the width of the flexural reinforcement (in.) (5.5.3.2)
R_r	=	factored resistance of the unstrengthened member (kip-in.) (3.1.4)
r_g	=	radius of gyration of gross cross section (in.) (5.4.1)
s_f	=	center-to-center spacing of FRP (in.) (6.3.2) (6.3.4)
s_{fs}	=	longitudinal spacing of spike anchors (in.) (5.5.2.4)
s_{max}	=	maximum permitted spacing of transverse reinforcement (in.) (6.3.3.3)
T_f	=	nominal strength in torsion provided by the externally bonded FRP system (kip-in.) (6.4.1) (6.4.2)
T_g	=	glass transition temperature (degrees F) (C4.2.5.1)
$T_{MaxDesign}$	=	maximum design temperature (degrees F) (4.2.5.1) (4.2.7.1)
T_n	=	nominal torsional resistance (kip-in.) (6.4.1)
T_r	=	factored torsional resistance (kip-in.) (6.4.1)
t_f	=	thickness of a single layer of FRP reinforcement (in.) (5.2.1) (5.5.1) (5.5.2.3) (6.3.2) (6.4.2) (7.3.2.2) (7.5.2)
V_c	=	nominal shear resistance of the concrete (kip) (5.5.2.2) (6.3.1)
V_f	=	nominal shear strength provided by the externally bonded FRP system (kip) (6.3.1)
V_n	=	nominal shear resistance (kip) (6.3.3.2)
V_p	=	component of the prestressing force in the direction of the shear force (kip) (6.3.1) (6.3.3.2)
V_r	=	factored shear resistance (kip) (6.3.1)
V_s	=	shear resistance provided by transverse reinforcement (kip) (6.3.1)
V_u	=	factored shear force (kip); factored shear force at end of flexural FRP reinforcement (kip) (5.5.2.2) (6.3)
v_u	=	shear stress (ksi) (6.3.3.3)
w_f	=	width of FRP reinforcement (in.) (5.5.2.3) (5.5.3.2) (6.3.2) (6.3.4) (7.5.2)
w_g	=	groove width for NSM FRP rods or strips (in.) (5.5.5)
w_s	=	thickness of NSM FRP strip (in.) (5.5.5)
x_l	=	lesser dimension of the member (in.) (6.4.2)
y_l	=	larger dimension of the member (in.) (6.4.2)
$(\cdot)_U$	=	U-wrap FRP properties (5.5.2.3)
α_{lf}	=	stress block factor when concrete compressive strain is less than 0.003 in./in. taken as the ratio of equivalent rectangular concrete compressive stress intensity to the compressive strength of concrete used in design (5.2.2) (5.4.1)

α_f	=	angle of inclination of FRP with respect to the longitudinal axis of the member (degrees) (6.3.2)
α_{fs}	=	splay angle of spike anchor (degrees) (5.5.3.2) (6.3.4)
β_{fs}	=	spike anchor installation angle (degrees) (6.3.4)
ϵ_c	=	compression strain in concrete (in./in.) (C5.4.1)
ϵ_{cu}	=	failure strain of concrete in compression (in./in.) (5.4.1)
ϵ_f	=	strain in FRP reinforcement (in./in.) (C5.4.1)
ϵ_{fe}	=	effective strain in FRP reinforcement (in./in.) (5.3.2) (5.5.2.3) (6.3.2) (7.3.2.2) (7.5.2)
ϵ_{fi}	=	strain at FRP–concrete interface (in./in.) (5.2.1) (5.5.2.3)
ϵ_{fu}	=	FRP ultimate strain (in./in.) (5.2.1) (5.3.2) (6.4.2) (7.3.2.2) (7.5.2)
ϵ_i	=	strain in concrete substrate at time of FRP installation (in./in.) (5.4.1)
ϵ_{pe}	=	effective strain in prestressing steel (in./in.) (5.4.1)
ϵ_{pi}	=	initial strain in prestressing steel (in./in.) (5.4.1)
ϵ_{pn}	=	net tensile strain beyond decompression in the prestressing steel (in./in.) (5.4.1)
ϵ_{ps}	=	strain in prestressing steel (in./in.) (5.4.1)
ϵ_{pu}	=	specified ultimate tensile strain of prestressing steel (in./in.) (5.4.1)
ϵ_s	=	strain in non-prestressed steel reinforcement (in./in.) (C5.4.1)
ϵ'_s	=	strain in compression steel reinforcement (in./in.) (C5.4.1)
ϵ_y	=	yield strain of non-prestressed steel reinforcement (in./in.) (C5.4.1)
η	=	strain limitation coefficient that is less than unity (5.3.2)
η_i	=	load modifier specified in AASHTO LRFD Article 1.3.2 (3.1.4)
ρ_f	=	FRP shear reinforcement ratio (6.3.2)
τ_a	=	average bond strength (ksi) (5.5.5)
τ_{int}	=	interface shear transfer strength (ksi) (5.5.1)
ϕ	=	resistance factor specified in AASHTO LRFD Article 5.5.4.2 (3.2.2) (5.4.2) (6.3.1) (6.4.1) (7.3.1) (7.5.2)
ϕ_c	=	0.75, material resistance factor for concrete (7.3.2.2)
ϕ_{EM}	=	environmental reduction factor for FRP material (4.2.4.2) (5.4.1) (6.3.2) (6.4.2) (7.3.2.2)
ϕ_{Ei}	=	environmental reduction factor for FRP–concrete bond (4.3.2.2) (5.4.1) (6.3.2) (7.5.2)
ϕ_f	=	FRP resistance factor (3.2.2) (5.4.2.1) (6.3.1) (6.4.1) (7.3.2.2) (7.5.2)

2.3—REFERENCES

AASHTO. *AASHTO LRFD Bridge Design Specifications*, 9th Ed. LRFDBDS-9. American Association of State Highway and Transportation Officials, Washington, DC, 2020.

ASTM. D3878, Standard Terminology for Composite Materials. ASTM International, West Conshohocken, PA.

ASTM. D907, Standard Terminology of Adhesives. ASTM International, West Conshohocken, PA.

SECTION 3: GENERAL REQUIREMENTS

TABLE OF CONTENTS

3.1—DESIGN BASIS	3-1
3.1.1—Design Objectives	3-1
3.1.2—Maximum Concrete Strength	3-1
3.1.3—FRP Design Properties.	3-1
3.1.4—Strengthening Requirement	3-2
3.2—LIMIT STATES	3-2
3.2.1—Service Limit States.	3-2
3.2.2—Strength Limit States.	3-3
3.2.3—Extreme Event Limit States	3-4
3.2.4—Fatigue Limit State	3-4
3.3—LOADS AND LOAD COMBINATIONS	3-5
3.3.1—Loads	3-5
3.3.2—Load Combinations	3-5
3.3.3—Bridge Load Ratings and Evaluation	3-5
3.4—LIVE LOAD LIMITS DURING FRP APPLICATION AND CURE	3-6
3.5—REFERENCES.	3-6

This page intentionally left blank.

SECTION 3:
GENERAL REQUIREMENTS

3.1—DESIGN BASIS

3.1.1—Design Objectives

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, economy, and aesthetics, as stipulated in AASHTO LRFD Article 2.5.

C3.1.1

The resistance criteria in these Guide Specifications were developed using modern principles of structural reliability analysis, which are consistent with those on which AASHTO LRFD is 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 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.

These Guide Specifications are intended to apply only to concrete bridge elements and components that have minimum strength requirements established in Article 3.1.4 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; 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.

3.1.2—Material Compressive Strength

The provisions of these Guide Specifications are limited to concretes with specified compressive strength, f'_c , not exceeding 10 ksi. For applications on existing concrete bridge elements with a compressive strength greater than 10 ksi, the analysis shall be based on limiting the concrete compressive strength to 10 ksi.

3.1.3—FRP Design Requirements

FRP samples shall be tested according to ASTM D3039/D3039M standard practice. The

C3.1.3

The gross FRP system area should be used in determining the design tensile properties

design value of the strength or the failure strain of the FRP reinforcement used for strengthening shall be computed in accordance with ASTM D7290 standard practice. For wet-layup FRP systems, the average ply thickness of the test specimens, as calculated according to ASTM D3039/D3039M, shall be used as the design FRP thickness.

3.1. —S r R r

The provisions of these Guide Specifications shall apply to concrete bridge elements for which the factored resistance satisfies the following requirement:

$$R_r \geq \eta_i [(DC + DW) + (LL + IM)] \quad (3.1.4-1)$$

where:

DC = force effects due to component and attachments

DW = force effects due to wearing surfaces and utilities

IM = dynamic load allowances

LL = force effects due to live loads

R_r = factored resistance of the unstrengthened member calculated in accordance with AASHTO LRFD Section 5 and these Guide Specifications

η_i = load modifier specified in AASHTO LRFD Article 1.3.2

3.2— IMIT STATES

Structural members shall satisfy AASHTO LRFD Equation 1.3.2.1-1 for each limit state, unless otherwise specified.

The load factor, γ , in AASHTO LRFD Equation 1.3.2.1-1 shall be as defined in AASHTO LRFD Tables 3.4.1-1, 3.4.1-2, and 3.4.1-3.

3.2.1—S r S

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

when tested according to ASTM D3039/D3039M specifications.

C3.1.

In cases where FRP is being considered for concrete bridge elements that may be subjected to force effects in addition to those identified in Equation 3.1.4-1 (e.g., Horizontal Earth Pressure E_H , Vertical Earth Pressure E_V), consideration should be given to determine reasonable strength requirements for the unstrengthened bridge element prior to FRP application.

C3.2

These Guide Specifications apply to Strength limit states I, II, III, IV and V; Service limit states I, III, and IV; Extreme Event limit states I and II; and the Fatigue limit state, as defined in AASHTO LRFD Article 3.4.

C3.2.1

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

- Service I—Load combination relating to the normal operational use of the bridge with a 70-mph wind and all loads taken at their nominal values.
- 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.

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.

The AASHTO 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 AASHTO LRFD reflects current exclusion weight limits mandated by various jurisdictions. Vehicles permitted under these limits were in service for many years prior to 1994, when the first *AASHTO LRFD Bridge Design Specifications* were published. 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 segmental concrete girders.

3.2.2—Strength Limit States

Structural members of a bridge shall satisfy AASHTO LRFD Equation 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 the design wind speed at the location of the bridge.
- Strength IV—Load combination emphasizing dead load force effects in bridge superstructures.
- Strength V—Load combination relating to normal random vehicular use of the bridge with wind of 80 mph velocity.

The resistance factors, ϕ , shall be as defined in AASHTO LRFD Article 5.5.4.2. The FRP resistance factors, ϕ_r , shall be taken as:

C3.2.2

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 AASHTO LRFD is presented in Nowak (1993).

For tension-controlled reinforced or prestressed concrete sections: 1.00

For shear in reinforced or prestressed concrete sections: 0.85

For torsion in reinforced or prestressed concrete sections: 0.65

For compression-controlled sections with spirals or ties: 0.65

For axial tension in reinforced concrete sections: 0.50

3.2.3—E r E S

Structural members of a bridge shall satisfy AASHTO LRFD Equation 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, check floods, and certain hydraulic events with a reduced live load other than that which is part of the vehicular collision load.

3.2. —F S

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

C3.2.3

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, ice floe, or possible foundation scour.

C3.2.

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 AASHTO 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.

3.3—LOADS AND LOAD COMBINATIONS

3.3.1—Loads

The loads defined in AASHTO LRFD Article 3.3.2 and characterized in AASHTO LRFD Articles 3.5 through 3.15 shall be applied for designing reinforced and prestressed concrete highway bridge members strengthened with externally bonded FRP reinforcement.

3.3.2—Load Combinations

The load combination requirements shall be determined in accordance with AASHTO LRFD Article 3.4.

3.3.3—Bridge Load Ratings and Evaluation

Bridge evaluations shall be performed using the evaluation criteria stipulated in the AASHTO *Manual for Bridge Evaluation*, Third Edition, 2018 (MBE). MBE Equation 6A.4.2.1-1 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 for 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 (e.g., axial-bending interaction or shear-bending interaction) shall be considered in developing the rating as provided in the MBE under the sections on resistance of structures.

C3.3.1

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

C3.3.3

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 AASHTO LRFD
- Legal Loads—(1) AASHTO legal loads, as specified in MBE Article 6A.4.4.2.1a, and (2) the notional rating load as specified in MBE Article 6A.4.4.2.1b 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.4.5 to determine the need for load posting or strengthening. Legal loads for ratings given in MBE Article 6A.4.4.2.1a 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 6A.4.4.2.1b represent the increasing presence of Formula B multi-axle specialized hauling vehicles in the traffic stream in many states.

3. — LIVE LOAD LIMITS DURING FRP APPLICATION AND CURE

The maximum permissible live load moment during FRP flexural strengthening application and cure is limited to $0.5M_{n,r}$.

The bridge or traffic lane over which the FRP flexural strengthening system is being applied, need not be closed or posted if the following condition is satisfied:

$$0.5M_{n,r} \geq (1 + IM) M_{LL,r} \quad (3.4-1)$$

where:

IM = dynamic load allowance

$M_{LL,r}$ = live load moment during FRP system application and cure (kip-in.)

$M_{n,r}$ = nominal flexural resistance of member prior to application of FRP system (kip-in.)

C3.

The limits are based on studies by Barnes and Mays (2001), Reed et al. (2005) and Wang et al. (2013). If the live load limit of Equation 3.4-1 is not satisfied, especially in the case of girders, the following options may be investigated: (a) bridge closure, (b) lane closure over member being retrofitted, (c) load post bridge/lane, (d) brace or support member being retrofitted.

3. —REFERENCES

AASHTO. *AASHTO LRFD Bridge Design Specifications*, 9th Ed. LRFD BDS-9. American Association of State Highway and Transportation Officials, Washington, DC, 2020.

AASHTO. *Manual for Bridge Evaluation*, 3rd Ed., with 2019 and 2020 Interim Revisions. MBE-3. American Association of State Highway and Transportation Officials, Washington, DC, 2018.

ASTM. D3039/D3039M, Standard Test Method for Tensile Properties of Polymer Matrix Composite Materials. ASTM International, West Conshohocken, PA.

ASTM. D7290, Standard Practice for Evaluating Material Property Characteristic Values for Polymeric Composites for Civil Engineering Structural Applications. ASTM International, West Conshohocken, PA.

Barnes, R. A. and G. C. Mays. The effect of traffic vibration on adhesive curing during installation of bonded external reinforcement. *Proceedings of the Institution of Civil Engineers Structures and Buildings*, 146(4), 2001, pp. 403–410.

Nowak, A. S. Live Load Model for Highway Bridges. *Journal of Structural Safety*, Vol. 13, Nos. 1 and 2, December, Elsevier, Maryland Heights, MO, December 1993, pp. 53–66.

Reed, M. W., R. W. Barnes, A. Schindler, and H-W. Lee. Fiber-Reinforced Polymer Strengthening of Concrete Bridges that Remain Open to Traffic. *ACI Structural Journal*, 102(6), 2005, pp. 823–831.

Wang, W.-W., -G. Dai, and . A. Harries. Performance Evaluation of RC Beams Strengthened with an Externally Bonded FRP System under Simulated Vehicle Loads. *Journal of Bridge Engineering*, 18(1), 2013, pp. 76–82.

TABLE OF CONTENTS

4.1—SCOPE	4-1
4.2—MATERIAL REQUIREMENTS	4-1
4.2.1—Material Acceptability	4-1
4.2.2—Material Content	4-1
4.2.3—Material Testing	4-1
4.2.4—Environmental Exposure	4-2
4.2.4.1—Environmental Exposure Categories	4-2
4.2.4.2—Environmental Reduction Factors for FRP Material.	4-2
4.2.5—Other Material Requirements.	4-3
4.2.5.1—Glass Transition Temperature	4-3
4.2.5.2—Moisture Equilibrium Content	4-3
4.2.5.3—Additional Requirements	4-3
4.2.6—Impact Tolerance.	4-5
4.2.7—Adhesive Requirements	4-5
4.2.7.1—Adhesive Glass Transition Temperature	4-5
4.3—BOND REQUIREMENTS	4-5
4.3.1—Adhesive Bond Strength.	4-5
4.3.2—Environmental Exposure	4-5
4.3.2.1—Environmental Exposure Categories	4-5
4.3.2.2—Environmental Reduction Factors for the FRP–Concrete Bond	4-6
4.4—REFERENCES	4-6

This page intentionally left blank.

SECTION 4:

MATERIAL & BOND REQUIREMENTS

4.1—SCOPE

This Section defines the requirements for externally bonded FRP composite material systems intended for use for repair and strengthening of concrete bridge elements.

4.2—MATERIAL REQUIREMENTS

4.2.1—Material Acceptability

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.

4.2.2—Material Content

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, fiber volume, 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.

4.2.3—Material Testing

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 ten.

2. —E r E r

2. .1—E r E r C r

The environmental exposure category of the FRP material shall be determined as follows:

Environmental Exposure Category A: Environmental Exposure Category A shall apply to overpass bridges and bridges over dry surfaces. Exposure to periodic rain or seasonal de-icing agent spray fall in this category.

Environmental Exposure Category B: Environmental Exposure Category B shall apply to bridges that are continuously or frequently in contact with water or located in high moisture environments. Bridges located in coastal regions, over waterways, or near water bodies fall in this category.

2. .2—E r R d F r r FR M r

For non-bond-critical FRP retrofit designs, the ultimate stress and strain for the FRP material shall be modified by the FRP material environmental reduction factor, ϕ_{EM} , provided in Table 4.2.4.2-1.

For bond-critical designs, the environmental reductions factors based on the FRP–concrete bond are provided in Article 4.3.2.

T 2. .2 1—E r R d F r r FR M r

FRP Material	ϕ_{EM}	
	Environmental Exposure Category	
	A	B
CFRP Fabric	0.85	0.80
CFRP Pultruded Laminate	0.95	0.90
GFRP Fabric	0.75	0.65

C 2. .1

The environmental exposure categories and the corresponding environmental reduction factors, ϕ_{EM} , in Table 4.2.4.2-1, are based on statistical analysis of experimental tests conducted on test specimens in accordance with ASTM D3039/D3039M or similar standard tests (NCHRP Report 20-07/Task 428).

Environmental Exposure Category A is for bridges in relatively moisture-free environments while Environmental Exposure Category B is for bridges in relatively high-moisture environments.

When FRP material is continuously subjected to more specific environmental exposures (e.g., different chemicals, free ing) in addition to the ones listed in Article 4.2.4.1, material tests should be performed to determine the environmental reduction factors.

C 2. .2

The strength of GFRP fabric has been found to degrade when exposed to Alkaline environments (NCHRP Report 20-07/Task 428). If GFRP fabric is used in an Alkaline environment, it should be considered as a separate environmental exposure category with $\phi_{EM} = 0.20$ (NCHRP Report 20-07/Task 428).

Other environmental exposures and reduction factors are presented in NCHRP Report 20-07/Task 428, such as a $\phi_{EM} = 0.55$ for CFRP fabric continuously exposed to moisture and susceptible to free ing or free e–thaw.

4.2.5—Other Material Requirements

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.

4.2.5.1—Glass Transition Temperature

The characteristic value of the glass transition temperature of the composite system, determined in accordance with ASTM E1640, shall be at least 40 degrees F higher than the maximum design temperature, $T_{MaxDesign}$, defined in Article 3.12.2.2 of AASHTO LRFD.

4.2.5.2—Moisture Equilibrium Content

The average value and coefficient of variation of the moisture equilibrium content determined in accordance with ASTM D5229/D5229M shall not be greater than two percent and ten percent, respectively. A minimum sample size of ten shall be used in the calculation of these maximum values.

4.2.5.3—Additional Requirements

The FRP strengthening system should retain 85 percent of its strength following exposure to the conditions identified in Table 4.2.5.3-1, when tested according to ASTM D3039/D3039M.

C4.2.5.1

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 (Zureick and Kahn 2001). In addition, it is to be noted that T_g decreases as the moisture content in the composite increases.

C4.2.5.2

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 becomes weak and the structural integrity of the composite system degrades.

C4.2.5.3

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 following:

- Chemical or physical changes in the polymeric matrix, or both.
- 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 4.2.5.3 are based on those developed for CALTRANS

Table 4.2.5.3-1—Environmental Test Requirements

Exposure Condition	Specification	Test Condition	Test Duration
Water	ASTM D2247, ASTM E104	100% RH at 100 ± 3 degrees F	3,000 hours
Saltwater	ASTM C581, ASTM D1141	Immersed at 73 ± 2 degrees F	3,000 hours
Alkali	ASTM C581	Immersed in a solution of calcium hydroxide (pH ~12) at 73 ± 3 degrees F	3,000 hours
Dry heat	ASTM D3045	140 ± 5 degrees F	3,000 hours
Ultraviolet light	ASTM G154	Cycle 1 (UVA-340 lamp): 8 h UV at 140 ± 5 degrees F with 4 h condensation at 122 ± 5 degrees F	3,000 hours
Cyclic weathering	ASTM G153	Cycle 5: 4 h light at 145 ± 5 degrees F with 4 h light and water spray	375 cycles
Freeze–thaw	AASHTO T 161 (ASTM C666)	Lower temperature of specimens from 40 to 0 degrees F and raise it from 0 to 40 degrees F in not less than 2 nor more than 5 h.	3,000 cycles

(Steckel et al., 1999; Hawkins et al., 1999), for GDOT (Zureick, 2002) and those recommended by ICRI 330.2 (2016) and AASHTO M 337.

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

4.2.6—Impact Tolerance

Where impact tolerance is stipulated by the Engineer of Record, the stipulated impact tolerance shall be determined by ASTM D7136/D7136M.

C4.2.6

Impact tolerance may be stipulated for members subject to impact, e.g., floating debris impacting a river pier or pile strengthened with externally bonded FRP.

4.2.7—Adhesive Requirements

When adhesive material is used to bond the FRP reinforcement to the concrete surface, the following requirements shall be met.

4.2.7.1—Adhesive Glass Transition Temperature

After conditioning in the environments stipulated in Article 4.2.5.3, the characteristic value of the glass transition temperature of the adhesive material determined in accordance with ASTM E1640, shall be at least 40 degrees F higher than the maximum design temperature, $T_{MaxDesign}$, defined in AASHTO LRFD Article 3.12.2.2.

4.3—BOND REQUIREMENTS**4.3.1—Adhesive Bond Strength**

After conditioning in the environments stipulated in Article 4.2.5.3, the bond strength (ksi), determined by tests specified by the Engineer of Record, shall be at least 0.2 ksi or $0.065 \sqrt{f'_c}$, whichever is greater. For NSM FRP, the bond strength shall be at least 1.0 ksi.

C4.3.1

The bond strength limit of $0.065 \sqrt{f'_c}$ is based on tests conducted by Naaman (1999) on reinforced concrete beams strengthened with externally bonded FRP reinforcement.

Pull-off tests, such as ASTM D7522/D7522M, should be applied to surface bonded FRP material. For NSM FRP, pull-out testing on specimens similar to the one developed by De Lorenzis et al. (2002), or as recommended by the Engineer of Record, should be carried out. The bond strength limit of 1.0 ksi is based on the review of published research by Hassan and Rizkalla (2003).

4.3.2—Environmental Exposure**4.3.2.1—Environmental Exposure Categories**

Environmental Exposure Categories A and B for FRP-concrete bond shall be determined in accordance with Article 4.2.4.1.

C4.3.2.1

The environmental exposure categories and the corresponding environmental reduction factors, ϕ_{Ei} , in Table 4.3.2.2-1 are based on statistical analysis of

experimental tests conducted on beam specimens, and single- and double-lap shear specimens (NCHRP Report 20-07/Task 428).

When FRP-concrete bond is continuously subjected to more specific environmental exposures (e.g., different chemicals, saturated at elevated temperatures, continuous free ing) in addition to the ones listed in Article 4.2.4.1, bond tests should be performed to determine the environmental reduction factors.

4.3.2.2—Environmental Reduction Factor for FRP-Concrete Bond

The environmental reduction factor for the FRP-concrete bond, ϕ_{Ei} , is applicable to the interface strain for bond-critical FRP applications. The effective strain at the FRP-concrete interface shall be modified using the environmental reduction factor for the FRP-concrete bond, ϕ_{Ei} , provided in Table 4.3.2.2-1.

Table 4.3.2.2-1—Environmental Reduction Factor for FRP-Concrete Bond

FRP material		ϕ_{Ei}	
		Environmental Exposure Category	
		A	B
CFRP Fabric/Prepregs		0.75	0.60
CFRP Pultruded Laminate		0.65	0.45
GFRP Fabric		0.75	0.60
NSM FRP	Epoxy Adhesive	0.90	0.80
	Cementitious Adhesive	0.50	0.50

For environmental exposures other than the ones in Article 4.3.2, the Engineer of Record shall stipulate the value of ϕ_{Ei} based on experimental data for the materials specified, and this value shall be provided in the contract documents.

5.—REFERENCES

AASHTO. *AASHTO LRFD Bridge Design Specifications*, 9th Ed. LRFDBDS-9. American Association of State Highway and Transportation Officials, Washington, DC, 2020.

C 4.3.2.2

Other environmental exposures and reduction factors are presented in NCHRP Report 20-07/Task 428, such as a $\phi_{Ei} = 0.45$ for specimens exposed to hygrothermal conditions where they are continuously submerged in a liquid medium (water/saltwater/alkaline solution) at elevated temperatures greater than or equal to 104 degrees F. A $\phi_{Ei} = 0.45$ is also derived for exposures to free ing temperatures where CFRP fabric specimens are continuously subjected to temperatures less than or equal to 0 degrees F.

- AASHTO. M 337, Standard Specification for Fiber-Reinforced Polymer Composite Materials for Highway and Bridge Structures. American Association of State Highway and Transportation Officials, Washington, DC.
- AASHTO. T 161, Standard Method of Test for Resistance of Concrete to Rapid Freezing and Thawing. American Association of State Highway and Transportation Officials, Washington, DC.
- ASTM. C581, Standard Practice for Determining Chemical Resistance of Thermosetting Resins Used in Glass-Fiber-Reinforced Structures Intended for Liquid Service. ASTM International, West Conshohocken, PA.
- ASTM. D1141, Standard Practice for Preparation of Substitute Ocean Water. ASTM International, West Conshohocken, PA.
- ASTM. D2247, Standard Practice for Testing Water Resistance of Coatings in 100 % Relative Humidity. ASTM International, West Conshohocken, PA.
- ASTM. D3039/D3039M, Standard Test Method for Tensile Properties of Polymer Matrix Composite Materials. ASTM International, West Conshohocken, PA.
- ASTM. D3045, Standard Practice for Heat Aging of Plastics Without Load. ASTM International, West Conshohocken, PA.
- ASTM. D5229/D5229M, Standard Test Method for Moisture Absorption Properties and Equilibrium Conditioning of Polymer Matrix Composite Materials. ASTM International, West Conshohocken, PA.
- ASTM. D7136/D7136M, Standard Test Method for Measuring the Damage Resistance of a Fiber-Reinforced Polymer Matrix Composite to a Drop-Weight Impact Event. ASTM International, West Conshohocken, PA.
- ASTM. D7522/D7522M, Standard Test Method for Pull-Off Strength for FRP Laminate Systems Bonded to Concrete or Masonry Substrates. ASTM International, West Conshohocken, PA.
- ASTM. E104, Standard Practice for Maintaining Constant Relative Humidity by Means of Aqueous Solutions. ASTM International, West Conshohocken, PA.
- ASTM. E1640, Standard Test Method for Assignment of the Glass Transition Temperature by Dynamic Mechanical Analysis. ASTM International, West Conshohocken, PA.
- ASTM. G153, Standard Practice for Operating Enclosed Carbon Arc Light Apparatus for Exposure of Nonmetallic Materials. ASTM International, West Conshohocken, PA.
- ASTM. G154, Standard Practice for Operating Fluorescent Ultraviolet (UV) Lamp Apparatus for Exposure of Nonmetallic Materials. ASTM International, West Conshohocken, PA.
- De Lorenzis, L., A. Rizzo, and A. La Tegola. A modified pull-out test for bond of near-surface mounted FRP rods in concrete. *Composites Part B: Engineering*. Vol. 33, Issue 8, December 2002, pp. 589–603. [https://doi.org/10.1016/S1359-8368\(02\)00052-5](https://doi.org/10.1016/S1359-8368(02)00052-5).
- Hassan, T., and S. Rizkalla. Investigation of Bond in Concrete Structures Strengthened with Near-Surface Mounted Carbon Fiber Reinforced Polymer Strips. *Journal of Composites for Construction*, Vol. 7, Issue 3, August 2003, pp. 248–257.
- Hawkins, G. F., E. Johnson, and J. P. Nokes. Qualifications for Seismic Retrofitting of Bridge Columns Using Composites. *Composite Properties Characterization*, Vol. 3. Prepared for the California Department of Transportation, Contract No. SA0A011, The Aerospace Corporation, El Segundo, CA, 1999.

- ICRI. 330.2, *Guide Specifications for Externally Bonded FRP Fabric Systems for Strengthening Concrete Structures*. International Concrete Repair Institute, St. Paul, MN, 2016.
- Naaman, A. E. *Repair and Strengthening of Reinforced Concrete Beams Using CFRP Laminates*, Vols. 1–7. Report No. RC-1372, Michigan Department of Transportation, Lansing, MI, 1999.
- National Cooperative Highway Research Program. *Update of the 2012 AASHTO Guide Specifications for Design of Bonded FRP Systems for Repair and Strengthening of Concrete Bridge Elements*, National Cooperative Highway Research Program Report 20-07/Task 428, Transportation Research Board, National Research Council, Washington, DC, Forthcoming.
- Steckel, G. L., J. L. Bauer, G. F. Hawkins, and M. W. Vanik. Qualifications for Seismic Retrofitting of Bridge Columns Using Composites. *Composite Properties Characterization*, Vols 1 & 2. Prepared for the California Department of Transportation, Contract No. SA0A011, The Aerospace Corporation, El Segundo, CA, 1999.
- Zureick, A. and L. Kahn. Rehabilitation of Reinforced Concrete Structures Using Fiber-Reinforced Polymer Composites. *ASM Handbook*, Vol. 21: Composites, ASM International, Materials Park, OH, 2001, pp. 906–913.
- Zureick, A. *Proposed Specifications—Polymeric Composite Materials for Rehabilitating Concrete Structures*. Report prepared for the Georgia Department of Transportation, Atlanta, GA, 2002.

SECTION 5: MEMBERS UNDER FLEXURE
TABLE OF CONTENTS

5.1—GENERAL REQUIREMENTS	5-1
5.2—DESIGN ASSUMPTIONS	5-1
5.2.1—FRP–Concrete Interface Strain	5-1
5.2.2—Concrete Stress Distribution	5-2
5.3—SERVICEABILITY AND FATIGUE LIMIT STATES	5-3
5.3.1—Serviceability Stresses	5-3
5.3.2—Fatigue Stresses	5-4
5.4—STRENGTH LIMIT STATES	5-4
5.4.1—Stress in Reinforcement at Nominal Flexural Resistance	5-4
5.4.2—Factored Flexural Resistance	5-7
5.4.2.1—Flanged Sections	5-8
5.4.2.2—Rectangular Sections	5-8
5.4.2.3—Other Cross Sections	5-8
5.4.3—Ductility Requirements	5-9
5.5—DETAILING REQUIREMENTS	5-9
5.5.1—Development Length	5-9
5.5.2—End Debonding	5-9
5.5.2.1—Detailing to Prevent End Debonding	5-9
5.5.2.2—Shear Limit to Prevent End Debonding	5-10
5.5.2.3—U-Wraps for End Debonding	5-11
5.5.2.4—Spike Anchors for End Debonding	5-12
5.5.3—Continuous Flexural Anchoring	5-12
5.5.3.1—U-Wraps for Continuous Anchoring	5-12
5.5.3.2—FRP Spike Anchors for Continuous Anchoring	5-13
5.5.4—Laps and Splices	5-15
5.5.5—Detailing for NSM FRP Rods and Strips	5-15
5.6—REFERENCES	5-16

This page intentionally left blank.

SECTION 5:
MEMBERS UNDER FLEXURE

5.1—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 these Guide Specifications.

Except where specifically provided below, all provisions of AASHTO LRFD shall apply.

5.2—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 bonding exists between the reinforcing steel and the concrete, and the 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 (in./in.).

5.2.1—FRP REINFORCEMENT STRAIN

Depending on the type of FRP reinforcement application, the maximum usable strain at the FRP-concrete interface, ϵ_{fi} , shall be taken as:

Externally bonded FRP fabric and precured FRP laminate

$$\alpha_{fi} \leq \epsilon_{fi} \leq 0.6\epsilon_{fu} \quad (5.2.1-1)$$

C 5.2

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.

C 5.2.1

The minimum strains depend on the type of FRP reinforcement, and are conservative estimates based on failures observed in experimental test data.

The use of FRP U-wraps and FRP spike anchors to anchor flexural FRP reinforcement, such as externally bonded FRP fabric and precured FRP laminates, has been shown to increase the strain at which failure occurs at the FRP-concrete interface.

The maximum strain limits are based on experimental data where, depending on the type of FRP strengthening, rupture failure was observed at values less than those predicted by material testing.

$$\epsilon_{fi} = 0.045 \sqrt{\frac{f'_c}{nE_f t_f}} \quad (5.2.1-2)$$

in which:

$$nE_f t_f \geq 1,900 \text{ kip/in.} \quad (5.2.1-3)$$

$$\alpha_{fi} = \begin{cases} 0.004 \text{ in./in. for partial or no anchors} \\ 0.006 \text{ in./in. for U-wrap anchors} \\ 0.007 \text{ in./in. for FRP spike anchors} \end{cases} \quad (5.2.1-4)$$

NSM FRP rods

$$\epsilon_{fi} = \text{larger of } \begin{cases} 0.006 \text{ in./in.} \leq 0.8\epsilon_{fu} \\ 0.40\epsilon_{fu} \leq 0.008 \text{ in./in.} \end{cases} \quad (5.2.1-5)$$

NSM FRP strips

$$\epsilon_{fi} = \text{larger of } \begin{cases} 0.007 \text{ in./in.} \leq 0.8\epsilon_{fu} \\ 0.45\epsilon_{fu} \leq 0.009 \text{ in./in.} \end{cases} \quad (5.2.1-6)$$

where:

- E_f = modulus of elasticity of FRP reinforcement (ksi)
- f'_c = compressive strength of concrete for use in design (ksi)
- n = number of layers of FRP
- t_f = thickness of a single layer of FRP reinforcement (in.)
- ϵ_{fi} = strain at FRP–concrete interface (in./in.)
- ϵ_{fu} = FRP ultimate strain (in./in.)

5.2.2—Concrete Stress Distribution

When concrete compressive strain is 0.003 (in./in.) under conditions of force equilibrium, it is permitted to model the distribution of concrete stress

Equation 5.2.1-2 is a modified form of the one proposed by Teng et al. (2003) and provides conservative failure load estimates based on experimental test data.

The limit on the axial stiffness per unit width (Equation 5.2.1-3) of 1,900 kip/in. is based on a limited set of experimental data (NCHRP Report 20-07/Task 428).

The FRP properties should be based on test data in accordance with ASTM D3039/D3039M as outlined in Articles 3.1.3 and C3.1.3. For wet-layup FRP systems, the average ply thickness of the test specimens, as calculated according to ASTM D3039/D3039M, should be used as the design FRP thickness.

In order to reach the strain limits provided in Equation 5.2.1-4, anchoring of the flexural FRP should be carried out as detailed in Article 5.5.3. Use of anchors less than those stipulated under Article 5.5.3 should be considered as partial anchors.

C5.2.2

The nominal resistance of FRP-strengthened concrete members will most likely be governed by debonding of FRP at the FRP–concrete interface

in compression as having a uniform stress of $0.85 f'_c$, over a depth $a = \beta_1 c$, in which c is the distance from compression face to the neutral axis and β_1 is the stress block factor specified in AASHTO LRFD Article 5.6.2.2.

When concrete compressive strain is less than 0.003 (in./in.) under conditions of force equilibrium, the concrete compression stress distribution shall be modeled according to the following equations:

$$f_c = \alpha_{1f} f'_c \quad (5.2.2-1)$$

in which:

$$\alpha_{1f} = 0.75 \text{ for } 2.5 \text{ ksi} \leq f'_c \leq 5 \text{ ksi} \quad (5.2.2-2)$$

$$\alpha_{1f} = 0.75 - 0.05 \times (f'_c - 5) \quad (5.2.2-3)$$

for $5 \text{ ksi} < f'_c \leq 10 \text{ ksi}$

where:

f_c = stress in concrete (ksi)
 α_{1f} = stress block factor when concrete compressive strain is less than 0.003 in./in. taken as the ratio of equivalent rectangular concrete compressive stress intensity to the compressive strength of concrete used in design

prior to the concrete compressive strain reaching 0.003 (in./in.) in the outermost concrete fiber in compression. When the compressive strain in the extreme fiber of the concrete is less than 0.003 (in./in.) at force equilibrium, the AASHTO LRFD equivalent rectangular concrete compressive stress block (AASHTO LRFD Article 5.6.2.2) 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.

The stress block represented in Equations 5.2.2-1, 5.2.2-2, and 5.2.2-3 is able to represent the concrete stress distribution when the compressive strain in the extreme fiber of the concrete is less than 0.003 in./in. It has a uniform compressive block that extends from the neutral axis (zero strain) to the outermost concrete fiber in compression, and is applicable to both rectangular sections as well as T-sections. The validation of the concrete stress block is presented in NCHRP Report 20-07/Task 428.

As per AASHTO LRFD Article 5.6.2.2, for sections that consist of a beam with a composite slab of different concrete strength, and the compression block includes both types of concrete, it is conservative to assume the composite beam to be of uniform strength at the lower of the concrete strengths in the flange and web. If a more refined estimate of flexural capacity is warranted, a more rigorous analysis method should be used.

5.3—SERVICEABILITY AND FATIGUE LIMIT STATES

5.3.1—Serviceability Stresses

Subjected to the Serviceability load combination specified in Article 3.4.1 of AASHTO LRFD, the compressive stress in the concrete, f_c , and the tensile stress in the steel reinforcement, f_s , of reinforced concrete members shall meet the following requirements:

$$f_c \leq 0.6 f'_c \quad (5.3.1-1)$$

$$f_s \leq 0.8 f_y \quad (5.3.1-2)$$

C5.3.1

Limiting the stress of the steel reinforcement under service load to 80 percent of the yield stress, and the compressive stress of concrete to $0.6 f'_c$ is equivalent to the recommendation of ACI Committee 440 (ACI 440.2R). The limitation in steel stress is based on the analytical work of El-Tawil et al. (2001).

Service stress checks for prestressed concrete members should be carried out as prescribed in Article 5.9.2.3.2 in AASHTO LRFD, and any variations approved by the Engineer of Record.

5.3.2—Fatigue Stresses

Subjected to the fatigue load combination specified in Article 3.4.1 of AASHTO LRFD, the stress in the FRP reinforcement, f_f , shall meet the following requirements:

$$f_f \leq \text{lesser of } \begin{cases} \eta E_f \epsilon_{fu} \\ E_f \epsilon_{fe} \end{cases} \quad (5.3.2-1)$$

where:

- E_f = modulus of elasticity of FRP reinforcement (ksi)
- ϵ_{fe} = effective strain in FRP reinforcement (in./in.)
- ϵ_{fu} = FRP ultimate strain (in./in.)
- 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 $\eta = 0.55$, 0.30 , and 0.20 shall be used for carbon, aramid, and glass fiber reinforcement, respectively.

5.4—STRENGTH LIMIT STATES

The following assumptions may be used for normal-weight concrete with design compressive strength up to 10 ksi.

5.4.1—Stress in Reinforcement at Nominal Flexural Resistance

For rectangular or flanged sections with bonded tendons utilizing seven-wire low relaxation prestressing strands subjected to flexure about one axis, where the approximate stress distribution in Article 5.2.2 is used, and for which f_{pe} is not less than $0.5f_{pu}$, the average stress in the prestressing steel may be taken as:

For 270 ksi strand:

$$f_{ps} = \begin{cases} 28,500\epsilon_{ps} & \text{for } \epsilon_{ps} \leq 0.0086 \\ 270 - \frac{0.04}{\epsilon_{ps} - 0.007} & \text{for } \epsilon_{ps} > 0.0086 \end{cases} \quad (5.4.1-1)$$

C5.3.2

Stress 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 0.55, 0.30, and 0.20 are based on the recommendation of the ACI Committee 440 (ACI 440.2R).

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

The effective strain in the FRP reinforcement should be based on Equation 5.4.1-6.

C5.4.1

The information provided for prestressed components with bonded tendons utilizing seven-wire low relaxation prestressing strands is based on the *PCI Bridge Design Manual* (2014). For bonded high-strength bars and unbonded sections, the appropriate stress-strain relationship should be used.

For 250 ksi strand:

$$f_{ps} = \begin{cases} 28,500\varepsilon_{ps} & \text{for } \varepsilon_{ps} \leq 0.0076 \\ 250 - \frac{0.04}{\varepsilon_{ps} - 0.0064} & \text{for } \varepsilon_{ps} > 0.0076 \end{cases} \quad (5.4.1-2)$$

The strain in the prestressing steel, ε_{ps} , is determined using strain compatibility.

$$\varepsilon_{ps} = \varepsilon_{pe} + \frac{P_e}{A_c E_c} \left(1 + \frac{e^2}{r_g^2} \right) + \varepsilon_{pn} \quad (5.4.1-3)$$

The depth to the neutral axis, c , is determined using the following equation:

For T-section behavior:

$$c = \frac{A_{ps}f_{ps} + A_s f_s + A_f f_{fe} - A'_s f'_s - \alpha_1 f'_c (b - b_w) h_f}{\alpha_1 f'_c b_w} \quad (5.4.1-4)$$

For rectangular section behavior, b_w shall be taken as b in Equation 5.4.1-4.

The internal strain and corresponding stress distribution within a prestressed T-section under flexure at the ultimate limit state is shown in Figure C5.4.1-1.

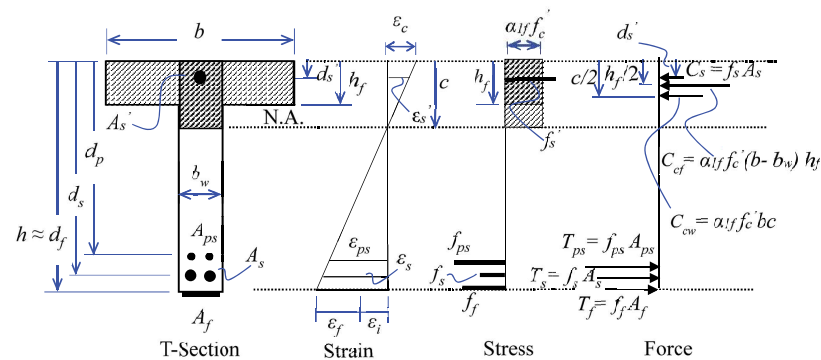


Figure C5.4.1-1—Internal strain and stress distribution within a prestressed T-section under flexure at the ultimate limit state

The effective stress in the non-prestressed reinforcement shall be taken as defined in Article 5.6.2.1 of AASHTO LRFD.

The effective stress in the FRP reinforcement shall be taken as:

$$f_{fe} = E_f \varepsilon_{fe} \quad (5.4.1-5)$$

in which the effective strain in the FRP reinforcement is given by:

$$\varepsilon_{fe} = \text{lesser of } \begin{cases} \phi_{EM} 0.6\varepsilon_{fu} \\ \phi_{Ei} \varepsilon_{fi} \\ \phi_{Ei} \left[\frac{\varepsilon_{CU} (d_f - c)}{c} - \varepsilon_i \right] \\ \phi_{Ei} \left[(\varepsilon_{pu} - \varepsilon_{pi}) \frac{(d_f - c)}{(d_p - c)} - \varepsilon_i \right] \end{cases} \quad (5.4.1-6)$$

where:

- A_c = Area of section calculated using the gross composite concrete section properties of the girder and the deck and the deck-to-girder modular ratio (in.²)
- A_f = area of FRP reinforcement (in.²)
- A_{ps} = area of prestressed steel tension reinforcement (in.²)
- A_s = area of non-prestressed steel tension reinforcement (in.²)
- A'_s = area of steel compression reinforcement (in.²)
- b = width of the compression face of the member (in.)
- b_w = web width (in.)
- c = distance from compression face to the neutral axis (in.)
- d_f = distance from compression face to the centroid of FRP reinforcement (in.)
- d_p = distance from compression face to the centroid of prestressed tension reinforcement (in.)
- E_c = modulus of elasticity of the concrete (ksi)
- e = eccentricity of prestressing steel (in.)
- f_{fe} = effective stress in FRP reinforcement (ksi)
- f_{ps} = average stress in prestressing steel at the time for which the nominal resistance of member is required (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)
- h_f = compression flange depth (in.)

The effective strain in the FRP reinforcement is governed by either the strain at the FRP–concrete interface at debonding from the concrete substrate, the strain in the FRP at concrete crushing, the strain at which the FRP ruptures, or the strain in the FRP at prestressing steel rupture.

For Grade 270 ksi and 250 ksi seven-wire low relaxation prestressing strands, ε_{pu} should be taken as 0.035 in./in. This is based on an ACI Committee 440 recommendation (ACI 440.2R).

The environmental reduction factors for the FRP material and FRP–concrete bond are defined in Articles 4.2.4 and 4.3.2, respectively.

- P_e = effective prestressing force (kip)
 r_g = radius of gyration of gross cross section (in.)
 α_{lf} = stress block factor when concrete compressive strain is less than 0.003 in./in. taken as the ratio of equivalent rectangular concrete compressive stress intensity to the compressive strength of concrete used in design (defined in Article 3.2.2)
 ε_{cu} = failure strain of concrete in compression (in./in.)
 ε_{fu} = FRP ultimate strain (in./in.)
 ε_i = strain in concrete substrate at time of FRP installation (in./in.) (5.4.1)
 ε_{pe} = effective strain in prestressing steel (in./in.)
 ε_{pi} = initial strain in prestressing steel (in./in.)
 ε_{pn} = net tensile strain beyond decompression in the prestressing steel (in./in.)
 ε_{ps} = strain in prestressing steel (in./in.)
 ε_{pu} = specified ultimate tensile strain of prestressing steel (in./in.)
 ϕ_{EM} = environmental reduction factor for FRP material
 ϕ_{Ei} = environmental reduction factor for the FRP–concrete bond

5.4.2—Factored Flexural Resistance

The calculation of the factored flexural resistance in rectangular and flanged sections shall be carried out in accordance with AASHTO LRFD Article 5.6.3.2. The approximate concrete stress distribution in Article 5.2.2 is used to determine the flexural resistance, M_r , taken as:

$$M_r = \phi M_n + \phi_f M_{nf} \quad (5.4.2-1)$$

where:

- M_n = nominal flexural resistance (kip-in.)
 M_{nf} = nominal FRP flexural resistance (kip-in.)
 ϕ = resistance factor defined in AASHTO LRFD Article 5.5.4.2
 ϕ_f = FRP resistance factor equal to 1.0 for flexure

C5.4.2

The factored resistance is in line with the design strength determination in accordance with AASHTO LRFD Article 5.6.3.2 and is written so that the design strength for a concrete flexural member is simply augmented by the contribution of the externally bonded FRP reinforcement. The FRP resistance factor $\phi_f = 1.0$ is based on NCHRP Report 20-07/Task 428, where the combined effect of the lower bound strain limits in Article 5.2.1 and the environmental reduction factors in Article 4.3.2.2 yield conservative values for the nominal flexural resistance.

. 2.1—Factored Strength

The factored flexural resistance, M_r , of a concrete T-section strengthened with FRP reinforcement externally bonded to the beam tension surface, where the compression flange depth is less than the neutral axis depth ($h_f < c$), shall be taken as:

$$M_r = \phi \left[A_{ps} f_{ps} \left(d_p - \frac{c}{2} \right) + A_s f_s \left(d_s - \frac{c}{2} \right) - A'_s f'_s \left(d'_s - \frac{c}{2} \right) - \alpha_{1f} f'_c (b - b_w) h_f \left(\frac{h_f}{2} - \frac{c}{2} \right) \right] + \phi_f A_f f_{fe} \left(d_f - \frac{c}{2} \right) \quad (5.4.2.1-1)$$

where:

d_f = distance from compression face to the centroid of FRP reinforcement (in.)

d_p = distance from compression face to the centroid of prestressed tension reinforcement (in.)

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

d'_s = distance from compression face to the centroid of compression reinforcement (in.)

h_f = compression flange depth (in.)

. 2.2—Nominal Strength

The nominal flexural resistance, M_n , of a concrete rectangular section strengthened with FRP reinforcement externally bonded to the beam tension surface, may be determined using Equation 5.4.2.1-1, where b_w shall be taken as b .

. 2.3—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 5.2.