Design and Construction Guidelines for Strengthening Bridges using Fiber Reinforced Polymers (FRP)

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This research concerns the development of guidelines for the design and use of externally-bonded FRP strengthening systems on bridges in Michigan. Six representative international FRP-related guidelines were analyzed and compared for applicability and effectiveness to MDOT needs. An experimental program involving natural and accelerated laboratory weathering was conducted to determine acceleration factors appropriate for the climate of Michigan. Bond pull-off and other specimen tests were used to establish rates of strength degradation, and an environmental reduction factor was estimated from these results. From the review of the existing guidelines as well as the test results, recommendations for design as well as installation, quality control, inspection, maintenance and repair were proposed that are based on modifications to as well as a synthesis of existing provisions.</td></tr>
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EXECUTIVE SUMMARY

Over the last two decades, various codes, standards, and other guidelines have been proposed for strengthening structures with externally-bonded fiber reinforced polymer (FRP) systems. However, these documents offer different design and installation procedures and recommendations, and were developed without reference to the particular needs of MDOT bridge components. From a review and analysis of these existing guidelines as well as experimental testing results, this project aims to provide suitable recommendations for design, construction, maintenance, and inspection when strengthening Michigan bridge elements with FRP composites. In particular, the primary goals of this study are to: develop a synthesis of representative design and use guidelines for FRP strengthening with specific attention to flexural, shear, and confinement; to develop recommendations for design and use specifically relevant to MDOT needs; and to construct selections of the recommendations based on laboratory and field testing.

As detailed in Chapter 2, the first task of this research was to conduct a comprehensive review of the existing FRP strengthening literature. From this review, six representative international guidelines were chosen for further analysis and evaluation:

- *American Concrete Institute (ACI) 440.2R-08, Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures;*
- *Intelligent Sensing for Innovative Structures (ISIS) Design Manual 4, FRP Rehabilitation of Reinforced Concrete Structures;*
- *American Association of State and Highway Transportation Officials (AASHTO) Guide Specifications for Design of Bonded FRP Systems for Repair and Strengthening of Concrete Bridge Elements;*
- *Japan Society of Civil Engineers (JSCE) Recommendations for Upgrading of Concrete Structures with use of Continuous Fiber Sheets;*
- *United Kingdom Concrete Society Technical Report 55 (TR55), Design Guidance for Strengthening Concrete Structures Using Fibre Composite Materials,* and;
- *Italian National Research Council Technical Document 200 (CNR-DT 200), Guide for the Design and Construction of Externally Bonded FRP Systems*.

In Chapter 3, the six guide provisions listed above were analyzed and compared with particular attention to flexural, shear, and confinement strengthening. In general, it was found that many analysis provisions were common to the six international guidelines, although various differences emerged when reduction factors, strain limits, and other details of implementation were included. For flexural strengthening, it was found that ACI, AASHTO, and ISIS were fairly consistent with capacity prediction. For the cases studied, ACI was found to have greatest capacity for lower FRP area and higher concrete strengths, while AASHTO and ISIS were found to have greatest capacity for higher FRP areas, and TR55 was found to be relatively conservative. In the case of shear, unfactored resistance values were similar, but when reduction factors were applied, AASHTO and ACI generally provided the largest design capacities while results for CNR, UK and ISIS were similar but lower. When the confinement of circular columns

was studied, it was found that CNR produced higher design values while AASHTO and TR55 were most conservative. For square columns, CNR similarly produced highest capacity while ACI was most conservative. However, the codes converge to similar values for the unfactored cases.

Chapter 4 presents the findings of a review of the installation and usage procedures presented in the guidelines. It was found that coverage of installation procedures varies widely among codes. While AASHTO and CNR contained little coverage of FRP installation, ACI and ISIS were the most comprehensive. Similarly, ACI and ISIS provided related coverage of evaluation and acceptance criteria, although ACI, TR55 and JSCE all present a relatively broad coverage of maintenance and repair. In contrast, CNR's coverage of maintenance and repair is brief, while ISIS and AASHTO do not provide detailed coverage of this subject.

As summarized in Chapter 5, a significant portion of the research effort of this project concerned experimental testing. The purpose of the laboratory testing was to assess FRP system degradation when exposed to the climate of Michigan and to develop recommendations for an acceleration and environmental reduction factor specific for Michigan weather conditions. In this effort, approximately 120 FRP-strengthened concrete specimens were prepared, and subjected to accelerated weathering as well as natural outdoor weathering. A valuable source of long-term Michigan exposure data was found from a 15 year old existing MDOT FRP installation that was included in the test results. Pull-off testing, flexural, and confinement strength tests were conducted to assess degradation and link accelerated test results to natural weathering results. In this effort, specimens strengthened with Sika, Fyfe, and BASF systems were subjected up to 240 freeze-thaw cycles and compared to natural weathering results. By linking bond degradation rates between the outdoor and laboratory-accelerated test results, short-term and long-term acceleration factors were developed that allow estimation of bond losses due to natural Michigan weathering based on accelerated test results.

From the results of the code analysis and testing portions of this project, Chapter 8 presents a series of recommendations for design and use. In general, the AASHTO design provisions were recommended with several modifications, including changes in strength reduction factors based in part on ACI guidelines, allowable strain limits when prestressed concrete girder strengthening is considered, and other factors. A second series of general recommendations for installation, inspection and quality control, and maintenance and repair were suggested based on a review of the available provisions. These recommendations were within the parameters set by existing guidelines on these topics.

CHAPTER 1: INTRODUCTION

1.1 Statement of the Problem

Highway bridges deteriorate from various causes such as environmental conditions, traffic loading, deferred maintenance, and collision damage. However, it is often expensive and disruptive to replace weakened structural members. Although various traditional methods are available to strengthen and repair bridge elements, a promising alternative is to utilize fiber reinforced polymer (FRP) composite materials. Advantages include a high strength-to- weight ratio, excellent corrosion resistance, and ease of installation. Common applications involve externally bonded composite fabrics on beams, columns, and bridge decks, where a significant improvement in compressive, shear, and flexural performance can be realized.

A significant number of general guidelines exist for the design and construction of FRP systems for strengthening structural components, as detailed in this report. However, there is a need for specific design guidelines for use of FRP composite materials for flexural, shear and confinement strengthening of bridge elements in Michigan. Under service conditions, Michigan bridges are subjected not only to traffic loads but also to a wide range of temperature and moisture changes. In winter, deicing salts are widely used, and their effect must be considered as part of the service environment. The effect of cold winters on FRP materials is of special concern. Sub-zero temperatures may cause changes in mechanical properties of FRP material and create additional damage, and the presence of deicing salts could accelerate such deterioration (Wu et al. 2006a). In summers, high temperature, especially coupled with high humidity, has a different but additional possible detrimental effect on FRP. The combination of complex environmental and mechanical loading complicates the durability assessment of FRP composites (Hollaway and Head 2001, Helbling et al. 2006), hence the long-term durability of FRP-bonded bridges remains a major concern. With these as well as other critical factors in mind, this project aims to provide a background for refined design, construction, maintenance, and inspection guidelines for strengthening deteriorated Michigan bridge elements with FRP composites.

1.2 Background

1.2.1 Introduction

The Michigan Department of Transportation (MDOT) conducts regular inspection of bridges and overpasses across the state. Of 4406 bridges recently inspected, 271 were classified as structurally deficient, while 634 bridges were classified as functionally obsolete. Moreover, 21% of the bridges inspected were designated as requiring some form of retrofit, modification, or upgrade to remove the deficiency and restore function (MDOT 2012). Bridge inspection and reporting in Michigan is part of the larger, federally mandated National Bridge Inspection Standards (NBIS) administered by the Department of Transportation (USDOT) Federal Highway Administration (FHWA) (USDOT-FHWA 2004**)**. The "2010 Status of the Nation's Highways, Bridges, and Transit: Conditions & Performance", published by FHWA (2010), presents similar deficiency levels for bridges nationally to that of the state of Michigan. Unsurprisingly, according to FHWA data, older bridges are more likely to be structurally deficient and functionally obsolete than newer ones. For example, the proportion of structurally deficient and functionally obsolete bridges exceeds 40 percent in the 55 to 59 years old category, and 50 percent in the 80 to 84 years old category (Memmott 2007).

The unique Michigan environmental conditions may further accelerate the rate of deterioration of highway bridges, as although bridges are expected to have a service life span of 50 to 100 years, many are showing signs of distress much earlier. Extreme temperatures in the summer and winter, including many cycles of freeze thaw, and the use of deicing salts, are factors that contribute to the progressive damage of bridge structural members (Staton & Knauff 2007). Other factors include heavy traffic loading, lack of adequate maintenance, and collision damage.
Depending on the bridge deficiency rating and nature, different corrective actions may be required. Traditional methods of strengthening and rehabilitating steel bridges include replacement of damaged structural members, repair of corroded beam ends, addition of stiffeners, and application of protective coatings (Wipf et al. 2003). For concrete bridges, traditional rehabilitation measures may include sealing of hairline cracks using epoxy injection, spot-patching of damaged areas, waterproofing, jacketing structural members to restore/increase their load carrying capacity, and cathodic protection against rebar corrosion. However, traditional methods have inherent limitations. For example, spot patching methods can mend corrosion-induced spalls, but typically do not retard chloride-induced corrosion since concrete containing chlorides is difficult to remove completely. The corrosion rates are observed to be higher at the perimeter of the patch and is independent of the type of patch material used (Tabatabai et al. 2005).

An alternative bridge rehabilitation method is to utilize FRP composite materials. Significant benefits include a high strength to weight ratio, excellent corrosion resistance, and ease of installation. Because of their light weight, FRP composites are cheaper to transport, require less or no scaffolding to install, and minimally adds to the structure's dead load. Due to the superior strength properties of FRP composites, only thin layers are needed to rehab beams and columns, minimally altering their dimensions. This is particularly important in maintaining bridge vehicular clearance and other tolerances within acceptable limits.

Various documents have been developed in the last two decades describing FRP strengthening systems. The most widely known guide in the US is ACI 440.2R, *Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures* (2008), though many others exist, as discussed in this report.

Although there exist a significant number of design guides, other than the referenced MDOT reports, these documents contain general FRP guidelines and consider generic applications, without reference to the particular needs of MDOT bridge components or Michigan vehicle load and weather conditions. Thus, a lack of knowledge exists for best practices for design, evaluation, construction, maintenance, and inspection procedures for FRP strengthened bridges in Michigan. Further, FRP materials and their technologies are relatively new to the Michigan construction industry, and hence a knowledge gap exists for best practices and implementation.

In general, FRP composites are composed of two major constituents: the fibers (usually of glass, aramid, or carbon), and a resin matrix. Typically, for the external application of FRP sheets, a layer of dry fiber sheet (usually unidirectional tape) is placed on top of a coat of polymer resin that hardens to bond the fiber sheet to the concrete structure. However, wet lay-up, precured, and near surface mounted construction techniques have been used in practice. When needed, multiple layers of fiber sheets can be sequentially added by repeating the same procedure. The resulting properties and potential failure modes of an FRP composite structure are a function of the properties of the fiber, the matrix, and their interfaces. The interface between the FRP sheet and the concrete is particularly important, since composite action requires a solid bond. Final failure is often caused by the debonding of the FRP sheet from the concrete substrate (Meier 1995, Buyukozturk and Hearing 1998). The degradation of a constituent in FRP over time affects various composite properties, and may even change the order of governing failure modes which may be matrix, fiber, or interface-dominated. This is a particularly important concern, as a FRP bonded structure could failure abruptly due to a change in dominant failure mode.

1.2.2 Current design approach concepts

FRP materials have been used for compressive, flexural, and shear strengthening of reinforced concrete structures since the late 1970s. FRP strengthening has since been established as a potentially efficient and economical technique for the repair and rehabilitation of deteriorating concrete structures, including bridges. To account for the complex behavior and various possible failure mechanisms of FRP bonded structures, extensive experimental investigations were carried out by numerous researchers (Seible et al. 1997, Mo et al. 2004, Nanni 2004, Ludovico et al. 2005, Walker and Karbhari 2006). For FRP bonded flexural beams, several failure modes were generally observed:

1. Crushing of the concrete in the compression zone before rupture of the FRP sheet or yielding of the reinforcing steel (brittle failure).
2. Yielding of the tension steel before concrete crushing or rupture of FRP sheet (ductile failure).
3. Yielding of the compression steel reinforcement of a doubly reinforced section (relatively ductile failure).
4. Rupture of the FRP sheet before steel yield and the compressive strain in the concrete is below its ultimate strain (the most brittle failure).
5. Anchorage failure (delamination) in the bond zone of the FRP sheet (often a ductile failure).
6. Peeling or shear/tension failure of the concrete substrate near the FRP sheet's cut off zone (brittle failure).

These six failure modes were classified into two types by Thomsen (2004). Type one includes modes exhibiting composite action up to failure, either due to concrete crushing, FRP rupture, or lack of shear resistance. Type two consists of failures by loss of composite action due to debonding of the FRP sheet, or by end peeling, where the concrete cover near the support regions peels off. To avoid detachment failure at the FRP/concrete interface, ACI 440.2R (2008) introduces a bond reduction factor (k_m) to limit the strain permitted in the FRP system. However, due to the high cost of experimental research, values for the bond reduction factor were based on a limited number of experimental investigations. Correspondingly, a critical factor potentially not adequately accounted for is the potential deterioration of the bond strength over time. According to ACI 440.2R, k_m is taken as a value no greater than 0.9, which reduces the usable strength of the FRP below its ultimate rupture strain. Even so, FRP rupture or delamination might occur if the FRP or bond strength later deteriorates. Thus for the flexural design of externally strengthened reinforced concrete beams, four failure modes are assumed possible, two corresponding to failure of the concrete in compression and two corresponding to failure of the FRP sheet (Choi et al. 2008): concrete crushing after steel yields; concrete crushing before steel yields; steel yield followed by FRP rupture; and debonding of the FRP at the FRP/concrete interface.

The additional shear strength that can be provided by the FRP is based on several factors, including the geometry of the beam or column, the wrapping technique, and the quality of the existing surface of the concrete. There are three typical types of FRP wrapping schemes, depending on the number of sides wrapped (4, 3, or 2). The four-sided wrapping scheme is most efficient. However, this is often impossible for bridge members, such as in the case of a T-beam integral with a deck slab above. In such situations, shear strength can also be improved by wrapping three sides (U-wrap) or bonding to two sides of the member, though the latter is the least effective (ACI 2007).

1.2.3 Delamination

A critical concern of externally bonded FRP reinforced structures exposed to severe weather environments is delamination, or debonding. Although detailed weather-induced bond deterioration is not fully understood, the results of debonding and its related failure phenomenon are well known. The quality of interfacial bonding has a strong influence on structural performance, as this significantly affects the composite action required for many applications, and ultimate failure of the strengthened component is often caused by debonding of the FRP sheet from the concrete substrate (Meier 1995, Buyukozturk and Hearing 1998).

1.2.4 Durability

The FRP-concrete bond line is the critical component to the effectiveness of most FRP structural strengthening applications, as this is the location where the transfer of stresses occurs. An exception to this would be a structural element that is confined with FRP, such as column wrapping. Field experience has shown that the bond between the composite and concrete cannot always be assured. The bond can degrade over time, eventually causing the system to become ineffective. Bond quality is influenced by the condition of the existing concrete, surface preparation of the concrete substrate, quality of the composite system application, quality of the composite and the durability of the epoxy primer and resin. A large number of parameters affect bond strength, including exposure to ultra-violet radiation, chemical activity, temperature, moisture, and stress level, as well as other factors (Karbhari 1997). As FRP composites for bridges have been in service for a relatively short period of time, there are few long-term data available to define environmental effects and the resulting degradation rate.

1.3 Objectives of the Study

The specific research objectives of this study are to:

- Develop a synthesis report that identifies primary factors in the selection, design, and use of FRP for strengthening, with application to specific bridge component failure modes such as flexure, shear, and confinement.
- Identify relevant design, maintenance, and inspection issues of FRP strengthening specific to MDOT bridges.
- Develop a user-friendly design procedure for FRP strengthening, a guide for construction, maintenance, and inspection, and accompanying examples of design calculations that demonstrate the methodology of FRP flexural, shear, and confinement strengthening and its application to Michigan bridge components.
- Validate selections of the recommended procedures with laboratory and field testing and a comparative strengthening example of an existing MDOT bridge.

1.4 Summary of Research Tasks

Task 1. Conduct a comprehensive review of the state-of-the-art technical literature.

Task 2. Address deficiencies in the existing guidelines.

Task 3. Conduct durability tests.

Task 4. Develop recommendations for flexure, shear, and confinement design and evaluation.

Task 5. Develop recommendations for use, construction, Inspection, and maintenance.

Task 6. Produce a strengthening design for an existing MDOT bridge using the recommendations for comparison and validation.

Task 7. Prepare project deliverables.

CHAPTER 2: LITERATURE REVIEW

2.1 Introduction

FRP has been used for numerous strengthening applications in various industries. However, common applications for bridge components involve externally bonded composite fabrics or jackets on beams, columns, and bridge decks, where significant improvements in compressive, shear, and flexural performance has been obtained (Nanni et al. 1992, Karbhari et al. 1993, Saadatmanesh et al. 1994, Chajes et al. 1995, Labossiere et al. 1995, Seible et al. 1997, Nanni 2000, Mo et al. 2004, Nanni 2004, Ludovico et al. 2005, Walker and Karbhari 2006; Mertz and Gillespie 1996, Miller et al. 2001, Tavakkolizadeh & Saadatmanesh 2003, Rizkalla et al. 2008, Elarbi 2011). A significant number of general guidelines exist for the design and construction of FRP systems, as identified further in this report. However, the combination of complex environmental and mechanical loading for Michigan bridges may complicate the durability assessment of FRP composites (Wu et al. 2006a, Hollaway & Head 2001, Helbling et al. 2006).

2.2 FRP Constituents

2.2.1 Fibers

FRP composites consist of two main constituents: a load bearing constituent, mainly fibers, and a polymeric matrix that serves as a binder and protector of the fibers. The matrix facilitates load transfer among fibers, and ensures that embedded fibers maintain their orientation and directional stability (Ansley et al 2009). As multi-phase materials, composites are generally anisotropic in nature exhibiting differing mechanical properties in 3 orthogonal directions. Properties of FRP composites have some variation depending on the manufacturing and fabrication processes employed (Elarbi 2011). Three types of reinforcing fibers are available commercially; glass, aramid, and carbon. Each type has different grades with varying properties. Carbon fibers have the highest modulus while glass fibers have the lowest modulus and largest elongation. All fiber types exhibit linear elastic behavior when tested.

The mechanical and physical properties of a representation of commercially available fibers are tabulated in Table 2.1 below. Values in the table are adapted from Mallick (2007). Note that the negative thermal expansion coefficient of aramid and carbon fibers indicates that shrinkage occurs when these materials are heated.

Glass fiber became commercially available in 1939 with the start-up of an Owens Corning production facility. Glass FRP (GRFP) composites have relatively low stiffness, high elongation, and moderate strength and weight. Glass is by far the most widely used fiber, because of the combination of low cost and excellent corrosion resistance. Glass fibers are classified into three types: E-glass, S-glass, and alkali resistant AR-glass fibers. Aramid fibers are synthetic and mainly used for aerospace and military applications, such as in ballistic rated body armor and as an asbestos substitute. Aramid fibers are sensitive to high heat and moisture. Carbon fibers have been commercially available since 1959. They are durable and perform very well under fatigue loading in hot moist environments.

Table 2.1 - Typical reinforcing fiber material properties

Fiber Type	Fiber Identification	Density (lbs/ft^3)	Tensile Modulus, (ksi)	Tensile Strength, (ksi)	Failure Strain, (%)	Thermal Expansion Coefficient, ($10^{-6}/^{o}F$)	Poisson's Ration
Glass	E-Glass	159	10,500	500	4.80	8.99	0.20
	S-Glass	155	12,600	625	5.00	5.22	0.22
Aramid	Kevlar 49	91	19,000	525	2.80	-3.60	0.35
	Technora	88	10,100	435	4.60	-10.79	0.35
Carbon	T-300	110	33,500	530	1.40	-1.08	0.20
	P-100	134	10,000	350	0.32	-2.61	0.20
	AS-4	112	36,000	590	1.65	-1.08	0.20
	IM-7	111	43,500	770	1.81	-1.35	0.20

Theoretically, carbon fibers could obtain mechanical properties of 15,000 ksi tensile strength and 145,000 ksi modulus of elasticity if the crystal structure can be optimally oriented and packed. However, if polymer chains are folded in the crystalline state, a typical occurrence, neither the theoretical strength nor modulus can be fully-developed. Carbon fiber composites are ideally suited to applications where strength, stiffness, lower weight, and outstanding fatigue characteristics are critical requirements. CFRP sheets and strips have been used to strengthen concrete structures such as beams, columns, slabs, piles, and decks (Elarbi 2011, Sika Corp. 2012).

2.2.2 Matrix

The most commonly used matrix for structural composites is thermosetting polymer. Polyester, vinyl-ester, and epoxy are the most common polymeric matrix materials used with high performance reinforcing fibers. They are all thermosetting polymers with good process ability and chemical resistance. Epoxies are more expensive than polyesters and vinyl-esters, but have in general better mechanical properties and outstanding durability. Thermoset polymers, including epoxy, are cured by chemical reactions, and the process of curing is irreversible (Elarbi 2011). Table 2.2 contains the mechanical properties of two commercially available epoxies widely used in FRP composites).

Table 2.2 - Mechanical properties of commonly used FRP epoxies

Epoxy Type	Sikadur 300 (psi)	Tyfo S Epoxy (psi)
Tensile Strength	8,000	10,500
Tensile Modulus	250,000	461,000
Tensile Elongation	3%	5%
Flexural Strength	11,500	17,900
Flexural Modulus	500,000	452,000

2.2.3 FRP strengthening systems

Early fiber composite applications have mainly been limited to aerospace, chemical and shipbuilding due to cost and research limitations (Emmons 1998). However, modern composites can be found in various forms from underground storage tanks to boat hulls and jet fighters. CFRP strip bonding for structural repair and strengthening applications was first introduced in Switzerland by Meier in 1984 (Barton 1997). Caltrans pioneered the use of FRP in the United States during the early 1990's by seismically upgrading bridge columns in California with GFRP fabrics (Sika Corp. 2012).

FRP is available in a variety of forms such as bars, grids, sheets, and tendons for prestressing. Modern rehabilitation methods include the use of FRP composite sheets in the form of beam wrapping to strengthen flexural and/or shear capacity, column wrapping to enhance seismic performance, bonded FRP flange plates to increase bending capacity, and epoxying FRP rods in grooves cut into the substrate to increase member strength (Khan 2010). One of the most flexible strengthening options is the use of externally-bonded (EB) FRP systems. Commercially available FRP systems are offers by suppliers such as Sika, Fyfe and QuakeWrap. Systems are offered in either unidirectional or bi-directional fiber orientation. These systems are made of carbon fibers, glass fibers, or, for bi-directional systems, carbon fibers in one direction and glass fibers in the other. Depending on the application, epoxy adhesives are an integral part of the system, and can be formulated to provide a range of application characteristics and mechanical properties when cured. Epoxy formulation provides the bond between the reinforcing plies (laminate) and the concrete base, the bond between different plies, and a protective coat to shield the laminate from the elements during service life. Tables 2.3 and 2.4 present the physical and mechanical properties of select unidirectional and bi-directional FRP strengthening systems offered by Sika and Fyfe.

2.2.4 Composite interfacial adhesion and debonding

The quality of the adhesive bond at the interfaces between plies and between concrete and the strengthening FRP layers is critical to the utilization of the composite constituents. Adhesive bond at the interface is achieved primarily by attaching the surfaces within a layer of molecular dimensions, that is, of the order of 0.1-0.5nm.

Table 2.3 - Properties of FRP systems from Sika and Fyfe

Physical & Mechanical Properties of Unidirectional FRP Strengthening Systems by Sika and Fyfe

		Sika Unidirectional (0°)						Fyfe Unidirectional (0°)							
	Unit	Sikawrap Hex 103C		Sikawrap Hex 117C		Sikawrap Hex 230C		Tyfo SCH-41		Tyfo SCH-41- 0.5X		Tyfo SCH-41- 2X		Tyfo SCH-41H	
Fiber Properties															
Tensile Strength	psi	550,000		550,000		500,000		550,000		550,000		550,000		675,000	
	Mpa	3,793		3,793		3,450		3,790		3,790		3,790		4,650	
Tensile Modulus	psi	33,000,000		34,000,000		33,400,000		33,400,000		33,400,000		33,400,000		42,000,000	
	GPa	228,000		234,000		230,000		230,000		230,000		230,000		289,600	
Ult. Elongation	%	1.50		1.50		1.50		1.70		1.70		1.70		1.70	
Density	$lbs/in.^3$	0.065		0.065		0.065		0.063		0.063		0.063		0.065	
	g/cc	1.8		1.8		1.8		1.74		1.74		1.74		1.8	
Laminate Physical Properties															
Fiber type		Carbon		Carbon		Carbon		Carbon		Carbon		Carbon		Carbon	
Color		Black		Black		Black		Black		Black		Black		Black	
Weight	OZ/Y^2	18		9		6.7		19		9.5		40		24	
	g/m^2	618		309		230		644		322		1424		814	
Ply Thickness	in	0.04		0.02		0.015		0.04		0.024		0.08		0.04	
	mm	1.016		0.51		0.381		1.0		0.6		2.0		1.0	
Cured Laminate Mechanical Properties															
		Test	Design	Test	Design	Test	Design	Test	Design	Test	Design	Test	Design	Test	Design
Tensile Strength	psi	123,000	104,000		105,000	129,800	104,000	143,000	121,000	137,000	116,000	143,000	121,000	200,000	170,000
	Mpa	849	717		724	894	715	986	834	944.6	799.8	986	834	1,380	1,170
Tensile Modulus	psi	10,239,800	9,446,600		8,200,000	9,492,300	8,855,000	13,900,000	11,900,000	14,500,000	12,300,000	13,900,000	11,900,000	15,500,000	13,100,000
	MPa	70,552	65,087		56,500	65,402	61,012	95,800	82,000	99,900	84,800	95,800	82,000	106,800	90,300
Tensile Elongation	%	1.12	0.98		1.0	1.33	1.09	1.00	0.85	0.95	0.80	1.00	0.85	1.30	1.10
Tensile Strength per inch width	lbs	4,928	4,160		2,100	1,947	1,560	5,720	4,840	3,288	2,784	11,440	9,680	8,000	6,800
	kN	21.9	18.5		9.3	8.7	6.9	25.4	21.5	14.6	12.4	50.9	43.1	35.6	30.2

Table 2.4 - Bidirectional laminate selection offered by Sika

		Sika Bidirectional ($0^o/90^o$)		
		Sikawrap Hex 113C		**Sikawrap Hex 115C**
Fiber Properties				
Tensile Strength	psi	500,000		550,000
	MPa	3,450		3,793
Tensile Modulus	psi	33,400,000		33,000,000
	GPa	230,000		234,500
Ult. Elongation	%	1.50		4.00
Density	lbs/in.^{3}	0.065		0.065
	g/cc	1.8		1.8
Laminate Physical Properties				
Fiber type		Carbon		Carbon
Color		Black		Black
Weight	OZ/Y^2	5.7		18.7
	g/m^2	196		638
Ply Thickness	in	0.01		0.04
	mm	0.25		1.00
Cured Laminate Mechanical Properties (with Sikadur Hex 300 epoxy)				
			Design	Test	Design
Tensile Strength	psi		66,000	83,980	70,870
	Mpa		456	579	489
Tensile Modulus	psi		6,000,000	7,017,555	6,149,730
	MPa		41,400	48,351	42,468
Tensile Elongation	%		1.20	1.14	0.98
Tensile Strength per inch width	lbs		660	3,359	2,835
	kN		2.90	14.9	12.6

The term ‘adhesion’ is associated with intermolecular forces acting across an interface and involves a consideration of surface energies and interfacial tensions. As a liquid, adhesives flow over and into the irregularities of a solid surface, coming into contact with it and as a result, interacting with its molecular forces. The adhesive then solidifies to form the “joint”. The basic requirements for good adhesion are thus intimate contact between the adhesive and the substrates and an absence of weak layers or contamination at the interface. Adhesive bonding therefore involves a liquid ‘wetting’ a solid surface, which implies the formation of a thin film spreading uniformly without breaking into droplets. Fundamentally, the surface tension of the adhesive should be lower than the surface energy of the solids involved, in this case, the treated surface of FRP and the exposed constituents of concrete. Because of the similarity in epoxy adhesive and composite matrix composition, values of surface tension and surface energy are similar. Both compositions contain polar molecular groups which are mutually attractive and chemically compatible. Thus, good adhesion is assured, provided that contamination is removed by surface preparation (Sika Corp. 2012). The quality of the adhesive bond at the interface has the potential to impact failure modes of an FRP composite structure. The interface between the FRP sheet and the concrete is particularly important, since composite action requires a perfect bond. Final failure is often caused by the debonding of the FRP sheet from the concrete substrate (Meier 1995, Buyukozturk & Hearing 1998). The degradation of a constituent in FRP over time affects various composite properties, and may even change the order of governing failure modes which may be matrix, fiber, or interface-dominated. This is a particularly important concern, as a FRP bonded structure could fail abruptly due to a change in dominant failure mode.

2.2.5 Current design standards and guides

Various documents have been developed in the last two decades describing FRP strengthening systems and provide guidelines for design assumptions and calculations. The most widely known guide in the US is ACI 440.2R, *Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures* (2008). A number of reports issued by the National Cooperative Highway Research Program (NCHRP) are design/construction guides of FRP systems and include:

- NCHRP Report 678, *Design of FRP Systems for Strengthening Concrete Girders in Shear* (Belarbi et al. 2011).
- NCHRP Report 655, *Recommended Guide Specification for the Design of Externally Bonded FRP Systems for Repair Strengthening of Concrete Bridge Elements* (Zureick et al. 2010).
- NCHRP Report 564, *Field Inspection of In-Service Bridge Decks* (Telang et al. 2006), which contains a manual for the in-service inspection of FRP bridge decks.
- NCHRP Report 514, *Bonded Repair and Retrofit of Concrete Structures Using FRP Composites* (Mirmiran et al. 2004), which describes recommended construction specifications and a control process for FRP strengthening including material preparation, application, a QA/QC plan, as well as training, qualification, inspection, and maintenance recommendations.

Other US and international guides include:

- AASHTO (2013), *Guide Specifications for Design of Bonded FRP Systems for Repair and Strengthening of Concrete Bridge Elements*.
- AC125 (2012), *Acceptance Criteria for Concrete and Reinforced and Unreinforced Masonry Strengthening Using Externally Bonded Fiber-Reinforced Polymer (FRP) Composite Systems.* AC125 is issued by ICC Evaluation Service to establish minimum requirements for the issuance of evaluation reports on fiber-reinforced polymer (FRP) composite systems under the 2012, 2009 and 2006 *International Building Code* (IBC) and the 1997 *Uniform Building Code* (UBC).
- ACI 440.3R (2004), *Guide Test Methods for Fiber-Reinforced Polymers (FRPs) for Reinforcing or Strengthening Concrete Structures.*
- ACI 440R (2007), *Report on Fiber-Reinforced Polymer (FRP) Reinforcement for Concrete Structures.*
- ACI SP-215 (2003), *Field Applications of FRP Reinforcement: Case Studies.*
- ISIS (2008), *Design Manual No. 4, FRP Rehabilitation of Reinforced Concrete Structures,* issued by the Canadian Network of Centers of Excellence on Intelligent Sensing for Innovative Structures.
- CEB-FEB (2001), Bulletin No. 14, *Externally Bonded FRP Reinforcement for RC Structures,* contains design guidelines for the use of FRP reinforcement in accordance with the design format of the CEB-FIP Model Code and Eurocode2.
- CNR-DT 200 (2004), *Guide for the Design and Construction of Externally Bonded FRP Systems.*
- Japanese Society of Civil Engineers (1997), *Recommendations for Design and Construction of Concrete Structures Using Continuous Fiber Reinforcing Materials.*
- Japan Concrete Institute (1997), *State of the Art Report on Retrofitting with CFRM, Guidelines for Design, Construction and Testing.*
- TCS (2000), *Design Guidance for Strengthening Concrete Structures Using Fiber Composite Materials*, Technical Report 55, UK.

A decade ago, MDOT released Research Report RC-1386, *Repair of Corrosion-Damaged Columns using FRP Wraps* (Harichandran and Baiyasi 2000), in which laboratory testing was conducted to assess the results of this particular application. In 2003, MDOT issued Research Report RC-1435, *Sensors to Monitor Bond in Concrete Bridges Rehabilitated with FRP* (Harichandran and Hong 2003), in which an experimental study to detect the debonding of EB CFRP on concrete structures was conducted using electrochemical impedance spectroscopy (EIS) sensing technology. The goal was to assess various environmental factors on debonding.

2.2.6 Designing with FRP reinforcement

A reinforced concrete element strengthened with FRP is a member consisting of three materials, concrete, steel, and FRP, where the FRP supplements the existing steel reinforcement. The application of FRP requires engineers to address the interaction of the three materials, with each material having different material properties and statistical variation. Design formulations addressing each material's contribution to the overall strength (R_n) of the element can be based on either suitable overall or individual reduction factors applied separately to each of the materials constituting the composite (concrete, steel and FRP), a combined reduction factor applied to the entire composite, or a combination of the two where both individual and combined reduction factors are applied to the system (Sika 2012). For example, ACI 440.2R-08 recommends the use of ACI-318 resistance (ϕ) factors applied to the overall element strength, in combination with an additional FRP reduction factor ψ, which takes into account the effects of FRP material property variation. The general philosophy can be expressed as: $\phi R_n = \phi$[steel & concrete + ψ (FRP strength)].

2.2.7 Flexural strengthening

In the ACI approach, which is similar to other provisions, the nominal moment capacity of a tension-reinforced section M_n can be obtained from eq. 1, where the first term is the moment capacity generated from the tension steel force and the second term is the moment capacity generated from the FRP sheet. Design capacity ϕM_n is then determined by reducing the nominal moment capacity by the appropriate resistance factor ϕ:

$$M_n = A_s f_s\left(d - \frac{\beta_1 c}{2}\right) + \psi A_f f_{fe}\left(h - \frac{\beta_1 c}{2}\right) \tag{2.1}$$

Parameter ψ represents a factor used to reduce the contribution of the FRP sheet, while other parameters in eq. 2.1 are similar to those used in typical reinforced concrete flexural capacity analysis (representing areas of tension material, A_s and A_f, their stresses at section capacity, f_s and f_{fe}, and their lever arms to the compression zone of the concrete, $d\text{-}\beta_1 c/2$ and $h\text{-}\beta_1 c/2$). A concern is that, in a recent study (Elarbi and Wu 2012), eq 2.1. was found to typically underestimate experimental results by 20% to 60%, which is discussed further below. However, of greater concern is the possibility of over-estimating capacity by not properly accounting for loss of FRP or bond strength over time, as outlined above.

An additional concern is the possibility of over-estimating capacity by not properly accounting for loss of FRP or bond strength over time. It was observed that more reliable strength reduction coefficients need to be developed to represent the long-term use of FRP exposed to the environmental and service parameters specific to a local region. To better develop design standards and construction guides, PennDOT commissioned a research program by selecting candidate bridges for NDT testing before and after the application of EB FRP for beam strengthening. Results from a finite element analysis of test data were used to develop draft PennDOT design standards and construction specifications and to apply lessons learned to the design and constructability of nearly 1,000 concrete T-beam bridges in Pennsylvania (Davalos et al. 2012).

2.2.8 Shear strengthening

FRP sheets have been shown to increase the shear strength of existing concrete beams and columns when used to wrap members (Chajes et al. 1995, Labossiere et al. 1995, Seible et al. 1997, Mo et al. 2004). The nominal shear strength of an FRP strengthened concrete member V_n

can be determined by adding the contribution of the FRP reinforcing (V_f) to the contribution of the steel shear reinforcement (V_s) and the concrete (V_c), as per eq. 2.2, which presents the ACI approach. The total design strength is then found by appropriately reducing V_n by the appropriate shear resistance factor ϕ.

$$V_n = \left(V_c + V_s + \psi V_f\right) \tag{2.2}$$

The contribution of the FRP sheet to shear strength is based on fiber orientation and the assumed shear crack pattern. The shear strength provided by the FRP reinforcement can be determined by calculating the force resulting from the tensile stress in the FRP across an assumed crack, in a similar fashion to the process used for design with steel stirrups. As given by ACI 440 (2007), the shear contribution from FRP is:

$$\mathrm{V_f} = \frac{\mathrm{A_f f_{fe}}\left(\sin\alpha + \cos\alpha\right)\mathrm{d_f}}{\mathrm{s_f}} \tag{2.3}$$

where A_f is the cross sectional area of the FRP sheet; f_{fe} is the tensile stress in the FRP reinforcement; d_f is the effective beam height; α is the orientation angle of the FRP and s_f is the spacing between adjacent FRP strips. To avoid delamination failure, mechanical anchorage can be used. However, the effective strain in FRP is limited (for example, to 0.004 per ACI 440), and the total shear reinforcement allowed is often limited as well. For example, FRP shear reinforcement in ACI is limited to the same criteria given for steel alone per ACI 318 (2011), given as: $V_s + V_f \leq 8\sqrt{f_c^{'}}\,b_w d$.

2.2.9 Numerical modeling

Numerous researchers have developed numerical models of FRP-strengthened concrete elements (Ouyang & Wan 2006, Ibrahim & Mahmood 2009, Obaidat et al. 2010, etc). Pesic and Pilakoutas (2003) conducted a finite element analysis to address concrete cover delamination and plate end failure of concrete beams with bonded flexural FRP reinforcement. Elarbi (2011) developed refined models in recent research using the finite element analysis code ABAQUS, to predict the deformation and failure of concrete beams strengthened with carbon FRP sheets applied to the bottom surface. This approach explicitly modeled the concrete and FRP materials with independent elements and nonlinear time-dependent material properties to account for deterioration. The two types of expected failure modes have been correctly predicted by the model: FRP rupture and FRP delamination. Two groups of beams were examined; one group with perfectly bonded FRP reinforcement and another group of aged beams with deteriorated FRP reinforcement bond. For the healthy FRP bonded beams in the study, the dominant failure mode was FRP rupture, where the concrete first began to crack on the midpoint of the tension side and propagated approximately one-third of the section height upwards, followed by rupture of the FRP sheet. For the aged (bond-deteriorated) FRP bonded beams, the dominant failure mode was delamination. FRP delamination began at the midpoint of the beam and then spread toward the beam ends. These two failure modes predicted from the model well-matched the observed experimental failure sequences.

Elarbi (2011) also conducted an extensive series of concrete beams strengthened with carbon FRP. Beams were conditioned to simulate the aging effect that weather-exposed components are expected to experience, and was conducted using various accelerated hygrothermal conditions. For the control specimens exposed to indoor conditions only, it was found that the ACI-based predictions of capacity significantly underestimated the failure load, whereas the numerical simulations agreed well with experimental results. Similar specimens were exposed to accelerated weathering cycles, under which bond strength deteriorated and led to delamination, a failure which is not currently considered in ACI 440 but accounted for in the numerical simulations.

2.2.10 Delamination

FRP delaminaton, or debonding, can be considered as the propagation of an interfacial crack with residual shear stress acting along the interface (Taljsten 1996, Leung & Tung 2006). In the case of FRP bonded to flat concrete members, debonding may occur at the end of the FRP sheet or initiate at an interior location where a stress concentration is present. Once debonding initiates, however, it may initially remain stable depending on how adjacent cracks interact, potentially allowing for further increases in load (Niu & Wu 1990, Chen et al. 2007).

To model debonding, an interfacial shear-slip relation is generally needed. Most interfacial relation models are based on two similar assumptions. First, the initiation of interfacial debonding begins when the interfacial stress has reached the prescribed interfacial strength (τ_s). Second, in the debonded zone, the residual shear stress (τ) softens linearly with the interfacial sliding (s), such as given by (Leung & Tung 2006): $\tau = \tau_0 - ks$, where τ_0 and k are the interfacial material parameters defining the initial residual shear stress and the shear softening rate after debonding. Based on this equation, the distribution of tensile stress (σ_p) along the debonded portion of FRP is calculated from:

$$\frac{d^2\sigma_P}{dx^2} + \alpha^2\sigma_P = 0 \tag{2.4}$$

Finally, in the elastic zone where debonding has not yet occurred, σ_p is given by:

$$\frac{d^2\sigma_P}{dx^2} - \beta^2\sigma_P = 0 \tag{2.5}$$

where α and β are parameters describing the structural configuration and material properties. Solving the above equations with appropriate boundary conditions provides the tensile stresses along the FRP sheet as well as interfacial shear stresses. Such an analytic approach can simulate the debonding process in great detail, permitting comparisons with experiments. However, such analytical solutions are available for simple cases only.

Another popular approach to model debonding is to use finite element analysis. By using suitable interface elements, very good results have been obtained (Wu et al. 2004, Mu et al. 2006, Elarbi & Wu 2012). This approach allows simulations of rather complex loading and geometrical configurations, but does not provide a detailed description of the delamination process. Therefore, both approaches may need to be considered to properly characterize delamination behavior.

2.2.11 Durability

Most FRP durability information has been gathered from laboratory simulations of harsh environments (Dutta & Hui 1996, Toutanji & Balaguru 1999, Karbhari et al. 2003). Karbhari (2004) found that low temperature thermal cycling appears to have a greater deteriorative effect on FRP than constant immersion at below-freezing temperatures, due in part to interface-level degradation. Here, micro-cracking at lower temperatures results in an increase of water absorption during warmer periods, followed by increased resin plasticization and hydrolysis. The expansion of frozen water collected in cracks results in debonding and transverse micro-crack growth (Rivera & Karbhari 2002). Various other researchers have found that such freeze/thaw exposures can lead to significant material degradation through matrix cracking and fiber-matrix debonding, as well as increased brittleness, resulting in a substantial change in the damage mechanisms commonly observed under ambient conditions (Dutta 1989, 1992, Lord & Dutta 1988, Haramis 2003, Karbhari et al. 1994, 2000, 2002). For glass and carbon-wrapped columns, however, freeze-thaw cycles alone appear to have no significant effect on compressive strength. It was also found that, using accelerated corrosion experiments, column wrapping significantly reduced reinforcing bar corrosion (Harichandran & Baiyasi 2000).

Wu et al. (2006, 2006a) completed a comprehensive study on the combined effects of low temperature thermal cycling, cycling frequency, sustained loads, and presence of salinity on FRP. It was found that sustained loads significantly accelerated degradation in all cases; that the significance of salinity depended on the cycling frequency; and that a high-humidity environment produced most damage. Moreover, when FRP materials were conditioned at higher temperatures, water absorption was increased under sustained loads (Gibson 1994, Kulkarni & Gibson 2003). Elarbi and Wu (2012) recently found that high temperature and high humidity environments have a very detrimental effect on the strength and stiffness of FRP materials, as well as the bonding between FRP and concrete. Although experimental data on bond deterioration due to natural weathering are unavailable, under accelerated laboratory environments, a large reduction (more than 80%) in bond strength between FRP and concrete in a high temperature environment has been reported, primarily due to the deterioration of the FRP material (Bank et al. 1998, Katz et al. 1999, Galati et al. 2006).

2.2.12 Accelerated weathering testing

At present, there is no standardized durability test procedure for FRP materials for infrastructure applications. However, an accelerated test procedure to simulate the effects of natural weathering on FRP has been developed by Wu et al. (2006, 2006a). This procedure, modified from ASTM C666 (2008), the standard freeze/thaw durability test for concrete, incorporates the combined effects of temperature, medium (i.e. immersion environment), and sustained load into a complete test program. Using this test procedure, it was found that failure modes of FRP composites most likely to be affected by environmental conditions are those associated with the polymer matrix material (Wu et al. 2006). It was also found that after 250 freeze/thaw cycles with less than 25% sustained load applied, flexural strength of an FRP-bonded concrete specimen experienced significant reductions if exposed to moisture (Wu et al. 2006).

Using tests similar to the above, degradation rates can be fundamentally calculated from the change in strength or stiffness versus time plots. However, to reduce experimental time, an accelerated test method uses one or several accelerating mechanisms to increase the rate of degradation. The rate of degradation under the accelerated condition is then related to the degradation rate under field service conditions by an acceleration factor, which is defined as the ratio of the degradation rate in the accelerated environment (i.e. laboratory) to that in the actual service environment. Acceleration factors for various environments may be determined using the framework outlined in ASTM E632 (1996). With appropriate acceleration factors known, short-term, relatively inexpensive laboratory experiments or FE simulations can be used to accurately simulate durability performance over real time (Yan 2005). Thus, the acceleration factor is a particularly valuable tool to evaluate degradation rates. Currently, however, appropriate acceleration factors for FRP use in Michigan are unknown.

2.2.13 Installation of externally bonded FRP systems

While installation procedures are straightforward, it is well recognized that even minor deviation from the prescribed procedures may dramatically affect the final system performance, impact adhesion quality, and cause premature delamination or bubbling at the concrete/FRP reinforcement interface (TRCF 1998). In many FRP composite applications, the FRP bond to the concrete substrate is critical. If the concrete is not properly prepared prior to wrapping, the FRP composite may not adhere adequately to the concrete surface. Surface preparation should include removal of all contaminants such as organic growth, old bituminous products, surface coats, oil and dirt, by high pressure water cleaning and/or sand blasting. If any of these materials remain, there are higher risks for bond deterioration (TRCF 1998). The concrete surface should also be dried and well cured before application of epoxy and FRP. A dry, open-pore structure in the concrete substrate allows the epoxy primer to be drawn in, forming “legs” into the concrete which create a mechanical anchoring bond. The concrete surface should also be free of any impregnating sealers. A simple test can be done by sprinkling water on the substrate and check for beading; the water should be absorbed immediately if the desired open pore structure exists.

Manufacturers of EB FRP products are aware of the importance of proper installation. Some offer extensive training modules for contractors, while others allow installation only through certified applicators. NCHRP 514 (2004) recommends that the manufacturer/supplier be prequalified for each FRP system after providing product data sheets, approved product testing data sets, and a comprehensive hands-on training program to qualify each FRP system contractor/applicator. It further recommends that the contractor is left responsible for the quality control of all materials and procedures in the project, and must submit a quality control and quality assurance (QC/QA) plan for approval. The plan should include specific procedures for personnel safety, tracking and inspection of all FRP components prior to installation, inspection of all prepared surfaces prior to FRP application, inspection of the work in progress to ensure conformity with specifications, QA samples, inspection of all completed work including necessary tests for approval, repair of any defective work, and clean-up. It was further recommended that any part of the work that fails to comply with the requirements be remedied or removed and replaced by the contractor. A proper installation plan includes adequate specification of safety procedures, equipment and materials preparation and procurement, surface preparation, precutting of fabric, determining the correct fabric to resin ratio, proper epoxy mixing and saturation of FRP sheets, how to properly apply the sheets to the structural member, and applying coatings as specified (Mirmiran et al. 2004).

CHAPTER 3: DESIGN PROVISIONS

3.1 Introduction

This chapter presents a review and evaluation of prevalent design guidelines for the use of externally applied FRP for strengthening reinforced concrete bridge members. Guided by the literature review presented in Chapter 2, six international representative strengthening guidelines covering practice in North America, Europe, and Japan were chosen for thorough investigation of the assumptions, procedures, and limits for design and analysis. The specific guidelines considered for review and evaluation are:

1. ACI 440.2R-08 - *Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures* (ACI 440.2R, 2008).
2. ISIS – *Design Manual 4 - FRP Rehabilitation of Reinforced Concrete Structures* (ISIS Design Manual 4, Version 2, 2008).
3. AASHTO *Guide Specification for the Design of Bonded FRP Systems for Repair and Strengthening of Concrete Bridge Elements* (AASHTO FRP Guide, 2013).
4. *JSCE Recommendations for Upgrading of Concrete Structures with use of Continuous Fiber Sheets* (JSCE Recommendations, 2001).
5. TR55 - *Design guidance for strengthening concrete structures using fibre composite materials* (TR55, 2000).
6. CNR-DT 200/2004 - *Guide for the Design and Construction of Externally Bonded FRP Systems* (CNR-DT 200, 2004).

Note that the review is limited to practically available documents. For example, CNR/2004, had a recent update, but this has not yet been translated to English, and thus was not included for evaluation in this report. The scope of the review in this chapter covers the analysis and design procedures for flexural strengthening, shear strengthening, and confinement using externally bonded FRP wrap. For reference, relevant and available code nomenclature is given in Appendix A.

Section 3.2 covers flexural strengthening, where the effect of concrete strength, FRP strength, existing steel reinforcement ratio, and other critical parameters, are considered for comparison among guidelines. Section 3.3 covers shear strengthening, where cases of continuous and spaced U-wrap, complete wrap, and two-sided strips are considered. Section 3.4 covers confinement, with consideration given to circular and square columns.

3.2 Flexural FRP Strengthening of RC/PC Bridge Members

3.2.1 Introduction

The items considered for review and comparison in this section are based on the ACI 440.2R-08 framework since it offers the most complete coverage of the subject. These specific items include:

- Strengthening limits
- Environmental reduction factors
- FRP strain limits
- Strength reduction factors
- Serviceability and service load limits
- Creep rupture and fatigue limits
- FRP end peeling and development length
- Flexural design approach and assumptions
- Nominal moment analysis and design procedure

The use of FRP strengthening systems can substantially enhance flexural strength. It has been documented that an increase in flexural strength from 10% to 160% can be obtained from FRP strengthening (Meier and Kaiser, 1991; Ritchie et al, 1991; Sharif et al, 1994). However, taking into account code-specified strengthening and ductility limits, strength increases of 40% are more reasonable.

3.2.2 Strengthening Limits

AASHTO, ACI, and ISIS set strengthening limits to qualify a structural member for strengthening. The limits ensure the member's ability to support a specified amount of service dead and live loads in the case of loss of strengthening due to construction error, severe environmental damage, vandalism, or fire (ACI 440.2R, 2008). Other guides emphasize the need for a thorough inspection of the existing structure, and a review of plans and specifications, the as-built plans, as well as any repair/maintenance documentation of the structure. Unlike AASHTO, ACI and ISIS, other guides leave the decision to strengthen a structure to the responsible agency on a case by case basis. The following paragraphs present strengthening limits specified by AASHTO, ACI, and ISIS.

3.2.2.1 AASHTO

The provisions of AASHTO are limited to concrete members with a specified compressive strength $f_c^{'}$ not exceeding 8 ksi and apply to bridge elements for which the factored resistance satisfies the following requirement (*AASHTO LRFD Bridge Specifications, 5th Edition*):

$$R_r \geq \eta_i[(DC + DW) + (LL + IM)] \qquad (3.2.1)$$

where:

R_r = factored resistance
DC = load effect due to component and attachments
DW = load effect due to wearing surfaces and utilities
LL = live load effect
IM = force effect due to dynamic load allowance
η_i = load modifier calculated using the following expression:

$\eta_i = \eta_D \eta_R \eta_I \geq 0.95$ (3.2.2)

where:

η_D = a factor relating to ductility;
≥ 1.05 for non-ductile components and connections
= 1.00 for conventional designs and details complying with the AASHTO LRFD Specifications
≥ 0.95 for components and connections for which additional ductility-enhancing measures have been specified beyond those required
For all other limit states: η_D = 1.00

η_R = a factor relating to redundancy
≥ 1.05 for non-redundant members
= 1.00 for conventional levels of redundancy
≥ 0.95 for exceptional levels of redundancy
For all other limit states: η_R = 1.00

η_I = a factor relating to operational classification
≥ 1.05 for critical or essential bridges
= 1.00 for typical bridges
≥ 0.95 for relatively less important bridges
For all other limit states: η_I = 1.00

Note that the resistance of components and connections is determined, in many cases, on the basis of inelastic behavior, although the force effects are determined by using elastic analysis. This inconsistency is common to most current bridge specifications as a result of incomplete knowledge of inelastic structural action.

3.2.2.2 ACI 440.2R-08

Similar to AASHTO, ACI sets limits to qualify a structure for strengthening. Here, two conditions must be met. The first condition is expressed by Equation 3.2.3 (ACI Equation 9.1) to ensure the structure's ability to sustain a minimum specified load after loss of strengthening Equation 3.2.3 states that the existing structural design capacity must equal or exceed the new service dead load by at least 10%, and must have the ability to support at least 75% of the new service live loads.

$(\varphi R_n)_{existing} \geq (1.1S_{DL} + 0.75S_{LL})_{new}$ (3.2.3 – ACI Eq. 9.1)

$(\varphi R_n)_{existing} \geq (1.1S_{DL} + 1.00S_{LL})_{new}$ (3.2.4)

To be considered for strengthening, a structural member must also meet the condition expressed by Equation 3.2.5 (ACI Equation 9.2), which ensures the structure is able to sustain load for the duration of its fire rating in the case of fire. The quantity $R_{n\theta}$ represents the nominal strength at elevated temperature in accordance with the guidelines of ACI 216R (ACI 216R-1989) or through testing. Fire scenarios considered should be in accordance with ASTM E119 (ASTM E119). The $R_{n\theta}$ value is to be computed for the structure's fire rating without considering the strength contribution from FRP. Figure 3.2.1 shows the required minimum relationship between $R_{n\theta}$ and the desired service live load, for various levels of existing dead load.

$$(R_{n\theta})existing \geq S_{DL} + S_{LL} \qquad \text{(3.2.5 - ACI Eq. 9.2)}$$

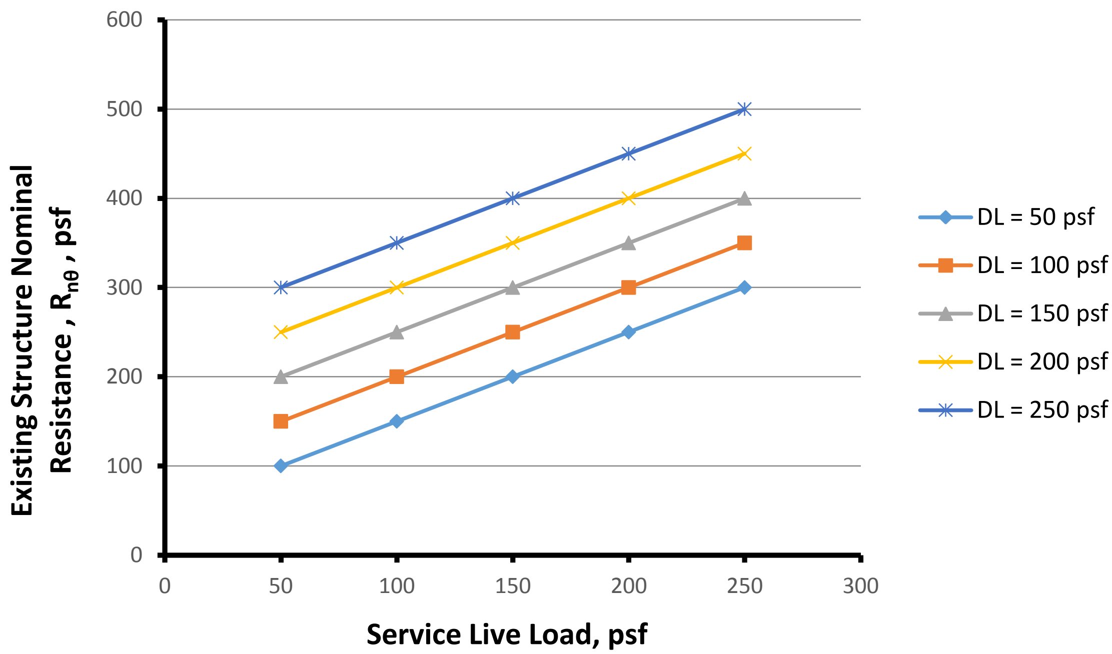

Figure 3.2.1 - ACI strengthening limits at elevated temperatures

3.2.2.3 ISIS

ISIS Design Manual 4 (Canada) recommends that an existing structure have the capacity to support dead loads and at least 50% of its live loads to qualify for strengthening (note although ISIS describes the requirement, it does not provide an explicit expression). The ISIS condition for strengthening may be expressed in a manner similar to that of ACI, as presented by Equation 3.2.6.

$$(\phi\, R_n)_{existing} \geq (1.0\, S_{DL} + 0.5\, S_{LL})_{new} \qquad (3.2.6)$$

The ISIS condition given above is specific to the S6-06 Canadian bridge code (CSA-S6-06, 2006). The live load factor of 0.5 serves as a minimum limit, but ISIS notes that it is subject to increase.

3.2.2.4 Other Codes

The UK Concrete Society Standard TR55 offers no specific quantitative expression as a set criterion to qualify a structure for strengthening. However, it states that the decision to strengthen should be guided by a careful assessment process that is independent of structure type and based on rigorous criteria and sound engineering judgment. TR55 refers to TR54 -*Diagnosis of deterioration in concrete structures* (TR54, 2000) and the Institution of Structural Engineers' *Appraisal of Existing Structures* (1996) for assistance in evaluating the condition of the structure.

Italy's CNR presents factors to consider that insure the durability of the member to be strengthened, while JSCE offers no specific quantitative expression or condition to serve as set criterion to qualify a structure for strengthening.

3.2.2.5 Summary

To illustrate the difference between the above strengthening limits, consider a case where the new service dead load (S_{DL}) is to be 50 psf and the new service live load (S_{LL}) is to be 140 psf. In this case, ACI limits strengthening to members that can support a minimum existing capacity of 160 psf, and, if fire is considered, the member must be able to sustain 190 psf. In contrast, the ISIS minimum load capacity is 120 psf.

Figure 3.2.2 presents a comparison of ACI, AASHTO, and ISIS strengthening limits. Note that ACI offers two scenarios, one for sustained live loads (generally not applicable to most bridges), and another for a typical transient live load. From the graph, it can be seen that AASHTO and ACI adopt similar limits for strengthening while ISIS is less conservative. The ACI fire endurance limit appears similar to the exiting strengthening limit provided by of AASHTO. Therefore, no special fire endurance condition appears needed and AASHTO provisions are recommended for use.

In summary, AASHTO and ACI have similar strengthening limits, where ISIS allows strengthening at a lower threshold of existing structure strength. As the LL/DL ratio increases, with differences increasing significantly while the remaining codes do not quantify strengthening limits.

3.2.3 Environmental Reduction Factors

Environmental factors are applied to FRP material properties to account for degradation in exposure conditions, as a function of exposure as well as FRP material and application.

3.2.3.1 ACI

ACI indicates that the values presented in Table 3.2.1 below (ACI Table 9.1) are conservative and based on the relative durability of each fiber type.

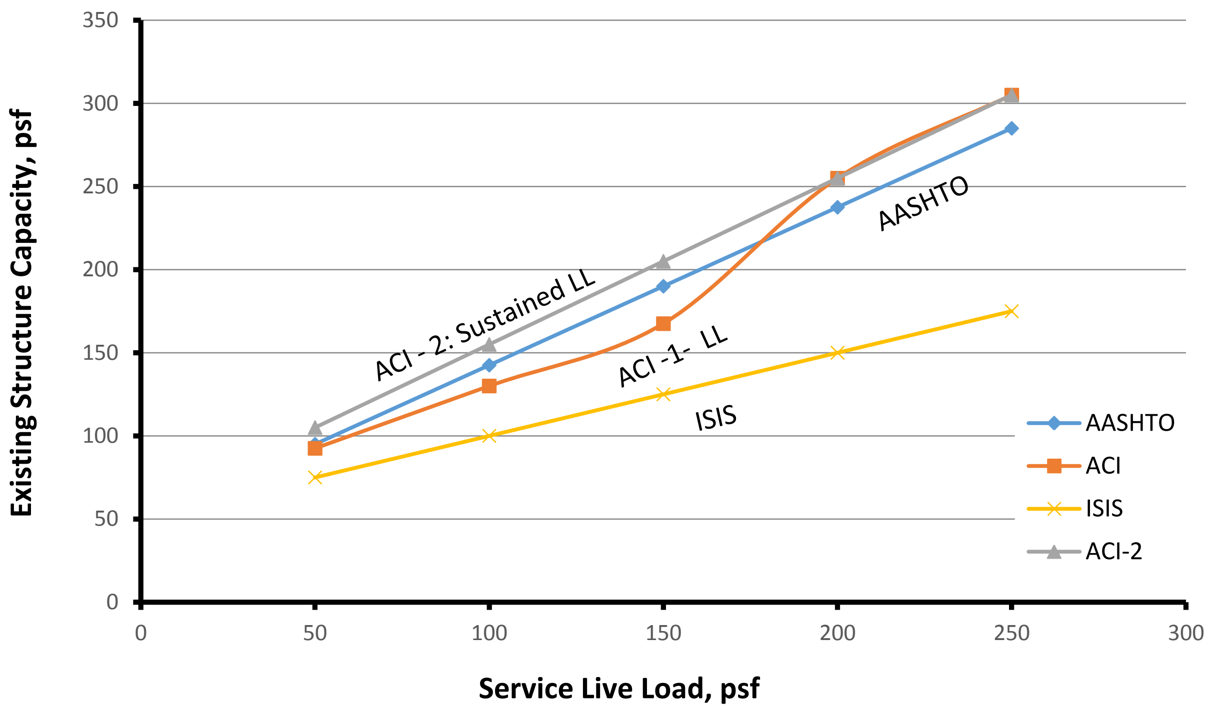

Figure 3.2.2 - FRP strengthening limits for different codes (with fixed dead load of 50 psf)

Table 3.2.1 – Environmental reduction factors

Exposure Conditions	Fiber type	Environmental reduction factor C_E
Interior exposure	Carbon Glass Aramid	0.95 0.75 0.85
Exterior exposure (bridges, piers, and unenclosed parking garages)	Carbon Glass Aramid	0.85 0.65 0.75
Aggressive environment (chemical and wastewater treatment plants)	Carbon Glass Aramid	0.85 0.50 0.70

3.2.3.2 CNR

The CNR code (Italy) recommends an environmental conversion factor ηa with values identical to those presented in the ACI standards above. The factor ηa is used to determine the maximum FRP design strain, ε_{fd}, defined as:

$$\varepsilon_{fd} = min\left\{\eta_a \frac{\varepsilon_{fk}}{\gamma_f}, \varepsilon_{fdd}\right\} \tag{3.2.7}$$

Generally, ε_{fdd} is the minimum governing value. Therefore, the environment factor usually does not govern the design. See Table 3.2.2 for values of η_a (Table 3.2.2 - CNR Table 3-4).

Table 3.2.2 – Environmental conversion factor H_A

Exposure Conditions	Type of fiber/resin	η_a
Internal	Glass/Epoxy	0.75
	Aramid/Epoxy	0.85
	Carbon/Epoxy	0.95
External	Glass/Epoxy	0.65
	Aramid/Epoxy	0.75
	Carbon/Epoxy	0.85
Aggressive environment	Glass/Epoxy	0.50
	Aramid/Epoxy	0.70
	Carbon/Epoxy	0.85

3.2.3.3 ISIS

ISIS does not directly specify an environmental reduction factor. Rather, ISIS considers "material resistance factors", which include an environmental reduction factor along with other partial safety factors combined in a single factor. ISIS presents two sets of factors depending on the code used; one for bridges and another for buildings. In this document, greater consideration is given to the Canadian bridge code S6-06 (CSA-S6-06, 2006) recommendations. Factors are developed based of several criteria including type of material, type of manufacturing process, and other durability and environmental considerations, as shown in Table 3.2.3 below (ISIS Table 3.3.4). Note that the reduction factor specific to pultruded CFRP (hand applied wet lay-up) is 0.56, which is considerably conservative compared to AASHTO's 0.85 reduction factor.

Table 3.2.3 - Material resistance factors

Material (installation process)	Bridges	Buildings
Pultruded Aramid FRP (NSMR)	$\phi_{FRP} = 0.60$	--
Pultruded Aramid FRP (externally-bonded plate)	$\phi_{FRP} = 0.50$	$\phi_{FRP} = 0.75$
Aramid FRP sheet (hand applied wet lay-up)	$\phi_{FRP} = 0.38$	$\phi_{FRP} = 0.75$
Pultruded Carbon FRP	$\phi_{FRP} = 0.75$	$\phi_{FRP} = 0.75$
Carbon FRP sheet (hand applied wet lay-up)	$\phi_{FRP} = 0.56$	$\phi_{FRP} = 0.75$
Pultruded Glass FRP	$\phi_{FRP} = 0.65$	$\phi_{FRP} = 0.75$
Glass FRP sheet (hand applied wet lay-up)	$\phi_{FRP} = 0.49$	$\phi_{FRP} = 0.75$
Concrete	$\phi_C = 0.75$	$\phi_C = 0.60$
Steel (passive reinforcement)	$\phi_S = 0.95$	$\phi_S = 0.85$
Steel (prestressing tendons)	$\phi_P = 0.95$	$\phi_P = 0.90$

3.2.3.4 TR55

The TR55 gives no explicit environmental factors. However, additional safety factors are used for FRP depending on the type and manufacturing process; these are discussed under the strength reduction factors section in this document.

3.2.3.5 JSCE

JSCE recommends using suitable reduction factors for environmental effects, and refers to several Japanese standards for further reference, including , *The Standard Specifications for Design and Construction of Concrete Structures* (1996), Proposed Specification for Durability Design of Concrete Structures, 1995 (limited to new construction), and *Guidelines for Maintenance of Concrete Structures* (1995). However, these guides are not available in English translation at the time of preparing this document.

3.2.3.6 AASHTO

AASHTO CFRP Standards (AASHTO FRP Guide, 2013) have no consideration for dedicated environmental reduction factors. However, *AASHTO LRFD Bridge specifications for GFRP-Reinforced Concrete Bridge Decks and Traffic Railings* (2009) recommends environmental reduction factors for GFRP bars. The recommended factors for embedded GFRP bars are applicable when concrete is exposed to earth and weather (see Table 3.2.4).

Table 3.2.4 - AASHTO environmental reduction factors for GFRP bars

Exposure condition	**Environmental Reduction Factor, C_E**
Concrete not exposed to earth and weather	0.80
Concrete exposed to earth and weather	0.70

3.2.3.7 Summary

In summary, ACI offers the most comprehensive coverage of environmental factors presented in Table 3.2.1 (ACI Table 9.1). CNR offers identical factors to ACI, but they are rarely incorporated in design since the condition to apply them (ε_{fd}) does not usually govern the design. Figure 3.2.3 presents a comparison of environmental reduction factors for ACI, CNR, and AASHTO (GFRP).

3.2.4 FRP Strain Limits

3.2.4.1 ACI

ACI sets limits to FRP strain in a strengthened section to prevent debonding cracks from developing. Equation 3.2.8 and 3.2.8 m (ACI Equation 10-2) below limit FRP strain at 90% of ε_{fu} or lower. The expressions are modifications of the work by Teng et al. (Teng et al., 2001, 2004) and are based on measured FRP strain at debonding cracks of flexural test samples.

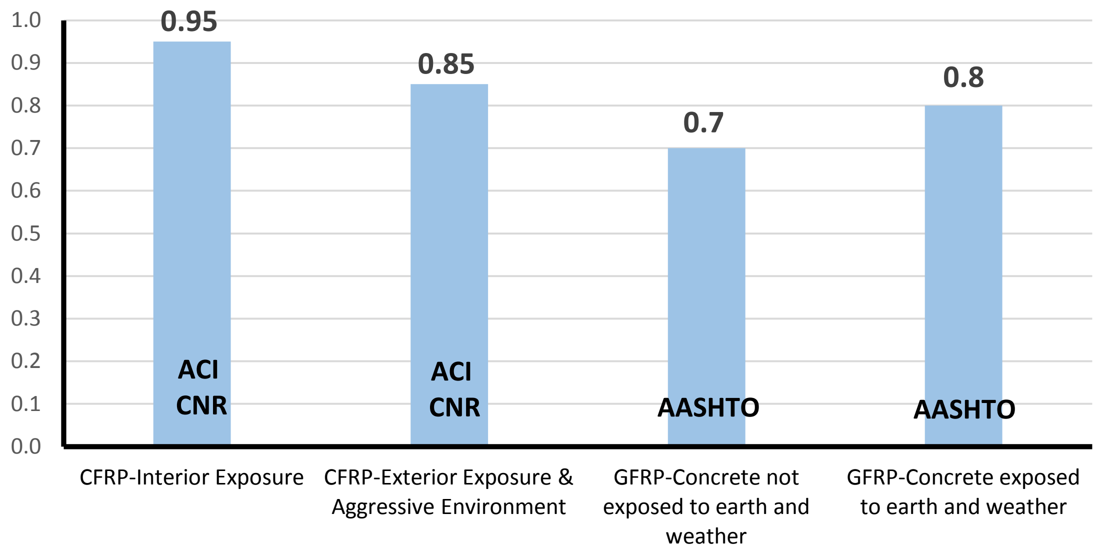

Figure 3.2.3 - Comparison of environmental reduction factors

$$\varepsilon_{fd} = 0.083\sqrt{\frac{f'_c}{nE_f t_f}} \leq 0.9\varepsilon_{fu} \quad in\ in.\!-\!lb\ units \qquad (3.2.8 – ACI Eq. 10.2)$$

$$\varepsilon_{fd} = 0.41\sqrt{\frac{f'_c}{nE_f t_f}} \leq 0.9\varepsilon_{fu} \quad in\ SI\ units \qquad (3.2.8 m – ACI Eq. 10.2)$$

The ACI code notes that transverse clamping of flexural samples has shown to improve bond behavior relative to that predicted by Equation 3.2.8 (ACI Eq. 10.2). Provision of transverse clamping FRP U-wraps along the length of flexural FRP reinforcement has been observed to increase FRP debonding strain by up to 30% (CECS- 146, 2003) prior to debonding failure.

To illustrate the condition expressed in Equation 3.2.8, assume a 4000 psi concrete beam strengthened with 3 plies of BASF MBRACE CF130 CFRP, where the thickness per ply is 0.0065 inch. Further assuming tensile properties as: ultimate tensile strength, f^*_{fu} = 550 ksi; tensile modulus, E_f = 33,000 ksi; ultimate rupture strain, ε^*_{fu} = 1.67%; nominal thickness, t_f = 0.0065 in/sheet. The application of Equation 2.8 (ACI Eq. 10-2) results in FRP strain ε_{fd} being limited to 0.0065 in/in, which is 46% of the ultimate FRP strain ε_{fu} value of 0.0142 in/in.

3.2.4.2 ISIS

ISIS considers debonding and anchorage failures as premature tension failures that require evaluation to reduce ε_{FRP} on a case by case basis through testing. However, the Code specifies a maximum value of ε_{FRP} = 0.006 as specified by the building code S806-02 (CAN/CSA-S806-02, 2002).

3.2.4.3 AASHTO

AASHTO requires that the strain developed in the FRP reinforcement at the ultimate limit state shall be equal to or greater than 2.5 times the strain in the FRP reinforcement at the point where the steel tension reinforcement yields (Equation 3.2.9). This limitation is present to ensure that the tension steel reinforcement yields before the point of incipient debonding of the externally bonded FRP reinforcement, thereby enabling the development of a ductile mode of flexural failure.

$$\frac{\varepsilon_{frp}^{u}}{\varepsilon_{frp}^{y}} \geq 2.5 \qquad (3.2.9)$$

With a maximum useable strain of 0.005 at the FRP reinforcement/concrete interface, the maximum strain developed in the FRP reinforcement is (Equation 2.10):

$$\varepsilon = 0.005 - \varepsilon_{bo} \qquad (3.2.10)$$

where:

ε_{bo} = The initial tensile strain at the bottom concrete surface as a result of the moment due to dead load (the existing tensile strain prior to FRP installation).

3.2.4.4 TR55

For strain limits, TR55 refers to Neubauer and Rostasy (1997), who suggest an ultimate limit of $5\varepsilon_y$ or half the ultimate plate strain, which for the materials tested was 0.75%. TR55 further suggests that other research has suggested somewhat lower limits, in the order of 0.6% for sagging moments and 0.4% for hogging moments. However, practical experience in the UK suggests that the higher strain limits are more reasonable. On this basis, TR55 recommends that, to avoid debonding failure, the strain in the FRP should not exceed 0.8% when the applied loading is uniformly distributed, and 0.6% if combined high shear forces and bending moments are present, such as where the load is concentrated at a point and at hogging regions close to supports.

3.2.4.5 CNR

Within CNR, the maximum FRP tensile strain, ε_{fd}, is calculated as follows:

$$\varepsilon_{fd} = min\left\{\eta_a . \frac{\varepsilon_{fk}}{\gamma_f}, \varepsilon_{fdd}\right\} \qquad (3.2.12)$$

where:

ε_{fk} = characteristic value of the adopted strengthening system
η_a = environmental factor
γ_f = partial factor for FRP rupture

$\varepsilon_{fdd} =$ maximum strain due to intermediate debonding

The above definitions are further explained in the CNR code.

3.2.4.6 JSCE

JSCE does not provide maximum FRP strain limit recommendations. In this context, JSCE cites the work to establish criteria of peeling of continuous fiber sheets by Wu and Niu (2000). The code describes the work as ongoing research and does not explicitly present research findings. JSCE uses equation 3.2.32 (JSCE Eq. 6.4.1) to set the FRP stress limit σ_f as a function of interfacial fracture energy G_f, which is determined from bond strength tests of FRP sheets to concrete. Refer to Equation 3.2.32 under Article 3.2.7.6 of this document for further description.

3.2.4.7 Summary

In summary, three standard guidelines have fixed strain limit values, AASHTO has a limit of 0.005 at the concrete/FRP interface, ISIS has a value of 0.006 (for bridges), and the TR55 strain value is 0.008 for uniform load application and 0.006 for combined high shear/bending applications. The effect of $f_c^{'}$ and the number of FRP plies are evaluated on these code limits, as presented in Figure 3.2.4. Here, $f_c^{'}$ was varied from 3 ksi to 8 ksi while the area of FRP plies is kept at 0.3315 in^2 (3 plies of BASF MBrace CF130). In Figure 3.2.5, the number of plies is varied from 1 to 5 (area changes from 0.11 to 0.55 in^2) while maintaining $f_c^{'}$ at 5500 psi. Figures 3.2.4 and 3.2.5 show the effect of these changes for various codes. From the graphs, it can be seen that the codes with fixed FRP strain limits are independent of changes of $f_c^{'}$ or the amount of FRP strengthening used. Only ACI and CNR exhibit an increase in the strain limit with an increase in $f_c^{'}$, and a reduction in strain limit when increasing the amount of FRP used. ACI appears to allow a maximum strain in excess of 0.009 at $f_c^{'} = 8\ ksi$. While this may appear less conservative, other factors such as environmental and strength reduction factors are used in combination with this limit. Note that ACI and CNR limits increase approximately linearly as $f_c^{'}$ increases while the limits decrease nonlinearly as FRP area increases.

The two graphs below (Figures 3.2.4 and 3.2.5) show that AASHTO's approach is relatively conservative and is similar to the results of ISIS, which is also relatively conservative (note that both of these codes are bridge codes, while the other codes are generally applicable). At 1 and 2 FRP plies, ACI reaches a relatively high strain limit value of 0.013. As FRP area increases, most guides converge between 0.004 and 0.006, except for the TR55 limit of 0.008. In general, ACI maximum strain limits are relatively unconservative at lower levels of FRP strengthening as well as at higher values of $f_c^{'}$.

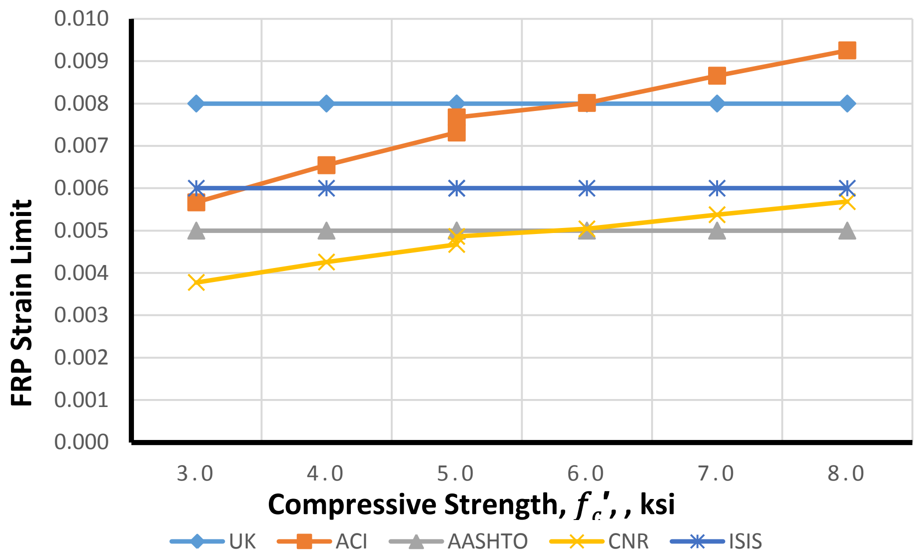

Figure 3.2.4 - Effect of concrete compressive strength f_c' on FRP strain limits (number of FRP layers = 3, area = 0.33 in^2)

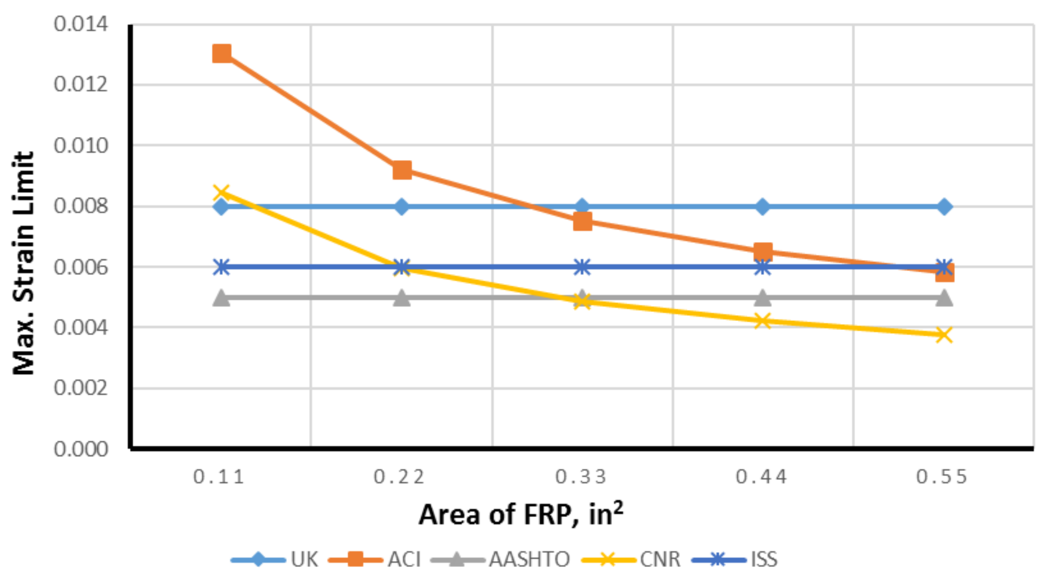

Figure 3.2.5 - Effect of amount of FRP strengthening on FRP strain limits, f_c' =5.5 ksi

3.2.5 Strength Reduction Factors

3.2.5.1 ACI

ACI follows the approach of ACI 318-05 philosophy in evaluating the strength reduction factor ϕ to promote ductile behavior. According to ACI 318-05, the strength reduction factor ϕ is applicable for steel grades with yield stress up to 60 ksi. The factor is a general factor applied to concrete, steel, and FRP equally. A maximum value of 0.9 is used when steel strain $\varepsilon_t \geq 0.005$ at ultimate capacity (i.e. when concrete has a strain value of 0.003 in compression), which represents a tension controlled failure characterized by ductile behavior. A minimum value of 0.65 is used when $\varepsilon_t \leq \varepsilon_{sy}$, signifying a compression controlled failure (non-ductile behavior). ϕ values follow a linear transition between the 2 extremes (see Equation 3.2-11 below). ϕ is applied to all moment components equally; it is considered a global factor.

$$\phi = \begin{cases} 0.90 \; for \; \varepsilon_t \geq 0.005 \\ 0.65 + \frac{0.25(\varepsilon_t - \varepsilon_{sy})}{0.005 - \varepsilon_{sy}} \quad for \; \varepsilon_{sy} < \varepsilon_t < 0.005 \\ 0.65 \; for \; \varepsilon_t \leq \varepsilon_{sy} \end{cases} \qquad \text{(3.2.11 –ACI Eq. 10-5)}$$

As observed, the values for ϕ depend on the steel strain level. Another recommended reduction factor specific to the FRP moment contribution is ψ_f, with a value of 0.85. The combined reduction factor for the FRP moment contribution is ϕ x ψ_f. ψ_f is a reduction factor to account for the uncertainty in the FRP-generated moment strength. Uncertainty is attributed variation in material properties and possible different failure modes observed for FRP-strengthened members (delamination of FRP reinforcement). The reduction factor is determined based on a statistical evaluation of variability in mechanical properties, predicted versus full-scale test results, and field applications. The FRP-related reduction factors are calibrated to produce reliability indices (β) typically above 3.5. However, reliability indices between 3.0 and 3.5 can be encountered in cases where relatively low ratios of steel reinforcement combined with high ratios of FRP reinforcement are used (Nowak and Szerszen, 2003; Szerszen and Nowak, 2003). Such cases are less likely to be encountered in design because they violate the strength increase limits of ACI Article 9.2 – Strengthening Limits.

Research conducted by Okeil (Okeil et al., 2007) presents the development of a resistance model for reinforced concrete bridge girders flexurally strengthened with externally bonded CFRP laminates. The resistance model is used to calculate the reliability index of CFRP strengthened cross-sections. From the mode, the reliability index for CFRP strengthened sections is greater than that for RC sections. This is primarily attributed to the low coefficient of variation for CFRP ultimate strength, which is lower than the coefficient of variation of the strength of steel or concrete. However, although reliability index improves with addition of CFRP, the flexural resistance of RC members strengthened with FRP is recommended to be reduced by 0.85 over a similar non-strengthened member. This recommendation is based on the recognition of the brittle nature of CFRP behavior.

3.2.5.2 ISIS

As noted earlier, ISIS combines various factors into "material resistance factors" which serve to account for resistance uncertainties as well as environmental reduction factors. ISIS presents two sets of factors, one for bridges and another for buildings. The factors provided are also classified by the FRP installation process, as shown in Table 3.2.3 (ISIS Table 3.4). The Table presents material resistance factors for concrete, steel, and several FRP schemes. The ISIS ϕ factors are applied separately to different materials.

The ISIS reduction value for carbon sheets using a wet lay-up is 0.56 for bridge applications and 0.75 for building applications. The ISIS low values are partially due to the fact that they represent the multiplication of 3 equivalent factors in ACI, namely C_E, ψ, and ϕ. ISIS refers to clauses in the S6-06 bridge code (Clauses 16.5.3 and 8.4.6) (CSA-S6-06, 2006) for further reference.

Table 3.2.5 below compares the resulting CFRP factor for a hand applied wet lay-up between ACI and ISIS, the latter of which results in a more conservative value.

Table 3.2.5 - ISIS and ACI reduction factor comparison

Type	ACI	ISIS
Environmental exterior exposure, CE	0.85	
Additional reduction Factor, y	0.85	0.56
$\boldsymbol{\phi}$	0.90	
Equivalent factor	0.65	0.56

3.2.5.3 AASHTO

AASHTO uses a fixed-value strength reduction factor ϕ_{frp} with a value of 0.85, for externally applied FRP. Note that the AASHTO strain limit is 0.005 for FRP, which limits the use of FRP to flexural members which will have relatively ductile failure conditions. In contrast, for comparison, in the case of internal GFRP bars used to reinforce bridges, AASHTO specifies the following values for ϕ:

$$\phi = \begin{cases} 0.55 & for\ p_f \le p_{fb} \\ 0.3 + .025\frac{p_f}{p_{fb}} & for\ \ p_{fb} < p_f < 1.4p_{fb} \\ 0.65 & for\ \ p_{fb} \ge 1.4p_{fb} \end{cases} \tag{3.2.13}$$

In which:

$$\rho_{fb} = 0.85\beta_1 \frac{f_c'}{f_{fd}} \frac{E_f \varepsilon_{cu}}{E_f \varepsilon_{cu} + f_{fd}} \tag{3.2.14}$$

where:

ρ_f = GFRP reinforcement ratio = A_f / bd

β_1 = Factor taken as 0.85 for concrete strength not exceeding 4 ksi. For concrete strengths exceeding 4 ksi, $\beta_1$is reduced at a rate of 0.05 for each 1 ksi of strength in excess of 4 ksi, except that β_1 is not be taken less than 0.65

f_c' = specified compressive strength of concrete, ksi

f_{fd} = design tensile strength of GFRP bars considering reduction for service environment, ksi

E_f = Modulus of elasticity of GFRP reinforcement, ksi
ε_{cu} = Ultimate strain in concrete
ρ_{fb} = GFRP reinforcement ratio producing balanced strain conditions

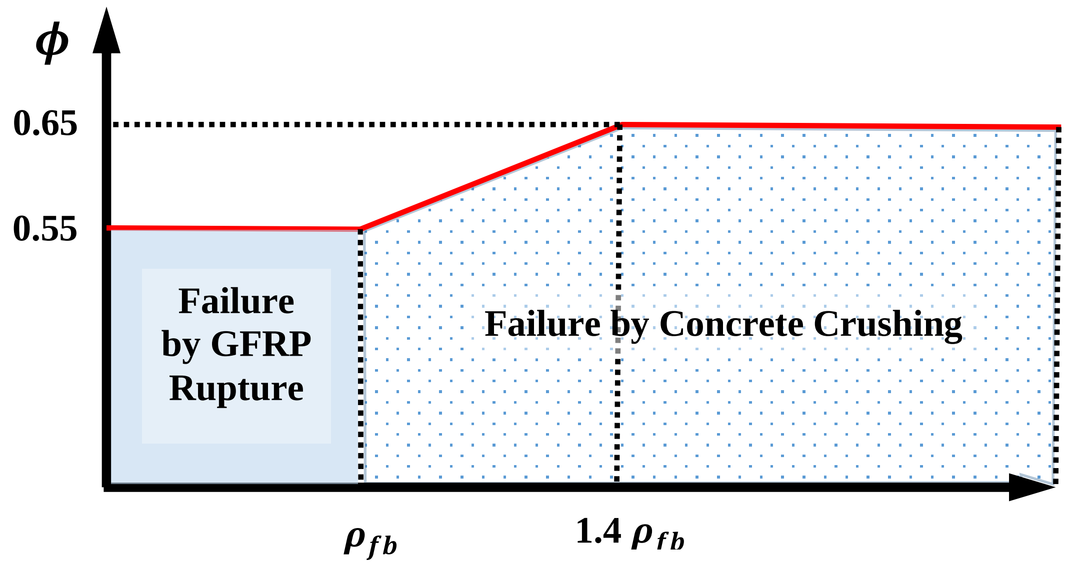

Figure 3.2.6 - Strength reduction factor for GFRP (adopted from AASHTO GFRP manual)

In ASSHTO GRRP, resistance factors ϕ for shear are to be taken as 0.75. If concrete strength is higher than specified, the member can fail due to GFRP rupture. To find a transition between two values of ϕ, a section controlled by concrete crushing is defined as a section in which $\rho_f \geq 1.4\ \rho_{fb}$, and a section controlled by GFRP rupture is defined as one in which $\rho_f \leq \rho_{fb}$.

3.2.5.4 TR55

In TR55 (UK), the characteristic material properties are divided by appropriate partial safety factors ($\gamma_{mf}, \gamma_{mm}, \gamma_{mE}$) to determine the appropriate design values. γ_{mf} is a factor based on the type of material used; γ_{mm} relates to the strengthening system and method of application, and γ_{mE} addresses the effect of material stiffness deterioration with time. The product of the three factors, ($\gamma_{mf} \cdot \gamma_{mm} \cdot \gamma_{mE}$), determines the final safety factor. Thus:

$$f_{mfd} = f_{fk} / (\gamma_{mf} \cdot \gamma_{mm} \cdot \gamma_{mE}) \qquad \text{(3.2.15 –TR55 Eq. 5.2)}$$

Where the values of ($\gamma_{mf}, \gamma_{mm}, \gamma_{mE}$) are determined from Tables 3.2.6, 3.2.7, and 3.2.8 (TR55 Tables 5.2, 5.3, and 5.4):

Table 3.2.6 - Example partial safety factors for strength at ultimate limit state (TR55 Table 5.2)

Material	Partial Safety Factor, γ_{mf}
Carbon FRP	1.4
Aramid FRP	1.5
Glass FRP	3.5

Table 3.2.7- Recommended partial safety factors for manufactured composites (TR55 Table 5.3)

Type of system (and method of application or manufacture)	Additional partial safety factor, (γ_{mm})
Plates	
Pultruded	1.1
Prepreg	1.1
Performed	1.2
Sheets or tapes	
Machine-controlled application	1.1
Vacuum infusion	1.2
Wet lay-up	1.4
Prefabricated (factory-made) shell	
Filament winding	1.1
Resin transfer molding	1.2
Hand lay-up	1.4
Hand-held spray application	2.2

Table 3.2.8- partial safety factor for modulus of elasticity at ultimate (TR55 Table 5.4)

Material	Factor of safety (γ_{mE})
Carbon FRP	1.1
Aramid FRP	1.1
Glass FRP	1.8

The product ($\gamma_{mf} \cdot \gamma_{mm}$) , labeled γ_{mF} , is a partial safety factor applied to the characteristic mechanical properties of the FRP system (Equation 3.2.16). The partial safety factor γ_{mF} is a function of the type of FRP material (γ_{mf}), and the manufacturing process (γ_{mm}).

$$\gamma_{mF} = \gamma_{mf} \cdot \gamma_{mm} \qquad \text{(3.2.16 –TR55 Eq. 5.3)}$$

The guide provides examples of how equation 3.2.16 (TR55 Eq. 5.3) is applied using the above tables. For example, for a carbon fiber pultruded plate, strength measured on the plate, γ_{mF} = 1.4 x 1.1 = 1.54; for an aramid sheet, strength measured on in situ specimens, γ_{mF} = 1.5 x 1.4 = 2.1; for a glass prefabricated shell, made by hand lay-up, γ_{mF} = 3.5 x 1.4 = 4.9.

In order to facilitate comparison of TR55 partial safety factors to strength reduction factors of other codes, the reciprocal values, (*1/x*), for TR55 are considered as an equivalent reduction factors. Figure 3.2.7 presents the equivalent reduction factors corresponding to γ_{mf} values in Table 3.2.6. Figure 3.2.8 presents equivalent reduction factor values corresponding to γ_{mm} of three select manufacturing processes from Table 3.2.7. Figure 3.2.9 presents equivalent reduction factors corresponding to γ_{mF} calculated values discussed in the previous paragraph.

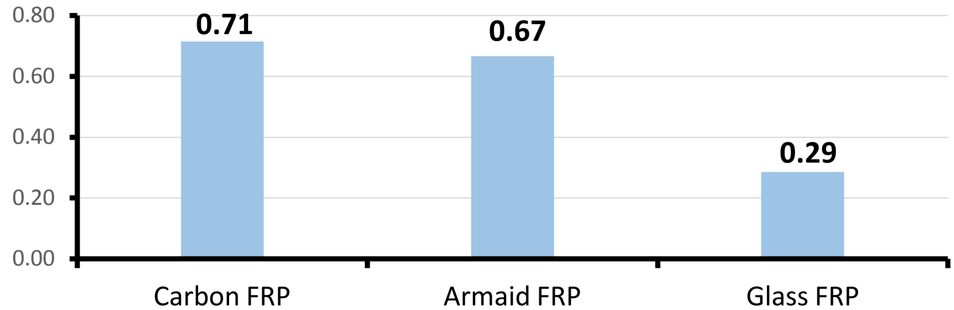

Figure 3.2.7 - Equivalent reduction factors corresponding to material partial safety factors, γ_{mf}, for ultimate strength limit state

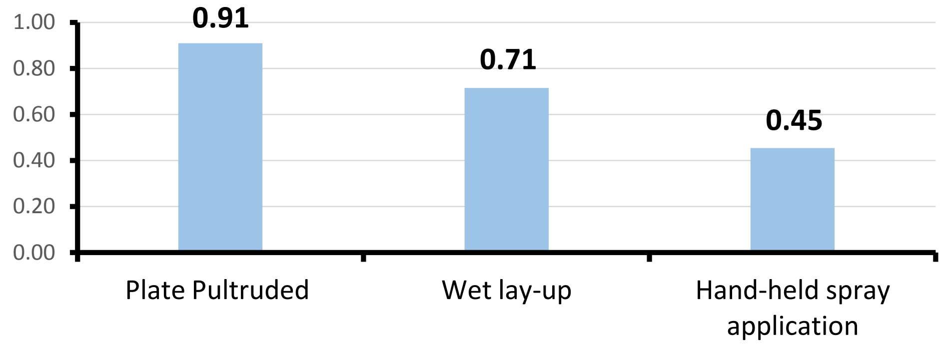

Figure 3.2.8 - Equivalent reduction factors corresponding to manufacturing process partial safety factors, γ_{mm}, for select processes

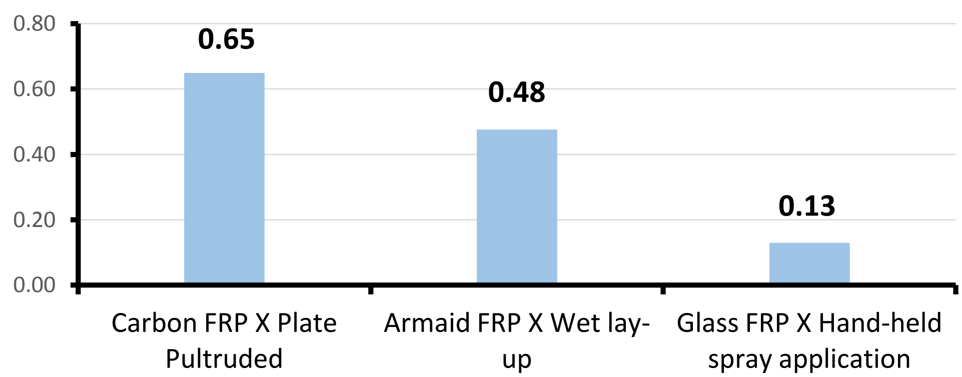

Figure 3.2.9 - Equivalent reduction factors corresponding to γ_{mf} of select materials and manufacturing processes

3.2.5.5 CNR

CNR specifies a partial safety factor, γ_{Rd} , that depends on the resistance model; either bending, shear, or confinement. These factors are applicable to ultimate limit states as presented in Table 3.2.9 (CNR Table 3-3).

Table 3.2.9 - CNR partial safety factor γ_{Rd}

Resistance model	$\boldsymbol{\gamma_{Rd}}$
Bending/Combined bending and axial load	1.0
Shear/Torsion	1.2
Confinement	1.1

Other partial factors include materials and products factors γ_m with values that depend on failure mode; either FRP rupture or FRP debonding. Values of γ_m are depending of application type; type A or type B. Type A applies to certified strengthening systems, while type B applies to uncertified strengthening systems. Certification of a strengthening system must be in accordance with CNR acceptance criteria stipulated in section 2.5 of the code (CNR-DT 200, 2004). Values of γ_m are presented in Table 3.2.10.

Table 3.2.10 – CNR partial factor for materials and products, γ_m

Failure mode	Partial factor	Type-A application[1]	Type-B application[2]
FRP rupture	γ_f	1.10	1.25
FRP debonding	$\gamma_{f,d}$	1.20	1.50

In the case of FRP wet lay-up systems, CNR considers the coefficients α_{fE} and α_{ff}. α_{fE} is a reduction factor for stiffness, while α_{ff} is a reduction factor for FRP tensile strength which should not exceed 0.90. In design applications, CNR considers partial safety factors of 1.15, 1.60, and 1.20 for steel, concrete, and FRP respectively. The reciprocals of the partial safety factors are considered equivalent reduction factors for steel, concrete, and FRP:

Steel equivalent reduction factor	= 0.87
Concrete equivalent reduction factor	= 0.63
FRP equivalent reduction factor	= 0.83

3.2.5.6 Summary

FRP strength reduction factor values are near 0.85 for the majority of codes. Certain codes offer separate factors depending of the material and manufacturing process used as in the cases of ISIS (Canada) and TR55 (UK). Depending on the manufacturing process, the factors considered by ISIS and TR55 are more conservative when compared to the fixed FRP reduction factor value for ASSHTO (0.85), and values considered by ACI. The multiplicative factors specified by TR55 and CNR can result in relatively large reductions in strength. The ACI resistance factor varies with the mode of failure and ductility. Depending on the application, ACI provides a more conservative strength reduction factor when compared with the fixed AASHTO factor of 0.85, but AASHTO is more restrictive with maximum FRP design strain. Table 3.2.11 summarizes the FRP reduction factors specified by various codes. Table 3.2.12 summarizes commonly used reduction factors of concrete, steel, and CFRP. Blank rows in the tables indicate non-applicable cases for the code considered.

Table 3.2.11 – Strength reduction factors with and without environmental factor

Code	**Application**	**Reduction Factor including environmental factor**	**Reduction Factor excluding environmental factor**
ACI	Interior Exposure Includes CE and ψ	0.81	0.85
	Exterior Exposure Includes CE, ψ, and ϕ	0.72	
ISIS	CFRP Sheet-Wet Lay-up	0.56	
	CFRP Plate-Wet Lay-up	0.75	
AASHTO	Fixed value for FRP		0.85
TR55	CFRP Wet Lay-up sheets (equiv.)		0.65
	CFRP Wet Lay-up pultruded plates (equiv.)		0.83
CNR	Fixed value for FRP		0.83

Table 3.2.12 - Reduction factor values for concrete, steel and CFRP

	ACI	**AASHTO**	**ISIS**	**UK**	**CNR**
Steel	$0.9 \geq \phi \geq 0.65$	$1.0 \geq \phi \geq 0.65$	0.90 Non-prestressed	0.87	0.87
			0.95 Prestressed		
Concrete			0.75	0.67	0.80
CFRP	0.85	0.85	0.56 Sheet-hand applied	0.65	0.83
			0.75 Pultruded		

3.2.6 Serviceability and Service Load Limits

3.2.6.1 ACI

For non-prestressed concrete members, ACI restricts stresses in reinforcing steel and concrete under service loads to avoid the development of inelastic deformation, especially when structures are subject to cyclic loading. The service limits for steel as well as concrete stresses are provided in Equations 3.2.17 and 3.2.18.

$$f_{s,s} \leq 0.80 f_y \qquad \text{(3.2.17 - ACI Eq. 10-6)}$$

$$f_{c,s} \leq 0.45 f'_c \qquad \text{(3.2.18 - ACI Eq. 10-7)}$$

Under service loads, the stress in steel can be computed using equation 3.2.19 (ACI Eq. 10-14), and the stress in the FRP strengthening system can be computed using equation 3.2.20 (ACI Eq. 10-15). The values calculated are compared to the service stress limits found in equations 3.2.17 and 3.2.18 (ACI Eqs. 10-6 and 10-7) to verify if serviceability conditions are met.

$$f_{s,s} = \frac{\left[M_s + \varepsilon_{bi} A_f E_f \left(d_f - \left(\frac{k_d}{3}\right)\right)\right](d-k_d)E_s}{A_s E_s \left(d - \left(\frac{k_d}{3}\right)\right)(d-k_d) + A_f E_f \left(d_f - \left(\frac{k_d}{3}\right)\right)(d_f - k_d)} \quad \text{(3.2.19 - ACI Eq. 10-14)}$$

$$f_{f,s} = f_{s,s}\left(\frac{E_f}{E_S}\right)\frac{(d_f-k_d)}{(d-k_d)}\varepsilon_{bi}E_f \quad \text{(3.2.20 - ACI Eq. 10-15)}$$

ACI does not provide references for equations 3.2.17 and 3.2.18 (ACI Eqs. 10-6 and 10-7).

3.2.6.2 AASHTO

AASHTO references *AASHTO LRFD Bridge Design Specifications*, 2010, for more detail on the load combinations specific to different limit states including serviceability, strength, extreme events and fatigue. Recommended values for AASHTO service stress limits are provided in Table 3.2.13 below.

3.2.6.3 ISIS

ISIS specifies no limits on concrete stresses under service loads, but stipulates that the stress in steel reinforcement under service loads may not exceed 80% of the yield stress, which is similar to ACI limit.

3.2.6.4 TR55

TR55 references BS 8110, part 2, 1985, or BS 5400, Part 4, 1990, for crack width limits not to be exceeded under service loads. A procedure to calculate crack width is referenced to Part 2 – section 3 of BS 8110, 1985. To avoid excessive deformations in bridges, the stresses in the steel reinforcement and concrete at working loads should not normally exceed $0.81f_y$ and $0.6f_{cu}$ (or 0.6 times the worst credible strength), respectively.

3.2.6.5 CNR

CNR specifies that when investigating a service limit state, stresses in the FRP system shall satisfy the limitation of:

$$\sigma_f \leq \eta\, f_{fk} \quad \text{(3.2.21)}$$

where:

f_{fk} = the FRP characteristic strength at failure.

η = a factor for environmental effects, long term effects, impact and explosive loadings and vandalism

CNR also specifies that the service stress in concrete and steel must be in accordance with limits stated in the current building code (CNR-DT 200, 2004).

3.2.6.6 JSCE

According to JSCE, cracks in FRP-strengthened members are relatively more dispersed than in unstrengthened members, and accordingly, the individual crack width is reduced. In pull-out tensile strength tests of members strengthened with CFRP sheets, the crack width is proportional to the average strain of the sheet and reinforcement, and is almost independent of the concrete cover, the steel rebar diameter, the rigidity of the continuous fiber sheets, and the compressive strength of concrete. At the level just before the steel yield point, the crack width is approximately 0.3 to 0.7 times the width of cracks in members not bonded with FRP.

For structures with large dead loads, JSCE recommends that crack width be calculated using Equation 7.4.1 of the *Standard Specifications for Design and Construction of Concrete Structures (Design*), 1996. The equation proposed is identical to Equation 3.2.22 below, but without the (0.7) multiplier.

For the specific case of large dead load, and the absence of concrete cracking prior to the application of FRP, or for structures governed by live loads, the flexural crack width may be calculated using Equation 3.2.22 (JSCE Eq. C6.5.1), in which the crack width is taken as 70% of the width calculated from Equation 7.4.1.

$$w = 0.7k[4c + 0.7(C_s - \phi)]\left[\frac{\sigma_{se}}{E_s}\left(or\,\frac{\sigma_{pe}}{E_p}\right) + \varepsilon'_{cs}\right] \qquad \text{(3.2.22 - JSCE Eq. C6.5.1)}$$

3.2.6.7. Summary

The service stress limits for different codes are summarized in the Table 3.2.13. The same steel limit is shared by all standards except JSCE and CNR codes. AASHTO and CNR allow the highest FRP limits, while ACI specifies the lowest.

Table 3.2.13 - Summary of service limit state

	ACI	**AASHTO**	**ISIS**	**UK**	**CNR**
Steel	$0.80f_y$	$0.80f_y$	$0.80f_y$	$0.80f_y$	
Concrete	$0.45f'_c$	$0.36f'_c$		$0.60f_{cu}$	
CFRP	$0.55f_{fu}$	$0.80f_{fu}$		0.65	$0.80f_{fu}$

3.2.7 Creep Rupture and Fatigue Stress Limits

3.2.7.1 ACI

According to ACI, of the available fiber types, CFRP is least affected by creep rupture. After 500,000 hours which ACI roughly equates to "about 50" years (actually 57), CFRP is predicted to maintain 90% of its initial ultimate stress (Yamaguchi et al. 1997; Malvar 1998). The values for GFRP and AFRP are 30% and 50% of the CFRP limit, respectively. Taking 0.6 as a safety factor (ACI 440.2R), the limit values for GFRP, AFRP, and CFRP are 20%, 30%, and 55% as shown in Table 3.2.14 below.

Table 3.2.14 – ACI sustained plus cyclic service load stress limit in FRP reinforcement

Fiber type		
GFRP	**AFRP**	**CFRP**
$0.20f_{fu}$	$0.30f_{fu}$	$0.55f_{fu}$

The tabulated limits in Table 3.2.14 represent fiber stress limits under service conditions $f_{f,s}$. An evaluation of $f_{f,s}$ using Equation 3.2.20 must be less or equal the tabulated limit in Table 3.2.14.

3.2.7.2 ISIS

ISIS provisions to guard against fatigue failure include limits on the difference between maximum and minimum stresses in the steel bars to 125 MPa (CAN/CSA S6-06 bridge code, 2006). ISIS imposes limits on the service stresses providing different limits for bridges and buildings to protect against creep rupture. Table 3.2.15 (ISIS Table 5.1) below presents these limits for AFRP, CFRP, and GFRP. ISIS limits are slightly higher than those provided by ACI since they are specific to creep rupture protection only, while the ACI limits include cyclic service loads.

Table 3.2.15 – ISIS maximum stress level against creep rupture

Material	Bridges	Buildings
Aramid FRP	0.35 f_{FRPU}	0.38 f_{FRPU}
Carbon FRP	0.65 f_{FRPU}	0.60 f_{FRPU}
Glass FRP	0.25 f_{FRPU}	0.25 f_{FRPU}

3.2.7.3 AASHTO

AASHTO imposes an FRP strain limit of 0.005 and at ultimate capacity, a maximum allowable FRP strain level of 2.5 times the FRP strain at steel yield. Subjected to the fatigue load combination of the AASHTO LRFD Bridge Design Specifications, the maximum compression strain in the concrete, ε_c, the strain in the steel reinforcement, ε_s, and the strain in the FRP reinforcement, ε_{frp}, are given as:

$$\varepsilon_c \leq 0.36 \frac{f_c^{'}}{E_c} \tag{3.2.22}$$

$$\varepsilon_s \leq 0.8\varepsilon_y \tag{3.2.23}$$

$$\varepsilon_{frp} \leq \eta \varepsilon_{frp}^u \tag{3.2.24}$$

where:

ε_{frp}^u = The characteristic value of the tensile failure of FRP reinforcement.

By limiting the maximum strain in the concrete to the above value, the stress range in the concrete is kept less than $0.40f_c^{'}$. Limiting the steel reinforcement strain under service load to 80% of yield strain is equivalent to the recommendation of ACI Committee 440, where the stress in the reinforcing steel under service load is limited to 80% of yield stress (Equation 3.2.17). This recommendation is based on the work of El-Tawil et al. (2001); Shahawy and Beitelman (1999, 2000); and Barnes and Mays (2000). It was found that concrete strengthened with CFRP and subjected to cyclic fatigue leads to stress results similar to those obtained for static creep, and limiting the service load stress of steel bars in reinforced concrete beams strengthened with CFRP to $0.85f_y$ is adequate.

Strain limits on the FRP reinforcement are also specified to avoid creep-rupture. AASHTO suggests that the creep rupture reduction factor, η should be based on experimental data and in the absence of such data, a value of η = 0.8, 0.5, and 0.3 shall be used for carbon, aramid, and glass fiber, respectively. However, FRP strains investigated under the service load combination are usually sufficiently low that creep rupture of the FRP is typically not of concern. Note that AASHTO, FIB 14 (Swiss code), and ACI base the selection of η values on the work of Yamaguchi and Malvar (1997), although ACI recommends using 0.9 rather than 0.8 for carbon.

Per AASHTO, the investigation of material strain limits can be done as follows. If the section cracking moment M_{cr} is less than the moment caused by the fatigue load combination, the portion of the concrete in tension is neglected and a transformed section of the cracked, FRP and steel-reinforced section is developed. The cracking moment can then be evaluated per Eq. 3.2.25.

$$M_{cr} = f_t \frac{I_g}{t} \tag{3.2.25}$$

where:

$$f_r = 0.24\sqrt{f_c^{'}} \tag{3.2.26}$$

From the FRP reinforcement load-strain data, E_{frp} can be evaluated.

$$E_{frp} = \frac{f_{frp}}{\varepsilon_{frp}} = \frac{N_b/t_{frp}}{\varepsilon_{frp}} \tag{3.2.27}$$

Modular ratio for concrete: $n_c = \frac{E_c}{E_{frp}}$ (3.2.28)

Modular ratio for steel $n_s = \frac{E_s}{E_{frp}}$ (3.2.29)

Once the transformed section is developed and the neutral axis location (z) is identified, strains in the concrete, steel reinforcement and FRP due to fatigue can then be calculated using Equations 3.2.30, 3.2.31, and 3.2.32.

$$\varepsilon_c = \frac{M_f z}{I_T E_{frp}} \tag{3.2.30}$$

$$\varepsilon_s = \frac{M_f (d-z)}{I_T E_{frp}} \tag{3.2.31}$$

$$\varepsilon_{frp} = \frac{M_f \left(h+t_{frp}-z\right)}{I_T E_{frp}} \tag{3.2.32}$$

3.2.7.4 CNR

CNR expresses the effect of fatigue, cyclic loading and continuous (creep) stress by use of a conversion factor for long term effects, η_1. Values for η_1 are provided in Table 3.2.16 for several FRP systems.

Table 3.2.16 - Conversion factor for long term effects for several FRP systems for service limit states

Loading mode	Type of fiber/resin	$\boldsymbol{\eta_1}$
Continuous (Creep and relaxation)	Glass/Epoxy	0.30
	Aramid/Epoxy	0.50
	Carbon/Epoxy	0.80
Cyclic (fatigue)	All	0.50

3.2.7.5 TR55

TR55 recommends that checks for fatigue should be carried out in accordance with the recommendations in Clause 4.7 of BS 5400, *Steel, Concrete, and Composite Bridges-Part 4, 1990*, and the stress range in the FRP should be limited to the appropriate values given in Table 3.2.17 (TR55 Table 6.1).

Table 3.2.17 – Maximum Stress range as a percentage of the design ultimate strength

Material	Stress range (%)
Carbon FRP	80
Aramid FRP	70
Glass FRP	30

TR55 stipulates that a stress rupture of the FRP may occur under sustained service loads. The code recommends that the maximum stress in the FRP at service loads, as a proportion of the design strength, should not exceed the values given in Table 3.2.18 (TR55 Table 6.2).

Table 3.2.18 - Maximum stress range under service loads to avoid stress rupture for different materials

Material	Maximum range (%)
Carbon FRP	65
Aramid FRP	40
Glass FRP	55

Figures 3.2.10 and 3.2.11 are visual representations of Tables 3.2.17 and 3.2.18 respectively. Figure 3.2.10 presents the permitted maximum service stress range as a percentage of the ultimate strength of the FRP material to protect against fatigue failure for bridges exposed to repeated live loads. Figure 3.2.11 presents the maximum service stress to protect against stress rupture. From the data, carbon fibers perform best under fatigue loads and to resist creep rupture. Aramid fiber, while performs well under fatigue load conditions, it performs poorly to protect against creep rupture. Glass fiber performs poorly under fatigue loading, but moderately resists creep rupture.

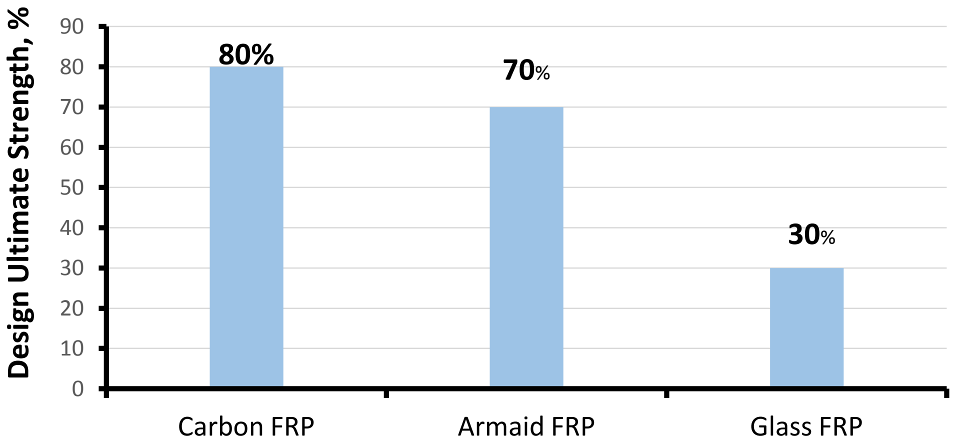

Figure 3.2.10 – TR55 Maximum stress range as a proportion of the design ultimate strength

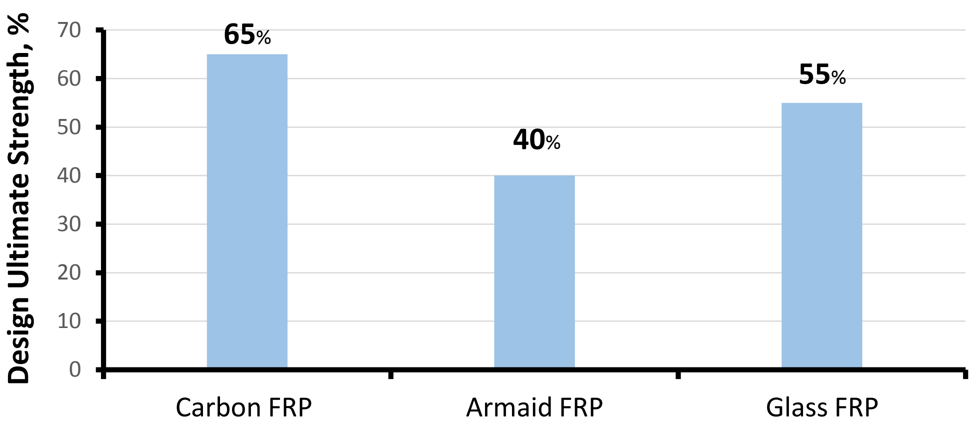

Figure 3.2.11 - TR55 Maximum stress range under service loads to avoid stress rupture

3.2.7.6 JSCE

According to JSCE, the design flexural fatigue resistance of members strengthened with FRP reinforcements shall be calculated considering the flexural fatigue characteristics of existing sections, the fatigue characteristics of the continuous fiber sheet and the characteristics of interfacial peeling fatigue failure between the continuous fiber sheet and the concrete. The code indicates that methods for accurate calculation of flexural capacity fatigue resistance have not yet been established. One recommended test method for performing tensile fatigue strength of continuous fiber sheets (JSCE-E 546, 2000).

To set a fatigue limit, JSCE uses its limit to avoid peeling failure using an interfacial fracture energy approach (Equation 3.2.32, JSCE Eq. 6.4.1). Equation 2.33 is modified to consider interfacial peeling fatigue failure for continuous fiber sheets and concrete substrate (Equation 3.2.34, JSCE Eq. C6.4.11). A reduction factor, μ , is introduced to limit the value of tensile fiber stress, σ_f, instead of being set for peeling failure limit to further accommodate fatigue loading.

$$\sigma_f \leq \sqrt{\frac{2G_f E_f}{n_f . t_f}} \qquad \text{(3.2.33 – JSCE Eq. 6.4.1)}$$

where:

n_f = Number of plies of continuous fiber sheets
E_f = Modulus of elasticity for continuous fiber sheet (*N/mm*^{2})
t_f = Thickness of one layer of continuous fiber sheet (*mm*)
G_f = Interfacial fracture energy between continuous fiber sheet and concrete (*N/mm*)

$$\sigma_f \leq \sqrt{\frac{2\mu G_f E_f}{n_f . t_f}} \qquad \text{(3.2.34 – JSCE Eq. C6.4.11)}$$

where:

μ = Reduction factor resulting from the influence of fatigue load on the interfacial fracture energy relating to the bond of continuous fiber sheets to concrete. In general, this value may be set to 0.7.

3.2.7.7. Summary

The factors for creep rupture and fatigue effects vary significantly. CNR and TR55 set fixed values for creep rupture and fatigue limits, while ACI and ISIS express this limit as a function of FRP strength. AASHTO provides the most elaborate checks for creep rupture and fatigue, providing strain (or stress) limits for concrete, steel and FRP. These limits are presented in Tables 3.2.19. Refer to Table 3.2.12 for recommended reduction factors for steel, concrete and FRP.

Figure 3.2.12 summarizes reduction factors for carbon, aramid, and glass fiber to guard against creep rupture and fatigue. In general, ACI, AASHTO, and ISIS factors are similar, while ACI and AASHTO generally provide the most conservative factors.

Table 3.2.19 - FRP limits due to creep rupture and cyclic loading

Code	**Formula/Description**	**Limit**
AASHTO	$\varepsilon_c = \dfrac{M_f z}{I_T\, E_{frp}}$	$\leq 0.36 \dfrac{f_c^{'}}{E_c}$
	$\varepsilon_s = \dfrac{M_f (d - z)}{I_T\, E_{frp}}$	$\leq 0.8\varepsilon_y$
	$\varepsilon_{frp} = \dfrac{M_f (h + t_{frp} - z)}{I_T\, E_{frp}}$	$\leq \eta \varepsilon_{frp}^{u}$ Note: η for CFRP is taken as 0.80
TR55	Stress range as a proportion of the design ultimate strength	80%
ACI	Sustained plus cyclic stress limit	0.55 f_{fu}
ISIS	Maximum stress level for creep	0.65 f_{FRP}^{u}
CNR	Partial factor for creep rupture & fatigue	0.8 for creep and relaxation 0.5 for fatigue
JSCE	σ_f	$\sigma_f \leq \sqrt{\dfrac{2\mu G_f E_f}{n_f . t_f}}$

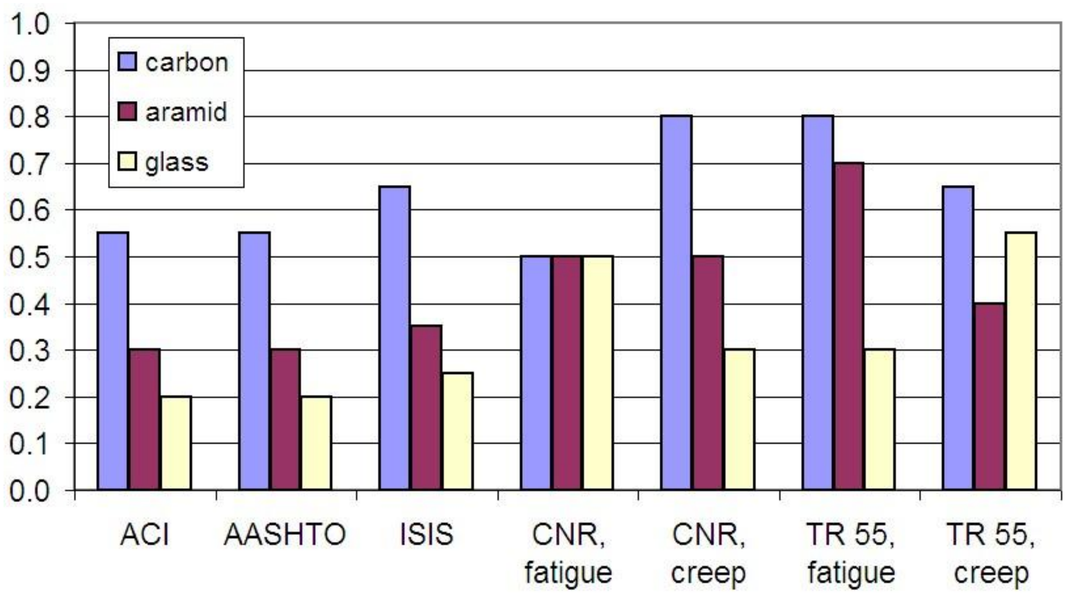

Figure 3.2.12 – Summary of FRP fatigue/creep rupture coefficients

3.2.8 End Peeling

3.2.8.1 ACI

An end peeling failure frequently occurs due to a high stress at the FRP termination point causing a splitting away of the concrete cover at plane of the steel reinforcement. In ACI, to prevent this failure, the FRP reinforcement requires anchorage if the factored shear V_u at the termination point is greater than 2/3 of the concrete shear strength V_c, or:

$$V_u > 0.67\, V_c \qquad (3.2.35)$$

where:

$$V_c = 2\,\lambda\,\sqrt{f_c'}\, b_w\, .d \qquad \text{(3.2.36 - ACI 318, equation 11-3)}$$

The anchoring is generally done using a transverse FRP U-wrap with an area A_f.

3.2.8.2 AASHTO

The peel stress at the point of end-termination of externally bonded reinforcement is required to meet following limit:

$$f_{peel} \leq 0.065\sqrt{f_c'} \qquad (3.2.37)$$

In which:

$$f_{peel} = \tau_{av}\left[\left(\frac{3E_a}{E_{frp}}\right)\frac{t_{frp}}{t_a}\right]^{1/4} \qquad (3.2.38)$$

where:

$$E_a = 2G_a(1 + \upsilon_a) \qquad (3.2.39)$$

υ_a = Poisson’s ratio of the adhesive, and is taken as 0.35.

$$\tau_{av} = \left[V_u + \left(\frac{G_a}{E_{frp}t_{frp}t_a}\right)^{1/2} M_u\right]\frac{t_{frp}(h-y)}{I_T} \qquad (3.2.40)$$

τ_{av} = The characteristic value of the limiting shear stress in the adhesive (ksi). In the absence of experimental data, a value of 5.0 can be used.

AASHTO discusses three possible modes of debonding at the termination point of an externally bonded FRP reinforcing system when the structure is subjected to shear and flexure. One possibility is critical diagonal crack debonding with or without concrete cover separation and plate end interfacial debonding. In this case, if the FRP termination point is in a zone of high shear and the amount of steel reinforcement is insufficient, critical diagonal cracking may occur.

In such cases, a critical diagonal shear crack forms and intersects the FRP, then propagates toward the end of the member. A second possibility occurs in beams with higher amounts of existing steel shear reinforcement, instead of a single critical diagonal crack, multiple diagonal cracks of smaller width may occur. In this case, concrete cover separation is generally the controlling debonding failure mode. Here, failure of the concrete cover is initiated by a crack near the FRP termination point. The crack then propagates to and then along the level of the steel tension reinforcement. This mode of failure has been demonstrated experimentally for beams with externally bonded steel plates and FRP reinforcement, and is the mode of failure discussed in section 3.2.8.1 above.

A third possibility occurs when high interfacial shear and normal stresses near the end of the FRP exceed the strength of the weakest element, generally the concrete, and plate-end interfacial debonding is initiated. Debonding in this case propagates from the FRP termination point toward the middle of the structural member, near the FRP-concrete interface. Note that this failure mode is only likely to occur when the FRP is significantly narrower than the beam section. AASHTO does not quantify a threshold for a ratio at which debonding initiates and propagates.

Although a wide range of predictive models that include numerical, fracture mechanics, data-fitting, and strength of material-based methods have been developed to address end peeling failures (Yao, 2004), AASHTO recommends a simplified equation based on the approximate analysis of Roberts (1989). At present, AASHTO does not specify a standard test method for determining the peel strength of an FRP reinforcement system from the concrete surface. However, AASHTO recommends for this purpose the use of ASTM Standard Test Method D 3167, *Standard Test Method for Floating Roller Peel Resistance of Adhesives* (2010). The ASTM method can be used for determining the peel resistance of adhesive bonds between rigid and flexible surfaces adhered together. For cases in which the peeling occurs within the concrete layer, AASHTO recommends that the peeling strength be limited to $0.065\sqrt{f_c'}$ and if the peeling stress exceeds $0.065\sqrt{f_c'}$, mechanical anchors at the FRP termination point must be used.

3.2.8.3 TR55

TR55 refers to early research on FRP separation failure and suggests a number of possible approaches for combating this problem, including: the use of plate end anchorage devices, flexible adhesives, and imposing limits on the plate aspect (i.e. breadth/thickness) ratio. Bolted systems, bonded angle sections and composite straps bonded across the soffit plate are examples of plate end anchorage devices that have been proposed as possible methods of preventing FRP separation failure. Generally, TR55 states that end plate separation failure can be avoided by meeting two criteria: 1) limiting the longitudinal shear stress between the FRP and the substrate, and 2) anchoring the FRP by extending it beyond the point at which it is theoretically no longer required.

To meet the first criterion, TR55 notes that field experience indicates that limiting the longitudinal shear stress at the ultimate limit state to a value no greater than 0.8 N/mm^2, premature peeling failure can be avoided. TR55 also presents a procedure to calculate the minimum anchorage length as it relates to the maximum ultimate bond force. The longitudinal shear stress, τ, can be calculated using the following expression (Eq. 3.2.41):

$\tau = V\alpha_f \alpha A_f(h\text{-}x) \, / \, I_{cs} b_a$ (3.2.41)

The relationship between the bond force, T_k, and the corresponding anchorage length, l_t is presented in Figure 3.2.13. It is further recommended that a minimum anchorage length of 500 mm should be provided. In situations where it is not possible to provide the maximum allowable anchorage length, the bond force should be less than the following:

$$T_k = \left(\frac{T_{k,max}\, l_t}{l_{t,max}}\right)\left[\frac{2 - l_t}{l_{t,max}}\right] \tag{3.2.42}$$

Additionally, TR55 considers using an anchorage device, provided its capacity has been verified by testing.

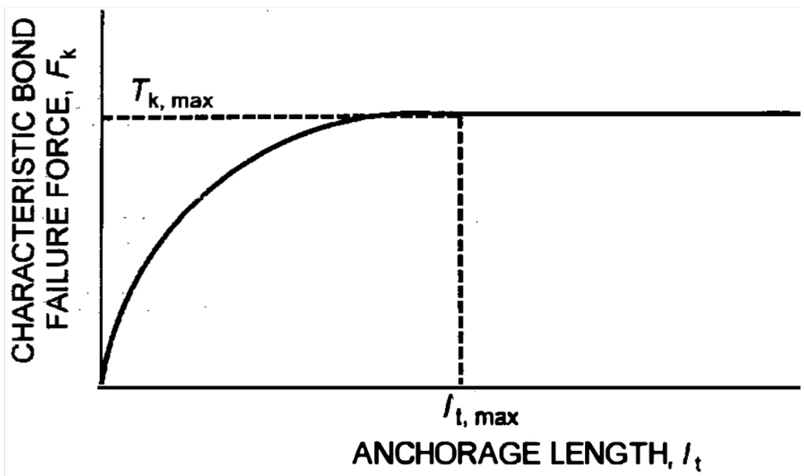

Figure 3.2.13 – Characteristic bond failure force vs. anchorage length (TR55)

3.2.8.4 JSCE

JSCE does not clearly specify design equations other than the following limiting peeling stress expression:

$$\sigma_f \leq \sqrt{\frac{2G_f E_f}{n_f . t_f}} \tag{3.2.43}$$

JSCE notes how the interfacial fracture energy factor G_f varies with the strengthening system, the number of plies, and the anchoring system used, and states that this value should be determined through testing. When testing cannot be performed, JSCE recommends using a G_f value of 4 lb/in (0.7 N/mm).

3.2.8.5 CNR

The following equation is used to limit the peeling stress in CNR:

$$\tau_{b,e} \leq f_{bd} \tag{3.2.44}$$

Per Eq. 3.2.44, to prevent peeling failure, the equivalent shear stress $\tau_{b,e}$ should be less than the design bond strength (f_{bd}). If the shear stress is higher, an anchorage device must be used. The equivalent shear stress is given by Eq. 3.2.45:

$$\tau_{b,e} = k_{id} \, . \, \tau_m \tag{3.2.45}$$

where:

k_{id} = A coefficient (≥1) accounting for shear and normal stresses close to the anchorage ends, assumed to be 1.0.

$$\tau_m = \frac{V_{(z=a)} . t_f . (h - x_e)}{I_c / n_f} \tag{3.2.46}$$

τ_m = average shear stress
$V_{(z=a)}$ = shear force acting on the section where FRP strengthening ends
t_f = fiber thickness
n_f = modular ratio, E_f / E_c E_c
x_e = distance from the extreme compression fiber to the neutral axis
I_c = moment of inertia of the transformed section

$$f_{bd} = k_b \, . \frac{f_{ctk}}{\gamma_b} \tag{3.2.47}$$

f_{bd} = design bond strength, a function of the characteristic tensile strength of the concrete, f_{ctk}

γ_b = 1.0 for rare loading combinations
= 1.2 for frequent loading combinations

The peeling stress is then given as:

$$f_{peel} = \tau_{av} \left[\left(\frac{3E_a}{E_{frp}} \right) \frac{t_{frp}}{t_a} \right]^{1/4} \tag{3.2.48}$$

where:

$$\tau_{av} = \left[V_u + \left(\frac{G_a}{E_{frp} t_{frp} t_a} \right)^{1/2} M_u \right] \frac{t_{frp}(h-y)}{I_T} \tag{3.2.49}$$

References are not provided for the basis of the safety factors γ_b. Environmental factors are not accounted for. A value of 116 psi (0.8 *N/mm*^{2}) is recommended for use as an upper limit to avoid premature peeling failure.

3.2.8.6 Anchorage methods for FRP

For cases of high peeling or shear stress, an anchorage system may be required to avoid debonding failure. In these cases, an anchorage system might allow the use of a FRP strengthening plan that otherwise would not meet design code provisions. For example, anchorage may allow greater strengthening or the use of a wider range of possible FRP geometries and material properties. A drawback of the use of many anchorage systems is the added cost and complexity of installation.

NCHRP 678 (Belarbi et al. 2011) describes several anchorage systems, including the near-surface mounted system (NSM), where the end of the FRP wrap or pre-formed plate is bent and embedded into a groove cut into the concrete, then secured with epoxy. Another system available involves anchoring the FRP to the concrete with a spike made from a bundle of fibers. In this process, half of the spike length is covered with resin and allowed to harden. The pre-cured spike end is then inserted into a resin-filled hole that is drilled into the concrete, and the dry fibers on the other end of the spike are spread apart on the surface of the FRP layer to be anchored, and then saturated with resin. Yet other types of anchorage systems involve embedding nails or rods into the concrete to secure steel plates on top of the FRP layer to be anchored.

ISIS also describes several common anchorage systems, including bonding an additional FRP strip along the edges of the applied FRP; anchoring FRP shear stirrups into the bottom of the slab in T-beam applications; and clamping the FRP strips with plates and bolts, as later illustrated in section 3.3.2.3.

3.2.8.7 Summary

A summary of peeling limits and procedures are given in Table 3.2.20.

Table 3.2.20- Peeling summary

Code	Quantity	Formula	Limit
ACI 440.2R	V_u	$A_{f_{anchor}} = \frac{(A_f f_{fu})_{longitudinal}}{(E_f \kappa_v \varepsilon_{fu})_{anchor}}$	$V_u \geq 0.67\, V_c$
AASHTO	f_{peel}	$f_{peel} = \tau_{av}\left[\left(\frac{3E_a}{E_{frp}}\right)\frac{t_{frp}}{t_a}\right]^{1/4}$	$f_{peel} \leq 0.065\sqrt{f_c'}$
CNR-DT 200	Bond shear strength	$\tau_{b,e} \leq f_{bd}$	$f_{bd} = k_b \,.\frac{f_{ctk}}{\gamma_b}$
TR55	Shear stress	$V_u \leq 116\, psi\ (0.8\, N/mm^2)$	$116\, psi\ (0.8\, N/mm^2)$
JSCE	Shear stress	$\Delta\sigma_f = \sqrt{\frac{2G_f E_f}{n_f . t_f}}$	$\sqrt{\frac{2G_f E_f}{n_f . t_f}}$

As shown in the table, different methods are used to evaluate peeling for the various codes. ACI uses a peeling limit evaluating the shear capacity of concrete at the FRP termination point, expressed as a function of $\sqrt{f_c'}$ and section shear capacity and demand ($V_u \geq 0.67\,V_c$). AASHTO and CNR limits are similar, and are a function of f_c' and $\sqrt{f_c'}$ respectively, while JSCE expresses the limit in terms of FRP related quantities. To compare code expressions, a $0.8\,N/mm^2$ rectangular concrete beam of h=30 in and b= 18 in is considered as an example. The beam is assumed to have the following properties: factored shear force at the reinforcement end termination = 100 kips; factored moment at the reinforcement termination = 500 kip-in; materials properties of Mbrace Saturant and Mbrace CF130 wrap are used, such that fiber thickness (one ply) = 0.0065 inch; fiber modulus of elasticity = 33000 ksi; adhesive thickness = 0.0022 inch; adhesive modulus of elasticity = 440 ksi; Poisson ratio of adhesive = 0.40; $\gamma_b = 1.2$ (assumes a frequent load combination (for CNR)). Peeling stresses and limits are evaluated as a function of the amount of FRP strengthening (Figure 3.2.14), and concrete compressive strength f_c' (Figure 3.2.15).

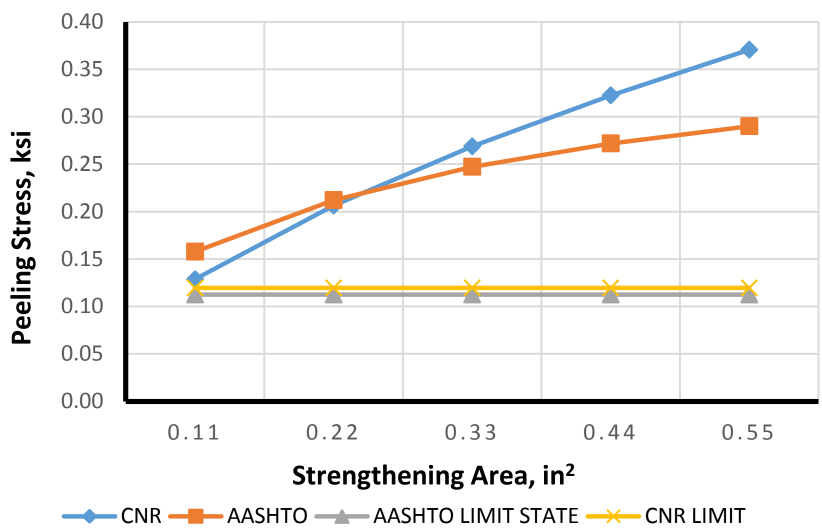

Figure 3.2.14 – Peeling stress vs. FRP strengthening area

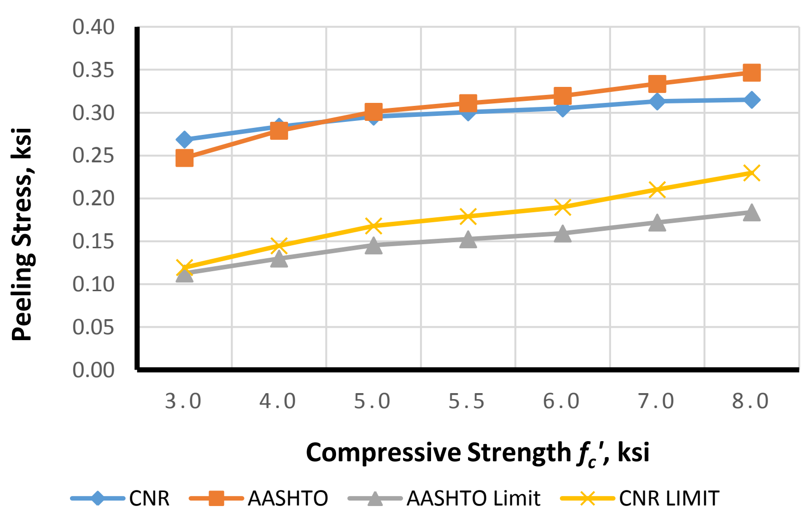

Figure 3.2.15 – Peeling Stress vs f'c

Two notable observations are that; 1) for the example beam, the peeling stress exceeds the peeling limit in all cases, and end anchorage is required; and 2) peeling stresses significantly increase as f_c' increases, and marginally increases as the amount of FRP strengthening increases. However, peeling limits are independent of the amount of FRP strengthening, but vary with the increase in f_c'. AASHTO and CNR are similar in the treatment of peeling stresses and peeling limits, although AASHTO peeling limits are slightly more conservative than those of CNR.

3.2.9 Development Length

3.2.9.1 AASHTO

In AASHTO, the tension development length, L_d, is taken as:

$$L_d \geq \frac{T_{frp}}{\tau_{int} b_{frp}} \tag{3.2.50}$$

where:

T_{frp} = tensile force in the FRP reinforcement corresponding to an FRP strain of 0.005
τ_{int} = interface shear transfer = 0.065 $\sqrt{f_c'}$

The specified development length is required to allow the full tension strength of the FRP to be developed in the region of maximum moment.

As will be shown below, the AASHTO development length calculations produce more conservative values as FRP area increases. It was also found that AASHTO development length values are most conservative at lower values of f_c', producing values as well as trends that are significantly different from the other codes studied (see Figures 3.2.16 and 3.2.17). Other code results appear to be independent of the changes in FRP area as well as f_c' (ISIS and TR55) or slightly dependent on these factors (ACI and CNR). To understand the reasons for these discrepancies, the basis of AASHTO's development length methodology is reviewed.

The interface shear transfer strength (τ_{int}) given in AASHTO is based on the recommendation of Naaman and Lopez (1999) and Naaman et al. (1999), who conducted tests on uncracked and precracked reinforced concrete beams externally bonded with FRP reinforcement and subjected to accelerated freeze-thaw cycles. This shear limit represents a lower bound of the experimental data found from short-term direct tension tests of FRP reinforcement bonded to concrete surfaces (Haynes, 1997; Binzindavji and Neale, 1999). This research was conducted for MDOT and is detailed in Report RC-1372, "Repair and strengthening of reinforced concrete beams using CFRP Laminates, Behavior of beams subjected to freeze-thaw cycles".

In the experimental program, reinforced concrete beams were subjected to a maximum of 300 freeze-thaw cycles according to ASTM C666. The parameters investigated were two different adhesive systems, the Tonen CFRP sheet system (MBrace), and the Sika CFRP system (Carbodur); as well as the degree of cracking prior to strengthening. It was found that freeze-thaw cycles influenced the behavior of reinforced concrete beams with glued-on Carbon Fiber

Reinforced Plastic (CFRP) laminates, and the recommended value of interface shear transfer strength (τ_{int}) represents the lower bound of the test results.

3.2.9.2 ACI

In ACI, the bond capacity of FRP is developed over a critical length l_{df}. To develop the effective FRP stress at a given section, the available anchorage length of FRP should exceed the value given in the equation below:

$$l_{df} = 0.057\sqrt{\frac{nE_f t_f}{\sqrt{f_c^{'}}}} \quad \text{in-lb units} \qquad (3.2.50)$$

$$l_{df} = \sqrt{\frac{nE_f t_f}{\sqrt{f_c^{'}}}} \quad \text{SI units} \qquad (3.2.50\ )$$

3.2.9.3 ISIS

ISIS specifies the minimum required anchorage length for the externally-bonded FRP beyond the point where no strengthening is required, l_a. According to the S6-06 bridge code, l_a may be evaluated as follows:

$$l_a = 0.5\sqrt{\mathrm{E_{FRP}t_{FRP}}} \geq 11.81 \text{ in } (300\text{mm}) \qquad (3.2.51 \text{ - ISIS Eq. } 5.29)$$

Anchorage lengths longer than l_a must be provided if required by the manufacturer's installation procedure. For cases where l_a is not provided, suitable anchorage mechanisms must be used.

3.2.9.4 – CNR

The optimal bonded length l_e may be estimated as follows:

$$l_e = \sqrt{\frac{E_f t_f}{2\sqrt{f_{ctm}}}} \qquad (3.2.52 \text{ - CNR Eq. } 4.1)$$

where:

l_e is in mm; the value from Eq. 3.2.53 should be multiplied by 0.03937 to obtain l_e in inches.

3.2.9.5 TR55

TR55 specifies a threshold anchorage length, $l_{t,max}$, above which no increase in the bond failure force is possible. The maximum anchorage length, $l_{t,max}$, needed to activate this bond force iscalculated using the following expressions:

$$l_{t,max} = 0.7\sqrt{E_{fd}t_f/f_{ctm}} \qquad \geq 19.69\ in\ (500\ mm) \qquad \text{(3.2.53 - TR55 Eq. 6.19)}$$

where:

$$f_{ctm} = 0.18(f_{cu})^{2/3} \qquad \text{(3.2.54 -TR55 Eq. 6.22)}$$

3.2.9.6 Summary

Generally, all expressions (with the exception of ISIS), express development length as a function of tension in the FRP and concrete compressive strength f_c'. The ISIS formula f_c'sets a minimum limit of 11.81 in (300 mm), while TR55 sets the development length minimum value at 19.69 in (500 mm). A comparative evaluation of development length is given in Figures 3.2.16 and 3.2.17 below (note that to evaluate CNR, f_{ctm} is assumed to be 0.3 $(f_c')^{2/3}$ in accordance with the Eurocode, since CNR does not provide an expression for the value). Two scenarios are considered; the first (Figure 3.2.16) varies f_c' , while the second (Figure 3.2.17) varies FRP area from 1 to 5 plies of BASF MBrace CF130 CFRP wrap (see article 3.2.8.7 in this document for example beam properties and other relevant data).

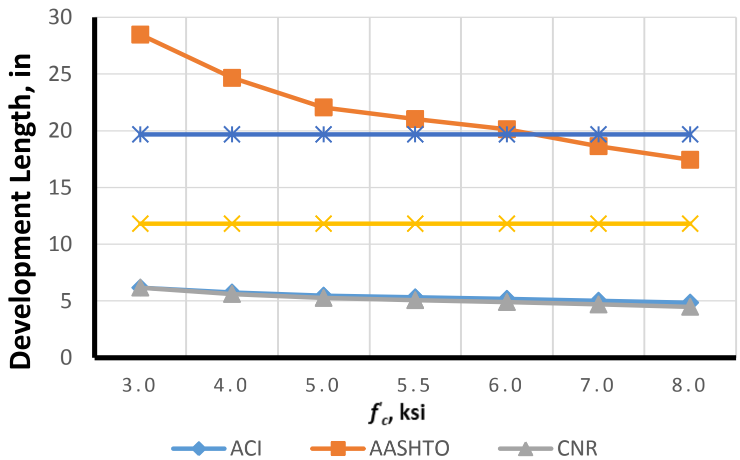

Figure 3.2.16 – Development length as a function of f_c', ksi

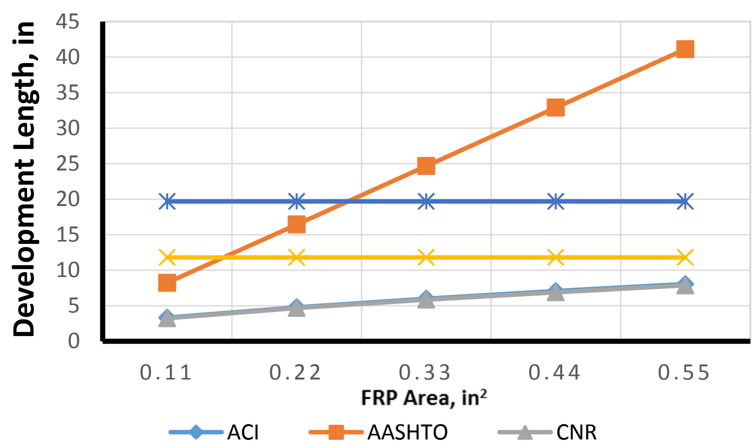

Figure 3.2.17 – Development length as a function of FRP area, in^2

It can be seen from the Figures that a significant variation in development length requirements exist. For ISIS and TR55, the lower limits specified generally controlled. For AASHTO, development length is independent of FRP modulus, while all other codes have expressions for development length as a function of FRP modulus. The AASHTO expression is also significantly more sensitive to FRP area. As discussed above, AASHTO recommendations differ significantly from the other codes and generally provide conservative results. However, the values considered by AASHTO were specifically based on testing conducted for MDOT at the University of Michigan that included freeze-thaw conditioning relevant to Michigan.

3.2.10 Flexural Design Approach and Assumptions

3.2.10.1 AASHTO

In AASHTO, the calculation of the flexural strength of reinforced concrete members externally reinforced with FRP materials assumes perfect bond between the reinforcing steel, FRP reinforcement and the concrete; that the contribution of tension stress in the concrete to flexural strength is neglected; the stress-strain behavior for FRP reinforcement is linear-elastic until 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; and the maximum usable strain at the FRP/concrete interface is 0.005.

When concrete compressive strain is less than 0.003, the concrete compression stress distribution is to be modeled with a parabolic shape according to the following equation:

$$f_c = \frac{2(0.9f'_c)(\varepsilon_c/\varepsilon_0)}{1+(\varepsilon_c/\varepsilon_0)^2} \qquad (3.2.56)$$

where:

$$\varepsilon_0 = 1.71\frac{f'_c}{E_c} \qquad (3.2.57)$$

ε_0 is the concrete strain corresponding to the maximum stress on the concrete stress-strain curve.

The factored resistance, Mr, of a steel reinforced concrete rectangular section strengthened with FRP externally bonded to the beam tension surface is be taken as:

$$Mr = 0.9\left[A_s f_s(d_s - k_{2c}) + A'_s f'_s(k_{2c} - d'_s)\right] + \phi_{FRP} T_{FRP}(h - k_{2c}) \qquad (3.2.58)$$

where:

ϕ_{FRP} = resistance factor = 0.85
T_{FRP} $= n\, b_{FRP} N_b$ (3.2.59)
n = number of FRP reinforcement plates

N_b = FRP reinforcement strength per unit width, corresponding to 0.5% strain in the FRP reinforcement when subjected to tension in accordance with ASTM D3039; assumed to be 1.07.

3.2.10.2 JSCE

JSCE considers carbon and aramid fibers. It employs 5 partial safety factors: material, load, member, structure, and analysis. However, JSCE does not include explicit design equations for flexure. Due to the lack of a clear procedural description of flexural capacity evaluation, JSCE was not further considered for moment capacity analysis and comparison.

3.2.10.3 CNR

Similar to JSCE, CNR also provides no explicit capacity expression, but the flexural failure mode and capacity can be determined from strain compatibility, section equilibrium, and the required material strength limits. In CNR, the concrete stress block is not specified, so the FIB 14 procedure (2001) is used, where the coefficient representing the resultant of the compressive stress can be expressed as follows:

ψ = 0.8 for $f_c^{'}$ < 7.3 ksi (50 MPa)
= 0.8 - ($f_c^{'}$-50)/400 for $f_c^{'}$ > 7.3 ksi (eq. in MPa)
λ = coefficient representing the extreme compression fiber
= 0.40 (from FIB 14, as no value is given in CNR)
K_b = geometric coefficient, ≥ 1.0

Finally, the maximum strain in FRP is to be calculated as follows:

$$\varepsilon_{fd}\varepsilon_{fd} = min\left\{\eta_a.\frac{\varepsilon_{fk}}{\gamma_f}, \varepsilon_{fdd}\right\} \tag{3.2.12}$$

where:

ε_{cu} = Ultimate strain of concrete in compression, set to 0.0035

For comparison to other codes (as applicable in Figures 3.2.18-3.2.35), the following assumptions are used in CNR flexural calculations: steel yield strength is reduced by the specified partial safety factor of γ_s = 1.15 such that $f_{yd} = f_y/\gamma_s$; the mean value of concrete tensile strength $f_{ctm} = 0.3\left(\sqrt{f_c^{'}}\right)^{2/3}$ (not explicitly defined in CNR, but taken from Eurocode; concrete compressive strength is taken as $f_{cd} = 0.85\,\frac{f_c^{'}}{\gamma_c}$, with the partial safety factor taken as $\gamma_s = f_{ctm} = \left(\sqrt{f_c^{'}}\right)^{2/3} f_{cd} = \frac{f_c^{'}}{\gamma_c}\gamma_c =$ 1.6.

3.2.10.4 ACI

ACI assumes that the maximum concrete compressive strain is 0.003. The ψ FRP strain limit is imposed using equation 3.2.8 as shown below:

$$\varepsilon_{fd} = 0.083\sqrt{\frac{f'_c}{nE_f t_f}} \leq 0.9\varepsilon_{fu} \quad in\ in.-lb\ units \qquad (3.2.8 – ACI Eq. 10-2)$$

$$\varepsilon_{fd} = 0.41\sqrt{\frac{f'_c}{nE_f t_f}} \leq 0.9\varepsilon_{fu} \quad in\ SI\ units \qquad (3.2.8 m)$$

Equation 3.2.60 (ACI Eq. 10-13) is typically used to evaluate the nominal moment capacity of the section:

$$M_n = A_s f_s \left[d - \left(\frac{\beta_{1c}}{2}\right)\right] + \phi A_f f_{fe} \left[h - \left(\frac{\beta_{1c}}{2}\right)\right] \qquad (3.2.60 - ACI Eq. 10-13)$$

For the special case when ε_c is smaller than 0.003, values for $\beta_1$and α_1 are evaluated as follows:

$$\beta_1 = \frac{4\varepsilon_c' - \varepsilon_c}{6\varepsilon_c' - 2\varepsilon_c} \qquad (3.2.61)$$

$$\alpha_1 = \frac{3\varepsilon_c'\varepsilon_c - \varepsilon_c^2}{3\beta_1 {\varepsilon_c'}^2} \qquad (3.2.62)$$

Using $\beta_1$and α_1, the neutral axis location can be evaluated from equation 3.2.63.

$$c = \frac{A_s \cdot f_y + A_f \cdot f_{fe}}{\alpha_1 \cdot f_c' \cdot \beta_1 \cdot b} \qquad (3.2.63)$$

Steel and FRP service stresses are calculated using equations 3.2.19 and 3.2.20 to ensure compliance with serviceability requirements.

$$f_{s,s} = \frac{\left[M_s + \varepsilon_{bi} A_f E_f \left(d_f - \left(\frac{k_d}{3}\right)\right)\right](d - k_d)E_s}{A_s E_s \left(d - \left(\frac{k_d}{3}\right)\right)(d - k_d) + A_f E_f \left(d_f - \left(\frac{k_d}{3}\right)\right)(d_f - k_d)} \qquad (3.2.19 - ACI Eq. 10-14)$$

$$f_{f,s} = f_{s,s}\left(\frac{E_f}{E_s}\right)\frac{(d_f - k_d)}{(d - k_d)}\varepsilon_{bi} E_f \qquad (3.2.20 - ACI Eq. 10-15)$$

An environmental reduction factor C_E (taken as 0.85 assuming exterior exposure) and the additional reduction factor ψ specific to FRP are considered in the factored moment calculations.

3.2.10.5 ISIS

The ISIS design procedure is straightforward with the assumptions given in the code. The neutral axis is determined using strain compatibility and section balance. In ISIS, it is assumed that concrete maximum compressive strain is ≤ 0.0035; the maximum FRP strain (per the bridge code) = 0.006; and, the appropriate reduction factors are used as specified earlier in this document.

3.2.10.6 TR55

TR55 is similar to the other codes in its assumptions of the applicability of basic engineering mechanics to establish section capacity. Specific to TR55 are the following recommendations for flexural capacity: the FRP strain limit to prevent debonding is taken as 0.008 for uniformly distributed loads and 0.006 for load effects of simultaneous high shear and moment; and the ultimate compressive strain in concrete $\varepsilon_{cu} = 0.0035$.

Unique to TR55 is that the neutral axis location (x) is calculated from the original section equilibrium and is not adjusted with the addition of FRP. An additional moment capacity, M_{add}, representing the FRP contribution, is added to the original section nominal moment capacity. Assuming a flexural section is to be strengthened to carry a larger moment M than its current capacity, with a singly-reinforced section, the nominal moment is evaluated as follows:

$$M_r = F_s z + F_f [z + (h - d)] \qquad (3.2.64 – UK Eq. 6.14)$$

where:

$M_r = M$ and $Fs = (f_y / \gamma_{ms}) \cdot A_s$

3.2.10.7 Summary

For comparison of different code results, a rectangular reinforced concrete beam with b = 18 in and h = 30 in is considered for strengthening with MBrace CF 130 CFRP. Figures 3.2.18 to 3.2.26 illustrate how moment capacity changes as a function of FRP area and initial steel reinforcement ratio. It was found that, as expected, an increase in moment capacity accompanies an increase in FRP area. An exception is observed for ACI with a steel ratio of 0.0171 (Figure 3.2.25 - factored moment case). This exception occurs because the steel reinforcement strain falls in the transition zone between a tension and compression controlled failure, causing a reduction in the value of ϕ and thus reducing the factored moment as FRP area increases. It was also found that as steel reinforcing ratio increases, a compression failure is observed in some codes but not in others. For example, AASHTO and CNR develop compression failures at a steel reinforcement ratio of 0.0171 for all values of FRP strengthening. It is important to note that the compression failures were obtained for the relatively low f'_c chosen for the evaluation beam of 3000 psi. This low value was specifically chosen such that the behavior of different code procedures considering different beam failure modes (i.e. tension and compression controlled) could be compared. In most cases, where f'_c is greater that 3000 psi, the likelihood of developing a compression failure is reduced for the same amount of FRP reinforcement.

It was also found that due to the need to maintain section equilibrium, when FRP area increases, FRP strain is reduced. However, when the code-specified FRP strain limits are imposed (for example, ISIS enforces a maximum FRP strain of 0.006), FRP rupture sometimes becomes the only possible mode of failure. Moreover, ACI exhibits a higher FRP strain at lower values for FRP area, relative to other codes. This can be attributed to one of the two ACI strain limit expressions, where the strain limit is inversely propotional to FRP area (Equation 3.2.8).

$$\varepsilon_{fd} = 0.083\sqrt{\frac{f'_c}{nE_f t_f}} \leq 0.9\varepsilon_{fu} \quad {\scriptstyle in\ in.-lb\ units} \qquad \text{(3.2.8 – ACI Eq. 10-2)}$$

$$\varepsilon_{fd} = 0.41\sqrt{\frac{f'_c}{nE_f t_f}} \leq 0.9\varepsilon_{fu} \quad {\scriptstyle in\ SI\ units} \qquad \text{(3.2.8 m)}$$

Although predicted FRP strains differ, AASHTO and ISIS, both bridge-specific codes, generally produce similar moment capacity values for the different cases investigated. ACI, AASHTO, and ISIS are also reasonably consistent with flexural capacity prediction. ACI has greatest capacity for lower FRP area and higher f'_c, while AASHTO and ISIS have greatest capacity for higher FRP areas. TR55 is more sensitive when there is a change in concrete compressive strength and is less affected by change in FRP area. It is the most conservative for almost all cases. Despite it largest FRP strain limit of 0.008, it produces the least capacity due to its conservative (non-adjustment of the neutral axis) flexural capacity calculation method. In most cases, FRP rupture was found to be the mode of failure.

Figures 3.2.27-3.2.35 illustrate the effect of changing f'_c on moment capacity, as a function of several other parameters. For this comparison, FRP area was fixed at 3 plies of 17 in wide of MBrace CF130 CFRP. As expected, higher FRP strains result in higher the moment capacities. It was also found that ACI results in higher values for FRP strain as well as factored and unfactored moments when compared to other codes. For lower steel reinforcement ratios (ρ = 0.0033 and 0.0064), tension failure is the dominant mode of failure, though the higher value for ρ (0.0171), when combined with the FRP, results in possible compression failures (for example, AASHTO and CNR transition from tension failure into compression failure at the 0.0171 steel reinforcement ratio).

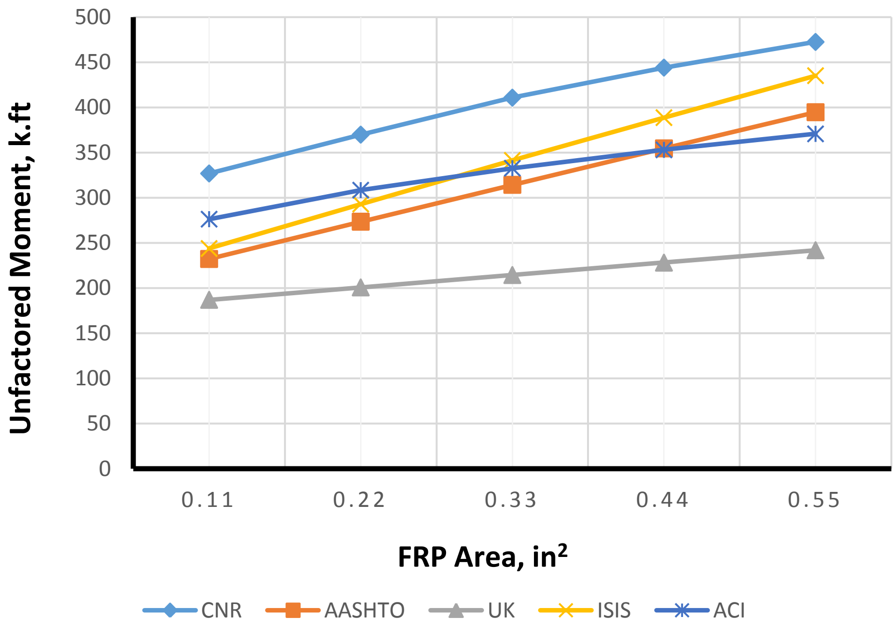

Figure 3.2.18 – Effect of amount of FRP flexural strengthening on unfactored moment ($\rho = 0.0033$)

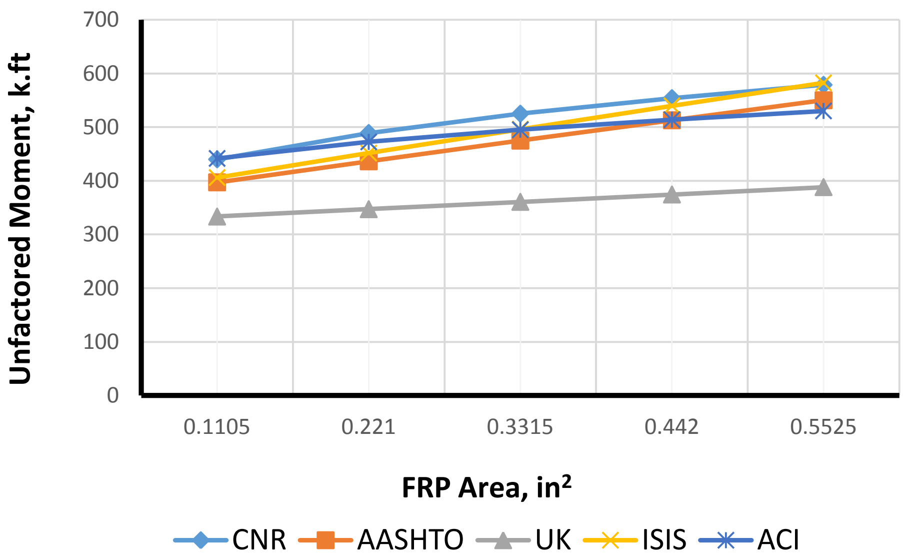

Figure 3.2.19 – Effect of amount of FRP flexural strengthening on unfactored moment ($\rho = 0.0064$)

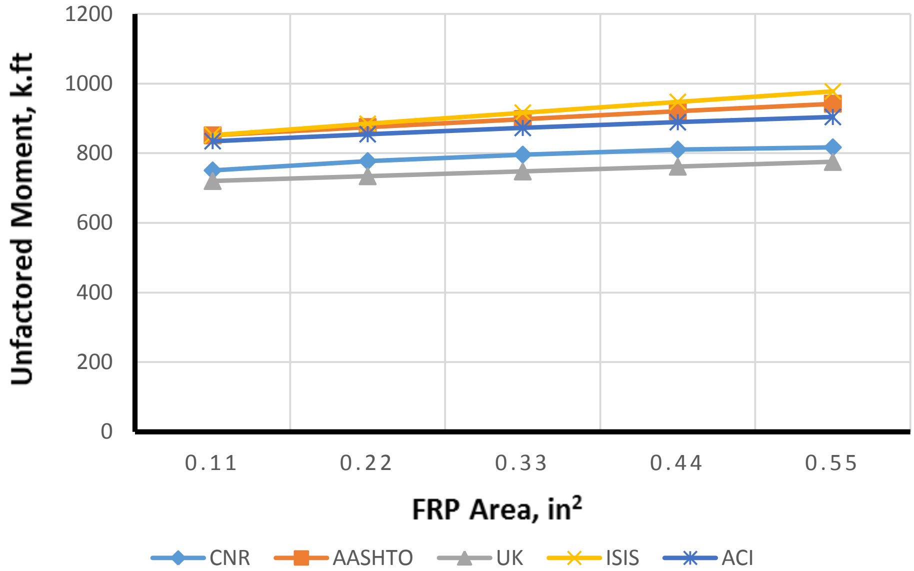

Figure 3.2.20 – Effect of amount of FRP flexural strengthening on unfactored moment (ρ = 0.0171)

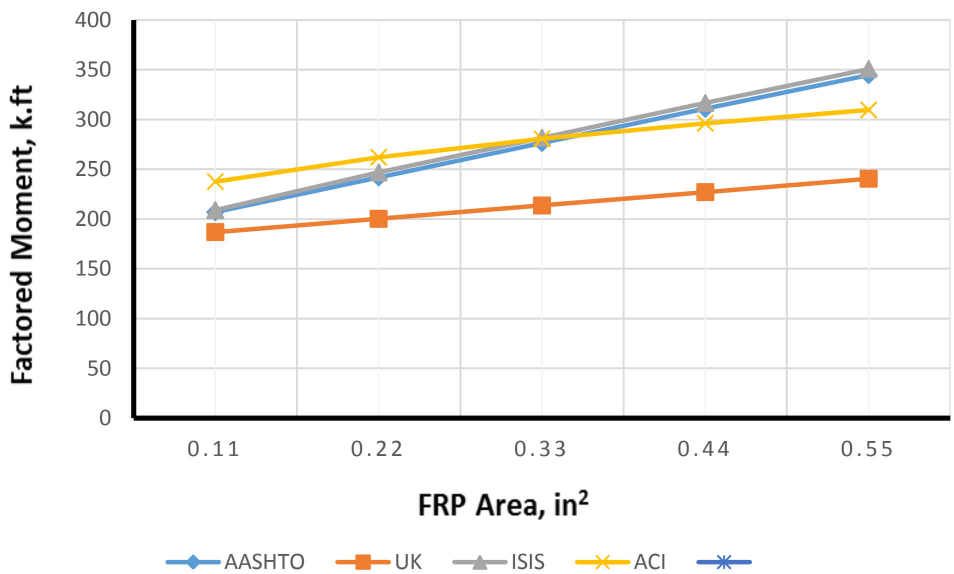

Figure 3.2.21 – Effect of amount of FRP flexural strengthening on factored moment (ρ = 0.0033)

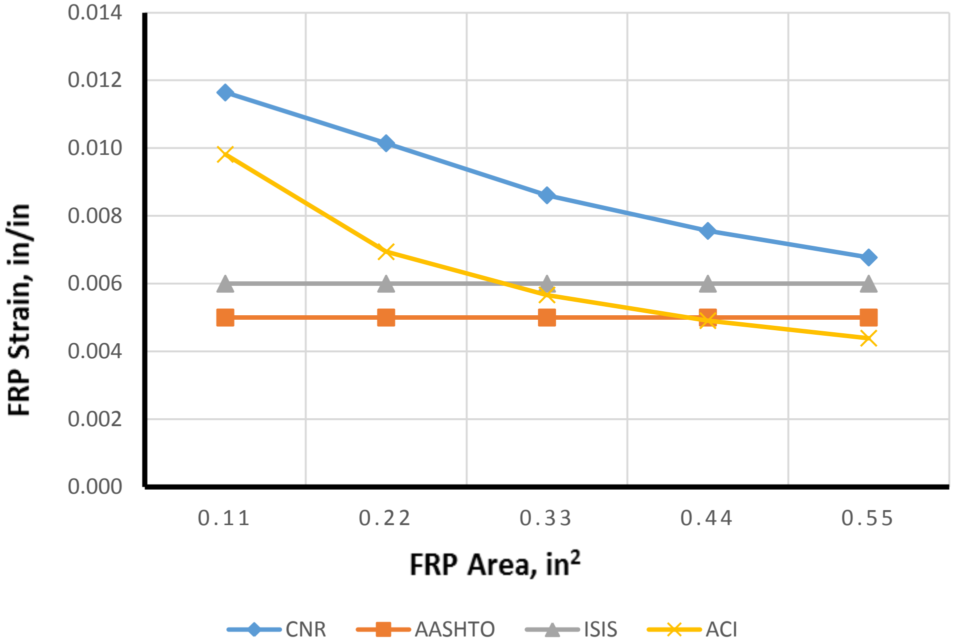

Figure 3.2.22 – Effect of amount of FRP flexural strengthening on FRP strain ($\rho = 0.0033$)

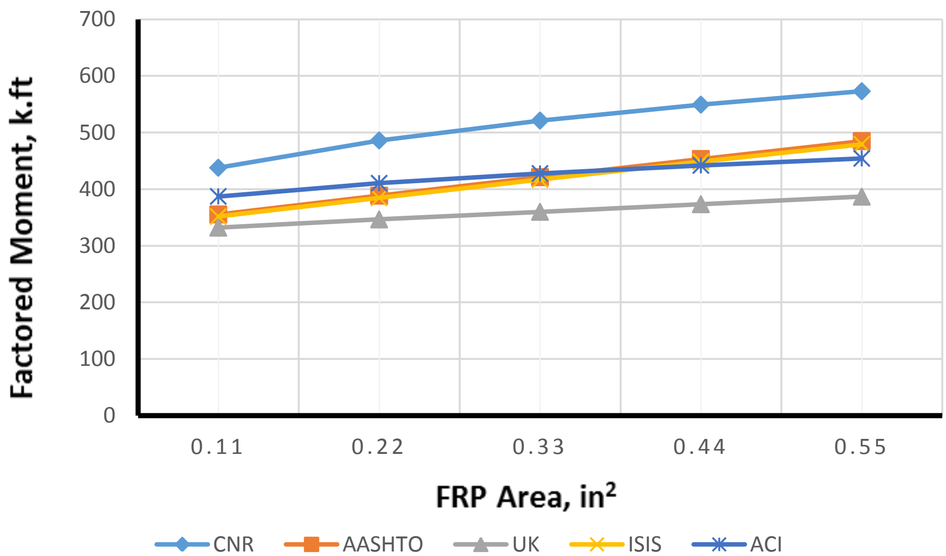

Figure 3.2.23 – Effect of amount of FRP flexural strengthening on factored moment ($\rho = 0.0064$)

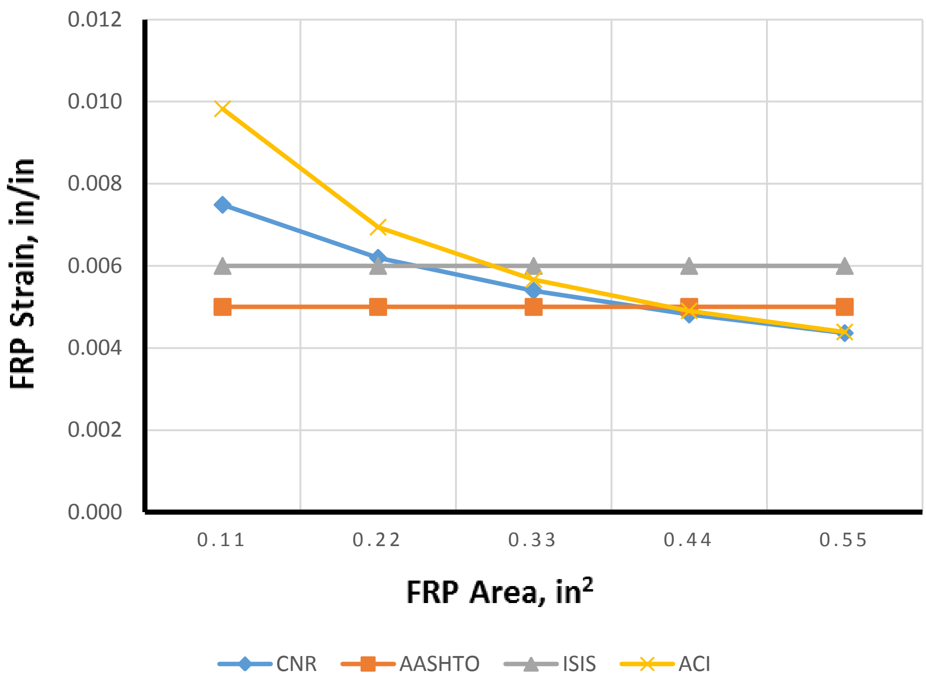

Figure 3.2.24 – Effect of amount of FRP flexural strengthening on FRP strain ($\rho = 0.0064$)

Note: As steel reinforcement ratio increases and FRP strain decreases, failure mode changes from compression to tension failure. Also, with increasing f'_c, the mode of failure changes from compression to tension failure.

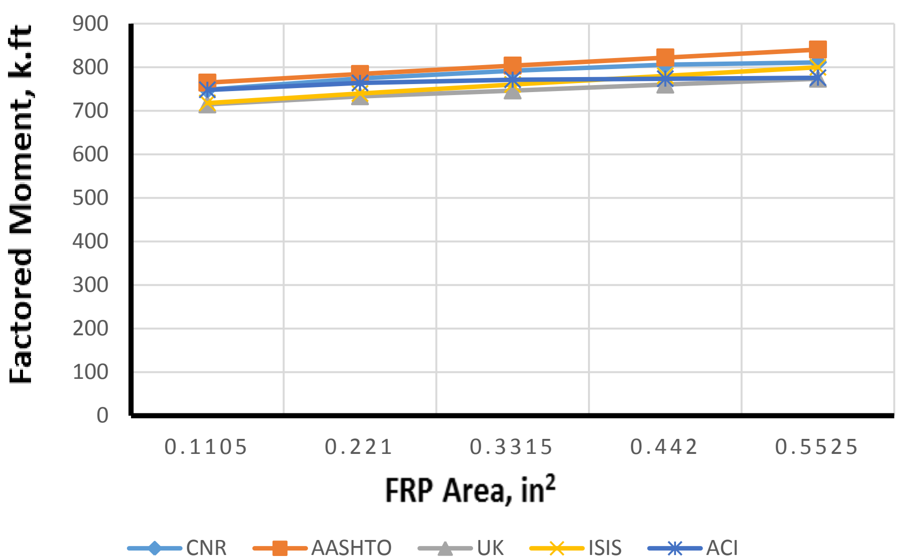

Figure 3.2.25 – Effect of amount of FRP flexural strengthening on factored moment ($\rho = 0.0171$)

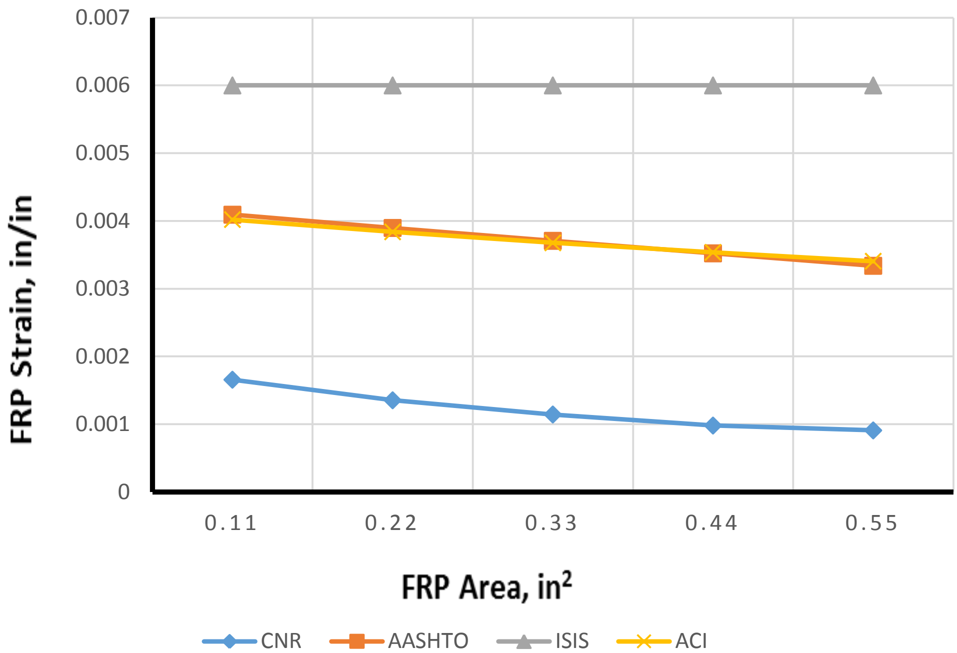

Figure 3.2.26 – Effect of amount of FRP flexural strengthening on FRP strain ($\rho = 0.0171$)

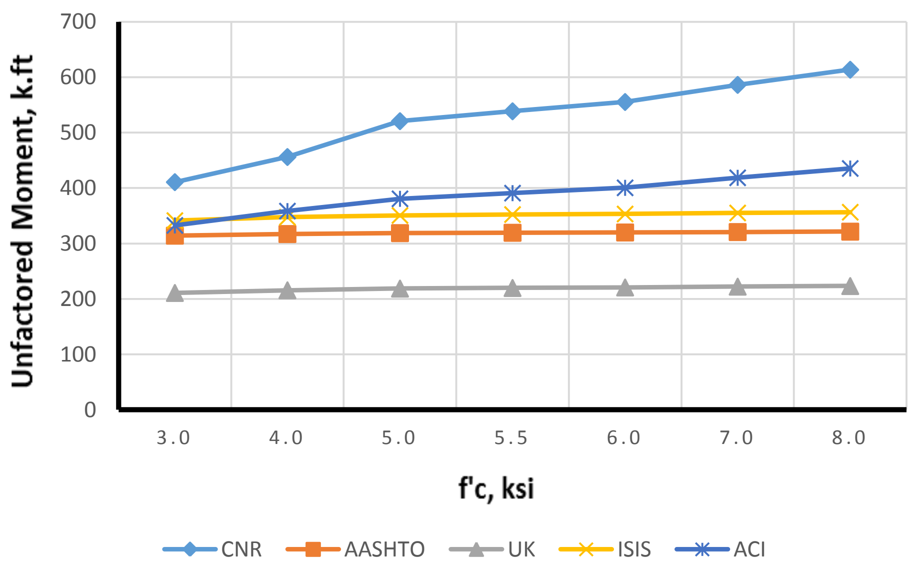

Figure 3.2.27 – Effect of f_c' *on unfactored moment* ($\rho = 0.0033$)

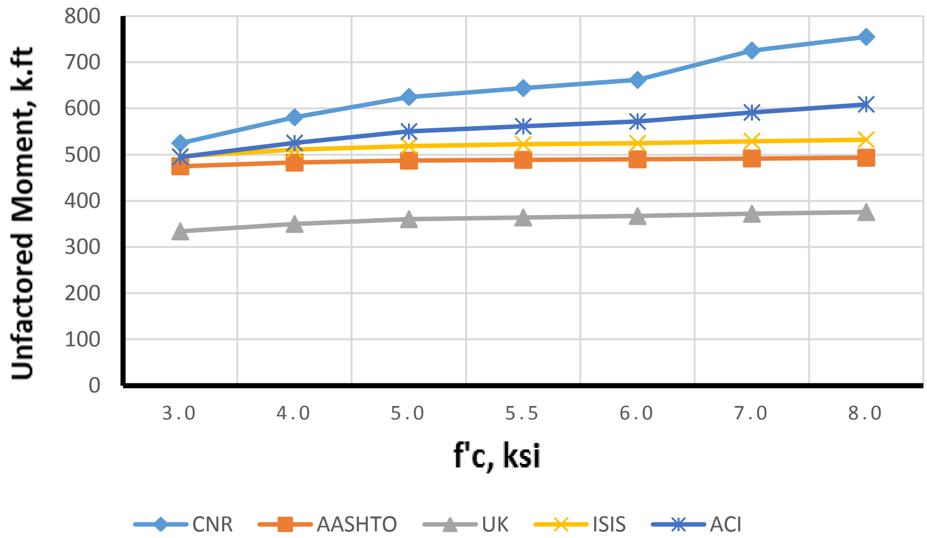

Figure 3.2.28 – Effect of f_c' *on unfactored moment* ($\rho = 0.0064$)

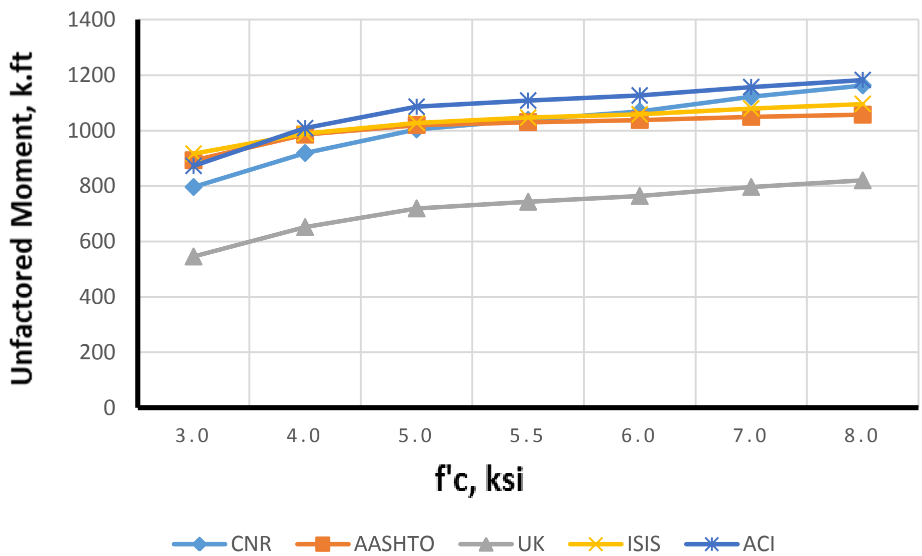

Figure 3.2.29 – Effect of f_c' *on unfactored moment* ($\rho = 0.0171$)

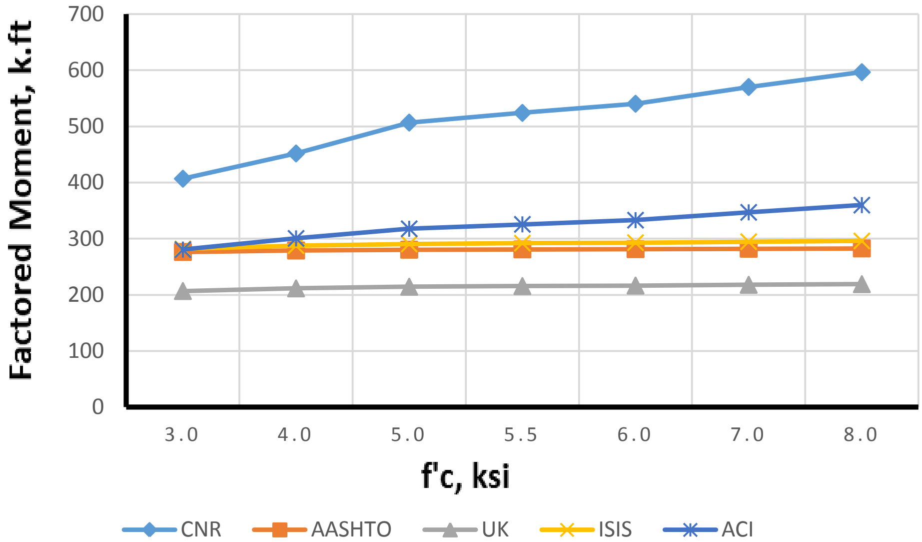

Figure 3.2.30 – Effect of f_c' *on factored moment* $(\rho = 0.0033)$

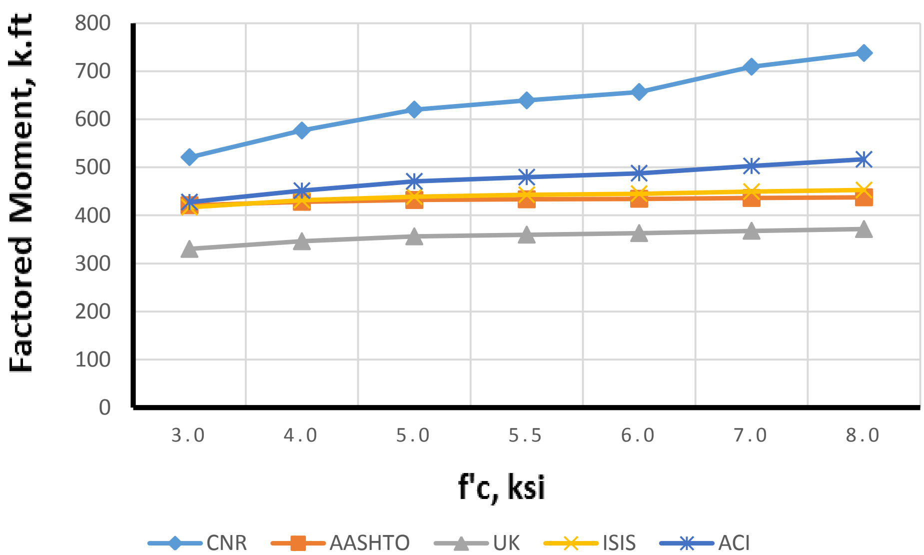

Figure 3.2.31 – Effect of f_c' *on factored moment* $(\rho = 0.0064)$

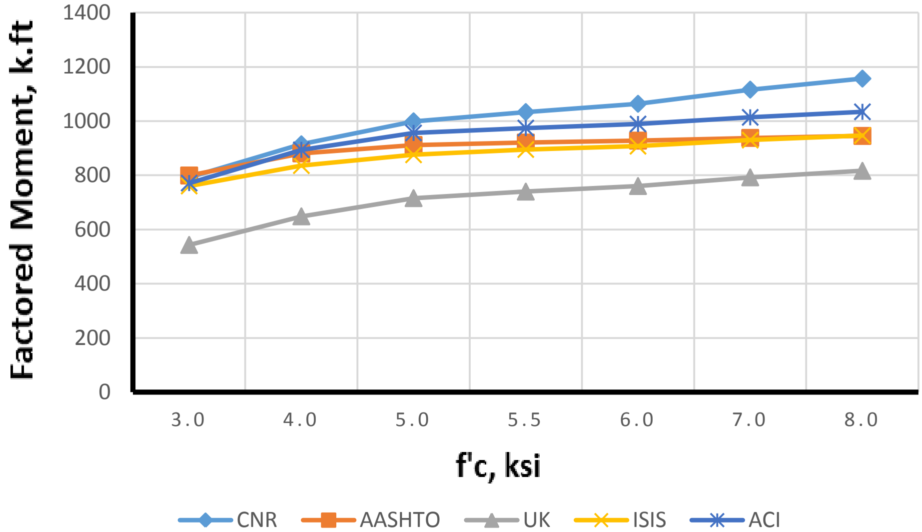

Figure 3.2.32 – Effect of f_c' on factored moment ($\rho = 0.0171$)

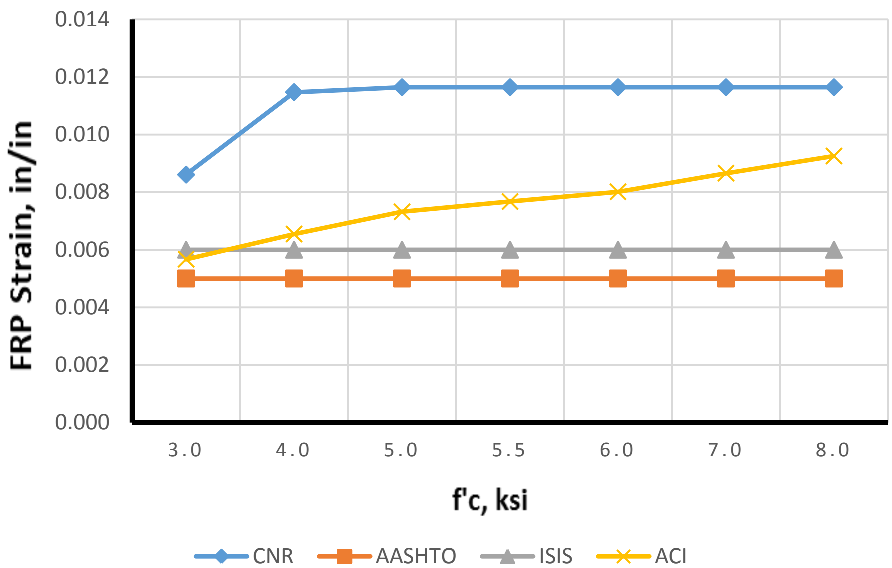

Figure 3.2.33 – Effect of f_c' on FRP strain ($\rho = 0.0033$)

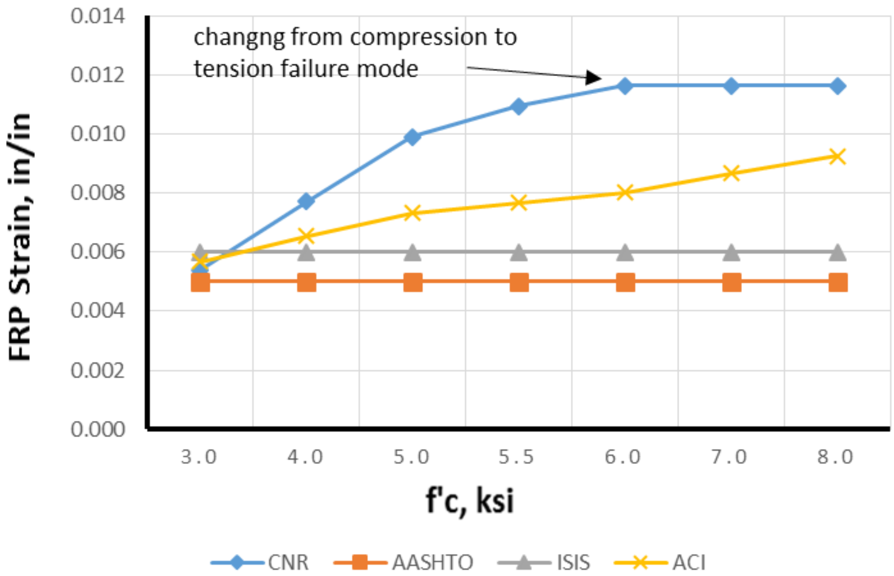

Figure 3.2.34 – Effect of f_c' *on FRP strain* ($\rho = 0.0064$)

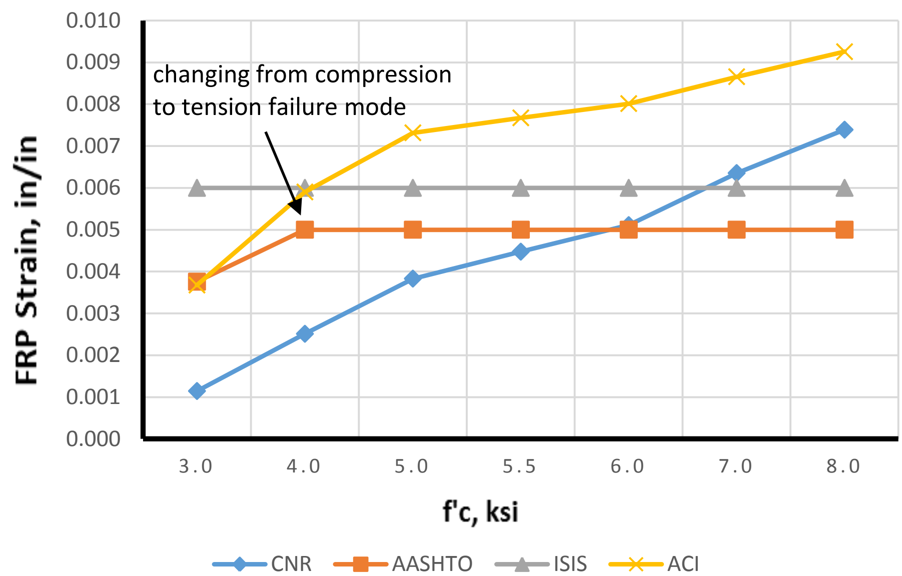

Figure 3.2.35 – Effect of f_c' *on FRP strain* ($\rho = 0.0171$)

3.3 Shear FRP Strengthening of RC/PC Bridge Members

3.3.1 Introduction

A review and analysis of the shear strengthening provisions for concrete bridge members with FRP is presented in this section for the six reviewed guidelines. The organization of this section is based on the framework presented in ACI 440.2R-08 since it offers the most complete coverage of the subject. The specific items for analysis and comparison include:

- Wrapping schemes
- Strength reduction factors
- Reinforcement and spacing limits
- FRP strain limits
- Shear design approach and assumptions
- Shear analysis procedure and results

3.3.2 Wrapping schemes

Three types of wrapping schemes are generally used to increase the shear strength of rectangular beams: 4-sided (complete or "closed") wrap, 3-sided (U-wrap), and 2-sided wrap, as shown in Figure 3.3.2. Each standard mentions these three types, and some offer recommendations and comments, which are summarized below.

3.3.2.1 ACI

ACI notes that the completely-wrapped scheme is the most efficient, followed by the three-sided U-wrap, although complete wrap is more common in columns. In all wrapping schemes, the FRP system can be installed continuously along the span of a member or placed as discrete strips. However, the use of continuous FRP that completely encases a member is discouraged since it potentially prevents migration of trapped moisture.

3.3.2.2 AASHTO

Similar to ACI, AASHTO notes that the 2-sided wrap is least effective, as it is subject to premature debonding under high shear loads. Similarly, U-wrap schemes may debond prior to a complete wrap scheme, but U-wrap is popular in practice because of its wide applicability and ease of installation. U-wrap, or U-jacketing, can be combined with anchorage to increase the effectiveness of the FRP by anchoring the fibers, preferably, in the compression zone. Properly design anchors can result in the fibers reaching their tensile capacity prior to debonding, permitting the jacket to behave as if it were completely wrapped.

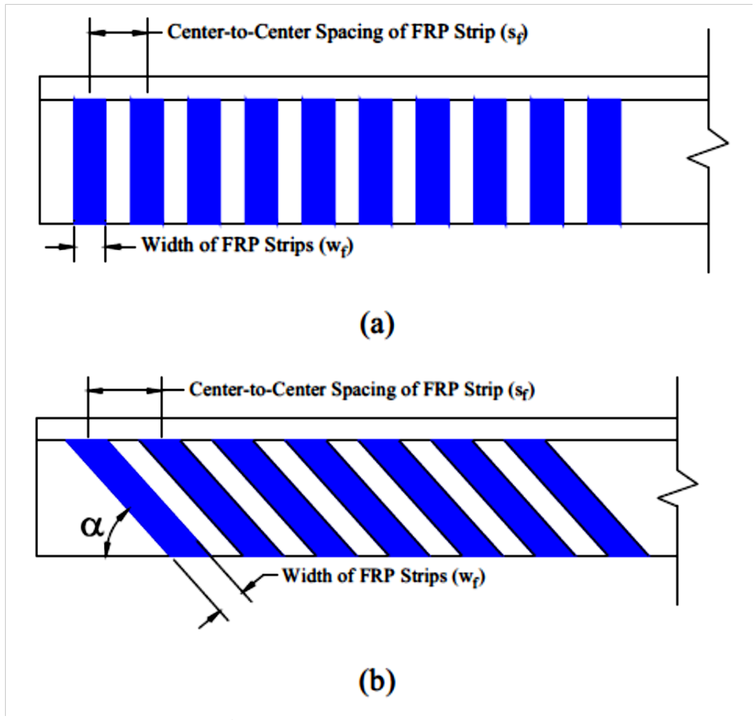

Figure 3.3.1 – a) Fibers at 90^o direction, b) Fibers at inclined direction (AASHTO)

3.3.2.3 ISIS

As with ACI and AASHTO, ISIS recommends the use of closed wrapping in beams whenever possible as this approach is most effective. U-shaped stirrups are recommended when access to the full perimeter of the beam is not possible, as in the case of a T-beam. In the case of AASHTO-type beams (Figure 3.3.2) or shear walls, side bonding is the only form of strengthening possible. ISIS notes that 2 and 3-sided schemes are bond-critical and, depending on shear value (discussed in ISIS section 7.4.2), may require anchorage. Figure 3.3 illustrates the typical anchoring systems described in ISIS.

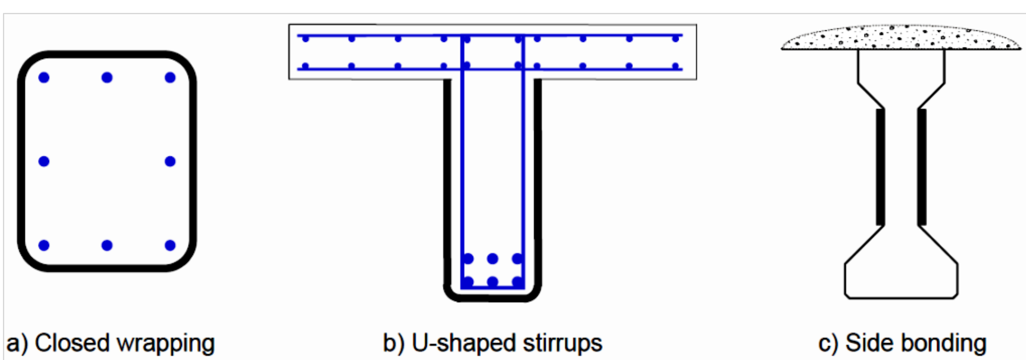

Figure 3.3.2 – Recommended application of wrapping schemes in ISIS (ISIS)

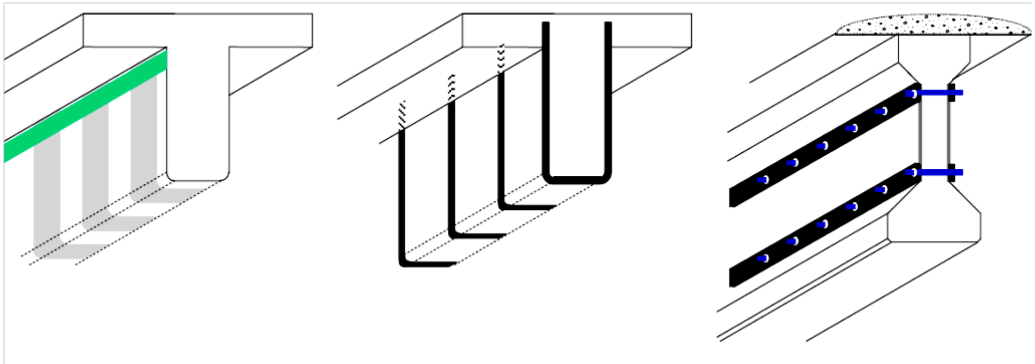

Figure 3.3.3 – Typical anchoring systems for FRP shear reinforcement described in ISIS (ISIS)

3.3.2.4 CNR

CNR notes that, For U-wrap schemes, delamination of the ends of the FRP reinforcement can be avoided by using anchors in the form of laminates/sheets and/or bars installed in the direction of the member longitudinal axis. In such cases, the behavior of the U-Wrap can be considered equivalent to that of a completely wrapped member.

3.3.2.5 TR55

TR 55 recommends that the FRP is placed such that the principal fiber orientation is either 45 or 90 degrees to the longitudinal axis of the member.

3.3.2.6 Wrapping schemes – summary

In summary, there is little difference in the wrapping schemes presented and recommended by the different codes. Essentially, completely wrapped sections, 3-sided wrap, and 2-wrap are considered, where the wrapping can be continuous or in parallel strips either at 90 degrees to the member direction or at an inclined angle. Detailed provisions to determine the strength of such schemes differs somewhat among codes and is discussed in section 3.3.6 below.

3.3.3 Strength reduction factors

FRP shear strength reduction factors are summarized in Figure 3.3.4 for the different codes. The largest reduction value (most conservative) is adopted by ISIS (0.56) and the smallest value (least conservative) is adopted by AASHTO (0.85). It is noted that AASHTO uses other limits such as restricting the maximum FRP stirrup spacing, depending on the total shear value. Similar to flexure, the highly conservative reduction factor for shear given by ISIS is attributed to the fact that the ISIS factor incorporates environmental and material reduction factors as well.

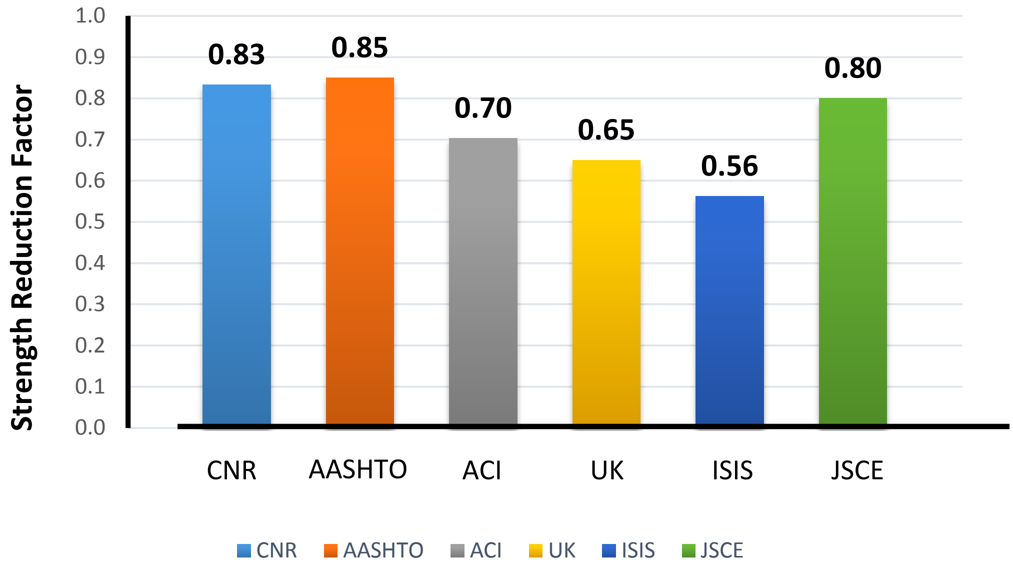

Figure 3.3.4 - FRP strength reduction factor

3.3.4 Reinforcement limits and spacing limits

3.3.4.1 ACI

3.3.4.1.1 Shear strengthening limits

The sum of the shear strengths provided by the FRP and existing shear reinforcement are to be limited to the criteria given for steel alone, as given in ACI 318. This limit is given in terms of four times the nominal shear strength of the concrete ($2\sqrt{f_c'}\, b_w d$), and is expressed as:

$V_s + V_f \leq 8\sqrt{f_c'}\, b_w d$ in-lb units (3.3.1a – ACI eq.11-11)

$V_s + V_f \leq 0.66\sqrt{f_c'}\, b_w d$ SI units (3.3.1b)

3.3.4.1.2 Spacing of FRP strips

For external FRP shear strengthening in the form of discrete strips, ACI specifies limits in articles 11.1 and 11.4.2. Article 11.1 stipulates that the center-to-center spacing between the strips should not exceed the sum of *d*/4 plus the width of the strip. Moreover, Article 11.4.2 states that spacing limits should follow those given in ACI 318, which are as follows (equations 3.3.2a and 3.3.2b):

For $V_s + V_f \leq 4\sqrt{f_c'}\, b_w\, d$ $S_{max} = \frac{d}{2} \leq 24$ in (3.3.2a)

For $V_s + V_f > 4\sqrt{f_c'}\, b_w\, d$ $S_{max} = \frac{d}{4} \leq 12$ in (3.3.2b)

3.3.4.2 AASHTO

3.3.4.2.1 Maximum FRP shear reinforcement

In AASHTO, the amount of FRP used cannot result in a section with nominal shear strength exceeding the limit given in equation 3.3.3:

$$V_n = 0.25\, f_c' b_v d_v + V_p \qquad \text{(3.3.3-AASHTO eq. 5.8.3.3-2)}$$

where:

$$V_n = V_c + V_s + V_f \qquad (3.3.4)$$

The factored shear strength, V_r, is defined as:

$$V_r = \phi(Vc + Vs + Vp) + \phi_{frp} V_{frp} \qquad \text{(3.3.5 – AASHTO eq. 4.3.1-1)}$$

where:

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

$$= 0.0316\beta\sqrt{f_c'}\, b_v d_y \qquad \text{(3.3.6 – AASHTO eq.5.8.3.3-3)}$$

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

$$= \frac{A_v . f_{yt} . d_v (\cot\theta + \cot\alpha \sin\alpha)}{s_v} \qquad \text{(3.3.7 - LRFD Eq.5.8.3.3-4)}$$

where:

$$\boldsymbol{d_v = max(\, d_{v1}, d_{v2}, d_{v3})} \qquad (3.3.8)$$

$$d_{v1} = d - \frac{a}{2} \quad , \; d_{v2} \; = 0.9d \; , \, d_{v3} \; = 0.72 h_T$$

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

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

ϕ = 0.9

ϕ_{frp}= FRP resistance factor = 0.85

3.3.4.3 Summary

Table 3.3.1 summarizes expressions for shear reinforcement limits by different codes, while Table 3.3.2 summarizes spacing limits.

Table 3.3.1- Maximum shear resistance allowed

Code	Limit	Equation	
AASHTO	Maximum total allowable shear	$V_n = 0.25f'_c b_v d_v + V_p$ where: $V_n = V_c + V_s + V_f$	
CNR	Maximum total allowable shear	$V_{Rd,max} = 0.3f_{cd}bd$	
ACI	Max. shear by steel & FRP	$V_s + V_f \leq 8\sqrt{f'_c}\, b_w d$ $V_s + V_f \leq 0.66\sqrt{f'_c}\, b_w d$	in-lbs units SI units
UK	Maximum permissible shear stress	$0.8\sqrt{f_{cu}}$ or 675 psi (5 N/mm^2)	
ISIS	Max. strengthening (bridge code)	$V_c + V_s + V_{FRP} \leq 0.25\phi_c f'_c b_v d_v$	

As shown in Table 3.3.1, the maximum combined shear contribution is a fixed value for a given section. For this reason, the amount of allowable FRP shear reinforcement depends upon the amount of steel shear reinforcement present in the section (and associated shear capacity of the steel stirrups V_s). The effect of the existing level of V_s on the allowed FRP shear capacity (V_f) is shown in Figures 3.3.5a-c, for different ratios of V_s/V_c and values of f'_c. An obvious observation is the increase in allowed FRP shear strengthening with an increase of f'_c. It is also seen that ACI is most restrictive, while CNR allows the most strengthening, while AASHTO and ISIS fall between these bounds. Notice that for ACI, with a V_s/V_c ratio of 4.0, no FRP shear strengthening is allowed (Fig. 3.3.5c).

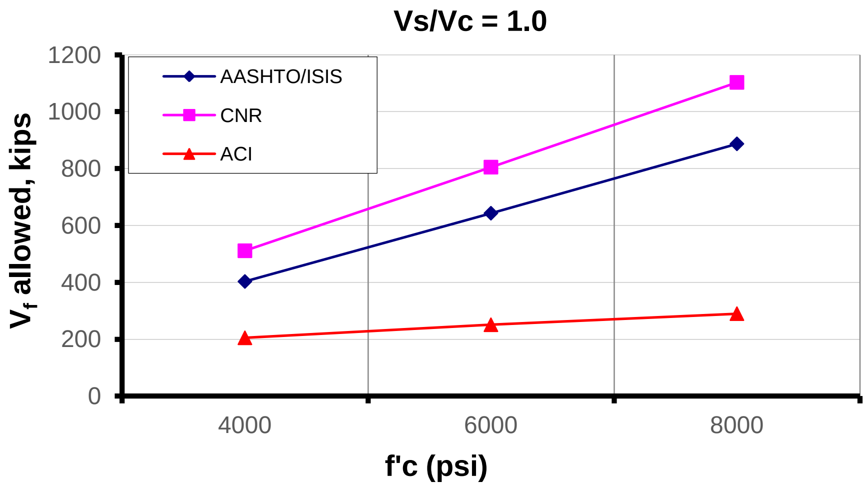

Figure 3.3.5a

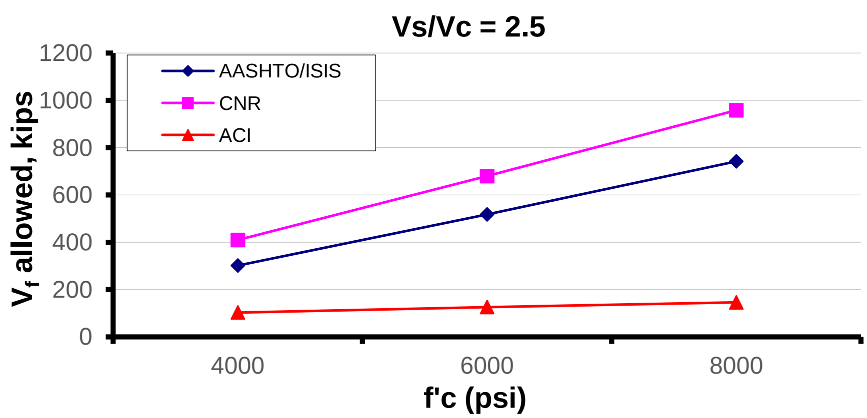

Figure 3.3.5b

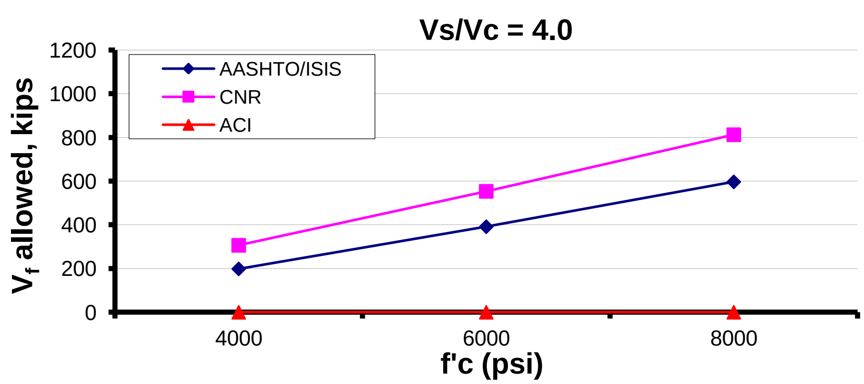

Figure 3.3.5c

Figure 3.3.5 – Effect of V_s/V_c on allowed V_f

Table 3.3.2 - Maximum spacing of FRP shear reinforcement

Code	Equation
AASHTO	For $v_u < 0.125f'_c$, use $S_{max} = 0.8d_v \leq 24\text{in}$ For $v_u > 0.125f'_c$, use $S_{max} = 0.4d_v \leq 12\text{in}$
CNR	2 in (50 mm) $\leq w_f \leq$ 10 in (250 mm), and $w_f \leq p_f \leq \min\{0.5d, 3w_f, w_f + 8 \text{ in } (200) \text{ mm}\}$.
ACI	$S_{FRP} \leq w_{FRP} + \frac{d_{FRP}}{4}$ For $V_s + V_f \leq 4\sqrt{f_c'}\, b_w\, d$, use $S_{max} = \frac{d}{2} \leq 24$ in For $V_s + V_f > 4\sqrt{f_c'}\, b_w\, d$, use $S_{max} = \frac{d}{4} \leq 12$ in
ISIS	Bridge Code S6-06: $S_{FRP} \leq w_{FRP} + \frac{d_{FRP}}{4}$ Building Code: $s \leq 0.75\, d_v \leq 24\, in\, (600\, mm)\, for\, \frac{V_f - V_p}{b_v d_v} < 0.1\, \phi_c f_c'$ $s \leq 0.33\, d_v \leq 12\, in\, (300\, mm)\, for\, \frac{V_f - V_p}{b_v d_v} \geq 0.1\, \phi_c f_c'$

The effect of the FRP strip width on spacing limits for different codes is evaluated for different beam depth using beam dimensions provided in section 3.3.6.7 of this report. Beam depths considered are 12, 24, 36, 48, and 60 in. Figures 3.3.7 to 3.3.11 present the maximum spacing limits as a function of strip width for different codes. Except for CNR and ISIS, codes have different maximum spacing limits depending the applied shear value. For most codes, spacing limits for low shear values fall between 10 - 20 in, while for high shear values, limits fall between 5-10 in. Limits are generally expressed as a function of concrete compressive strength. However, the ISIS S6-06 bridge code spacing limit is independent of concrete compressive strength f_c' and is a function of the depth d_{FRP} and strip width w_{FRP}.

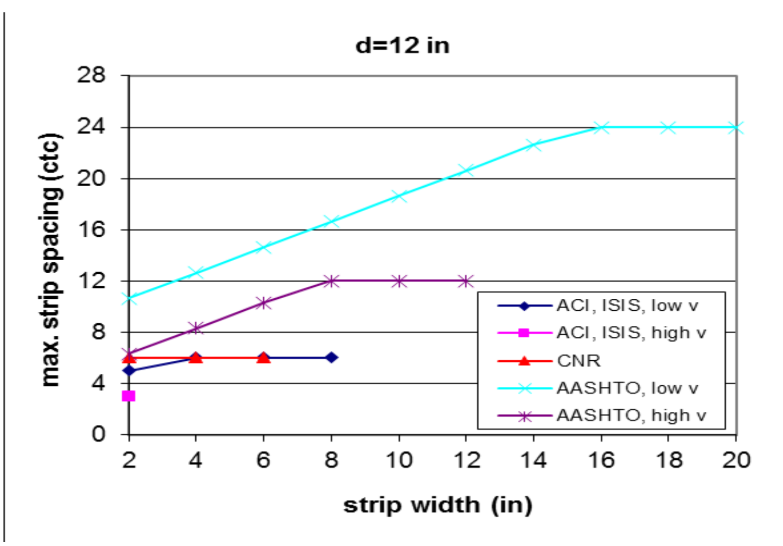

Figure 3.3.6– Effect of strip width on maximum strip spacing, d = 12

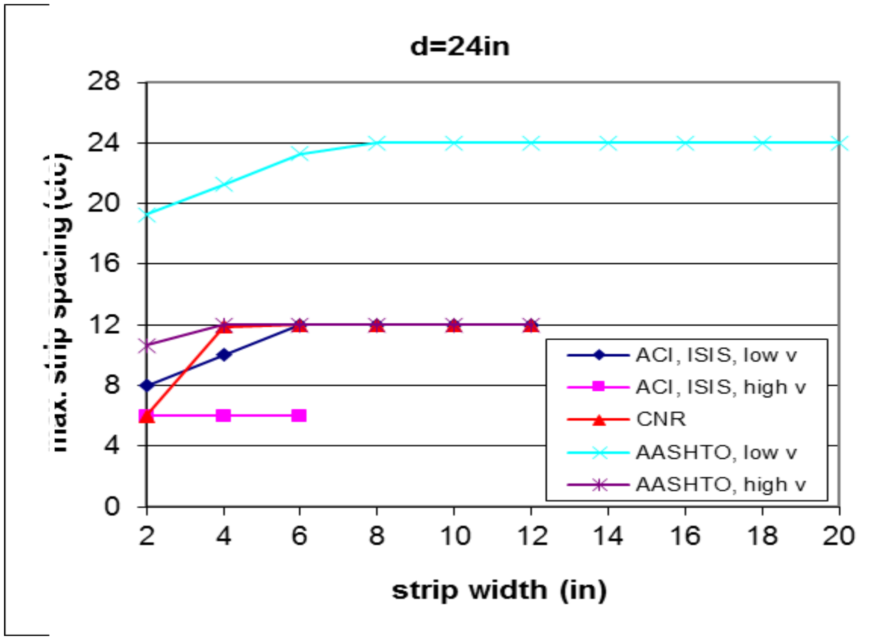

Figure 3.3.7– Effect of strip width on maximum strip spacing, d = 24

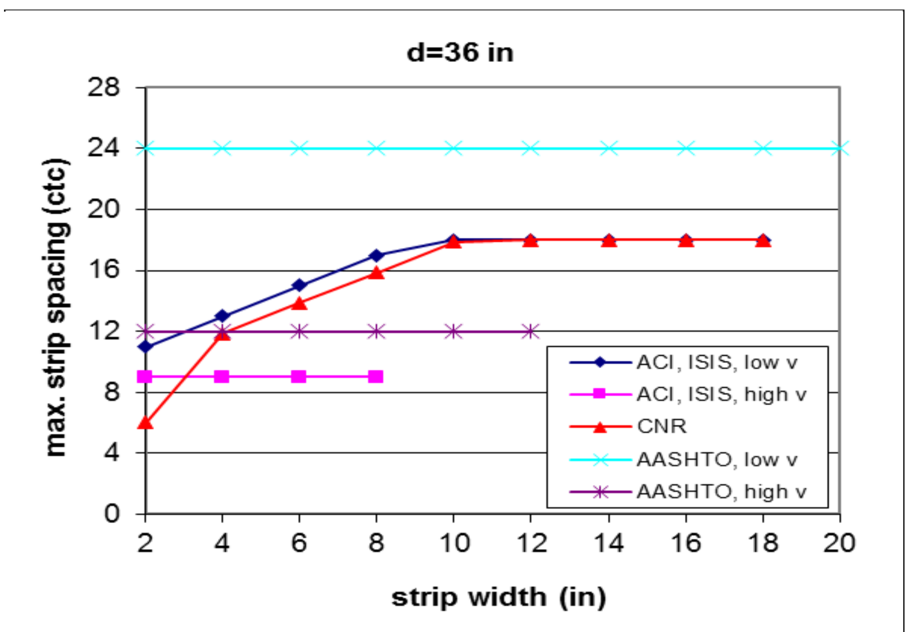

Figure 3.3.8– Effect of strip width on maximum strip spacing, d = 36

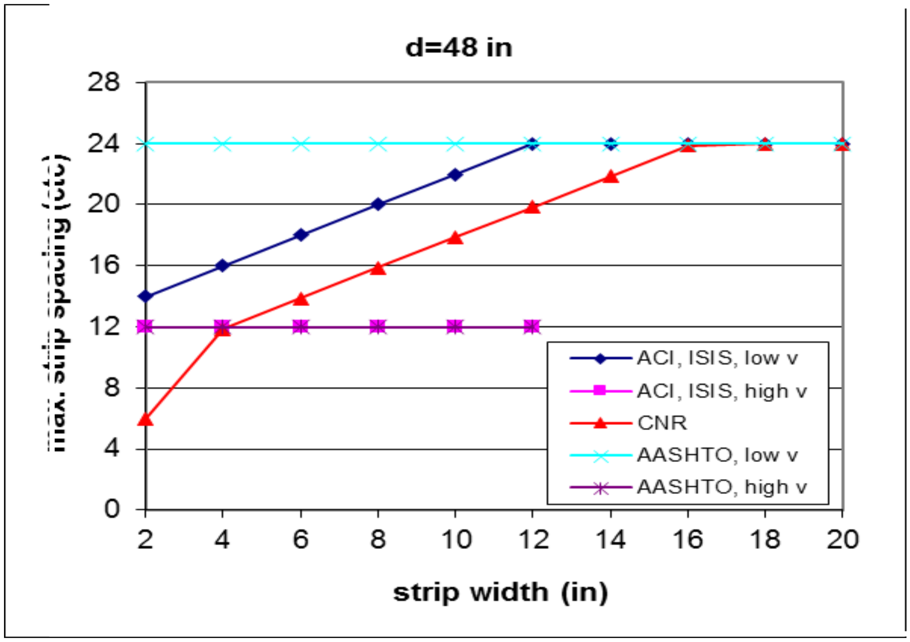

Figure 3.3.9 – Effect of strip width on maximum strip spacing, d = 48

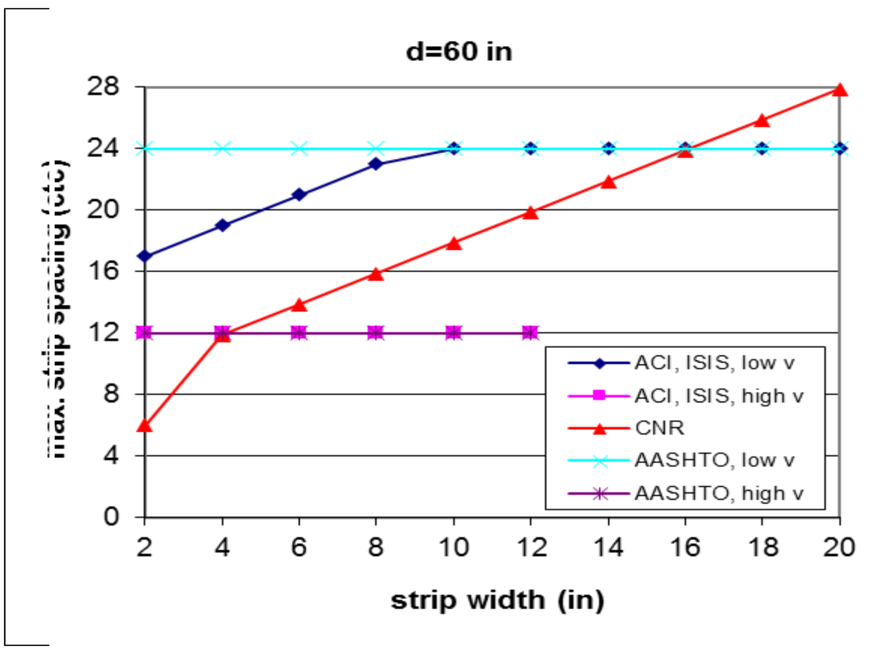

Figure 3.3.10 – Effect of strip width on maximum strip spacing, d = 60

3.3.5 FRP design strain limits

3.3.5.1 ACI

For completely wrapped members with FRP, ACI sets the following FRP strain limit to prevent wide cracks from forming that could cause a loss of aggregate interlock in the concrete (equation 3.3.18, ACI equation 11-6a):

$\varepsilon_{fe} = 0.004 \leq 0.75\varepsilon_{fu}$ (3.3.18 – ACI eq. 11-6a)

For U-wrap and 2-sided wrap schemes, aggregate interlock is preceded by FRP delamination. Therefore, setting strain limits to prevent delamination failure is the control criterion. The following strain limit equation incorporates a bond-reduction coefficient κ_v applicable to shear (equation 3.3.19, ACI equation 11-6b):

$\varepsilon_{fe} = \kappa_v \varepsilon_{fu} \leq 0.004$ (3.3.19 – ACI eq. 11-6b)

where:

$\kappa_v = \frac{k_1 k_2 L_e}{468\varepsilon_{fu}} \leq 0.75 \quad (in - lb\ units)$ (3.3.20 - ACI eq. 11-7)

$\kappa_v = \frac{k_1 k_2 L_e}{11900\varepsilon_{fu}} \leq 0.75$ (SI units)

$L_e = \frac{2500}{(n_f t_f E_f)^{058}} \quad (in - lb\ units)$ (3.3.21 - ACI eq. 11-8)

$L_e = \frac{23300}{(n_f t_f E_f)^{058}}$ (SI units)

$k_1 = \left(\frac{f'_c}{4000}\right)^{2/3}$ (in - lb units) (3.3.22 - ACI eq. 11-9)

$k_1 = \left(\frac{f'_c}{27}\right)^{2/3}$ (SI units)

$$k_2 = \begin{cases} \frac{d_{fv} - L_e}{d_{fv}} & for\ U - wraps \\ \frac{d_{fv} - 2L_e}{d_{fv}} & for\ two\ sides\ bonded \end{cases}$$ (3.3.23 - ACI eq. 11-10)

3.3.5.2 AASHTO

AASHTO specifies the effective FRP strain, ε_{fe}, which represents the average strain experienced by the FRP at shear failure of the strengthened member. When the FRP is fully anchored, as in the case of complete wrap or for U-wrap with anchors, FRP rupture is the expected mode of failure. The strain limit to prevent such a failure is expressed as follows:

$\varepsilon_{fe} = R_f \varepsilon_{fu}$ (3.3.24)

where:

$$R_f = 0.088 \le 4\left(\rho_f E_f\right)^{-0.67} \le 1.0 \tag{3.3.25}$$

In the case of side bonding or U-wrap with other anchorage causing non-rupture failures to be the more likely mode of failure, the expression for strain limit is as follows:

$$\varepsilon_{fe} = R_f \varepsilon_{fu} \le 0.004 \tag{3.3.26}$$

where:

$$R_f = 0.06 \le 3\left(\rho_f E_f\right)^{-0.67} \le 1.0 \tag{3.3.27}$$

The reduction factor, $\boldsymbol{R_f}$, is to be found from tests in which the load is applied at a distance from the support sufficient to assume that plane sections before deformation remain plane after deformation, i.e. slender beam behavior. Thus, these provisions are only applicable to beams with a shear span-to-depth ratio greater than 2.5.

3.3.5.3 ISIS

The reviewed ISIS procedure to establish strain limits is taken from the S6-06 bridge code. Strain limits are specified in Equations 3.3.28a-c (7.3) with limits for FRP strength given by Eq. 3.3.28a (7.3a), aggregate interlock by Eq. 3.3.28b (7.3b), and bond critical applications such as U-shaped FRP stirrups by Eq. 3.3.28c (7.3c).

$\varepsilon_{FRPe} \le 0.75\varepsilon_{FRPu}$	FRP Strength	(3.3.28a – ISIS eq 7-3a)
$\varepsilon_{FRPe} \le 0.004$	Aggregate interlock	(3.3.28b – ISIS eq 7-3b)
$\varepsilon_{FRPe} \le \kappa_v \varepsilon_{FRPe}$	Bond capacity (U-wrap only)	(3.3.28c – ISIS eq 7-3c)

Equation 3.3.28c requires the evaluation of the bond reduction coefficientκ_v. The steps for calculating κ_v are identical to those described for ACI, above.

3.3.5.4. Summary

Table 3.3.3 summarizes the shear strengthening strain limits. UK and CNR have fixed strain limits of 0.004 for U-wrap and side wrap, while CNR has a limit of 0.005 for a completely wrapped system.

Table 3.3.3 - Specified maximum FRP strain for different codes

Code	Application	Equation
AASHTO	U-Wrap and 2-sided	$\varepsilon_{fe} = R_f \varepsilon_{fu} \leq 0.004$ *where:* $R_f = 0.06 \leq 3(\rho_f E_f)^{-0.67} \leq 1.0$
	Completely wrapped	$\varepsilon_{fe} = R_f \varepsilon_{fu}$ *where:* $R_f = 0.088 \leq 4(\rho_f E_f)^{-0.67} \leq 1.0$
CNR	Completely wrapped	ε_f, max = 0.005
	U-wrap and 2-sided	0.004
UK	U-Wrap	0.004
ACI	Completely wrapped	$\varepsilon_{fe} = 0.004 \leq 0.75\varepsilon_{fu}$
	U-wrap and 2-sided	$\varepsilon_{fe} = K_v \varepsilon_{fu} \leq 0.004$
ISIS	All wrapping cases	$\varepsilon_{FRPe} \leq 0.75\varepsilon_{FRPu}$ $\varepsilon_{FRPe} \leq 0.004$
	U-shaped FRP stirrups only	$\varepsilon_{FRPe} \leq K_v \varepsilon_{FRPe}$

3.3.6 Shear design approach and assumptions

3.3.6.1 ACI

The goal of the analytical procedure described in ACI is to evaluate FRP shear resistance V_f while applying applicable strain limits necessary to meet strength, aggregate interlock and bond requirements. The following are the steps needed for FRP shear calculation.

Step 1: Evaluate the bond reduction coefficient κ_v (provided in the strain limits section above).

$$\kappa_v = \frac{k_1 k_2 L_e}{468\varepsilon_{fu}} \leq 0.75 \ \ (in - lb\ units) \qquad \text{(3.3.20 - ACI eq. 11-7)}$$

$$\kappa_v = \frac{k_1 k_2 L_e}{11900\varepsilon_{fu}} \leq 0.75 \quad \text{(SI units)}$$

Step 2: Evaluate ε_{fe} for U-wrap case using the strain limit equation:

$$\varepsilon_{fe} = K_V \varepsilon_{fu} \leq 0.004 \qquad \text{(3.3.19 – ACI eq. 11-6b)}$$

Step 3: Calculate the effective stress of FRP, f_{fe}:

$f_{fe} = E_f \varepsilon_{fe}$ (3.3.29 - ACI eq. 11-5)

Step 4: Calculate FRP shear area, A_{fv}:

$A_{fv} = 2nt_f w_f$ (3.3.30 - ACI eq. 11-4)

Step 5: Calculate shear resistance for FRP, V_f:

$V_f = \frac{A_{fv} f_{fe} (\sin\alpha + \cos\alpha) d_{fv}}{s_f}$ (3.3.31 - ACI eq. 11-3)

For a 90 degree angle (i.e. vertically placed strips), (sin α + cos α) = 1, as per the assumption in eq. 3.3.33.

Step 6: Calculate shear resistance by concrete and steel:

$V_c = 2\lambda\sqrt{f_c'} b_w d$ (3.3.32)

$V_s = \frac{A_v f_y d}{s}$ (3.3.33)

Step 7: Calculate nominal and factored shear by concrete, steel, and FRP:

$V_n = V_c + V_s + V_f$

$\phi V_n = \phi(V_c + V_s + V_f)$

3.3.6.2 ISIS

The ISIS procedure to calculate the shear resistance of FRP is as follows:

Step 1: Evaluate d_{FRP}:

d_{FRP} = The greater of 0.72*h* or 0.9*d*

Step 2: Choose the smallest FRP strain value from the following conditions (ISIS eq. 7.3):

$\varepsilon_{FRPe} \leq 0.75\varepsilon_{FRPu}$	FRP Strength	(3.3.28a – ISIS eq. 7-3a)
$\varepsilon_{FRPe} \leq 0.004$	Aggregate interlock	(3.3.28b – ISIS eq. 7-3b)
$\varepsilon_{FRPe} \leq \kappa_v \varepsilon_{FRPe}$	Bond capacity (U-wrap only)	(3.3.28c – ISIS eq. 7-3c)

Step 3: Use equation 3.32 to evaluate V_{FRP}:

$$V_{FRP} = \frac{\phi_{FRP} E_{FRP} \varepsilon_{FRP} A_{FRP} d_{FRP} (\cot\theta + \cot\beta)\sin\beta}{s_{FRP}}$$ (3.3.32 – ISIS eq. 7-3c)

Step 4: Calculate the shear resistance of concrete and steel (building code):

$$V_c = 0.2\lambda\phi_c\sqrt{f_c'}b_v d$$ For beams (3.3.33a – ISIS eq. 7-22a)

$$V_c = 0.2\lambda\phi_c\sqrt{f_c'}0.8\,A_g$$ For columns (3.3.33b – ISIS eq. 7-22b)

$$V_c = 0.2\lambda\phi_c\sqrt{f_c'}0.8\,b_v L$$ For walls (3.3.33c – ISIS eq. 7-22c)

$$V_s = \frac{\phi_s f_y A_v d}{s}$$ (3.3.34 – ISIS eq. 7.21)

Step 6: Calculate the total shear capacity of the concrete, steel, and FRP:

$$V_r = V_c + V_s + V_{FRP}$$

Note: when computing the unfactored total shear, the reduction factors ϕ_c, ϕ_s, and ϕ_{FRP} are dropped from the equations for V_c, V_s, and V_{FRP}.

3.3.6.3 AASHTO

The AASHTO procedure is as follows:

Step 1: Evaluate the nominal shear resistance of the concrete:

$$V_c = 0.0316\,\beta\,\sqrt{f_c'}\,b_v\,d_v$$ (3.3.35)

Assuming β =2 and θ = 45^o (simplified procedure), $d_v = Max\,(\,d - \frac{a}{2}, 0.9d, 0.72\,h_T)$

Step 2: Evaluate the nominal shear resistance of the steel:

$$V_s = \frac{A_v\,f_y\,d_v\,(cot\,\theta + cot\,\alpha)\,sin\,\alpha}{S}$$ (3.3.36)

where:

S = internal shear reinforcement spacing
A_v = area of internal shear reinforcement

Step 3: Evaluate the nominal shear resistance of the FRP:

$$V_{frp} = \rho_f E_f\,\varepsilon_{fe} b_v d_f\left(\sin\alpha_f + \cos\alpha_f\right)$$ (3.3.37, AASHTO eq. 4.3.2-1)

where:

ρ_f = reinforcement ratio of FRP

for discrete strips, $\rho_f = \frac{2n_f\, t_f\, w_f}{b_v s_f}$ (3.3.38a)

for continuous sheets, $\rho_f = \frac{2n_f\, t_f}{b_v}$ (3.3.38b)

t_f = FRP reinforcement thickness
w_f = width of the strip
s_f = center-to-center spacing of FRP
b_v = effective web width taken as the minimum web width within the effective depth (d_f)
f_{fe} = effective stress of FRP
d_f = effective depth of FRP measured from the top of FRP reinforcement to the centroid of the longitudinal reinforcement
α_f = angle of inclination of FRP with respect to the longitudinal axis of the member
E_f = modulus of elasticity of FRP
$\mathcal{E}_{fe}$ = effective strain of FRP (refer to equations 3.3.24 - 3.3.27)

Step 4: Evaluate the nominal shear resistance of the member:

$$V_n = V_c + V_s + V_{FRP}$$

3.3.6.3.1 AASHTO GFRP

The steps to calculate shear resistance for AASHTO GFRP are as follows:

Step 1: Evaluate the nominal shear resistance provided by the concrete:

$$V_c = 0.16\sqrt{f_c'}\, b_w\, c \leq 0.32\sqrt{f_c'}\, b_o\, c \qquad (3.3.39)$$

where:

c = distance from extreme compression fiber to neutral axis, in
$= kd$
k= ratio of depth of neutral axis to reinforcement depth

$$k = \sqrt{2\rho_f n_f + \left(\rho_f n_f\right)^2} - \rho_f n_f \qquad \text{(3.3.40 – AGFRP eq. 2.7.3-4)}$$

b_w = width of web, in
d = distance from extreme compression fiber to centroid of tension reinforcement, in

b_o = perimeter of critical section computed at $d/2$ away from the concentrated load (in)
Note: the shape of the critical section is taken as the same shape of the concentrated load.

Step 2: Evaluate design tensile strength for shear:

$$f_{fv} = 0.004E_f \leq f_{fb} \tag{3.3.41}$$

where:

$$f_{fb} = \left(0.005\frac{r_b}{d_b} + 0.3\right) f_{fd} \leq f_{fd} \tag{3.3.42}$$

E_f = modulus of elasticity of GFRP reinforcement, ksi
f_{fb} = strength of the bent portion of a GFRP bar, ksi
r_b = internal radius of the bent GFRP bar, in
d_b = GFRP bar diameter, in
f_{fd}= design tensile strength of GFRP bars considering reduction for service environment, ksi.

Step 3: Evaluate the nominal shear resistance by the shear reinforcement, V_f .

$$V_f = \frac{A_{fv}f_{fv}d}{s} \tag{3.3.43}$$

Step 4: Evaluate the nominal shear resistance, V_n :

$$V_n = V_c + V_f \tag{3.3.44}$$

3.3.6.4 CNR

The CNR procedure is as follows (see Figure 3.3.11):

Step 1: Evaluate the shear resistance of concrete:

$$V_{Rd,ct} = 0.6f_{ctd}bd\delta \tag{3.3.45}$$

where:

δ = 1

$$f_{ctd} = 0.7\frac{f_{ctm}}{\gamma_c} \tag{3.3.46}$$

$$f_{ctm} = 0.3\, f_c'^{0.67} \tag{3.3.47}$$

γ_c = 1.6

Step 2: Evaluate the shear resistance of steel:

$$V_{Rd,s} = \frac{A_{sw}}{s} \cdot f_{ywd} \cdot 0.9d \tag{3.3.48}$$

where:

A_{sw}, s, and f_{ywd} represent area, spacing, and steel stirrups yield strength, respectively.

Step 3: Evaluate the effective FRP design strength:

For a U-wrap configuration:

$$f_{fed} = f_{fdd} . \left[1 - \frac{1}{3} . \frac{l_e . \sin\beta}{min\{0.9.d, h_w\}}\right] \tag{3.3.49}$$

For completely wrapped members having rectangular cross sections:

$$f_{fed} = f_{fdd} . \left[1 - \frac{1}{6} . \frac{l_e . \sin\beta}{min\{0.9.d, h_w\}}\right] + \frac{1}{2} \left(\phi_R . f_{fd} - f_{fdd}\right) . \left[1 - \frac{l_e . \sin\beta}{min\{0.9.d, h_w\}}\right] \tag{3.3.50}$$

where:

f_{fd} = design strength of FRP reinforcements

$$\phi_R = 0.2 + 1.6 . \frac{r_c}{b_w}, \text{ and } 0 \leq \frac{r_c}{b_w} \leq 0.5 \tag{3.3.51}$$

Step 4: Evaluate the FRP contribution to the shear capacity:

For a rectangular cross-section and FRP side bonding configuration:

$$V_{Rd,f} = \frac{1}{\gamma_{Rd}} min\{0.9d, h_w\} f_{fed} 2t_f \frac{\sin\beta}{\sin\theta} \frac{w_f}{p_f} \tag{3.3.52a}$$

For U-wrapped or completely wrapped configurations:

$$V_{Rd,f} = \frac{1}{\gamma_{Rd}} 0.9d f_{fed} 2t_f (\cot\theta + \cot\beta) \frac{w_f}{p_f} \tag{3.3.52b}$$

Step 5: Evaluate the total shear capacity:

$$V_{Rd} = \min\{V_{Rd,ct} + V_{Rd,s} + V_{Rd,f}, V_{Rd,max}\} \tag{3.3.53 – CNR eq. 4.24}$$

where:

$$V_{Rd,max} = 0.3 f_{cd} bd \tag{3.3.54 – CNR eq. 10.14}$$

γ_{Rd} = 1.20
d = member effective depth
h_w = stem depth
f_{fed} = effective FRP design strength
t_f = thickness of the adopted FRP system
w_f = FRP width
p_f = FRP spacing

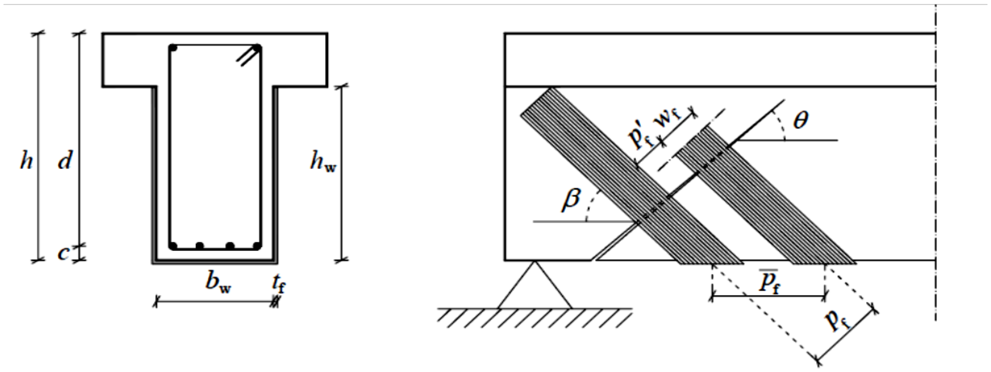

Figure 3.3.11 - CNR shear strengthening notation (CNR)

3.3.6.5 JSCE

The JSCE procedure is as follows:

Step 1: Evaluate the shear resistance of concrete:

$$V_{cd} = \beta_d \beta_p \beta_n f_{vcd} b_w \frac{d}{\gamma_b} \qquad (3.3.55 – \text{JSCE eq. } 6.4.4)$$

where:

$$f_{vcd} = 0.2\sqrt[3]{f'_{cd}} \ (N/mm^2) \leq 0.72(N/mm^2) \qquad (3.3.56 – \text{JSCE eq. } 6.4.5)$$

$$\beta_d = \sqrt[4]{1/d}\ (d{:}\ m), 1.5\ when\ \beta_d > 1.5$$

$$\beta_p = \sqrt[3]{100p_w}(d{:}\ m), 1.5\ when\ \beta_p > 1.5$$

$$\beta_n = 1 + M_0/M_d(N'_d \geq 0), when\ \beta_n > 2$$

$$= 1 + 2\,M_0/M_d(N'_d \geq 0), when\ \beta_n > 0$$

N'_d = design axial compressive force

M_d = design bending moment

M_0 = decompression moment

b_w = web width

d = effective depth
p_w = $A_s/(b_w \times d)$
A_s = cross-sectional area of reinforcing bars in tension side
f'_{cd} =design compressive strength of concrete (N/mm^2)
γ_b = member factor (in general, may be set to 1.3)

Step 2: Evaluate the shear resistance of steel:

$$V_{sd} = \left[A_w f_{wyd}(\sin\alpha_s + \cos\alpha_s)/S_s\right] Z/\gamma_b \quad \text{(3.3.57 – JSCE eq. 6.4.6)}$$

where:

A_w = total cross-sectional area of shear reinforcement in space *s*s
f_{wyd} = design tension yield strength of shear reinforcement [58 ksi max. (400 N/mm^2)]
α_s = angle formed by shear reinforcement about the member axis
s_s = spacing of shear reinforcement
z = lever arm length (generally may be set to d/1.15)
γ_b = member factor (generally may be set to 1.15)

Step 3: Evaluate the effective FRP design strength:

$$V_{fd} = K\left[A_f f_{fud}\left(\sin\alpha_f + \cos\alpha_f\right)/S_f\right] Z/\gamma_b \quad \text{(3.3.58 – JSCE eq. 6.4.7)}$$

where:

K = shear reinforcing efficiency of continuous fiber sheets according to Equation 3.59 (JSCE eq. 6.4.8)

$$K = 1.68 - 0.67R, however, 0.4 \le K \le 0.8 \quad \text{(3.3.59 – JSCE eq. 6.4.8)}$$

$$R = \left(\rho_f E_f\right)^{1/4}\left(\frac{f_{fud}}{E_f}\right)^{2/3}\left(\frac{1}{f'_{cd}}\right)^{1/3}, however, 0.5 \le R \le 2.0$$

$$\rho_f = A_f/\left(b_w s_f\right)$$

Step 4: Evaluate the total shear capacity:

$$V_{fyd} = V_{cd} + V_{sd} + V_{fd} \quad \text{(3.3.60 – JSCE eq. 6.4.3)}$$

3.3.6.6 UK

The maximum allowable design shear force due to ultimate loads, $V_{R,max}$ at any cross-section, is obtained from:

$$V_{R,max} = v_{max} \,.\, b.d \qquad (3.3.61 – UK TR55 eq. 7.1)$$

where:

b = width of section
d = effective depth of section
$V_{R,max} = \min\{ 0.8\ (f_{cu})^{0.5}, 5N/mm^2\}$

Step 1: Evaluate the shear resistance of concrete (per BS 8110):

$$v_c = \frac{0.79}{\gamma_m}\left\{\frac{100A_s}{(b_v d\}}\right\}^{1/3} (\frac{400}{d})^{1/4}\ N/mm^2 \qquad (3.3.62)$$

Step 2: Evaluate the shear resistance of steel (BS 8110):

$$V_s = \frac{0.87A_{sv}f_{yv}d}{s_v} \qquad (3.3.63)$$

Step 3: Evaluate the effective FRP design strength:

The amount of FRP required can be calculated using the same principles as in conventional reinforced concrete design. The shear resistance of the FRP is given by:

$$V_{Rf} = \left(\frac{1}{\gamma_{mf}}\right) A_{fs}\left(E_{fd}\varepsilon_{fe}\right) \sin\beta\ (1 + \cot\beta)(d_f/s_f) \qquad (3.3.64 – UK TR55 eq. 7.3)$$

where:

A_{fs} = area of FRP shear reinforcement
= $2t_f\ w_{fe}$ assuming that the FRP is placed on both sides of the member
w_{fe} = effective width of FRP, which is a function of shear crack angle and FRP strengthening configuration, equal to (d_f - L_e) where FRP is in the form of a U-jacket and (d_f - $2L_e$), where FRP is bonded to side faces
L_e = effective bond length= $461.3 / (t_f\ .E_{fd}\ )^{0.58}$
ε_{fe} = design strain in the FRP
β = angle between FRP and the longitudinal axis of the member = $45°$ or $90°$
d_f = effective depth of FRP shear reinforcement, usually equal to d for rectangular sections and (d - slab thickness) for T -sections

s_f = spacing between the centre line of FRP plates (see Section 3.7.3). Note that for continuous sheet reinforcement $s_f = w_{fe}$

γ_{mf} = partial safety factor for FRP

Step 4: Evaluate the total shear capacity. Add the concrete, steel, and FRP shear components:

$$V_T = V_c + V_s + V_{Rf}$$

3.3.6.7 Summary

To identify the effect of different parameters on the shear capacity of beams according to the different code procedures, an example singly-reinforced rectangular beam section was examined. The example beam has a f'_c = 4 ksi, and grade 60 longitudinal steel with area A_s = 3 in^2, and #3 stirrups spaced at 12 in (A_v=0.22 in^2). The analysis includes the use of a single CFRP ply of BASF MBrace CF130 to strengthen the beam in shear with a U-wrap scheme. The critical variables explored include the width of the FRP shear strengthening strip and the spacing between strips.

All applicable code reduction factors are removed when calculating unfactored FRP shear results, which are presented in Figures 3.3.12-3.3.15 for FRP stirrups. As shown in the figures, unfactored shear resistance values are very close for the different codes. As expected, the FRP shear capacities increase as FRP spacing decreases. Strip width has a minimal impact on capacity.

When code reduction factors are applied, some significant differences in capacity are observed, as shown in Figures 3.16-3.19 for FRP strips. Here, AASHTO and ACI consistently provide the largest design capacity values for FRP. Factored shear values for CNR, UK and ISIS result in nearly identical values as well, but significantly lower than AASHTO and ACI, with shear resistance values approximately 30 to 40% higher than those provided by CNR, UK and ISIS. Note that some code values do not appear on the graphs; this is due to a maximum strip spacing restriction. This is particularly apparent for ACI, which has relatively strict spacing requirements.

Figures 3.3.20 and 3.3.21 illustrate the FRP shear resistance for continuous U-wrap. As shown in Figure 3.3.30, results are similar for all codes. However, large differences emerge when reduction factors are applied, as shown in Figure 3.3.21. Similar to the FRP strip case, AASHTO and ACI provide the highest (and similar) design capacities, while the remaining codes provide similar, lower capacities.

Figures 3.3.22 and 3.3.23 compare 2-sided to U-wrap results. In all cases, 2-sided results consistently provide less capacity than U-wrap, as expected. Note, however, that AASHTO limits the maximum strain of FRP to 0.004 for both U-wrap and 2 sided schemes, and provides the same formula for shear resistance. Moreover, TR-55 does not provide an explicit procedure to calculate 2-sided shear capacity. JSCE and ISIS do not offer any formula for calculating the shear resistance of two sided wrap.

In Figure 3.3.24, the effect of changing FRP strip width on the components of the total shear resistance (for a constant gap between strips of $d/4$, which is the maximum allowed for ACI code). ACI, AASHTO and ISIS have somewhat similar results, while TR55 and CNR are most conservative. JSCE does not provide enough information to calculate the shear resistance of steel or concrete. As shown, increasing the FRP strip width does not result in significant changes in shear resistance (note that the maximum spacing allowed in CNR is exceeded in the last two widths shown in the graph, and is therefore not shown). This can be verified with Figure 3.3.25, which illustrates the relatively change in FRP capacity with strip width, for a constant gap between strips. Note that a 12 in strip width exceeds the maximum allowed in CNR. It appears that all codes have similar trends.

Figure 3.3.26 presents the effect of the ratio of the effective depth of FRP shear reinforcement to the effective depth of the section on unfactored FRP shear resistance. In the figure, the effective depth of the section is taken as 26 in and the effective depth of FRP is varied from 16 to 24 in. Note that the CNR approach for calculating shear resistance has no relationship to the effective depth of FRP, and was thus not included on the graph. As shown, AASHTO and ISIS results are very close, with increasing ratios of (d_{frp}/d) resulting in increased capacities. Although TR55 provides significantly lower capacities, trends are similar.

In Figures 3.3.27-3.3.29, the effect of changing the beam height on the components of total unfactored shear resistance is examined (for the case of continuous U-wrap). In the figures, the effective depth of the section is assumed to equal 4 in less than the beam height, while the effective depth of the FRP is assumed to be equal to 0.9 of the effective depth of the section, while the steel stirrups are assumed to be #3 bars. Changing the height of the beam has the most effect on the total shear resistance, which is most sensitive to the concrete component of resistance. ACI provides the highest capacities, although AASHTO and ISIS are similar but slightly lower.

Figures 3.3.30-3.3.32 show the effect of changing beam height on the shear capacity of each of the contributing components (concrete, steel, FRP) individually. In general, AASHTO, ACI, ISIS, and CNR provide similar values for FRP and concrete, while TR55 is significantly more conservative, especially at larger beam depths. Code results are more similar for steel capacity, where ISIS and CNR are most conservative.

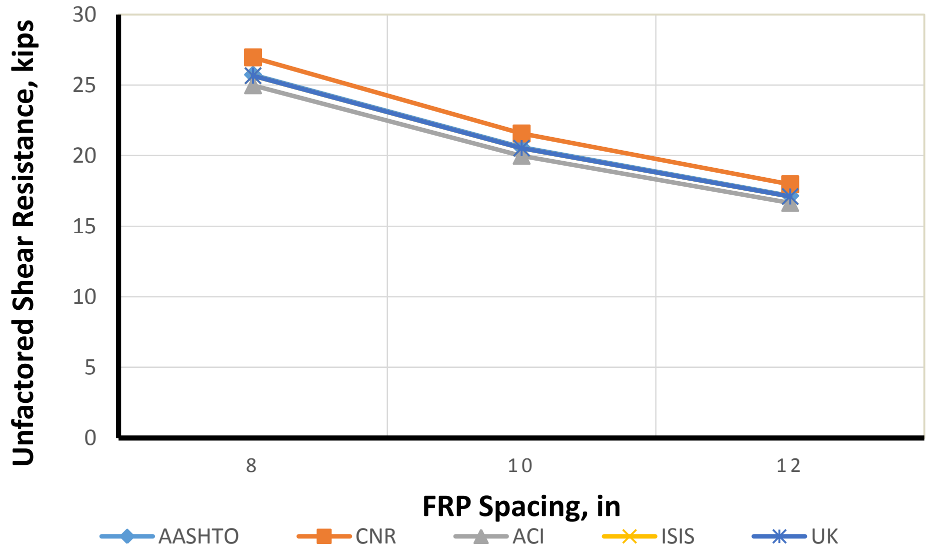

Figure 3.3.12 - Unfactored fiber shear resistance vs FRP spacing, width = 6 in, U-wrap

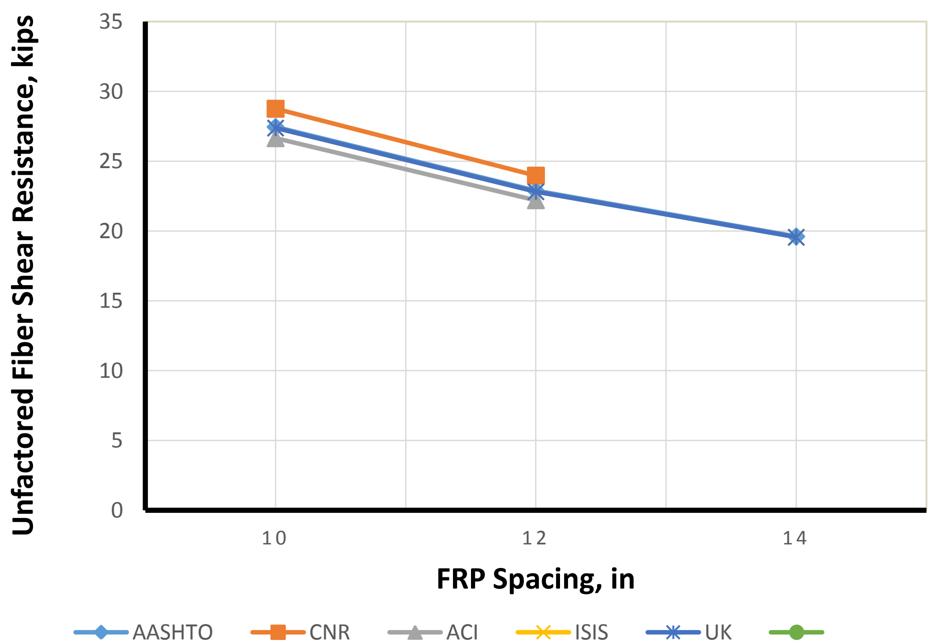

Figure 3.3.13 - Unfactored fiber shear resistance vs FRP spacing, width = 8 in, U-wrap

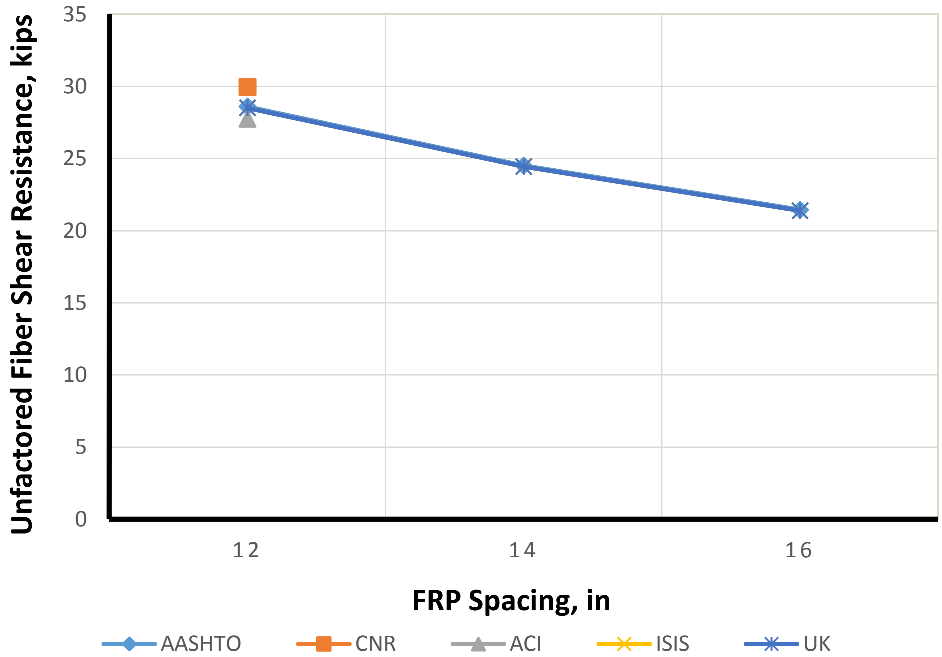

Figure 3.3.14 - Unfactored fiber shear resistance vs FRP spacing, width = 10 in, U-wrap

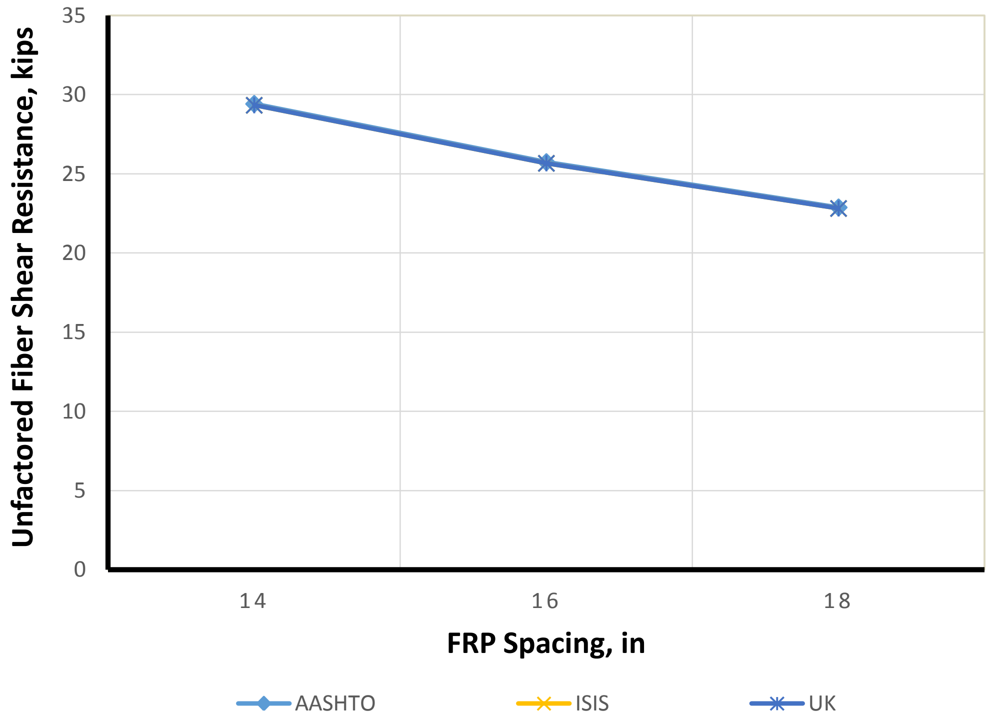

Figure 3.3.15 - Unfactored fiber shear resistance vs FRP spacing, width = 12 in, U-wrap

Note that a 12 in FRP width exceeds the maximum allowed in CNR (10 in), and CNR results thus do not appear.

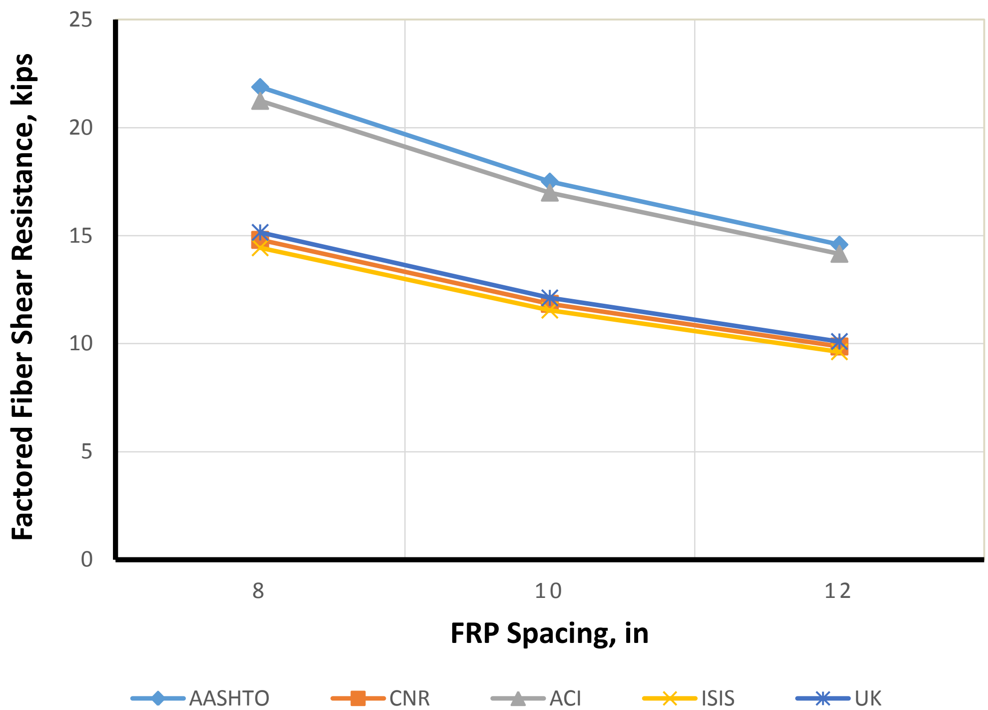

Figure 3.3.16 - Factored fiber shear resistance vs FRP spacing, width = 6 in, U-wrap

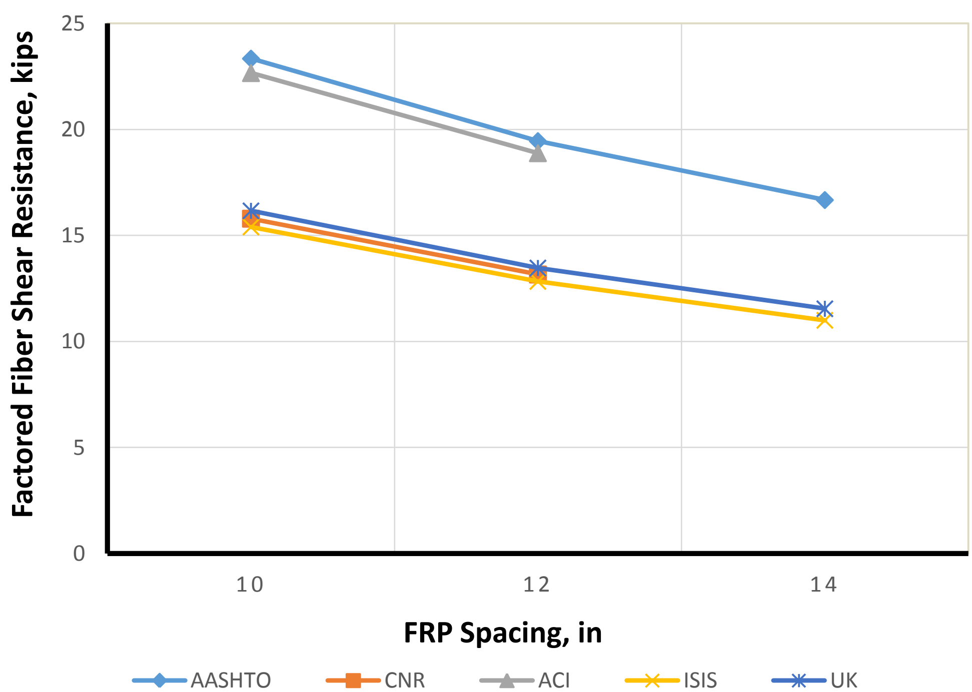

Figure 3.3.17 - Factored fiber shear resistance vs FRP spacing, width = 8 in, U-wrap

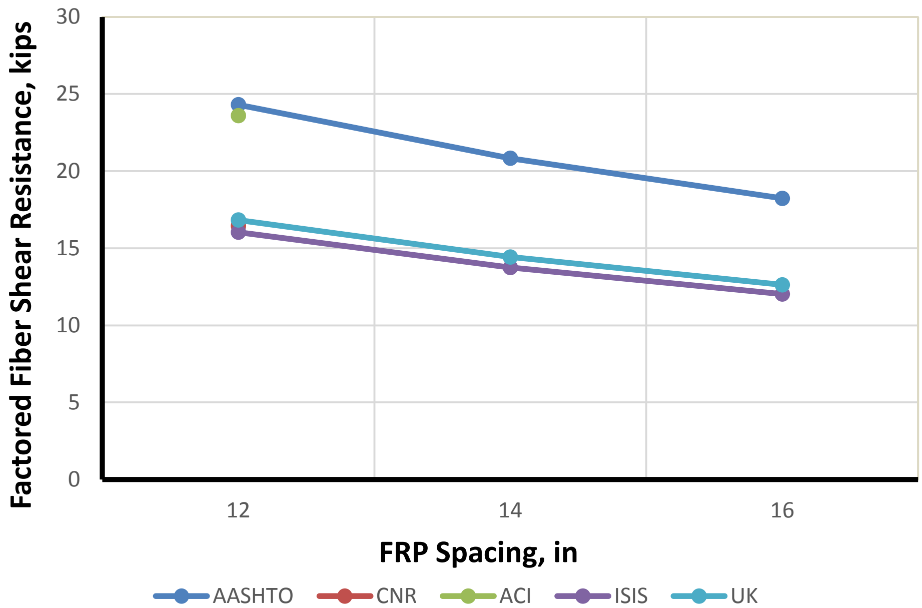

Figure 3.3.18 - Factored fiber shear resistance vs FRP spacing, width = 10 in, U-wrap

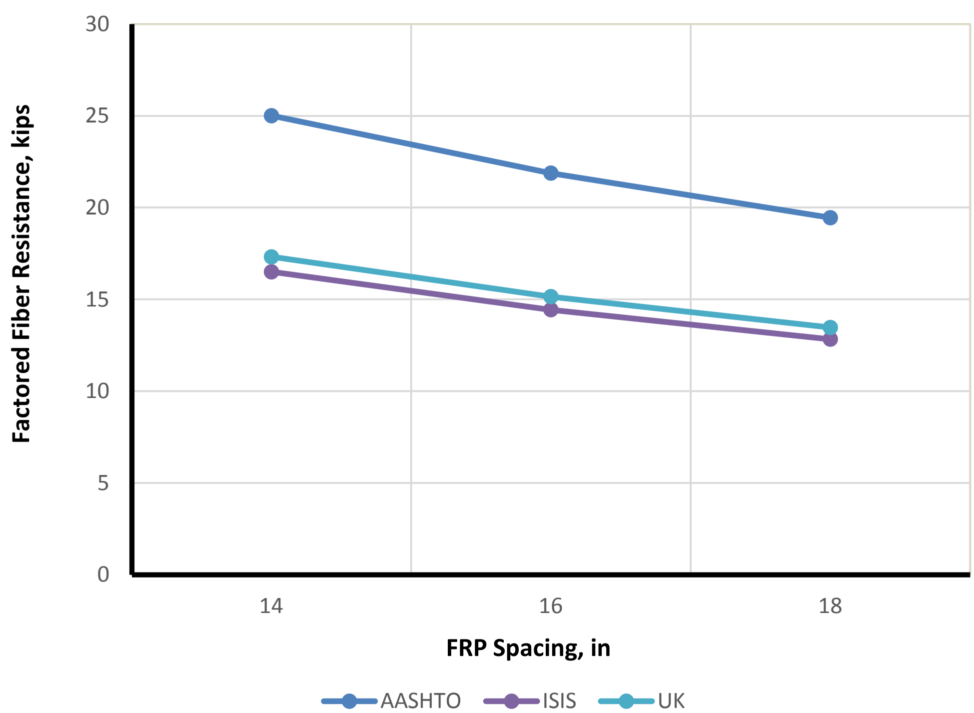

Figure 3.3.19 - Factored fiber shear resistance vs FRP spacing, width = 12 in, U-wrap

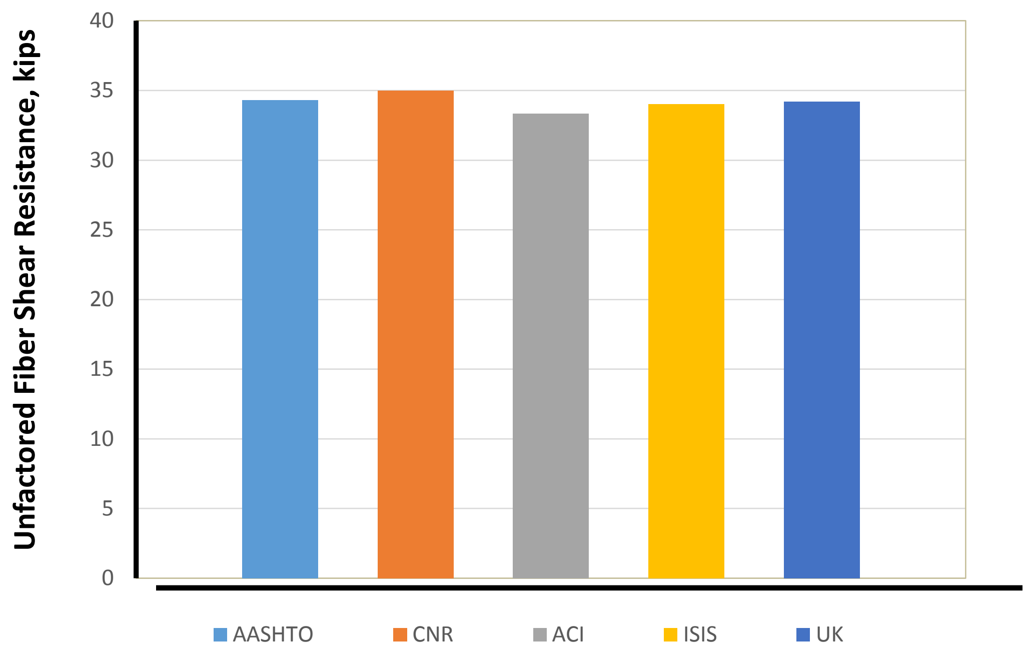

Figure 3.3.20 - Unfactored fiber shear resistance, U-wrap, continuous

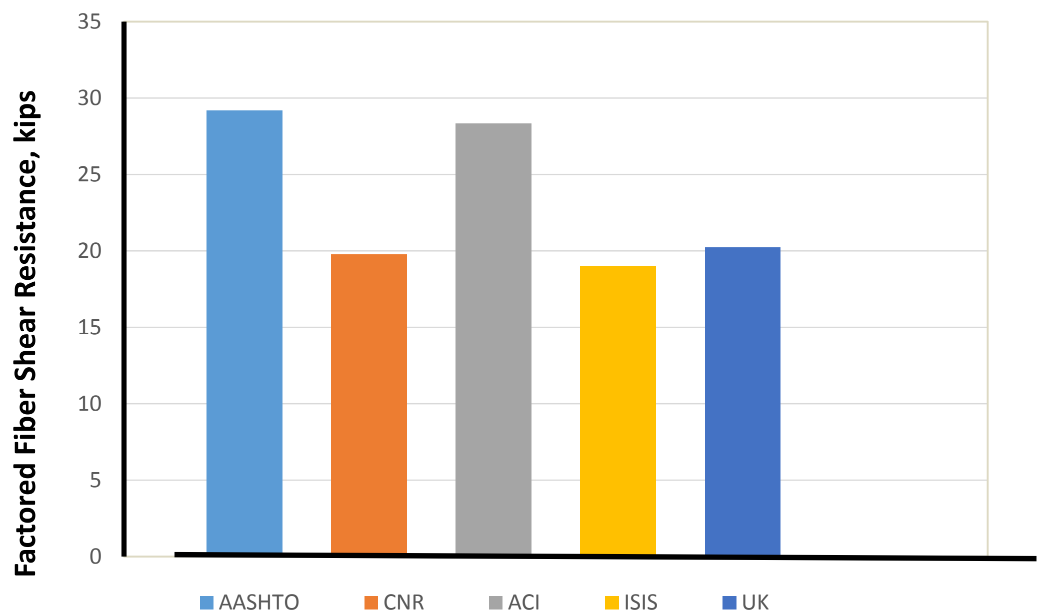

Figure 3.3.21 - Factored fiber shear resistance, U-wrap , continuous

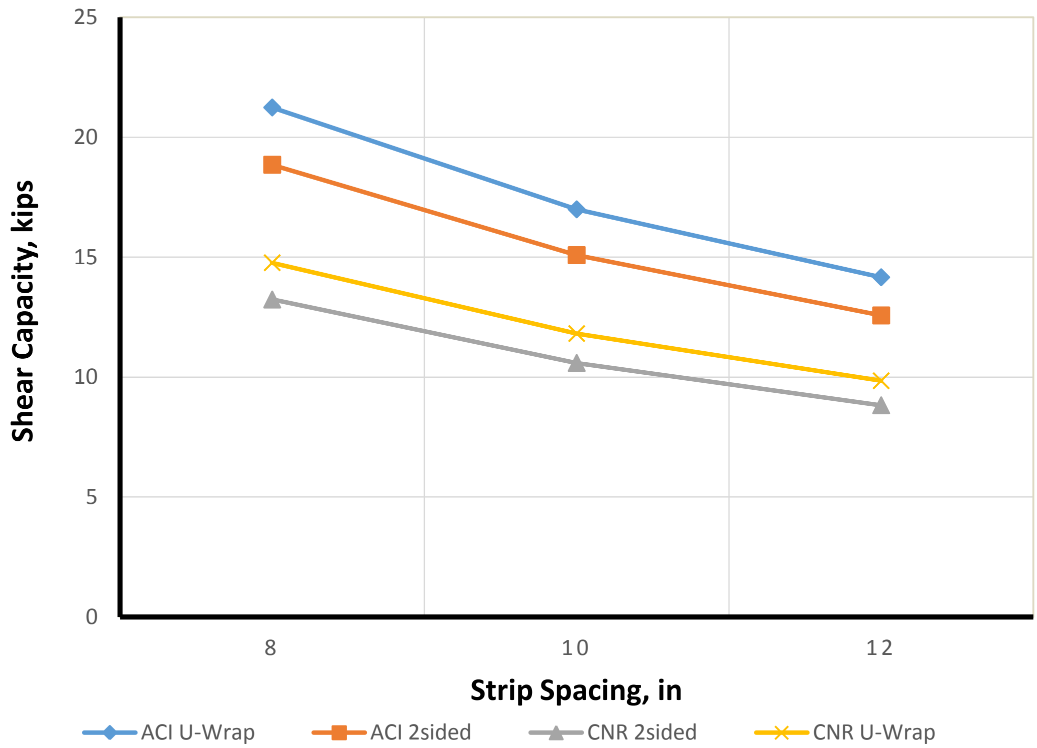

Figure 3.3.22 - Effect of strip spacing on U-wrap and 2 sided wraping, $w_f = 6$ in

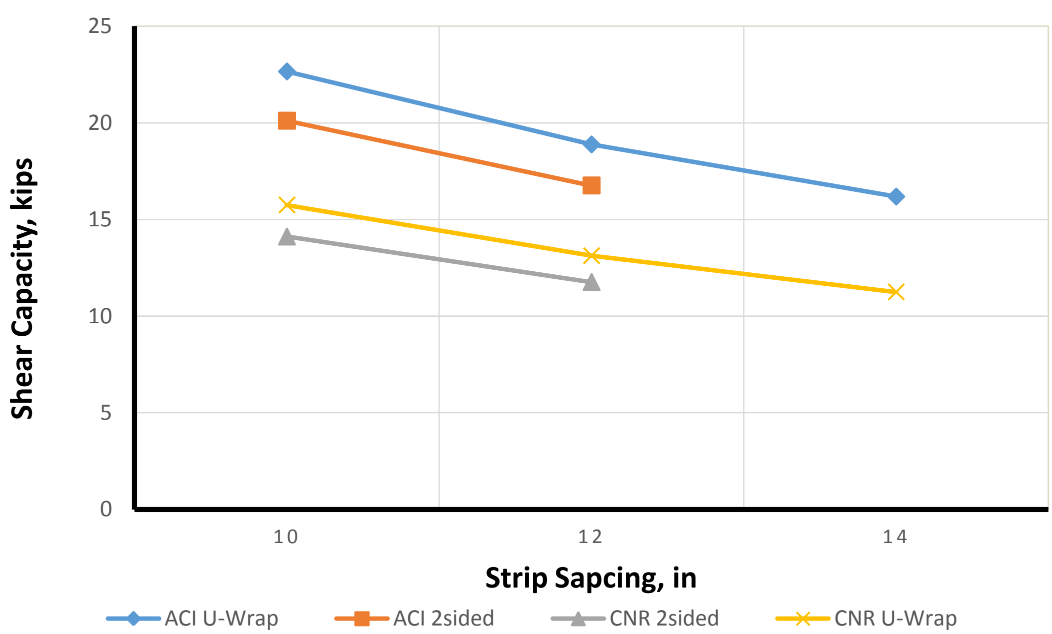

Figure 3.3.23 - Effect of strip spacing on U-wrap and 2 sided wraping, $w_f = 8$ in

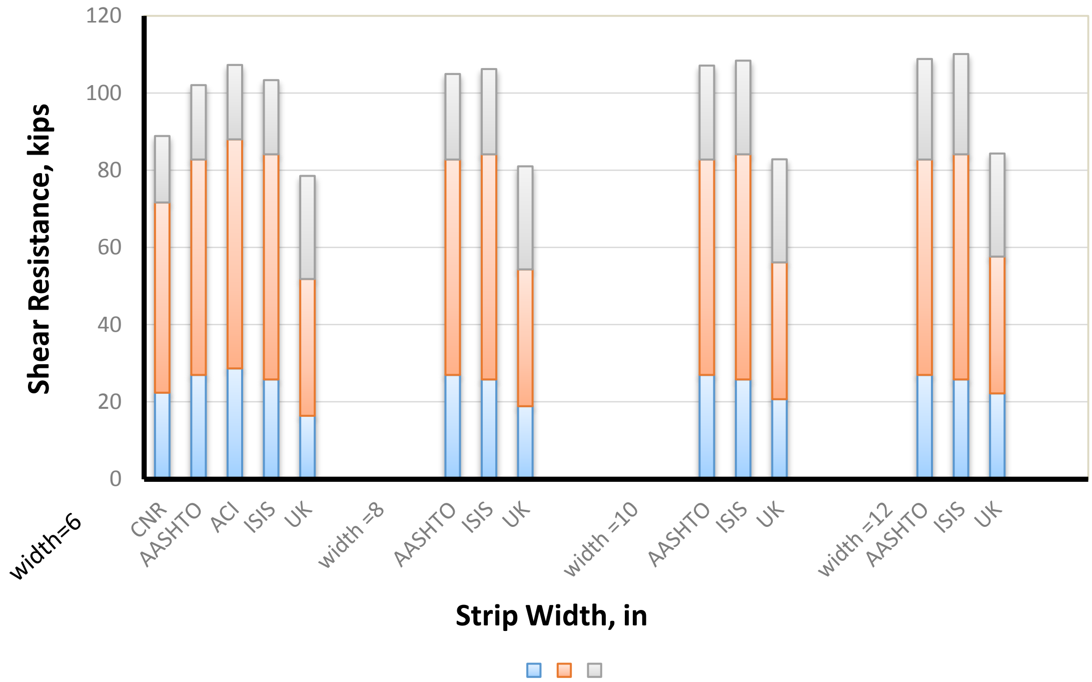

Figure 3.3.24 - Effect of FRP strip width on total shear resistance when gap = d/4

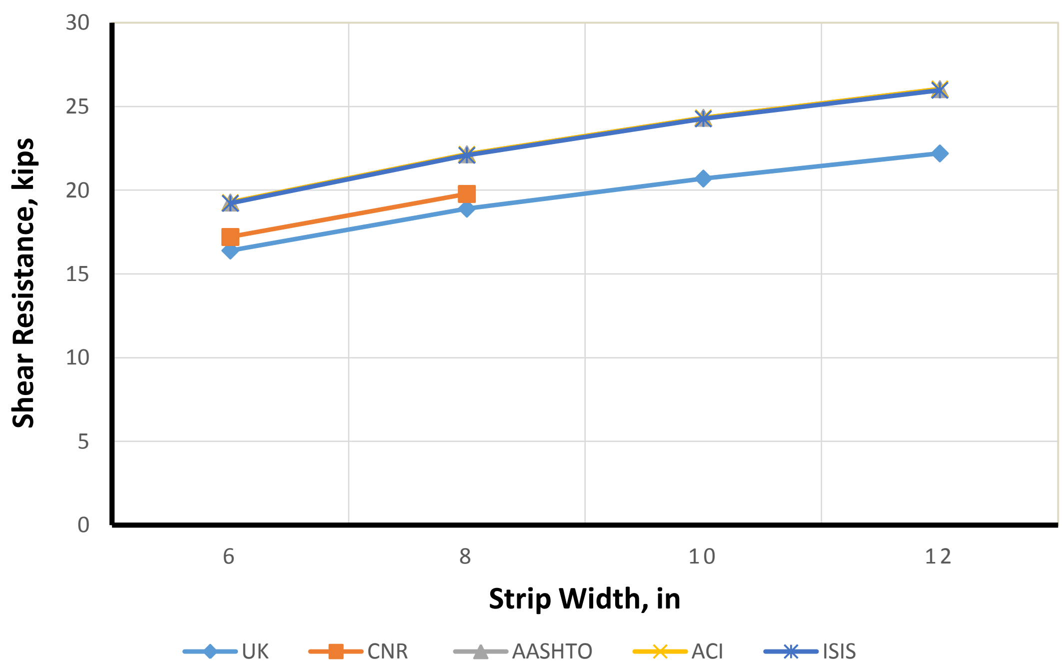

Figure 3.3.25 - Effect of FRP strip width on FRP shear resistance when gap = d/4

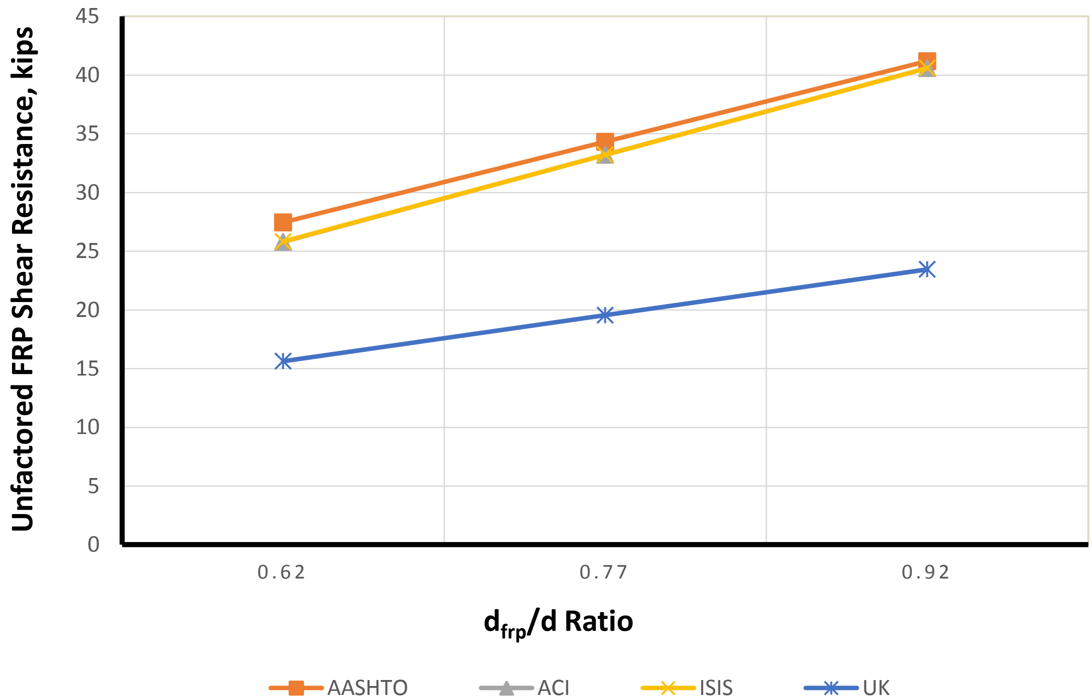

Figure 3.3.26 - Effect of the ratio of d_{frp}/d on Unfactored FRP shear resistance

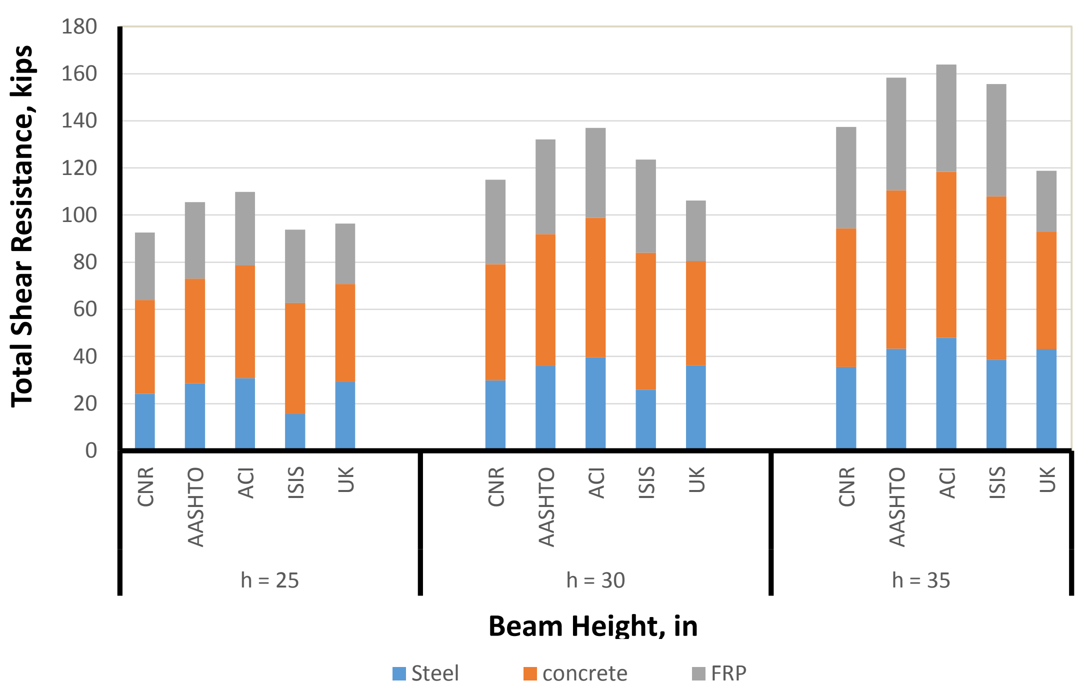

Figure 3.3.27 - Effect of beam height on total shear resistance, steel stirrup spacing = 9 in

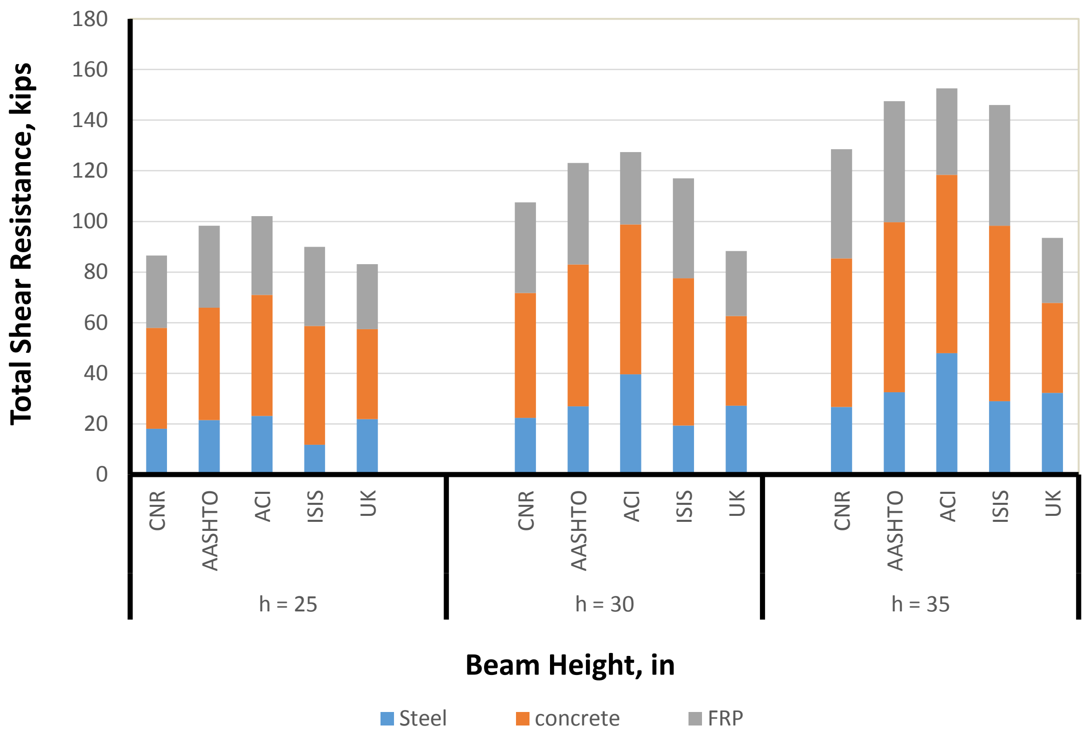

Figure 3.3.28 - Effect of beam height on total shear resistance, steel stirrup spacing = 12 in

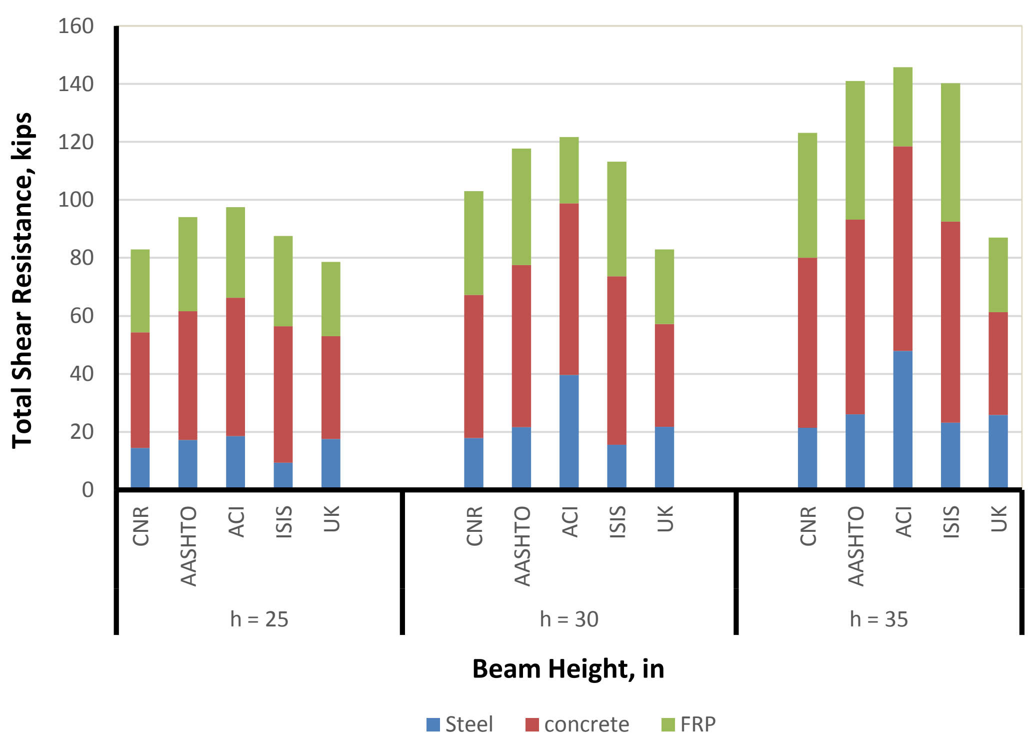

Figure 3.3.29 - Effect of beam height on total shear resistance, steel stirrup spacing = 15 in

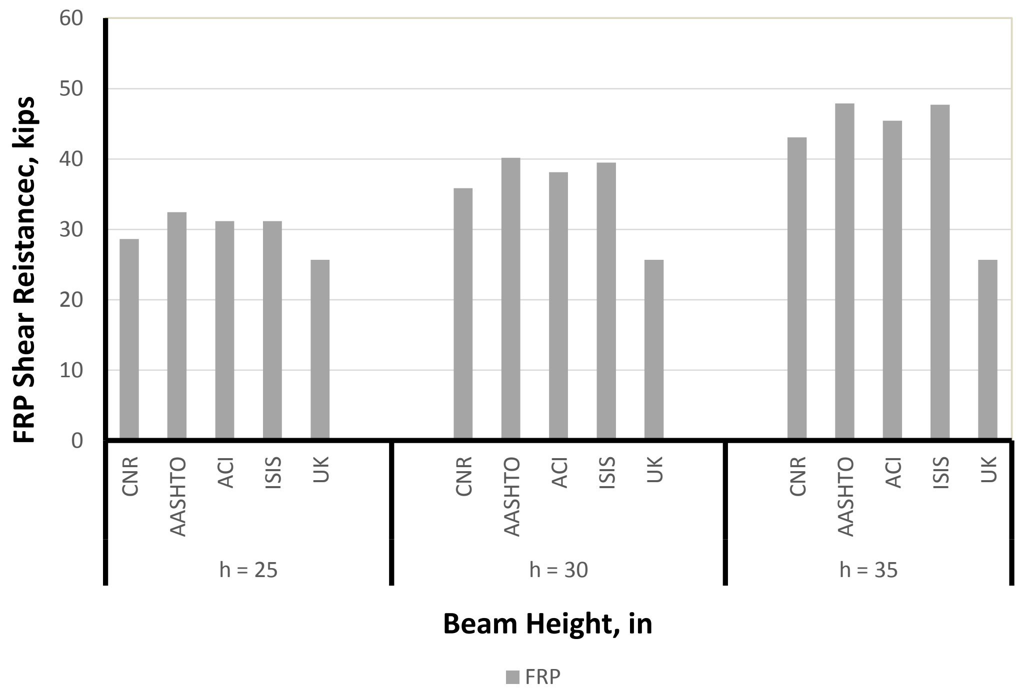

Figure 3.3.30 - Effect of beam height on FRP shear resistance, kips steel stirrup spacing = 9 in

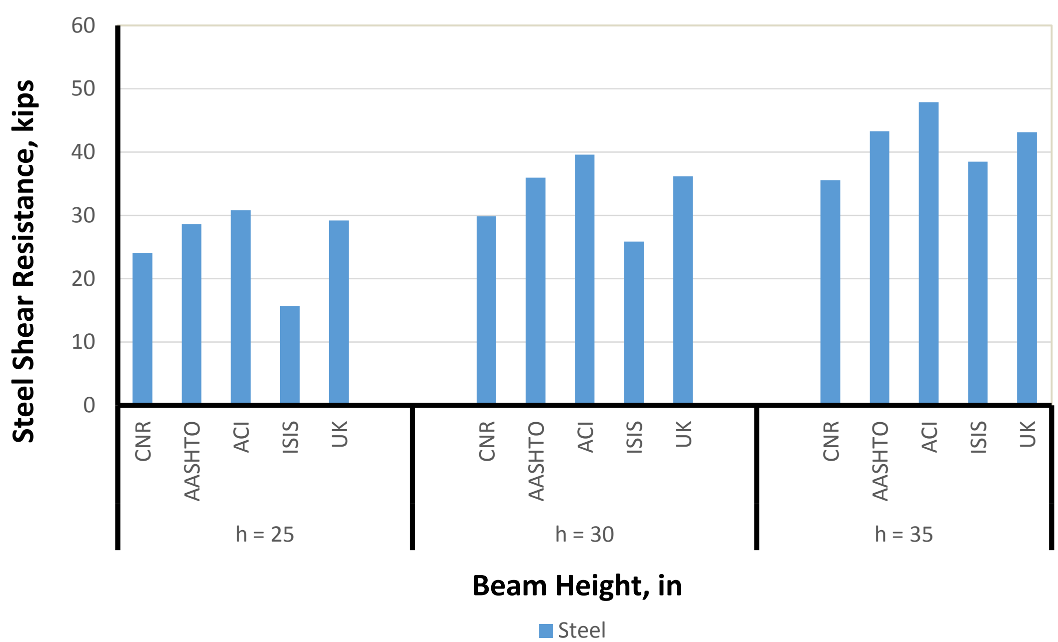

Figure 3.3.31 - Effect of beam height on steel shear resistance steel stirrup spacing = 9 in

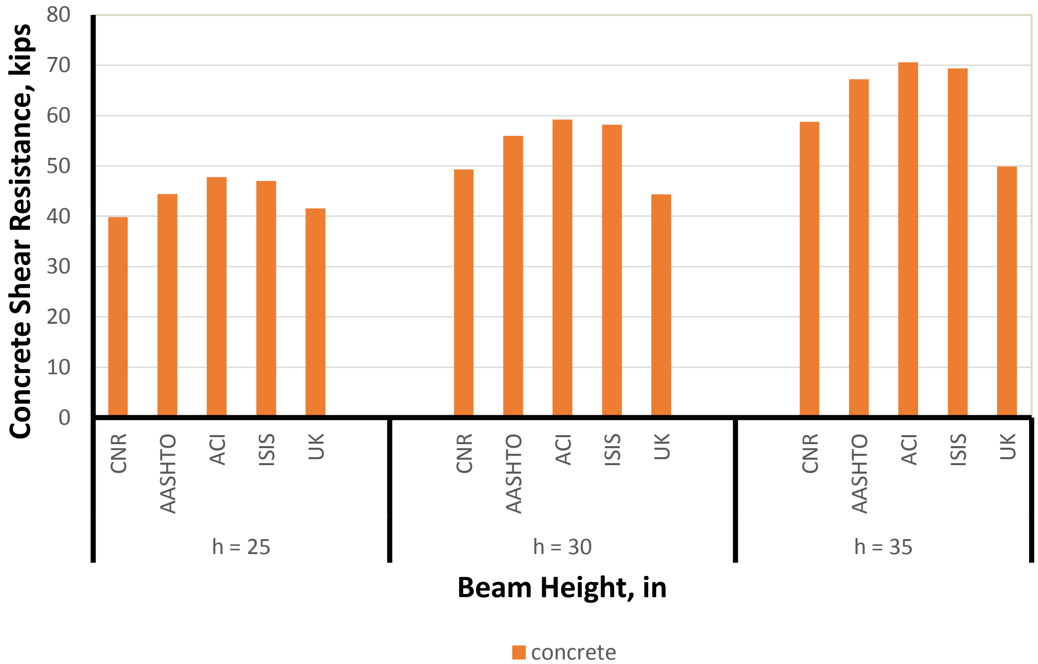

Figure 3.3.32 - Effect of beam height on concrete shear resistance, steel stirrup spacing = 9 in

3.4 FRP Confinement Strengthening of RC/PC Bridge Members

3.4.1 Introduction

As with previous sections, the items considered for review and comparison in this section are based on the organization of ACI 440.2R-08. These items include:

- Design considerations
- Strength reduction factors
- Maximum FRP strain due to confinement
- FRP stress limits
- Design procedures and analysis

3.4.2 Design considerations

3.4.2.1 Strength reduction factors

Table 3.4.1 presents strength reduction factors recommended for use by different codes. Most range from 0.65-0.75, although, as discussed in previous sections, how these factors are applied in design is not consistent among codes.

Table 3.4.1 - Strength reduction factor

Code	Reduction factor	
AASHTO	Reduction factor: Confinement =	0.65
	Resistance factor: Spiral =	0.75
	Ties =	0.65
CNR	Reduction factor for FRP =	0.90
UK	Concrete:	0.67
ISIS	Concrete:	0.75
	Steel:	0.90
	CFRP (embedded in f_l equation):	0.56
ACI	Spiral:	0.75
	Ties:	0.65

3.4.2.2 Maximum FRP strain due to confinement

3.4.2.2.1 CNR

Failure of an RC confined member may occur by fiber rupture. However, beyond a critical value of hoop strain, the concrete loses effective confinement as it expands laterally and axial strength and stiffness are no longer enhanced with the FRP. As a result, according to CNR, failure of the FRP-confined RC member is also reached when the FRP strain reaches a value of 0.4 % or above.

3.4.2.2.2 AASHTO

Similar to CNR, AASHTO specifies a FRP strain limit of 0.004 when used for confinement of axial compression members. For comparison, Table 3.4.2 presents other strain limits depending on the type of stresses applied to the section.

3.4.2.2.3 ACI

ACI introduces the following expression to calculate the maximum confinement strain:

$$\varepsilon_{ccu} = \varepsilon'_c \left(1.50 + 12\kappa_b \frac{f_l}{f'_c}\left(\frac{\varepsilon_{fe}}{\varepsilon'_c}\right)^{0.45}\right) \qquad \text{(3.4.1 – ACI eq. 12-6)}$$

The parameter κ_b accounts for the geometry of the section and can be taken as 1.0 for circular cross sections. To prevent excessive cracking and potential loss of concrete integrity, the strain calculated in Equation 3.4.1 should be limited to the value given in equation 3.4.2 (ACI eq. 12-7):

$$\varepsilon_{ccu} \leq 0.01 \qquad \text{(3.4.2 – ACI eq. 12-7)}$$

3.4.2.2.4 ISIS

As with CNR and AASHTO, ISIS specifies a maximum confinement strain of 0.004 in the Canadian building code.

$$f_{lFRP} = \frac{2t_{FRP}\phi_{FRP}f_{FRPu}}{D_g} \leq \frac{2t_{FRP}E_{FRP}(0.004)}{D_g} \qquad \text{(3.4.3 – ISIS eq. 6-2b)}$$

The bridge code, however, does not include specific limits on FRP hoop strain, but rather imposes a material reduction factor, ϕ_{FRP}.

$$f_{lFRP} = \frac{2t_{FRP}\phi_{FRP}f_{FRPu}}{D_g} \qquad \text{(3.4.4 – ISIS eq. 6-2a)}$$

Taking $\phi_{FRP}f_{FRPu} = E_{FRP}(\phi_{FRP}\varepsilon_{FRPu})$, and considering a typical case of MBrace CF130 FRP with $\varepsilon_{FRPu} = 0.0167$ and $\phi_{FRP} = 0.5625$, the resulting factored strain becomes 0.0094, a value which is close to the ACI limit of 0.01.

3.4.2.2.5 TR55

Note that although TR55 specifies a maximum strain fixed value of 0.004, which matches the AASHTO and CNR values, this is in fact for the case of shear. When axial confinement is considered, however, the hoop strain limit is specified as 0.01. This value corresponds to an effective enhanced cube strength of $1.5f_{cu}$.

3.4.2.2.6 Summary - Maximum FRP strain due to confinement

A summary of the FRP strain limits for confined sections is presented in Table 3.4.2. As shown, most are from 0.003-0.005.

Table 3.4.2 Maximum FRP strain due to confinement

Code	Maximum Strain due to Confinement
AASHTO	axial compression: 0.004 axial tension: 0.005 Max. design (axial tension) completely wrapped: $\varepsilon_{fe} = 0.004 \le 0.75\,\varepsilon_{fu}$ Combined axial compression & bending: 0.003
ACI	$\varepsilon_{ccu} = \varepsilon'_c\left(1.50 + 12k_b \frac{f_l}{f'_c}\left(\frac{\varepsilon_{fe}}{\varepsilon'_c}\right)^{0.45}\right)$ $\varepsilon_{ccu} \le 0.01$
CNR	$\varepsilon_{fd,rid} = \min\{\eta_a.\frac{\varepsilon_{fk}}{\gamma_f}; 0.004\}$
ISIS	Building Code: 0.004 Bridge Code: no limit provided
UK	Confinement: 0.010 Shear: 0.004

3.4.2.3 FRP stress limits

Limits for confinement stress are generally a function of the stress–strain model used by a particular code. AASHTO adopts a bilinear model similar to that of ISIS. AASHTO takes a minimum strain limit of 600 psi to reflect the fact that confinement pressure effectiveness is attained after a certain level of ductility is achieved. Note that AASHTO's maximum confinement stress introduces errors for the case of rectangular columns (D becomes the diagonal length instead of the radius). The errors are tolerated by AASHTO since they are on the conservative side.

ACI and CNR both assume a two-stage stress-strain model; an initial parabolic stage to represent unconfined concrete behavior, followed by a linear stage for confined behavior. The maximum confined stress limits are identical for both codes, while the minimum limits vary slightly.

TR55 discusses several stress-strain behavior models and recommends a model proposed by Lillistone and Jolly (1997). The general expression for FRP confined concrete stress is presented in equation 3.4.3 (TR55 eq. 8.3).

$$f_{cc} = \frac{(0.67/\gamma_{mc})(E_i - E_p)\varepsilon_{cc}}{1+\left\{\frac{\varepsilon_{cc}(E_i - E_p)}{f_o}\right\}} + E_p\varepsilon_{cc} \qquad (3.4.5 \text{ –TR55 eq. 8.3})$$

where:

E_i =Initial tangent modulus of concrete

$$= 21500\,\{\frac{f_{cu}+8}{10}\}^{1/3} \quad \text{(3.4.6 –TR55 eq. 8.4)}$$

E_p =Post-crushing tangent modulus of concrete

$$= 1.282\,\left(\frac{2t_f}{D}\right)E_{fd} \quad \text{(3.4.7 –TR55 eq. 8.5)}$$

$$f_o = f_{cu}(E_i - E_p)/(E_i - E_l) \quad \text{(3.4.8 –TR55 eq. 8.6)}$$

$$E_l = (f_{cu} + 8)/\varepsilon_{cu} \quad \text{(3.4.9 –TR55 eq. 8.7)}$$

To evaluate the maximum confinement pressure, TR55 recommends equation 3.4.10 (TR55 8.16):

$$f_{ccd} = \frac{0.67 f_{cu}}{\gamma_{mc}} + 0.05\,(\frac{2t_f}{D})E_{fd} \quad \text{(3.4.10 –TR55 eq. 8.16)}$$

Table 3.4.3 summarizes minimum and maximum confined stress limits.

Table 3.4.3 *FRP stress limits*

Code	Minimum Confined Stress	Maximum Confined Stress
AASHTO	600 psi	$f_l = \phi_{frp}\frac{2N_{frp}}{D} \leq \frac{f_c'}{2}\left(\frac{1}{k_e\phi} - 1\right)$
CNR	Confinement is effective if: $f_{l,eff}/f_{cd} > 0.05$	$f_{ccd} = f_{cd} + E_t\varepsilon_{ccu}$
ACI	$f_l/f_c' \geq 0.08$	$f_{cc,u}' = f_c' + E_2\varepsilon_{ccu}$
ISIS	$0.1\,f_c'$	$0.33\,f_c'$

3.4.3 Analysis and Design Procedures

3.4.3.1 AASHTO

3.4.3.1.1 Axial capacity of confined columns in compression

The design procedure for columns strengthened with FRP is the same as that for reinforced concrete columns without strengthening. However, the concrete compressive strength f_c' is replaced by the increased confined concrete compressive strength f_{cc}'.

The factored axial load resistance, P_r, for a confined column is taken as:

For members with spiral reinforcement:

$$P_r = 0.85\phi\left[0.85 f_{cc}'\left(A_g - A_{st}\right) + f_y A_{st}\right] \qquad \text{(3.4.11 – AASHTO eq. 5.3.1-1)}$$

For members with tie reinforcement:

$$P_r = 0.80\phi\left[0.85 f_{cc}'\left(A_g - A_{st}\right) + f_y A_{st}\right] \qquad \text{(3.4.12 – AASHTO eq. 5.3.1-2)}$$

where:

ϕ = resistance factor
A_g = gross area of section (in^2)
A_{st} = total area of longitudinal reinforcement, (in^2).
f_y = yield strength of reinforcement (ksi)
f_{cc}' = compressive strength of the confined concrete.

The multipliers 0.85 and 0.80 are intended to account for accidental load eccentricity (0.05h and 0.10h for columns with spiral or tied reinforcement, respectively). Columns with larger eccentricities are to be designed using the provisions of AASHTO section 5.5-Axial Tension.

3.4.3.1.2 Evaluation of confined compressive strength f_{cc}'

The compressive strength of the confined concrete, f_{cc}', is determined from:

$$f_{cc}' = f_c'\left(1 + \frac{2f_l}{f_c'}\right) \qquad \text{(3.4.13 – AASHTO eq. 5.3.2.2-1)}$$

where:

$f_l =$ The confinement pressure due to FRP strengthening for circular columns

$$= \phi_{frp}\frac{2N_{frp}}{D} \leq \frac{f_c'}{2}\left(\frac{1}{k_e\phi} - 1\right) \qquad \text{(3.4.14 – AASHTO eq. 5.3.2.2-2)}$$

k_e = strength reduction factor applied for unexpected eccentricities
= 0.80 for tied columns, and
= 0.85 for spiral columns.
N_{frp} = Strength per width of FRP reinforcement corresponding to a strain of 0.004
ϕ_{frp}= 0.65

For rectangular columns, the diameter *D* is taken as the smaller dimension of the width and depth. When equation 3.4.14 (AASHTO eq. 5.3.2.2-2) is applied to rectangular columns after replacing *D* with the smaller dimension of the rectangular section, the factored axial strength estimated from equation 3.4.11 or 3.4.12 (AASHTO 5.3.1-1 or 5.3.1-2) is conservative. The calculated gain in strength provided by the confinement of a rectangular section is very little compared to that attainable for circular section. As a result, neither minimum nor maximum limits are specified for rectangular sections, since the attainable confinement pressure, which relies on ductility development, is very limited for rectangular columns.

3.4.3.2 ACI

3.4.3.2.1 Axial capacity of confined columns in compression

ACI provides an expression similar to AASHTO to evaluate the axial capacity of confined columns. The confined stress-strain behavior model adopted by ACI is based on the stress-strain model developed by Lam and Teng (2003). The expressions used to evaluate axial design capacity for spiral and tied columns are provided in Equations 3.4.15a and 3.4.15b (ACI 12-1a & 12-1b). These equations are follow ACI 318 column design equations, but concrete compressive strength f_c' is replaced with the confined compressive strength f_{cc}'.

For nonprestressed columns with steel tie reinforcement:

$$\phi P_n = 0.85\phi\left[0.85 f_{cc}'\left(A_g - A_{st}\right) + f_y A_{st}\right] \qquad \text{(3.4.15a – ACI eq. 12-1a)}$$

For nonprestressed with tie reinforcement:

$$\phi P_n = 0.80\phi\left[0.85 f_{cc}'\left(A_g - A_{st}\right) + f_y A_{st}\right] \qquad \text{(3.4.15b – ACI eq. 12-1b)}$$

3.4.3.2.2 Stress –strain model for confined reinforced concrete columns

The Lam and Teng (2003) model adopted by ACI is shown in Figure 3.4.1. Equation 3.4.16a provides the general expression for concrete stress for the nonlinear/unconfined portion as well as the linear/confined portion. Equation 3.4.16b evaluates the slope for the linear portion of the stress-strain model, E_2. Equation 3.4.16c evaluates the transition strain between the nonlinear and linear portions of the stress-strain curve. Equations 3.4.1 and 3.4.2 are discussed in section 3.4.2.2.3.

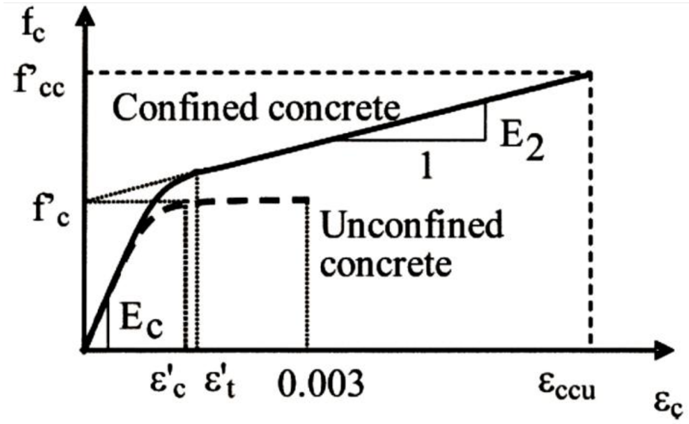

Figure 3.4.1 – ACI stress-strain model for FRP-confined concrete (Lam and Teng, 2003a)

$$f_c = \begin{cases} E_c\varepsilon_c - \frac{(E_c-E_2)^2}{4f'_c}{\varepsilon_c}^2 & 0 \le \varepsilon_c \le {\varepsilon'}_t \\ f'_c + E_2\varepsilon_c & {\varepsilon'}_t \le \varepsilon_c \le \varepsilon_{ccu} \end{cases} \quad \text{(3.4.16a – ACI eq. 12-2a)}$$

where:

$E_2 =$ The slope of the linear portion of the stress-strain model for FRP confined concrete, psi.

$$= \frac{f'_{cc}-f'_c}{\varepsilon_{ccu}} \quad \text{(3.4.16b – ACI eq. 12-2b)}$$

$$\varepsilon'_t = \frac{2f'_c}{E_c-E_2} \quad \text{(3.4.16c – ACI eq. 12-2c)}$$

$$\varepsilon_{ccu} = \varepsilon'_c\left(1.50 + 12\kappa_b\frac{f_l}{f'_c}\left(\frac{\varepsilon_{ccu}}{\varepsilon_{ccu}}\right)^{0.45}\right) \quad \text{(3.4.1 – ACI eq. 12-6)}$$

$$\varepsilon_{ccu} \le 0.01 \quad \text{(3.4.2 – ACI eq. 12-7)}$$

3.4.3.2.3 Evaluation of confined compressive strength f'_{cc}

$$f'_{cc} = f'_c + \psi_f 3.3\kappa_a f_l \quad \text{(3.4.17 – ACI eq. 12-3)}$$

$$f_l = \frac{2E_f n t_f \varepsilon_{fe}}{D} \quad \text{(3.4.18 – ACI eq. 12-4)}$$

$$\varepsilon_{fe} = \kappa_\varepsilon\varepsilon_{fu} \quad \text{(3.4.19 – ACI eq. 12-5)}$$

κ_ε= Strain efficiency factor (accounts for the possibility of premature failure of FRP)
= 0.586 for CFRP

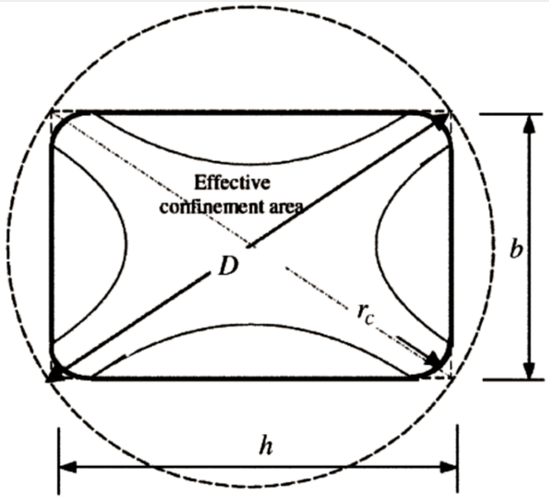

Figure 3.4.2 – Equivalent circular cross section (Lam and Teng 2003)

D is the diameter of a circular column. For noncircular cross sections, an equivalent circular cross section with diameter *D* equal to the diagonal of the rectangular cross section is used. The equivalent *D* value for a rectangular column cross section is presented in equation 3.4.20 (ACI 12-8):

$$D = \sqrt{b^2 + h^2} \qquad \text{(3.4.20 – ACI eq. 12-8)}$$

For circular cross sections, the shape factors κ_a and κ_b appearing in Equations 3.4.1 and 3.4.17 above can be taken as 1.0. For rectangular cross sections, κ_a and $\kappa_b$are expressed in Equation 3.4.21 (ACI eq. 12-9) and 3.4.22 (ACI eq. 12-10) respectively. Their values depend on two parameters: the side-aspect ratio *h*/*b* (see Figure 3.4.2), and the ratio of the effective area of the confined concrete A_e to the total area of the concrete section A_c. The ratio A_e/A_c is evaluated in Equation 3.4.23 (ACI eq. 12.11.

$$\kappa_a = \frac{A_e}{A_c}\left(\frac{b}{h}\right)^2 \qquad \text{(3.4.21 – ACI eq. 12-9)}$$

$$\kappa_b = \frac{A_e}{A_c}\left(\frac{b}{h}\right)^{0.5} \qquad \text{(3.4.22 – ACI eq. 12-10)}$$

$$\frac{A_e}{A_c} = \frac{1 - \frac{\left[\left(\frac{h}{b}\right)(h-2r_c)^2 + \left(\frac{h}{b}\right)(h-2r_c)^2\right]}{3A_g} - \rho_g}{1-\rho_g} \qquad \text{(3.4.23 – ACI eq. 12-11)}$$

3.4.3.2.4 Serviceability considerations

ACI stipulates that the transverse strain in the concrete should remain below its cracking strain at service load levels, which is to be taken as an equivalent concrete compressive stress limit of 0.65f_c'. ACI also states that the service stress in the longitudinal steel should remain below 0.60f_y to avoid plastic deformation under sustained or cyclic loads.

3.4.3.3 ISIS

3.4.3.3.1 Axial capacity of confined columns in compression

The analysis considered in this document is based on the S6-06 bridge code equations and procedure. The factored axial resistance of a section in compression, P_r, is given by Equation 3.4.24 (ISIS eq. 6.5a):

$$P_r = 0.80\left[\alpha_1 \phi_c f'_{cc}\left(A_g - A_s\right) + \phi_s f_y A_{st}\right] \qquad \text{(3.4.24 - ISIS eq. 6-5a)}$$

where:

ϕ_c = 0.75
ϕ_s = 0.9
α_1 = 0.85 – 0.0015 $f'_c \geq$ 0.67 (3.4.25 – ISIS eq. 5-1)

3.4.3.3.2 Evaluation of the confined compressive strength f'_{cc}

The model used in S6-06 to evaluate the confined compressive strength f'_{cc} is given by equation 3.4.26 (ISIS eq. 6-3). This equation provides a reasonably conservative estimate of f'_{cc}, based on the research by Thériault and Neale (2000); Teng et al., (2002), and Bisby et al. (2005).

$$f'_{cc} = f'_c + 2f_{lFRP} \qquad \text{(3.4.26 – ISIS eq. 6-3)}$$

where:

$f'_c \leq 7.25\ ksi\ (50\ MPa)$

$f_{lFRP} =$ Confinement pressure due to FRP strengthening at ultimate capacity (MPa)

$$= \frac{2t_{FRP}\phi_{FRP}f_{FRPu}}{D_g} \qquad \text{(3.4.27 – ISIS eq. 6-2a)}$$

D_g = Diameter of a circular column, or diagonal of a rectangular column provided that the section edges are rounded, $h/b \leq$ 1.5, (see figure 3.4.2), and $b \leq$ 31.5 in (800mm).

3.4.3.4 CNR

3.4.3.4.1 Axial capacity of FRP-confined members under concentric or slightly eccentric force

Confinement action on reinforced concrete columns becomes significant only after cracking of the concrete and yielding of the steel reinforcement when increased lateral expansion occurs; prior to concrete cracking, the FRP is practically unloaded.

Ultimate strength design requires that both the factored design axial load, N_{Sd}, and the factored axial capacity, $N_{Rcc,d}$, satisfy the following equation:

$$N_{Sd} \leq N_{Rcc,d} \qquad \text{(3.4.28 – NCR eq. 4.39)}$$

For non-slender FRP confined members, the factored axial capacity can be calculated as follows:

$$N_{Rcc,d} = \frac{1}{\gamma_{Rd}} A_c f_{ccd} + A_s f_{yd} \qquad \text{(3.4.29 – NCR eq. 4.40)}$$

where:

$\gamma_{Rd} =$ partial factor taken equal to 1.10
A_c = member cross-sectional area
f_{ccd} = design strength of confined concrete
A_s = area of existing steel reinforcement
$f_{yd} =$ yield strength of existing steel reinforcement

3.4.3.4.2 Evaluation of the confined compressive strength f_{ccd}

The design strength, f_{ccd} , of the confined concrete is evaluated as follows:

$$\frac{f_{ccd}}{f_{cd}} = 1 + 2.6\left(\frac{f_{1,eff}}{f_{cd}}\right)^{2/3} \qquad \text{(3.4.30 – CNR eq. 4.41)}$$

where:

$f_{cd} =$ the design strength of unconfined concrete
$f_{1,eff} =$ the effective confined lateral pressure. For effective confinement, use $\left(\frac{f_{1,eff}}{f_{cd}}\right) > 0.05$.

3.4.3.4.3 Evaluation of the confined lateral pressure

The effective confined lateral pressure, $f_{1,eff}$, is a function of the member cross section and FRP configuration, as indicated in the following equation:

$$f_{1,eff} = k_{eff} \cdot f_1 \qquad \text{(3.4.31 – CNR eq. 4.42)}$$

where:

$k_{eff} =$ A coefficient of efficiency (≤1), defined as the ratio of the volume, $V_{c,eff}$, of the effectively confined concrete and the volume, V_c , of the concrete member (neglecting the area of existing internal steel reinforcement).

f_1 = The confined lateral pressure.

$= \frac{1}{2} . \rho_f . E_f . \varepsilon_{fd,rid}$ (3.4.32 – CNR eq. 4.43)

ρ_f = The geometric strengthening ratio as a function of section shape (circular or rectangular) and FRP configuration (continuous or discontinuous wrapping).

E_f = Young's modulus of the FRP in the direction of fibers.

$\varepsilon_{fd,rid}$ = A reduced FRP design strain.

$= \min \{\eta_a * \frac{\varepsilon_{fk}}{\gamma_f}; 0.004\}$ (3.4.33 – CNR eq. 4.47)

η_a = Environmental conversion factor (CNR Table 3-4)

γ_f = Partial factor (CNR Table 3-2)

$k_{eff} = k_H . k_V . k_a$ (3.4.34 – CNR eq. 4.44)

3.4.3.4.4 Evaluation of k_H, k_V, and k_a

The coefficient of horizontal efficiency, k_H , depends on the cross-section shape. The coefficient of vertical efficiency, k_V, depends on the FRP configuration. Regardless of the section shape, the efficiency coefficient, k_a, is used when fibers are spirally installed with an angle α_f with respect to the member cross-section. k_a is evaluated using equation 3.4.35 (CNR eq. 4.46).

$k_a = \frac{1}{1+(tan\, \alpha_f)^2}$ (3.3.35 – CNR eq. 4.46)

For reinforced concrete columns confined using continuous wrapping, $k_V = 1.0$. For the case of discontinuous FRP wrapping, FRP strips are installed with a center-to-center spacing of p_f, and clear spacing of p'_f (see Figure 3.4.3). A reduction in confinement effectiveness occurs due to the diffusion of stresses (approximately at 45°) between two subsequent wrappings. The reduction is independent of column cross section shape. The coefficient of vertical efficiency, k_V, can be evaluated using the following expression:

$k_V = \left(1 - \frac{p'_f}{2 * d_{min}}\right)^2$ (3.3.36 – CNR eq. 4.45)

where:

d_{min} = the minimum cross sectional dimension of the member
p'_f $\leq (d_{min}/2)$ for discontinuous wrapping

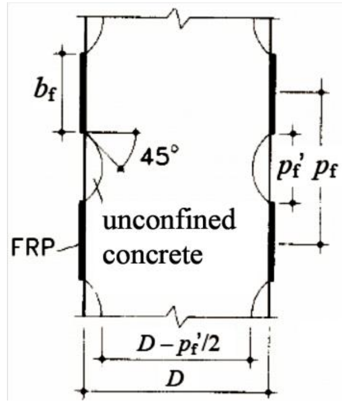

Figure 3.4.3 – Elevation view of circular member confined with FRP strips (CNR)

For circular cross sections subjected to either concentric or slightly eccentric axial load, confinement is most effective and k_H = 1.0. For members with square or rectangular cross sections, FRP-confinement produces only marginal increases in the member's compressive strength, and CNR specifies additional special limitations in this case; for example, the strengthening effect of FRP confinement is neglected for rectangular cross sections having *b/d* > 2, or max$\{b, d\}$> 35.4 in (900 mm) unless otherwise demonstrated in experimental tests.

In CNR, similar to other codes, for rectangular cross sections, the effectively confined concrete area is taken as a fraction of the overall concrete cross section, due to the "arch effect" as shown in Figure 3.4.4. Such an effect depends on the corner radius r_c. CNR recommends the following minimum limit to r_c :

$r_c \geq 0.79\ in\ (20mm)$ (3.3.37 – CNR eq. 4.49)

For rectangular cross sections, the coefficient of horizontal efficiency, k_H, accounts for the arch effect and is evaluated as follows:

$k_H = 1 - \frac{b'^2 + d'^2}{3A_g}$ (3.3.38 – CNR eq. 4.51)

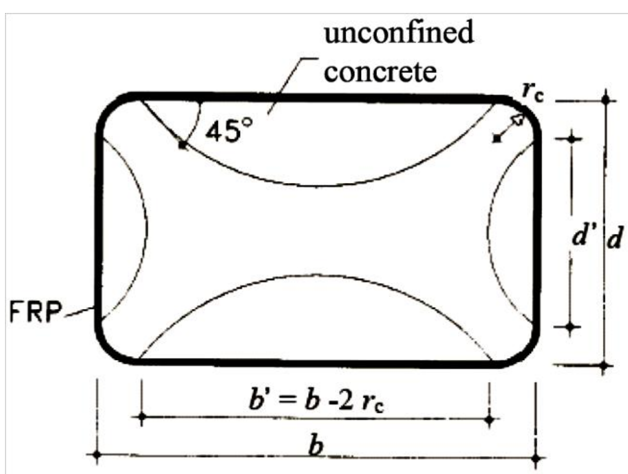

Figure 3.4.4 – Confinement of rectangular sections (CNR)

3.4.4 Summary

Code results are compared in Figures 3.4.5 - 3.4.34. Results in the figures are calculated with the following assumptions: a circular cross section is considered with a diameter of 26 inches and a square cross section with side lengths of 23 in (both columns have the same area of 530 in^2). When the effects of f_c' are evaluated, FRP wrapping is taken as 3 plies with a total thickness of 0.02 in (0.51 mm). Here, BASF MBrace CF130 CFRP is considered. When the amount of CFRP is varied from 1, 2, 3, 4, and 5 plies, f_c' is taken as 4 ksi, and the corresponding CFRP thicknesses are 0.0065, 0.013, 0.0195, 0.026, and 0.033 respectively (0.17, 0.33, 0.50, 0.66, 0.83mm). Moreover, for sake of comparison, for AASHTO results, the minimum confinement pressure limit is ignored. Figures 4.5-4.10 present the effect of changing f_c' on the axial capacity of a square column with different reinforcement ratios (0.02, 0.03, and 0.04). Factored (including applicable reduction factors) and unfactored (excluding applicable reduction factors from calculations) axial loads are evaluated. Also note that for TR55, reduction factors are embedded and only the partial safety factor for concrete could be removed for the unfactored cases.

It was found that the unfactored results are very similar among codes, and as expected, when the effect of various reduction factors are included, greater differences emerge. Here, CNR is least conservative while the other codes show, in general, closer agreement, with AASHTO and TR55 (as expected, as some implicit reduction factors are present, as noted above) most conservative. It was also found that axial capacity is more sensitive to reinforcement ratio ρ at lower values of f_c' than higher values; at a compressive strength of 8 ksi, no increase in axial load capacity is observed with an increase of ρ from 0.02 to 0.04.

In Figures 3.4.11-3.4.16, the effect of changing f_c' on circular column capacity with different reinforcement ratios is examined. As shown, although trends are similar, the circular column generally has greater capacity then the corresponding square column of the same area. Considering the unfactored case, ACI generally has the highest capacity. However, for the factored cases, CNR produces the greatest capacity.

In Figures 3.4.17-3.4.22, the effect of number of plies on the axial capacity of square columns is shown. In these figures, it can be seen that, for the factored case, CNR has the highest capacity while ACI is most conservative. However, the codes converge to similar values for the unfactored case. Figures 3.4.23- 3.4.28 illustrate the effect of changing the number of plies on the axial load capacity of circular columns. For circular columns, axial capacity was more sensitive to f_c' then the amount of FRP confinement. For the circular column, AASHTO, ISIS and TR55 are most conservative (as compared to ACI for square columns), while CNR is least conservative in both cases. Figures 3.4.29 and 3.4.30 present the effect of number of plies on axial capacity. It can be seen that increasing the number of layers has little effect on improving column axial capacity for both circular and square columns, and AASHTO and ISIS (both bridge codes) produce similar results that are slightly more conservative as compared to ACI and CNR.

Figures 3.4.31 and 3.4.32 show the effect of changing f_c' on circular and square column capacity, while Figure 3.4.33 compares circular and square column results. As shown earlier, increasing f'_c clearly increases axial capacity, while the bridge codes (AASHTO and ISIS) are slightly conservative in predicting the axial capacity of circular columns, and TR55 is most conservative

for square columns. Due to the reduction of effective area of confinement, square columns clearly have lower axial capacity than a corresponding circular column. It is also clear that ACI and CNR produce similar results for circular columns. Figure 3.4.34 shows the effect of changing the number of plies on axial load capacity, for both circular and rectangular columns. ACI and CNR provide higher capacities for circular columns while for square columns, ACI is conservative. For both circular and square columns, AASHTO and ISIS produce similar results.
In summary, the following general observations can be made. The different codes produce more consistent values of capacity when f_c' is varied than when ply number is varied. This is because changing the number of plies results in different FRP stress and strain limits imposed by different codes, which becomes the underlying cause of lack of agreement. Only a small change in axial capacity occurred when ply number changed from 1 to 5, while similarly changing f_c' resulted in large differences in capacity. Overall, CNR is least conservative for most cases, while AASHTO and ISIS produce close results for most cases.

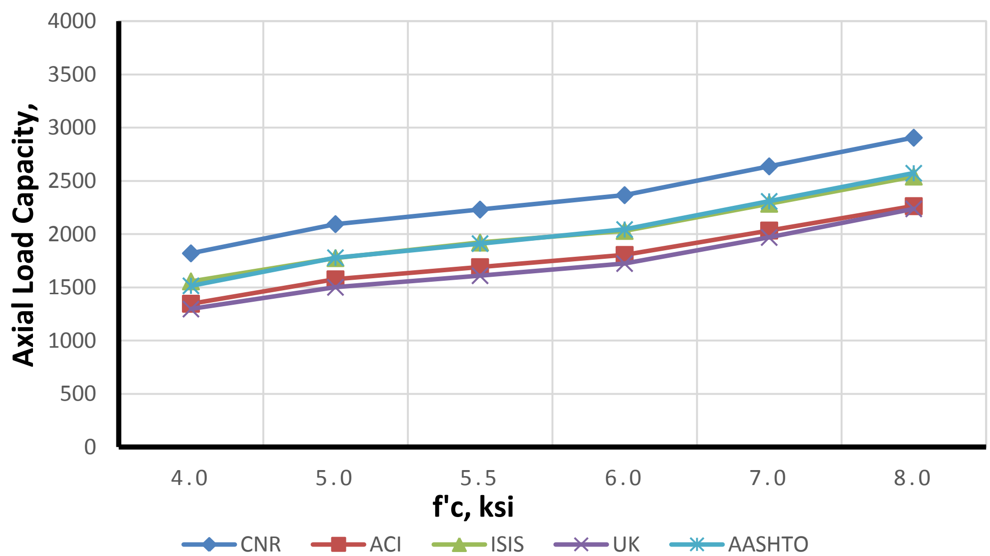

Figure 3.4.5 - Effect of changing f_c' on factored axial load resistance, square column, $\rho = 0.02$

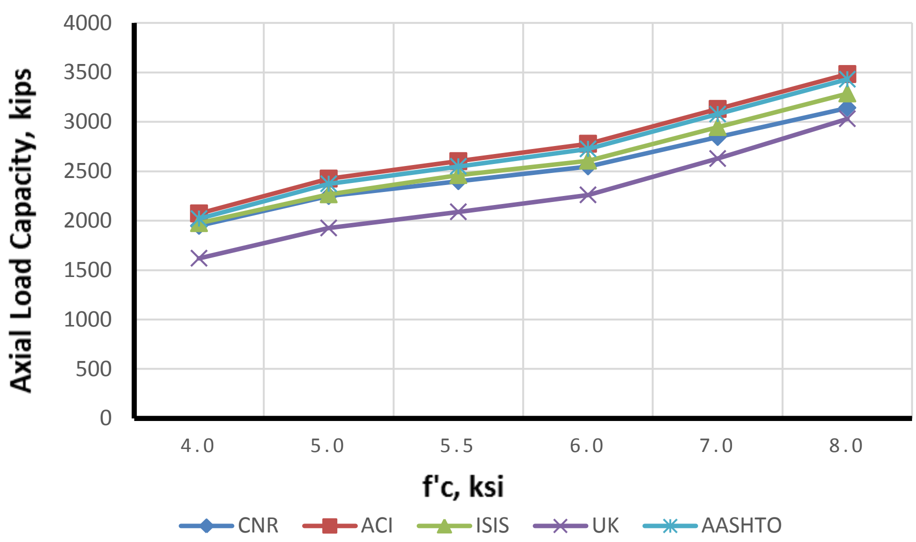

Figure 3.4.6 - Effect of changing f_c' on unfactored axial load resistance, square column, $\rho = 0.02$

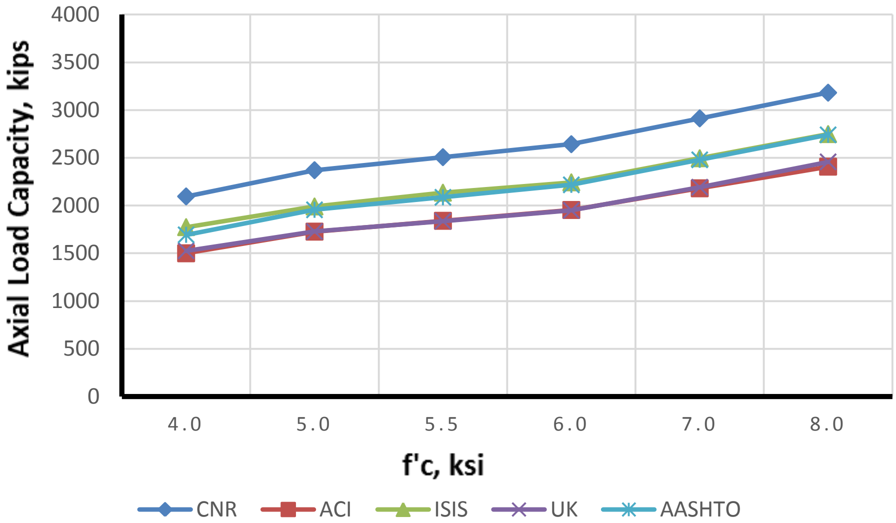

Figure 3.4.7 - Effect of changing f_c' on factored axial load resistance, square column,

$\rho = 0.03$

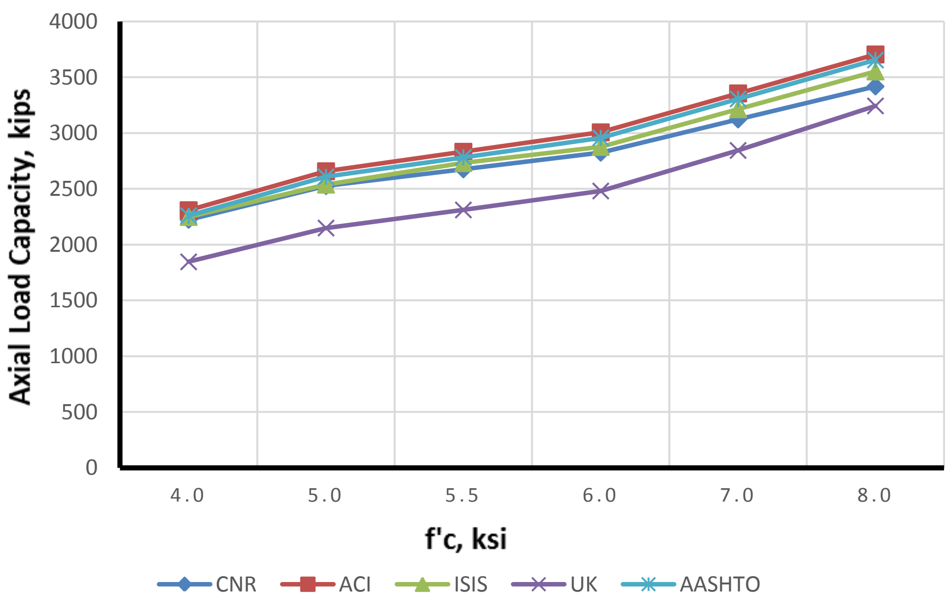

Figure 3.4.8 - Effect of changing f'_c on unfactored axial load resistance, square column,

$\rho = 0.03$

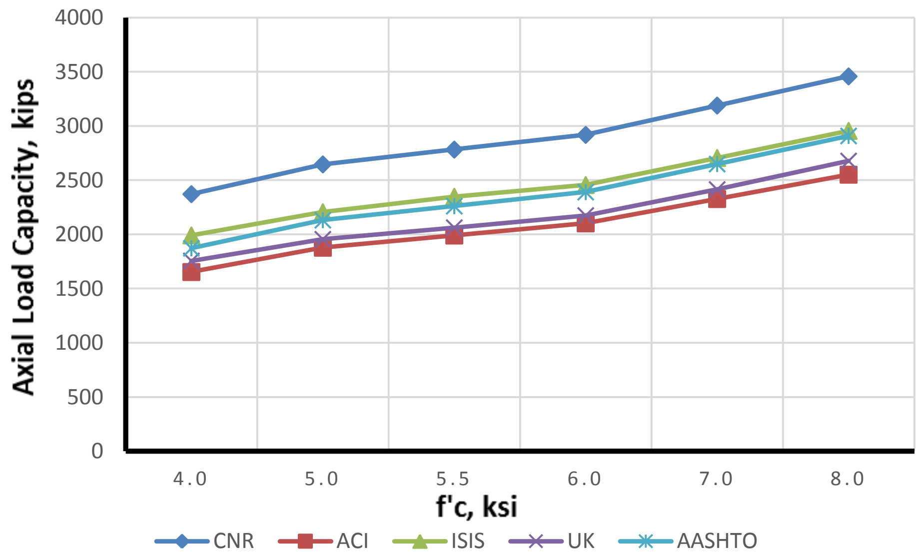

Figure 3.4.9 - Effect of changing f'$_c$ on factored axial load resistance, square column,

$\rho = 0.04$

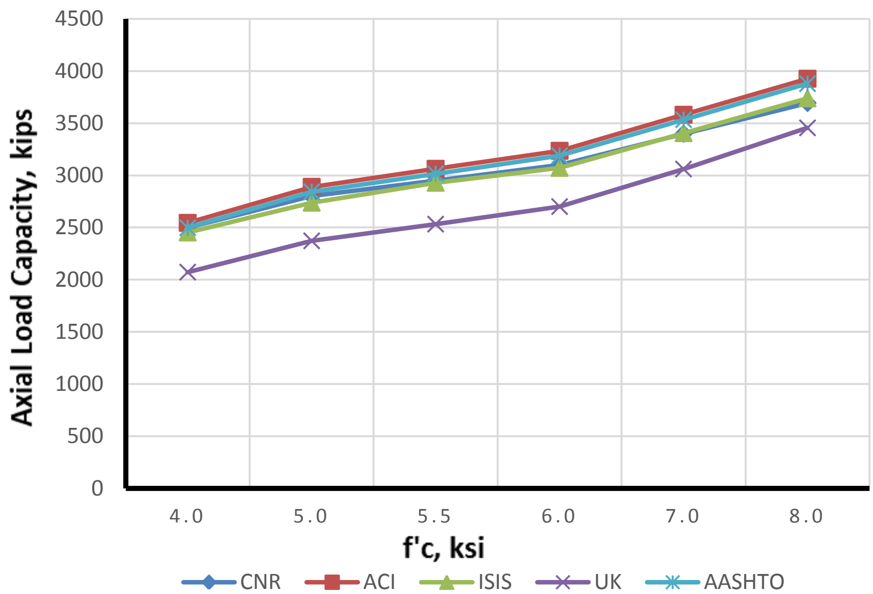

Figure 3.4.10 - Effect of changing f'$_c$ on unfactored axial load resistance, square column,

$\rho = 0.04$

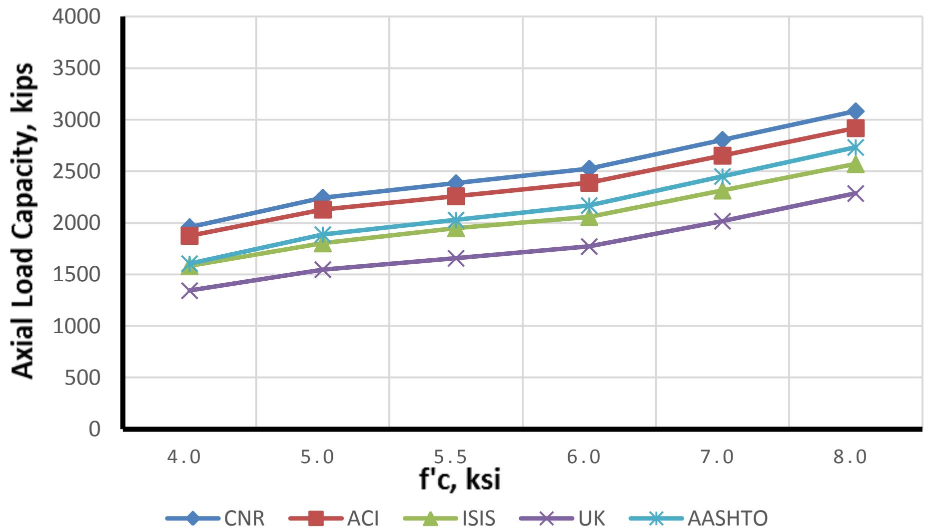

Figure 3.4.11 - Effect of changing f'_c on factored axial load resistance, circular column,

$\rho = 0.02$

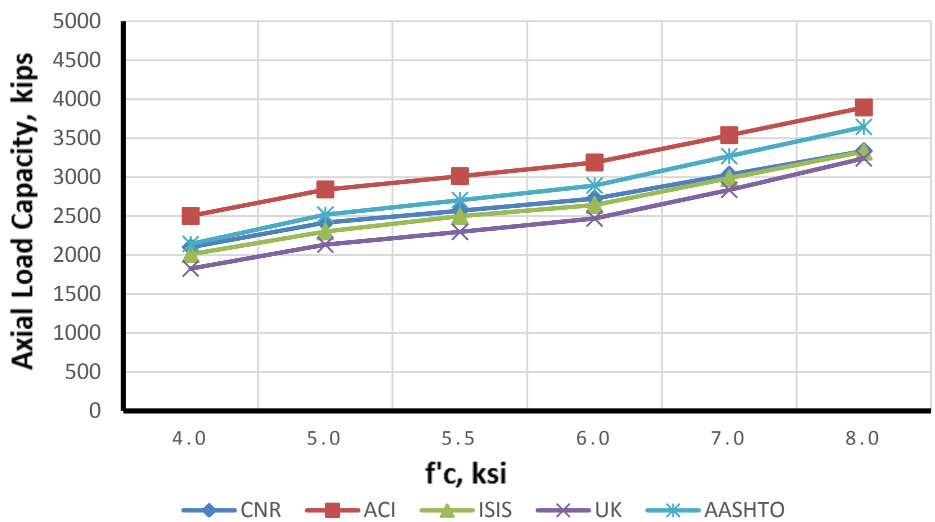

Figure 3.4.12 - Effect of changing f_c' on unfactored axial load resistance, circular column,

$\rho = 0.02$

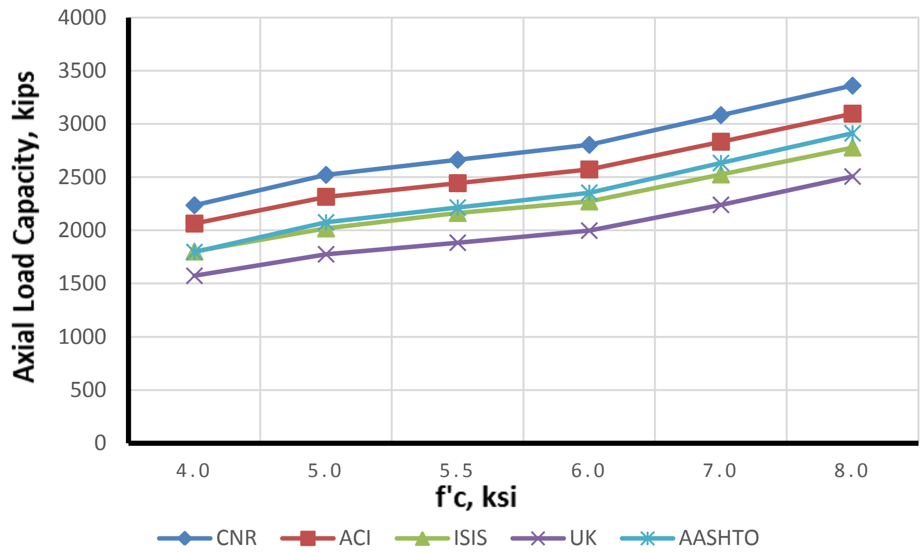

Figure 3.4.13 - Effect of changing f_c' on factored axial load resistance, circular column,

$\rho = 0.03$

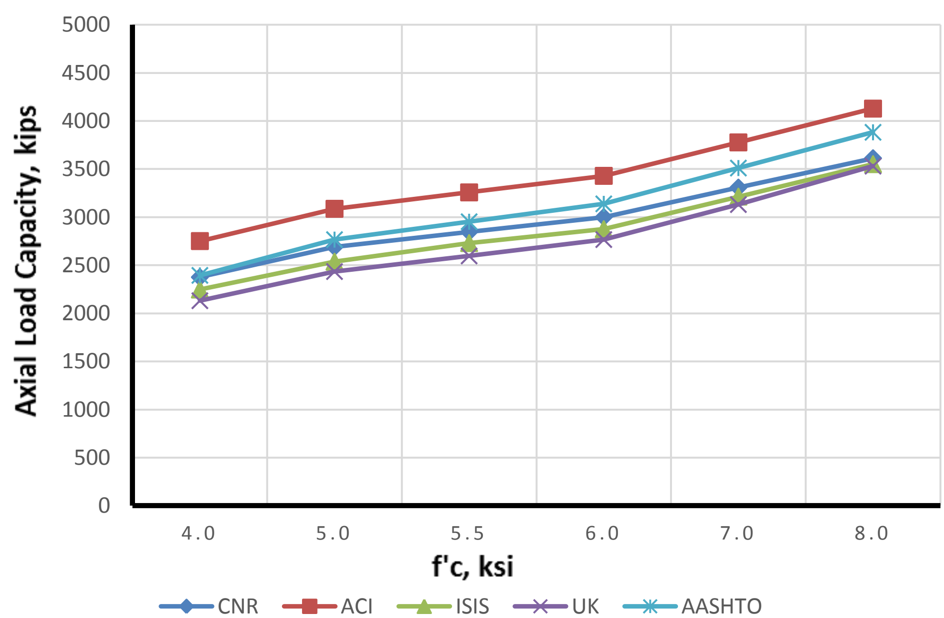

Figure 3.4.14 - Effect of changing f_c' on unfactored axial load resistance, circular column,

$\rho = 0.03$

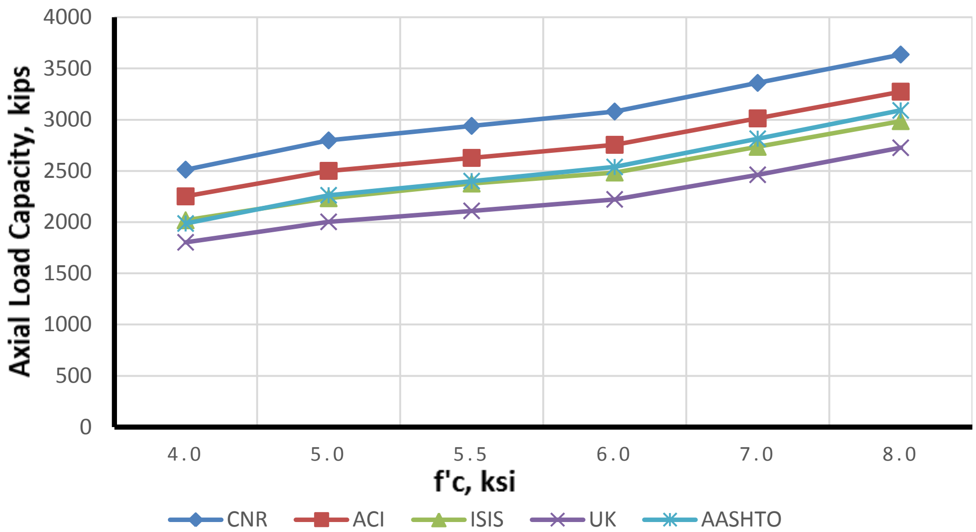

Figure 3.4.15 - Effect of changing f'_c on factored axial load resistance, circular column,

$\rho = 0.04$

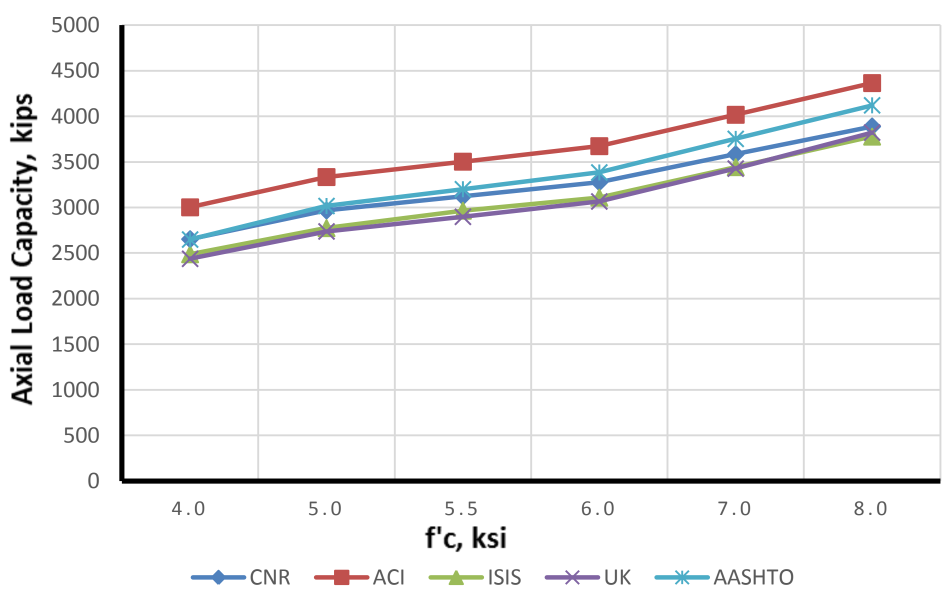

Figure 3.4.16 - Effect of changing f_c' on unfactored axial load resistance, circular column,

$\rho = 0.04$

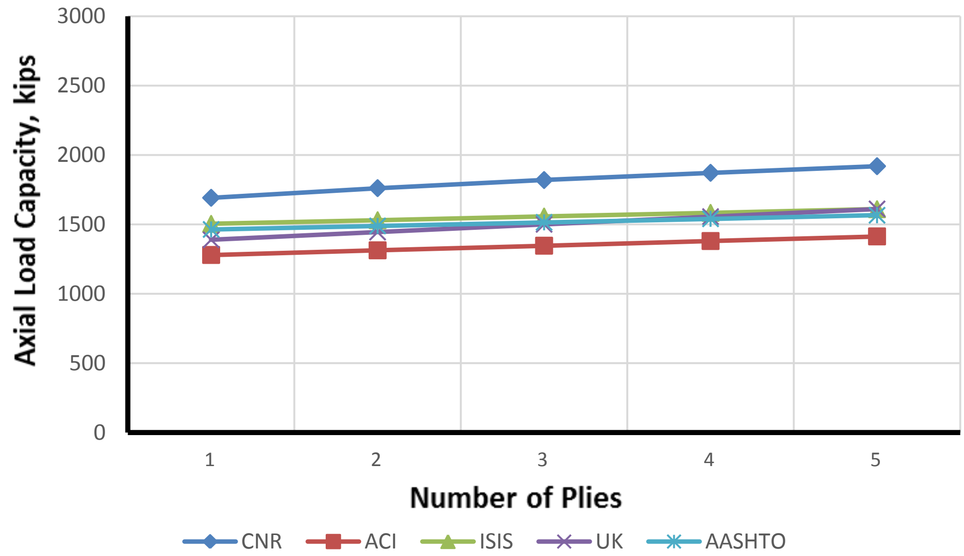

Figure 3.4.17 - Effect of changing number of plies on factored axial load resistance, square column, $\rho = 0.02$

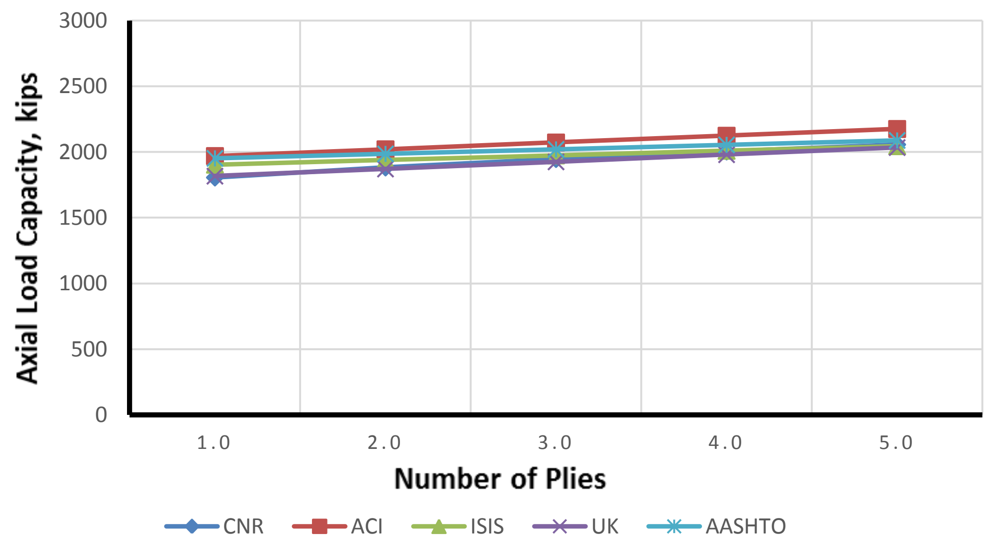

Figure 3.4.18 - Effect of changing number of plies on unfactored axial load resistance, square column, $\rho = 0.02$

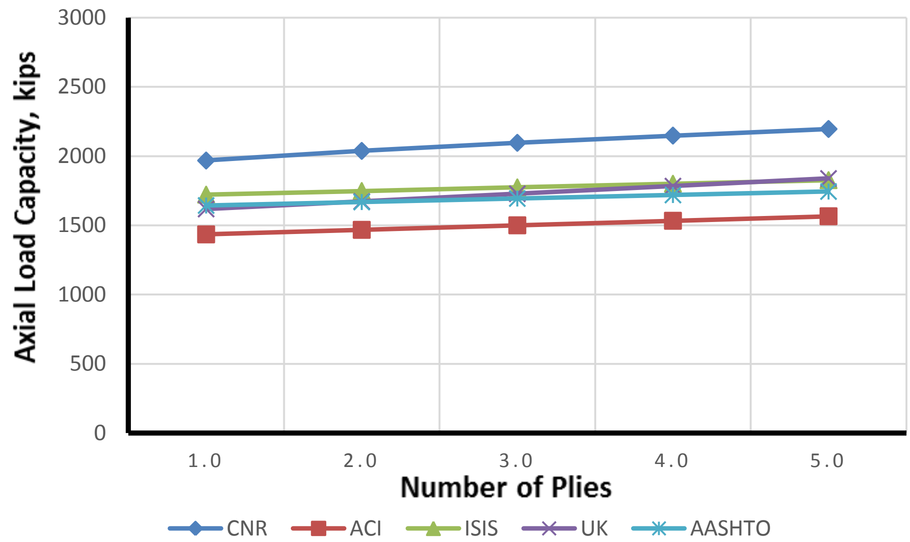

Figure 3.4.19 - Effect of changing number of plies on factored axial load resistance, square column, $\rho = 0.03$

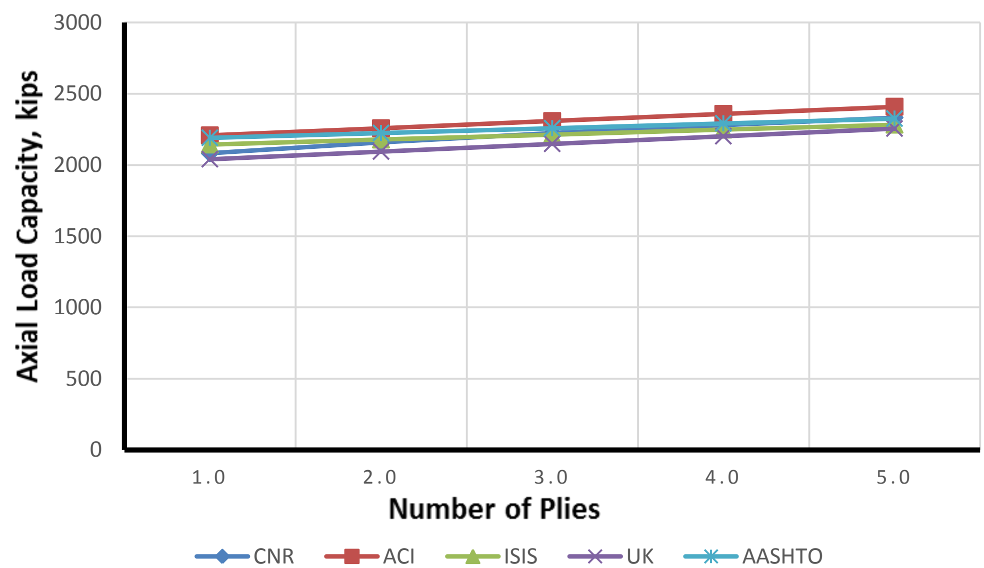

Figure 3.4.20 - Effect of changing number of plies on unfactored axial load resistance, square column, $\rho = 0.03$

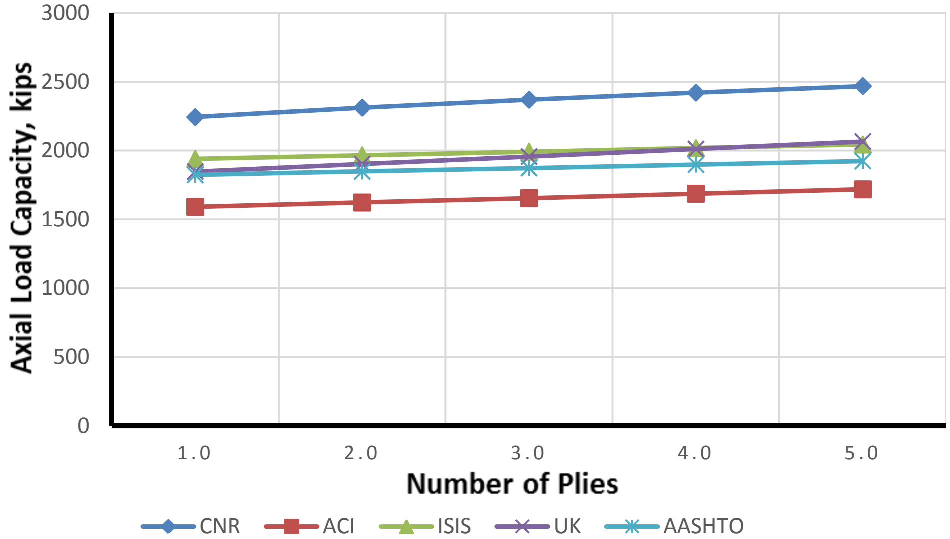

Figure 3.4.21 - Effect of changing number of plies on factored axial load resistance, square column, $\rho = 0.04$

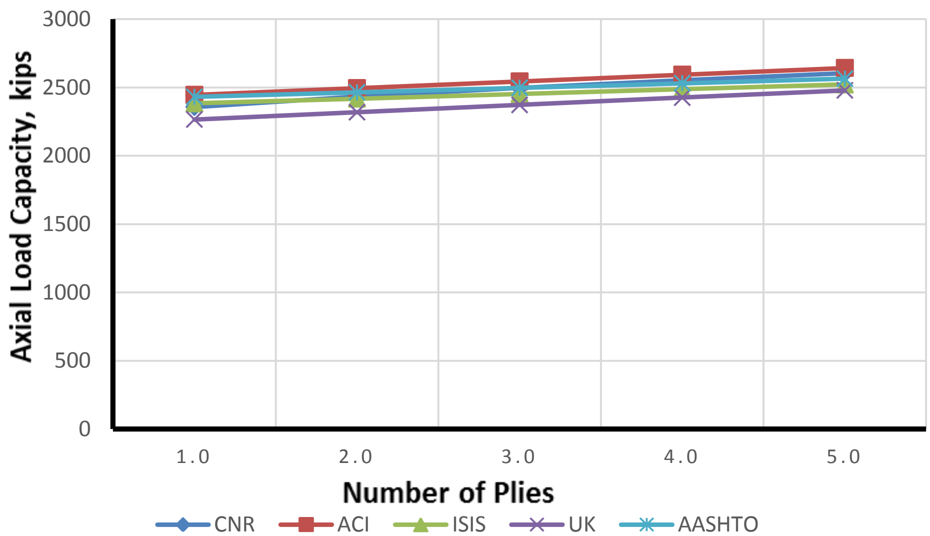

Figure 3.4.22 - Effect of changing number of plies on unfactored axial load resistance, Square column, $\rho = 0.04$

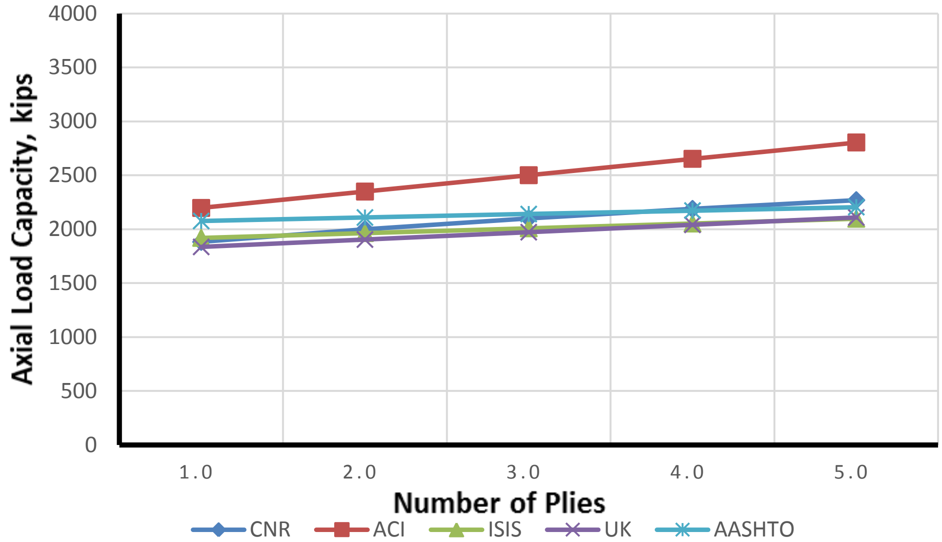

Figure 3.4.23 - Effect of changing number of plies on unfactored axial load resistance, circular column, $\rho = 0.02$

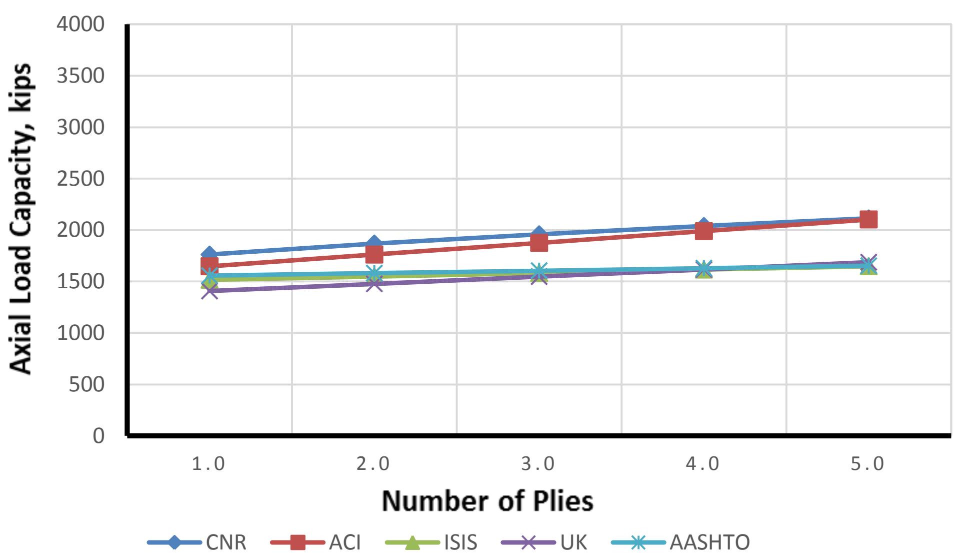

Figure 3.4.24 - Effect of changing number of plies on factored axial load resistance, circular column, $\rho = 0.02$

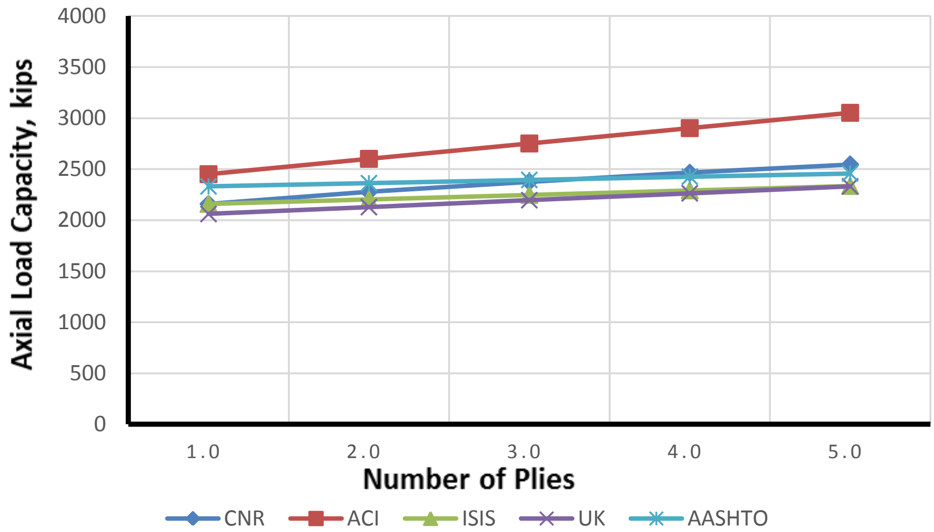

Figure 3.4.25 - Effect of changing number of plies on unfactored axial load resistance, circular column, $\rho = 0.03$

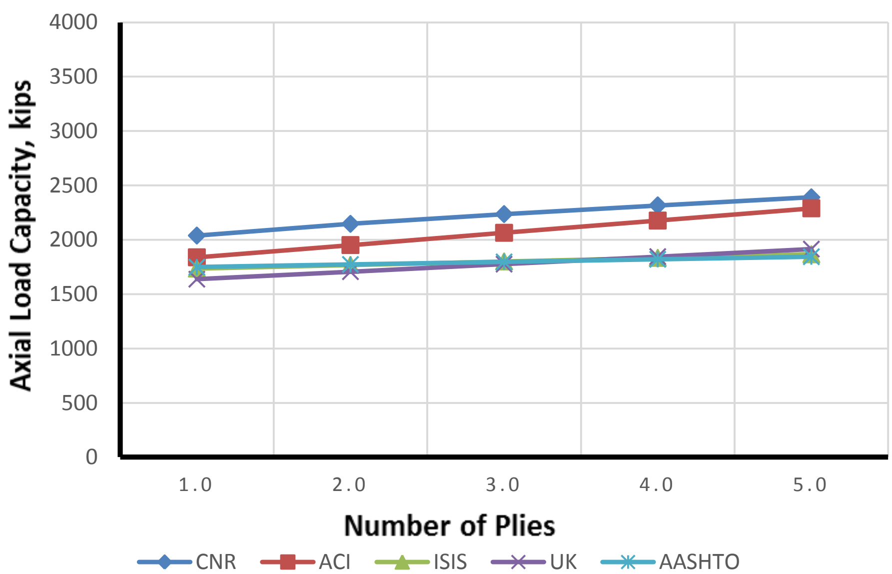

Figure 3.4.26 - Effect of changing number of plies on factored axial load resistance, circular column, $\rho = 0.03$

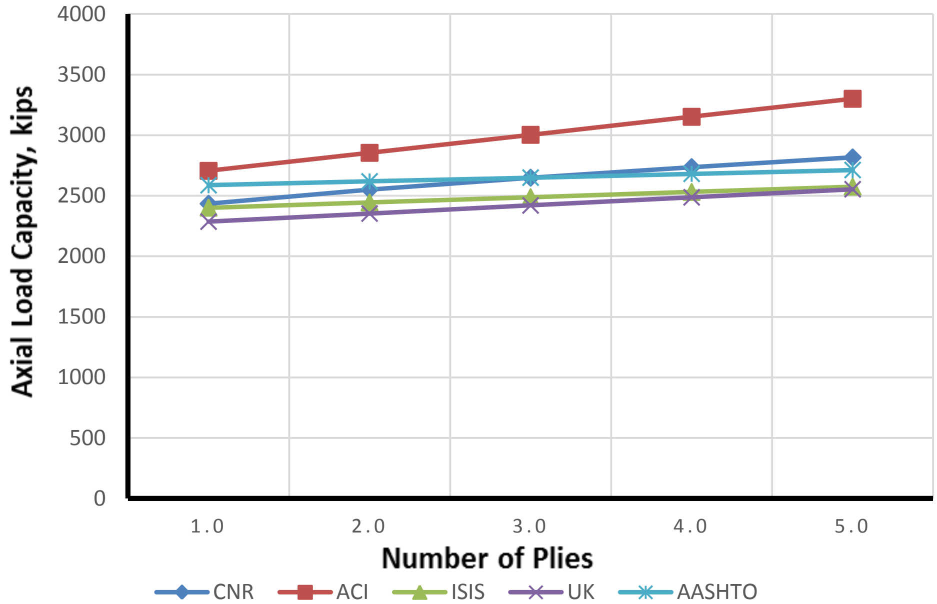

Figure 3.4.27 - Effect of changing number of plies on unfactored axial load resistance, circular column, $\rho = 0.04$

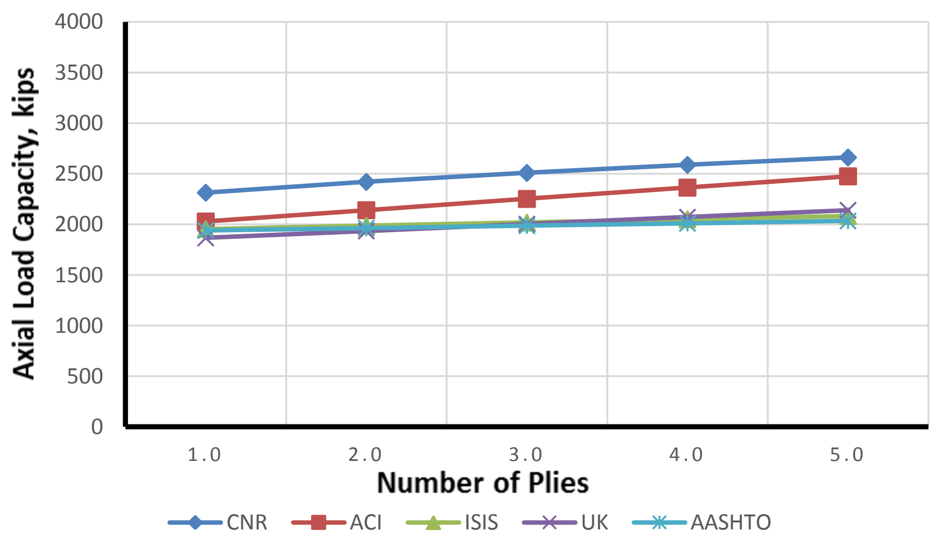

Figure 3.4.28 - Effect of changing number of plies on factored axial load resistance, circular column, $\rho = 0.04$

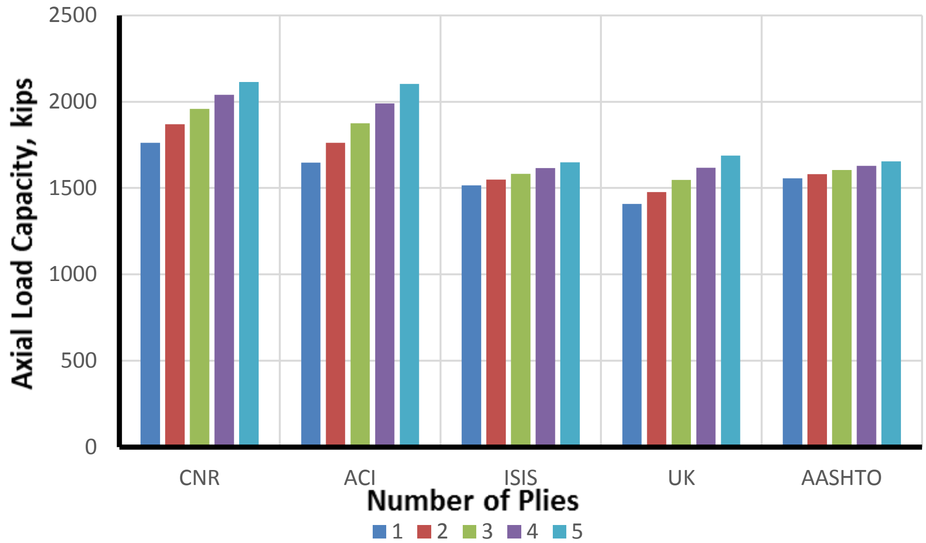

Figure 3.4.29 - Effect of changing number of plies on axial load capacity, circular column,

$\rho = 0.02$

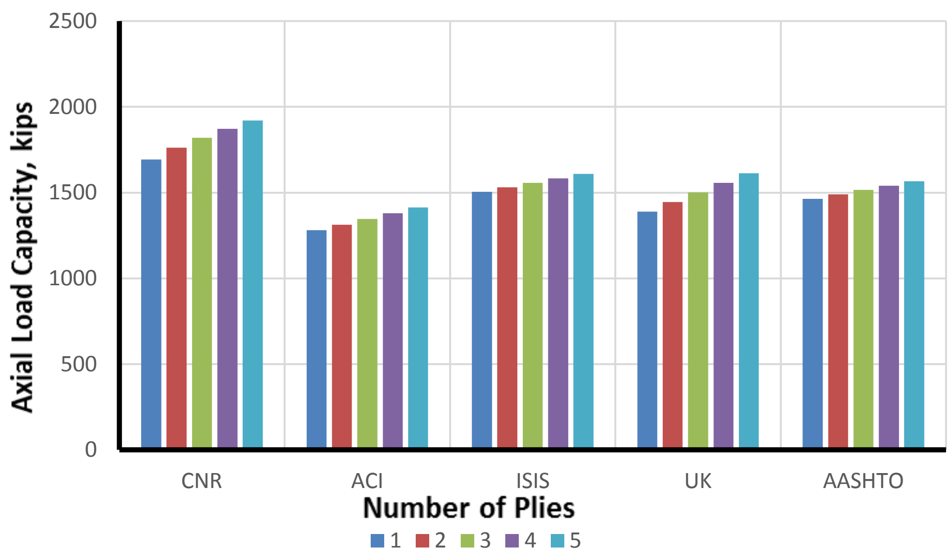

Figure 3.4.30 - Effect of changing number of plies on axial load capacity, square column,

$\rho = 0.02$

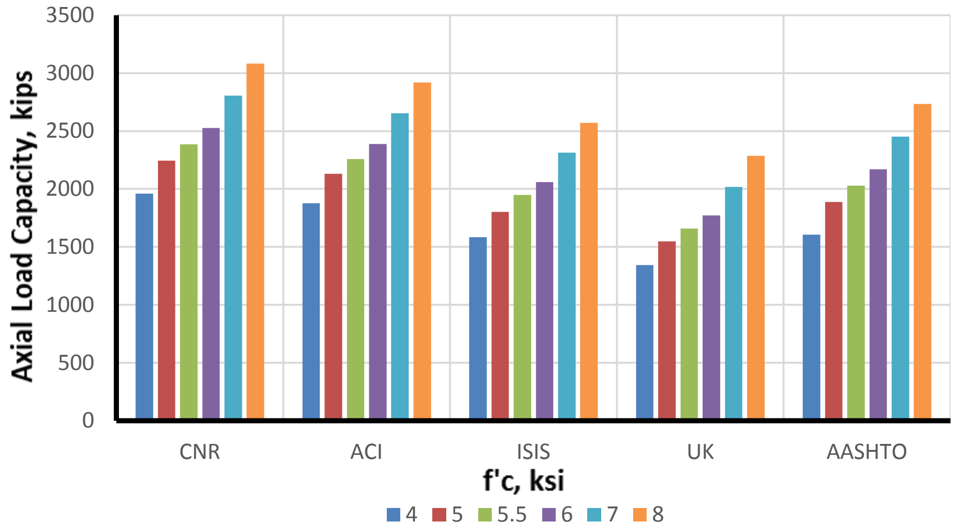

Figure 3.4.31 - Effect of changing f'_c on axial load capacity, circular column, $\rho = 0.02$

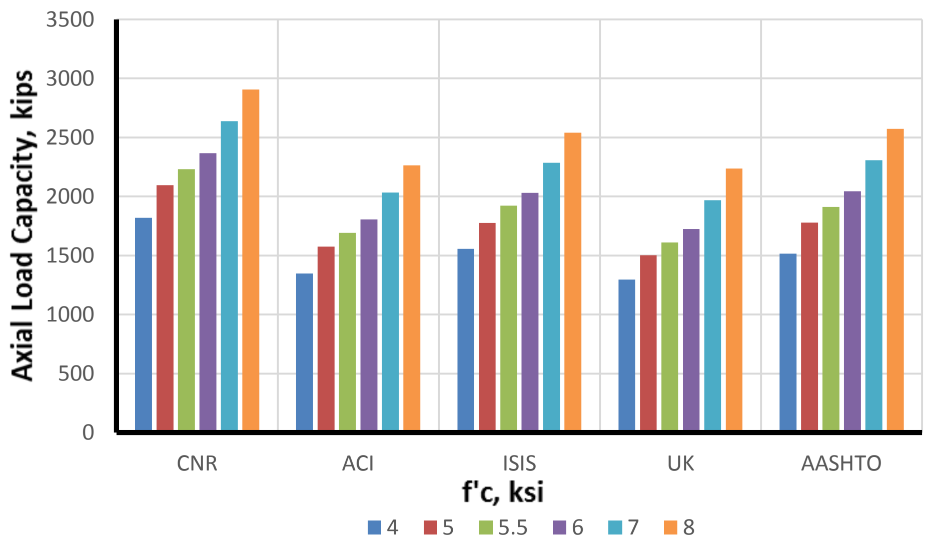

Figure 3.4.32 - Effect of changing f'_c on axial load capacity, square column, $\rho = 0.02$

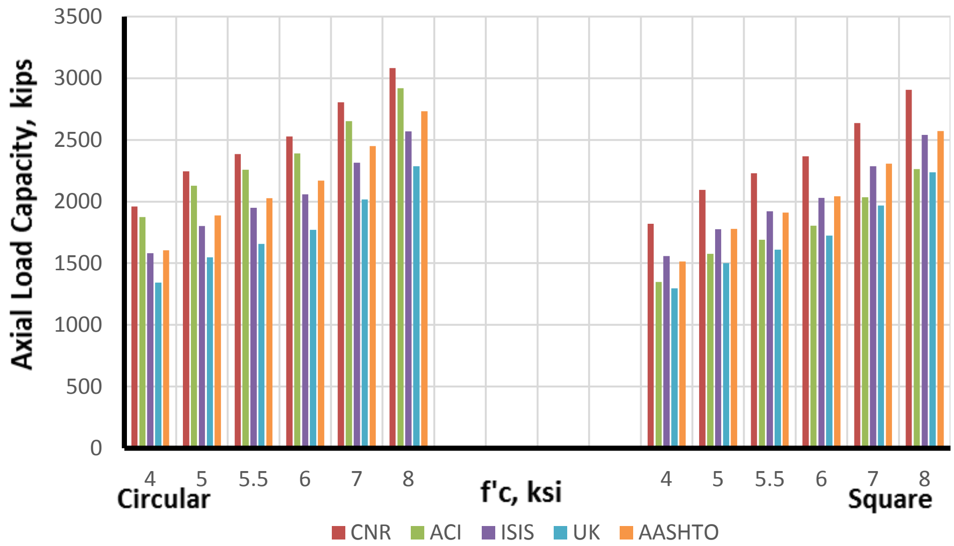

Figure 3.4.33 - Effect of changing f'$_c$ on axial load capacity, both circular and rectangular columns, $\rho = 0.02$

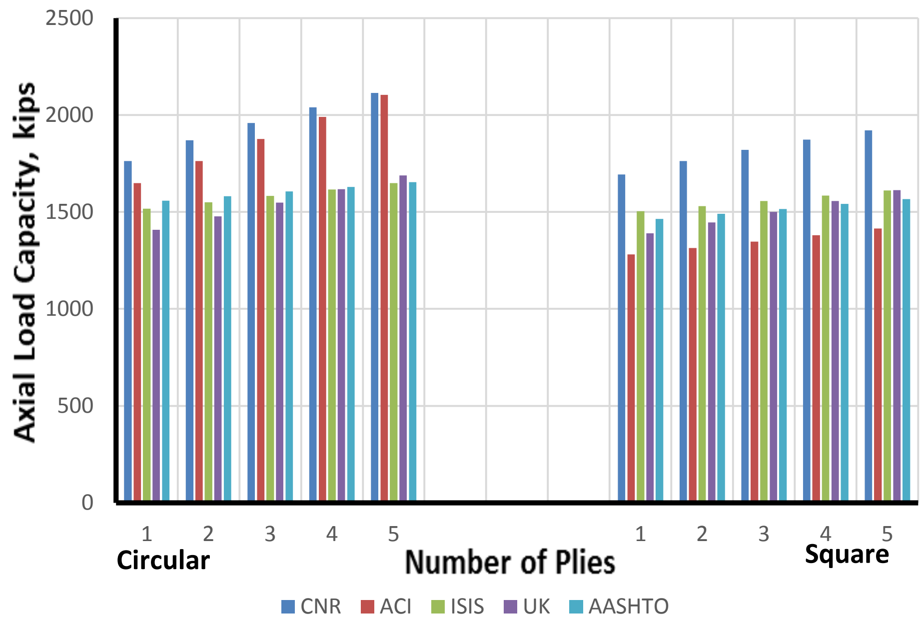

Figure 3.4.34 - Effect of changing number of plies on axial load capacity, both circular and rectangular columns, $\rho = 0.02$

3.5 Witness Panels

As further discussed throughout Chapter 4, witness panels are often part of the FRP system design. The panels are used as test panels to verify the adequacy of the FRP installation. As such, they are to be produced on-site and exposed to the same construction and weathering conditions as the strengthening system. Some codes recommend the use of witness panels in general, while others suggest that this decision should be left to the project engineer. Although there are no universal guidelines to their use, in general, as the size, complexity, and importance of the strengthening project increases, the benefits to witness panel fabrication become more compelling. The panels may be actual areas on the structure itself that are similar in character to the strengthened areas. In this case, additional strengthening material is applied on other areas of the structure for later testing. Alternatively, the panels may be smaller, portable specimens apart from the structure. As the panels may remain available for use for long periods of time, they can be used only to verify initial quality of the installation but long-term performance characteristics as well.

Typical witness panel sizes are from 6 in – 24 in^2, and are best if placed on the structure itself in different locations. In general, the number of panels increases with project complexity as well as the size of the strengthened area. As such, careful planning in the design phase of the project is necessary to decide upon the number and type of panels (i.e. on or off of the structure), and if on the structure, panel placement. Additional detail is provided in Chapters 4 and 8.

CHAPTER 4: PROVISIONS FOR INSTALLATION, QC, AND MAINTENANCE

4.1 Introduction

This chapter presents a review of field installation provisions of CFRP strengthening systems detailed in the six international codes considered in Chapter 3 (ACI-440.2R-08; ISIS Manual 4; AASHTO FRP Guide Specification; JSCE Recommendations; TR55; and CNR-DT 200).

Section 4.2 concerns installation procedures and related matters including shipping, storage, and handling of FRP system components as well as contractor qualifications to perform the work needed. Section 4.3 covers all quality assurance/quality control aspects of FRP installation including inspection, evaluation and acceptance criteria. Section 4.4 covers maintenance and repair including inspection, assessment, and repair techniques.

4.2. Installation of CFRP Strengthening Systems

4.2.1 Introduction

The review of installation procedures is based on the framework presented in the available guidelines, and includes the following items:

- FRP shipping, handling, and storage
- Contractor qualification
- Installation procedures

4.2.2 Shipping, storage, and handling

4.2.2.1 ACI

4.2.2.1.1 Shipping

In ACI, packaging, labeling, and shipping for thermosetting resin materials are to be controlled by the *Code of Federal Regulations 49* (CFR 49). Many materials are classified as corrosive, flammable, or poisonous in Subchapter C (CFR 49) under "Hazardous Materials Regulations." As such, FRP system constituent materials are to be packaged and shipped in a manner that conforms to all applicable federal and state packaging and shipping codes and regulations.

4.2.2.1.2 Storage

To preserve critical properties and maintain safety while storing FRP materials, ACI suggests that manufacturer's recommendations should be followed. Of particular concern are reactive curing agents, hardeners, initiators, catalysts, and cleaning solvents which have special safety-related requirements, and are to be stored in a manner recommended by the manufacturer as well as OSHA. Catalysts and initiators (usually peroxides) should be stored separately.

Properties of uncured resin components may change with time, temperature, and humidity. Any component material that has exceeded its shelf life, has deteriorated, or has been contaminated should not be used. FRP materials deemed unusable should be disposed of in a manner specified by the manufacturer and acceptable to state and federal environmental control regulations.

4.2.2.1.3 Handling

Thermosetting resins describe a generic family of products that includes unsaturated polyesters, vinyl esters, epoxy, and polyurethane resins. The materials used with them are generally described as hardeners, curing agents, peroxide initiators, isocyanates, fillers, and flexibilizers. ACI notes some general health hazards that may be encountered when handling thermosetting resins, from skin irritation to explosive reactions.

ACI recommends that product hazard labels and associated Material Safety Data Sheets (MSDS) are to be read and understood by those working with these products. CFR 16, Part 1500 (2009), regulates the labeling of hazardous substances and includes thermosetting-resin materials. ANSI Z-129.1 (2010) provides further guidance regarding classification and precautions. ACI states that disposable suits and gloves are suitable for handling fiber and resin materials. Rubber or plastic gloves resistant to resins and solvents are recommended and should be discarded after each use. Safety glasses or goggles should be used when handling resin components and solvents. Respiratory protection, such as dust masks or respirators, should be used when fiber fly, dust, or organic vapors are present, or during mixing and placing of resins if required by the FRP system manufacturer.

The workplace in which composite materials are prepared and installed should be well ventilated, and surfaces should be covered as needed to protect against contamination and resin spills. The manufacturer's literature should be consulted for proper mixing procedures. ACI notes that ambient cure resin formulations produce heat when curing, which accelerates the reaction. Uncontrolled reactions, including fuming, fire, or violent boiling, may occur in containers holding a mixed mass of resin; therefore, such containers should be monitored.

As cleanup can involve flammable solvents, appropriate safety precautions are suggested. However, cleanup solvents are available that do not present flammability concerns. All waste materials are to be disposed of as prescribed by the prevailing environmental authority.

4.2.2.2 ISIS

As with ACI, according to ISIS, the shipping, handling and storage of all fiber, resin and FRP materials are to be performed in accordance with manufacturer's specifications. The shipping, handling and storage of FRP materials is covered in Clause A16.1.2 of the S6-06 bridge code (CSA-S6-06, 2006) and Clause 14.3 of the S806-02 building code (CSA-S806-02, 2002). Governing safety and environmental regulations must be followed, and appropriate documentation is to be provided that specifies composite material properties, installation requirements, and safety considerations for workers, the environment, and the public. This documentation includes technical data sheets of FRP products, as well as MSDSs.

4.2.2.2.1 Shipping

ISIS states that FRP materials are to be packaged and shipped in a manner that conforms to applicable packaging and shipping regulations, with particular concern to thermosetting resin materials that are classified as corrosive, flammable or poisonous, that must follow appropriate regulations for hazardous materials. It is the duty of the contractor and supplier to ensure that the packaging and shipping methods used do not negatively impact material properties and performance. All FRP components must be shipped with their respective MSDSs. ISIS recommends that all components of the FRP system are inspected upon delivery to the construction site, and the use of opened or damaged containers should only proceed with written authorization by the project engineer.

4.2.2.2.2 Storage

Proper storage of FRP components is in a clean, dry area, sheltered from the sun, which is well ventilated and temperature controlled and in accordance with the manufacturer's recommendations. As with ACI, ISIS notes that special safety requirements are required in the storage and handling of certain components such as reactive curing agents, hardeners, initiators, catalysts, and cleaning solvent. Catalysts and initiators (e.g., peroxides) should be stored separately.

The manufacturer is to provide a recommended shelf life within which the properties of the resin-based materials should continue to meet or exceed the stated performance, and the contractor must follow these time limits. Materials that have exceeded their shelf life or have otherwise exhibited signs of deterioration should be disposed of in a manner specified by the manufacturer.

4.2.2.2.3 Handling

ISIS refers to ACI 503R (1998) for detailed handling information and potential hazards of FRP components. However, ISIS recommends that special care should be taken to avoid material contact with water, dust or other contaminants. Moreover, excessive bending, crushing and other sources of mechanical damage to the fibers must be avoided. All involved in handling thermosetting resins are to read and understand product labels and MSDSs.

Personal protection precautions include the use of disposable rubber or plastic gloves that are resistant to resins and solvent penetration (which should be discarded after each use), safety glasses or goggles, as well as respiratory protection such as dust masks or respirators when handling resin components and solvents, or for operations where fiber fly, dust, or organic vapors are present. In poorly ventilated areas, the use of respiratory protection is required, preferably with a fresh air supply.

Information regarding proper storage, handling, and mixing resin components and potential hazards should be provided by the manufacturer and made available at the construction site. As with ACI recommendations, ISIS notes that mixed resin containers should be frequently monitored since uncontrolled reactions, including fuming and fire, may occur. Similarly, ISIS notes that the workplace should be adequately ventilated, and surfaces covered as needed to protect against contamination and resin spills.

All waste materials are to disposed of as prescribed by the prevailing environmental authority, and appropriate precautions observed during clean-up, since some cleaning solvents may be flammable.

4.2.2.3 AASHTO

AASHTO does not provide specific recommendations for shipping, storage, and handling. However, NCHRP Report 609 (2008), upon which other AASHTO recommendations are based, offers the following guidelines.

4.2.2.3.1 Shipping and storage

All FRP system components must be delivered and stored in the original factory-sealed, unopened packaging with labels identifying the manufacturer, brand name, system identification number and date. Catalysts and initiators are to be stored separately. All components must be protected from dust, moisture, chemicals, direct sunlight, physical damage, fire, and temperatures outside the range specified in system data sheets.

NCHRP 609 notes that typically, temperature in the storage area should be within 50°–75°F (10°–24°C), unless otherwise noted on the system data sheet, and components should be stored in a dry environment, unless an acceptable moisture level is specified on the system data sheet. It is further stated that any component that has been stored in a condition different from that stated above must be disposed of.

4.2.2.3.2 Handling

All FRP components, but especially fiber sheets, must be handled with care according to manufacturer recommendations to protect them from damage and to avoid misalignment or breakage of the fibers. NCHRP 609 notes that higher modulus fibers are more susceptible to misalignment damage, and therefore should be handled with greater care. After cutting, sheets shall be either stacked dry with separators, or rolled gently at a radius no tighter than 12 in (305 mm), or as recommended by the manufacturer.

All components of the FRP system, especially resins and adhesives, must be handled with care to avoid safety hazards, including but not limited to skin irritation and breathing vapors and dusts. Resin mixing is to be monitored to avoid fuming and production of excessive inflammable vapors, fire hazards, or violent boiling. It is the contractor's responsibility to ensure that all components of the FRP system at all stages of work conform to governing environmental and safety regulations.

4.2.2.4 JSCE

4.2.2.4.1 Shipping

JSCE recommends that the handling precautions relating to material deterioration and safety during delivery, storage, mixing, processing, and use are to be confirmed in advance and strictly observed. Materials are to be properly shipped and stored to ensure that no deterioration occurs.

4.2.2.4.2 Storage and handling

JSCE notes that continuous fiber sheets and strands are easily damaged before being impregnated with resin, and some types of continuous fibers may deteriorate if exposed to ultraviolet light and moisture. Therefore, in general, the FRP materials should be stored in a cool, dark place without exposure to direct sunlight. As resins are potentially harmful to workers, resin containers must be sealed securely and stored in a cool, dark place. As resins are also flammable, fire precautions should be observed and storage quantities kept within limits prescribed by fire regulations (JSCE references the Japanese Fire Defense Law). JSCE further recommends that consideration is given to the handling manuals prepared by the material manufacturer.

4.2.2.5 TR55

TR55 does not have a specific section discussing shipping, storage and handling. The code requires, however, that all installation related work abide by governing safety laws (TR55 references The Health and Safety at Work Act and The Control of Substances Hazardous to Health Regulations). The code emphasizes that a certificate of conformity to these laws must be provided with the materials from the supplier.

TR55 provides few additional material storage guidelines, suggesting that all materials should be stored and used strictly in accordance with the manufacturer's instructions, with particular attention to maintaining proper temperature, and for adhesives, a dry storage area. TR55 suggests that adhesive and material delivery dates should be recorded, and these items used in rotation based on the date of arrival. It further states that materials should be stored at the construction site in a way that damage and contamination are avoided.

4.2.2.6 CNR

4.2.2.6.1 Shipping and storage

CNR notes that each component of the FRP system is to be suitably packaged and transported according to governing safety regulations. FRP materials are to be stored according to the recommendations provided by the supplier/manufacturer. CNR notes that the properties of non-polymerized resins may change over time and are affected by moisture and temperature. Temperature may also affect the mixture reactivity and properties of polymerized resin. Suitable environmental conditions for storage are suggested to be from 50–75°F (10-24 °C) and in a dry environment with less than 20% humidity, unless otherwise suggested by the manufacturer.

In storage, care is to be taken to avoid laminate and other preformed material damage due to bending or improper stacking. It is especially important that some potentially hazardous constituents such as reactive reticulating agents, initiators, catalysts, solvents for surface cleaning, etc., are stored according to manufacturer requirements or official standards. As with other code recommendations, CNR specifies that catalysts and initiators (typically peroxides) are stored separately from other reagents to avoid any accidental contact leading to premature polymerization.

Manufacturers are to indicate the storage time (shelf life) that ensures thermo-setting resin properties are maintained. Constituents exceeding their shelf life or suffering degradation or contamination are not to be used, and those deemed unusable are to be disposed of according to manufacturer specifications as well as the governing safety laws.

4.2.2.6.2 Handling

CNR states that the manufacturer is to provide the MSDSs for all FRP constituents. It notes that substances used in combination with thermoset resins are typically hardeners, cross linkers, initiators (peroxides), and fillers, and these are associated with potential health hazards. Personnel working with these substances are to read all labels as well as MSDS to minimize risks. When handling fibers and resins, disposable gloves and work-suits, as well as protective glasses are suggested. Rubber or plastic gloves are to be solvent-resistant. In the presence of fiber fragments, dusts or solvent vapors, or when mixing and applying resins, respiratory protection devices are needed, in accordance to the suggestion of the FRP manufacturer. The working site must always be properly ventilated.

4.2.2.7 Summary of shipping, storage, and handling

ACI, ISIS, and CNR stipulate that packaging, labeling, and shipping are to be done in accordance with applicable national and local packaging and shipping codes and regulations. ISIS stresses the need to abide by the manufacturer's guidelines for shipping as well. AASHTO and TR55 have no specific regulations for shipping. JSCE emphasizes the need to properly ship FRP materials to prevent material deterioration during shipping.

AASHTO does not have provisions regarding FRP storage, and storage recommendations are taken from NCHRP 609. ACI, NCHRP 609, ISIS, CNR, and TR55 state that storage of FRP materials should be done in accordance with manufacturer's recommendations to preserve the material properties and maintain worker safety. ACI adds that reactive curing agents and cleaning solvents should also be stored as recommended by OSHA. ACI, NCHRP 609, ISIS and CNR require that catalysts and initiators (usually peroxides) be stored separately.

Since FRP materials have a prescribed shelf life, there is a chance that stored materials exceed this limit and require disposal. ACI requires that material disposal be conducted in accordance with the manufacturer's guidelines and be acceptable to state and federal environmental control regulations. CNR states that disposal should be done in accordance to the manufacturer as well as the provisions of safety laws and regulations. ISIS relies only on the manufacturer's recommendations for materials disposal. AASHTO has no specific recommendations for materials disposal. However, NCHRP 609, Section 3.4 stipulates that disposal of expired materials be performed in a manner to protect the environment and follow the manufacturer's recommendations as stated in the MSDS.

Storage temperature, moisture, and other considerations are discussed with varying details among different codes. AASHTO specifies a storage temperature of 50-70^{o}F (10-21^{o}C), a dry enclosure protected from dust, moisture, direct sun, and the risk of fire or damage. CNR recommends the same temperature range as NCHRP 609, but requires a dry environment with a moisture not to exceed 20% or as recommended by the manufacturer. ACI has no specific stated temperature or moisture recommendations but suggests following the recommendations of the manufacturer. TR55 states that storage temperature range is to be in accordance with the manufacturer, while JSCE places emphasis on fire protection of the storage facility based on limits prescribed by the Fire Defense Law. JSCE adds several requirements for storage such as maintaining a cool temperature and a dark environment with no direct exposure to sunlight.

ACI, NCHRP Report 609, ISIS, and CNR require that manufacturer's recommendations are to be used when handling FRP materials to protect against health risks from skin exposure or fume inhalation. ACI refers to CFR 16, Part 1500 for regulations on hazardous substance labeling, and ANSI Z-129.1 for further guidance regarding classifications and precautions. ACI, ISIS, and CNR require individuals handling FRP system components to wear disposable suites, gloves, eye protection, and respirators for safe handling of the materials. The three codes recommend the use of a well-ventilated work place. TR55 and JSCE have no coverage of FRP materials handling. ACI and ISIS state that for cleanup and disposal, the use of cleanup solvents that do not present a high flammability risk is advisable. The codes require that all waste materials be contained and disposed as prescribed be the prevailing environmental authority.

4.2.3 Contractor qualifications

4.2.3.1 ACI

ACI suggests that the FRP system installation contractor should demonstrate competency for surface preparation and application of the FRP system to be installed. This competency can be demonstrated by providing evidence of training and documentation of related work previously completed, or by an actual demonstration of surface preparation and installation. It is recommended that the FRP system manufacturer or its authorized agent should train the contractor's application personnel in the installation procedure for the specific system to be used.

4.2.3.2 ISIS

ISIS stipulates that the contractor is responsible for providing the training of his staff and shall provide proof of the qualification or experience of his staff to the project engineer. The staff, which includes the installation crew, supervisors, and safety officers, must be properly trained on the specific tasks within their responsibility. For example, a site safety officer must be trained on the course of action in the case of accident in accordance with materials safety regulations provided in the MSDSs.

4.2.3.3 AASHTO

As with ACI, AASHTO recommends that FRP application must be performed by a contractor trained in accordance with the installation procedures specified by the manufacturer. For further information, AASHTO references NCHRP 609 (2008). According to NCHRP 609, the manufacturer/supplier may be pre-qualified for each FRP system to be installed, after providing the following information:

- System data sheets and MSDS for all components of the FRP system;
- Documentation of a minimum of 5 years' experience with the FRP system, or 25 documented similar field applications with acceptable reference letters from respective owners;
- Documentation of a minimum of 50 test data sets from an independent agency approved by the owner verifying the mechanical properties, aging and environmental durability of the proposed FRP system, and;
- Documentation of the availability of a comprehensive hands-on training program for each FRP system that can be taken by the staff of the contractor/applicator.

The owner may also require the manufacturer/supplier to provide a specified number of samples of the components and the complete FRP system for in-house or independent testing prior to qualification. The training program conducted by the manufacturer/supplier should provide hands-on experience with surface preparation and installation of the same FRP system for which the certificate is issued. The contractor/applicator may be pre-qualified by the owner for each FRP system, after providing documentation of a minimum of 3 years' experience or 15 similar field applications with acceptable reference letters from respective owners, and a certificate of completed training from the manufacturer/supplier for at least one staff member who will be present on site throughout the project.

4.2.3.4 JSCE

JSCE recommends that work should be performed under the supervision of an engineer who has thorough knowledge of FRP strengthening. However, the code does not specifically discuss criteria for qualifying a contractor.

4.2.3.5 TR55

TR55 recommends that various issues should be taken into account when selecting a contractor. In particular, it suggests that the contractor provide evidence of experience in strengthening work, has quality assurance procedures in place, and is accredited and audited in accordance with ISO 9002. TR55 further suggests that the contractor should be a member of the Concrete Repair Association or has a record of successful projects involving the installation of composites. Moreover, the contractor should provide a detailed statement of the method which will be used to install the composites as well as an assessment of the risks involved. This should include discussion of the procedure which will be used to minimize the risks to the workforce and to any other persons (especially children) who may be affected by the work. The contractor's personnel should be supplied with the correct protection equipment for use when handling the materials, and trained and qualified in the application technique specified by the manufacturer of the system. Finally, the contractor should provide a safe means of access to the work location and maintain an environment suitable for the successful use of structural adhesives.

4.2.3.6 CNR

CNR does not provide specific recommendations for qualifying a contractor. However, it defines several responsibilities of the contractor as follows. The contractor is to obtain the material indicated by the designer through suppliers/manufacturers who guarantee the quality of their products, and is to ensure that the products are accompanied by technical data sheets, reporting both mechanical and physical characteristics, and possibly laboratory test certificates. Finally, the contractor is to see that these products comply with the provisions indicated by the designer, and if the material with the indicated requirements is not available, the contractor is to work with the designer to find a viable alternative.

4.2.3.7 Summary of contractor qualifications

ACI, ISIS, and TR55 require that the contractor provide evidence of training on the FRP strengthening system to be used, as well as provide evidence of past experience of similar projects. CNR and JSCE do not provide any specific contractor qualifications requirements. AASHTO refers to NCHRP 609 for recommendations with regard to contractor qualifications. This report offers a comprehensive list of qualification requirements for the supplier and the contractor with specifics such as documentation to verify the required years of experience for the supplier (5 years). The report generally requires the submittal of documented evidence for all items needed for qualification (see Article 4.2.3.3, above).

4.2.4 Installation Procedures

4.2.4.1 ACI

ACI notes that procedures for installing FRP systems have been developed by the system manufacturers and often differ between systems. In addition, installation procedures can vary for the same system, depending on the type and condition of the structure. ACI recommends that deviations from the procedures developed by the manufacturer should not be allowed without manufacturer approval.

4.2.4.1.1 Temperature, humidity, and moisture considerations

ACI emphasizes that temperature, relative humidity, and surface moisture at the time of installation can affect the performance of the FRP system. It suggests that primers, saturating resins, and adhesives should generally not be applied to cold or frozen surfaces. When the surface temperature of the concrete surface falls below a minimum level as specified by the FRP system manufacturer, improper saturation of the fibers and improper curing of the resin constituent materials can occur, compromising the integrity of the system. It suggests that a non-contaminating heat source can be used to raise the ambient and surface temperatures during installation.

With regard to moisture, ACI states that resins and adhesives should generally not be applied to damp or wet surfaces unless they have been formulated for such applications. Moreover, FRP systems should not be applied to concrete surfaces that are subject to moisture vapor transmission. ACI warns that the transmission of moisture vapor from a concrete surface through the uncured resin materials typically appears as surface bubbles and can compromise the bond between the FRP system and the substrate.

4.2.4.1.2 Equipment

Some installation procedures specify unique equipment designed for the application of that particular system. This equipment can include resin impregnators, sprayers, lifting/positioning devices, and winding machines. ACI suggests that all equipment should be clean and in good operating condition. All supplies and equipment should be available in sufficient quantities to allow continuity in the installation project and quality assurance.

4.2.4.1.3 Surface preparation

Successful strengthening with FRP systems is dependent on a sound concrete substrate and proper preparation of the concrete surface, as an improperly prepared surface can result in debonding or delamination. Although ACI presents general guidelines intended to all externally bonded FRP systems, it notes that specific guidelines for a particular FRP system should be obtained from the manufacturer. For general methods of concrete repair and surface preparation, ACI refers to ACI 546R (2004) and ICRI 03730 (2008). ACI suggests that the FRP system manufacturer should be consulted to ensure compatibility of the FRP system with the materials used for repairing the concrete substrate, if necessary. If corrosion-related concrete deterioration is detected, ACI suggests that the cause of the corrosion be addressed, and the associated deterioration repaired before application of the FRP system.

To repair cracks, those wider than 0.010 in (0.3 mm) can be pressure injected with epoxy before FRP installation, in accordance with ACI 224.1R (2007). Cracks of smaller width may require resin injection or sealing to prevent corrosion of existing reinforcement. ACI refers to ACI 224.1R (2007) for crack-width limitations based on different exposure conditions.

ACI classifies surface preparation into two major categories: bond-critical or contact-critical. Bond-critical applications require an adhesive bond between the FRP and the concrete, and usually involve systems for flexural or shear strengthening. ACI suggests that surface preparation for bond-critical applications should be in accordance with ACI 546R (2004) and ICRI 03730 (2008). A summary of recommendations is given as follows:

- The surface on which the FRP system is to be applied should be free of loose or unsound materials, and where fibers wrap around the corners of rectangular cross sections, the corners should be rounded to a minimum 0.5 in (13 mm) radius to prevent stress concentrations in the FRP as well as voids between the FRP and the concrete. This applies to both vertical and horizontally-oriented corners. Note that ACI does not specifically mention allowing an existing or created chamfered edge to substitute for a rounded edge. Roughened corners can be smoothed with putty. Inside corners and concave surfaces may require special detailing to ensure bond between the FRP system and the concrete.

- Consideration should be given to removing obstructions and embedded objects in the concrete before installing the FRP system, and surface preparation can be accomplished with either abrasive or water-blasting techniques. Bug holes and other small surface voids should be completely exposed during surface profiling. After profiling, the surface should be cleaned and protected.
- The concrete surface should be prepared to a minimum concrete surface profile (CSP) 3, as defined by ICRI surface profile chips, and localized out-of-plane variations, including form lines, should not exceed 1/32 in (~1 mm) or the tolerances recommended by the FRP system manufacturer.
- Localized variations can be removed by grinding, before abrasive or water blasting, or can be smoothed over using resin-based putty if variations are small. Bug holes and voids should be filled with resin-based putty. All surfaces to be strengthened should be as dry as recommended by the FRP system manufacturer.

Contact-critical applications only require close contact between the FRP and concrete, and are generally reserved for confinement strengthening. In applications involving confinement, the surface should be prepared such that continuous contact between the concrete and the FRP system is maintained. Moreover, surfaces to be wrapped should be flat or convex, and large surface voids should be patched. Materials with low compressive strength and elastic modulus, such as plaster, should be removed.

4.2.4.1.4 Mixing of resins

According to ACI, the manufacturer should supply recommended batch sizes, mixture ratios, mixing methods, and mixing times, and all mixing should be done according to the manufacturer's recommendations. Resin components should be at the proper temperature and mixed until there is a uniform and complete mixing of components. Resins should be mixed for the prescribed mixing time and visually inspected for uniformity of color. As resin components are often contrasting colors, ACI notes that full mixing is usually achieved when color streaks are eliminated. Resin mixing should be in quantities sufficiently small to ensure that all mixed resin can be used within the resin's pot life.

4.2.4.1.5 Application

ACI recommends that FRP systems be selected with consideration for their impact on the environment, including emission of volatile organic compounds and toxicology.

Where required, primer should be applied to all areas on the concrete surface where the FRP system is to be placed. Here, primer should be placed uniformly at the manufacturer's specified rate of coverage. Once applied, the primer should be protected from dust, moisture, and other contaminants before applying the FRP system. Putty should be used in an appropriate thickness and sequence with the primer as recommended by the FRP manufacturer, and only to fill voids and smooth surface discontinuities. Rough edges or trowel lines of cured putty should be ground smooth, and primer and putty should be cured as specified by the FRP system manufacturer before applying the FRP resin or adhesive. Note that after the putty and primer are cured, the FRP system manufacturer may require additional surface preparation before the application of the saturating resin or adhesive.

The use of solvents to clean the FRP surface before applying a coating is not recommended due to the damaging effect that solvents may have on resin, and the FRP system manufacturer should approve the use of any solvent wipe. Coatings should be periodically inspected and maintenance provided as needed to ensure the effectiveness of the coating.

Wet lay-up FRP systems are typically installed by hand using dry fiber sheets and a saturating resin, and should use the manufacturer’s installation recommendations. Generally, saturating resin should be applied uniformly to all prepared surfaces where the system is to be placed, then the reinforcing fibers should be gently pressed into the resin. Entrapped air between layers should be released or rolled out before the resin sets. Sufficient resin should be applied to achieve full saturation of the fibers, and successive layers of saturating resin and fibers should be placed before the complete cure of the previous resin layer. If previous layers have cured, interlayer surface preparation, such as light sanding or solvent application as recommended by the system manufacturer, may be required.

Wrapping machines are primarily used for concrete columns. Machine-applied systems can use resin pre-impregnated tows or dry-fiber tows; prepreg tows are impregnated with saturating resin off-site and delivered to the work site as spools, while dry fibers are impregnated at the job site during the winding process. ACI emphasizes that the FRP system manufacturer’s recommendations should be followed in all application steps. After wrapping, prepreg systems should be cured at an elevated temperature in accordance with the manufacturer’s recommendations.

Precured systems include shells, strips, and open grid forms that are typically installed with an adhesive. General recommendations are as follows. Surfaces to be bonded should be clean and prepared in accordance with the manufacturer’s recommendations, with a minimum concrete surface profile (CSP) of 3 (ICRI 03732). The adhesive should then be applied uniformly at the rate recommended by the FRP manufacturer. After the precured sheets are placed into the wet adhesive, entrapped air between layers should be released or rolled out before the adhesive sets. Any protective coatings that are used should be compatible with the FRP strengthening system and applied in accordance with the manufacturer’s recommendations.

4.2.4.1.6 Alignment of FRP materials

Proper FRP alignment is critical, as even small variations in angle (as little as 5 degrees) from that intended can cause a substantial reduction in strength and stiffness. Therefore, materials should be handled such that correct fiber straightness and orientation are preserved, and deviations in ply orientation should be made only if approved by the project engineer. Moreover, kinks, folds, waviness, or other forms of substantial material malformation should be reported for evaluation.

4.2.4.1.7 Multiple plies and lap splices

Multiple plies may be used, provided that all plies are fully impregnated with resin the shear strength is sufficient to transfer the shearing load between plies, and the bond strength between the concrete and FRP system is sufficient. Lap splices may be used for long spans as necessary, provided that they are staggered, unless otherwise approved by the project engineer. Lap splice details, including lap length, should be based on testing and installed in accordance with the manufacturer's recommendations. Specific guidelines on lap splices are given in ACI Chapter 13.

4.2.4.1.8 Curing

Ambient-cure resins can take several days to reach full cure, and temperature extremes or fluctuations can retard or accelerate curing time. All resins should be cured according to the manufacturer's recommendations, and field modification of resin chemistry should not be permitted. The cure status of installed plies should be verified to be sufficient before placing subsequent plies, and the installation of successive layers should be halted if there is a curing anomaly.

To meet the manufacturer's curing recommendations, FRP systems may require protection such as tents or plastic screens, during installation and curing against adverse temperatures, direct contact by rain, dust, or dirt, excessive sunlight, high humidity, and vandalism. If temporary shoring is required, the FRP system should be fully cured before the shoring is removed. If damage to the FRP system is suspected during installation, the project engineer should be notified and the FRP system manufacturer should consulted for evaluation.

4.2.4.2 ISIS

As with ACI, ISIS stresses the importance of following the specific recommendations of the FRP system manufacturer.

4.2.4.2.1 Temperature, humidity, and moisture considerations

ISIS notes that the temperature, humidity, and dew point at the time of installation can affect the performance of the FRP system. It suggests that in general, primers, saturating resins and adhesives should not be applied to cold or frozen surfaces. Per Clause A16.1.3.5 of the S6-06 bridge code, during the installation of the FRP, ambient air and concrete surface temperature should be 50°F (10°C) or more; the concrete surface temperature should be at least 5°F (3 °C) higher than the actual dew point; and atmospheric relative humidity should be less than 85%. To meet these conditions in colder temperatures, it may be necessary to provide a non-contaminating auxiliary heat source to the FRP material during the installation and curing processes.

Moreover, resin and adhesive materials should not be applied to wet surfaces unless they are specifically formulated for this purpose, and FRP materials should not be applied to concrete surfaces that are subject to condensation, vapor transmission, or water ingression unless such issues are clearly addressed by the system design and the resin systems are specifically formulated for use in such conditions.

4.2.4.2.2 Equipment

As with ACI, ISIS suggests that all equipment used in the installation process should be clean and in good operating condition, and should be accessible for inspection by the project engineer. The contractor should have qualified personnel sufficiently trained to install and operate system-specific equipment such as resin impregnators, sprayers, lifting/positioning devices, and winding machines. All materials, and supplies, and personal protective equipment should be available in sufficient quantities to allow safe construction continuity and quality assurance.

4.2.4.2.3 Surface Preparation

For concrete surface preparation details, ISIS refers to Clause A16.1.4 of the S6-06 bridge code (2006) and Clause 14.9 of the S806-02 building code (2002). ISIS suggests that the concrete surface preparation should be inspected and approved by the engineer or the supervisor prior to the application of the FRP. Some specific recommendations are as follows. ISIS recommends that the surface preparation should be performed according to the FRP system manufacturer's guidelines. Surfaces in good condition may only require cleaning, but all signs of deterioration, including that caused by steel corrosion, should be repaired prior to the application of the FRP system. Before the repair process, the concrete surface must be free of particles and pieces that no longer adhere to the structure, and cleaned from oil and other contaminants. An inspection and approval of the surfaces is then required before the repair may begin. During the repair, the concrete surfaces must be repaired or reshaped in accordance with the original section, and sections with sharp edges must be rounded to a minimum radius of 1.4 in (35 mm), in accordance with the S6-06 bridge code (2006), before installing the FRP system.

Cracks wider than 0.01 in (0.3 mm) should be pressure injected with epoxy in accordance with the guidelines of ACI 224.1R (2007), and smaller cracks in aggressive environments may also require epoxy injection to prevent corrosion of steel reinforcement. All surface repairs should meet the requirements of the FRP system manufacturer.

For bond-critical applications, the method of surface preparation should depend on the existing surface condition. A smooth concrete surface can be sandblasted or otherwise abraded until aggregates become visible according to the relevant preparation recommendations. After blast cleaning, the surface should be protected within an appropriate amount of time prior to the FRP installation. Areas that are very rough can be leveled using a material approved by the manufacturer and/or the engineer, and out-of-plane variations should be within the tolerances recommended by the FRP system manufacturer, where small holes and voids should be filled with putty or a mortar polymer. The maximum allowed depth of depressions (see Clause A16.1.4 of the S6-06 bridge code) is shown in Table 4.2.1 (ISIS Table 8.1).

Table 4.2.1 – Maximum depth of depressions on the concrete surface

Type of FRP	Max. depth for length of 12 in (0.3 m), in	Max. depth for length of 80 in (2.0 m), in
Plates ≥ 0.04 in (1.0 mm)	0.16 in (4.0 mm)	0.39 in (10.0 mm)
Plates < 0.04 in (1.0 mm)	0.08 in (2.0 mm)	0.24 in (6.0 mm)
Sheets	0.08 in (2.0 mm)	0.16 in (4.0 mm)

All dirt, oil, existing coatings, or other matter that could interfere with the bond of the FRP should be removed. The concrete surface must have a tensile and shear strength high enough to ensure an efficient bonding; Clause A16.1.4 of the S6-06 bridge code requires a minimum tensile strength of 218 psi (1.5 MPa) as measured by a pull-off tension test in accordance with ASTM D4541 (2002). Rectangular cross-sections should have corners rounded or reshaped to a minimum radius of 1.4 in (35 mm), and roughened corners should be smoothed with an epoxy gel.

All surfaces to which the strengthening system will be applied should be dry according to the FRP system manufacturer's requirements, and the moisture content should be evaluated according to the requirements of ACI Standard 503.4 (2003).

For contact-critical applications such as confinement, the surface preparation should guarantee a continuous contact between the concrete and the FRP confinement system. Rounding of corners, filling holes and eliminating depressions are most critical.

For all types of applications, if water seepage through the concrete is found, special resins designed for this bond condition must be used. Both the engineer and the contractor must verify that the pressures will neither cause debonding nor affect the integrity of the reinforced structure. If FRP is to be installed underwater, the method to be used must be prescribed in detail according to the recommendations of the manufacturer and must be approved by the engineer.

4.2.4.2.4 Mixing of resins

When mixing resins, all components should be mixed at a proper temperature and in the correct ratio until there is uniform and complete mixing and the product is free from trapped air. The resin mixing should be done in quantities sufficiently small to ensure that all mixed resin will be used within the resin's pot life.

4.2.4.2.5 Application

ISIS references Clause A16.1.3 of the S6-06 bridge code for FRP application. ISIS recommends that all materials, including primer, putty, saturating resin and fibers, are part of the same system. ISIS notes that appropriate installation procedures should depend on the specific FRP system and the structure for strengthening.

For hand-applied wet lay-up systems, manufacturer recommendations must be followed in all application steps, and the following additional recommendations are suggested. Putty should be used only to fill voids and smooth surface discontinuities prior to the application of other materials, and should be used in an appropriate thickness and in sequence with the primer as recommended by the FRP manufacturer. Primer should be placed uniformly on the prepared surface at the specified rate of coverage, and should have sufficiently low viscosity to penetrate the surface of the concrete substrate. Rough putty edges or trowel lines should be smoothed before installation of the fiber sheets.

Hand-applied, wet lay-up materials include dry as well as pre-impregnated fiber sheets and fabrics used with a saturating resin applied on site. In the latter case, the saturating resin should be applied uniformly to all prepared surfaces where the FRP is to be placed. The resin should have sufficiently low viscosity such that the fiber reinforcement becomes fully impregnated with resin prior to curing. Once the FRP is applied, entrapped air under the sheet or between layers should be released or rolled out before the resin sets. In doing so, it is recommended to work the FRP materials parallel to the fibers, proceeding in one direction from the center or from one extremity and to avoid any backward and forward movements. A protective finish compatible with the proposed system should be applied in accordance with the manufacturer's recommendations.

When using pre-cured systems (i.e. surface bonded plates), the pre-cured laminate surfaces to be bonded should be cleaned and prepared in accordance with the manufacturer's recommendations. Adhesive should be applied uniformly to the prepared surfaces where the laminates are to be placed. Care should be taken to use an application method to avoid entrapping air under the laminate, because such a condition is difficult to detect as well as to rectify. In contrast to hand-applied, wet lay-up materials, stacking multiple layers of FRP plate is usually not permitted, except for the overlapping portion of prefabricated L-shaped stirrups. At intersections of FRP plates, care should be exercised to minimize curvature; grooving the concrete for the layer underneath is sometimes used to allow full contact between the plate and the concrete surface underneath.

For FRP material used to wrap the base of a reinforced concrete column that is in contact with the ground, the wrapping should extend a minimum of 20 in (500 mm) below the ground surface to prevent water and air infiltration. ISIS also states that FRP stirrup strips for shear reinforcement must be anchored in a satisfactory manner at both extremities; this anchorage is to be specified by the designer.

4.2.4.2.6 Alignment of FRP materials

ISIS notes that the alignment of the FRP material is critical, and the ply orientation and stacking sequence must be specified in the design prior to installation. As with ACI, ISIS specifies that sheet and fabric materials should be handled in a manner to maintain the fiber straightness and orientation, as even small variations in the intended orientation angle can cause a reduction in strengthening. Any observed deviation in angle for an installed FRP are to be approved by the project engineer.

4.2.4.2.7 Multiple plies and lap splices

ISIS allows the use of multiple layers of FRP materials, provided that all layers are fully impregnated within the resin system, that the resin shear strength is sufficient to transfer the shearing load between layers, and that the FRP-to-concrete adhesive strength is sufficient. However, the project engineer or the manufacturer may limit the maximum number of consecutive layers, and/or define the installation period between successive layers. When several superposed layers of FRP materials are required, care must be taken not to move or otherwise disturb the preceding layers where the resin has not set. In the absence of other prescriptions, a minimum overlap length parallel to the fibers of 6 in (150 mm) is suggested. If an interruption of the FRP system laying up process occurs, inter-layer surface preparation such as cleaning or light sanding may be required.

4.2.4.2.8 Curing

FRP materials are to be cured according to the recommendations of the manufacturer. Unless otherwise specified, ISIS recommends the following curing provisions. A minimum curing time of 24 hours should be allowed before further work is performed, unless the curing process is accelerated by heating (via a chemical reactant or other external supply). For the entire curing duration, the temperature must be maintained above the minimum required curing temperature; condensation on the surface must be prevented; and chemical contamination from gases, dust or liquid sprays must be prevented during curing. ISIS notes that although successful rehabilitation of beams under simulated traffic loads have been reported (Recuero et al., 2004), mechanical stresses should be minimized during curing.

4.2.4.2.9 Protective coating

When the surface of the FRP materials is sufficiently dry or hard, a protective coating and/or paint compatible with the installed reinforcement system can be added. A minimum of 24 hours should be allowed for the protective coating/paint to dry, or as recommended by the manufacturer. Further, the contractor is required to provide a certificate of compatibility of the protection system prepared by the FRP manufacturer, and the contractor is to provide a guarantee for the performance of the proposed protection system for the expected exposure conditions. The protection system must provide sufficient protection against ultraviolet radiation. It may include a wearing layer if the FRP reinforcement materials are expected to be subjected to abrasive effects. The wearing layer must not be considered to be structural reinforcement, and it must be inspected and maintained regularly. If the FRP reinforcement must be protected against fire, the protection system proposed must be approved by the engineer, and the contractor or manufacturer must guarantee its compliance.

4.2.4.3 AASHTO

AASHTO requirements are written in a similar manner to those given by ACI and ISIS. According to AASHTO, procedures for the installation of FRP systems are developed by the manufacturer and can vary between different systems. Procedures may also vary depending on the type and condition of the structure to be strengthened. AASHTO notes that the application of

FRP systems will not stop the ongoing corrosion of existing steel reinforcement, and the cause of any corrosion should be addressed and corrosion-related deterioration should be repaired prior to application of any FRP system.

4.2.4.3.1 Temperature, humidity, and moisture considerations

When temperatures exceed 90°F, the epoxy may be difficult to apply due to an accelerated hardening rate. AASHTO thus recommends that work should be scheduled to avoid high temperatures. If it is necessary to apply epoxy compounds in high temperatures, the work should be supervised by a person experienced in applying epoxy under such conditions. AASHTO also notes that epoxy systems formulated for elevated temperatures are available and should be considered (see ACI 530R-93). At temperatures below 40°F, application difficulties may also occur due to a deceleration of the rate of curing, and the presence of frost or ice crystals may be detrimental to the bond between the FRP and the concrete.

4.2.4.3.2 Surface preparation

Prior to FRP application, the concrete surface should be prepared to a minimum concrete surface profile (CSP) 3. Proper preparation and profiling of the concrete surface is necessary to achieve the specified bond strength; improper surface preparation can lead to debonding or delamination. AASHTO recommends that localized out-of-plane variations, including form lines, should not exceed 1/32 inch or the tolerances recommended by the FRP system manufacturer, whichever is smaller. Bug holes and voids are to be filled with epoxy putty. It is recommended that surface preparation be accomplished using abrasive or water-blasting techniques, and all contaminants that could interfere with the bond between the FRP system and concrete substrate should be removed. When fibers are wrapped around corners, corners should be rounded to a minimum radius of 1/2 inch to prevent stress concentrations in the FRP system as well as voids between the FRP and the concrete. Rough edges can be smoothed by grinding or with putty.

4.2.4.4 JSCE

4.2.4.4.1 Temperature, humidity, and moisture considerations

When bonding or wrapping with continuous fibers, at each stage of the work it should be verified that environmental conditions are suitable. Suitable conditions for epoxy resin applications are a 9°F (5°C) or higher temperature and humidity no more than 85%. In lower temperatures, the construction site should be warmed or a low-temperature primer and resin may be used. If the surface of the concrete is not dry, special primers for wet surfaces should be used. It should also be verified that the concrete surface preparation is suitably performed; that the mixing and coating of primer are appropriately performed; and that the mixing and coating of a smoothing agent are appropriately performed.

To prevent improper hardening of the primer and smoothing agent, the materials should be applied to a dry surface. After application, the primer and smoothing agent should be allowed to harden until firm, and should be checked visually and by touch to make sure there is no dust or moisture on the surface. If there is condensation or other moisture on the surface before initial hardening, indicated by whitening, the area should be wiped with solvent or the effected portion of primer or smoothing agent removed with sandpaper.

4.2.4.4.2 Surface preparation

Prior to application of FRP, JSCE recommends that construction defects, remarkable deterioration, and surface cracking in the concrete should be repaired. These defects include problems such as rock pockets, honeycombs, level differences or other surface imperfections, which deviate from a smooth surface and can reduce the effectiveness of the FRP strengthening.

When continuous fiber sheets and continuous fiber strands are placed perpendicular to corner angles, the angles should be rounded by chipping, polishing, or the use of a smoothing agent.
To ensure proper bond between the FRP and the concrete surface, deteriorated layers, oils and other contaminants should be removed from the surface.

4.2.4.4.3 Application

When using continuous fiber sheets, JSCE notes that it is critical that the sheets are attached with the specified position, direction, and number of plies. A working diagram matching the actual structure should be prepared based on the design. The diagram should clearly identify the reference point for attachment, the overlap splice positions and the number of plies to enable the sheets to be attached properly. It must also be ensured that the sheet is bonded or sealed securely to the concrete surface, and that the resin is suitably mixed and applied and has thoroughly impregnated the sheet. This is particularly important in the overlap splice sections, where the impregnation resin should thoroughly penetrate between the fibers and sheets. After attaching the continuous fiber sheets, an inspection should be done visually or through sounding to verify the absence of lift, swelling, peeling, slackness, wrinkles, and voids in the epoxy resin impregnation.

When using continuous fiber strands, it must be verified that the strand winding interval is appropriate; the strand winding tension is constant; the strand winding speed is appropriate; the strands are thoroughly impregnated with resin; that the resin has been suitably mixed and applied; and that the impregnation resin is cured thoroughly. JSCE notes that if carbon fiber strands are wound by hand, the tension force applied is not constant, resulting in variations of stress distribution in the strands after completion. Moreover, since the winding speed is not constant, there may also be variations in the degree of permeation of the impregnation resin. These issues may affect the tensile strength of the strands. For these reasons, the use of a machine to wind the strands, to control the winding interval, tension and speed, is recommended.

JSCE notes that when wrapping FRP around corners, it is important that a sufficient radius of curvature is maintained, as when the radius of curvature is small, as noted by AASHTO, stress concentrations occur, decreasing the effective tensile strength of the FRP. In general, the chamfer radius should be between 0.4 in – 2.00 in (10 mm - 50 mm). However, the type and thickness of FRP used have a large influence on the necessary chamfer radius.

4.2.4.4.4 Lap splices

JSCE states that the required lap length is to be determined through testing in accordance with JSCE-E 542 (2000). JSCE note that carbon and aramid fiber sheets have been found to require an overlap splice length of approximately 4 in (100 mm) at the lower stress level produced in the splice zone and about 8 in (200 mm) for strengthening for shear capacity and ductility. However, depending on the type of FRP and resin used, the splice strength may be lower than the tensile strength of the FRP and failure may occur in the bonded layers of the splice even if the overlap length is long.

When only one layer of continuous fiber sheet is used for reinforcement, variations in overlap splice strength due to construction errors may significantly influence the splice strength. In such cases, it is recommended to elongate the overlap splice length and to attach one more layer of FRP over the splice section. When more than one layer is used, the overlap splices should not be placed at the same section, since this reduces the overlap splice strength. Overlap splices should not be placed at locations subjected to large bending moments.

4.2.4.4.5 Curing

After applied, resin should be cured for a suitable period of time before the next sheet is attached. Before the initial setting of the impregnation resin, JSCE recommends that the surface should be protected with vinyl sheets from rain, dust, and sudden climatic changes. It must also be verified that the impregnation resin is cured thoroughly.

4.2.4.4.6 Anchorage length

JSCE notes that an item requiring verification is the end anchorage of FRP, for which it should be confirmed that the strand is wound with the required number of turns at the section to be anchored. The required number of turns for anchorage should be determined through testing. JSCE states that one to two wraps is sufficient anchorage for carbon fiber strands composed of 12,000 filaments.

If mechanical anchorage is to be used, it can be by way of anchor bolts and plates, and should be verified by confirming that the anchorage detail has sufficient strength to prevent anchorage failure. JSCE suggests that when reinforcing bridge pier foundations, mechanical anchoring with anchor plates and bolts may be necessary because attaching FRP sheets to the footing surface is generally insufficient anchorage. When FRP is bonded to the sides of beams for shear reinforcement, anchorage should be provided by anchor bolts and plates.

4.2.4.5 TR55

4.2.4.5.1 Temperature, humidity, and moisture considerations

Before application of FRP, the concrete surface should be dry for normal applications. If surface dryness is not achievable, a suitable epoxy adhesive for non-dry surfaces should be selected.

Suitable environmental conditions including temperature, relative humidity and surface moisture must be maintained during surface preparation, strengthening system application, and the curing period. During surface preparation, environmental control consists of a system to extract dust from the work area and the exclusion of any material that might contaminate the prepared surface. During installation, a clear access path from the adhesive work area to the location of the concrete surface should be maintained in order to minimize contamination risk. During the curing period, the adhesive temperature must be maintained within the specified limits. The work area should be kept dry.

4.2.4.5.2 Surface Preparation

TR55 suggests that a trial run of the surface preparation process should be conducted to determine the best technique for the FRP system to be used. As with other codes, TR55 notes that the concrete surface must be cleaned to remove contaminants. Even new concrete should be cleaned to remove mold release agents and curing membranes. The preparation process should remove the surface layer to expose small particles of aggregate without causing damage to the substrate. The surface should not be polished or roughened excessively. Sharp edges, shutter marks or other irregularities should be removed to achieve a flat surface.

Various preparation techniques can be effective, including wet, dry, and vacuum-abrasive blasting; high-pressure washing, with or without emulsifying detergents, and using biocides (where necessary); steam cleaning alone or in conjunction with detergents; and, for smaller areas, mechanical wire brushing or surface grinding. TR55 warns that special care should be taken when using some methods, including mechanical impact methods such as needle gunning and bush hammering, which are often too aggressive and may cause micro-cracks and/or an irregular concrete texture.

Washing techniques may be ineffective in some cases, and can simply spread the contaminants further. The use of solvent-based and sodium hydroxide-based products in the form of a gel or poultice can be effective in drawing out the contaminants, but such products must be then completely removed from the surface. If wet grit-blasting is done, the concrete surface must be allowed to completely dry before proceeding. TR55 further recommends vacuum dry-blasting over "open" blasting, the former of which is safer for workers and the environment.

TR55 states that, prior to FRP installation, defects in the concrete surface should be repaired, and if cementitious repairs are used, they should be allowed 28 days to cure. TR55 notes that it is important that the prepared surface should allow application of the resin in a layer of uniform thickness; the thickness of the adhesive layer is commonly between 0.04 – 0.20 in (1 – 5 mm). To facilitate this, the surface should be smoothed by removing any steps and filling hollows with a suitable repair mortar. Minor imperfections in the concrete surface can be treated with epoxy materials which can be applied in thin layers. The flatness of the surface should be such that the gap under a 3.3 ft (1 m) straight-edge does not exceed 0.20 in (5 mm). When fabric is to be wrapped around corners, the corners should be rounded to a minimum radius of 0.6 in (15 mm), or as recommended by the manufacturer. TR55 notes that some bonding systems require the use of a primer to be applied once the surface preparation is complete if so, primer should be applied in strict accordance with the manufacturer's instructions.

The final assessment of surface quality can be made with pull-off tests. If primer was used, the primed surface should be tested. Here, a minimum of three tests are carried out, as described in BS 1881: Part 207 (1992).

4.2.4.5.3 Mixing of resins

All equipment used for the mixing and application of the adhesive and materials should be kept clean and maintained in good operating condition, and all operators should be suitably trained in the use of such equipment. The mixing and application of the adhesive should be in accordance with the manufacturer's instructions. For accurate mix proportioning, pre-batched quantities of resins and hardeners should be used. The materials should be mixed thoroughly per the supplier’s instructions. The volume of adhesive mixed at one time should be such that it may be applied within the pot life of the adhesive; adhesive remaining at the end of the specified pot life must be discarded.

4.2.4.5.4 Application

If the concrete surface is to be strengthened using FRP plates, the mixed adhesive is to be applied to the bonding area by hand, using plastering techniques. The thickness of the adhesive should be maintained from 0.04 – 0.08 in (1–2 mm). Before applying the adhesive to the FRP plate, the plate surface should be prepared in accordance with the manufacturer's recommendations; in general, this involves application of light abrasion and cleaning with a solvent. No additional treatment is required for materials with an additional peel ply which, upon removal, exposes a clean surface with the appropriate roughness. The adhesive layer should be applied to the plates to form a slightly convex profile across the plate. Extra thickness along the center-line helps to reduce the risk of void formation.

If FRP fabric is to be applied, a hand-held foam roller or brush can be used to apply the bonding adhesive to the concrete surface. The adhesive layer should be evenly applied to saturate the concrete surface· and adhere to the FRP. Dry fabric can be directly applied to the resin-saturated concrete surface without adhesive being applied to the fabric. For wet fabric, the resin must be applied to the fabric before it is installed. The resin can be applied to the fabric using handheld foam rollers or brushes, or an impregnation machine.

4.2.4.6 CNR

4.2.4.6.1 Temperature, humidity, and moisture considerations

CNR recommends that FRP should not be installed in very moist environments, as a high degree of humidity may delay resin curing and affect strengthening effectiveness, especially for wet lay-up applications. Moreover, FRP should not be applied to substrates having a surface humidity greater than 10%, as such conditions could delay the penetration of the primer and generate air bubbles that could compromise bond. Substrate humidity can be evaluated with a hygrometer for mortar or by employing absorbent paper. FRP material should also not be applied if temperatures are too low, as resin curing and fiber impregnation could be compromised. It is also recommended not to install FRP when the concrete surface is heavily exposed to sunlight. CNR suggests that the range of suitable temperature for FRP application is generally within 50-95°F (10-35°C). In low temperature environments, artificial heat can be provided. If resin curing takes place under rainy conditions, protective measures should be employed to ensure proper curing.

4.2.4.6.2 Surface preparation

Prior to FRP installation, CNR recommends that the soundness of the concrete substrate is checked, and in any case, concrete compressive strength should not be less than 2.18 ksi (15 N/mm^2) below which the FRP strengthening may not be effective. CNR states that deteriorated concrete should be removed, and an assessment of the existing steel reinforcing bars should be made if exposed. Corroded steel bars are to be protected against further deterioration. Once the deteriorated concrete has been removed, suitable measures taken to prevent further corrosion, and additional protective measures, if needed, to prevent other sources of concrete degradation (such as water leakage), concrete restoration using shrinkage-free cement grouts is to be performed. A concrete surface roughness with profile differences greater than 0.4 in (10 mm) is to be leveled with a compatible epoxy paste, and specific filling materials are to be used for unevenness greater than 0.8 in (20 mm). Cracks wider than 0.02 in (0.5 mm) should be stabilized using epoxy injection methods before FRP strengthening can take place. Once the flatness and soundness of the concrete is restored sandblasting should be done to provide a roughness of at least 0.01 in (0.3 mm); level of roughness can be measured by suitable instruments such as a laser profilometer or an optical profile-measuring device. All inside and outside corners and sharp edges shall be rounded or chamfered to a minimum radius of 0.8 in (20 mm). Finally, the concrete surface should be cleaned to remove any dust, laitance, or any other bond-inhibiting material.

4.2.4.6.3 Application

Proper fibers alignment is to be observed, and waving of FRP reinforcement must be avoided during installation. If carbon fiber is used and there is potential for direct contact between the carbon and existing steel reinforcement, insulating material should be installed to prevent galvanic corrosion. CNR recommends that an anchorage length of at least 8 in (200 mm) be used for the end portion of FRP systems. Alternatively, mechanical connectors may be used.

4.2.4.6.4 Witness areas

If semi-destructive tests are planned, it is suggested to provide additional strengthening areas ("witness areas") in properly selected parts of the structure of size of at least 20 in X 8 in (500 mm x 200 mm), with a minimum extension of 155 in^2 (0.1 m^2), but not less than 0.5% of the overall area to be strengthened. Witness areas are to be strengthened at the same time of the main FRP installation, using the same materials and procedures. In addition, witness areas are to be exposed to the same environmental conditions of the main FRP system and should be uniformly distributed on the strengthened structure.

4.2.4.6.5 Protective coating

The FRP system should be protected from direct sunlight, which may produce chemical-physical alterations in the epoxy matrix. This can be achieved with protective acrylic paint, provided that the composite surface is cleaned with soap beforehand. Alternatively, a better protection can be achieved by applying a plaster or mortar layer (preferably concrete-based) to the installed system.

For fire protection, two different solutions may be adopted: the use of intumescent panels or the application of protective plasters. In both cases, the manufacturer is to indicate the degree of fire protection provided, as a function of the panel/plaster thickness. The panels, generally based on calcium silicates, are to be applied directly on the FRP system, provided that fibers will not be cut during their installation. Fire-protective coatings that can keep the FRP temperature below 176°F (80°C) for 90 minutes are also available.

4.2.4.7 Summary of installation procedures

Coverage of installation procedures varies widely among codes. AASHTO and CNR are primarily design codes with little coverage of many aspects of FRP installation. The limited areas that are covered by the two codes include surface preparation of the concrete substrate and site environmental conditions at the time of installation. CNR covers three additional topics; minimum width for crack injection repair; lap splices; and temporary protection of the FRP system during resin curing. AASHTO's lack of coverage is supplemented by NCHRP 609, which provides description of a recommended installation procedure not found in AASHTO. JSCE is similarly brief in coverage, and includes site environmental conditions at the time of installation; surface preparation of the concrete substrate; mixing of resin; and lap splicing. TR55 covers the same items as JSCE, with the exception of lap splicing. TR55 also adds detail on FRP application in the form of primer/putty application and wet lay-ups.

The most comprehensive installation procedure detailed in the reviewed codes is provided by ACI and ISIS. The two codes have similar coverage, although ISIS provides more quantitative limits. The topics covered by ACI and ISIS include site environmental conditions at installation; use of equipment; surface preparation; resin mixing; application of the FRP system; protective coatings; alignment of FRP materials; multiple plies and lap splices; resin curing; and temporary protection. Comparative summaries are provided in Tables 4.2.2 – 4.2.4. Table 4.2.2 compares maximum allowable crack width beyond which concrete injection is required as part of surface repair prior to FRP application. Table 4.2.3 compares the minimum allowable lap splice length. Table 4.2.4 compares code recommendations for minimum radius for roundness of concrete corners for FRP application.

Table 4.2.2 – Maximum allowable concrete crack width beyond which injection is required

ACI	ISIS	CNR
0.01 in (0.3 mm)	0.01 in (0.3 mm)	0.02 in (0.5 mm)

Table 4.2.3 – Minimum allowable lap splice length

ISIS	JSCE	CNR
6 in (150 mm)	4 in (100 mm) at lower stress levels 8 in (200 mm) for strengthening of shear capacity and ductility	8 in (200 mm)

Table 4.2.4 – Minimum radius for roundness of concrete corners

ACI	ISIS	AASHTO	JSCE	CNR	TR55
0.5 in (13 mm)	1.4 in (35 mm)	0.5 in (13 mm)	0.4 – 2.0 in (10-50 mm)	0.8 in (20 mm)	0.6 in (15 mm)

4.3 Inspection, Evaluation, and Acceptance

4.3.1 Introduction

Section 4.3 reviews inspection procedures, evaluation, and acceptance criteria for the installed FRP strengthening system. An assessment survey inspection, as part of a long-term maintenance program, is covered in Section 4.44.

Inspection evaluation procedures cover the materials used, conformance to the strengthening design plan, and post-installation inspection of the finished product to detect any deficiencies such as delamination, air bubbles, fiber waviness or misalignment, and adhesion strength. Evaluation of deficiencies, if any, is conducted based on acceptance criteria that sets limits between acceptable minor deviations and deficiencies that require corrective repairs.

4.3.2 Inspection

4.3.2.1 ACI

ACI states that FRP systems and all associated work should be inspected as required by applicable codes. The inspection should be conducted by or under the supervision of the project engineer or a qualified inspector. The qualified inspector should look for compliance with the design drawings and project specifications. During the installation of the FRP system, daily inspection should be conducted and should note, as applicable:

- Date and time of installation;
- Ambient temperature, relative humidity, and general weather observations;
- Surface temperature of concrete;
- Surface dryness per ACI 503.4 (2003);
- 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 or precured laminate batch number(s) and approximate location in 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 resin cure;
- Conformance with installation procedures;
- Pull-off test results: bond strength, failure mode, and location;
- FRP properties from tests of field sample panels or witness panels, if required;
- Location and size of any delaminations or air voids; and
- General progress of work.

The inspector should provide the inspection records and witness panels. ACI defines witness panels as structural samples manufactured on site as true representatives of the materials and FRP system used. They are to be kept under the same conditions as the structure for future testing and evaluation. In some cases, FRP system is applied to areas of the structure not in need of strengthening for future testing and assessment. Records and witness panels should be retained for a minimum of 10 years or a period specified by the project engineer. The installation contractor should retain sample cups of mixed resin and maintain a record of the placement of each batch.

4.3.2.2 ISIS

ISIS divides inspection tasks into several phases; inspection of the concrete substrate; materials inspection; and inspections before installation, during installation, and at completion of the project.

4.3.2.2.1 Contractor's QC responsibilities

ISIS requires that the FRP material suppliers, the installation contractors and all others associated with the FRP strengthening project maintain a comprehensive quality assurance (QA) and quality control (QC) program (per Annex A16.2 of S6-06 bridge code). Quality assurance (QA) is achieved through a set of inspections, measurements, and applicable tests to document the acceptability of the surface preparation and the installation of FRP, and the quality control (QC) should cover all aspects of the strengthening project, and will depend on the size and complexity of the project.

The FRP material suppliers are responsible for training the installation crew and as well as certifying their competency for the surface preparation and installation of the FRP materials. ISIS notes that only qualified inspectors should be used.

FRP materials should be qualified on the basis of the engineer's plans and specifications. ISIS notes that two types of specifications are possible; descriptive or performance. In descriptive specifications, the engineer specifies the length, width, orientation, installation sequence and other requirements of a particular, selected FRP material, and perhaps acceptable equivalents. In performance specifications, the engineer specifies requirements in term of strength, stiffness, or other necessary properties and characteristics, and the contractor is responsible for selecting an appropriate FRP system and submitting it for approval.

The FRP manufacturer is to provide documentation demonstrating that the proposed system meets all design requirements such as tensile strength, type of fibers and resin, durability, resistance to creep, bonding to substrate, and glass transition temperature. Independent tests of the FRP constituent materials and laminates fabricated with them are essential and should be mandatory.

Selection of contractors should be based on evidence regarding their qualifications, their demonstrated skills and ability through experience or training for FRP strengthening projects.

4.3.2.2.2 Inspection of concrete substrate

The concrete surface should be inspected and tested before application of the FRP material. The inspection should include an examination for surface smoothness, protuberances, holes, cracks, corners, and other imperfections and characteristics. Pull-off tests should be performed to determine the tensile strength of the concrete for bond-critical applications, in accordance with Clause A16.1.4 of the S6-06 bridge code (2006). The degree of surface dryness, including the potential for condensation, should be in accordance with the criteria established by the FRP manufacturer.

4.3.2.2.3 Inspection before installation

Inspections of the FRP material is to be conducted before, during and after installation. The inspection program should cover such aspects such as raw materials, the presence and extent of delamination, the cure of the installed system, adhesion, laminate thickness, fiber alignment and material properties.

Before construction, the FRP material supplier should submit a certification and identification of all FRP materials to be used, and the installation procedure should be submitted with information on shelf life and resin working time related to temperature. Performance tests on the supplied materials should be performed according to the QC test plan and should meet the requirements specified in the engineer’s performance specifications. Testing may include parameters such as tensile strength, glass transition temperature, gel time, pot life, as well as the adhesive shear strength.

4.3.2.2.4 Inspection during installation and at completion

During construction, special care should be taken to keep all records on the quantity of mixed resin during a one-day period, the date and time of mixing, the mixture proportions, and identification of all components, ambient temperature, humidity, and other factors that may affect resin properties. These records should also identify the FRP sheets used each day, their location on the structure, the ply count and direction of application, and all other relevant information in accordance with Clause A16.2.3.3 of the S6-06 bridge code.

Sample cups of mixed resin should be prepared according to a predetermined sampling plan and retained for testing to determine the level of cure. Sample FRP plate specimens should be fabricated according to a predetermined sampling plan, under the same ambient conditions and procedures used to apply the FRP material to the concrete surfaces. Performance tests on these FRP specimens may be conducted as needed. Moreover, a visual inspection of fiber orientation, and waviness for specific FRP material systems, should be performed. Fiber misalignment of more than 5 degrees (1/12 slope) from the specified angle should be reported to the engineer.

4.3.2.2.5 Inspection at completion

At project completion, the inspector is to check for delamination, the overall quality of cured system, and test for adhesion. Moreover, a record of all final inspection and test results related to the FRP material should be retained. It should include a description of any delamination and repairs, on-site bond tests, anomalies and correction reports as well as all test results from designated testing facilities. Samples of the cured FRP materials should be retained by the engineer. The required reports and tests are specified in Clause A16.2.3.4 of the S6-06 bridge code (2006).

4.3.2.3 AASHTO

AASHTO recommends that systems considered for FRP strengthening should be inspected by a licensed engineer or qualified inspector knowledgeable in FRP systems and installation procedures. The following should 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 the resulting surface profile using ICRC surface profile-chips;
- Qualitative description of surface cleanliness;
- Type of auxiliary heat source, if used;
- Widths of cracks not injected with epoxy;
- Fiber or pre-cured laminate batch number(s) and approximate locations in the structure where each was used;
- 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 resin curing;
- Conformance with installation procedures;
- Location and size of any delaminations or air voids;
- General progress of work;
- Level of resin curing, in accordance with ASTM D2582 (2009);
- Adhesion strength.

4.3.2.4 JSCE

JSCE recommends that inspections are conducted at each stage of the work, which are: material inspections when first received; inspections of the storage condition of these materials; surface preparation inspections; and, inspections of the bonding or jacketing condition of the FRP after the work is completed. Further description of these inspections is provided below.

4.3.2.4.1 Materials and storage inspection

When material is first received, the FRP, primer, smoothing agent, impregnation resin, and other materials should be inspected for quality and damage. Inspection of materials should be done in accordance with the quality assurance sheet, test results, or other relevant documents issued by the manufacturer. If the materials have suffered damage during shipment, long-term storage at the site, or during construction, before use, they should be tested to confirm quality.

The storage condition of the materials is to be inspected as well. In general, materials should be stored indoors in a well-ventilated location away from direct sunlight, flame, and rain, and at appropriate temperature and humidity conditions. Laws relating to storage should be strictly observed.

4.3.2.4.2 Inspection of the existing structure

At the site, a detailed inspection of the existing structure to be strengthened should be conducted, with attention to the following:

- Existing structural configuration and site conditions. Due to construction errors or undocumented changes, existing structures may not necessarily have been completed as specified in the available design documents. The dimensions of the existing structure should be verified in advance. Moreover, as climatic conditions affect curing and bond, it is necessary to determine the wind, sunshine, temperature changes and other conditions expected at the site.

- Necessity for impact protection. If the applied FRP sheets may be damaged by impacts, it is necessary to study whether surface protection should be implemented. Accordingly, the potential for damage to a strengthening system by impacts and the degree of damage to the existing structure from impacts should be estimated.

- Surface quality. Since bond strength is crucial for effectiveness, the degree of surface deterioration and damage of the existing concrete surface should be carefully determined, so necessary repair measures can be implemented.

- Cause and degree of existing deterioration. The deterioration progress of the upgraded concrete structure depends on the type and degree of the causes of deterioration. Accordingly, when damaged concrete structures are upgraded, it is necessary to pre-examine the type and degree of external factors causing deterioration. Particularly, when concrete damaged by alkali- aggregate reaction is strengthened, volumetric expansion may occur after construction. Therefore, the quantity and area of FRP coverage should be determined by taking account of the amount of residual expansion.

4.3.2.4.3 Inspection before, during, and after installation

Before application of FRP, concrete surface preparation is to be inspected with respect to the completeness of sectional restoration work, surface smoothness, processing of corner angles, primer coating, and smoothness.

During and after construction, FRP should be inspected for attachment position, lifting, peeling, slackness, wrinkles, overlap splice length, number of plies, and quantity of the resin coating. Moreover, a bond strength test should be conducted as needed. If the scale of construction is large or construction conditions are severe, it is best to conduct confirmation tests using test specimens fabricated at the site. The same materials as those used on site should be used for tests, and the tests should be performed on the concrete at the site. However, if it is difficult to perform the test on the concrete at the site, the test may be performed on a slab specimen with concrete properties representative of the site concrete. If wound on site, FRP strands are to be inspected for winding position, winding interval, winding tension and winding speed, and that fibers are thoroughly impregnated with resin.

4.3.2.5 TR55

TR55 states that its recommendations for inspection, evaluation, and acceptance are not intended to be a specification, but are meant to suggest what significant points should be included in a specification. TR55 suggests that the manufacturer should supply characteristic values of the mechanical properties to be used for design purposes (e.g. strength, elastic modulus, etc.), which TR55 defines as the mean value minus 2 standard deviations. It further suggests that sufficient tests should be carried out at regular intervals to ensure that the values reported are statistically valid.

4.3.2.5.1 Material quality control requirements

TR55 suggests that all materials used should have been manufactured under an approved quality scheme, such as ISO 9000, and conform to relevant ISO specifications, Euronorms, or other equivalent international standards. In addition, the traceability of all materials should be ensured and materials should be supplied with a certificate of conformity to the relevant standards. Further, all external or independent testing to determine material properties should be carried out in approved laboratories in accordance with relevant international standards or by the manufacturer under an approved quality scheme. When no international standards exist, an industry or company standard or method with a recognized history should be used.

4.3.2.5.2 Inspection before installation

Testing should consist of visual checks on the basic materials and where appropriate, physical tests on the finished elements as detailed below. For fabric materials, the properties of a specified width of finished material should be checked by testing samples. The frequency of testing should be stated in the quality plan. A minimum of one sample should be taken at the start and finish of each production run. Minimum properties to test for should include unit weight, elastic modulus, and tensile strength. Samples of FRP may be made into laminates, using the appropriate specified resin, and the laminate then tested to determine composite properties. Where appropriate, properties should be determined in the transverse direction as well as in the longitudinal direction. All individual rolls of material should be appropriately labeled.

For pultruded plates, the supply of fibers to the pultrusion line should be monitored on a regular basis, with a frequency of at least once per hour. The speed of processing, processing temperature, and other manufacturing parameters should be maintained within agreed limits and checked and recorded regularly. The properties of the plate should be checked by testing samples, and the frequency of testing should be stated in the quality plan. A minimum of one sample should be taken at the start and finish of each production run. The samples should also be checked for dimensional accuracy, and plates should be marked with a unique batch number at regular intervals.

When received from the supplier, all materials should be accompanied by a certificate of conformity to appropriate standards. When received, all materials should be stored and used strictly in accordance with the manufacturer's instructions. Accurate records should be maintained of all materials used (e.g. delivery notes, batch numbers) and, when required, the ambient conditions (e.g. temperature, relative humidity). If plates are used, visual checks should be carried out to ensure that the plate material is as specified and undamaged. When bonded, the plate should be checked by tapping or other means to ensure continuous adhesion.

4.3.2.5.3 Inspection during installation

For wet lay-up laminates, visual checks should be carried out on mats, unidirectional tapes/fabrics, woven rovings and multi-axial fabrics to ensure uniformity and conformity. The completed laminate should be checked visually for defects. When required by the contract, trial pieces to verify properties such as strength and elastic modulus should be made at the same time and by the same process. Care should be taken to ensure that the trial pieces are representative of the material in the finished unit.

4.3.2.6 CNR

CNR contains little information relevant to this topic, and simply indicates that the inspector’s responsibilities are to check that the quality of the materials are in compliance with the manufacturer specifications; to verify that all materials have been accepted by the construction manager; and to check the results of any experimental tests required.

4.3.3 Evaluation and acceptance

4.3.3.1 ACI

ACI states that FRP systems should be evaluated and accepted or rejected based on conformance to the design drawings and specifications. FRP system material properties, installation within specified placement tolerances, presence of delaminations, cure of resins, and adhesion to substrate should be included in the evaluation. Placement tolerances including fiber orientation, cured thickness, ply orientation, width and spacing, corner radii, and lap splice lengths should be evaluated. ACI further suggests that witness panels and pull-off tests are to be used to evaluate the installed FRP system. In-place proof load testing can also be used to confirm the installed behavior of the FRP-strengthened member (Nanni and Gold 1998).

4.3.3.1.1 Evaluation and acceptance before starting the project

Before starting the project, the FRP system manufacturer should submit a certification of specified material properties and identification of all materials to be used. Based on the needs of the project, additional material testing can be conducted if deemed necessary. Evaluation of delivered FRP materials, in accordance with the QC test plan, may include tests for tensile strength, an infrared spectrum analysis, glass transition temperature, gel time, pot life, and adhesive shear strength. Materials that do not meet the minimum requirements as specified by the licensed design professional should be rejected.

For FRP systems such as pre-cured and machine-wound systems, that do not lend themselves to the fabrication of small, flat witness panels, the project engineer can require test panels or samples to be provided by the manufacturer. During installation, sample cups of mixed resin should be prepared according to a predetermined sampling plan and retained for testing to determine the level of cure.

4.3.3.1.2 Evaluation and acceptance at project completion

Fiber or pre-cured-laminate orientation should be evaluated by visual inspection. Fiber or pre-cured laminate misalignment of more than 5 degrees [approximately 1 in/ft (80 mm/m)] from that specified should be reported to the engineer for evaluation and acceptance.

The cured FRP system should be evaluated for delaminations or air voids between multiple plies or between the FRP system and the concrete. Inspection methods should be capable of detecting delaminations as small as 2 in^2 (1300 mm^2), and may include acoustic sounding (hammer sounding), ultrasonics, and thermography. Delamination size, location, and quantity relative to the overall application area should be considered in the evaluation.

For wet lay-up systems, small delaminations less than 2 in^2 (1300 mm^2) are permissible as long as the delaminated area is less than 5% of the total laminate area and there are no more than 10 such delaminations per 10 ft^2 (1 m^2). Large delaminations, greater than 25 in^2 (16,000 mm^2), can significantly affect the performance of the installed FRP and should be repaired by selectively cutting away the affected sheet and applying an overlapping sheet patch of equivalent plies. Delaminations less than 25 in^2 (16,000 mm^2) may be repaired by resin injection or ply replacement, depending on the size and number of delaminations and their locations.

For precured FRP systems, ACI states that each delamination should be evaluated and repaired in accordance with the instructions of the project engineer (for MDOT projects, this responsibility would fall to the person responsible for QA or the manufacturer's representative).

Upon completion of the repairs, the laminate should be re-inspected to verify that the repair was properly accomplished.

The relative cure of FRP systems can be evaluated by laboratory testing of witness panels or resin-cup samples using ASTM D3418 (2003). 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 retained resin samples. The FRP system manufacturer should be consulted to determine the specific resin-cure verification requirements.

Adhesion strength is a critical inspection parameter. For bond-critical applications such as flexural or shear strengthening, tension adhesion testing of cored samples should be conducted using the methods in ACI 503R (1998), ASTM D4541 (2002), or the method described by ACI 440.3R (2004), Test Method L.1. Successful tension adhesion strengths should exceed 200 psi (1.4 MPa) and exhibit failure of the concrete substrate. Lower strengths or failure between the FRP system and concrete or between plies should be reported to the engineer.

Cured thickness also should be verified. Small core samples, typically 0.5 in (13 mm) in diameter, may be taken to visually ascertain the cured laminate thickness or number of plies; however, taking samples from high stress or splice areas should be avoided. The cored hole can generally be filled and smoothed with a repair mortar or the FRP system putty. However, if required, a 4–8 in (100–200 mm) overlapping FRP sheet patch of equivalent plies may be applied over the filled and smoothed core hole immediately after taking the core sample.

4.3.3.2 ISIS

The categories covered under evaluation and acceptance by ISIS are similar to those covered in ACI. These include a pre-installation evaluation as well as an evaluation at project completion, which involves checking for items such as fiber orientation, delamination, cure of strengthening system, adhesion, laminate thickness, and material properties.

4.3.3.2.1 Evaluation and acceptance before starting the project

For materials qualification, ISIS recommends that FRP systems without fully established properties through appropriate testing should not be considered for use, and all constituent materials should be acceptable by applicable codes and known for their good performance. Mechanical properties of FRP systems should be determined from plate specimens manufactured in a process representative of their field installation, and from tests based on established standards. However, modification of standard procedures may be permitted in order to better represent field assemblies. The specified material qualification programs should include sufficient laboratory testing to measure the repeatability and reliability of critical properties, and testing multiple batches of FRP materials is recommended.

4.3.3.2.2 Evaluation and acceptance at project completion

Visual inspection of fiber orientation, and waviness for specific FRP material systems, should be performed; fiber misalignments of more than 5 degrees from the specified angle should be reported to the engineer.

An inspection of the FRP system for delamination should be conducted after the full cure. Delaminations or other anomalies that are detected should be evaluated considering their size and number relative to the overall application area, as well as their location with respect to structural load transfer. Inspection methods may include acoustic sounding (hammer sounding), ultrasonics, and thermography, and should be capable of detecting delaminations as small as 1 in^2 (600 mm^2) as well as any defects with a leading edge greater than 1 in (25 mm). Repairs are required for delaminations of size greater than 2.3 in^2 (1500 mm^2) or 5% of the total laminate area. Cutting away the affected sheet and applying an overlapping sheet patch of equivalent plies may be considered a repair option for delaminated FRP areas. Appropriate bond lengths must be used to ensure the integrity of the repaired area. The repaired area should then be re-inspected, and the resulting delamination map or scan compared with that of the initial inspection to verify that repair soundness.

Evaluation of the relative cure of FRP materials can be performed by laboratory testing of sample plate specimens or resin samples using ASTM Standard D3418 (2003). At the construction site, curing evaluation is achieved by physical observation of resin tackiness and hardness of work surfaces or retained resin samples. For pre-molded systems, adhesive hardness measurements should be made in accordance with the manufacturer's recommendations.

Tension adhesion testing of cored samples should be conducted using known methods such as those described in ASTM D4541 (2002) or ACI 503R (1998). However, care should be taken to avoid coring in high stress or splice areas. The tested areas must be repaired unless they are located in areas where the FRP is unstressed. Sampling frequency will depend on the size and complexity of the project. Tensile bond strength values less than 220 psi (1.5 MPa) are unacceptable.

The laminate thickness or number of plies used can be inspected with small core samples, typically 0.6 in (15 mm) in diameter; these cores may be those used for adhesion testing. As with coring for adhesion testing, highly stressed or splice areas should be avoided. After coring, the hole should be filled and smoothed. A 4 - 8 in (100–200 mm) overlapping sheet patch of equivalent plies should be applied when required.

Confirmation of the strength and elastic modulus of the FRP materials can be determined using tension tests on panels fabricated from construction site specimens. Tension testing should follow procedures such as those prescribed in Annex F of the S806-02 building code (2002) or ASTM Standard D3039 (2008). The lap strength, tensile strength, and elastic modulus of the FRP materials can also be determined using burst testing of ring specimens. FRP materials can be tested in accordance with existing ASTM test methods, but all exceptions to the method should be listed in the test report. Durability related tests use the same test methods, but require application of specific preconditioning of the specimens.

For verification of the qualification testing results, samples of the FRP system should be prepared at the construction site and tested at an approved laboratory, with the properties to be verified specified. In-place load testing can be used to confirm the behavior of the FRP-strengthened member. Such testing should be performed under the supervision of an experienced engineer and precaution must be exercised to avoid damaging the structure.

4.3.3.3 AASHTO

4.3.3.3.1 Contractor submittals

For evaluation and acceptance, AASHTO recommends that the contractor be required to submit evidence of acceptable quality control procedures that were followed in the manufacture of the FRP system. The quality control procedure should at least include the specifications for raw material procurement, 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.

The contractor also must provide information describing the fiber, matrix, and adhesive systems to be used that is sufficient to define their engineering properties. Descriptions of the fiber system should include the fiber type, percent of fiber orientation in each direction, and fiber surface treatments. When required by the engineer, 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.

Further, AASHTO suggests that the contractor should submit 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. These tests shall be conducted by a testing laboratory approved by the engineer. For each property value, the batches from which test specimens were drawn are to be identified and the number of tested specimens from each batch, and the mean, minimum, and maximum value, as well as the coefficient of variation, must be reported. The minimum number of tested samples is 10.

4.3.3.3.2 Moisture content and epoxy requirements

When cured under conditions identical to those of the intended use, the composite material system as well as the adhesive system are to conform to the following requirements. The characteristic value of the glass transition temperature of the composite system, determined in accordance with ASTM D4065 (2012), should be at least 40^{o}F higher than the maximum design temperature, defined in Section 3.12.2.2 of the *AASHTO LRFD Bridge Design Specifications* (2012). The characteristic value of the tensile failure strain in the direction corresponding to the highest percentage of fibers must not be less than 1% if the tension test is conducted according to ASTM D 3039 (2008).

When moisture migrates through the matrix and reaches the fiber-matrix interface, adhesion of the matrix to the fibers weakens. AASHTO thus specifies that the moisture content must be limited; the mean and coefficient of variation of the moisture equilibrium content, as determined in accordance with ASTM D 5229/D 5229M (2010), must not be greater than 2% and 10%, respectively. A minimum sample size of 10 should be used in the calculation of these values.

4.3.3.3.3 Environmental conditioning

After conditioning in the various environments listed below, the characteristic value of the glass transition temperature, determined in accordance with ASTM D4065 (2012), and that of tensile strain, determined in accordance with ASTM D3039 (2008), of the composite in the direction of interest shall retain 85% of the required values above. The conditioning environments are as follows:

- Water: Samples shall be immersed in distilled water having a temperature of 100 ± 3°F (38 ± 2°C) and tested after 1,000 hours of exposure.
- Alternating ultraviolet light and condensation humidity: Samples shall be conditioned in an apparatus under Cycle 1-UV exposure condition according to ASTM G154 (2012) Standard Practice. Samples shall be tested within two hours after removal from the apparatus.
- Alkali: The sample shall be immersed in a saturated solution of calcium hydroxide (pH ~11) at ambient temperature of 73 ± 3^{o}F (23 ± 2^{o}C) for 1000 hours prior to testing. The pH level shall be monitored and the solution shall be maintained as needed.
- Freeze-thaw: Composite samples shall be exposed to 100 repeated cycles of freezing and thawing in an apparatus meeting the requirements of ASTM C666 (2008).

Further, if impact tolerance is stipulated by the engineer, impact tolerance should be determined according to ASTM D7136 (2007).

When adhesive material is used to bond the FRP reinforcement to the concrete surface, the following requirements shall be met. After conditioning in the environments noted above, the characteristic value of the glass transition temperature of the adhesive material, determined in accordance with ASTM D 4065 (2012), must be at least 40^{o}F higher than the maximum design temperature as defined in Section 3.12.2.2 of AASHTO LRFD Bridge Design Specifications (2012). Moreover, after conditioning in the environments noted above, the bond strength, determined by tests specified by the engineer shall be at least $0.065\sqrt{f_c'}$ (ksi) (Naaman,1999). Also, AASHTO recommends to evaluate whether moisture will collect at the bond lines between the concrete and epoxy adhesive before the epoxy has cured. This may be checked by taping a 4 x 4 ft (1.2 x 1.2 m) polyethylene sheet to the concrete surface. If moisture collects on the underside of the sheet before the time required to cure the epoxy, then the concrete should be allowed to dry sufficiently to prevent the possibility of a moisture barrier forming between the concrete and epoxy per ACI 530R-05 (2005).

4.3.3.3.4 Epoxy physical and adhesive properties testing

During installation, sample cups of mixed resin should be prepared according to a predetermined sampling plan and retained for testing to determine the level of curing, in accordance with ASTM D2583 (2007). The relative cure of the resin can also be evaluated on the project site by physical observation of resin tackiness and hardness of the work surfaces and retained resin samples.
For bond-critical (i.e. flexural or shear) applications, tension adhesion testing of cored samples should be conducted using the methods in ACI 530R (2005), ASTM D 7234 (2012), or the method described by ISIS (1998). The sampling frequency should be specified. Tension adhesion

strengths should exceed 200 psi (1.4 MPa) and exhibit failure of the concrete substrate before failure of the adhesive per ACI 440.2R-08 (2008).

4.3.3.4 JSCE

JSCE recommendations for evaluation and acceptance include criteria for fire safety, collision protection, and finishing work.

4.3.3.4.1 Fire safety

In JSCE, the required level of fire safety depends on the structure to be upgraded and the surrounding environment. In general, three levels of safety are available:

- Flame-retardant. Here, the combustibility of the FRP is low and it can be confirmed that, even if damaged in a fire, it can be repaired.
- Noncombustible and quasi-noncombustible. In a fire, the FRP sheets are not ignited and no harmful fumes are produced. However, they are not required to maintain their load-carrying capacity during and after the fire.
- Fire-resistant. The structure will not collapse during the fire, and the FRP sheets are required to maintain full strength after the fire and either full or partial strength during the fire, without repair.

For verification of fire safety, a test specimen with the same protective coating as the actual structure should be manufactured and subjected to combustion tests. During the combustion test, ignition, the generation of gases, harmful surface deformation, and changes in the quality of the FRP after the fire are to be studied according to the level of fire safety required.

One simple method of performing the combustion test that can be used to check the flame retardant level of protection is to bring a flame near the FRP and observe whether the sheet is ignited and whether there is any residual flame on the surface of the test specimen when the burner flame is removed.

In a normal fire, the carbon fibers used in FRP do not combust or produce a chemical reaction. Even when ignited with an external flame, the fibers are self-extinguishing once the flame is removed. Therefore, they are thought to be flame retardant even without surface covering. In general, when a FRP strengthening system is used, the existing concrete structure supports dead loads and other permanent loads, while the FRP is used to support live loads. Thus, even if continuous fiber sheets fail during a fire, there is usually no danger of the structure collapsing immediately. In light of this, JSCE notes that if there is little danger of a fire occurring; no danger of spread if a fire should occur; adequate refugee space is available; and there is little likelihood of danger to human life, then there is no particular need to provide a flame resistant covering on the FRP system.

When preventing combustion during a fire is a design objective and nonflammable or quasi-nonflammable coverings are used with the assumption that the sheets will be repaired after the fire, coverings may generally consist of mortar, a rock wall, or similar materials. One method of checking flame resistance in this design category is to fabricate a test specimen by bonding FRP sheets to the concrete members, then heating the test specimen in a furnace at the prescribed temperature for the prescribed duration, then immediately testing the FRP sheets to determine if the properties of the FRP sheets are altered in undesirable ways.

When the FRP sheets are expected to maintain strength without repair even after the fire, the temperature of the continuous fiber sheets during the fire should be kept below that at which resin decomposes (for epoxy resins, about 500°F (260°C). One way to achieve this is to cover with approximately 2 in (50 mm) of mortar.

4.3.3.4.2 Collision resistance

Collision resistance is another area addressed by JSCE. When there is a possibility that the FRP may be subjected to impact, one of several methods may be used to confirm performance requirements. One possibility is that the magnitude of the impact design load is estimated using statistical data, and then performance requirements are to be verified by conducting an impact load test. The normal impact resistance of the structure can be evaluated through drop impact tests and pendulum impact tests. Possible impacts during the service life of the structure can be estimated based on damage surveys of structures thought to have been subjected to the same impacts that the structure being verified is prone to. When it is difficult to make statistical calculations of the impacts applied to the structure during its service life, more simple methods can be used that are based on the results of a survey of existing structures. When the structure is expected to be subjected to impacts only on very rare occasions, verification may be conducted to ensure that, even if the performance drops temporarily after an impact, the structure can be quickly repaired.

4.3.3.4.3 Finishing work

Finishing work includes providing coatings for durability, appearance and fire protection. JSCE recommends that upgraded surfaces are finished appropriately to ensure that the performance requirements, including sunlight and weatherproofing, fire resistance, shock resistance, roughness, and appearance, are satisfied. JSCE notes that the excellent durability of FRP has been confirmed through outdoor exposure tests and accelerated exposure tests. However, depending on the type of fiber, durability may be impaired by conditions of use, and hence the finishing should be planned after carefully considering the properties of the FRP. Resin may deteriorate and whiten when exposed to ultraviolet light and ozone, and its appearance is easily marred. Accordingly, when an aesthetic appearance and illumination are required in the environments exposed to direct sunlight, the FRP should be finished with protective paint.

4.3.3.5 TR55

Detailed evaluation and acceptance criteria as presented in ACI and ISIS are missing in TR55. The code, however, introduces test methods for shear and adhesive bond using mechanical and non-destructive test methods. The proposed test method for shear is the double lap shear test (see Figure 4.3.1). TR55 notes that various non-destructive tests may be used to inspect a completed and cured bond. The most common is acoustic sounding (hammer tapping). Thermography may be used to survey large areas. Pull-off dollies can also be installed at the time of strengthening, and pull-off tests can be performed at various times to test adhesive strength as a function of time. Similarly, additional double lap shear test specimens can be prepared at the time of strengthening and tested at various times.

TR55 states that delamination risk is dependent on the type of strengthening and the location on the structure. For example, an area of delamination in the wrapping of a column will probably have a limited effect on performance, while delamination of a plate on the soffit of a beam, particularly at points of high shear, will have a significant effect. TR55 references ACI 440 to suggest the extent of delamination that may be acceptable.

For major structures, it may be appropriate to install instrumentation prior to the strengthening to assess the structural response before and after strengthening.

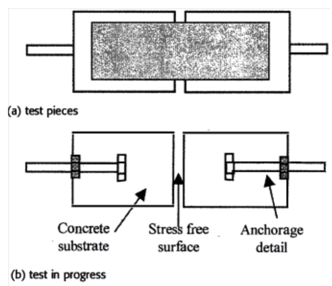

Figure 4.3.1 – Double lap shear test

4.3.3.6 CNR

4.3.3.6.1 Evaluation and acceptance before starting the project

CNR recommends before starting the project that FRP materials should be subjected to a series of controls to ensure appropriate mechanical and physical characteristics. For construction materials, specific standards are available for the determination of minimum values of physical and mechanical properties, test procedures, as well as acceptance criteria. For further information about mechanical characterization tests for fiber reinforcement materials, CNR refers to the following documents:

- *ACI 440.3R 04 "Guide Test Methods for Fiber-reinforced Polymers for Reinforcing or Strengthening Concrete Structures";*
- *JSCE (1995) "Test methods for continuous fiber reinforcing materials";*
- *JSCE (2000) "Test methods for continuous fiber sheets";*
- *ISO (TC71/SC6N) "Non-conventional strengthening of concrete - Test methods-Part 1: Fiber strengthened polymer (FRP) bars and grids";*
- *ISO (TC71/SC6N) "Non-conventional strengthening of concrete - Test methods-Part 2: Fiber strengthened polymer (FRP) sheets".*

4.3.3.6.2 Responsibilities of the construction manager

CNR defines the process of evaluation and acceptance as the responsibility of the construction manager. Additional responsibilities of the construction manager are as follows. The construction manager is to make decisions as to the acceptance of products; check the compliance of the material to the designer's provisions; check the origin of the supplied material (pultruded materials are typically marked by the manufacturer for their identification, while other materials must have labels or tags with the necessary information for traceability); and to check the mechanical and physical characteristics of products using the test certificates provided by the manufacturer. Additional responsibilities are to determine whether experimental tests are to be required to evaluate material quality and compliance with the values provided by the manufacturer. Such tests are to be carried out in laboratories with sufficient experience and equipment to characterize FRP materials. Acceptance criteria may be based on the maximum acceptable deviation of results from the values obtained during production. In some cases, tests may be required to evaluate both mechanical and physical properties of unconditioned and conditioned specimens to take into account temperature and moisture variation.

CNR defines Type-A and Type-B applications. For Type-A, certification is obtained for each component as well as the final product to be applied. However, it is the decision of the construction manager to require acceptance tests for the installed system. For Type-B applications, each component requires certification but not the FRP system. In this case, the construction manager shall require a number of tests to ensure proper quality of both the FRP system and installation procedures as suggested in CNR sections 4.8.3 and 5.8.3 for reinforced concrete and masonry structures, respectively.

4.3.3.6.3 Evaluation and acceptance at project completion

CNR specifies that quality control tests are needed during FRP installation. Tests include at least one cycle of semi-destructive tests for the mechanical characterization of the installation itself, and at least one non-destructive mapping to ensure uniformity.

For semi-destructive tests, both pull-off and shear tearing tests may be conducted. Semi-destructive tests shall be carried out on witness panels and, where possible, in non-critical strengthened areas at the rate of one test for every 53.82 ft^2 (5 m^2) of application, and in any case, not less than 2 per each type of test. The pull-off test is used to assess properties of the concrete substrate, and is carried out by using 0.8 in (20 mm) thick circular steel plates with a diameter of at least 3 times the characteristic size of the concrete aggregate, but not less than 1.6 in (40 mm). These plates are adhered to the surface of the FRP with an epoxy adhesive. After the steel plate is firmly attached to the FRP, it is isolated from the surrounding FRP with a core drill rotating at a speed of at least 2500 rpm. Here, particular care shall be taken to avoid heating of the FRP system while a 0.04-0.08 in (1-2 mm) incision of the concrete substrate is achieved. FRP application may be considered acceptable if at least 80% of the tests (both tests in case of only two tests) return a pull-off stress not less than 130-174 psi (0.9-1.2 MPa), provided that failure occurs in the concrete substrate.

The shear tearing test is particularly significant to assess the quality of bond between the FRP and concrete. It may be carried out only when it is possible to pull a portion of the FRP system in its plane located close to an edge detached from the concrete substrate. Results may be considered acceptable if at least 80% of the tests (both in the case of two tests) return a peak tearing force not less than 5.4 kips (24 kN).

Nondestructive tests may be used to characterize the uniformity of FRP application, starting from an adequate two-dimensional survey of the strengthened surface with a spatial resolution as a function of the strengthening area (see Table 4.3.1).

Table 4.3.1 – Minimum resolution for defects to be identified with non-destructive tests

Shear stress transfer at interface	**Example**	**Non-destructive test**	**Surface mapping grid**	**Minimum resolution for defects thickness**
Absent	Wrapping, with the exception of the overlapping area in a single-layer application	Optional	10 in (250 mm)	0.12 in (3.0 mm)
Weak	Central area of extensive plane reinforcement	Optional	10 in (250 mm)	0.12 in (3.0 mm)
Moderate	Central area of longitudinal flexural strengthening	Suggested	4 in (100 mm)	0.02 in (0.5 mm)
Critical	Anchorage area, overlapping areas between layers, stirrups for shear strengthening, interface area with connectors, or an area with large roughness or cracks in the substrate	Required	2 in (50 mm)	0.004 in (0.1 mm)

The non-destructive tests described by CNR are:

Stimulated Acoustic testing. In a simple version, this test can be conducted by a technician hammering the composite surface and listening to the sound from the impact. However, more objective results may be obtained with automated systems.

High-frequency ultrasonic testing. This are best conducted with a reflection method using frequencies no less than 1.5 MHz and probes with diameter no greater than 0.98 in (25 mm), using the technique based on the first peak amplitude variation due to localize defects.

Thermographic test. According to CNR, this is effective only for FRP systems with low thermal conductivity, and should not be applied to carbon or metallic FRP strengthening systems unless special precautions are taken, as the heat developed during the test must be smaller than the glass transition temperature of the FRP system.

Acoustic emission test. This test is particularly suited for detecting defects in FRP systems applied on RC structures, as well as delamination of the concrete substrate.

4.3.3.7 Summary of evaluation and acceptance

ACI and ISIS provide similar coverage of evaluation and acceptance criteria including materials evaluation, materials/FRP system qualification tests, and field testing and sample collections. The two codes present criteria for acceptance of fiber alignment, delamination, cure of resin, adhesion strength, and cured thickness.

AASHTO states the contractor’s responsibilities for evaluation and acceptance. The contractor is to submit a complete QC plan detailing inspection, sampling, testing, and criteria for acceptance. The contractor is required to also provide detailed information on materials used as part of the FRP system, as well as test results to verify the materials meet required design properties. AASHTO provides a detailed list of acceptance criteria for materials and FRP system including a description of testing required and sample sizing. AASHTO places emphasis on testing materials under various environmental factors.

JSCE places particular emphasis on three areas; fire resistance, collision safety, and finishing work. The code offers a reasonable coverage of the three areas but lacks coverage in areas of evaluation and acceptance covered by other codes. TR55 does not have clear boundaries separating aspects of quality control, inspection, and testing from evaluation and acceptance criteria. So, items discussed in inspection overlaps evaluation and acceptance.

CNR offers a list of reference codes it recommends as basis for its materials/system qualification. The code provides broad guidelines for evaluation and acceptance but lacks detail. The code offers two testing categories; semi-destructive and nondestructive. Semi destructive testing includes pull-off and shear tearing tests. Nondestructive testing includes stimulated acoustic testing, acoustic emission testing, and thermographic testing.

A comparison of the minimum acceptable tension strength of adhesive is presented in Table 4.3.2, and a comparison of the maximum allowable area of delamination is presented in Table 4.3.3.

Table 4.3.2– Minimum acceptable tension strength of adhesive

ACI	ISIS	CNR
200 psi (1.4 MPa)	220 psi (1.5 MPa)	130-175 psi (0.9-1.2 MPa)

Table 4.3.3 – Maximum allowable area of delamination

ACI	ISIS
2 in^2 (1300 mm^2) Or 5% of the total laminate area	2.33 in^2 (1500 mm^2) Or 5% of the total laminate area

4.4 Maintenance and Repair

4.4.1 Introduction

Section 4 covers elements of a maintenance program with periodic inspection and testing to identify any damage, degradation, or deficiencies to the FRP strengthening system and to make any necessary repairs. A maintenance assessment is made from test data as well as observations, and may include recommendations to help slow down degradation and propose necessary repairs.

4.4.2.1 ACI

4.4.2.1.1 Inspection and assessment

ACI suggests that periodic inspection and assessment are needed to verify the long-term performance of the FRP system. The causes of any damage or deficiencies detected during routine inspections should be identified and addressed before performing any repairs or maintenance.

A general inspection consists of observation for changes in color, debonding, peeling, blistering, cracking, crazing, deflection, indications of reinforcing-bar corrosion, and other anomalies. Other inspection methods such as ultrasonic, acoustic sounding (hammer tap), or thermographic tests may be used to identify signs of progressive delamination.

Test data and observations are used to assess any damage and the structural integrity of the strengthening system. Testing can include pull-off tension tests or conventional structural load tests. The assessment can include a recommendation for repairing any deficiencies and preventing recurrence of degradation.

4.4.2.1.2 Repair techniques

Prior to repair, the causes of the damage must first be identified and addressed. The method of repair should depend on the cause of damage, the type of material, the form of degradation, and the level of damage. Minor damage should be repaired, including localized FRP laminate cracking or abrasions that affect the structural integrity of the laminate. Such damage can be repaired by bonding FRP patches with the same FRP characteristics over the damaged area. The FRP patches should possess the same characteristics, as thickness or ply orientation, as the original laminate. Minor delaminations can be repaired by resin injection.

Major damage, including peeling and debonding of large areas, may require removal of the affected area, reconditioning of the concrete, and replacement of the FRP. ACI does not mention techniques that may be used to remove the damaged FRP. However, some recommendations for removal are provided in Section 8.2.6.2 of this report. FRP patches should be installed in accordance with the material manufacturer's recommendation.

If the surface protective coating is to be replaced, the FRP should be inspected for structural damage or deterioration. The surface coating should be replaced using a process approved by the system manufacturer.

4.4.2.2 ISIS

ISIS does not address long-term maintenance, assessment and repair.

4.4.2.3 AASHTO

AASHTO recommends that the following documents be considered for evaluation and repair of existing concrete structures and post-repair evaluation criteria:

- *ACI 201.1R: Guide for Making a Condition Survey of Concrete in Service*
- *ACI 224.1R: Causes, Evaluation, and Repair of Cracks in Concrete*
- *ACI 364.1R-94: Guide for Evaluation of Concrete Structures Prior to Rehabilitation*
- *ACI 440.2R-08: Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures*
- *ACI 503R: Use of Epoxy Compounds with Concrete*
- *ACI 546R: Concrete Repair Guide*
- *International Concrete Repair Institute (ICRI) ICRI 03730: Guide for Surface Preparation for the Repair of Deteriorated Concrete Resulting from Reinforcing Steel Corrosion*
- *International Concrete Repair Institute (ICRI) ICRI 03733: 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*

AASHTO states that relevant specifications and guidelines provided by FRP manufacturers should also be carefully reviewed.

4.4.2.4 JSCE

JSCE recommends that concrete structures strengthened with FRP should be maintained with a systematic combination of deterioration prediction, inspection, evaluation and judgment, countermeasures, and records.

Anticipated deterioration should be accounted for in the design and maintenance inspections by upgrading and repairing appropriately. At the time of a maintenance event, more accurate predictions of deterioration should be made.

4.4.2.4.1 Inspection and assessment

Inspections consist of initial, daily, periodic, detailed, and extraordinary inspections. These should be based on the performance requirements and the predictions of performance deterioration. Inspections should be conducted visually or using appropriate inspection equipment, with consideration given to both performance requirements and the mechanism of deterioration. Deterioration may affect the FRP material itself or the resin individually, the FRP system as a composite material (interfacial deterioration), and deterioration of bond to the concrete. The visual features of this deterioration may include swelling, peeling, lifting, softening, discoloration, whitening, chalking, cracking, wearing, erosion, pinholes, scratches, deformation, and embrittlement. However, JSCE notes that FRP sheets will block or limit the intrusion of various external substances. As such, improved concrete durability can be anticipated with respect to the salt attack, carbonation, freeze-thaw, alkali aggregate reaction, chemical attacks, and fatigue. Deterioration may change various properties, including weight, volume, mechanical properties (hardness, bond strength, tensile strength, modulus of elasticity, elongation, etc.), and physical properties (electrical properties, thermal properties, optical properties, etc.) Inspections using a combination of observation as well as suitable inspection equipment should be conducted.

JSCE notes that two stages of evaluations and judgments are used: those based primarily on visual inspections, and those based on detailed inspections. For visual inspections, a judgment is made as to whether a detailed inspection is required. In a detailed inspection, the need for countermeasures is evaluated and judgment is used to select the type of countermeasure.

4.4.2.4.2 Repair techniques

Countermeasures are implemented to satisfy performance requirements, based on the results of evaluation and judgment. Countermeasures include stricter inspections, service restrictions, repair of the FRP system, additional upgrading, improvement of appearance, and dismantling and disposal. For minor deterioration, countermeasures should consist primarily of stricter inspections and repair of the FRP system. The method selected should depend on the deterioration mechanism and the extent of changes observed. However, JSCE suggests the following for consideration. For swelling, peeling, and lifting, resin fill can be used, while for cracking, wearing, and erosion, patching can be used. When serious deterioration or deterioration over a wide area is observed, additional FRP upgrading should be performed. In such cases, the existing FRP should be removed and the upgrading plan reexamined.

To implement suitable maintenance, the results of design, construction, inspection, evaluations and judgments, repairs, additional upgrading, and so on, are to be recorded and the records maintained. The ease of maintenance is affected by the upgrading plan and by design and construction. More specifically, the placement of access paths to the structure that allow inspection and monitoring equipment affects ease of maintenance. For this reason, it is recommended to give thorough consideration to maintenance considerations in the upgrading plan, as well as in design and construction.

4.4.2.5 TR55

4.4.2.5.1 Inspection and assessment

Similar to ACI, TR55 emphasizes that the FRP strengthening system should be monitored and inspected regularly. A general inspection is recommended once a year, while a detailed inspection is recommended once every 6 years.

A general (visual) inspection primarily consists of a surface inspection. The inspector looks for signs of crazing, cracking, delamination, or evidence of deterioration, in addition to local damage due to impact or surface abrasion. Signs of concrete deterioration in the form of cracking or corrosion should also be reported. Any required identification or warning labels should be checked for, and missing ones should be replaced.

The condition of the FRP protective layer, if any, should also be inspected. Damaged protective coatings should be replaced in accordance with the supplier's recommendations. It is not appropriate to remove the protective layer to facilitate inspection.

A detailed inspection considers various items. Debonding of the FRP from the concrete may be determined by tapping or thermography. However, TR55 reports that no nondestructive tests are available to assess the condition of the adhesive bond. Therefore, adhesive bonding is evaluated by puIl-off tests on the control specimens at regular intervals. PuIl-off tests should be carried out as part of a detailed inspection, although there may be a requirement to test samples more frequently, particularly soon after strengthening.

To facilitate inspection, instrumentation may be installed as part of the assessment process, for example to measure strains due to live loading on the structure. Such instrumentation can be used to indicate changes in structural response. If significant changes are observed, it is necessary to determine whether they are due to changes in the strengthening system (such as delamination) or due to overall changes in the concrete structure (such as additional cracking or corrosion) so that appropriate action can be taken. The Health and Safety File for the structure should include details of any instrumentation that was installed as part of the strengthening exercise, along with any data obtained before and after strengthening.

Information on the materials used in strengthening should be included in the Health and Safety File for the structure. This File should also include reported minor areas of delamination and any initial faults detected in the strengthening system; identification of critical strengthening regions such as anchorage zones and high stress regions; and procedures prepared by the engineer on actions to be taken for various forms of damage to the FRP strengthening system, which should be tailored to each particular structure.

In order to facilitate the testing and evaluation of the FRP strengthening system, it is recommended that additional areas of the strengthened structure, away from the regions that were strengthened, are also bonded with the FRP system for future testing that does not impact the system performance. TR55 reports that this approach has been adopted on a number of structures including the Barnes Bridge in Manchester and the John Hart Bridge in British Columbia.

Alternatively, FRP can be bonded to concrete samples which can be stored near the structure. Samples can be inspected and tested as part of the inspection regime. To aid inspection, some or all of the samples should not be covered with any protective layer. They should thus indicate a lower bound of performance of the composites bonded to the structure. Details should be included in the structure's Health and Safety File along with recommendations for the frequency of testing.

4.4.2.5.2 Repair techniques

For localized damage to the FRP, repairs can be made with resin injection or plate overlapping. For major damage, such as peeling and debonding of large areas, the defective material should be removed to an extent that material on the periphery of the repair is fully bonded. The concrete surface should then be prepared, and an FRP patch installed that allows adequate overlap between the new and old materials. The compatibility of the proposed repair material with the materials already in place should be checked. The repair material must have similar characteristics to the material in place such as fiber orientation, volume fraction, strength, stiffness and overall thickness.

4.4.2.6 CNR

CNR recommends that, due to the poor availability of data regarding long term behavior of FRP systems used for strengthening, appropriate monitoring of the installed FRP system should be performed with periodic semi- and non-destructive tests The aim of the monitoring process is to identify potential problems with the temperature of the installed FRP system; environmental humidity; displacements and deformations of the strengthened structure; fiber damage; and defects or delaminations in the installed FRP system.

4.4.2.7 Summary of maintenance and repair

ACI, TR55 and JSCE all present a relatively broad and reasonable coverage of maintenance and repair issues. They each call for development and implementation of a periodic maintenance program including visual inspection, specified testing, an assessment of damage to evaluate the structural integrity of the system, and recommendations for repair. Based on the type and severity of damage, the three codes offer suggestions for appropriate repair methods. CNR's coverage on maintenance and repair is brief, and states the need for periodic monitoring and inspection using semi and non-destructive tests for assessment of the system. No discussion of repair recommendation is presented. ISIS does not cover this topic, while AASHTO does not provide detailed coverage, but refers to a list of specialized documents from ACI, ICRI, and NCHRP for further suggestions.

CHAPTER 5: LABORATORY TESTING

5.1 Durability Testing Overview

5.1.1 Introduction

The purpose of the laboratory testing is to assess FRP system degradation when exposed to the climate of Michigan and to develop a corresponding environmental reduction factor. To achieve this, two primary groups of FRP-strengthened concrete samples were prepared. Group 1 samples concern material and bond degradation assessment and are used to generate the primary data for environmental response by way of pull-off testing. Group 2 specimens are used as ‘spot checks’ to verify the applicability of the material-level response results to structural-level degradation with respect to failure modes such as flexure, shear, and axial compression.

Group 1 consists of two identical sets of FRP-strengthened concrete test samples. One set is kept outdoors, bearing full exposure to natural weathering effects including exposure to the sun and variation in moisture and temperature levels. The second set undergoes acceleration testing in an environmental chamber using a predetermined environmental acceleration cycle. Pull-off adhesive bond testing is conducted on samples from Group 1 sets one and two at appropriate testing frequencies, as discussed below. A calibration of bond degradation between set one and set two makes it possible to establish acceleration factors that are appropriate for the specific climate in Michigan.

Group 2 samples include sample beams for flexural, 2-sided shear, and u-wrap (3-sided shear) tests, and wrapped cylinder samples for confinement testing. Fabric as well as FRP plates were used for flexural strengthening. A selection of the samples (flexural and shear-strengthened) were kept outdoors for environmental weathering while identical samples were reserved for accelerated testing.

Concrete samples used for the Group 2 beam specimens have dimensions of 16” x 4.3” x 4.1” (length x width x height). These dimensions are based on the work of Elarbi (2011) and are in accordance with ASTM C293-08. Cylindrical samples are standard 4” x 8” (diameter x height) size compressive strength cylinders in accordance with ASTM C39-08. Group 1 pull-off test samples are sized to be approximately half the width of the Group 2 beam samples. These dimensions are 16” x 2.0” x 4.1”. Note that multiple pull-off tests can be conducted on a single Group 1 test specimen.

5.1.2 Materials procurement

Concrete was acquired from a commercial batch plant (McCoig Materials – MDOT plant No. M-11) with experience and knowledge of MDOT concrete mix designs for bridge applications. Several suppliers of FRP were used for strengthening. In particular, Sika, Fyfe, and BASF provided material for CFRP wraps, epoxy adhesives, and protective coating materials. The fabric wraps were primarily unidirectional CFRP, although bi-directional wraps from Sika and Fyfe were also used. In addition, CFRP plates from Sika and Fyfe as well as protective coatings (FRP paint) were obtained for testing. Section 5.2 presents the manufacturer-specified material properties of the FRP systems.

5.1.3 Testing program

FRP-strengthened samples were prepared with several variations for testing, using:

- Unidirectional and bi-directional fabrics
- Specimens to evaluate bond (Group 1) as well as to check structural-level behavior (Group 2)
- Different types of epoxy
- Different fabric characteristics
- Specimens with and without a protective coating
- CFRP plates as an alternative to fabric

A total of 127 concrete samples were strengthened. Thirteen representative samples were placed outdoors under the exposure of natural weathering, with identical samples reserved for accelerated testing indoors.

5.2 Materials Properties

5.2.1 Concrete sample preparation and properties

As noted above, a concrete sample size of 16” x 4.3” x 4.1” was selected for Group 2 structural testing (shear and flexure) in accordance with ASTM 293-08, while the adhesive bond pull-off test samples (Group 1) were half-width (2.0” wide). For casting, plywood mold assemblies were prepared (Figure 5.2.1) and a standard MDOT 2012 Grade D concrete mix was ordered from McCoig Materials. Pour dates were February 1 and April 5, 2013 (Figure 5.2.2). Fresh concrete properties were determined on-site with standard ASTM tests (Figure 5.2.3).

Figure 5.2.1 – Typical plywood mold assembly

Figure 5.2.2 – Fresh concrete surface finishing

Figure 5.2.3 – Fresh concrete onsite testing

Concrete properties were as follows:

Coarse aggregate (L6AA-OTT)	1720 lbs.
Fine aggregate (2NS-AAR)	1240 lbs.
Cement (Type 1)	658 lbs.
Water	41.6 gallons
W/C ratio	0.4
Slump	5.0 in
Air content	6%

Test cylinders with 4” diameter formed at the time of casting were moist-cured by submersion in a water bath along with the cast specimens and later tested for evaluation of compressive strength using an MTS-290 testing machine. The specified 28 day compressive strength of the mix was 5,500 psi. Table 5.2.1 provides the average compressive strength of the cylinder tests at different times. A minimum of three cylinders were used for each set. The target compressive strength of 5500 psi was exceeded at 26 day sample testing.

Table 5.2.1 - Concrete compressive test results

Set No.	Age at testing, days	Mean strength, lbs.	Mean Compressive Strength, psi
1	5	33712	2683
2	7	48080	3826
3	10	60160	4787
4	26	72131	5740

5.2.2 FRP material properties

As noted above, FRP strengthening systems with carbon fiber and epoxy adhesives were obtained from Sika, Fyfe, and BASF. Epoxies consisted of resins and hardeners. In the following sections, products and their physical and mechanical properties are presented.

5.2.2.1 Sika material properties

FRP wraps from Sika include Sikawrap Hex 113C bi-directional, Sikawrap Hex 103C, Sikawrap Hex 230C, and Sika carboDur S1014 plate (4” wide strip). Table 5.2.2 presents Sika fiber properties, while Tables 2.3 and 2.4 provide the physical and mechanical properties of Sika laminate. Sika epoxies are used as part of the FRP strengthening system to achieve the bond between FRP wrap and the concrete substrate. Three types of epoxies are included; SikaDur 300, SikaDur 330, and SikaDur 30. SikaDur 300 and SikaDur 330 are suitable for wet lay-up applications of FRP wrap with SikaDur 300 having the lower viscosity of the two. SikaDur 30 is designed to work with FRP plates. Table 5.2.5 presents the mechanical properties of SikaDur 300 and SikaDur 330. Table 5.2.6 presents the mechanical properties of SikaDur 30.

Table 5.2.2 - Sika fiber properties

Fiber Properties	Unit	Sikawrap Hex 103C	Sikawrap Hex 113C	Sikawrap Hex 230C
Tensile Strength	psi	550,000	66,000	500,000
	MPa	3,793	456	3,450
Tensile Modulus	psi	33,000,000	6,000,000	33,400,000
	GPa	228,000	41,400	230,000
Ultimate Elongation	%	1.50	1.20	1.50
Density	lbs/in^3	0.065	0.065	0.065
	g/cc	1.8	1.8	1.8

Table 5.2.3 – Physical properties of Sika laminate

Laminate Physical Properties	**Unit**	**Sikawrap Hex 103C**	**Sikawrap Hex 113C**	**Sikawrap Hex 230C**
Fiber type		Carbon	Carbon	Carbon
Color		Black	Black	Black
Weight	OZ/Y^2	18	5.7	6.7
	g/m^2	618	196	230
No. of Filaments		24,000		
Ply Thickness	in	0.04	0.01	0.015
	mm	1.016	0.25	0.381

Table 5.2.4 - Mechanical properties of Sika laminate

Cured Laminate Mechanical Properties		**Sikawrap Hex 103C**		**Sikawrap Hex 230C**	
	Unit	Test	Design	Test	Design
Tensile Strength	psi	123,000	104,000	129,800	104,000
	MPa	849	717	894	715
Tensile Modulus	psi	10,239,800	9,446,600	9,492,300	8,855,000
	MPa	70,552	65,087	65,402	61,012
Tensile Elongation	%	1.12	0.98	1.33	1.09
140 deg. Tensile Strength	psi	123,000	101,400	118,200	102,000
	MPa	847	699	814	703
140 deg. Tensile Modulus	psi	10,136,900	9,156,500	9,789,000	8,693,000
	MPa	69,843	63,088	67,450	59,896
140 deg. Tensile Elongation	%	1.13	0.97	1.16	1.00
Comp. Strength	psi	113,000	103,800	113,000	97,000
	MPa	779	715	779	668
Comp. Modulus	psi	9,726,000	8,930,600	9,724,700	9,230,000
	MPa	67,014	61,532	67,003	63,597
90 deg. Tensile Strength	psi	3,500	2,300	3,965	390
	MPa	24	16	27	23
90 deg. Tensile Modulus	psi	705,500	576,700	852,800	799,000
	MPa	4,861	3,973	5,876	5,502
90 deg. Tensile Elongation	%	0.45	0.33	0.46	0.4

Table 5.2.4 - Mechanical properties of Sika laminate, Cont.

Cured Laminate Mechanical Properties		**Sikawrap Hex 103C**		**Sikawrap Hex 230C**	
	Unit	Test	Design	Test	Design
In-plane Shear Strength	psi	7,500	6,700	9,100	8,100
	MPa	52	46	63	56
In-plane Shear Modulus	psi	362,500	347,500	421,200	406,000
	MPa	2,498	2,394	2,902	2,800
Tensile Strength per inch width	lbs	4,928	4,160	1,947	1,560
	kN	21.9	18.5	8.7	6.9

Table 5.2.5 - SikaDur 300 and SikaDur Hex 330 properties

Epoxy Properties	**Unit**	**Sikadur Hex 330**(2)	**Sikadur 300**(1)
Tensile Strength	psi	4,900	8,000
	MPa	33.8	55
Tensile Modulus	psi		250,000
	MPa		1,724
Tensile Elongation	%	1.2	3
Flexural Strength	psi	8,800	11,500
	MPa	60.6	79
Flexural Modulus	psi	506,000	500,000
	MPa	3,489	3,450

1. *Material properties after 14 day cure*
2. *Material properties after 7 day cure*

Table 5.2.6 - SikaDur 30 properties

Epoxy Properties	**Unit**	**Sikadur 30**
Tensile strength after 7 days	psi	3600
	MPa	24.8
Modulus of elasticity after 7 days	psi	65,000,000
	MPa	4,482
Flexural strength	psi	6,800
	MPa	46.8
Shear strength after 14 days	psi	3,600
	MPa	24.8
Elongation at break	%	1

5.2.2.2 Fyfe material properties

FRP wraps obtained from Fyfe include Tyfo SCH-41, Tyfo SCH-41-.5X, and Tyfo SCH-41H. Table 5.2.7 provides the mechanical properties of Fyfe fiber wraps. Tables 5.2.8 and 5.2.9 provide the physical and mechanical properties of Fyfe laminate. Tyfo S epoxy is the main adhesive used in the Fyfe FRP system. Tyfo SW is a variation of Tyfo S formulated for underwater applications. Table 5.2.10 provides the properties of Tyfo S epoxy.

Table 5.2.7 - Fyfe fiber material properties

Fiber Properties	**Unit**	**Tyfo SCH-41**	**Tyfo SCH-41- .5X**	**Tyfo SCH-41H**
Tensile Strength	psi MPa	550,000 3,790	550,000 3,790	675,000 4,650
Tensile Modulus	psi GPa	33,400,000 230,000	33,400,000 230,000	42,000,000 289,600
Ultimate Elongation	%	1.70	1.70	1.70
Density	$lbs/in.^3$ g/cc	0.063 1.74	0.063 1.74	0.065 1.8

Table 5.2.8 - Physical properties of Fyfe laminate

Laminate Physical Properties	**Unit**	**Tyfo SCH-41**	**Tyfo SCH-41- .5X**	**Tyfo SCH-41H**
Fiber type		Carbon	Carbon	Carbon
Color		Black	Black	Black
Weight	OZ/Y^2 g/m^2	19 644	9.5 322	24 814
Ply Thickness	in mm	0.04 1.0	0.024 0.6	0.04 1.0

Table 5.2.9 - Mechanical properties of Fyfe laminate

Cured Laminate Mechanical Properties	Unit	Tyfo SCH-41		Tyfo SCH-41- .5X		Tyfo SCH-41H	
		Test	Design	Test	Design	Test	Design
Tensile Strength	psi	143,000	121,000	137,000	116,000	200,000	170,000
	MPa	986	834	944.6	799.8	1,380	1,170
Tensile Modulus	psi	13,900,000	11,900,000	14,500,000	12,300,000	15,500,000	13,100,000
	MPa	95,800	82,000	99,900	84,800	106,800	90,300
Tensile Elongation	%	1.00	0.85	0.95	0.80	1.30	1.10
Flexural Strength	psi	17,900	15,200				
	MPa	123.4	104.8				
Flexural Modulus	psi	452,000	384,200				
	MPa	3,120	2,650				
Flexural Failure Strain	%	1.00	0.85				
Comp. Strength	psi	50,000	42,500				
	MPa	344.8	293				
Comp. Modulus	psi	11,200,000	9,500,000				
	MPa	77,200	65,500				
Tensile Strength per inch width	lbs	5,720	4,840	3,288	2,784	8,000	6,800
	kN	25.4	21.5	14.6	12.4	35.6	30.2

Table 5.2.10 - Fyfe epoxy properties

Epoxy properties	Unit	Tyfo S Epoxy
Tensile Strength	psi	10,500
	MPa	72.4
Tensile Modulus	psi	461,000
	MPa	3,180
Tensile Elongation	%	5
Flexural Strength	psi	17,900
	MPa	123.4
Flexural Modulus	psi	452,000
	MPa	3,120

5.2.2.3 BASF materials properties

The FRP strengthening system obtained from BASF includes a set of products with the commercial name MBrace. One type CFRP wrap (MBrace CF130) was available. The MBrace surface treatment and epoxy adhesive products obtained were:

- MBrace putty for plugging holes and leveling depressions on the concrete surface.
- MBrace primer to enhance the adhesive bond between the saturant and concrete substrate.
- MBrace saturant resin to saturate the wrap and achieve adhesive bond to primer and other plies.

Tables 5.2.11, 5.2.12, 5.2.13 and 5.2.14 provide material properties of MBrace CF130, MBrace primer, saturant, and putty, respectively.

Table 5.2.11 - Material properties of MBrace CF130

Fiber Properties	**Unit**	**MBrace CF130**
Tensile Strength	psi MPa	550,000 3,800
Tensile Modulus	psi GPa	33,000,000 227,000
Ult. Elongation	%	1.67
Nominal thickness	in/ply mm/ply	0.0065 0.165
Ultimate tensile strength per unit width	Kips/in/ply KN/mm/ply	3.57 0.625
Tensile modulus per unit width	Kips/in/ply KN/mm/ply	215 38

Table 5.2.12 - Material properties of MBrace primer

Fiber properties	**Unit**	**Tensile properties**	**Compressive properties**	**Flexural Properties**
Yield strength	psi MPa	2100 14.5	3800 26.2	3500 24.1
Elastic modulus	psi MPa	105 717	97 670	86.3 595
Rupture strain	%	40	10	Large deformation with no rupture
Ultimate strength	psi MPa	2500 17.2	4100 28.3	3500 24.1

Table 5.2.13 - Material properties of MBrace saturant

Material properties	Unit	Tensile properties	Compressive properties	Flexural Properties
Yield strength	psi	7900	12500	20000
	MPa	54	86.2	138
Elastic modulus	psi	440000	380000	540000
	MPa	3034	2620	3724
Rupture strain	%	3.5	5	5
Ultimate strength	psi	8000	12500	20000
	MPa	55.2	86.2	138

Table 5.2.14 – Material properties of MBrace putty

Fiber properties	Unit	Tensile properties	Compressive properties	Flexural Properties
Yield strength	psi	1800	3300	3800
	MPa	12	22.8	26.2
Elastic modulus	psi	260000	155000	130000
	MPa	1800	1076	895
Rupture strain	%	7	10	7
Ultimate strength	psi	2200	3300	4000
	MPa	15.2	22.8	27.6

5.2.3 Sample preparation for FRP strengthening

At the conclusion of concrete sample curing, the samples were allowed to dry for 48 hours prior to surface preparation. The procedure includes the following steps:

- Surface grinding with an angle grinder to remove any weak surface concrete and remove all impurities.
- Grinding to round edges for U-wrap samples.
- Wire brushing of the surface to reach depressed areas and surface imperfections.
- Surface filling with putty of large defects to ensure a level surface.
- Air pressure cleaning to remove dust prior to FRP application.

Figure 5.2.4 – Surface grinding of concrete samples

5.2.4 Application of FRP to concrete samples

Manufacturer guidelines were followed closely during FRP installation, including use of appropriate procedures, tools, and safety requirements. Representatives from Sika and BASF supervised the installation to ensure adherence to guidelines. Figures 5.2.5 and 5.2.6 show the installation of the FRP and the completed samples. Based on this experience, two observations are as follows:

- It was difficult to fully saturate FRP fabric with high viscosity resin. If this combination of strengthening materials are used for wall and/or slab applications, special care is recommended to ensure that the fabric is fully saturated. The higher viscosity resins are recommended for FRP sheet (laminates) applications rather than fabrics.

- Strict adherence to the epoxy pot life and operating temperature range during the FRP strengthening is necessary to avoid impacting the quality of installation. Correspondingly, advanced planning of installation procedures is crucial to a successful installation. Careful adherence to the manufacturer's recommended guidelines is critical to avoid an early set of the epoxy.

These recommendations are noted in Chapter 8 (sections 8.2.3.6 and 8.2.3.7).

Figure 5.2.5 – FRP installation using Fyfe FRP system

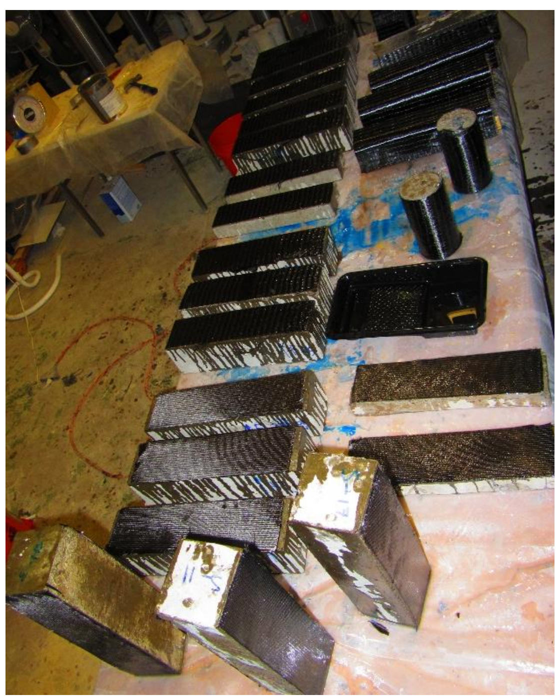

Figure 5.2.6 – Installed FRP system

5.3 Test Plan and Procedures

5.3.1 Summary of test specimens

A summary of all specimen characteristics is presented in Table 5.3.1 for flexure beam samples, Table 5.3.2 for shear beam samples, Table 5.3.3 for confinement cylinders, and Table 5.3.4 for pull-off test samples. Table 5.3.5 summarizes the number of each type of sample, while Figures 5.3.1-5.3.7 show select samples for different testing purposes.

Table 5.3.1 – FRP strengthened samples for flexural structural testing

Epoxy	Wrap	Uni/ Bi	No. of Samples	sample designation	sample cast date	Sample Wrapping date	Indoor/ Outdoor	Outdoor Samples	Paint Y/N	sample size
Sika										
SikaDur 300	103C	U	3	19, 20, 21	03/29/13	05/16/13	I	N	N	4"x4"x16"
	230C	U	3	22, 23, 24	03/29/13	05/16/13	I	N	N	4"x4"x16"
SikaDur 330	103C	U	3	1, 2, 3	02/01/13	03/07/13	I	N	N	4"x4"x16"
	230C	U	3	7, 8, 9	02/01/13	03/07/13	I	N	N	4"x4"x16"
	113C	B	3	4, 5, 6	02/01/13	03/07/13	I	N	N	4"x4"x16"
SikaDur 30	4" Strip	U	3	47, 48, 49	02/01/13	03/07/13	I	N	N	4"x4"x16"
Fyfe										
Tyfo S	SCH-41	U	4	50, 51, 52, 53	03/29/13	05/15/13	I	N	y	4"x4"x16"
	SCH-41	U	2	54, 55	03/29/13	05/15/13	O	54, 55	y	4"x4"x16"
	SCH-41 H	U	3	56, 57, 58	03/29/13	05/15/13	I	N	N	4"x4"x16"
	BCC	B	3	59, 60, 61	03/29/13	05/15/13	I	N	N	4"x4"x16"
Tyfo SW	SCH-41	U	3	62, 63, 64	03/29/13	05/15/13	I	N	N	4"x4"x16"
	SCH-41-0.5X	U	3	65, 66, 67	03/29/13	05/15/13	I	N	N	4"x4"x16"
MB-3	SCH-41	U	2	68, 69	03/29/13	05/15/13	I	N	N	4"x4"x16"
	SCH-41-0.5X	U	3	70, 71, 72	03/29/13	05/15/13	I	N	N	4"x4"x16"
BASF										
Mbrace Primer/ Saturant	Mbrace CF130	U	3	98, 99, 100	03/29/13	05/16/13	O	98, 99, 100	y	4"x4"x16"
		U	2	101, 102	03/29/13	05/16/13	I	N	y	4"x4"x16"
		U	2	103, 104	03/29/13	05/16/13	I	N	y	4"x4"x16"
		U	2	105, 106	03/29/13	05/16/13	I	N	Y	4"x4"x16"
Mbrace Primer/ Concresive	Mbrace S&P Laminate	U	3	107, 108, 109	03/29/13	05/16/13	I	N	N	4"x4"x16"

Table 5.3.2 – FRP strengthened samples for shear structural testing

Epoxy	Wrap	Uni/ Bi	No. of Samples	sample designation	sample cast date	Sample Wrapping date	Indoor/ Outdoor	Outdoor Samples	Apply paint Y/N	Shear Type
SIKA										
SikaDur 300	103C	U	3	25, 26, 27	03/29/13	05/16/13	I	N	N	2-sided
	113C	B	3	28, 29, 30	03/29/13	05/16/13	I	N	N	2-sided
SikaDur 330	103C	U	3	10, 11, 12	02/01/13	03/07/13	I	N	N	2-sided
	113C	B	3	13, 14, 15	02/01/13	03/07/13	I	N	N	2-sided
SikaDur 300	113C	B	3	31, 32, 33	03/29/13	05/16/13	I	N	N	U-wrap
	230C	U	2	42, 43	03/29/13	05/16/13	I	N	N	U-wrap
SikaDur 330	113C	B	3	16, 17, 18	02/01/13	03/07/13	I	N	N	U-wrap
FYFE										
Tyfo S	SCH-41	U	3	73, 74, 75	03/29/13	05/15/13	I	N	N	2-sided
	BCC	B	3	76, 77, 78	03/29/13	05/15/13	I	N	N	2-sided
MB-3	SCH-41	U	2	79, 80	03/29/13	05/15/13	I	N	N	2-sided
Tyfo S	SCH-41	U	3	81, 82, 83	03/29/13	05/15/13	I	N	N	U-wrap
BASF										
Mbrace Primer/ Saturant	Mbrace CF130	U	2	110, 111	03/29/13	05/16/13	O	110, 111	y	2-sided
		U	2	112, 113	03/29/13	05/16/13	I	N	y	2-sided
		U	2	114, 115	03/29/13	05/16/13	I	N	y	2-sided
		U	3	116, 117, 118	03/29/13	05/16/13	I	N	y	U-wrap

Table 5.3.3 – FRP strengthened samples for confinement structural testing

Epoxy	Wrap	Uni/ Bi	No. of Samples	sample designation	sample cast date	Sample Wrapping date	Indoor/ Outdoor	Outdoor Samples	Apply paint Y/N	Sample Type
SIKA										
SikaDur 300	103C	U	3	44, 45, 46	03/29/13	05/16/13	I	N	N	4"x8" Cyl.
FYFE										
Tyfo S	SCH-41	U	3	92, 93, 94	03/29/13	05/15/13	I	N	N	4"x8" Cyl.
Tyfo SW	SCH-41-0.5X	U	3	95, 96, 97	03/29/13	05/15/13	I	N	N	4"x8" Cyl.
BASF										
Mbrace Primer/ Saturant	Mbrace CF 130	U	3	119, 120, 121	03/29/13	05/16/13	I	N	N	4"x8" Cyl.

Table 5.3.4 – FRP strengthened samples for pull-off adhesion testing

Epoxy	Wrap	Uni/ Bi	No. of Samples	sample designation	sample cast date	Sample Wrapping date	Indoor/ Outdoor	Outdoor Samples	Apply paint Y/N	Type
SIKA										
SikaDur 300	103C	U	2	34, 35	03/29/13	05/16/13	I/O	34	1/2 both samples	4"x2"x16"
	113C	B	2	36, 37	03/29/13	05/16/13	I	1	N	4"x2"x16"
SikaDur 330	103C	U	2	38, 39	02/01/13	03/07/13	I/O	38	1/2 both samples	4"x2"x16"
	113C	B	2	40, 41	02/01/13	03/07/13	I	1	N	4"x2"x16"
FYFE										
Tyfo S	SCH-41	U	2	84, 85	03/29/13	05/15/13	I/O	84	1/2 both samples	4"x2"x16"
	BCC	B	2	86, 87	03/29/13	05/15/13	I	N	N	4"x2"x16"
Tyfo SW	SCH-41	U	2	88, 89	03/29/13	05/15/13	I/O	88	1/2 both samples	4"x2"x16"
MB-3	SCH-41	U	2	90, 91	03/29/13	05/15/13	I	N	N	4"x2"x16"
BASF										
Mbrace Primer/ Saturant	Mbrace CF 130	U	3	122, 123, 124	03/29/13	05/16/13	I/O	122	Y	4"x8" Cyl.
		U	3	125, 126, 127	03/29/13	05/16/13	I/O	125	N	4"x8" Cyl.

Table 5.3.5 – FRP strengthened samples for pull-off adhesion testing

Strengthening type	Sample type	Strengthened side/s	No. of Samples
Flexural	beam	bottom	53
Shear	beam	2 sides	26
Flexural/Shear	beam	3 sides	14
Pull-off Test	½-thickness beam	1 side	22
Confinement	cylinder	around	12
		TOTAL	**127**

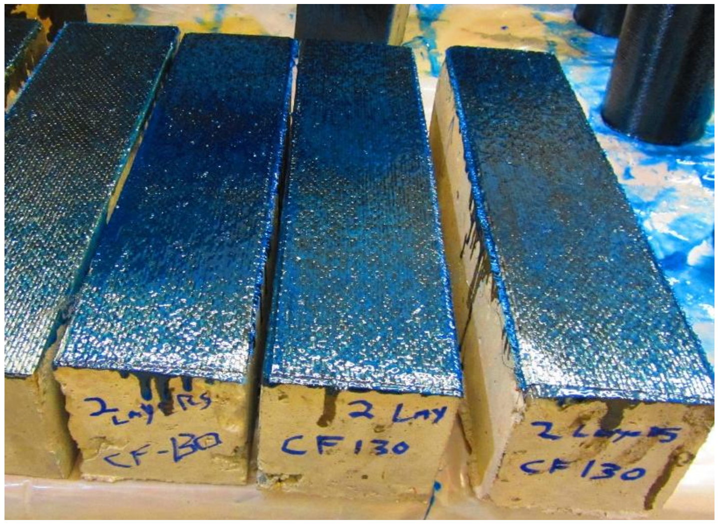

Figure 5.3.1 – FRP strengthened samples for flexure using FRP wrap (fabric)

Figure 5.3.2 – FRP strengthened samples for flexure using FRP plates (sheets)

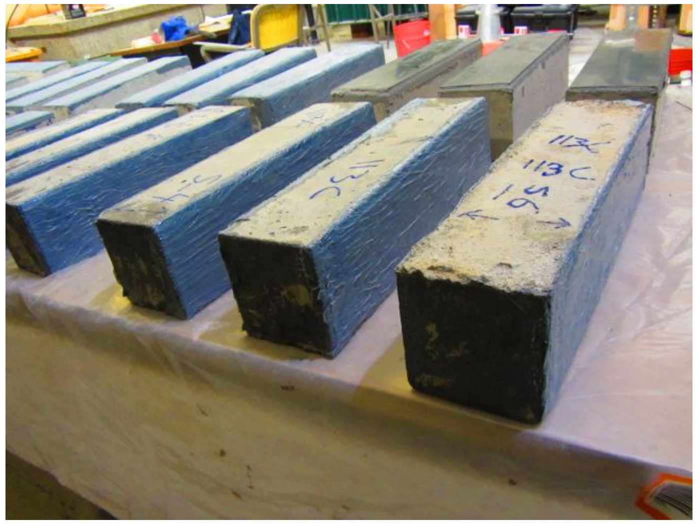

Figure 5.3.3 – FRP strengthened samples for 2-sided shear

Figure 5.3.4 – FRP strengthened samples using U-wrap

Figure 5.3.5 – Pull-off test samples using FRP wrap

Figure 5.3.6 – FRP strengthened confinement samples

Figure 5.3.7 – Outdoor samples exposed to natural weathering

5.3.2 Test plan

As noted earlier in this Chapter, for each outdoor sample set, an identical set is kept for accelerated testing and a control set is kept indoors at room temperature. Pull-off testing on outdoor samples, identical samples exposed to accelerated testing, and control samples at appropriate time intervals help quantify the acceleration factor. Once established, the acceleration factor can be used to estimate performance loss without the need for time-intensive outdoor weathering tests.

Outdoor samples were tested at 9 months and 14 months. Additional samples have been saved for future testing to refine the acceleration factors. Accelerated samples are conditioned in a

Tenney T10RC-1.5 SPL environmental chamber (Figure 5.3.8) in programmed runs. Each run consists of 60 designed cycles followed by pull-off testing of (see Figure 5.3.9 for a single accelerated testing cycle). As conditioning progressed, accelerated test samples were tested at 60, 120, 180, and 240 cycles for half-painted samples. These samples have been saved for additional weathering and future pull-off testing to further refine the results as needed. Tables 5.3.6 and 5.3.7 present the testing schedule for accelerated and outdoors test samples.

Figure 5.3.8 – Tenney environmental chamber

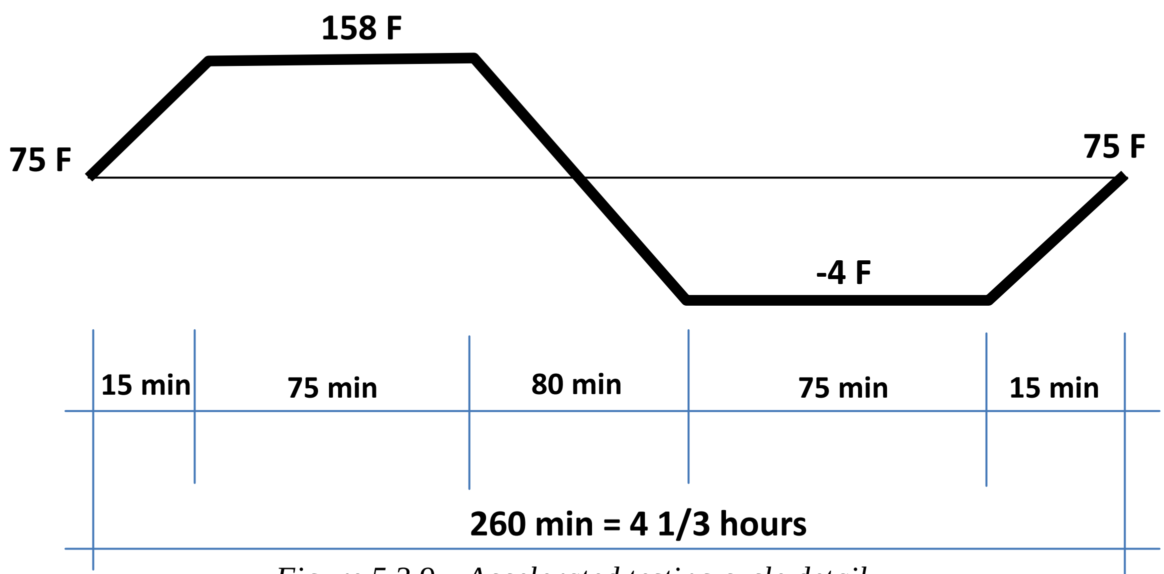

Figure 5.3.9 – Accelerated testing cycle detail

Table 5.3.6 – Accelerated specimens subjected to pull-off tests

Epoxy	Wrap	Uni/ Bi	Sample No.	Indoor/ Outdoor	Paint Y/N	Type
SIKA						
SikaDur 300	103C	U	**35**	I	half painted	4"x2"x16"
SikaDur 330	103C	U	**39**	I	1/2 painted	4"x2"x16"
FYFE						
Tyfo S	SCH-41	U	**85**	I	1/2 painted	4"x2"x16"
Tyfo SW	SCH-41	U	**89**	I	half painted	4"x2"x16"
BASF						
Primer/ Saturant	CF 130	U	124	I	Y	4"x2"x16"
Primer/ Saturant	CF 130	U	127	I	Y	4"x2"x16"
Submerged sample in distilled water						

Note: 1/2 painted samples are large enough for 5 sets of 4-dollies (20 pull-off tests) each, while fully painted or unpainted samples have 10 sets (40 pull-off tests) each.

Table 5.3.7 – Outdoor specimens subjected to pull-off tests

Epoxy	Wrap	Uni/ Bi	Sample No.	Indoor/ Outdoor	Paint Y/N	Size
SIKA						
SikaDur 300	103C	U	**34**	O	1/2 painted	4"x2"x16"
SikaDur 330	103C	U	**38**	O	1/2 painted	4"x2"x16"
FYFE						
Tyfo S	SCH-41	U	**84**	O	1/2 painted	4"x2"x16"
Tyfo SW	SCH-41	U	88	O	1/2 painted	4"x2"x16"
BASF						
Primer/ Saturant	CF 130	U	122	O	Y	4"x2"x16"
Room Temperature - control sample						
Primer/ Saturant	CF 130	U	125	O	Y	4"x2"x16"
Primer/ Saturant	CF 130	U	126	I	Y	4"x2"x16"
Room Temp Control sample	Submerged-distilled water					

5.3.3 Pull-off testing

Pull-off adhesion testing is accomplished using a DeFelsko PosiTest AT-A automatic adhesion tester (Figure 5.3.10). The test is conducted in accordance with ASTM D4541-09 *Standard test method for pull-off strength of coatings using portable adhesion testers*. The test involves several steps:

<u>Step 1 – Dolly and surface preparation</u>
Contaminants are removed from the dolly attachment surface and the sample surface using abrasive pads. Residue left from the abrading process is then removed using a dry cloth or paper towel. The area is then degreased with acetone or alcohol.

<u>Step 2 – Adhesive and dolly application</u>
Epoxi-Patch adhesive (Loctite 907- Hysol) is applied uniformly on the base of the dolly, then the dolly is attached to the sample surface. The dolly is pressured in place and excess adhesive is removed from around the dolly.

<u>Step 3 – Test area separation after full curing of adhesive</u>
A coring tool that fits the outside of the dolly is used to cut the FRP and just reach the concrete substrate. This helps to obtain a true measure of adhesion not affected by the surrounding fibers.

<u>Step 4 – Pull-off test</u>
The test is performed according to the PosiTest instructions using hydraulic pressure to separate the dolly from the sample surface. The tensile strength value is displayed on a digital screen and results may be saved to computer though a USB port.

A 20 mm diameter dolly size was used along with a coring bit with a 23 mm outside diameter for dolly separation in Step 3. A table-top drill press is used to achieve dolly separation prior to pull-off testing. Figure 5.3.11 shows two sets of pull-off dollies installed on sample no. 85. Figure 5.3.12 shows pull-off testing in progress.

Figure 5.3.10 – Pull-off adhesion tester (DeFelsko)

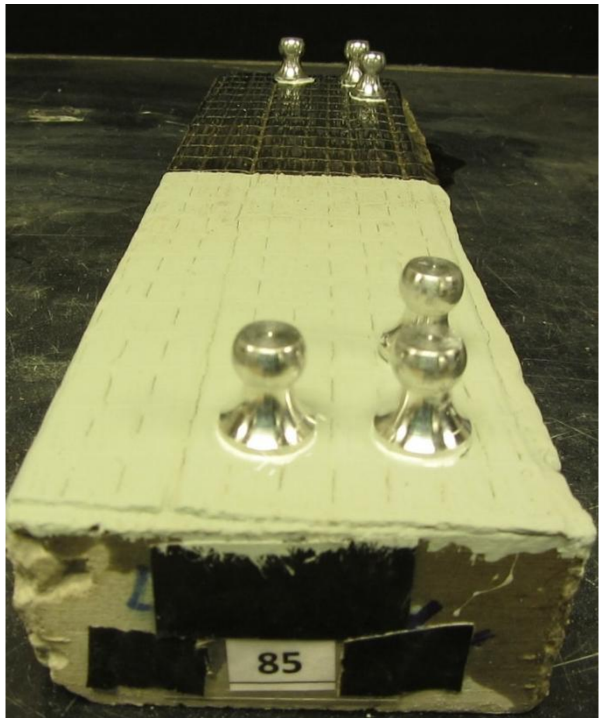

Figure 5.3.11 – Installation of dollies on a half-painted sample

Figure 5.3.12 – Pull-off adhesion testing underway

5.3.4 Flexural, shear and confinement testing

A 3-point loading scheme was used for the flexural samples. A span of 14 in was considered for the beam samples strengthened at the bottom face with a single ply of FRP fabric. Table 5.3.1 lists the flexural sample number designations and the flexural strengthening fabric used. Figure 5.3.13 illustrates the flexural sample loading set-up.

Preliminary tests of shear samples revealed that, after trying several load arrangements, it was not possible to fail the shear-strengthened specimens in shear prior to a moment failure. Therefore, the remaining shear specimens were saved for future consideration.

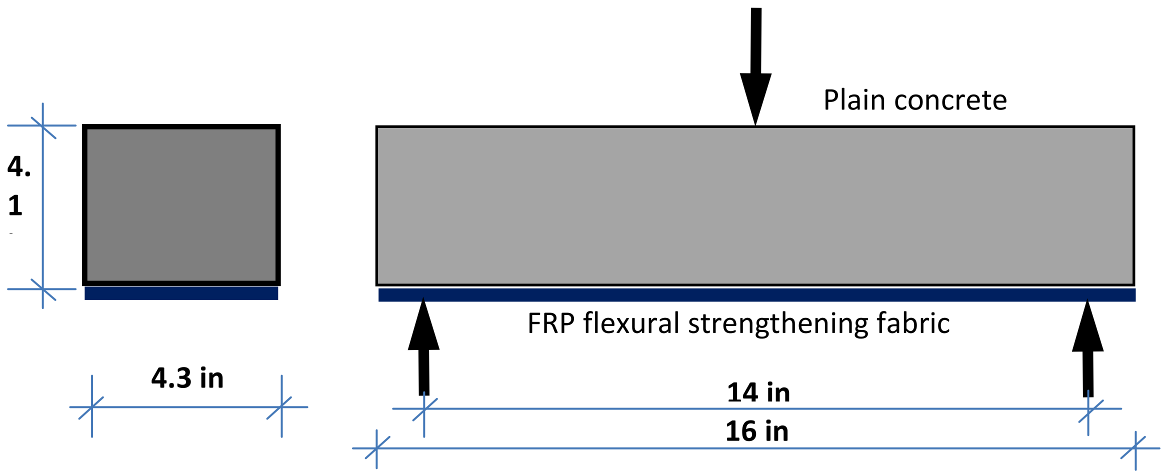

Figure 5.3.13 – Loading set-up of a flexural sample

5.4 Test Results

Durability testing involved pull-off testing as well as structural testing of flexural and confinement samples. Pull-off testing was conducted on outdoors, accelerated, and control samples to assess degradation and quantify acceleration factors. Structural testing was conducted on accelerated and control samples. In the following sections, pull-off and structural test data and analysis are presented.

5.4.1 Pull-off testing

Table 5.4.1 presents pull-off test data for outdoor samples while Table 5.4.2 presents data for accelerated samples.

Table 5.4.1 – Results of completed pull-off testing of outdoor samples

Sample Number	Epoxy	Wrap	Paint Y/N	control pull off test results, psi	Type of failure	9-month Pull off test result, psi	Type of failure	14-month Pull off test result, psi	Type of failure
34	SikaDur 300	103C	Y	1014	concrete	802	concrete	833	concrete
				424	concrete	929	concrete	645	concrete
				950	concrete	861	concrete	749	concrete
				930	concrete			693	concrete
			N	1014	concrete	571	Adhesive	856	concrete
				424	concrete	835	concrete	795	FRP
				950	concrete	695	concrete	576	FRP
				930	concrete			850	concrete
38	SikaDur 330	103C	Y			552	concrete	636	concrete
						729	concrete	807	concrete
						724	concrete	686	concrete
						724	concrete	854	concrete
			N	715	concrete	755	concrete	647	concrete/ FRP
				740	concrete	566	concrete	787	concrete/ FRP
				614	concrete	625	FRP	718	concrete
				932	concrete			542	concrete

Table 5.4.1 – Results of completed pull-off testing of outdoor samples, Cont.

Sample Number	Epoxy	Wrap	Paint Y/N	control pull off test results, psi	Type of failure	9-month Pull off test result, psi	Type of failure	14-month Pull off test result, psi	Type of failure
84	Tyfo S	SCH-41	Y			828	concrete	833	concrete
						1083	concrete	645	concrete
						875	concrete	749	concrete
								693	concrete
			N	720	concrete	995	concrete	404	FRP
				619	concrete	1028	concrete	652	concrete
				530	concrete	779	concrete	609	FRP
				659	concrete			613	FRP
88	Tyfo SW	SCH-41	Y			487	concrete/ FRP	487	concrete/FRP
						201	concrete/ FRP	201	concrete/FRP
						712	concrete/ FRP	712	concrete/ FRP
			N	581	Adhesive	408	concrete/ FRP	408	concrete/ FRP
				387	Adhesive	284	concrete/ FRP	284	concrete/ FRP
				397	Adhesive	178	concrete/ FRP	178	concrete/ FRP
				339	concrete/ FRP				
122	Primer/ Saturant	CF130	Y	1015	concrete	704	concrete	888	concrete
				1078	concrete	748	concrete	899	concrete
				867	concrete	741	concrete	497	concrete
				1135	concrete			1161	concrete
				1183	concrete				
125	Primer/ Saturant	CF130	Y	1015	concrete	578	concrete	591	concrete
				1078	concrete	556	concrete	933	concrete
				867	concrete	622	concrete	900	concrete
				1135	concrete			993	concrete
				1183	concrete				
126	Primer/ Saturant	CF130	Y	1015	concrete	834	concrete	887	concrete
				1078	concrete	638	concrete	979	concrete
				867	concrete	898	concrete	572	concrete
	Submerged -control - INDOOR			1135	concrete	1135		946	concrete
				1183	concrete	1183			

Table 5.4.2 – Results of completed pull-off testing of accelerated samples

Sample No.	Epoxy	Wrap	Paint Y/N	60 cycles, psi	Failure Mode	120 cycles, psi	Failure Mode	180 cycles, psi	Failure Mode	240 cycles, psi	Failure Mode
35	SikaDur 300	103C	Y	674	concrete	551	concrete/FRP	925	concrete/FRP	398	concrete
				396	concrete	656	concrete	809	concrete	642	concrete/ FRP
				724	concrete	705	concrete	589	concrete	732	concrete/ FRP
						662	concrete	795	concrete/FRP	823	conc
			N	843	concrete	772	concrete	545	concrete	694	concrete/ FRP
				292	concrete	838	concrete	679	concrete	685	concrete
				573	concrete	682	concrete	708	concrete	599	concrete
						935	concrete	725	concrete	723	concrete
85	Tyfo S	SCH-41	Y	636	concrete	617	concrete	421	concrete	60	concrete
				758	concrete	584	concrete	662	concrete	242	concrete
				773	concrete	528	concrete	0	epoxy	562	concrete
						483	concrete	496	concrete	0	concrete-disqualify
			N	727	concrete	591	concrete	302	concrete	179	conc/FRP
				721	concrete	395	concrete	221	concrete	536	concrete
				550	concrete	571	concrete	117	concrete	737	concrete/ FRP
						664	concrete	711	concrete	0	concrete-disqualify
89	Tyfo SW	SCH-41	Y	439	concrete	483	concrete/FRP	512	concrete	398	concrete
				249	concrete	339	FRP	541	concrete	212	concrete
				143	concrete	492	FRP	342	concrete	473	concrete
						285	FRP	369	concrete/ FRP	276	concrete
			N	402	concrete	251	FRP	316	concrete	263	concrete/ FRP
				386	concrete	240	FRP	260	concrete/ FRP	174	adhesive/concrete
				421	concrete	323	FRP	311	concrete	270	concrete
						375	FRP	313	concrete	325	concrete
124	Primer/ Saturant	CF130	N	720	concrete	707	concrete	971	concrete	537	concrete/ FRP
				721	concrete	803	concrete	951	concrete	684	concrete
				894	concrete	692	concrete	471	concrete	983	concrete
						816	concrete	933	concrete	536	concrete
127 Submerged in distilled water	Primer/ Saturant	CF130	Y	838	concrete	693	concrete	520	concrete	865	concrete
				895	concrete	779	concrete	1049	concrete	986	concrete
				791	concrete	736	concrete	698	concrete	939	concrete
						821	concrete	916	concrete	933	concrete

5.4.1.1 Evaluation of acceleration factor

To best estimate the acceleration factor, it is desirable to use the same FRP materials throughout the time period of consideration, and this time period should be as long as possible. As discussed in Chapter 6, a valuable long-term data point (15 year) was established for BASF (MBrace) material via field testing. Therefore, BASF (MBrace) laboratory samples 124 (accelerated) and 125 (outdoor) are used in addition to the field test result. Pull-off test results of outdoor sample 125, along with the field test result, are plotted in Figure 5.4.1a. Pull-off test results of the identical accelerated sample 124 are plotted in Figure 5.4.1b.

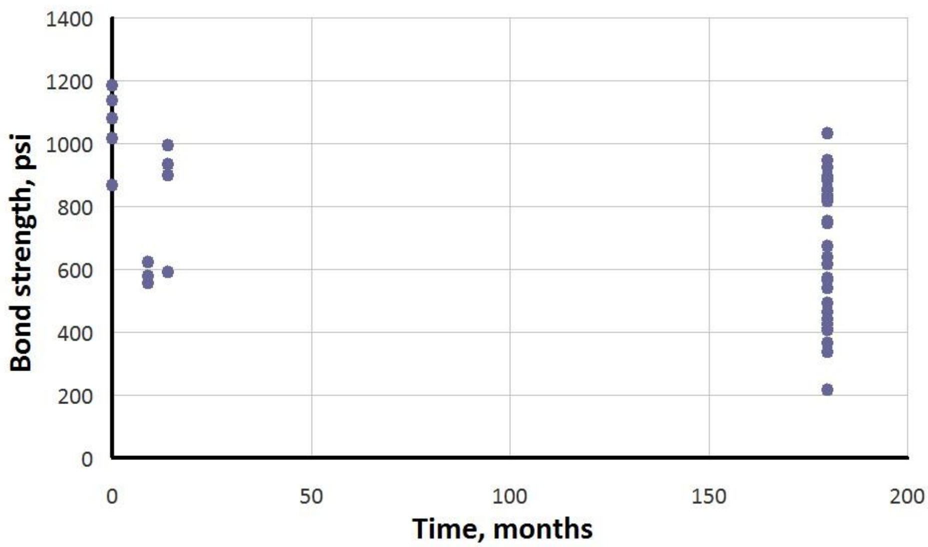

Figure 5.4.1a – Bond strength vs time for outdoor samples

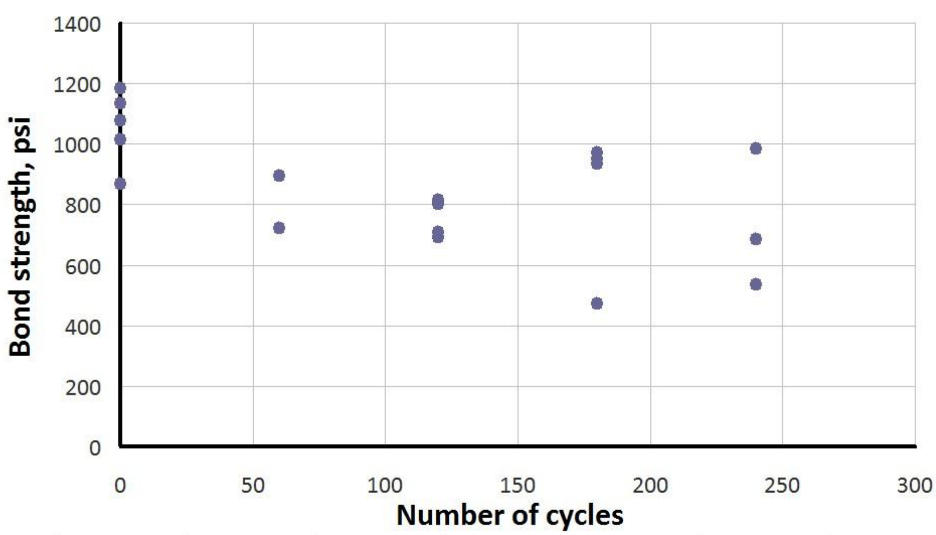

Figure 5.4.1b – Bond strength vs number of cycles for accelerated sample 124

Referring to the data presented in Figure 5.4.1b, the mean bond strength of the specimens clearly decreases for each cycle, and is as follows:

Number of Cycles	Mean bond strength
0	1056
60	778
120	754
180	832
240	685

Considering the data in Figures 5.4.1a and 5.4.1b, the largest losses occur within the first 60 cycles (accelerated) and 9 months (outdoors), then the rate of loss significantly decreases. Eliminating the first point on the graphs (i.e. time zero), a linear regression is conducted to best-fit the tail of the data for long-term use. These are given below in Figures 5.4.2a and 5.4.2b.

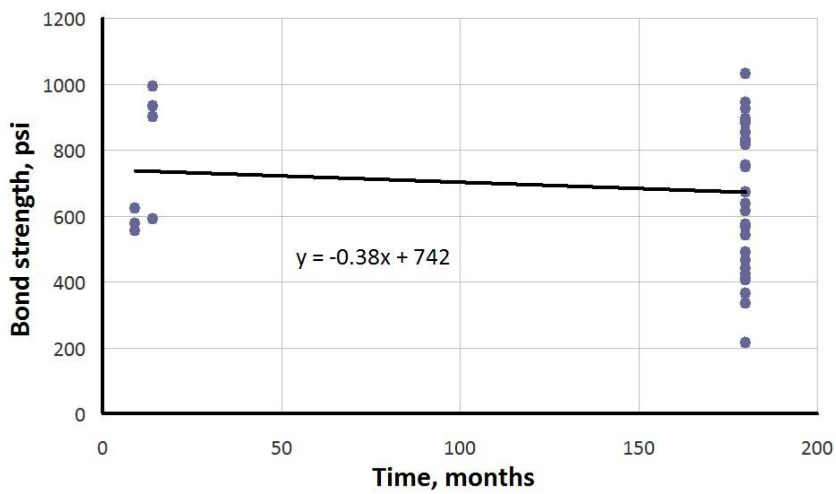

Figure 5.4.2a – Bond strength vs time for outdoor samples

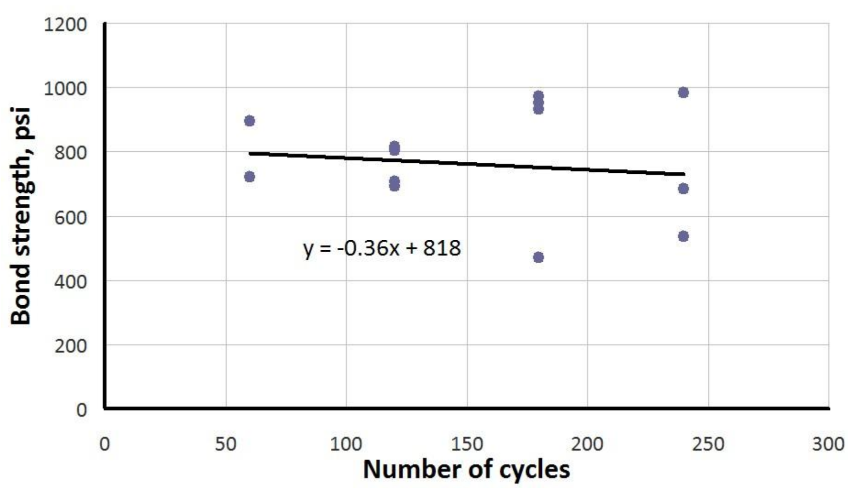

Figure 5.4.2b – Bond strength vs number of cycles for accelerated sample 124

For the accelerated data, the resulting regression line becomes: b=818-0.36c, where b is bond strength and c is number of cycles. For the outdoor samples, the line becomes: b=742-0.38t, where t is time in months.

To evaluate the long-term acceleration factor, the time and number of cycles required for the same loss of strength are related. For example, referring to the regression line for the accelerated samples, a loss of 100 psi of strength (i.e. b=718 psi) can be found to be 278 cycles. Similarly, for the outdoor samples, it requires 263 months, or 22 years of exposure, for the same loss of strength (b=642).

The long-term acceleration factor is therefore estimated to be 278 cycles / 22 years = approximately 12.7 cycles per year.

A similar short-term acceleration factor can be found by using the same process that was used to develop the long term factor, but now only the initial loss of strength (i.e. at time zero and at the first 60 cycles) is considered.

For the outdoor specimens, the best-fit regression line between the 0 month and the 9 – 14 month data is: b = 1003 -19t. For the accelerated specimens, the relationship describing the loss of strength from time 0 to 60 cycles is: b = 1056 - 4.62c. This results in an estimated short-term acceleration factor of approximately 207 cycles / 48 months = 4.3 cycles / month = 52 cycles/year (valid for about the first year of exposure).

These acceleration factors may thus be used in future work efforts to estimate bond degradation without the need for long-term outdoor tests. However, as preliminary estimates for acceleration factors based on limited data sets and exposure conditions, the values requires refinement with additional testing. As noted earlier in this chapter, both indoor and outdoor test samples made during this project continue to be studied in the long-term at Wayne State University.
Using the acceleration factors or the regression fit to the outdoor weathering data directly, long-term capacity loss can be approximated. In particular, for the outdoor specimens, assuming an initial strength of 1003 psi, bond strength at the end of the first-year loss is expected to be b = 1003 - 19(12)) = 775 psi, which represents a proportional reduction to 775/1000 = 0.775. In 50 years, for example, the expected bond strength (assuming an initial starting value of 742 psi, per the regression equation) = 742 - 0.38(49x12) = 519 psi, or an expected additional proportional reduction to 519/742 = 0.70, which represents a total expected 50 year reduction to 0.775x0.70. = 0.54 or 54%.

Based on these results, the expected bond strength reduction factor is given below:

Time (years)	Reduction factor
10	0.73
25	0.66
50	0.54
75	0.42

Possible refinements in this estimate can be made with future testing. As noted above, additional specimens have been saved for this purpose, and continue to be subjected to outdoor and accelerated weathering. Any necessary adjustments to the acceleration factor, and resulting recommendations for environmental reduction factors, will be made available as additional testing is conducted in future years.

5.4.2 Testing of structural samples

Two types of structural testing are conducted; flexural and confinement. As noted, it was not possible to obtain the desired failure mode for the shear-strengthened specimens. Flexural samples include accelerated BASF samples 101 and 102 and identical BASF samples 103 and 104 as control samples. Sample dimensions and flexural test set-up are provided in Section 5.3.5. Three confinement samples were also used; two accelerated and one control. BASF samples 119 and 120 were accelerated for 60 cycles. The use of this small number of samples stems from a desire to save as many samples as possible for future use, where longer-term data are more valuable.

5.4.2.1 Flexural testing

As shown in Figure 5.3.14, the 16” x 4.3” x 4.1” strengthened beams were tested as simply supported over a 14 in span and loaded at the middle of the span according to ASTM C 293. The load was applied monotonically using a high–capacity MTS testing machine. Test data is recorded up to the sample failure. Four beam samples were tested: samples 101 and 102 (accelerated) and samples 103 and 104 (indoor control samples).

Figure 5.4.3 shows a plot of the applied load vs. displacement of accelerated (101 and 102) and control (103 and 104) beam samples. Test results are presented in Figures 5.4.4-5.4.7. Accelerated samples (101 and 102) primarily exhibited peeling and debonding failures while the failure mode of the control beam samples (103 and 104) appears to be combined with shear.. Table 5.4.3 presents the maximum recorded load during testing and the failure mode. Results reveal that conditioning for only 60 cycles has a significant impact, altering both load carrying capacity (30% reduction) as well as failure mode.

Table 5.4.3 – Results of accelerated and control flexural beam sample testing

Sample type	Sample No.	Max. Load, lbs	Average/type	Failure Mode
Accelerated	Beam 101	1301	1412	Peeling
	Beam 102	1522		Debonding/peeling
Control	Beam 103	1624	2026	Shear/peeling
	Beam 104	2427		Shear/debonding

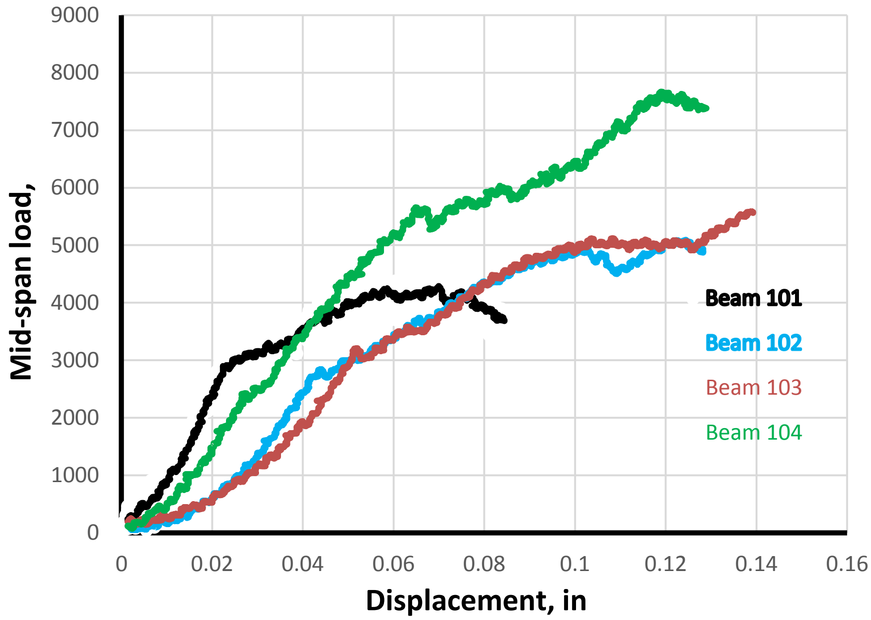

Figure 5.4.3 – Load vs displacement for accelerated (101, 102) and control (103, 104) beams

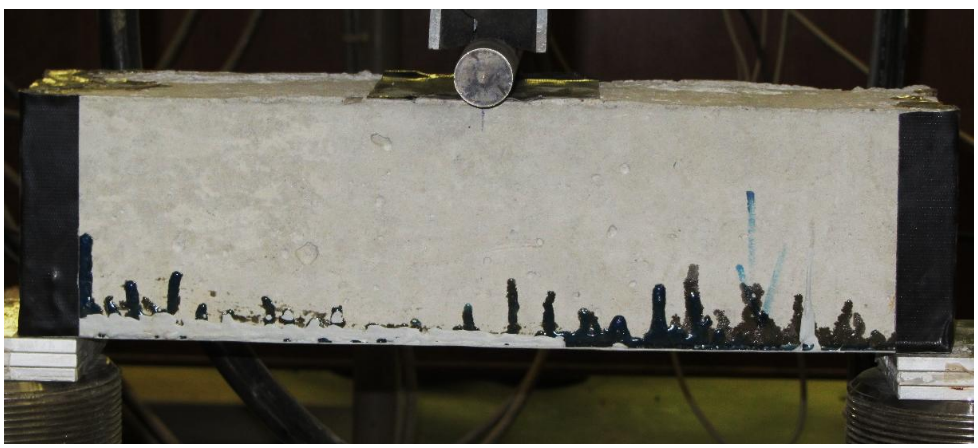

Figure 5.4.4a – Set up of accelerated sample 101

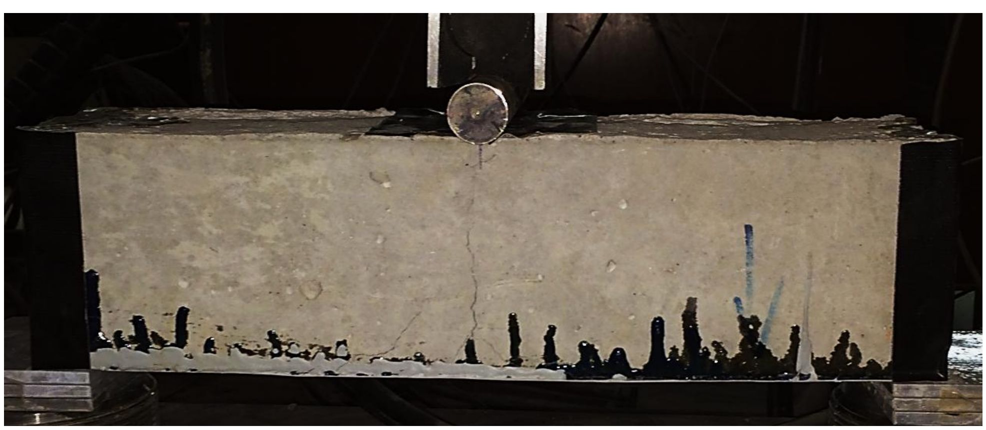

Figure 5.4.4b –Accelerated sample 101 initial cracking

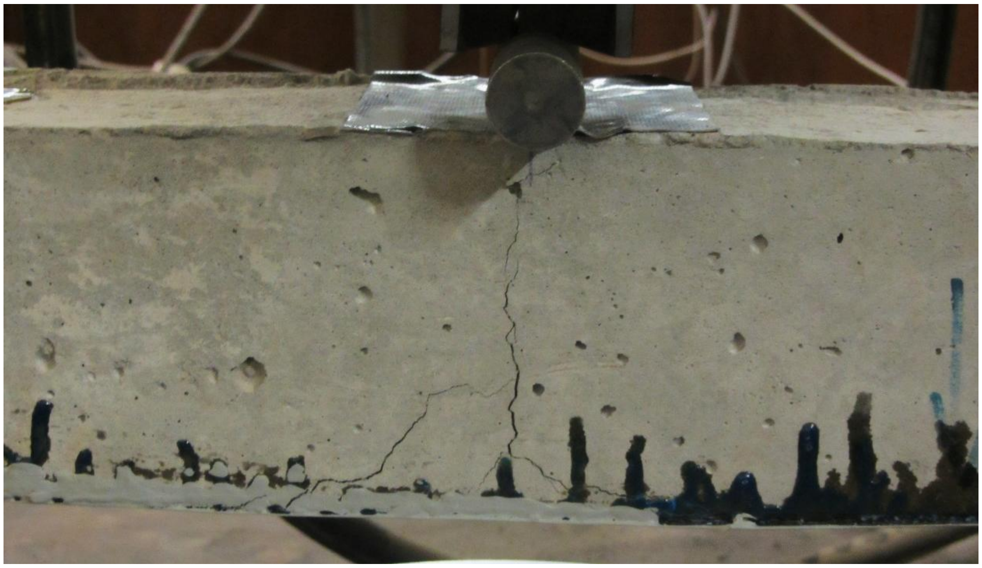

Figure 5.4.4c –Accelerated sample 101 initial cracking

Figure 5.4.4d –Accelerated sample 101 peeling failure

Figure 5.4.4e –Accelerated sample 101 failure plane

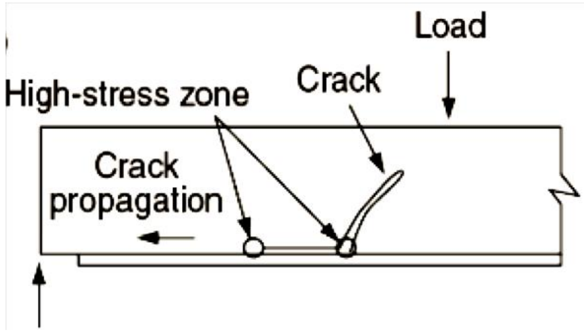

Figure 5.4.5 –Failure mode observed in sample 101 as peeling or shear/tension failure of concrete substrate

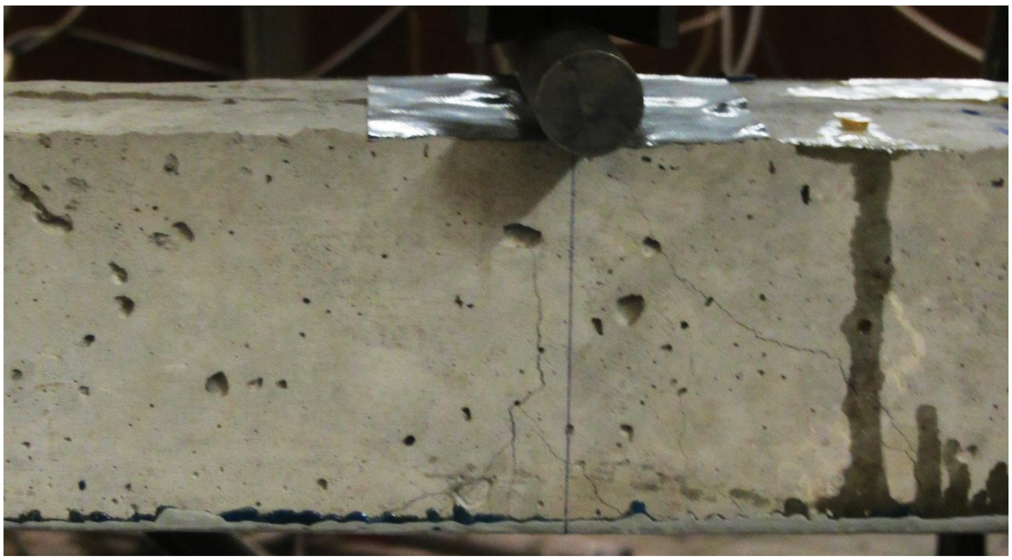

Figure 5.4.6a –Accelerated sample 102 initial cracks- note shear crack to right

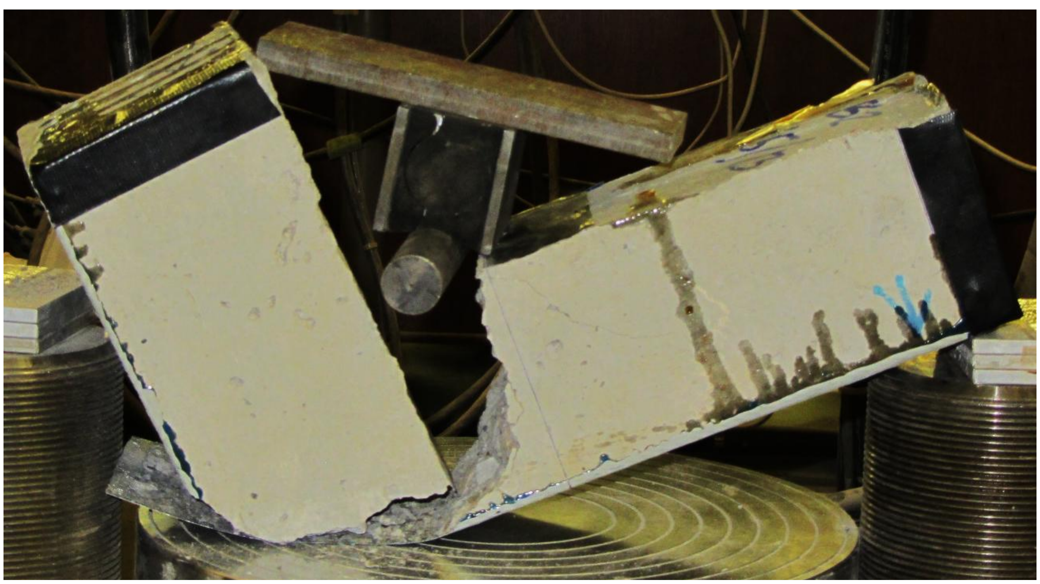

Figure 5.4.6b –Accelerated sample 102 failure (debonding/peeling)

Figure 5.4.7a –Control sample 103 initial hairline cracks (flexure and shear cracks)

Figure 5.4.7b –Control sample 103 shear crack

Figure 5.4.7c –Control sample 103 failure (shear/peeling)

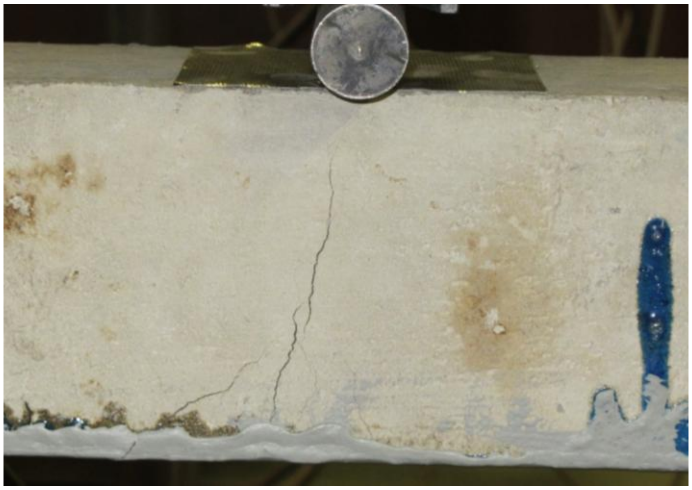
Figure 5.4.8a –Control sample 104 initial flexure cracking

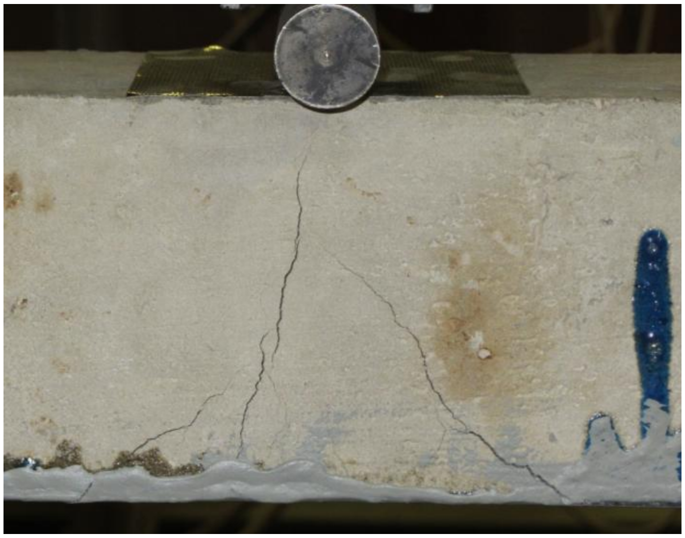
Figure 5.4.8b –Control sample 104 progressive cracking

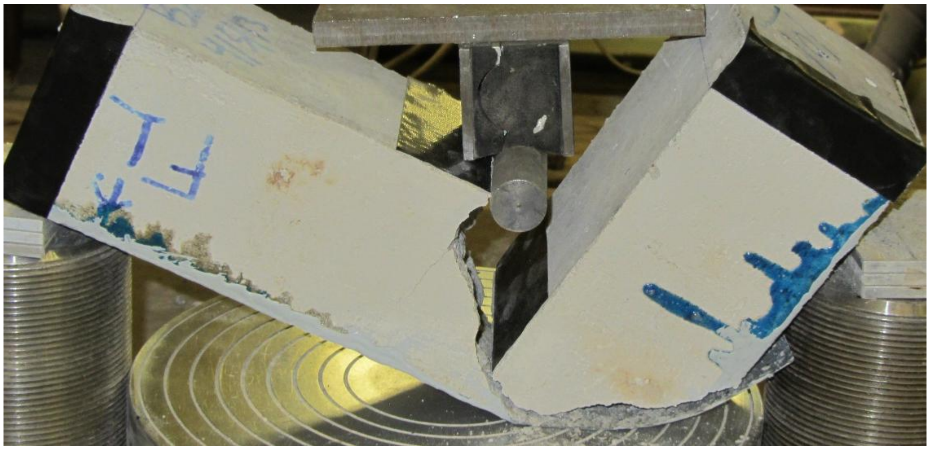
Figure 5.4.8c –Control sample 104 (shear/debonding)

5.4.2.2 Confinement testing

Three 4 in x 8 in cylinders were confined using MBrace CF130. Two of the three cylinders (samples 119 and 120) were conditioned in Tenney environmental chamber for 60 cycles. The remaining (sample 121) is a control sample maintained in indoor temperature and humidity. The control sample failed prematurely during testing due to a deficiency at the lower end of the cylinder causing the concrete to crumble and unwrap the FRP fabric in a domino effect with the unwrapping travelling upward as the test progressed. The axial load capacity of the control sample is therefore severely reduced and no meaningful comparison can be made with the accelerated samples.

However, the accelerated sample testing produced close results between the two samples. Sample number 119 failed due to FRP failure at the sample mid-section, with a failure load of 141 kips, corresponding to an ultimate compressive strength of 11.2 ksi. Confined sample number 120 failed due to FRP failure at mid-section as well. Failure load for sample 120 was 133 kips, corresponding to an ultimate compressive strength of 10.6 ksi. Figures 5.4.9-5.4.12 present compressive loads and compressive strengths of accelerated samples, in addition to photos showing the progressive damage of samples during compressive testing.

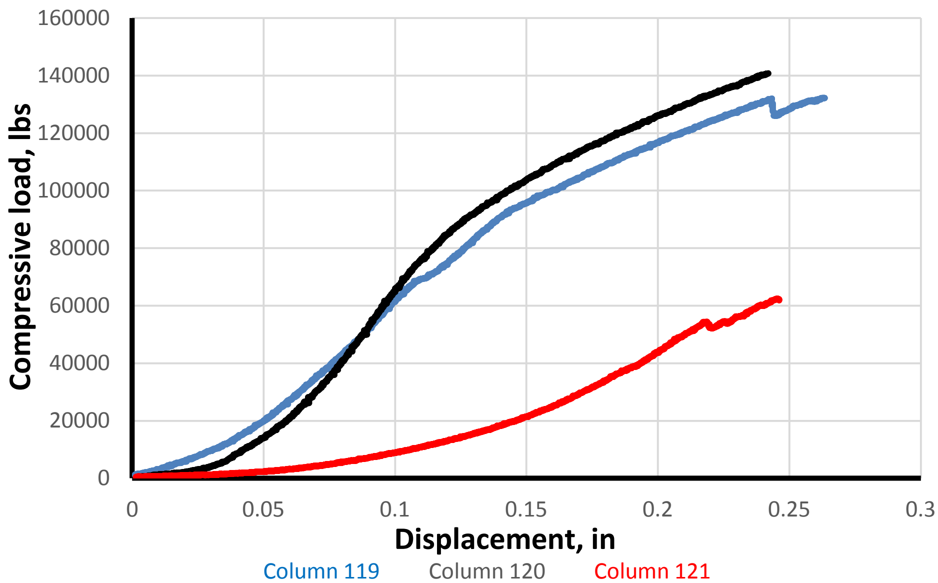

Figure 5.4.9 –Compressive strengths of accelerated confined samples 119 and 120, and failed control sample 121.

Figure 5.4.10 – Confinement sample 121 FRP failure due to lack of end confinement at the bottom prompting FRP to unravel upward

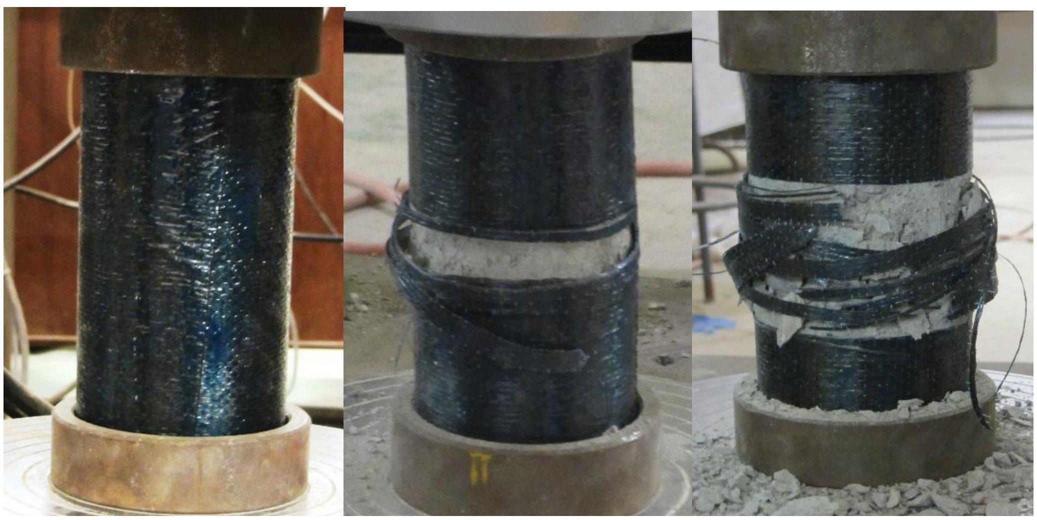

Figure 5.4.11 –Confinement sample 119 failure due to FRP rupture

Figure 5.4.12 –Confinement sample 120 failure due to FRP rupture

5.4.2.3 Summary

Of the structural-level specimen tests conducted, only the flexural specimens provided suitable results. However, the flexural test results do appear to correspond reasonably well to the pull-off test results. Figure 5.4.13 replots the accelerated test data (small circles; blue) along with the flexural tests (large circles; red). On the figure, the flexural test data in Table 5.4.3 are normalized such that the highest strength control specimen is equated to the highest strength pull-off specimen, with the remaining three flexural test results proportionally decreased. As shown on the figure, the normalized flexural test results follow the pull-off test trend closely. Thus, the corresponding rate of strength reduction at the bond level appears reasonably correlated to that at the structural failure mode level.

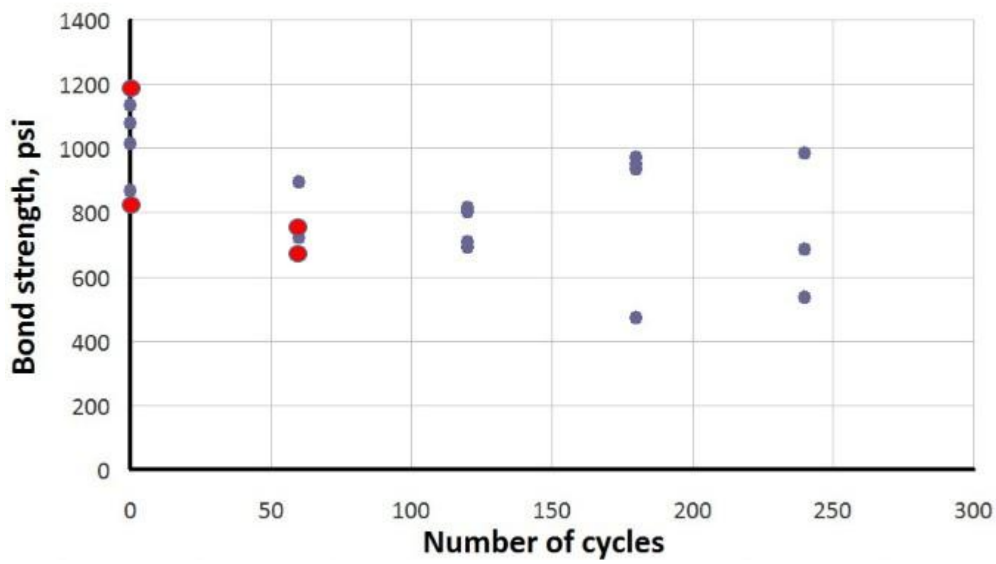

Figure 5.4.13 – Bond strength for pull-off specimens and flexural tests (larger symbols)

CHAPTER 6: FIELD TESTS

6.1 Introduction

To assess the pull-off strength of a relatively long-term FRP application, testing of adhesive bond was conducted on carbon-wrapped columns (numbers 2 and 6; see Figure 6.1) located beneath an existing bridge (#S09 and S10 of 23152) on west-bound I-96 over Lansing Road in Lansing, Michigan. The free-standing columns are located on the south/west-bound side of Lansing Road between the bridge piers and abutment. The tested columns were wrapped with Masterbuilders MBrace CF130 CFRP in July, 1999. Pull-off sample dollies ware installed on August 18, 2014 and tested on August 21, 2014. Work was conducted in accordance with standard installation and test procedures recommended by the test equipment manufacturer. In this case, 20 mm test dollies were installed using Loctite 907-Hysol Epoxi-Patch adhesive. A total of 33 dollies were installed on all sides of the two columns, with 4 dollies per face, except for face number 4 of column 2, where 5 dollies were installed due to the presence of visible corrosion on the concrete surface (see Figure 6.2). To ensure dollies maintain in contact with the column surface, rubber elastic membranes were taped to secure the dollies in position until the adhesive bond was fully developed (Figure 6.3).

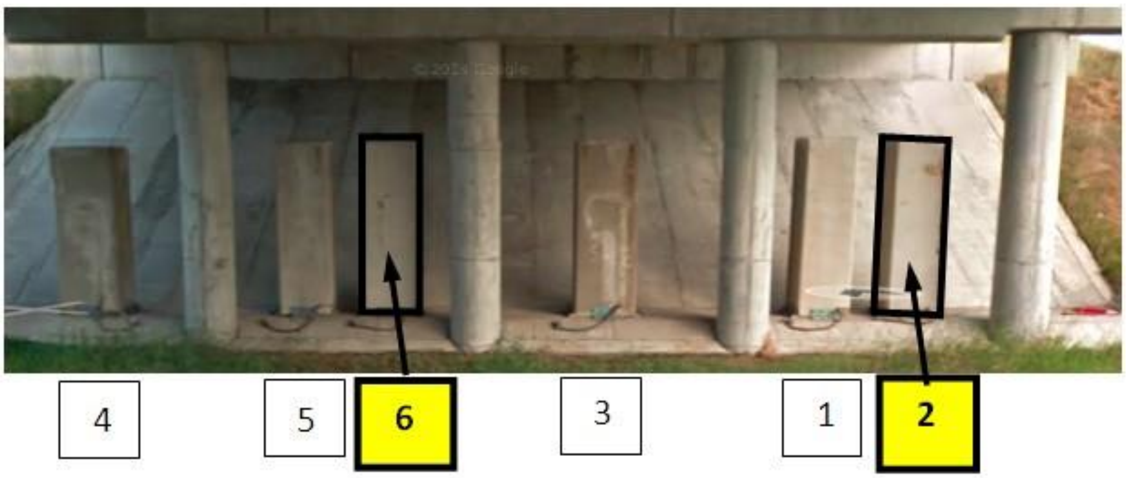

Figure 6.1 - Test columns

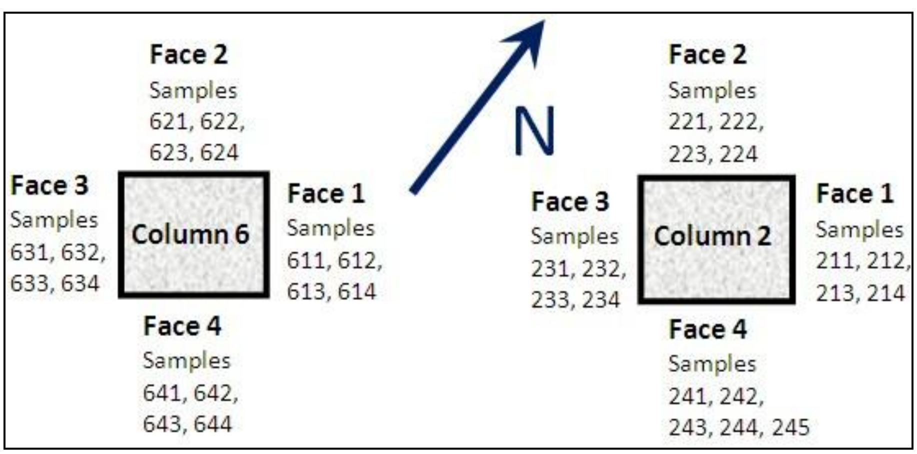

Figure 6.2 - Test sample location

Figure 6.3 - Secured test dollies

Test samples were designated with 3-digit numbers indicating column and face. For example, sample 621 represents a sample used on column 6, face 2, and number 1 (of 4) on that particular column face.

6.2 Dolly Installation and Testing

Field work took place on two days; Monday 8/18/14 (installation) and Thursday 8/21/14 (pull-off testing). Site work on Monday involved the installation of 33 dollies on columns 6 and 2. The installation process involved cleaning the column surfaces where test dollies were to be installed; roughening the base surfaces of the dollies using abrasive pads to enhance their adhesion; applying epoxy to the base of the dollies; then securing the dollies in place to allow the epoxy to cure by the use of a taped elastic membrane (Figures 6.3 and 6.4).

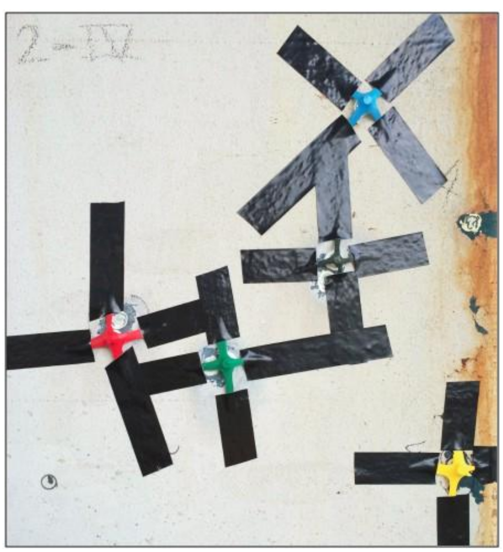

Figure 6.4 - Test dollies installed on column 2, face 4

To allow the epoxy adhesive sufficient time to develop full strength as recommended by the manufacturer, pull-off testing was conducted three days after installation. The following procedure was used to prepare samples for testing in accordance with manufacturer's recommendations and in conformance to ASTM D4541 (2002):

1. Remove tape and rubber membranes to expose the dollies (Figure 6.5).
2. Core around the base of the dollies to just cut through the carbon strengthening wrap using a 22 mm diamond coring bit and a drill guide (Figure 6.6). Care was taken not to cut into the concrete surface, which can influence test results.
3. Conduct adhesion testing with a PosiTest AT-A Automatic Adhesion Tester and record test data (Figure 6.7).

Figure 6.5 – Removal of tape and elastic membrane to expose dollies for testing

Figure 6.6 – Coring around dollies for proper adhesive bond testing

Figure 6.7 –Adhesive bond testing using PosiTest AT-A Automatic Adhesion Tester

6.3 Test Results

Test results are presented in Tables 6.1-6.8, while specimen failure surfaces are given in Figures 6.8 and 6.9. As seen in the tables, there is significant variation in test results, with mean overall bond strength of 668 psi and mean coefficient of variation (COV) of 0.29 (with range from 0.14-0.44 per column face). Several failure modes were observed, including concrete, FRP interface, and combined concrete/FRP interface failures. This variation in failure mode contributes to the large variation in pull-off strength. However, the majority were concrete failures, indicating the bond strength was greater than the concrete strength (in pull-off tension) in most cases. In three cases (samples 634, 644, and 224), the epoxy adhering the dolly to the FRP failed (called "adhesive" failures in the tables). These results were included, however, as they were within the strength range of the concrete and FRP failure modes, and represent the lower bound of possible strength for those particular test cases.

Studying the failure surfaces of the non-concrete failure cases, two reasons were uncovered that contributed to failure of the adhesive and/or FRP. The first is that the FRP was not fully saturated with epoxy in all cases. This can be seen most clearly in the case of sample 623, which had zero pull-off strength as there was practically no epoxy in the FRP at the test location, as shown in Figure 6.10. The second case was that of air bubbles present in the failure surface. This was seen throughout multiple samples (212, 612, 232, and 233), as shown in Figure 6.11.

The results of these tests were considered in development of the environmental factors, as discussed in Chapter 5.

Table 6.1 - Column 6, Face 1 test results and failure modes

Sample	Adhesive Bond Strength, psi	Average, psi	Standard Deviation (COV)	Failure Mode
611	881	796	121 (0.15)	Conc./FRP
612	616			Conc./FRP
613	834			Conc./FRP
614	851			Conc./FRP

Table 6.2 - Column 6, Face 2 test results and failure modes

Sample	Adhesive Bond Strength, psi	Average, psi	Standard Deviation (COV)	Failure Mode
621	1032	843	180 (0.21)	conc.
622	825			conc.
623	0*			FRP
624	673			conc./FRP

*Not included in calculations. Sample fell off upon completion of coring; no pull-off test possible.

Table 6.3 - Column 6, Face 3 test results and failure modes

Sample	Adhesive Bond Strength, psi	Average, psi	Standard Deviation (COV)	Failure Mode
631	854	650	194 (0.30)	conc.
632	410			conc.
633	747			conc.
634	588*			adhesive

Table 6.4 - Column 6, Face 4 test results and failure modes

Sample	Adhesive Bond Strength, psi	Average, psi	Standard Deviation (COV)	Failure Mode
641	573	548	159 (0.29)	conc.
642	376			FRP*
643	491			conc.
644	753			adhesive

*Evidence of air bubbles.

Table 6.5 - Column 2, Face 1 test results and failure modes

Sample	Adhesive Bond Strength, psi	Average, psi	Standard Deviation (COV)	Failure Mode
211	465	389	144 (0.37)	Conc.
212	335			Conc./FRP*
213	541			Conc.
214	215			Conc.

*Evidence of air bubbles.

Table 6.6 - Column 2, Face 2 test results and failure modes

Sample	Adhesive Bond Strength, psi	Average, psi	Standard Deviation (COV)	Failure Mode
221	854	800	112 (0.14)	conc.
222	892			conc.
223	638			Conc./FRP*
224	816			adhesive

*Primarily concrete failure

Table 6.7 - Column 2, Face 3 test results and failure modes

Sample	Adhesive Bond Strength, psi	Average, psi	Standard Deviation (COV)	Failure Mode
231	894	693	275 (0.40)	conc.
232	365			Conc./FRP*
233	566			Conc./FRP*
234	945			conc.

*Evidence of air bubbles.

Table 6.8 - Column 2, Face 4 test results and failure modes

Sample	Adhesive Bond Strength, psi	Average, psi	Standard Deviation (COV)	Failure Mode
241	405	624	275 (0.44)	conc.
242	424			conc./FRP
243	925			conc.
244	442			conc.
245	924			conc.

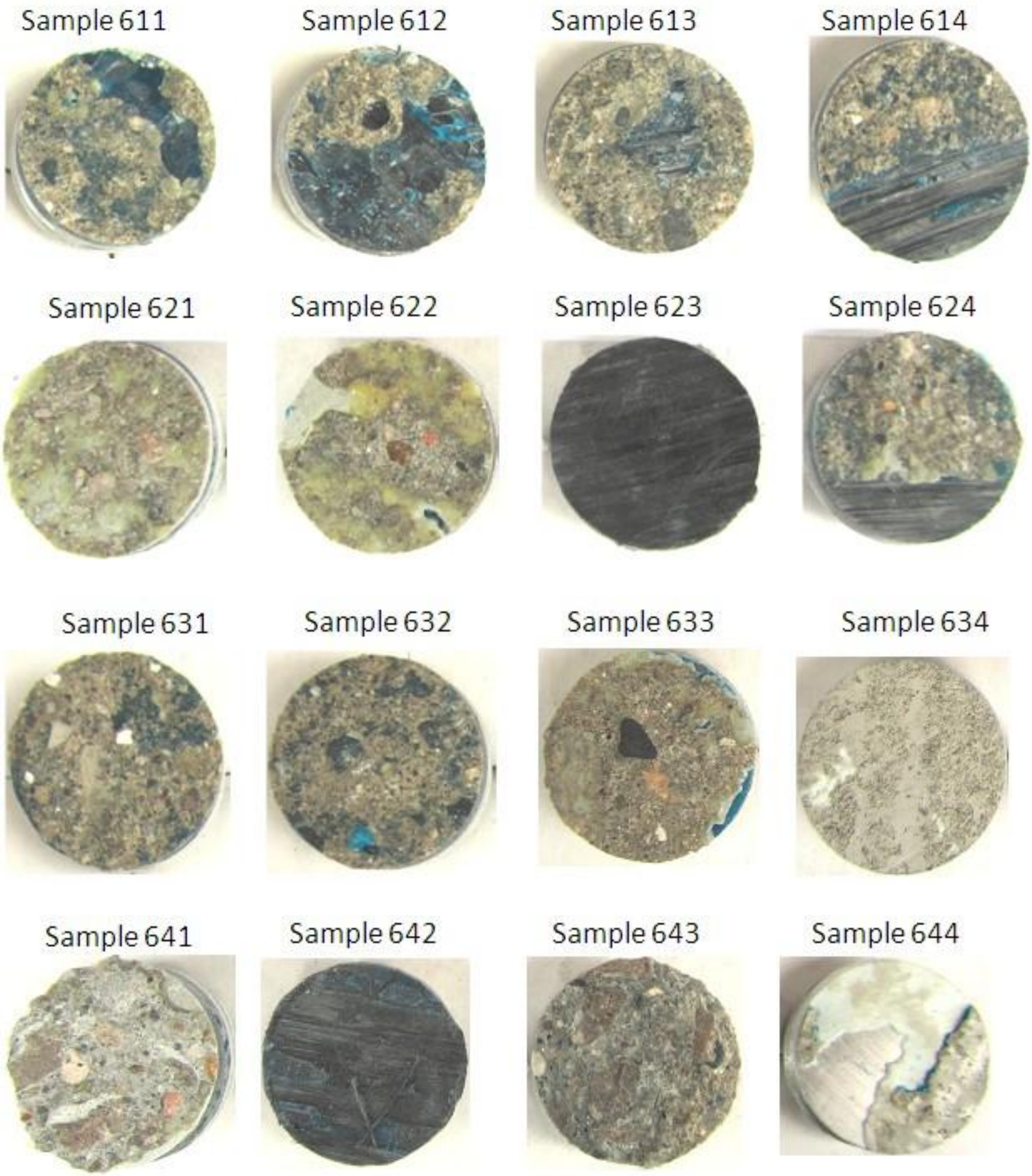

Figure 6.8 - Column 6 samples after testing

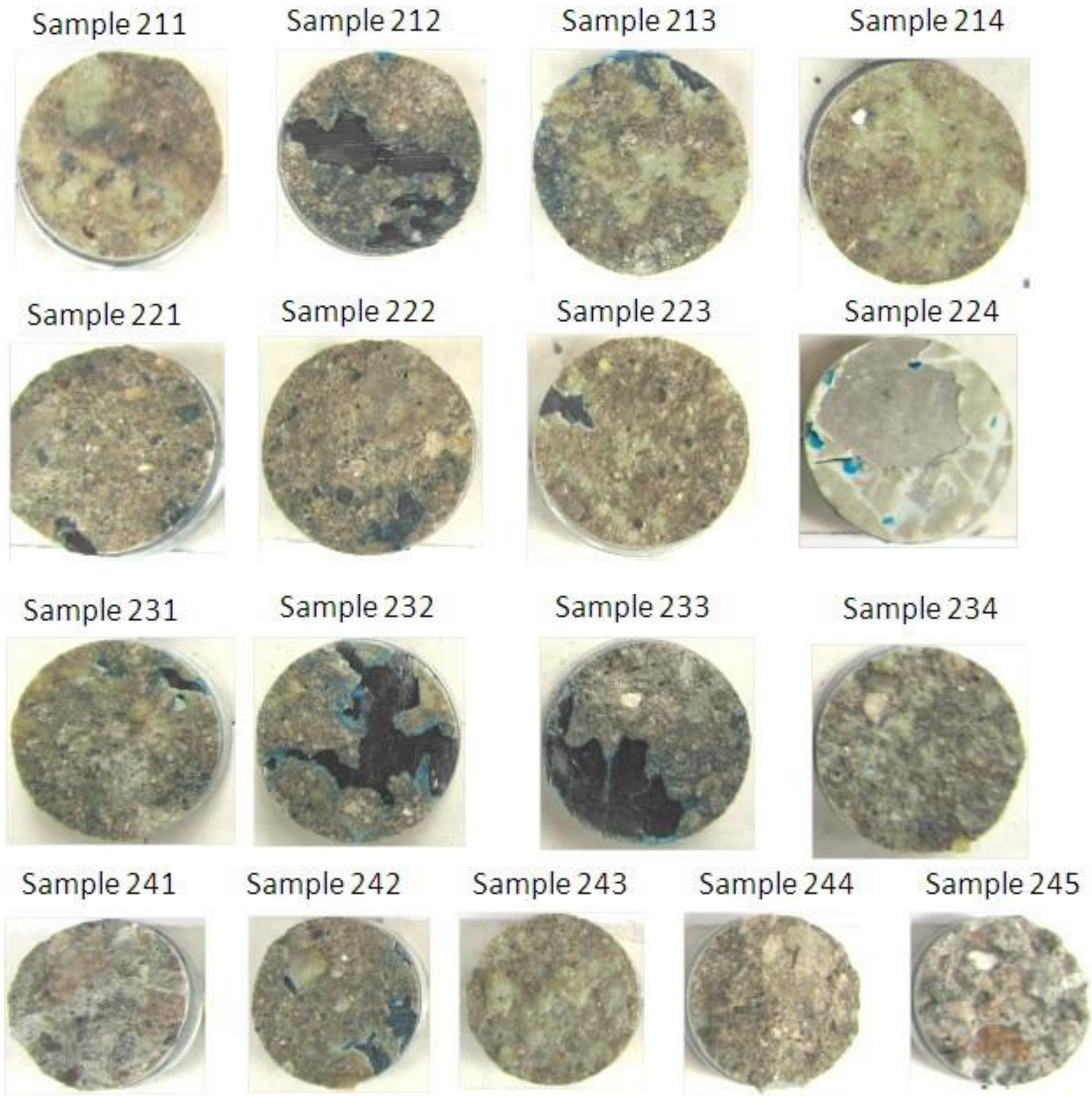

Figure 6.9 - Column 2 samples after testing

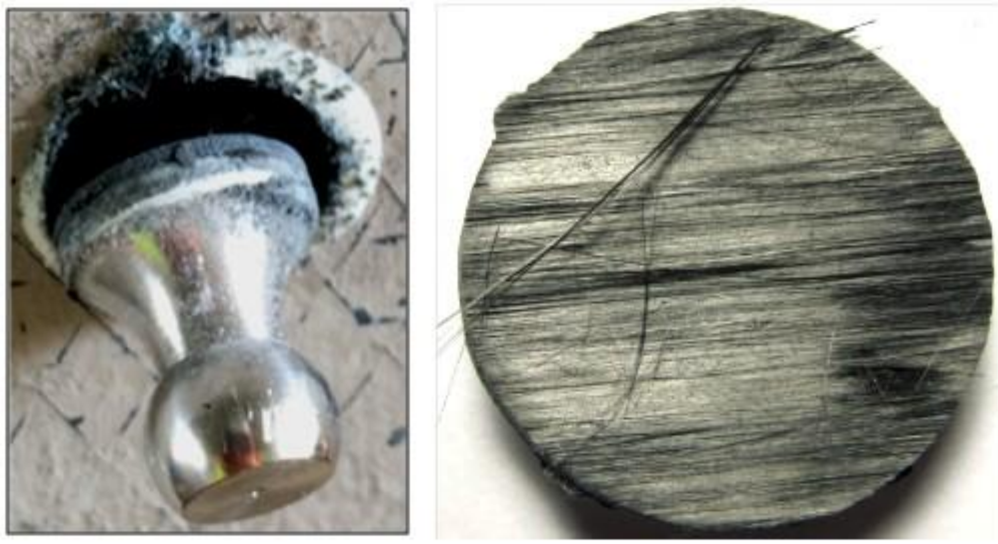

Figure 6.10 –Lack of adhesive bond due to lack of epoxy saturation of sample 623

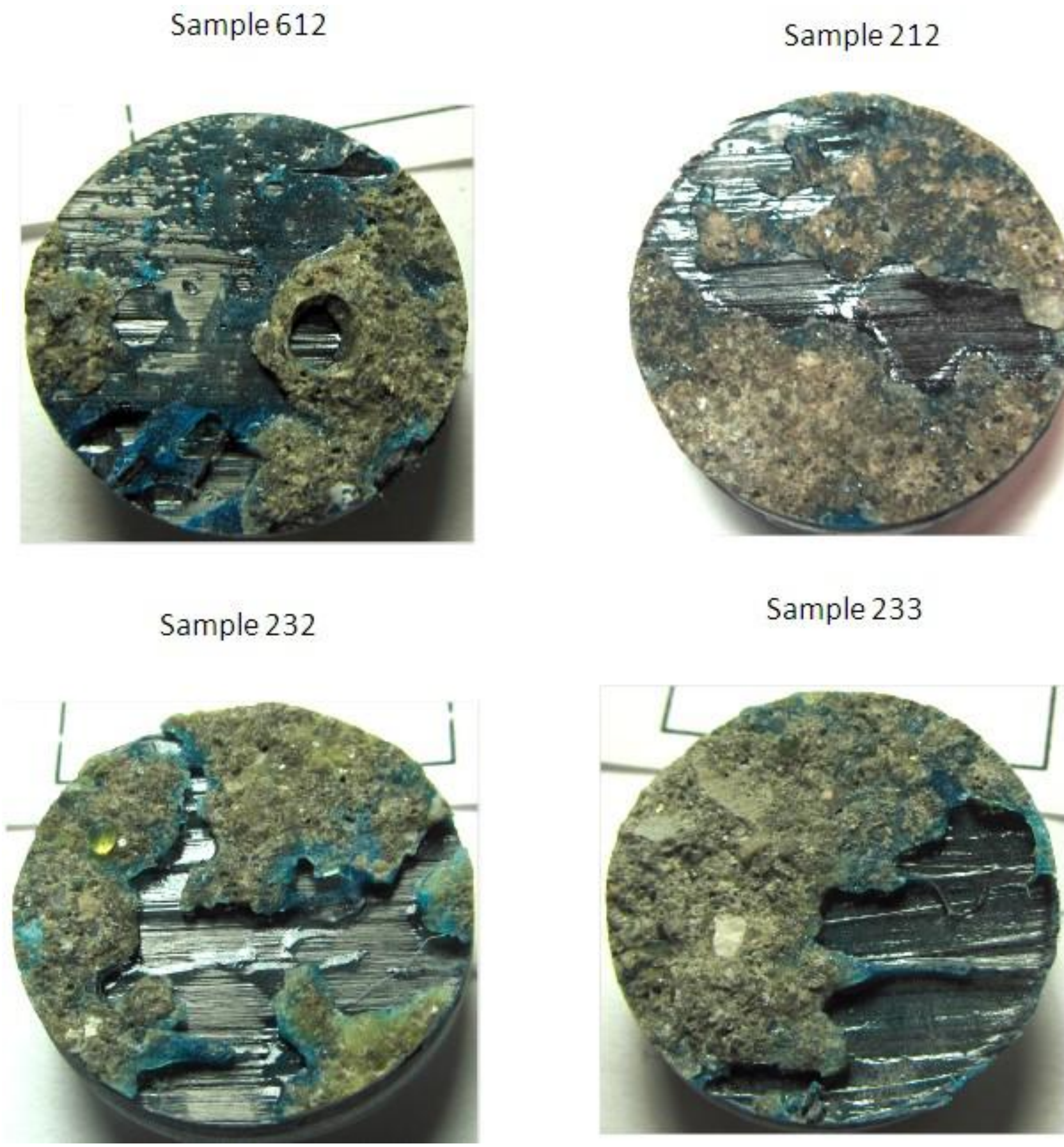

Figure 6.11 –Presence of air bubbles in failure surface of select samples

CHAPTER 7: COMPARISON DESIGN

Although the comparison of design provisions in Chapter 3 concerned hypothetical members, in this Chapter, an existing ACI-based column strengthening design provided by MDOT is compared to an alternative design using AASHTO provisions. Based on the design specifications provided, the column and FRP are assumed to have the properties given in Section 7.1.

7.1. Design Assumptions

7.1.1 Column

Original column section dimensions: b = 42'', h = 84''; gross area A_g = 3528 in^2

As required by ACI, column edges are assumed to be rounded to a minimum radius of r_c = 0.5 in

Total area of steel reinforcement: A_{st} = 34.36 in^2, with yield stress f_y = 60 ksi

Concrete compressive strength = 5 ksi

7.1.2 FRP reinforcement

Thickness, t_f = 0.04 in

Failure strength, f_{fu} = 340 ksi

Modulus of elasticity, E_f = 23000 ksi

Failure strain, ε_{fu} = 0.0147 in/in

Number of layers n = 5

7.1.3 Additional assumptions

ACI 440.2R (ACI Section 12.1.2) states that the provisions provided are not recommended for columns with a side aspect ratio of h/b greater than 2.0, or face dimension h or b exceeding 36 in, unless testing demonstrates the effectiveness of the provisions for these cases. As the column dimensions exceed these limits, it is assumed that the ACI provisions were verified to be applicable to the column.

As the environmental reduction factor was not explicitly used in the original ACI design, it is assumed that this factor was accounted for in the provided FRP reinforcement data, which was reported to have been obtained from laboratory testing.

The original design uses an efficiency factor κ_ε=0.58. However, ACI specifies this factor to be κ_ε=0.55 (ACI Section 2.1 and in ACI design examples). Therefore, it is assumed that κ_ε=0.55 for this analysis.

7.2 Calculations

7.2.1 ACI design, rectangular column

See Section 3.4.3.2 of this report for an outline of the design equations used below.

Axial capacity of the unstrengthened column:

P_n= 0.80 [0.85 f 'c (A_g- A_{st}) + f_y A_{st}] = 0.80 [0.85 × 5 (3528 – 34.36) + 60 × 34.36] = 13528 kips

Axial capacity of the strengthened column:

$$D = (b^2 + h^2)^{0.5} = (42^2 + 84^2)^{0.5} = 93.91\ in^2$$

$$\rho_g = \frac{A_{st}}{A_g} = \frac{34.36}{3528} = 0.0097$$

$$\frac{A_e}{A_c} = \frac{1 - \frac{\left[\left(\frac{b}{h}\right)(h-2r_c)^2 + \left(\frac{h}{b}\right)(b-2r_c)^2\right]}{3A_g} - \rho_g}{1-\rho_g} = \frac{1 - \frac{\left[\left(\frac{42}{84}\right)(84-2\times 0.5)^2 + \left(\frac{84}{42}\right)(42-2\times 0.5)^2\right]}{3\times 3528} - 0.0097}{1-0.0097} = 0.351$$

$$\kappa_a = \frac{A_e}{A_c}\left(\frac{b}{h}\right)^2 = 0.351\left(\frac{42}{84}\right)^2 = 0.088$$

$$\varepsilon_{fe} = \kappa_\varepsilon \varepsilon_{fu} = 0.55 \times 0.0147 = 0.0081$$

$$f_l = \frac{2\,nE_f\,t_f\varepsilon_{fe}}{\sqrt{b^2+h^2}} = \frac{2\times 5\times 23000\times 0.04\times 0.0081}{\sqrt{42^2+84^2}} = 0.793 \text{ ksi}$$

$$\frac{f_l}{f'_c} = \frac{0.793}{5} = 0.159 \geq 0.08 \text{ O.K.}$$

$$\varepsilon_{ccu} = \varepsilon'_c\left(1.5 + 12\ \kappa_b \frac{f_l}{f'_c}\left(\frac{\varepsilon_{fe}}{\varepsilon'_c}\right)^{0.45}\right.$$

where:

$$\varepsilon'_c = \frac{1.71\, f'_c}{E_c} = \frac{1.71\times 5}{4031} = 0.0021$$

$$\kappa_b = \frac{A_e}{A_c}\left(\frac{h}{b}\right)^{0.5} = 0.351\left(\frac{84}{42}\right)^{0.5} = 0.496$$

$$\varepsilon_{ccu} = 0.0021\left(1.5 + 12 \times 0.496 \times 0.159\left(\frac{0.0081}{0.0021}\right)^{0.45}\right) = 0.0068 < 0.01 \text{ O.K.}$$

$f'_{cc} = f'_c + (0.95 \times\ 3.3 \times\ \kappa_a \times f_l) = 5 + (0.95 \times 3.3 \times\ 0.088 \times\ 0.793)$ = 5.218 ksi

$P_{nc} = 0.8[0.85\ f'_{cc}\ (A_g - A_{st}) + f_y\ A_{st}]\ = 0.8[0.85 \times 5.218(3528 - 34.36) + 60 \times 34.36] = 14045$ kips

This produces a change in capacity of: $\Delta P_n = \frac{Pnc - Pn}{Pn} = \frac{14045 - 13528}{13528} = 3.8\ \%$

7.2.2 ACI design, modified column

As an option presented in the original design, the FRP can be applied around a near-circular steel shell (in plan) which is structurally linked to the column. This allows use of the much more efficient shape factors κ_a and κ_b =1.0 for circular sections rather than the greatly reduced factors required for rectangular sections. This option is presented below.

Axial capacity of the strengthened column:

D = $(b^2 + h^2)^{0.5}\ = (42^2 + 84^2)^{0.5}$ = 93.91 in^2

$\rho_g\ = \frac{A_{st}}{A_g} = \frac{34.36}{3528}$= 0.0097

$\frac{A_e}{A_c} = 1$

$\kappa_a\ = 1$

$\kappa_b = 1$

κ_ε = 0.55

$\varepsilon_{fe} = \kappa_\varepsilon \varepsilon_{fu} = 0.55\ \times 0.0147 = 0.0081$

$f_l = \frac{2\ nE_f\ t_f \varepsilon_{fe}}{\sqrt{b^2 + h^2}} = \frac{2 \times 5 \times 23000 \times 0.04 \times 0.0081}{\sqrt{42^2 + 84^2}}$ = 0.793 ksi

$\frac{f_l}{f'_c} = \frac{0.793}{5} = 0.159 \geq 0.08$ O.K.

$f'_{cc} = f'_c + (0.95 \times\ 3.3 \times\ \kappa_a \times f_l) = 5 + (0.95 \times 3.3 \times\ 1 \times\ 0.793)$ = 7.486 ksi

$\varepsilon_{ccu}\ = \varepsilon'_c(1.5 + 12\ \kappa_b \frac{f_l}{f'_c}\ (\frac{\varepsilon_{fe}}{\varepsilon'_c})^{0.45}$

where:

$$\varepsilon'_c = \frac{1.71\, f'_c}{E_c} = \frac{1.71 \times 5}{4031} = 0.0021$$

$$\varepsilon_{ccu} = 0.0021\,(1.5 + 12 \times 1 \times 0.159\,(\frac{0.0081}{0.0021})^{0.45}) = 0.0105 < 0.01 \text{ N.G.}$$

$$E_2 = \frac{f'_{cc} - f'_c}{\varepsilon_{ccu}} = \frac{7.486 - 5}{0.0105} = 236.76 \text{ ksi}$$

$$f'_{cc} = f'_c + (E_2 \times \varepsilon_{ccu}) = 5 + (236.76 \times 0.01) = 7.37 \text{ ksi}$$

$P_{nc} = 0.8[0.85\, f'_{cc}\,(A_g - A_{st}) + f_y\, A_{st}]$ = 0.8[0.85×7.37 (3528 – 34.36) + 60 ×34.36] = 19158 kips

This produces a change in capacity of: $\Delta P_n = \frac{Pnc - Pn}{Pn} = \frac{19158 - 13528}{13528} = 42\ \%$

7.2.3 AASHTO design, rectangular column

See Section 3.4.3.1 of this report for an outline of the design equations used below.

7.2.3.1 Column with confinement pressure limitation imposed

Axial capacity of the unstrengthened column:

$P_n = 0.80\,[0.85\, f'c\,(A_g - A_{st} - A_{ps}) + f_y\, A_{st} - A_{ps}\,(f_{pe} - E_p\, \varepsilon_{cu})]$

$= 0.80\,[0.85 \times 5 \times (3528 - 34.36 - 0) + 60 \times 34.36 - 0] = 13528$ kips

FRP reinforcement strength at a strain of 0.004:

$$N_{frp} = \frac{0.004 \times 13.6}{0.0147} = 3.70 \text{ kip/in}$$

$$f_l = \phi_{frp}\, \frac{2\, Nfrp}{D} = 0.65\, \frac{2\,(5)(3.70)}{42} = 0.57 \text{ ksi}$$

However, per AASHTO Article 5.3.2.2, the confinement pressure must be no less than 600 psi. Therefore, revise design to ensure $f_l = 0.6$ ksi.

Set $f_l = 0.6$ ksi and check adequacy:

$$f_l = 0.60 \text{ ksi} \le (\frac{f'c}{2})(\frac{1}{k_{e.}\,\phi} - 1) = (\frac{4}{2})(\frac{1}{0.8 \times 0.75} - 1)) = 1.33 \text{ ksi} \quad \text{O.K.}$$

$$N_{frp} = \frac{f_l\, D}{2\, \phi_{frp}} = \frac{0.6 \times 42}{2 \times 0.65} = 19.38 \text{ kip/ in}$$

Required number of plies for f_l = 0.6 ksi : $n = \frac{N_{frp}}{N_{frpo}} = \frac{19.38}{3.70} = 5.24$

Therefore, a minimum of 6 layers is required to meet the minimum confinement pressure.

Axial capacity of the strengthened column (with 6 layers):

$$f_l = \phi_{frp} \frac{2\,\mathrm{Nfrp}}{D} = 0.65 \frac{2\,(6)(3.70)}{42} = 0.69 \text{ ksi}$$

$$f'_{cc} = f'c\,(1 + 2\,\frac{f\,l}{f'c}) = 5\,(1 + 2\,\frac{0.69}{5}) = 6.38 \text{ ksi}$$

$$P_n = 0.8\,[0.85\, f'_{cc}\,(A_g - A_{st}) + f_y A_{st}] = 0.8\,[0.85 \times 6.38\,(3528 - 34.36) + 60 \times 34.36] = 16806 \text{ ksi.}$$

This produces a change in capacity of: $\Delta P_n = \frac{Pnc - Pn}{Pn} = \frac{16806 - 13528}{13528} = 24\%$

7.2.3.2 Column with confinement pressure limitation eliminated

For comparison, the confinement pressure limitation is removed, and 5 layers are used as with the ACI design.

Axial capacity of the unstrengthened column:

$$P_n = 0.80\,[0.85\, f'c\,(A_g - A_{st} - A_{ps}) + f_y A_{st} - A_{ps}\,(f_{pe} - E_p\,\varepsilon_{cu})]$$

$$= 0.80\,[0.85 \times 5 \times (3528 - 34.36 - 0) + 60 \times 34.36 - 0] = 13528 \text{ kips}$$

FRP reinforcement strength at a strain of 0.004.

$$N_{frp} = \frac{0.004 \times 13.6}{0.0147} = 3.70 \text{ kip/in}$$

$$f_l = \phi_{frp} \frac{2\,\mathrm{Nfrp}}{D} = 0.65 \frac{2\,(5)(3.70)}{42} = 0.57 \text{ ksi}$$

$$f\,l = 0.57 \text{ ksi} \le (\frac{f'c}{2})(\frac{1}{k_{e.}\,\phi} - 1) = (\frac{4}{2})(\frac{1}{0.8 \times 0.75} - 1)) = 1.33 \text{ ksi} \quad \text{O.K.}$$

$$N_{frp} = \frac{f_l\, D}{2\,\phi_{frp}} = \frac{0.57 \times 42}{2 \times 0.65} = 18.4 \text{ kip/ in}$$

Required number of plies for f_l = 0.57 ksi : $n = \frac{N_{frp}}{N_{frpo}} = \frac{18.4}{3.70} = 5$

Axial capacity of the strengthened column (with 5 layers):

$f_l = \phi_{frp} \frac{2\,N_{frp}}{D} = 0.65 \frac{2(5)(3.70)}{42} = 0.57$ ksi

$f'_{cc} = f'c\,(1 + 2\,\frac{f\,l}{f'c}) = 5\,(\,1 + 2\frac{0.57}{5}\,) = 6.14$ ksi

$P_n = 0.8\,[0.85\,f'_{cc}\,(A_g - A_{st}) + f_y\,A_{st}] = 0.8\,[0.85 \times 6.14\,(3528 - 34.36) + 60 \times 34.36] = 16236$ ksi.

This produces a change in capacity of: $\Delta P_n = \frac{P_{nc} - P_n}{P_n} = \frac{16236 - 13528}{13528} = 20\%$

7.2.4 AASHTO design, modified column

7.2.4.1 Modified column with confinement pressure limitation imposed

Axial capacity of the unstrengthened column:

$P_n = 0.80\,[0.85\,f\,'c\,(A_g - A_{st} - A_{ps}) + f_y\,A_{st} - A_{ps}\,(\,f_{pe} – E_p\,\varepsilon_{cu})]$

$= 0.80\,[0.85 \times 5 \times (3528 - 34.36 - 0) + 60 \times 34.36 - 0] = 13528$ kips

FRP reinforcement strength at a strain of 0.004:

$N_{frp} = \frac{0.004 \times 13.6}{0.0147} = 3.70$ kip/in

$f_l = \phi_{frp} \frac{2\,N_{frp}}{D} = 0.65 \frac{2\,(5)(3.70)}{93.91} = 0.256$

However, per AASHTO Article 5.3.2.2, the confinement pressure must be no less than 600 psi. Therefore, revise design to ensure $f_l = 0.6$ ksi.

Set $f_l = 0.6$ ksi and check adequacy:

$f\,l = 0.60 \text{ ksi} \leq (\frac{f\,'c}{2})(\frac{1}{k_{e,}\,\phi} - 1) = (\frac{4}{2})(\frac{1}{0.8 \times 0.75} - 1)) = 1.33$ ksi O.K.

$N_{frp} = \frac{f_l\,D}{2\,\phi_{frp}} = \frac{0.6 \times 93.91}{2 \times 0.65} = 43.35$ kip/ in

Required number of plies for $f_l = 0.6$ ksi: $n = \frac{N_{frp}}{N_{frpo}} = \frac{43.34}{3.70} = 11.71$

Therefore, a minimum of 12 layers is required to meet the minimum confinement pressure.

Axial capacity of the strengthened column (with 12 layers):

$f_l = \phi_{frp} \frac{2\,Nfrp}{D} = 0.65 \frac{2\,(12)(3.70)}{93.91} = 0.61$ ksi

$f'_{cc} = f'c\,(1 + 2\,\frac{f\,l}{f'c}) = 5\,(1 + 2\,\frac{0.61}{5}) = 6.22$ ksi

$P_n = 0.8\,[0.85\,f'_{cc}\,(A_g - A_{st}) + f_y\,A_{st}] = 0.8\,[0.85 \times 6.22\,(3528 - 34.36) + 60 \times 34.36] = 16426$ ksi.

This produces a change in capacity of: $\Delta P_n = \frac{Pnc - Pn}{Pn} = \frac{16426 - 13528}{13528} = 21\,\%$

7.2.4.2 Modified column with confinement pressure limitation eliminated

For comparison, the confinement pressure limitation is removed, and 5 layers are used as with the ACI design.

Axial capacity of the unstrengthened column:

$P_n = 0.80\,[0.85\,f'c\,(A_g - A_{st} - A_{ps}) + f_y\,A_{st} - A_{ps}\,(f_{pe} - E_p\,\varepsilon_{cu})]$

$= 0.80\,[0.85 \times 5 \times (3528 - 34.36 - 0) + 60 \times 34.36 - 0] = 13528$ kips

FRP reinforcement strength at a strain of 0.004:

$N_{frp} = \frac{0.004 \times 13.6}{0.0147} = 3.70$ kip/in

$f_l = \phi_{frp} \frac{2\,Nfrp}{D} = 0.65 \frac{2\,(5)(3.70)}{93.91} = 0.256$

$f\,l = 0.256 \text{ ksi} \leq (\frac{f'c}{2})(\frac{1}{k_{e.}\,\phi} - 1) = (\frac{4}{2})(\frac{1}{0.8 \times 0.75} - 1)) = 1.33$ ksi O.K.

$N_{frp} = \frac{f_l\,D}{2\,\phi_{frp}} = \frac{0.256\,X\,93.91}{2\,X\,0.65} = 18.5$ kip/ in

Required number of plies for $f_l = 0.256$ ksi: $n = \frac{N_{frp}}{N_{frpo}} = \frac{18.5}{3.70} = 5$

Axial capacity of the strengthened column (with 5 layers):

$f_l = \phi_{frp} \frac{2\,Nfrp}{D} = 0.65 \frac{2(5)(3.70)}{93.91} = 0.256$ ksi

$f'_{cc} = f'c\,(1 + 2\,\frac{f\,l}{f'c}) = 5\,(1 + 2\,\frac{0.256}{5}) = 5.51$ ksi

$P_n = 0.8\,[0.85\,f'_{cc}\,(A_g - A_{st}) + f_y\,A_{st}] = 0.8\,[0.85 \times 5.51\,(3528 - 34.36) + 60 \times 34.36] = 14740$ ksi.

This produces a change in capacity of: $\Delta P_n = \frac{Pnc - Pn}{Pn} = \frac{14740-13528}{13528} = 9\%$

7.3 Summary

As was shown in Chapter 3 (Section 3.4.4), for the same number of layers, ACI predicts larger capacity increases for strengthening circular columns, while AASHO predicts larger capacity increases for strengthening square columns.

Using 5 layers of FRP, the ACI design produced a capacity increase of 4% for the rectangular column and 42% for the modified (circular) column. Similarly, in the AASHTO procedure, if the minimum confinement strain requirement of 0.6 ksi is not enforced, 5 layers produces a 9% increase for the rectangular column and a 20% increase for the modified column. Imposing the confinement strain provision in AASHTO results in a requirement for 6 layers but provides a 24% increase in capacity for the rectangular column. For the modified column, however, imposing the confinement strain provision requires 12 layers and only provides a 21% increase in capacity.

CHAPTER 8: RECOMMENDATIONS

8.1 Analysis and Design Recommendations

8.1.1 Criteria for recommendations

Most of the codes reviewed have reasonable coverage of the major design and analysis areas (flexure, shear, confinement). To guide selection of appropriate provisions, the following primary criteria were considered:

1) Accuracy. It is essential that a recommended provision accounts for the phenomenon that it attempts to address with sufficient accuracy. In general, this can be established with adequate documentation in the code commentary, to demonstrate theoretical soundness and/or convincing experimental evidence for empirically-based provisions. Documentation may include references to appropriate technical papers, reports, or other published sources, most of which are included in the literature review section of this report. As shown in earlier sections of this document, the reviewed codes contain varying levels of documentation for their recommendations.

2) Format compatibility. From a practical point of view, it is important that recommended provisions are compatible with existing bridge design and analysis procedures used by MDOT. This is particularly important when considering strength reduction factors that modify expressions for capacity. Such factors may have been developed for building loads, for example, which have different levels of uncertainty than traffic loads. Similarly, manufacturing and environmental factors may reflect conditions that are not representative of those in Michigan.

3) Clarity and ease of use. Often, provisions produce similar outcomes, but achieve the outcome with different procedures, equations, and processes. Clearly, a balance must be drawn between accuracy and ease of use. As this report concerns recommendations for design and evaluation, a large increase in complexity that provides a minimal gain in accuracy may not be appropriate. Clarity and ease of use can be evaluated is in terms of the procedures and models used, and even the symbols and nomenclature used in the relevant expressions. Clarity also concerns similarity to the existing methods that designers are familiar with.

8.1.2 General recommendation and discussion

Based on the criteria considered above, it is recommended that the AASHTO guidelines (AASHTO *Guide Specification for the Design of Bonded FRP Systems for Repair and Strengthening of Concrete Bridge Elements*) are used as the base analysis and design document, with some recommended modifications, as discussed below. The AASHTO Guide covers most areas of concern with good documentation of accuracy, and results calculated from the AASHTO provisions generally fall with the range of outcomes of the other codes considered. Moreover, the document is specifically written for bridge structures, and is directly compatible with the AASHTO LRFD design provisions. The format, notation, and procedures within are also familiar to MDOT engineers. A brief summary of the reasoning for recommending the AASHTO provisions is discussed below. Refer to Chapter 3 of this report for additional detail.

Strengthening limits for fire endurance. ACI 440.2R provides strengthening limitations to allow a structure to maintain a minimum specified strength if the external FRP strengthening system is weakened in a fire. The use of this expression, however, requires evaluation of existing structural capacity at high temperatures, which introduces complexity that may not be desirable. Although AASHTO does not directly address this issue, AASHTO does provide a general strengthening limit expression when FRP is considered. As discussed in Section 3.2.2, this existing provision provides a strengthening limit outcome similar to that presented by ACI. Therefore, the existing AASHTO expression is recommended without modification.

FRP strain limits. To prevent debonding, AASHTO specifies that the FRP strain may not exceed 0.005 less initial strain (for flexure). AASHTO adds a second strain condition to ensure ductile behavior post steel yielding. AASHTO stipulates that the ultimate FRP strain equals 2.5 times FRP strain at the point of steel yielding. These strain limit requirements are slightly more conservative than some other codes (see Section 3.2.4 of this report and Figures 3.2.4 and 3.2.5), but are deemed reasonable for reinforced concrete structures. However, as discussed below, these may not be suitable for prestressed concrete girders.

Strength reduction factors for FRP. The majority of codes use a FRP strength reduction factor close to 0.85 as indicated in Table 3.2.10 and discussed in Section 3.2.5. Some codes such as TR55 adopt a more detailed procedure to account for different FRP materials, manufacturing processes, and exposure conditions to produce a combined factor for specific cases. This approach offers flexibility, but unfortunately, most of the values recommended are undocumented or rely on manufacturer test data that may not follow common standards or regulations. The common global value adopted by AASHTO of 0.85 is based the work by Okeil et al. (2007) that is based on reliability analysis and accounting for the brittle nature of CFRP. Due to the lack of information to justify a more detailed system, and as the AASHTO-specified value is similar to that adopted by other codes, it is recommended for use.

Serviceability, service load limits, creep rupture and fatigue limits. The AASHTO serviceability limits are $0.80\, f_y$ for steel; 0.36 f'_c for concrete; and $0.80\, f_{fu}$ for CFRP. These values are similar to those adopted by ACI, except for the FRP limit since AASHTO has no considerations for an environmental reduction factor which is built in the ACI factor of $0.55 f_{fu}$. Both ACI and AASHTO reference the work of Yamaguchi et al. (1997) and Malvar (1998) as a basis for fatigue limits. No other codes offer a complete set of serviceability limits. AASHTO fatigue limits are comprehensive as well, with strain limits given for concrete, steel, and FRP, and are recommended.

FRP end peeling. End peeling limits vary from a constant value specified by TR55, an expression by ACI relating the shear force to concrete shear resistance (as a function of section geometry and concrete compressive strength), and expressions by AASHTO that account for FRP thickness, adhesive properties and concrete compressive strength. As shown in Figures 3.2.15 and 3.2.16, the AASHTO peeling limits compare closely to those of CNR. The work of Naaman et al. (1999) is the basis for the limits adopted by AASHTO. This work specifically considered environmental conditions similar to those in Michigan, as discussed in Section 3.2.7, and is considered to be most appropriate for MDOT bridges.

Development length. The expression for development length presented by AASHTO is somewhat different from that of the other codes; it is substantially more sensitive both to f'_c as well as the amount of FRP provided, as shown in Section 3.2.9. However, the AASHTO expression was developed by Naaman et al. (1999) and specifically considered Michigan environmental conditions. It is therefore recommended for use on this basis.

Shear and confinement strengthening. Variations in shear and confinement strengthening provisions are minimal among the different codes. Based on this similarity, there is no theoretically compelling reason to deviate from the AASHTO provisions. However, due to the AASHTO requirement that a minimum confinement pressure of 0.6 ksi is met (see Table 3.4.3), AASHTO may often require a larger amount of FRP than theoretically necessary to increase capacity. Therefore, two alternatives are recommended for consideration of confinement strengthening: the AASHTO provisions as well as the ACI provisions. The latter is recommended as it may be more economically feasible than the AASHTO approach for some columns. In particular, as ACI does not specify a minimum required confinement pressure, it may achieve the same strengthening result as AASHTO but with less FRP material.

8.1.3 Recommended modifications to AASHTO provisions

Although the AASHTO provisions are generally recommended, some shortcomings in the AASHTO guidelines have been identified. To address these shortcomings, the following modifications are suggested:

8.1.3.1 Environmental reduction factors

AASHTO provides no specific environmental reduction factors. Rather, a minimum assumed interface shear strength is provided (τ_{int}= 0.065 $\sqrt{f_c'}$), which is also used to evaluate peeling (eq. 3.2.37) as well as development length (eq. 3.2.50). Therefore, the reduction factor below should be applied to the interface shear strength of the system, τ_{int}, the value of which is obtained from the manufacturer or from appropriate experimental testing as noted in section 8.2.4.2. Based on the experimental phase of this project (Chapter 5), the following Michigan-specific reduction factors for CFRP interface bond strength are recommended (linear interpolation is permitted):

Time (design years)	Reduction Factor
10	0.73
25	0.66
50	0.54
75	0.42

These values are to be multiplied by the τ_{int} determined for the specific FRP system (not the value of 0.065 $\sqrt{f_c'}$ provided by AASHTO, which is likely much lower than the actual τ_{int}). It should be emphasized that the reduction factor is applied to τ_{int} only and not directly to the section capacity in the form of an overall strength reduction factor.

As detailed in Chapter 5, a 15-year reduction factor was determined experimentally from measured reductions in bond strength of CFRP wrap exposed to Michigan weather, while shorter and longer term reduction factors were determined by extrapolation with best-fit linear regression models.

Multiple-year reduction factors are presented since it may make sense to apply a reduction factor corresponding to the anticipated remaining service life of the component to which the FRP is applied. However, if uncertain, this time should be conservatively estimated.

Although long-term reduction factors are large, if common epoxy strengths and appropriate surface preparation and installation techniques are used, as in accordance with the recommendations of this chapter, it is likely that no environmental reductions in capacity will occur in most cases. This is because the interface shear strength assumed by AASTHO is very low (τ_{int}= 0.065 $\sqrt{f_c'}$), and most FRP installations are expected to have initial τ_{int} values greater than twice this amount. Thus, even applying a large, long-term reduction factor for 50-75 years would result in a design shear strength greater than the minimum specified by AASHTO, resulting in a potential increase in peeling strength (eq. 3.2.37) and decrease in allowable development length (eq. 3.2.50).

8.1.3.2 Flexural design when considering compression failures

Although tension failures are clearly desired, in many cases it is not possible to meaningfully strengthen with FRP and maintain a tension-controlled section. This is particularly so for prestressed concrete sections, but is not directly addressed in AASHTO. In the case of a compression controlled section, it is recommended that in the capacity analysis, the AASTHO-specified FRP strain limit (at ultimate capacity) should correspond to a maximum concrete compressive strain in the girder of 0.003. If the resulting FRP strain is less than 0.005, the FRP reinforcement strength per unit width, N_b, should be calculated based on the FRP strain calculated at concrete compressive failure, not 0.005.

8.1.3.3 Initial strain for prestressed sections

For the strengthening analysis of prestressed concrete sections, the initial strain ε_{bo} at the bottom of the beam (FRP/concrete interface) must be calculated. Although not addressed in AASHTO, this expression can be theoretically derived and is given in ACI, which is recommended for use (ACI Eq. 10-18):

$$\varepsilon_{bo} = \frac{-p_e}{E_c A_{cg}}\left(1 + \frac{e\, y_b}{r^2}\right) + \frac{M_{DL}\, y_b}{E_c I_g} \qquad (8.1)$$

where:

ε_{pi} = initial strain level in prestressed steel reinforcement, in/in (mm/mm)
P_e = effective force in prestressing reinforcement (after allowance for all prestress losses), lb (N)
A_p = area of prestressed reinforcement in tension zone, in^2 (mm^2)
E_p = modulus of elasticity of prestressing steel, psi (MPa)

e = eccentricity of prestressing steel with respect to centroidal axis of member at support, in (mm)
r = radius of gyration of a section, in (mm)

8.1.3.4 Strength reduction factors and ductility provisions considering prestressed sections

To ensure ductility, AASHTO limits specify that the strain developed in the FRP reinforcement at section ultimate capacity must be equal to or greater than 2.5 times the strain in the FRP reinforcement at the point where the centroid of steel tension reinforcement yields. With a maximum useable strain of 0.005 at the FRP reinforcement/concrete interface, the maximum effective strain developed in the FRP reinforcement is $\varepsilon = 0.005 - \varepsilon_{bo}$, where ε_{bo} is the initial tensile strain at the bottom concrete surface as a result of the moment due to dead load (the existing tensile strain prior to FRP installation). When the FRP is applied, the steel will have some initial strain due to dead load moment (ε_d). Since the FRP is bonded to the outside of the beam, it is placed further away from the neutral axis in the section than the steel. Therefore, the FRP will have a minimum strain value of ε_y-ε_d when the steel yields, where ε_y is steel yield strain. Following the AASHTO provisions, at ultimate flexural capacity, strain in the FRP must be greater than the minimum possible value of $2.5(\varepsilon_y - \varepsilon_d)$. For prestressed beams, the steel effective prestrain (ε_p) must also be considered, such that the strain in the FRP at ultimate flexural capacity must be greater than $2.5(\varepsilon_y - \varepsilon_d - \varepsilon_p)$. However, for the high-grade steel used for prestressing, ε_y is large, and the value of $2.5(\varepsilon_y - \varepsilon_d - \varepsilon_p)$ may exceed the allowable FRP strain limit of 0.005 - ε_{bo}. This limitation will not allow any flexural FRP strengthening to be applied to prestressed beams in many cases. Reasonable changes in the concrete constitutive model does not alter this result. ACI provisions solve this problem by allowing such sections to be strengthened, but to account for the possible loss in ductility, the section strength is penalized with a lower resistance factor. However, for the practical cases of prestressed concrete girders investigated, it was found that the ACI approach often leads to high requirements of FRP reinforcement as well as a resistance factor of 0.65. The former occurs because the ACI debonding limit, which limits the usable strain in the FRP (eq. 3.2.8), is a function of the amount of FRP applied; as the amount of FRP is increased to meet flexural demand, this strain limit decreases. It was found that the resulting value is often lower than the value of 0.005 specified with the AASHTO provisions. Moreover, as FRP area is increased, strain in the prestressing steel at failure generally decreases, thus lowering resistance factor, often to 0.65. Therefore, direct use of the ACI strengthening provisions with prestressed griders was also found to be impractical in many cases. Therefore, an alternative approach is recommended for consideration, based on a modification of the existing AASHTO procedure.

This approach utilizes the AASHTO FRP strain limit addressed in Section 8.1.2. For consistency, the same methodology is recommended for both non-prestressed as well as prestressed sections, with appropriate adjustments in strain limits. This approach combines AASHTO and ACI limits and reduction factor concepts to produce a more practical strengthening procedure for some cases. For non-prestressed sections, the resistance factor for flexural capacity for concrete beams with FRP strengthening is recommended to be taken as:

$$\begin{cases} \phi = 0.90 \quad \text{for} \quad (\varepsilon_{FRPu} \geq 2.5\varepsilon_{FRPy} \text{ AND } \varepsilon_t \geq 0.005) \\ \phi = \min\begin{cases} 0.65 + \left(\dfrac{0.25}{1.5}\right)(\varepsilon_{FRPu} - 1) \ \mathit{for} \ \varepsilon_{FRPy} < \varepsilon_{FRPu} < 2.5\varepsilon_{FRPy} \\ 0.65 + \dfrac{0.25(\varepsilon_t - \varepsilon_{sy})}{0.005 - \varepsilon_{sy}} \ \mathit{for} \ \varepsilon_{sy} < \varepsilon_t < 0.005 \end{cases} \\ \phi = 0.65 \quad \text{for} \quad \varepsilon_{FRPu} \leq \varepsilon_{FRPy} \text{ OR } \varepsilon_t \leq \varepsilon_{sy} \end{cases} \tag{8.2}$$

where:

ε_t = strain in the lowest layer (i.e. furthest away from the compression side of the beam) of steel at ultimate capacity, where ultimate capacity is limited by either (whichever occurs first) concrete crushing, FRP rupture, or FRP debonding (at an assumed maximum interface strain of 0.005 given by AASHTO)

ε_{FRPu} = strain in the FRP at ultimate capacity

ε_{FRPy} = strain in the FRP when steel yields

The above factor is to be applied to the entire flexural capacity expression, including steel and FRP-based capacity components. In addition, the FRP portion of capacity is further reduced by the additional resistance factor given as:

$$\begin{cases} \phi_{FRP} = 0.85/0.90 = 0.94 \ \text{when} \ \phi = 0.9 \\ \phi_{FRP} = 0.38\phi + 0.6 \quad \text{when} \ 0.65 < \phi < 0.9 \\ \phi_{FRP} = 0.85 \quad \text{when} \quad \phi = 0.65 \end{cases} \tag{8.3}$$

Note that practically, due to the usable FRP strain limit specified by AASHTO of 0.005, $\phi = 0.90$ cannot be achieved due to the requirement of $\varepsilon_t \geq 0.005$. However, in most cases, if maximum FRP strain is set to 0.005, ε_t will only be slightly less than this value, resulting in ϕ between 0.85-0.90.

For prestressed concrete members (using 250-270 ksi prestress steel), the following is recommended:

$$\begin{cases} \phi = 0.90 \quad \text{for} \quad (\varepsilon_{FRPu} \geq 2.5\varepsilon_{FRPy} \text{ AND } \varepsilon_{ps} \geq 0.013) \\ \phi = \min\begin{cases} 0.65 + \left(\dfrac{0.25}{1.5}\right)(\varepsilon_{FRPu} - 1) \ \mathit{for} \ \varepsilon_{FRPy} < \varepsilon_{FRPu} < 2.5\varepsilon_{FRPy} \\ 0.65 + \dfrac{0.25(\varepsilon_{ps} - 0.010)}{0.013 - 0.010} \ \mathit{for} \ 0.010 < \varepsilon_{ps} < 0.013 \end{cases} \\ \phi = 0.65 \quad \text{for} \quad \varepsilon_{FRPu} \leq \varepsilon_{FRPy} \text{ OR } \varepsilon_{ps} \leq 0.010 \end{cases} \tag{8.4}$$

where:

ε_{ps} = The prestressing steel strain (at steel centroid) at ultimate capacity. The same FRP reduction factor as above is applied to the FRP portion of resistance.

$$\varepsilon_{ps} = \varepsilon_{pe} + \frac{p_e}{A_c E_c}\left(1 + \frac{e^2}{r^2}\right) + \varepsilon_{pnet} \leq 0.035 \quad (3.5.5)$$

$$\varepsilon_{pe} = \frac{f_{pe}}{E_p} \quad (3.5.6)$$

$$\varepsilon_{pnet} = \frac{d_p - c}{h - c}\,\varepsilon_{frp}^{u} \quad (3.5.7)$$

Note that practically, due to the usable FRP strain limit specified by AASHTO of 0.005, $\varepsilon_{ps} \leq 0.010$ and thus $\phi = 0.65$. However, the above equations retain validity if the AASHTO FRP strain limit of 0.005 is adjusted to a new value. Increasing this value will allow an increase in the strength reduction factor for prestressed members beyond 0.65.

In most cases, this approach will provide a resistance factor that is very close (slightly conservative) to that specified by AASHTO if AASHTO ductility criteria are met. If AASHTO criteria are not met, beam strengthening is allowed but with a reduced resistance factor similar to the ACI approach as a function of beam ductility.

Note that, to determine the most appropriate resistance factors and associated strain limits, a structural reliability analysis of FRP-strengthened prestressed concrete girders, subjected to Michigan bridge loads, is needed. However, this analysis is beyond the scope of this report. Therefore, with the recognition that the existing provisions were not specifically developed for strengthening prestressed concrete bridge girders, the recommended procedure may be a reasonable alternative to consider until desired reliability or revised guidelines results become available. Some design examples using the recommendations are given in Appendix B.

8.2 Installation, QC, and Maintenance Recommendations

For all sources examined, there is a nearly universal suggestion or requirement to follow the FRP system manufacturer's recommendations. Therefore, it is recommended that:

1) The installer is required to provide documentation of manufacturer recommendations for installation, QC, and maintenance, as available, and that these recommendations are followed. Deviation from these recommendations are to be made only for compelling reasons, and should have approval of the design engineer (who should have extensive experience with FRP strengthening of bridge structures), and MDOT. The proposed deviation should be accompanied with sufficient documentation and/or justification, preferably with quantitative analysis and/or experimental test data.

2) If the manufacturer does not provide recommendations in a particular instance, the following guidelines are recommended. In all cases, it is suggested the manufacturer is consulted for comment on the provisions before use.

A checklist of inspection items following the recommendations below is given in Appendix C, while suggested changes to the existing MDOT Provisions for FRP strengthening are given in Appendix D.

8.2.1 Shipping, storage, and handling

8.2.1.1 Shipping

As AASHTO has no specific shipping recommendations, in general, the ACI shipping guidelines are recommended. In particular, packaging, labeling, and shipping for thermosetting resin materials are to be controlled by the *Code of Federal Regulations 49* (CFR 49), in which some FRP system materials may be classified as corrosive, flammable, or poisonous in Subchapter C (CFR 49). As such, FRP system constituent materials are to be packaged and shipped in a manner that conforms to all applicable federal and state packaging and shipping codes and regulations. The following additional provisions are suggested:

It is the duty of the contractor and supplier to ensure that the packaging and shipping methods used do not negatively impact material properties and performance.

All FRP components must be shipped with their respective SDSs.

All components of the FRP system are to be inspected upon delivery to the construction site, and the use of opened or damaged containers should only proceed with written authorization by the project Engineer.

8.2.1.2 Storage

Although AASHTO provides no storage guidelines, ACI Article 5.2 offers the most complete coverage of storage and disposal of expired materials (Article 1.2), and these provisions are recommended in general, with the following additional comments.

All FRP system components must be stored in the original factory-sealed, unopened packaging with labels identifying the manufacturer, brand name, system identification number and date.

Proper storage of FRP components is in a clean, dry area, sheltered from direct sunlight, which is well ventilated and temperature controlled within 50°–75°F (10°–24°C). Catalysts and initiators (e.g., peroxides) should be stored separately.

As possible, components stored on the jobsite should be periodically inspected in accordance with the inspection checklist presented in Appendix C; components that have been stored in a condition different from that stated above are to be rejected.

Of particular concern are reactive curing agents, hardeners, initiators, catalysts, and cleaning solvents which have special safety-related requirements, and are to be stored in securely sealed containers in a manner recommended by OSHA. As resins are also flammable, fire precautions should be observed and storage quantities kept within limits prescribed by fire regulations.

The manufacturer is to provide a recommended shelf life within which the properties of the resin-based materials should continue to meet or exceed the stated performance, and the contractor must follow these time limits.

Any component material that has exceeded its shelf life, has deteriorated, or has been damaged or otherwise contaminated should not be used. Care must be taken to avoid laminate and other preformed material damage due to bending or improper stacking. FRP materials deemed unusable should be disposed of in a manner acceptable to state and federal environmental control regulations.

8.2.1.3 Handling

ACI Article 5.3 provides the most complete coverage of safe handling of FRP materials as compared to other codes. ACI is therefore generally recommended for use, with the following additional comments.

Information regarding proper storage, handling, and mixing resin components and potential hazards should be made available at the construction site.

Product hazard labels and associated SDSs are to be read and understood by those working with these products. CFR 16, Part 1500 (2009), regulates the labeling of hazardous substances and includes thermosetting-resin materials. Such labeling guidelines should be followed.

Disposable suits and gloves resistant to resins and solvents should be used for handling fiber and resin materials, where gloves should be discarded after each use. Safety glasses or goggles should be used when handling resin components and solvents, and surfaces should be covered as needed to protect against contamination and resin spills.

Respiratory protection, such as dust masks or respirators, should be used when fiber fly, dust, or organic vapors are present, or during mixing and placing of resins. The workplace in which composite materials are prepared and installed should be well ventilated; in poorly ventilated areas, the use of respiratory protection with a fresh air supply is recommended.

Uncontrolled reactions, including fuming, fire, or violent boiling, may occur in containers holding a mixed mass of resin; therefore, such containers should be monitored.

As cleanup can involve flammable solvents, appropriate safety precautions are suggested. All waste materials are to be disposed of as prescribed by the prevailing environmental authority.

It is the contractor's responsibility to ensure that all components of the FRP system at all stages of work conform to governing environmental and safety regulations.

All FRP components, but especially fiber sheets, must be handled with care to protect them from damage and to avoid misalignment or breakage of the fibers; excessive bending, crushing and other sources of mechanical damage to the fibers must be avoided. Higher modulus fibers are particularly susceptible to such damage. After cutting, sheets should be either stacked dry with separators, or rolled gently at a radius no tighter than 12 in (305 mm). Special care should be taken to avoid material contact with water, dust or other contaminants.

8.2.2 Manufacturer and contractor qualification

For manufacturer and contractor qualification, AASHTO refers to NCHRP 609, which gives relatively comprehensive qualification requirements as compared to other sources. NCHRP 609 is thus recommended as a qualification guide, with the following summary of these recommendations, as well as additional comments.

FRP application must be performed by a contractor specifically trained in accordance with the installation procedures specified by the manufacturer. These procedures may differ from one system to another, and thus general knowledge of FRP installation is of itself not sufficient.

The proposed manufacturer may be pre-qualified for each FRP system to be installed, after providing the following information to a qualified engineer for review and consideration:

- System data sheets and SDS for all components of the FRP system;
- Documentation of a minimum of 5 years' experience with the FRP system, or 25 documented similar field applications with acceptable reference letters from respective owners;
- Documentation of test data sets from an independent agency verifying the mechanical properties, aging and environmental durability of the proposed FRP system, and;
- Documentation of the availability of a comprehensive hands-on training program for each FRP system that can be taken by the staff of the contractor/applicator.

MDOT may further opt to require the manufacturer to provide samples of the components, as well as of the complete FRP system for in-house or independent testing prior to qualification.

The training program conducted by the manufacturer should provide hands-on experience with surface preparation and installation of the specific FRP system which is to be installed.

Additionally, it is recommended that:

MDOT may further opt to require that the contractor demonstrate competency by an actual demonstration of surface preparation and installation.

The contractor must provide a detailed statement of the method which will be used to install the composites as well as an assessment of the risks (failing to meet system performance requirements as well as health and construction hazards) involved. This should include discussion of the procedure which will be used to minimize these risks. The contractor should further provide description of a plan that will be used to maintain an environment on-site that is suitable for the successful use of structural adhesives. Note that many of these items are typical requirements for the contractor-submitted QC plan.

8.2.3 Installation

Most codes reviewed provided similar recommendations. Provisions provided by ACI are generally most complete and are recommended, with some additional provisions from AASHTO and other codes, as detailed below.

8.2.3.1 Temperature, humidity, and moisture considerations

Primers, saturating resins, and adhesives should generally not be applied to cold or frozen surfaces, which can affect bond as well as rate of curing, and the presence of frost or ice crystals may be detrimental to the bond between the FRP and the concrete. In particular, ambient air and concrete surface temperature should be 50° F (10° C) or more; the concrete surface temperature should be at least 5° F (3° C) higher than the actual dew point; and atmospheric relative humidity should be less than 85%. A non-contaminating heat source can be used to raise the ambient and surface temperatures during installation if needed.

During application of epoxy, work should be scheduled to avoid air and surface temperatures exceeding 90° F (32° C). If necessary, then the work should be supervised by a person experienced in applying epoxy under such conditions. Epoxy systems formulated for elevated temperatures are available, and should be considered as well (see ACI 530R-93). It is also recommended not to install FRP when the concrete surface is heavily exposed to sunlight.

Adhesives should not be applied to damp or wet surfaces with surface humidity greater than 10%, unless they have been formulated for such applications. Substrate humidity can be evaluated with a mortar hygrometer or other quantitative means. Moreover, FRP systems should not be applied to concrete surfaces that are subject to condensation, moisture vapor transmission, or water ingression, unless such issues are clearly addressed by the system design and the resin systems are specifically formulated for use in such conditions. ACI Standard 503.4 (2003) provides additional moisture content requirements. Note that the transmission of moisture vapor from a concrete surface through the uncured resin materials typically appears as surface bubbles and can compromise the bond between the FRP system and the substrate. Condensation or other moisture on the resin surface before initial hardening, indicated by whitening, the area should be wiped with solvent or the effected portion of primer or smoothing agent removed with sandpaper.

8.2.3.2 Equipment

All equipment should be clean, in good operating condition, and accessible for inspection by the project engineer. Note that some FRP systems recommend or require specific equipment for application, such as resin impregnators, sprayers, lifting/positioning devices, and winding machines. The contractor is to have qualified personnel sufficiently trained to install and operate such system-specific equipment, such as by training and/or certification from the FRP system manufacturer, if available. All materials, and supplies, and personal protective equipment should be available in sufficient quantities to allow safe construction continuity and quality assurance.

8.2.3.3 Substrate repair

The surface on which the FRP system is to be applied should be free of loose and unsound materials; unfit existing conditions may require repair. The quality of the substrate can be checked with testing. Compressive strength should not be less than 2.2 ksi (15 N/mm^2), and the concrete surface must have a minimum tensile strength of 220 psi (1.5 MPa), as measured by a pull-off tension test in accordance with ASTM D4541 (2002). For concrete surfaces that require repair, general methods are provided in ACI 546R (2004) and ICRI 03730 (2008). However, the concrete surfaces must be repaired or reshaped in accordance with the original section.
Before any epoxy or putty-based products are applied for repair, and all contaminants that could interfere with the bond should be removed. The concrete surface preparation should be inspected and approved by the project engineer before such repairs. If cementitious materials are used for repair, they should be allowed to sufficiently cure (i.e. when it is expected to have reached its minimum specified compressive strength) reach before further surface preparation. If corrosion-related concrete deterioration is detected, the cause of the corrosion should be addressed, and the associated deterioration repaired before application of the FRP system.

To repair cracks, those wider than 0.010 in (0.3 mm) should be pressure injected with epoxy before FRP installation, in accordance with ACI 224.1R (2007). Cracks of smaller width may require resin injection or sealing to prevent corrosion of existing reinforcement. ACI 224.1R (2007) provides additional crack-width limitations based on different exposure conditions.

If there is uncertainty in the best approach, a trial run of the surface preparation process should be conducted to determine an effective technique for the FRP system to be used, and approved by the project engineer.

8.2.3.4 Surface smoothness

For contact-critical applications such as confinement (i.e. those in which loss of bond between the concrete and FRP is not critical, but contact between the surfaces must be maintained), the surface preparation should guarantee a continuous contact between the concrete and the FRP confinement system. After repair (if needed), the concrete surface should be prepared to a minimum concrete surface profile (CSP) 3, as defined by ICRI surface profile chips, and localized out-of-plane variations, including form lines, should not exceed 1/32 in (1 mm). Localized variations can be removed by grinding, or can be smoothed over using resin-based epoxy if variations are small. It is best that epoxy materials are applied in thin layers to build up to the desired flatness. Bug holes and voids should be filled with resin-based putty. However, before application of any fillers, the surface must be appropriately cleaned to ensure bond of the repair materials. The maximum size allowed for small depressions is shown in Table 8.2.1.

Table 8.2.1 – Maximum depth of depressions on the concrete surface

Type of FRP	Max. depth for length of 12 in (0.3 m), in	Max. depth for length of 80 in (2.0 m), in
Plates $\geq$ 0.04 in (1.0 mm)	0.16 in (4.0 mm)	0.40 in (10.0 mm)
Plates $<$ 0.04 in (1.0 mm)	0.08 in (2.0 mm)	0.24 in (6.0 mm)
Sheets	0.08 in (2.0 mm)	0.16 in (4.0 mm)

Rectangular cross-sections should have corners rounded to a minimum radius of at least 0.5-1.4 in (13-35 mm). This applies for corners both horizontally and vertically oriented. Chamfered corners are not recommended as a substitute for rounded corners. Larger radii should be used if forming and bonding the FRP to the member corner becomes difficult, which may occur as the thickness and stiffness of the FRP material used increases. Rough corners should be smoothed with epoxy or putty. Inside corners and concave surfaces may be difficult for FRP application, may require special attention to ensure that bond is achieved between the FRP and the concrete.

8.2.3.5 Surface cleanliness

Before application of the FRP, the concrete surface should be cleaned to remove any dust, laitance, oils, dirt, or any other bond-inhibiting material. Even new concrete should be cleaned to remove mold release agents and curing membranes.

Various cleaning techniques can be effective, including wet, dry, and vacuum-abrasive blasting; high-pressure washing, with or without emulsifying detergents, and using biocides (where necessary); steam cleaning alone or in conjunction with detergents; and, for smaller areas, mechanical wire brushing or surface grinding. However, the use of some impact methods such as needle gunning and bush hammering may be too aggressive and are to be carefully monitored, as they and may cause micro-cracks and/or an irregular concrete texture.

Washing techniques may be ineffective in some cases, and can simply spread contaminants further; surface cleanliness should be carefully checked. If solvent-based and sodium hydroxide-based products are used, they must be completely removed from the surface. Vacuum dry-blasting is recommended over "open" blasting, the former of which is safer for workers and the environment. After cleaning, the surface should be protected from contamination prior to FRP installation if necessary. The concrete surface preparation should be inspected and approved by the project engineer before application of the FRP.

8.2.3.6 Resin mixing

Resin components should be mixed at the proper temperature, proportions and the appropriate time until there is a uniform and complete mixing of components that is free from trapped air. For accurate mix proportioning, pre-batched quantities of resins and hardeners are recommended. As resin components are often contrasting colors, full mixing is usually achieved when color is uniform and streaks are eliminated; the mixed result should be visually inspected for this condition. Resin mixing should be in quantities sufficiently small to ensure that all mixed resin can be used within its pot life; adhesive remaining at the end of the specified pot life must be discarded. Strict adherence to the epoxy pot life and operating temperature range during installation is necessary to avoid negatively impacting the quality of installation. Correspondingly, advanced planning is crucial.

8.2.3.7 Application of FRP systems

All materials, including primer, putty, saturating resin and fibers, should be part of the same system. If the use of primer is required of the FRP system, it should be uniformly applied to all surfaces of the concrete where the FRP system is to be placed. Primer should have sufficiently low viscosity to penetrate the surface of the concrete substrate. In particular, it is recommended that higher viscosity resins are limited to plates, and special care is taken to ensure full saturation of FRP fabrics. Once applied, the primer should be protected from dust, moisture, and other contaminants, and allowed to cure before applying the FRP. It should be checked visually and by touch to make sure it is cured and that there is no dust or moisture on the surface.

For FRP installation, a working diagram matching the actual structure should be prepared based on the design. The diagram should clearly identify the reference point for attachment, the overlap splice positions, and the number of plies to be attached.

For wet layup systems, which are typically installed by hand using dry fiber sheets and a saturating resin, the resin should be applied uniformly to all prepared concrete surfaces where the system is to be placed, then the reinforcing fibers gently pressed into the resin. Note that dry fabrics can be directly applied to the resin-saturated concrete surface without adhesive being applied to the fabric, but for wet fabric systems, the resin must be applied to the fabric before it is installed. Resin with sufficient quantity and sufficiently low viscosity should be applied to achieve full saturation of the fibers. A hand-held foam roller or brush can be used to apply the bonding adhesive to the concrete surface or resin, as required; an impregnation machine can also be used to apply resin to the fabric for wet fabric systems. Entrapped air between layers should then be released or rolled out before the resin sets. In doing so, it is recommended to work the FRP materials parallel to the fibers, proceeding in one direction from the center or from one extremity and to avoid any backward and forward movements. After attaching the continuous fiber sheets, an inspection should be done visually or through sounding to verify the absence of lift, swelling, peeling, slackness, wrinkles, and voids in the epoxy resin impregnation.

When using pre-cured systems such as surface bonded plates, the pre-cured laminate surfaces to be bonded should be properly prepared. This may involve application of light abrasion and cleaning. No additional treatment is required for materials with an additional peel ply which, upon removal, exposes a clean surface with the appropriate roughness. The mixed adhesive is then to be applied to the concrete bonding area by hand, using plastering techniques. The thickness of the adhesive should be maintained from 0.04-0.08 in (1-2 mm). The adhesive layer should be applied to the plates to form a slightly convex profile across the plate. Extra thickness along the center-line helps to reduce the risk of void formation. Stacking multiple layers of FRP plate is usually not permitted, except for the overlapping portion of prefabricated L-shaped stirrups. At intersections of FRP plates, care should be exercised to minimize curvature; grooving the concrete for the layer underneath is sometimes used to allow full contact between the plate and the concrete surface.

A protective finish compatible with the proposed system that provides ultraviolet light protection should be applied when the surface of the FRP material has sufficiently cured and has been cleaned. Alternatively, protection can be achieved by applying a plaster or mortar layer (preferably concrete-based) to the installed system. The protective coating may include a wearing layer for protection against abrasive conditions as well, but such a layer is not to be considered as structural reinforcement.

If required for fire protection, intumescent panels or the application of protective plasters may be used. In both cases, the degree of fire protection provided is to be specified, as a function of the panel/plaster thickness. The panels, generally based on calcium silicates, are to be applied directly on the FRP system, provided that fibers will not be cut during their installation. The protection system proposed must be approved by the engineer.

The contractor is to provide a certificate of compatibility of the protective system and the FRP system, prepared by the manufacturer, and the contractor is to provide a guarantee for the performance of the proposed protection system for the expected exposure conditions. Once applied, a minimum of 24 hours should be allowed for the protective coating to dry.

When FRP material is used to wrap the base of a reinforced concrete column that is in contact with the ground, the wrapping should extend a minimum of 20 in (500 mm) below the ground surface to prevent water and air infiltration.

When using continuous fiber strands, the use of a machine to wind the strands, to control the winding interval, tension and speed, is recommended. In this case, it must be verified that the strand winding interval is appropriate; the strand winding tension is constant; the strand winding speed is appropriate; the strands are thoroughly impregnated with resin; that the resin has been suitably mixed and applied; and that the impregnation resin is cured thoroughly.

If carbon fiber is used and there is potential for direct contact between the carbon and existing steel reinforcement, insulating material should be installed to prevent galvanic corrosion.

8.2.3.8 Alignment

The ply orientation and stacking sequence must be specified in the design prior to installation. Materials should be handled such that correct fiber straightness and orientation are preserved. Moreover, kinks, folds, waviness, or other forms of substantial material malformation must be reported for evaluation, as well as angle deviations greater than 5 degrees.

8.2.3.9 Multiple plies and lap splices

When multiple plies are used, it must be verified the resin shear strength is sufficient to transfer the shearing load between plies, and the bond strength between the concrete and FRP system is sufficient. The former can be verified with representative specimen test results (i.e. shear or flexure) provided by the manufacturer/installer. However, the project engineer may limit the maximum number of consecutive layers allowed, and/or restrict the installation period between successive layers. When several superposed layers of are used, care must be taken not to move or otherwise disturb the preceding layers where the resin has not set. If an interruption of the FRP system lay-up process occurs, inter-layer surface preparation such as cleaning or light sanding may be required. All plies must be fully impregnated with resin.

Lap splices may be used, provided that they are staggered, unless otherwise approved by the project engineer. Lap splice details, including required lap length, should be based on testing. Specific guidelines on lap splices are given in ACI Chapter 13. In the absence of manufacturer requirements, a minimum lap splice length of 8 in (200 mm) is recommended. If only one layer of FRP sheet is used within a lap or splice, it is recommended to elongate the overlap splice length and to attach one more layer of FRP over the splice section. When more than one layer is used, the overlap splices should not be placed at the same section, since this reduces the overlap splice strength. Overlap splices should not be placed at locations subjected to large bending moments.

8.2.3.10 Curing

Ambient-cure resins may take several days to reach full cure, and temperature fluctuations can retard or accelerate curing time. The cure status of installed plies should be verified to be sufficient before placing subsequent plies, and the installation of successive layers should be halted if there is a curing anomaly. Successive layers of saturating resin and fibers should be placed before the complete cure of the previous layer; if previous layers have cured to an advanced degree, interlayer surface preparation, such as light sanding or solvent application, may be required.

A minimum curing time of 24 hours should be allowed before further work is performed, unless the curing process is accelerated by heating. For the entire curing duration, the temperature must be maintained above the minimum required curing temperature; condensation on the surface must be prevented; and chemical contamination from gases, dust or liquid sprays must be prevented. Mechanical stresses on the FRP should be minimized during curing. Before the initial setting of the impregnation resin, the surface should be protected with vinyl sheets from rain, dust, excessive sunlight, high humidity, and sudden climatic changes, as necessary. If temporary shoring is used, the FRP system should be fully cured before the shoring is removed.

8.2.4 Inspection

The completeness and quality of inspection procedures for different stages of work vary among the standards reviewed. ISIS inspection procedures are in general recommended to help assess the exiting concrete surface condition and determine the repair method needed, if any. The ISIS inspection procedure to verify the quality of the concrete surface repair prior to FRP application is also recommended. AASHTO and ACI have similar inspection procedures to monitor the quality of the FRP installation. These procedures are reasonably detailed and discuss documentation, sample collection, the use of witness panels, and applicable testing. The AASHTO procedures are ultimately recommended for this purpose. Finally, the inspection procedure to be used at the completion of the installation as presented by ISIS is recommended, due to its comparative completeness. A summary of these recommended provisions, with additional suggestions, is provided below.

Inspection should be conducted by or under the supervision of the project engineer or a qualified inspector knowledgeable of FRP systems and installation procedures. In general, the inspector should look for compliance with the design drawings and project specifications, with details discussed below.

Note that two types of specifications are feasible; descriptive or performance. In a descriptive-focused specification, the engineer specifies the length, width, orientation, installation sequence and other requirements of a particular, selected FRP material, and perhaps acceptable equivalents. In a performance-focused specification, the engineer specifies requirements in term of strength, stiffness, or other necessary properties and characteristics, and the contractor is responsible for selecting an appropriate FRP system and submitting it for approval. Either type of specification is acceptable, provided that sufficient quantitative analysis and /or test data is provided to demonstrate that the required performance requirements will be met with the proposed strengthening plan.

The time and effort spent on inspection is expected to reasonably correlate with the importance level of the structure, as well as the size and complexity of the strengthening scheme. Also of consideration is how critical is the consequence of non-satisfactory system performance. Moreover, the extent of the inspection may be influenced in part by what is initially uncovered. For example, the identification of one or several flaws in the beginning of the inspection phase, or of a systematic error in application, should lead to a more detailed and extensive scrutiny of the system.

However, this is not to suggest that some structures are to be inspected more casually than others. Rather, some strengthening schemes, by nature of their construction, will inherently require more inspection effort. For example, section 8.2.4.4 (see below) recommends that the number of witness panels constructed varies with the size of the strengthened area. In any case, the extent of the planned inspection effort should be influenced by the factors noted in the paragraphs above.

8.2.4.1 Quality assurance and control program

FRP material suppliers, installation contractors, and other relevant parties associated with the FRP strengthening project are to maintain and submit for review a comprehensive quality assurance (QA) and quality control (QC) program. Quality assurance (QA) is achieved through a set of inspections, measurements, and applicable tests to document the acceptability of the surface preparation and the installation of FRP, and the quality control (QC) should cover all aspects of the strengthening project, and will depend on the size and complexity of the project.
All materials used should be manufactured under an approved quality scheme (for example, such as ISO 9000), and conform to a relevant specification or international standard. In addition, the traceability of all materials should be ensured. All external or independent testing to determine material properties should be carried out in approved laboratories in accordance with relevant standards or by the manufacturer under an approved quality scheme. The types and frequency of testing should be stated in the quality plan. A minimum of one sample should be taken at the start and finish of each production run.

8.2.4.2 Material inspection

The FRP manufacturer is to provide documentation demonstrating that the proposed system meets all design requirements such as (but not limited to) tensile strength, type of fibers and resin, durability, resistance to creep, bonding to substrate, and glass transition temperature.

Independent test results, performed according to the QC test plan for the FRP constituent materials and laminates, are required. Test results may include parameters such as tensile strength, glass transition temperature, gel time, pot life, as well as the adhesive shear strength. Note this last value is needed to apply the environmental reduction factors recommended in Section 8.1.3.1.

When material is first received, the FRP, primer, smoothing agent, impregnation resin, and other materials should be inspected for quality and damage. Inspection of materials should be done in accordance with the quality assurance sheet, test results, or other relevant documents issued by the manufacturer. If the materials have suffered damage during shipment, storage at the site, or during construction, they should be rejected or tested to confirm quality.

When received from the supplier, all materials should be accompanied by a certificate of identification and conformity to appropriate standards. Accurate records should be maintained of all materials used (e.g. delivery notes, batch numbers) and, when required, the ambient conditions (e.g. temperature, relative humidity). Visual checks should be carried out to ensure that the material is as specified.

The storage condition of the materials is to be inspected as well, as discussed earlier in these recommendations.

8.2.4.3 Inspection of concrete substrate

The concrete surface should be inspected and tested before application of the FRP material. The inspection should include an examination for completeness of restoration work, processing of corner angles, primer coating, surface smoothness, protuberances, holes, cracks, corners, and other imperfections and characteristics. Pull-off tests should be performed to determine the tensile strength of the concrete for bond-critical applications, as discussed earlier in these recommendations. The degree of surface dryness, including the potential for condensation, should be verified in accordance with the criteria established by the FRP manufacturer.

8.2.4.4 Inspections during installation

During construction, special care should be taken to keep all records on the quantity of mixed resin during a one-day period, the date and time of mixing, the mixture proportions, and identification of all components, ambient temperature, humidity, and other factors that may affect resin properties. These records should also identify the FRP sheets used each day, their location on the structure, the ply count and direction of application, and all other relevant information. This information is useful even if witness panels are tested for acceptance. This is because it is possible that witness panel tests may be found satisfactory even though the manufacturer's requirements for installation are not met. Such a finding may suggest the possibility of long-term durability problems with the installation, even though the initial quality appears satisfactory.

As specified by the project engineer, witness panels are to be manufactured on site and applied on equivalent concrete surfaces using the same preparation and installation procedures as used on the structure. Ideally, the witness areas are in the form of additional strengthening areas applied on the actual structure of size of at least 2-5 ft^2, but not less than 0.5% of the overall area to be strengthened. Witness areas are to be strengthened at the same time of the main FRP installation, and are as uniformly distributed on the structure as possible. The panels are to be kept under the same conditions as the strengthened structure for future testing and evaluation, and fabricated and tested according to a predetermined sampling plan. It is generally recommended that at least some witness panels are constructed for all strengthening installations.

During the installation of the FRP system, daily inspection should be conducted and should note, as applicable:

- Date and time of installation;
- Ambient temperature; relative humidity, and general weather observations;
- Surface temperature of concrete;
- Surface dryness per ACI 503.4 (2003);
- 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 or precured laminate batch number(s) and approximate location in 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 resin cure;
- Conformance with installation procedures;
- Pull-off test results: bond strength, failure mode, and location;
- FRP properties from tests of field sample panels or witness panels, if required;
- Location and size of any delaminations or air voids;
- General progress of work;
- Level of resin curing, in accordance with ASTM D2582 (2009); and
- Adhesion strength.

The FRP system should be further inspected with particular attention to attachment position, orientation and alignment, laminate thickness, waviness, lifting, peeling, slackness, wrinkles, overlap splice length, number of plies, and quantity of the resin coating. If wound on site, FRP strands are to be inspected for winding position, winding interval, winding tension and winding speed, and that fibers are thoroughly impregnated with resin.

8.2.4.5 Project completion

At project completion, a record of all inspections and test results related to the project should be retained. It should include a summary assessment of any problems identified and repairs, on-site bond tests, anomalies, as well as all test results from designated testing facilities.

8.2.5 Evaluation and acceptance

Evaluation is covered thoroughly in ACI Article 7.2 (see Section 3.3.1 of this document), while Chapter 14 of ACI discusses specification requirements and submittals required by the contractor. FRP sample specification requirements are also provided in NCHRP 609, which expand on the requirements stated in Chapter 14 of ACI. AASHTO covers acceptance criteria scope and detail level well. Thus, the evaluation procedures presented by ACI and acceptance criteria presented by AASHTO, supplemented with NCHRP 609, together offer a thorough, compatible, and complementary treatments of this topic, and are recommended. These procedures, together with additional suggestions from other standards, are summarized below.

FRP systems should be evaluated and accepted or rejected based on conformance to the design drawings and specifications. In general, items that were inspected require evaluation. A summary of most critical issues is given below.

8.2.5.1 Materials

As noted earlier in the recommendations, for evaluation and acceptance, the contractor is required to submit evidence of acceptable QC FRP system manufacturing procedures. This should at least include the specifications for raw material procurement, 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. Some acceptable test results are given later in this section. For those not specified, including material and bond properties, unless the project engineer has reason to specify otherwise, no additional specific limitations are recommended, provided that the required design values are met.

The contractor also must provide information describing the fiber, matrix, and adhesive systems to be used that is sufficient to define their engineering properties. Descriptions of the fiber system should include the fiber type, percent of fiber orientation in each direction, and fiber surface treatments. The matrix and the adhesive should 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.

Further, the contractor is to submit 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. These tests are to be conducted by a testing laboratory approved by the engineer. For each property value, the batches from which test specimens were drawn are to be identified and the number of tested specimens from each batch, and the mean, minimum, and maximum value, as well as the coefficient of variation, must be reported. The minimum number of tested samples is 10. In accordance with the QC test plan, test results may include tensile strength, elastic modulus, an infrared spectrum analysis, glass transition temperature, gel time, pot life, and adhesive shear strength, among other parameters relevant to the project. Tensile testing should follow a standard procedure, such as that described by ASTM D3039 (2008).

For FRP systems such as pre-cured and machine-wound systems, that do not lend themselves to the fabrication of small, flat witness panels, the engineer can require test panels or samples provided by the manufacturer.

For bond-critical applications, tension adhesion testing of cored samples (on the structure, after installation) should be conducted using a standard method such as that described in ACI 530R (2005), ASTM D4541 (2002), ASTM D7234 (2012), or the method described by ACI 440.3R (2004), Test Method L.1. Successful tension adhesion strengths should exceed 200 psi (1.4 MPa) and exhibit failure of the concrete substrate. Lower strengths or failure between the FRP system and concrete or between plies should be reported to the engineer. Care should be taken to avoid coring in high stress or splice areas. The tested areas must be repaired unless they are located in areas where the FRP is unstressed. As noted above, sampling frequency may be influenced by the size, complexity, and importance of the project, among other factors.

In-place load testing can also be used to confirm the installed behavior of the FRP-strengthened member. For major structures, it may be appropriate to install instrumentation prior to the strengthening to assess the structural response before and after strengthening.

Once the above information is gathered, the composite material system as well as the adhesive system are to be evaluated for conformance to the requirements that follow. It is assumed that the test specimens were cured under conditions equivalent to those during installation. Ideally, these tests are performed on witness panel samples created for all strengthening projects. However, if the project engineer approves, previous test results of an identical FRP system may be considered for evaluation.

1. The characteristic value of the glass transition temperature of the composite system, determined in accordance with ASTM D4065 (2012), should be at least 40^{o}F higher than the maximum design temperature, defined in Section 3.12.2.2 of the *AASHTO LRFD Bridge Design Specifications* (2012). The characteristic value of the tensile failure strain in the direction corresponding to the highest percentage of fibers must not be less than 1% if the tension test is conducted according to ASTM 3039 (2008).

2. The mean and coefficient of variation of the moisture equilibrium content, as determined in accordance with ASTM D 5229/D 5229M (2010), must not be greater than 2% and 10%, respectively. A minimum sample size of 10 should be used to calculate these values.

3. After conditioning in the various environments listed below, the characteristic value of the glass transition temperature, determined in accordance with ASTM D4065 (2012), and that of tensile strain, determined in accordance with ASTM D3039 (2008), of the composite in the direction of interest, is to retain 85% of the required values given in item 1), above. The conditioning environments are as follows:

- Water: Samples shall be immersed in distilled water having a temperature of 100 ± 3°F (38 ± 2°C) and tested after 1,000 hours of exposure.
- Alternating ultraviolet light and condensation humidity: Samples shall be conditioned in an apparatus under Cycle 1-UV exposure condition according to ASTM G154 (2012) Standard Practice. Samples shall be tested within two hours after removal from the apparatus.
- Alkali: The sample shall be immersed in a saturated solution of calcium hydroxide (pH ~11) at ambient temperature of 73 ± 3^{o}F (23 ± 2^{o}C) for 1000 hours prior to testing. The pH level shall be monitored and the solution shall be maintained as needed.

- Freeze-thaw: Composite samples shall be exposed to 100 repeated cycles of freezing and thawing in an apparatus meeting the requirements of ASTM C666 (2008).

4. If impact tolerance is stipulated by the engineer, impact tolerance should be determined according to ASTM D7136 (2007).

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

- After conditioning in the environments noted in item 3), the characteristic value of the glass transition temperature of the adhesive material, determined in accordance with ASTM D 4065 (2012), must be at least 40°F higher than the maximum design temperature as defined in Section 3.12.2.2 of AASHTO LRFD Bridge Design Specifications (2012).

- Before conditioning, the concrete-FRP interface (resin) shear strength is to be at least (0.065 $\sqrt{f'_c}$)/*RF*, where *RF* is the reduction factor given in section 8.1.3.1 of this chapter, for the anticipated number of design years (f'_c in ksi). After conditioning, the bond strength as well as the concrete-FRP interface (resin) shear strength are to be at least 0.065$\sqrt{f'_c}$ (ksi).

If specified in the project, for verification of fire safety, a test specimen with the same protective coating as the actual structure should be manufactured and subjected to combustion tests. During the combustion test, ignition, the generation of gases, harmful surface deformation, and changes in the quality and strength of the FRP and laminates after the fire are to be studied according to the level of fire safety required.

8.2.5.2 Cure

The relative cure of FRP systems can be evaluated by laboratory testing of witness panels or resin-cup samples using ASTM D3418 (2003) and ASTM D2583 (2007). 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 retained resin samples. The FRP system manufacturer should be consulted to determine the specific resin-cure verification requirements.

It should be evaluated whether moisture will collect at the bond lines between the concrete and epoxy adhesive before the epoxy has time to cure. This may be checked by taping a 4 x 4 ft (1 x 1 m) polyethylene sheet to the concrete surface. If moisture collects on the underside of the sheet before the time required to cure the epoxy, then before application of the adhesive, the concrete should be allowed to dry sufficiently to prevent the possibility of a moisture barrier forming between the concrete and epoxy per ACI 530R-05 (2005).

8.2.5.3 Orientation, placement, and thickness

Installation within specified placement tolerances including width and spacing, corner radii, and lap splice lengths should be evaluated, as well as fiber and pre-cured-laminate orientation and waviness. Misalignment of more than 5 degrees (approximately 1 in/ft [80 mm/m]) from that specified is generally not acceptable without further evaluation, and should be reported to the project engineer.

Cured thickness and/or number of plies used should be verified. Small core samples, typically 0.5 in (13 mm) in diameter, may be taken to visually ascertain the cured laminate thickness or number of plies; however, taking samples from high stress or splice areas should be avoided. These cores may be those used for adhesion testing. The cored hole can generally be filled and smoothed with a repair mortar or the FRP system putty. However, if required, a 4 to 8 in (100 to 200 mm) overlapping FRP sheet patch of equivalent plies may be applied over the filled and smoothed core hole immediately after taking the core sample.

8.2.5.4 Delamination

The cured FRP system should be evaluated for delaminations or air voids between multiple plies or between the FRP system and the concrete. Inspection methods should be capable of detecting delaminations as small as 2 in^2 (1300 mm^2), and may include acoustic sounding (hammer sounding), ultrasonics, and thermography. Delamination size, location, and quantity relative to the overall application area should be considered in the evaluation. For wet layup systems, the need for delamination repair depends on the size and number of delaminations. Small delaminations less than 2 in^2 (1300 mm^2) are permissible as long as the delaminated area is less than 5% of the total laminate area and there are no more than 10 such delaminations per 10 ft^2 (1 m^2). Delaminations exceeding these limits are to be repaired by either resin injection or ply replacement, depending on delamination size. Large delaminations, greater than 25 in^2 (16,000 mm^2), should be repaired by selectively cutting away the affected sheet and applying an overlapping sheet patch of equivalent plies with appropriate overlap length. Delaminations less than 25 in^2 (16,000 mm^2) may be repaired by either resin injection or ply replacement. For precured FRP systems, each delamination should be evaluated and repaired in accordance with the instructions of the engineer. Upon completion of repairs, the laminate should be re-inspected to verify that the repair was properly accomplished.

8.2.6 Maintenance and repair

A maintenance program involves periodic inspection and testing to identify any damage, degradation, or deficiencies to the FRP strengthening system and to make any necessary repairs. A maintenance assessment is made from test data as well as observations, and may include recommendations to help slow down degradation and propose repairs.

AASHTO provides a comprehensive reference list of ACI, ISRI, NCHRP documents dealing with the necessary evaluation criteria for repair as well as post repair evaluation. ACI is one of the listed references which offers a brief, but concise account of inspection and assessment, repair of the strengthening system, and repair of surface coatings (ACI Chapter 8). JSCE Article 9.3 offers some coverage of repair techniques as well (Article 4.2.4). As ACI Chapter 8 is also implicitly included in AASHTO, this, in addition to some of the provisions given by JSCE Article 9.3 for repair techniques (Article 4.2.4 of this document) are generally recommended, with a summary of these provisions and additional suggestions given below.

8.2.6.1 Maintenance Inspections

To verify the long-term performance of the FRP system, a general inspection is recommended at least once a year, while a detailed inspection is recommended at least once every 6 years.

A general (visual) inspection primarily consists of a surface inspection. The inspector looks for changes in color, signs of crazing, cracking, delamination/debonding, peeling, blistering, deflection, or evidence of other deterioration, in addition to local damage due to impact or surface abrasion and other damage and anomalies. Signs of concrete deterioration in the form of cracking or steel reinforcement corrosion should also be reported. The condition of the FRP protective layer, if any, should also be inspected.

Other inspection methods such as ultrasonic, acoustic sounding (hammer tap), or thermographic tests should be used to identify signs of progressive delamination. Although acoustic sounding is recommended for a general inspection, more accurate and time-consuming techniques may be reserved for a detailed inspection.

A detailed inspection should attempt to more accurately quantify the performance and condition of the FRP system. Most signs of FRP degradation, including debonding, can be determined with the test methods noted above. However, adhesive bond strength must be evaluated by puIl-off tests on the control specimens, and should be carried out as part of a detailed inspection. As noted earlier in these recommendations, to quantify the performance of the FRP strengthening system during inspection, and in particular, bond strength, it is recommended that additional areas of the strengthened structure, away from the regions that were strengthened, are also bonded with the FRP system for future testing at the time of installation. Alternatively, FRP can be bonded to long-term witness panels that are stored near the structure, to be inspected and tested as part of the inspection regime.

8.2.6.2 Repair

The causes of any damage or deficiencies detected during inspections should be identified and addressed before performing any repairs or maintenance.

The method of repair should depend on the cause of damage, the type of material, the form of degradation, and the level of damage. Even minor damage should be repaired, including localized FRP laminate cracking or abrasions that may affect the structural integrity of the laminate.

For cracking, wearing, and abrasion, patching should be used. Here, FRP patches are bonded over the damaged area. The FRP patches should possess the same characteristics as the material in place, such as fiber orientation, volume fraction, strength, stiffness, and overall thickness. Minor delaminations over small areas that include swelling, peeling, and lifting, can be repaired by resin injection.

For major damage, such as peeling and debonding of large areas, the defective material should be removed to an extent that material on the periphery of the repair is fully bonded. There is little information available about effective methods of FRP removal. However, a technique that has been successfully used involves first cutting the FRP fabric into pieces of manageable size with an appropriate tool such as an angle grinder, then peeling the sheets from the concrete surface with a crowbar or similar prying tools (conversations with Fyfe, 2014). Since the adhesion strength of the resin is often greater than that of the concrete surface itself, the concrete surface may peel off with the FRP sheet. This will then require repair of the substrate and re-smoothing the surface, as described in Sections 8.2.3.3 and 8.2.3.4. An FRP patch can then be installed that allows for adequate overlap between the new and old materials. The use of chemicals to facilitate FRP sheet removal which may penetrate and damage the concrete surface is not recommended, as the extent of such damage may be difficult to detect and this may cause future bond problems. Due to the lack of standard procedures guiding FRP removal, it is recommended that the contractor submit his plan for FRP removal to the project engineer for approval.

When serious deterioration or deterioration over a wide area is observed, additional FRP upgrading should be performed. In such cases, the existing FRP should be removed and the upgrading plan reexamined.

If the surface protective coating is to be replaced, the FRP should be further inspected for structural damage or deterioration once the coating is removed.

Specific repair techniques for concrete are well-documented. The following additional guides are recommended for identifying and repairing such damage:

- *NCHRP Report 609: Recommended Construction Specifications Process Control Manual for Repair and Retrofit of Concrete Structures Using Bonded FRP Composites*
- *ACI 201.1R: Guide for Making a Condition Survey of Concrete in Service*
- *ACI 224.1R: Causes, Evaluation, and Repair of Cracks in Concrete*
- *ACI 364.1R-94: Guide for Evaluation of Concrete Structures Prior to Rehabilitation*
- *ACI 503R: Use of Epoxy Compounds with Concrete*
- *ACI 546R: Concrete Repair Guide*
- *International Concrete Repair Institute (ICRI) ICRI 03730: Guide for Surface Preparation for the Repair of Deteriorated Concrete Resulting from Reinforcing Steel Corrosion*
- *International Concrete Repair Institute (ICRI) ICRI 03733: Guide for Selecting and Specifying Materials for Repairs of Concrete Surfaces*

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APPENDIX A: NOMENCLATURE

Note: with most codes, for design purposes, FRP properties such as FRP thickness t_{frp} may be calculated based on the single-layer and minimum resin parameters (i.e. using the fiber thickness), or as a composite unit with thickness including the resin. In either case, related material properties such as modulus of elasticity, tensile strength and strain at failure must also be calculated in a manner consistent with which approach is taken.

A.1 AASHTO

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

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

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

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

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

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

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

b = width of rectangular section (in)

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

b_v = effective shear web width (in)

b_w = girder width (in)

C = clamping force across the crack face (kips)

c = depth of the concrete compression zone (in)

D_g = external diameter of circular column (in)

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

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

d_v = effective shear depth (in)

E_a = modulus of elasticity of adhesive (ksi)

E_c = modulus of elasticity of the concrete (ksi)

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

f_c = stress in concrete at strain ε_c (ksi)

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

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

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

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

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

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

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

f_y = specified yield stress of steel reinforcement (ksi)

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

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

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

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

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

k_e = strength reduction factor applied for unexpected eccentricities

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

L_d = development length (in)

l_u = unsupported length of compression member (in)

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

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

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

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

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

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

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

P_r = factored axial load resistance (kips)

r = girder corner radius (in)

s_v = spacing of FRP reinforcement (in)

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

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

t_a = thickness of the adhesive layer (in)

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

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

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

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

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

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

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

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

v_u = effective shear stress (ksi); see AASHTO LRFD 5.8.2.9

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

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

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

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

ε_c = strain in concrete

ε_{frp} = strain in FRP reinforcement

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

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

μ = coefficient of friction

η = strain limitation coefficient that is less than unity

ν_a = Poisson's ratio of adhesive

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

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

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

A.2 ACI 440.2R 08

$\mathrm{A_c}$ = cross-sectional area of concrete in compression member, in^2 (mm^2)

$\mathrm{A_e}$ = cross-sectional area of effectively confined concrete section, in^2 (mm^2)

A_f = area of FRP external reinforcement, in^2 (mm^2)

$\mathrm{A_{fanchor}}$= area of transverse FRP U-wrap for anchorage of flexural FRP reinforcement

A_{fv} = area of FRP shear reinforcement with spacing *s*, in^2 (mm^2)

A_g = gross area of concrete section, in^2 (mm^2)

A_p = area of prestressed reinforcement in tension zone, in^2 (mm^2)

A_s = area of nonprestressed steel reinforcement, in^2 (mm^2)

A_{si} = area of i-th layer of longitudinal steel reinforcement, in^2 (mm^2)

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

a_b = smaller cross-sectional dimension for rectangular FRP bars, in (mm)

b = width of compression face of member, in (mm)

= short side dimension of compression member of prismatic cross section, in (mm)

b_b = larger cross-sectional dimension for rectangular FRP bars, in (mm)

b_w = web width or diameter of circular section, in (mm)

C_E = environmental reduction factor

c = distance from extreme compression fiber to the neutral axis, in (mm)

D = diameter of compression member of circular cross section, in (mm)

d = distance from extreme compression fiber to centroid of tension reinforcement, in (mm)

d_f = effective depth of FRP flexural reinforcement, in (mm)

d_{fv} = effective depth of FRP shear reinforcement, in (mm)

d_i = distance from centroid of i-th layer of longitudinal steel reinforcement to geometric centroid of cross section, in (mm)

d_p = distance from extreme compression fiber to centroid of prestressed reinforcement, in (mm)

D = diagonal distance of prismatic cross section (diameter of equivalent circular column), in (mm) = $\sqrt{b^2 + h^2}$

E_2 = slope of linear portion of stress-strain model for FRP-confined concrete, psi (MPa)

E_c = modulus of elasticity of concrete, psi (MPa)

E_f = tensile modulus of elasticity of FRP, psi (MPa)

E_{ps} = modulus of elasticity of prestressing steel, psi (MPa)

E_s = modulus of elasticity of steel, psi (MPa)

e_s = eccentricity of prestressing steel with respect to centroidal axis of member at support, in (mm)

e_m = eccentricity of prestressing steel with respect to centroidal axis of member at midspan, in (mm)

f_c = compressive stress in concrete, psi (MPa)

f'_c = specified compressive strength of concrete, psi (MPa)

$\overline{f'_c}$ = mean ultimate tensile strength of FRP based on a population of 20 or more tensile tests per ASTM D3039, psi (MPa)

$\sqrt{f'_c}$ = square root of specified compressive strength of concrete

f'_{cc} = compressive strength of confined concrete, psi (MPa)

f'_{co} = compressive strength of unconfined concrete; also equal to 0.85 f'_c, psi (MPa)

$f_{c,s}$ = compressive stress in concrete at service condition, psi (MPa)

f_f = stress level in FRP reinforcement, psi (MPa)

f_{fd} = design stress of externally bonded FRP reinforcement, psi (MPa)

f_{fd} = design stress of externally bonded FRP reinforcement, psi (MPa)

f_{fe} = effective stress in the FRP; stress level attained at section failure, psi (MPa)

$f_{f,s}$ = stress level in FRP caused by a moment within elastic range of member, psi (MPa)

f_{fu} = design ultimate tensile strength of FRP, psi (MPa)

f^*_{fu} = ultimate tensile strength of the FRP material as reported by the manufacturer, psi (MPa)

f_l = maximum confining pressure due to FRP jacket, psi (MPa)

f_{ps} = stress in prestressed reinforcement at nominal strength, psi (MPa)

f_{pu} = specified tensile strength of prestressing tendons, psi (MPa)

f_s = stress in nonprestressed steel reinforcement, psi (MPa)

f_{si} = stress in the *i*-th layer of longitudinal steel reinforcement, psi (MPa)

$f_{s,s}$ = stress level in nonprestressed steel reinforcement at service loads, psi (MPa)

f_y = specified yield strength of nonprestressed steel reinforcement, psi (MPa)

h = overall thickness or height of a member, in (mm)

= long side cross-sectional dimension of rectangular compression member, in (mm)

h_f = member flange thickness, in (mm)

I_{cr} = moment of inertia of cracked section transformed to concrete, in^4 (mm^4)

I_{tr} = moment of inertia of uncracked section transformed to concrete, in^4 (mm^4)

k = ratio of depth of neutral axis to reinforcement depth measured from extreme compression fiber

k_1 = modification factor applied to k_v to account for concrete strength

k_2 = modification factor applied to k_v to account for wrapping scheme

k_f = stiffness per unit width per ply of the FRP reinforcement, lb/in (N/mm);

$$k_f = E_f t_f$$

L_e = active bond length of FRP laminate, in (mm)

l_{db} = development length of near-surface-mounted (NSM) FRP bar, in (mm)

l_{df} = development length of FRP system, in (mm)

M_{cr} = cracking moment, in-lb (N-mm)

M_n = nominal flexural strength, in-lb (N-mm)

M_{nf} = ontribution of FRP reinforcement to nominal flexural strength, lb-in (Nmm)

M_{np} = contribution of prestressing reinforcement to nominal flexural strength, lb-in (N-mm)

M_{ns} = contribution of steel reinforcement to nominal flexural strength, lb-in (N-mm)

M_s = service moment at section, in-lb (N-mm)

M_{snet} = service moment at section beyond decompression, in-lb (N-mm)

M_u = factored moment at a section, in-lb (N-mm)

n = number of plies of FRP reinforcement

n_f = modular ratio of elasticity between FRP and concrete = $\frac{E_f}{E_c}$

n_s = modular ratio of elasticity between steel and concrete = $\frac{E_s}{E_c}$

p_e = effective force in prestressing reinforcement (after allowance for all prestress losses), lb (N)

p_n = nominal axial compressive strength of a concrete section, lb (N)

$\overline{P_{fu}}$ = mean tensile strength per unit width per ply of FRP reinforcement, lb/in (N/mm)

P_{fu}^* = ultimate tensile strength per unit width per ply of FRP reinforcement, lb/in (N/mm); P_{fu}^* = $f_{fu}^*\ t_f$

R_n = nominal strength of a member

$R_{n\phi}$ = nominal strength of a member subjected to elevated temperatures associated with a fire

r = radius of gyration of a section, in (mm)

r_c = radius of edges of a prismatic cross section confined with FRP, in (mm)

S_{DL} = dead load effects

S_{LL} = live load effects

T_g = glass-transition temperature, °F (°C)

T_{gw} = wet glass-transition temperature, °F (°C)

T_{ps} = tensile force in prestressing steel, lb (N)

t_f = nominal thickness of one ply of FRP reinforcement, in (mm)

V_c = nominal shear strength provided by concrete with steel flexural reinforcement, lb (N)

V_f = nominal shear strength provided by FRP stirrups, lb (N)

V_n = nominal shear strength, lb (N)

V_s = nominal shear strength provided by steel stirrups, lb (N)

w_f = width of FRP reinforcing plies, in (mm)

y_b = distance from centroidal axis of gross section, neglecting reinforcement, to extreme bottom fiber, in/in (mm/mm)

y_t = vertical coordinate within compression region measured from neutral axis position. It corresponds to transition strain ε_t' , in (mm)

α_1 = multiplier on fc' to determine intensity of an equivalent rectangular stress distribution for concrete

α_L = longitudinal coefficient of thermal expansion, in/in/°F (mm/mm/°C)

α_T = transverse coefficient of thermal expansion, in/in/°F (mm/mm/°C)

β_1 = ratio of depth of equivalent rectangular stress block to depth of the neutral axis

ε_b = strain level in concrete substrate developed by a given bending moment (tension is positive), in/in (mm/mm)

ε_{bi} = strain level in concrete substrate at time of FRP installation (tension is positive), in/in (mm/mm)

ε_b = strain level in concrete, in/in (mm/mm)

ε'_c = maximum strain of unconfined concrete corresponding to f'_c, in/in (mm/mm); may be taken as 0.002.

ε_{ccu} = ultimate axial compressive strain of confined concrete corresponding to 0.85fcc' in a lightly confined member (member confined to restore its concrete design compressive strength), or ultimate axial compressive strain of confined concrete corresponding to failure in a heavily confined member.

$\varepsilon_{c,s}$ = strain level in concrete at service, in/in (mm/mm)

ε_{ct} = concrete tensile strain at level of tensile force resultant in post-tensioned flexural members, in/in (mm/mm)

ε_{cu} = ultimate axial strain of unconfined concrete corresponding to 0.85fco' or maximum usable strain of unconfined concrete, in/in (mm/mm), which can occur at 0.85fc' or 0.003, depending on the obtained stress-strain curve

ε_{f} = strain level in the FRP reinforcement, in/in (mm/ mm)

ε_{fd} = debonding strain of externally bonded FRP reinforcement, in/in (mm/mm)

ε_{fe} = effective strain level in FRP reinforcement attained at failure, in/in (mm/mm)

$\overline{\varepsilon_{fu}}$ = mean rupture strain of FRP reinforcement based on a population of 20 or more tensile tests Per ASTM D3039, in/in (mm/mm)

ε_{fu}^{*} = ultimate rupture strain of FRP reinforcement, in/in (mm/mm)

ε_{pe} = effective strain in prestressing steel after losses, in/in (mm/mm)

ε_{pi} = initial strain level in prestressed steel reinforcement, in/in (mm/mm)

ε_{pnet} = net strain in flexural prestressing steel at limit state after prestress force is discounted (excluding strains due to effective prestress force after losses), in/in (mm/mm)

ε_{ps} = strain in prestressed reinforcement at nominal strength, in/in (mm/mm)

ε_{s} = strain level in nonprestessed steel reinforcement, in/in (mm/mm)

ε_{sy} = strain corresponding to yield strength of nonprestressed steel reinforcement, in/in (mm/mm)

ε_t = net tensile strain in extreme tension steel at nominal strength, in/in (mm/mm)

$\mathcal{E}_t'$ = transition strain in stress-strain curve of FRPconfined concrete, in/in (mm/mm)

ϕ = strength reduction factor

κ_a = efficiency factor for FRP reinforcement in determination of f_{cc}' (based on geometry of cross section)

κ_b = efficiency factor for FRP reinforcement in determination of ε_{ccu} (based on geometry of cross section)

κ_v = bond-dependent coefficient for shear

κ_ε = efficiency factor equal to 0.55 for FRP strain to account for the difference between observed rupture strain in confinement and rupture strain determined from tensile tests

ρ_f = FRP reinforcement ratio

ρ_g = ratio of area of longitudinal steel reinforcement to cross-sectional area of a compression member (*As* /*bh*)

ρ_s = ratio of nonprestressed reinforcement

σ = standard deviation

τ_b = average bond strength for NSM FRP bars, psi (MPa)

ψ_f = FRP strength reduction factor

= 0.85 for flexure (calibrated based on design material properties)

= 0.85 for shear (based on reliability analysis) for three-sided FRP U-wrap or two-sided strengthening schemes

= 0.95 for shear fully wrapped sections

A.3 ISIS

Symbol	Description	S6-06	S806-02
a	depth of an equivalent rectangular stress block, mm	a	
A_{FRP}	area of cross-section of an FRP bar, plate, sheet or tendon, mm^2	A_{FRP}	A_F
A_g	gross area of section, mm^2		A_g
A_p	area of pre-stressing tendons in tension zone, mm^2	A_{ps}	
A_{ps}	total area of steel tendons, mm^2		
A_s	area of steel tension reinforcement, mm^2	A_s	A_s
A'_s	area of compression steel reinforcement, mm	A'_s	A'_s
A_{sf}	area of tension steel reinforcement which equilibrates the compression force in the slab portion of a T-section, mm^2		
A_{sj}	area of intermediate longitudinal steel reinforcement placed on the sides of the element, mm^2		
A_{sw}	remaining area of tension steel reinforcement in a T-section, mm^2		
A_v	area of shear steel reinforcement perpendicular to the axis of a member within a distance s , mm^2	A_v	A_v
b	width of a rectangular section, mm	b	b
b_e	effective width of compression face of member, mm		
b_{FRP}	width of FRP shear reinforcement measured perpendicular to the longitudinal axis of the element, mm		
b_v	effective web width within depth d_v , mm	b_v	
b_w	width of web of a T-section, mm	b_w	b_w
c	distance from extreme compression face to neutral axis, mm	c	c
c'	distance from the neutral axis to the position where the compression strain is ε'_c in the concrete, mm		
C_b	distance from extreme compression face to neutral axis for balanced conditions, mm		
C_c	compressive resultant force from the concrete stressed lower than f'_c, N		
C_{cc}	compressive resultant force from the confined concrete, N		

C_f	compressive resultant force from the concrete in the slab portion of a T-section, N		
C_s	compressive resultant force from the compression steel, N		
C_w	compressive resultant force from the concrete in the web portion of a T-section, N		
d	effective depth of a reinforced concrete component, being the distance from the extreme compression face to the centroid of the tension steel reinforcement, mm	d	d
D	dead load, N	D	D
d'	distance from extreme compression face to the centroid of compression steel reinforcement, mm		
d_{FRP}	effective shear depth for FRP, calculated similar to d_V for steel reinforcement in accordance with Clause 8.9.1.5 of CSA S6-06, or the distance from extreme compression fibre to centroid of tension FRP reinforcement, mm	d_{FRP}	D_f
D_g	diameter of a circular column or equivalent diameter of a rectangular column, mm	D_g	D
d_{sj}	distance from extreme compression face to the position of intermediate steel reinforcement, mm		
d_V	S6-06: the effective shear depth for internal steel, as defined in Clause 8.9.1.5 of CSA S6-06, mm	d_V	
E_C	modulus of elasticity of concrete, MPa	E_C	E_C
E_f	modulus of elasticity of the fibres, MPa		
E_{FRP}	modulus of elasticity of FRP, MPa	E_{FRP}	E_F
EI	flexural stiffness of the element, N.mm2	EI	
E_m	modulus of elasticity of the resin matrix, MPa		
E_p	modulus of elasticity of steel tendons, MPa	E_p	E_p
E_s	modulus of elasticity of steel, MPa	E_s	E_s
F	live load capacity factor	F	
f_c	compression stress in concrete, MPa		
f'_c	specified compressive strength of concrete, MPa	f'_c	f'_c
f'_{cc}	compressive strength of confined concrete, MPa	f'_{cc}	f'_{cc}
f_{cr}	cracking strength of the concrete, MPa	f_{cr}	
f_f	tensile strength of the fibres, MPa		

f_{FRP}	tensile stress in a given direction, generally the fibre direction, of a FRP, MPa	f_{FRP}	f_F
f_{FRPu}	specified tensile strength of an FRP bars, plates, sheets, or tendons, MPa	f_{FRPu}	f_{Fu}
f_{lFRP}	confinement pressure due to FRP strengthening at the ULS, MPa	f_{lFRP}	f_l
f_m	tensile strength of the resin matrix, MPa		
f_{po}	stress in prestressed steel reinforcement when stress in the surrounding concrete is zero, MPa	f_{po}	
f_{pu}	specified tensile strength of prestressing steel, MPa	f_{pu}	
f_s	tensile stress in steel reinforcement, MPa	f_s	
f'_s	compressive stress in steel reinforcement, MPa		
f_{se}	effective stress in the prestressing steel after losses, MPa	f_{se}	
F_{sj}	resultant force from the intermediate steel, N		
f_Y	specified yield strength of steel reinforcement, MPa	f_Y	f_Y
h	overall thickness of a component, mm lateral dimension of the cross-section in the direction considered, mm longer dimension of a column, mm	h	h
h_f	slab thickness, mm		
h_{FRP}	height of FRP bonded on the lateral side of the member, mm		
k	effective length factor for compression elements	k	
k_1	concrete strength factor as defined in Clause 16.11.3.2 of CSA S6-06	k_1	
k_2	a non-dimensional factor as defined in Clause 16.11.3.2 of CSA S6-06	k_2	
k_c	confinement coefficient		k_c
k_e	strength reduction factor applied for unexpected eccentricities		
k_l	confinement parameter		k_l
K_v	bond reduction coefficient for externally-bonded FRP stirrups	K_v	
L	width of shear wall, mm unsupported length of a flexural member measured from centre to centre of supports, m	L	L

	live load, N		
l_a	minimum required anchorage length for externally-bonded FRP beyond the point where no strengthening is required, mm	l_a	
L_e	effective anchorage length of external FRP shear reinforcement, mm	L_e	
l_n	clear span of flexural member, mm		
l_u	unsupported length of a compression member, mm	l_u	
M_1	value of the smaller end moment at the ULS due to factored loads acting on a compression member, to be taken as positive if the member is bent in single curvature and negative if it is bent in double curvature, N.mm	M_1	
M_2	value of the larger end moment at the ULS due to factored loads acting on a compression member, always taken as positive, N.mm	M_2	
M_f	factored moment at a section, N.mm	M_f	
M_r	factored flexural resistance of a section in bending, N.mm	M_r	
M_{rf}	resisting moment corresponding to slab portion of a T-section, N.mm		
M_{rw}	resisting moment corresponding to web portion of a T-section, N.mm		
N_f	factored axial load normal to the cross-section occurring simultaneously with V_f , N	N_f	
P_D	axial dead load, N		
P_E	axial earthquake load, N		
P_f	factored axial load at a section at the ULS, N	P_f	P_f
P_L	axial live load, N		
P_r	factored axial load resistance, N factored axial resistance of a section in compression with minimum eccentricity, N	P_r	
P_{ro}	factored axial load resistance at zero eccentricity of the unconfined section, N		P_{ro}
r	radius of gyration of gross cross-section, mm	r	
s	spacing of steel shear reinforcement measured parallel to the longitudinal axis of the member, mm	s	s

S_{FRP}	spacing of externally-bonded FRP bands on concrete for shear strengthening measured along the axis of the member or unit width (i.e., 1.0) of a continuous FRP shear reinforcement, mm	S_{FRP}	S_F
S_{ze}	equivalent crack spacing parameter, mm	S_{ze}	
t_{FRP}	total thickness of externally-bonded FRP plates or sheets, mm	t_{FRP}	t_i
T_{FRP}	tensile resultant force from the longitudinal FRP, N		
$T_{FRP,force}$	tensile resultant force from the longitudinal FRP bonded on the tension face, N		
$T_{FRP,side}$	resultant force from the longitudinal FRP subjected to tension and bonded on the sides of the section, N		
T_g	glass transition temperature, °C		
T_{gw}	wet glass transition temperature, °C	T_{gw}	
T_s	tensile resultant force from the tension steel, N		
V_c	factored shear resistance attributed to the concrete, N	V_c	V_c
ν_f	volumetric ratio of fibres within the FRP		
V_f	factored shear force at the section, N	V_f	
V_{FRP}	factored shear resistance provided by FRP shear reinforcement, N	V_{FRP}	V_F
ν_m	volumetric ratio of resin matrix within the FRP		
V_P	factored shear resistance provided by the component in the direction of the applied shear of all the effective prestressing forces; positive if resisting the applied shear, N	V_P	V_P
V_r	factored shear resistance, N	V_r	V_r
V_s	factored shear resistance provided by steel shear reinforcement, N	V_s	V_s
w_{FRP}	width of FRP sheet measured perpendicular to the direction of main fibres, mm	w_{FRP}	
α	depth of an equivalent rectangular stress block, mm	α	
α_1	ratio of average stress in rectangular compression block to the specified concrete compressive strength	α_1	α_1
α_D	load factor for dead load	α_D	α_D
α_L	load factor for live load	α_L	α_L

α_P	angle of inclination of steel tendon force to the longitudinal axis of the member	α	
β	angle of inclination of the transverse reinforcement to the longitudinal axis of the member	β	θ
β_1	ratio of the depth of rectangular compression block to the depth of the neutral axis	β_1	β_1
β_v	factor to account for the shear resistance of cracked concrete	β	
δ	design lateral drift ratio		δ
Δf_c	additional concrete stress due to confinement, MPa		
ε_c	strain in concrete		
ε_{ci}	initial strain in concrete		
ε'_c	strain in concrete at f'_c		
ε'_{cc}	strain in concrete at f'_{cc}		
ε_{cu}	ultimate compressive strain of concrete		ε_{cu}
ε_{ft}	initial tensile strain at the location of FRP before applying FRP		ε_{Fi}
ε_{FRP}	strain in FRP reinforcement		ε_F
ε_{FRPe}	effective strain in FRP	ε_{FRPe}	
ε_{FRPt}	maximum permitted tensile strain in FRP flexural strengthening system		
ε_{FRPu}	ultimate strain of FRP	ε_{FRPu}	ε_{Fu}
ε_s	tensile strain in tension steel reinforcement		
ε'_s	compressive strain in compression steel reinforcement		
ε_{si}	initial tensile strain in tension steel reinforcement		
ε'_{si}	initial compressive strain in compression steel reinforcement		
ε_{sj}	tensile strain in intermediate steel reinforcement		
ε_x	longitudinal strain	ε_x	
ε_y	yield strain of steel		
ϕ_c	resistance factor for concrete	ϕ_c	ϕ_c
ϕ_{FRP}	resistance factor for FRP	ϕ_{FRP}	ϕ_F

γ_c	mass density of concrete, kg/m	γ_c
λ	parameter depending on the density of concrete	λ
θ	angle of inclination of the principal diagonal compressive stress to the longitudinal axis of the member	θ
ω_D	linear dead load, kN/m	
ω_L	linear live load, kN/m	

A.4 CNR-DT 200/2004

General notation

$(.)_c$ = value of quantity (.) for concrete

$(.)_{cc}$ = value of quantity (.) for confined concrete

$(.)_d$ = design value of quantity (.)

$(.)_f$ = value of quantity (.) for the fiber-reinforced composite

$(.)_k$ = characteristic value of quantity (.)

$(.)_{mc}$ = value of quantity (.) for confined masonry

$(.)_R$ = value of quantity (.) as resistance

$(.)_s$ = value of quantity (.) for steel

$(.)_S$ = value of quantity (.) as demand

Uppercase Roman Letters

A_c = area of concrete cross-section, net of steel reinforcement

A_f = area of FRP reinforcement

A_{fw} = area of FRP shear reinforcement

A_l = overall area of longitudinal steel reinforcement

A_{sw} = area of one stirrup leg

As_1 = area of steel reinforcement subjected to tension

A_{s2} = area of steel reinforcement subjected to compression

E_c = Young's modulus of elasticity of concrete

E_f = Young's modulus of elasticity of FRP reinforcement

E_{fib} = Young's modulus of elasticity of fiber itself

E_m = Young's modulus of elasticity of matrix

E_s = Young's modulus of elasticity of steel reinforcement

$F_{max,d}$ = design value of the maximum tensile force transferred by FRP reinforcement to the concrete support

F_{pd} = design value of the maximum anchorage force transferred by FRP reinforcement bonded on a masonry structure in the presence of a force perpendicular to the bonded surface area

G_a = shear modulus of adhesive

G_c = shear modulus of concrete

I_o = moment of inertia of cracked and un- strengthened reinforced concrete section

I_1 = moment of inertia of cracked and FRP-strengthened reinforced concrete section

I_c = moment of inertia of transformed section

I_f = moment of inertia of FRP reinforcement about its centroidal axis, parallel to the beam neutral axis

M_{Rd} = flexural capacity of FRP-strengthened member

M_{Sd} = factored moment

M_o = bending moment acting before FRP strengthening

M_1 = bending moment applied to the RC section due to loads applied after FRP strengthening

$N_{Rcc,d}$ = axial capacity of FRP-confined concrete member

$N_{Rmc,d}$ = axial capacity of FRP-confined masonry

N_{Sd} = factored axial force

P_{fib} = weight fraction of fibers

P_m = weight fraction of the matrix

T_g = glass transition temperature of the resin

T_m = melting temperature of the resin

T_{Rd} = torsional capacity of FRP-confined concrete member

$T_{Rd,f}$ = FRP contribution to the torsional capacity

$T_{Rd,max}$ = torsional capacity of the compressed concrete strut

$T_{Rd,s}$ = steel contribution to the torsional capacity

T_{Sd} = factored torsion

T_x = Yarn count in x direction

V_{fib} = volumetric fraction of fibers

V_{Rd} = shear capacity of FRP-strengthened member

$V_{Rd,ct}$ = concrete contribution to the shear capacity

$V_{Rd,max}$ = maximum concrete contribution to the shear capacity

$V_{Rd,s}$ = steel contribution to the shear capacity

$V_{Rd,f}$ = FRP contribution to the shear capacity

$V_{Rd,m}$ = masonry contribution to the shear capacity

V_{Sd} = factored shear force

Lowercase Roman Letters

b_f = width of FRP reinforcement

d = distance from extreme compression fiber to centroid of tension reinforcement

f_{bd} = design bond strength between FRP reinforcement and concrete (or masonry)

f_{bk} = characteristic bond strength between FRP reinforcement and concrete (or masonry)

f_c = concrete compressive strength (cylindrical)

f_{ccd} = design strength of confined concrete

f_{cd} = design concrete compressive strength

f_{ck} = characteristic concrete compressive strength

f_{ctm} = mean value of concrete tensile strength

f_{fd} = design strength of FRP reinforcement

f_{fdd} = design debonding strength of FRP reinforcement (mode 1)

$f_{fdd,2}$ = design debonding strength of FRP reinforcement (mode 2)

f_{fed} = effective design strength of FRP shear reinforcement

f_{fk} = characteristic strength of FRP reinforcement

f_{fpd} = design debonding strength of FRP reinforcement

f_{mk} = characteristic compressive strength of masonry

f_{mk}^{h} = characteristic compressive strength of masonry in the horizontal direction

f_{mcd} = characteristic compressive strength of FRP-confined masonry

f_{md} = design compressive strength of masonry

f_{md}^{h} = design compressive strength of masonry in the horizontal direction

f_{mtd} = design tensile strength of masonry

f_{mtk} = characteristic tensile strength of masonry

f_{mtm} = mean value of the tensile strength of masonry

f_{vd} = design shear strength of masonry

f_{vk} = characteristic shear strength of masonry

f_y = yield strength of longitudinal steel reinforcement

f_{yd} = design yield strength of longitudinal steel reinforcement

f_{ywd} = design yield strength of transverse steel reinforcement

f_l = confining lateral pressure

$f_{l,eff}$ = effective confining pressure

h = section depth

k_{eff} = coefficient of efficiency for confinement

k_H = coefficient of efficiency in the horizontal direction

k_V = coefficient of efficiency in the vertical direction

k_α = Coefficient of efficiency related to the angle of fibers respect to the longitudinal axis

l_b = bond length

l_e = optimal bond length

p_b = distance between layers of bars in the confinement of masonry columns

p_f = spacing of FRP strips or discontinuous FRP U-wraps

s = interface slip

s_f = interface slip at full debonding

t_f = thickness of FRP laminate

w_f = width of FRP laminate

x = distance from extreme compression fiber to neutral axis

Uppercase Greek Letters

Γ_{Fk} = characteristic value of specific fracture energy

Γ_{Fd} = design value of specific fracture energy

Lowercase Greek Letters

α_{fE} = safety coefficient for fabric stiffness

α_{ff} = safety coefficient for fabric strength

γ_{m} = partial factor for materials

γ_{Rd} = partial factor for resistance models

ε_{o} = concrete strain on the tension fiber prior to FRP strengthening

ε_{c} = concrete strain on the compression fiber

$\varepsilon_{\mathrm{ccu}}$ = design ultimate strain of confined concrete

$\varepsilon_{\mathrm{co}}$ = concrete strain on the compression fiber prior to FRP strengthening

$\varepsilon_{\mathrm{cu}}$ = ultimate strain of concrete in compression

ε_{f} = strain of FRP reinforcement

$\varepsilon_{\mathrm{fd}}$ = design strain of FRP reinforcement

$\varepsilon_{\mathrm{fd,rid}}$ = reduced design strain of FRP reinforcement for confined members

$\varepsilon_{\mathrm{fk}}$ = characteristic rupture strain of FRP reinforcement

$\varepsilon_{\mathrm{fdd}}$ = maximum strain of FRP reinforcement before debonding

$\varepsilon_{\mathrm{mcu}}$ = ultimate compressive strain of confined masonry

$\varepsilon_{\mathrm{mu}}$ = ultimate compressive strain of masonry

$\varepsilon_{\mathrm{s1}}$ = strain of tension steel reinforcement

$\varepsilon_{\mathrm{s2}}$ = strain of compression steel reinforcement

$\varepsilon_{\mathrm{yd}}$ = design yield strain of steel reinforcement

η = conversion factor

ν_{fib} = Poisson’s ratio of fibers

ν_{m} = Poisson's ratio of matrix

ρ_{fib} = fiber density

ρ_{m} = matrix density

σ_{c} = stress in the concrete

σ_{f} = stress in FRP reinforcement

σ_{s} = stress in tensile steel reinforcement

σ_{Sd} = stress normal to masonry face acting on the bonded surface area between FRP reinforcement and masonry

$\tau_{\mathrm{b,e}}$ = equivalent shear stress at the adhesive-concrete interface

ϕ_{u} = curvature at ultimate

ϕ_{y} = curvature at yielding

A.5 TR- 55

A_f = area of FRP

A_{fa} = area of effectively anchored additional FRP tensile reinforcement

A_{fa} = area of FRP shear reinforcement

A_s = area of tensile steel reinforcement

A'_s = area of compression steel reinforcement

b = width of section

b_a = width of adhesive layer

b_f = width of plate

b_w = beam width or plate spacing for solid slab

D = diameter of column

d = effective depth of section

d' = effective depth of compression steel

d_f = effective depth of FRP shear reinforcement

E_c = modulus of elasticity of concrete

E_f = modulus of elasticity of FRP

E_{fd} = design elastic modulus of FRP

E_{fk} = characteristic elastic modulus of FRP

E_i = initial tangent modulus of concrete

E_o , E_1 = secant modulus of concrete

E_p = post-crushing tangent modulus

E_s = modulus of elasticity of steel

$E_{\theta\theta}$ = hop modulus of FRP

F_f = tensile force in FRP

F_s = tensile force in steel reinforcement

F_s' = compressive force in steel reinforcement

f_{cc} = confined concrete compressive strength

f_{ccd} = design confined concrete compressive strength

f_{cck} = characteristic confined concrete compressive strength = f_{co}+ 4.1 × 0.85 f_{fu} t_f / R

f_{ck} = characteristic compressive cylinder strength of concrete ≈ 0.85 f_{cu}

f_{co} = unconfined concrete compressive strength

f_{ctm} = tensile strength of concrete = 0.18 $(f_{cu})^{2/3}$

f_{cu} = characteristic compressive cube strength of concrete

f_f = design tensile strength of FRP

f_{fd} = ultimate design tensile strength of FRP

f_{fk} = characteristic tensile strength of FRP

f_{fm} = mean tensile strength of FRP

f_0 = intercept of post-crushing tangent modulus with the stress axis = $f_{cu}(E_i$ - $E_p)$ / $(E_i$- $E_l)$

f_r = confinement pressure

f_y = characteristic tensile strength of steel reinforcement

f_y' = compressive strength of steel reinforcement

G = dead load

h = overall depth of member

I_{ce} = second moment of area of existing concrete equivalent transformed cracked section

I_{cs} = second moment of area of strengthened concrete transformed cracked section

k_b = factor, defined in Equation 6.21

L_e = effective bond length

l_t = anchorage length

$l_{t,max}$ = maximum anchorage length

M = design ultimate moment

M_{add} = additional required moment capacity

M_0 = moment capacity of existing beam

M_r = design resistance moment of strengthened section

$M_{r,b}$ = balanced moment of resistance

M_s = service moment based on unfactored permanent loads

M_u = ultimate moment

Q = live load

R = radius of column

s_f = spacing of FRP strips

T_k = characteristic bond failure force

$T_{k,max}$ = ultimate bond failure force

t_f = thickness of FRP

V = ultimate shear force at the plate end

V_{Rc} = shear resistance of concrete

V_{Re} = shear resistance of existing member

V_{Rf} = shear resistance of FRP

V_{Rl} = shear resistance of links

$V_{R,max}$ = maximum shear resistance of member

V$_{Rs}$ = shear resistance of strengthened member

V$_{s}$ = shear force due to ultimate loads

V$_{sd}$ = design shear force

V$_{max}$ = maximum permissible shear stress

w$_{f}$ = width of FRP shear reinforcement strips

w$_{fe}$ = effective width of FRP

x = depth of neutral axis of existing member

z = lever arm

α_c = modular ratio of steel to concrete

α_f = modular ratio of FRP to concrete

β = angle between FRP and the longitudinal axis of the member = 45° or 90°

γ_{mA} = partial safety factor for adhesive

γ_{mc} = partial safety factor for concrete

γ_{mE} = partial safety factor for modulus of elasticity of FRP

γ_{mF} = partial safety factor for FRP

γ_{mf} = partial safety factor for strength of FRP

γ_{mm} = partial safety factor for manufacture of FRP

γ_{ms} = partial safety factor for steel

ϵ_{cc} = axially confined concrete strain

ϵ_{ccu} = ultimate axial confined concrete strain = $\epsilon_{fu}/\ \nu_c\ (1 + \sqrt{\frac{f_{c\,0}R}{f_{fu}\,t_f}}\ )$

ϵ_{cft} = final tensile strain of concrete

ϵ_{cic} = initial compressive strain of concrete due to M_s

ϵ_{cit} = initial tensile strain of concrete due to M_s

ϵ_{cu} = ultimate compressive strain of (unconfined) concrete = 0.0035

ϵ_{fe} = effective FRP strain

ϵ_{fk} = characteristic failure strain of FRP

ϵ_{fu} = design ultimate strain of FRP

ϵ_{y} = yield strain of steel = 0.002

υ_{c} = poisson's ratio for concrete = 0.2

ρ_{f} = FRP shear reinforcement ratio

τ = longitudinal shear stress

APPENDIX B: DESIGN EXAMPLES

The following examples illustrate the recommended design procedures presented in Chapter 8:

Example 1: Flexural strengthening of a reinforced concrete girder (modified AASHTO)

Example 2: Flexural strengthening of prestressed concrete girder (modified AASHTO)

Example 3: Shear strengthening with 2-sided wrap (AASHTO)

Example 4: Shear strengthening with U-wrap (AASHTO)

Example 5: Axial strengthening of a confined circular column (AASHTO procedure)

Example 6: Axial strength of a confined square column (AASHTO procedure)

Example 7: Axial strength of a confined circular column (ACI procedure)

Example 8: Axial strength of a confined circular column (ACI procedure)

Note:

The following abbreviations are used for document references:

AASHTO: AASHTO Guide Specifications for Design of Bonded FRP Systems for Repair and Strengthening of Concrete Bridge Elements, 1st Ed. (2014)
LRFD: AASHTO LRFD Bridge Design Specifications (2012)
NCHRP: NCHRP Report No. 655 (2010)
ACI: 440.2R, Guide for the Design and Construction of Externally Bonded FRP Systems (2008)

B1 Flexural strengthening of a simply supported cast in-place reinforced concrete girder

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

Geometry and material properties:

Bridge span= 55 feet

Concrete compression strength, f'_c= 4 ksi

Reinforcing steel yield strength, f_y= 60 ksi

Steel reinforcement = 10 # 10 (A_s= 12.70 in^2)

Effective flange width, b_{eff} = 72 in

Ultimate tensile strain in the FRP reinforcement, ε_{frp}^{tu} = 0.0167

Strength in the FRP reinforcement at 1.67% strain, P_{frp}= 3.57 kips/in

Shear modulus of the adhesive, G_a = 160 ksi

Glass transition temperature = 165°F

New nominal loads for Strength I Limit State: M_{DC}= 540 kip-ft, M_{DW} = 50 kip-ft, M_{L+I}=1075 kip-ft

New nominal loads for fatigue limit state: M_{L+I}= 550 kip-ft

Factored shear force at the reinforcement end-termination, V_u= 150 kips

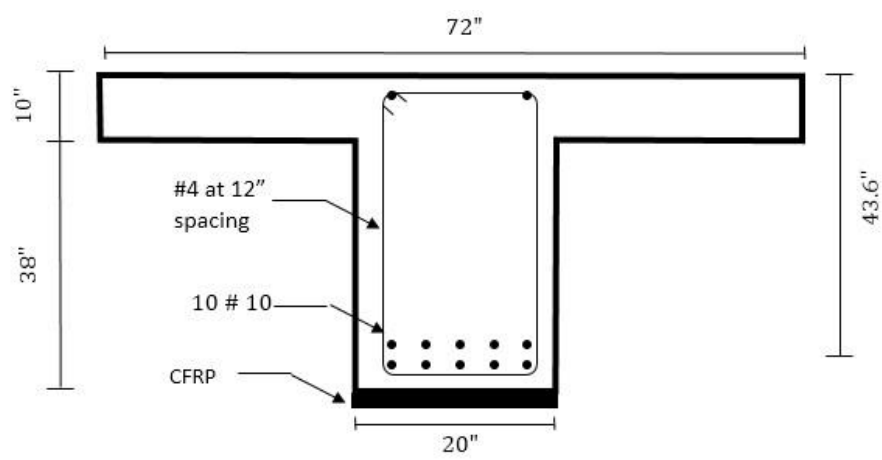

Figure B1 – Beam section

Step 1:

Determine if the FRP reinforcement material is in compliance with AASHTO Section 2.2.4.1, which specifies that the glass transition temperature must be higher than the maximum design temperature by 40°F.

The maximum design temperature, $T_{\text{Max Design}}$, is determined from Article 3.12.2.2 of the AASHTO LRFD Bridge Design Specifications for the location of the bridge (Michigan):

$T_{\text{Max Design}}$ = 105°F

$T_{\text{Max Design}}$ + 40°F = 105°F + 40°F = 145°F < Tg = 165°F

Step 2:

Establish the linear stress-strain relationship of the FRP reinforcement based on the design assumptions specified in article of 3.2 of the AASHTO and compute the tensile strength according to a strain value of 0.005.

$N_b = \frac{0.005}{0.0167}$ (3.57) = 1.07 kip / in

Step 3:

Neglecting the possible contribution of steel in the compression zone to flexure strength, the depth of the concrete compressive stress block is:

$a = \frac{As\ fy}{0.85\, f'c\ be} = \frac{12.7 \times 60}{0.85 \times 4 \times 72} = 3.11$ in

Since $a < h_f = 10$in, the stress block is within the flange and calculation is correct.

The depth of neutral axis $c = \frac{a}{\beta 1} = \frac{3.11}{0.85} = 3.66$ in

Strain in the tension steel is:

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

$\varepsilon_s = \frac{43.6-3.66}{3.66} \times 0.003 = 0.033$

Since $\varepsilon_s = 0.033 > \frac{f_y}{E_s} = \frac{60}{29000} = 0.0021$, the assumption that the tension steel yielded is correct.

$M_n = A_s f_y (d - \frac{a}{2})$

$M_n = 12.70 \times 60 (43.6 - \frac{3.11}{2}) = 32038$ kip- in

since $\varepsilon_t \geq 0.005$, phi = 0.9.

$\phi M_n = 0.9 \times 32038$ kips- in = 28834 kip-in = 2403 kip-ft

Step 4:

Determine if the existing beam requires strengthening.

The factored moment for Strength I limit state is:

M_u =1.25 M_{DC} + 1.5 M_{DW} + 1.75 M_{L+I} = 1.25 × 540 + 1.5 × 50 + 1.75 × 1075 = 5094 kip-ft = 2631 kip-ft

Since ϕMn < M_u, the beam requires strengthening.

Step 5:

For a preliminary estimate of the amount of FRP reinforcement necessary, the following approximation is used:

Tension force required of FRP, $T_{frp} \approx \frac{M_u - \phi M_n}{h} = \frac{(2631-2403)\times 12}{48}$ = 57 kips

Number of layers of FRP required, $n = \frac{T_{frp}}{N_b\, b_{frp}} = \frac{57}{1.07 \times 18}$ = 2.96

Try 3 layers, for which $T_{frp} = n \times N_b \times b_{frp} = 3 \times 1.07 \times 18$ = 57.7 kips

Step 6:

Compute the factored flexural resistance of the strengthened T-beam.

By trial and error, the depth of the neutral axis can be determined from strain compatibility and force equilibrium.

Guess c = 7.36 in

Strain in concrete = $\varepsilon_c = \frac{c}{h-c}\,\varepsilon_{frp}^{u} = \frac{7.36}{48-7.36} \times 0.005 = 9 \times 10^{-4}$ (where 0.005 is the assumed FRP limiting strain)

Modulus of concrete = $E_c = 57\sqrt{f'c} = 57\sqrt{4000}$ = 3605 ksi

Since ε_c < 0.003, use the parabolic concrete stress block model:

$\varepsilon_0 = \frac{1.71 \times f'_c}{E_c} = \quad = \frac{1.71 \times 4}{3605} = 0.0019$ (NCHRP 3.2)

$\frac{\varepsilon c}{\varepsilon 0} = \frac{9 \times 10^{-4}}{0.0019} = 0.48$

$\beta 2 = \frac{\text{Ln}\,[\,1 + (\varepsilon c/\varepsilon 0)^2\,]}{(\varepsilon c/\varepsilon 0)} = \frac{\text{Ln}\,[\,1+(0.48)^2]}{0.48} = 0.43$ (NCHRP 3.3)

Compression force in the concrete:

$C_c = 0.9\, f'_c\, \beta_2\, c\, b_e = 0.9 \times 4 \times 0.43 \times 7.36 \times 72$ = 820 kips

Strain in the steel:

$\varepsilon_c = \frac{d-c}{c}\ \varepsilon_c = \frac{43.6-7.36}{7.36} \times 9 \times 10^{-4} = 4.92 \times 10^{-3} > \varepsilon_y = \frac{fy}{Es} = \frac{60}{29000} = 0.00207$

Tension force in steel:

$T_s = A_s f_y = 12.7 \times 60 = 762$ kips

Tension force in the FRP reinforcement:

$T_{frp} = n \times N_b \times b_{frp} = 3 \times 1.07 \times 18 = 57.7$ kips

Total tension force:

$T = T_{frp} + T_s = 57.72 + 762 = 820$ kips

Since equilibrium of the force is satisfied ($T = 819 \approx C = 820$), the neutral axis position is correct.

If $T \neq C$, a new guess for the neutral axis is required.

Step 7:

Determine the initial strain resulting from the service dead load

Modular ratio $n_c = \frac{E_s}{E_c} = \frac{29000}{3605} = 8.04$

Assuming that the neutral axis lies within the flange of the T-section, the depth of the neutral axis can be computed from:

$Y_N = \frac{n\,As}{be}\left(-1 + \sqrt{1 + \frac{2\,d\,be}{n\,As}}\right) = \frac{8.04 \times 12.70}{72}\left(-1 + \sqrt{1 + \frac{2 \times 43.6 \times 72}{8.04 \times 12.70}}\right) = 9.79$ in

Since $Y_N < t_f$, the assumption is correct.

Cracked moment of inertia, $Icr = \frac{b\,h^3}{3} + n\ As\ (d - y_n)^2 = \frac{72 \times 9.79^3}{3} + 8.04 \times 12.70\ (43.6 - 9.79)^2 = 139241\ in^4$

Initial tensile stress at the bottom concrete surface, $\sigma_{bo} = \frac{Md\,(h-yn)}{I\,cr} = \frac{590 \times 12\,(48-9.79)}{139241} = 1.94$ ksi

At the time of installation of the externally bonded FRP, the dead load initial strain at the bottom surface of the beam is: $\varepsilon_{bo} = \frac{\sigma_{bo}}{E_c} = \frac{1.94}{3605} = 5.39 \times 10^{-4}$

Step 8:

Calculate resistance factor and the design moment capacity

$M_r = \phi\ ([\ A_s f_s\ (ds - k_2\ C\ ] + \phi_{frp}\ (h - k_2 C)\ )$

With $k_2 = 1 - \frac{2\left[\left(\frac{\varepsilon c}{\varepsilon 0}\right) - \arctan\left(\frac{\varepsilon c}{\varepsilon 0}\right)\right]}{\beta 2\left(\frac{\varepsilon c}{\varepsilon 0}\right)^2} = 1 - \frac{2[(0.48) - \arctan(0.48)]}{0.431 \times (0.48)^2} = 0.35$ (NCHRP 3.5)

Per recommendations in Section 8.1.3.4:

$$\begin{bmatrix} \phi = 0.9 & \text{for} & (\varepsilon_{frp}^{u} \geq 2.5\,\varepsilon_{frp}^{y} \text{ and } \mathcal{E}_t \geq 0.005 \\ \phi = \min \begin{bmatrix} 0.65 + \left(\frac{0.25}{1.5}\right)(\varepsilon_{frp}^{u} - 1) & \text{for} & \varepsilon_{frp}^{y} < \varepsilon_{frp}^{u} < 2.5\varepsilon_{frp}^{y} \\ 0.65 + \left(\frac{0.25\,(\mathcal{E}_t - \varepsilon_s^{y})}{0.005 - \varepsilon_s^{y}}\right) & \text{for} & \varepsilon_s^{y} < \mathcal{E}_t < 0.005 \end{bmatrix} & & \\ \phi = 0.65 & \text{for} & (\varepsilon_{frp}^{u} \leq \varepsilon_{frp}^{y} \text{ or } \mathcal{E}t \leq \varepsilon_s^{y} \end{bmatrix}$$

Where strain in the FRP at steel yield is:

$$\varepsilon_{frp}^{y} = \frac{h-c}{d-c}(\varepsilon_s^{y}) - \varepsilon_{bo} = \varepsilon_{frp}^{y} = \frac{48-7.36}{43.6-7.36}(0.0021) - 5.39 \times 10^{-4} = 0.0018$$

The ratio of FRP ultimate strain to strain at steel yield is: $\frac{\varepsilon_{frp}^{u}}{\varepsilon_{frp}^{y}} = \frac{0.005}{0.0018} = 2.81$

This exceeds the minimum required value of 2.5 as specified by AASHTO. If this ratio < 2.5, the design is not valid.

Strain in the bottom layer of steel at ultimate capacity is: $\varepsilon_t = \frac{d_t - c}{h-c}(\varepsilon_{frp}^{u}) = \frac{45.5-7.36}{48-7.36}\,0.005 = 0.0047$

$$\phi = 0.65 + \left(\frac{0.25\,(\mathcal{E}_t - \varepsilon_s^{y})}{0.005 - \varepsilon_s^{y}}\right) = 0.65 + \left(\frac{0.25\,(0.0047 - 0.0021)}{0.005 - 0.0021}\right) = 0.87$$

Per recommendations in Section 8.1.3.4, the additional reduction factor to the FRP contribution to moment capacity is:

$$\begin{bmatrix} \phi_{frp} = 0.94 & \text{when } \phi = 0.9 \\ \phi_{frp} = 0.38\,\phi + 0.6 & \text{when } 0.65 < \phi < 0.9 \\ \phi_{frp} = 0.85 & \text{when } \phi = 0.65 \end{bmatrix}$$

$\phi_{frp} = 0.38\,\phi + 0.6 = 0.38 \times 0.87 + 0.6 = 0.932$

The final moment capacity of the strengthened section is:

$M_r = 0.87\,[12.7 \times 60\,(43.6 - 0.35 \times 7.36) + 0.932 \times 57.72 \times (48 - 0.35 \times 7.36)]$

= 29463 kip-in= 2445 kip-ft

M_r= 2455 kip-ft < Mu= 2631 kip-ft

Since $M_r < M_u$, the strength is insufficient. Solutions are to increase the strength of FRP, the number of layers, or width of FRP (as feasible).

Increasing the number of layers to n=7, following the above procedure, the location of neutral axis and M_r can be found as:

c=7.71 inch and M_r= 2683 kip-ft > Mu= 2631 kip-ft, which is sufficient capacity.

Step 9:

Compute required development length.

Development length required: $L_d = \frac{T_{frp}}{\tau_{int}\, b_{frp}} = \frac{134.68}{0.065 \times \sqrt{4} \times 18} = 57.56$ in= 4.8 ft

Step 10:

Check fatigue limit state.

For the fatigue load combination: 0.75 M_{L+I} = 0.75 × 550 = 412.5 kip-ft = 4950 kip-in

Cracking moment, $M_{cr} = f_r \frac{I_g}{y_t}$

where:

$f_r = 0.24 \sqrt{f'_c} = 0.24 \sqrt{4} = 0.48$ ksi

$I_g = 310417$ in^4

$y_t = 30.68$ in

$M_{cr} = 0.48 \frac{310417}{30.68} = 4857$ kip-in

M_{cr} = 4857 kip-in ≤ 4950 kip-in O.K

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

$\varepsilon_c = \frac{M_f\, z}{I_T\, E_{frp}} = \frac{(550)(12)(10.14)}{16616 \times 33000} = 1.22 \times 10^{-4} < 0.36 \frac{f'c}{Ec} = 0.36 \frac{4}{3605} = 4 \times 10^{-4}$

$\varepsilon_s = \frac{M_f\,(d - z)}{I_T\, E_{frp}} = \frac{(550)(12)(43.6-10.14)}{16616 \times 33000} = 4 \times 10^{-4} < 0.8\, \varepsilon_y = 0.8 \times 0.0021 = 1.66 \times 10^{-3}$

$\varepsilon_{frp} = \frac{M_f\,(h + n\, t_{frp} - z)}{I_T\, E_{frp}} = \frac{(550)(12)\,(48 + 7 \times 0.0065 - 10.14)}{16616 \times 33000} = 4.56 \times 10^{-4} < \eta\ \varepsilon_{frp}^{u} = 0.8\ (0.0167) =$

0.013

Step 11:

Check reinforcement end termination peeling

M_u = 810 kip- ft

V_u = 150 kips

Shear modulus of FRP, $G_a = \frac{E_a}{2\,(1+\upsilon)} = \frac{440}{2\,(1+0.35)}$ = 163 ksi

Moment of inertia of beam section including FRP (use transformed section), I_T = 16616 in^4

Peeling stress, $f_{peel} = \tau_{av}\ [\ (\frac{3Ea}{E_{frp}})\ \frac{t_{frp}}{t_a}]^{1/4}$

where:

Average shear stress at FRP/concrete interface, $\tau_{av} = [\ V_u + (\frac{G_a}{E_{frp}\ t_{frp}\ t_a})^{\frac{1}{2}}\ M_u\ ]\ \frac{t_{frp}(h-z)}{I_T}$

$\tau_{av} = [\ 150 + (\frac{163}{33000 \times 7 \times 0.0065\ \times 0.020})^{\frac{1}{2}} \times 810 \times 12\ ]\ \frac{7 \times 0.0065 \times (\ 48-10.14\ )}{16616}$ = 2.36 ksi

$f_{peel} = 1.83\ [\ (\frac{3 \times 440}{33000})\ \frac{7 \times 0.0065}{0.02}]^{1/4}$ = 1.29 ksi $> \ \ 0.065\sqrt{4} = 0.13$

Since f_{peel} > limit, mechanical anchors at the FRP reinforcement is required.

B2 Flexural strengthening of a simply supported prestressed concrete girder

This example illustrates the flexural strengthening of a PC I-beam with an externally bonded CFRP system to accommodate higher loading.

Geometry and material properties:

Concrete compressive strength of the deck, $f'_{cd} = 4$ ksi

Modulus of elasticity, $E_c = 57\sqrt{f'_c} = 57\sqrt{4000} = 3605$ ksi

$\beta_1 = 0.85$ (for $f'_c \leq 4.0$ ksi)

New nominal loads for Strength I Limit State:

M_{DC} = 1700 kip-ft , M_{DW} = 200 kip-ft and M_{L+I} = 1525 kip-ft

Precast Beam (AASHTO-Type IV):

Concrete compressive strength, $f'_c = 5$ ksi

Pre-tensioning Strands:

Area of one tendon, A_{ps}= 0.153 in^2

Diameter = 0.5 in

Total area of the 26 strands, $A_{ps} = 3.98\ in^2$

Ultimate stress, $f_{pu} = 270$ ksi

Yield strength, $f_{py} = 0.9\ f_{pu} = 243$ ksi

Modulus of elasticity, $E_p = 28500$ ksi

Initial pre-tensioning at service limit state, $f_{pe} = 0.8 f_{py} = 194$ ksi

Factor related to the type of strands, $k = 2\ (1.04 - \frac{f_{py}}{f_{pu}}) = 0.28$ for low relaxation strands (LRFD 5.7.3.1.1-2)

FRP Reinforcement:

Shop-fabricated carbon fiber/ epoxy composite plates

Plate thickness, t_f = 0.039 in

Tensile strain in the FRP reinforcement at failure, ε_{frp}^{tu}= 0.013

Tensile strength in the FRP reinforcement at 1 % strain, P_{frp} = 9.3 kips / in

Glass transition temperature, Tg = 165°F

Geometric Properties:

Total height including deck slab, h_T = 64 in

Flange thickness, h_f = 9 in

Effective width of the Flange, b_{eff} = 96.38 in

Internal shear reinforcement = #3 at 12 in spacing

The distance from the extreme compression fiber to the center of gravity of the strands at the midspan, d_p = 57.62 in

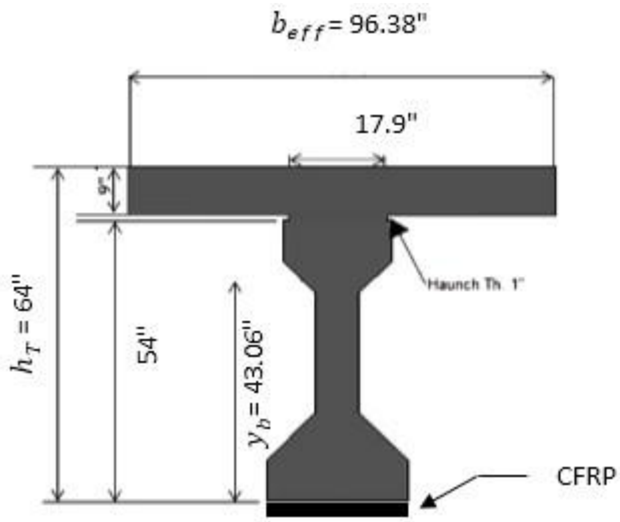

Figure B2 – Beam section

Step 1:

Determine if the FRP reinforcement material is in compliance with AASHTO Section 2.2.4.1, which specifies that the glass transition temperature must be higher than the maximum design temperature by 40°F.

The maximum design temperature, $T_{\text{Max Design}}$, is determined from Article 3.12.2.2 of the AASHTO LRFD Bridge Design Specifications for the location of the bridge (Michigan):

$T_{\text{Max Design}}$ = 105°F

$T_{\text{Max Design}}$ + 40°F = 105°F + 40°F = 145°F < Tg = 165°F

Step 2:

Neglecting the possible contribution of steel in the compression zone to flexural strength, the depth of the neutral axis is:

$$c = \frac{A_{ps}f_{pu} + A_s f_y}{0.85 f'_c \beta_1 b + k A_{ps}\left(\frac{f_{pu}}{d_p}\right)} \qquad \text{(LRFD 5.7.3.1.1-4)}$$

$$c = \frac{(3.98)(270)+0}{(0.85)(4)(0.85)(96.38)+(0.28)(3.98)\left(\frac{270}{57.62}\right)} = 3.79 \text{ in}$$

The depth of concrete compressive block: $a = \beta_1 c = (0.85)(3.79) = 3.22$ in

Since a < h_f, the stress block is within the flange and the calculation of a is correct.

$M_n = A_{ps} f_{ps} (d_p - \frac{a}{2})$ (LRFD 5.7.3.2.2-1)

The above equation is simplified from (LRFD 5.7.3.2.2-1) because no compression reinforcement or mild tension reinforcement is considered and the section behaves as a rectangular section.

$f_{ps} = f_{pu} (1 - k \frac{c}{d_p})$ (LRFD 5.7.3.1.1-1)

$f_{ps} = 270 (1 - 0.28 \frac{3.79}{57.62}) = 265$ ksi

$M_n = 3.98 \times 265 \times (57.62 - \frac{3.22}{2}) = 59074$ kip-in = 4923 kip-ft

$\phi M_n = 1 \times 59074$ kip-in = 59074 kip-in = 4923 kip-ft

Step 3:

Determine if the beam requires strengthening.

The factored moment for Strength I limit state is:

$Mu = 1.25\ M_{DC} + 1.5\ M_{DW} + 1.75\ M_{L+I} = 1.25 \times 1700 + 1.5 \times 200 + 1.75 \times 1525 = 5094$ kip-ft

= 61128 kip- in

Since $\phi M_n < Mu$, the beam requires strengthening.

Step 4:

Determine the initial strain resulting from the service dead load per recommendations in Section 5.2.2.3:

$\varepsilon_{bo} = \frac{-p_e}{E_c A_{cg}} (1 + \frac{e\, y_b}{r^2}) + \frac{M_{DL}\, y_b}{E_c I_g}$

where:

Radius of gyration, r = 22.04 in

Effective prestressing strain, $p_e = A_{ps} f_{pe} = 3.98 \times 194 = 773$ ksi

Eccentricity of prestressing force, e = 36.68 in

Gross moment of inertia, $I_g = 768283\ in^4$

Cross-sectional area, $A_{cg} = 1674$

Distance from extreme bottom fiber to the section centroid, $y_b = 43.06$ in

$\varepsilon_{bo} = \frac{-773}{3605 \times 1674.28} (1 + \frac{36.68 \times 43.06}{22.04^2}) + \frac{1700 \times 12 \times 43.06}{3605 \times 768283} = -2.3 \times 10^{-4}$

Step 5:

Establish the linear stress-strain relationship of the FRP reinforcement based on the design assumptions specified in article 3.2 of AASHTO and compute the tensile strength according to a strain value of 0.005.

$N_b = \frac{0.005}{0.01}$ (9.3) = 4.65 kip / in

Step 6:

Compute the factored flexural resistance of the strengthened I-beam. By trial and error the depth of the neutral axis can be determined from strain compatibility and force equilibrium.

Guess c = 12.22 in

Strain in concrete, $\varepsilon_c = \frac{c}{h-c}\, \varepsilon^{u}_{frp} = \frac{12.22}{64-12.22} \times 0.005 = 0.0012$ (where 0.005 is the assumed FRP limiting strain).

Since $\varepsilon_c < 0.003$, use the parabolic concrete stress block model:

$\varepsilon_0 = \frac{(1.71 \times f'c)}{Ec} = \frac{1.71 \times 4}{3605} = 0.0019$ (NCHRP 3.2)

$\frac{\varepsilon c}{\varepsilon 0} = \frac{0.0012}{0.0019} = 0.64$

$\beta 2 = \frac{\mathrm{Ln}\,[\,1+(\varepsilon c/\varepsilon 0)^2\,]}{(\varepsilon c/\varepsilon 0)} = \frac{\mathrm{Ln}\,[\,1+(0.64)^2\,]}{0.64} = 0.54$ (NCHRP 3.3)

Compression force in the concrete:

$C_c = 0.9\, f'_c\, \beta 2\; c\; b_e + kA_{ps}\left(\frac{f_{pu}}{d_p}\right)$

$Cc = 0.9 \times 4 \times 0.54 \times 9 \times 96.38 + 0.9 \times 5 \times 0.54 \times 3.22 \times 20 + 0.28 \times 3.98 \times \left(\frac{270}{57.62}\right) = 1855$ kips

Tension force in the FRP reinforcement:

Guided by step 5 in Example 1 and through trial and error, it is found that 7 plies are required to accommodate the ultimate moment.

$T_{frp} = n \times N_b \times b_{frp} = 7 \times 4.65 \times 24 = 781$ kips

Tension force in the prestressed steel:

$T_{ps} = A_{ps} f_{pu} = 3.98 \times 270 = 1074$ kips

Total tension force:

$T = T_{frp} + T_{ps} = 781 + 1074 = 1855$ kips

Since equilibrium of the force is satisfied (T = 1855 = C = 1855), the neutral axis position is correct. If T ≠ C, a new guess for the neutral axis is required.

Step 7:

Calculate resistance factor and the design moment capacity per recommendations in Section 8.1.3.4:

$$\begin{bmatrix} \phi = 0.9 & \text{for} & (\varepsilon_{frp}^{u} \geq 2.5\, \varepsilon_{frp}^{y} \text{ and } \mathcal{E}_{ps} \geq 0.013 \\ \phi = \min \begin{bmatrix} 0.65 + \left(\frac{0.25}{1.5}\right)(\varepsilon_{frp}^{u} - 1) & \text{for} & \varepsilon_{frp}^{y} < \varepsilon_{frp}^{u} < 2.5\varepsilon_{frp}^{y} \\ 0.65 + \left(\frac{0.25\,(\mathcal{E}_{ps} - 0.010)}{0.013 - 0.010}\right) & \text{for} & 0.010 < \mathcal{E}_{ps} < 0.013 \end{bmatrix} \\ \phi = 0.65 & \text{for} & (\varepsilon_{frp}^{u} \leq \varepsilon_{frp}^{y} \text{ or } \mathcal{E}ps \leq 0.010 \end{bmatrix}$$

where:

$\varepsilon_{ps} = \varepsilon_{pe} + \frac{p_e}{A_c E_c}\left(1 + \frac{e^2}{r^2}\right) + \varepsilon_{pnet} \leq 0.035$

$\varepsilon_{pe} = \frac{f_{pe}}{E_p} = \frac{194}{28500} = 0.0068$

$\varepsilon_{pnet} = \frac{d_p - c}{h - c}\, \varepsilon_{frp}^{u} = \frac{57.62 - 12.54}{64 - 12.54} \times 0.005 = 0.0044$

$\varepsilon_{ps} = 0.0068 + \frac{773}{1674 \times 3605}\left(1 + \frac{36.68^2}{22.04^2}\right) + 0.0044 = 0.0117$

The strain in the FRP at the point where the prestressing steel yields is:

$\varepsilon_{frp}^{y} = \frac{h - c}{d_p - c}\varepsilon_{ps}^{y} - \varepsilon_{bo}$

$\varepsilon_{ps}^{y} = \frac{f_{ps}}{E_{ps}} = \frac{243}{28500} = 0.0085$

$\varepsilon_{frp}^{y} = \frac{64 - 12.54}{57.62 - 12.54} \times 0.0085 - (-2.3 \times 10^{-4}) = 0.0099$

$\varepsilon_{frp}^{u} = 0.005 \leq \varepsilon_{frp}^{y} = 0.0099$. Thus, $\phi = 0.65$ and $\phi_{frp} = 0.85$

The final moment capacity of the strengthened section per recommendations in Section 5.1.4 is:

$M_r = \phi\,([A_{ps} f_{ps}\,(dp - k_2\, c] + \phi_{frp}\, T_{frp}\,(h - k_2\, c))$

where:

$k_2 = 1 - \frac{2[\left(\frac{\varepsilon c}{\varepsilon 0}\right) - \arctan\left(\frac{\varepsilon c}{\varepsilon 0}\right)]}{\beta 2\left(\frac{\varepsilon c}{\varepsilon 0}\right)^2} = 1 - \frac{2[\,0.64 - \arctan\,(0.64)]}{0.54\ (0.64)^2} = 0.36$ (NCHRP 3.5)

$f_{ps} = f_{pu}\left(1 - k\frac{c}{f_{pu}}\right) = 270\left(1 - 0.28\,\frac{12.54}{270}\right) = 266$ ksi

$M_r = 0.65 ([3.98 \times 266 \times (57.62 – 0.36 \times 12.54] + 0.85 \times 781\times (64 - 0.36\times 12.54)) = 62329$ kip-in = 5194 kip-ft

Mr = 5194 kip-ft > Mu = 5094 kip-ft O.K

B3 Shear strengthening of a prestressed concrete beam using 2-sided wrap

This example illustrates the shear strengthening of a PC I-beam with a 2-sided CFRP system. Note that the process and results are identical if a 3-sided (U-wrap) system that otherwise has the same configuration, is applied.

General Properties:

Bridge span = 42 feet

Concrete compressive strength of the deck, $f'_{cd} = 4$ ksi

Modulus of elasticity, $E_c = 57\sqrt{f'_c} = 57\sqrt{4000} = 3605$ ksi

$\beta_1 = 0.85$ (for $f'_c \leq 4.0$ ksi)

Factored shear force to resist, $V_u = 150$ kips

Precast Beam (AASHTO-Type IV):

Concrete compressive strength, $f'_c = 5$ ksi

Pre-tensioning Strands:

Area of one tendon, $A_{ps} = 0.153\ \text{in}^2$

Diameter = 0.5 in

Total area of the 26 strands, $A_{ps} = 3.98\ \text{in}^2$

Ultimate stress, $f_{pu} = 270$ ksi

Modulus of elasticity, $E_p = 28500$ ksi

Factor related to the type of strands, $k = 2\left(1.04 - \frac{f_{py}}{f_{pu}}\right) = 0.28$ for low relaxation strands (LRFD 5.7.3.1.1-2)

Internal Steel Shear Reinforcement:

Yield strength, $f_y = 60$ ksi

Modulus of elasticity, $E_s = 29000$ ksi

FRP Reinforcement:

Thickness, $t_f = 0.0065$ in

Failure strength, $f_{fu} = 550$ ksi

Modulus of elasticity, $E_f = 33000$ ksi

Failure strain, $\varepsilon_{fu} = 0.0167$ inch/inch

Geometric Properties:

Total height including deck slab, $h_T = 64$ in

Flange thickness, h_f = 9 in

Width of the web, b_v = 8 in

Effective width of the Flange, b_{eff} = 96.38 in

Internal shear reinforcement = #3 at 12 in spacing

Av = 0.22 in^2 s_v = 12 in α = 90 deg

Distance from the center of gravity of strands to the bottom fiber of the beam, Y_{bs} = 6.38 in

The distance from the extreme compression fiber to the center of gravity of the strands at the midspan, dp = 57.62 in

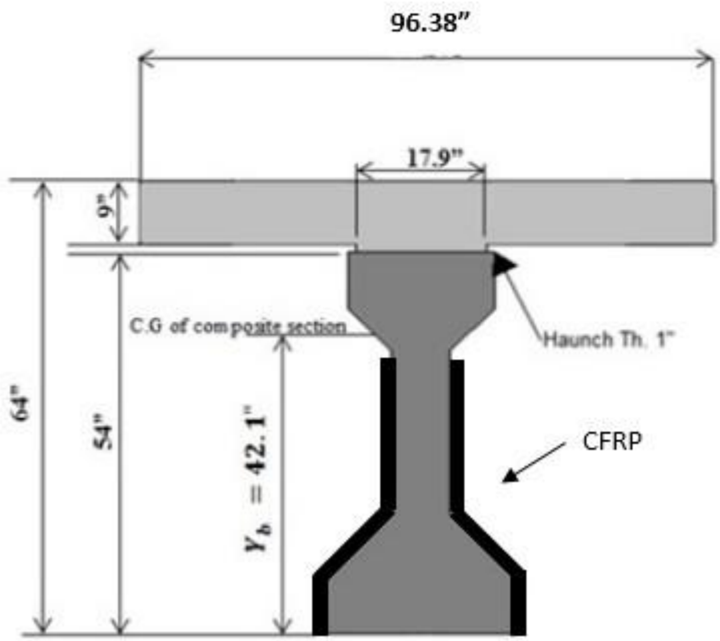

Figure B3a - Beam section

Step 1:

Determine the nominal shear resistance.

The effective shear depth, dv, is taken as the distance, measured perpendicular to the neutral axis, between the resultants of the tensile and compressive forces due to flexure; it need not be taken to be less than the greater of 0.9de or 0.72h (LRFD Article 5.8.2.9).

Neglecting the possible contribution of steel in the compression zone to flexural strength, the depth of the neutral axis is:

$$c = \frac{A_{ps}f_{pu} + A_s f_y}{0.85 f'_c \beta_1 b + kA_{ps}(\frac{f_{pu}}{d_p})}$$

$$c = \frac{(3.98)(270)+0}{(0.85)(4)(0.85)(96.38)+(0.28)(3.98)(\frac{270}{57.62})} = 3.79 \text{ in}$$

The depth of concrete compressive block: $a = \beta_1 c = (0.85)(3.79) = 3.22$ in

Since a < h_f, the stress block is within the flange and the calculation is correct.

d_e= effective depth from the extreme compression fiber to the centroid of the tensile force in the tension reinforcement = $h_c - Y_{bs} = 64 - 6.38 = 57.62$ in

Check governing case of d_v:

$d_{v1} = d_e - \frac{a}{2} = 57.62 - \left(\frac{3.22}{2}\right) = 56.01$ in

$d_{v2} = 0.9d_e = (0.9)(57.62) = 51.85$ in

$d_{v3} = 0.72h = (0.72)(64) = 46.08$ in

d_v = max (56.01 , 51.85 , 46.08) = 56.01 in

The nominal shear resistance provided by the concrete, V_c , is calculated in accordance with LRFD Eqn. 5.8.3.3-3 as:

$V_c = 0.0316\, \beta \sqrt{f_c'}\, b_v\, d_v$

For this example, the simplified method is followed (θ = 45 degrees and β=2). However, the iterative AASHTO LRFD Sectional Method could also be used if desired.

$V_c = 0.0316\,(2)(\sqrt{5})(8)(56.01) = 63$ kips

The nominal shear resistance provided by the internal steel reinforcement is :

$V_s = \frac{A_v f_y d_v (\cot\theta + \cot\alpha)\sin\alpha}{s} =$ (LRFD 5.8.3.3-4)

$\frac{(0.22)(60)(56.01)(1)(1)}{12} = 62$ kips

The nominal shear resistance provided by the vertical component of prestressing strands is V_p=0 (this example assumes straight strands; harped or draped strands will have a V_p component).

The nominal shear resistance of the member is:

$V_n = V_c + V_s + V_p = 63 + 62 + 0 = 125$ kips

Step 2:

Check whether strengthening is required.

Strength reduction factor for shear: ϕ = 0.9

$\phi V_n = 0.9(125) = 112.5$ kip

$\phi\ V_n$ = 112.5 kips < $V_{u\text{-crit}}$ = 150 kips. Therefore, the beam requires strengthening.

Step 3:

Selection of FRP strengthening scheme.

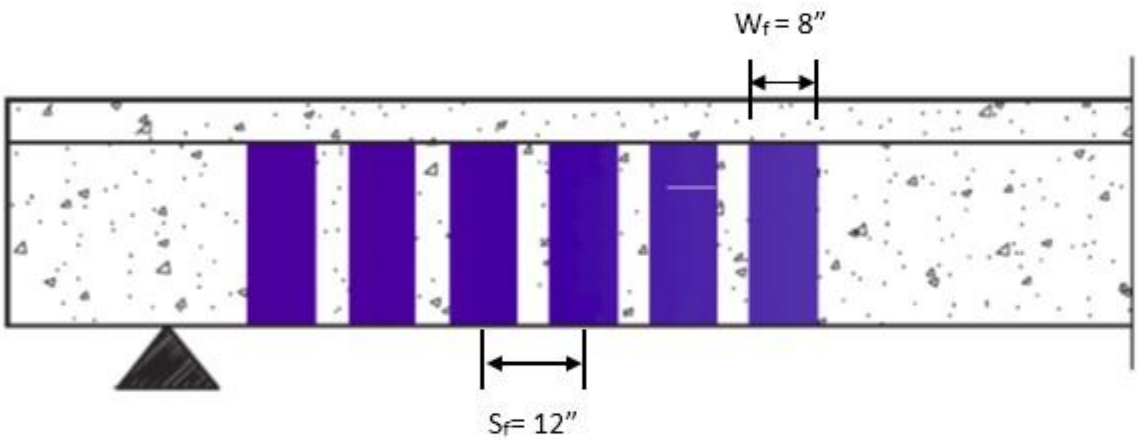

Figure B3b – FRP reinforcement scheme

A 2-sided (non-continuous) configuration is used without an anchorage system. The FRP sheets will be applied at 90 degrees with respect to the longitudinal axis of the girder. Note that the following calculation to determine FRP shear resistance for the 2-sided scheme is identical for a U-wrap scheme.

Assume one layer of FRP is sufficient	$n_f = 1$
Width of FRP sheets	W_f= 8 inch
Center-to-center spacing of FRP sheets	S_f = 12 inch
Orientation of FRP sheets	α_f = 90 degrees
Effective depth of FRP sheets	$d_f = d_p - h_f$

d_f= 57.62 - 9= 48.62 inch

Check if the selected spacing (12 in) is acceptable: Shear stress in concrete is:

$$V_u = \frac{V_{u-crit} - \phi\ Vp}{\phi\ .bv\ .dv} = \frac{150}{0.9 \times 8 \times 56.01} = 0.38 \text{ ksi} \qquad \text{(LRFD 5.8.2.9-1)}$$

Maximum spacing of FRP strips:

$$\text{Smax} = \begin{vmatrix} \min(0.8\text{ dv}, 24) & \text{if Vu} < 0.125\ f'\text{c} & \text{(LRFD 5.8.2.7} - 1) \\ \min(0.4\text{ dv}, 12) & \text{if Vu} \geq 0.125\text{ f'c} & \text{(LRFD 5.8.2.7} - 2) \end{vmatrix}$$

$0.125\ f'_c$ = 0.125 × 5 = 0.63 ksi

Since V_u = 0.38 ksi < 0.63 ksi, Smax = min (0.8 d_v , 24)

= min (0.8 × 56.01, 24)= 24 in 12 in < 24 in; the selected spacing is acceptable.

Step 4:

Determine FRP shear resistance, V_{frp}

The FRP reinforcement ratio is:

$\rho_f = \frac{2\ n_f t_f w_f}{b_v\ s_f}$ (AASHTO 4.3.2-2)

$= \frac{2 \times\ 1 \times 0.0065 \times 8}{8 \times 12}\ = 1.083 \times 10^{-3}$

The FRP strain reduction factor for side bonding or U-wrap without anchorage is:

$0.066 \leq R_f = 3\ (\rho_f \times E_f)^{-0.67} \leq 1.0$ (AASHTO 4.3.2-5)

$= 3 \times (1.083\ \times\ 10^{-3}\ \times 33000)^{-0.67} = 0.27$

The effective strain $\varepsilon_{fe} = R_f\ \varepsilon_{fu} \leq 0.004$

$\varepsilon_{fe} = 0.27 \times 0.0167 = 0.0046$

The effective strain is thus taken as 0.004.

$V_{frp} = \rho_f\ E_f\ \ \varepsilon_{fe}\ b_v\ d_f\ (\mathrm{Sin}(\alpha_f) + \mathrm{Cos}\ (\alpha_f))$ (AASHTO 4.3.2-1)

$V_{frp} = 1.083 \times 10^{-3} \times 33000 \times 0.004 \times 8 \times 48.62\ (\mathrm{Sin}\ (90) + \mathrm{Cos}\ (90)) = 56$ kips

Step 5:

Determine the design shear resistance of the member.

$\phi\ _{Vn\text{-}total} = \phi\ (V_c + V_p + V_s) + \phi\ _{frp}\ \ V_{frp}$ (AASHTO 4.3.1-1)

$= 0.9\ (63 + 0 + 62) + (0.85 \times 56) = 160$ kips

$\phi\ _{Vn\text{-}total} = 160$ kips $> V_{u\text{-}crit} = 150$ kips. Thus, one layer of FRP is sufficient.

Step 6:

Check maximum FRP shear reinforcement limitations.

$V_n \leq 0.25\ f'_c\ \ b_v\ d_v + V_p$ (AASHTO 5.8.3.3-2)

$V_n = V_c + V_s + V_{frp} = 63 + 62 + 56 = 181$ kips $\leq 0.25 \times 5 \times 8 \times 56.01 + 0 = 560$ kips

Thus, the web crushing failure limit is O.K.

B4 Shear strengthening of a T-beam using U-Wrap

This example illustrates the shear strengthening of a T-beam with a U-wrap system.

General Properties:

Concrete compressive strength, $f_c' = 3$ ksi

Modulus of elasticity $E_c = 57\sqrt{f_c'} = 57\sqrt{3000} = 3122$ ksi

$\beta_1 = 0.85$ (for $f_c' \leq 4.0$ ksi)

Factored shear force to resist, $V_u = 200$ kips

Internal Steel Shear Reinforcement:

Yield strength, $f_y = 60$ ksi

Modulus of elasticity, $E_s = 29000$ ksi

FRP Reinforcement:

Thickness, $t_f = 0.0065$ in

Failure strength, $f_{fu} = 550$ ksi

Modulus of elasticity, $E_f = 33000$ ksi

Failure strain, $\varepsilon_{fu} = 0.0167$ inch/inch

Geometric properties:

Beam height, $h_T = 48$ inch.

Width of the web, $b_v = 18$ inch

Effective flange width, $b_{eff} = 54$ in

Tensile reinforcement = 10#11, $A_s = 15.60$ in^2

Internal shear reinforcement = #4 at 12 in spacing

$Av = 0.40$ in^2 $\quad$ $s_v = 12$ in $\quad$ $\alpha = 90 \cdot \text{deg}$

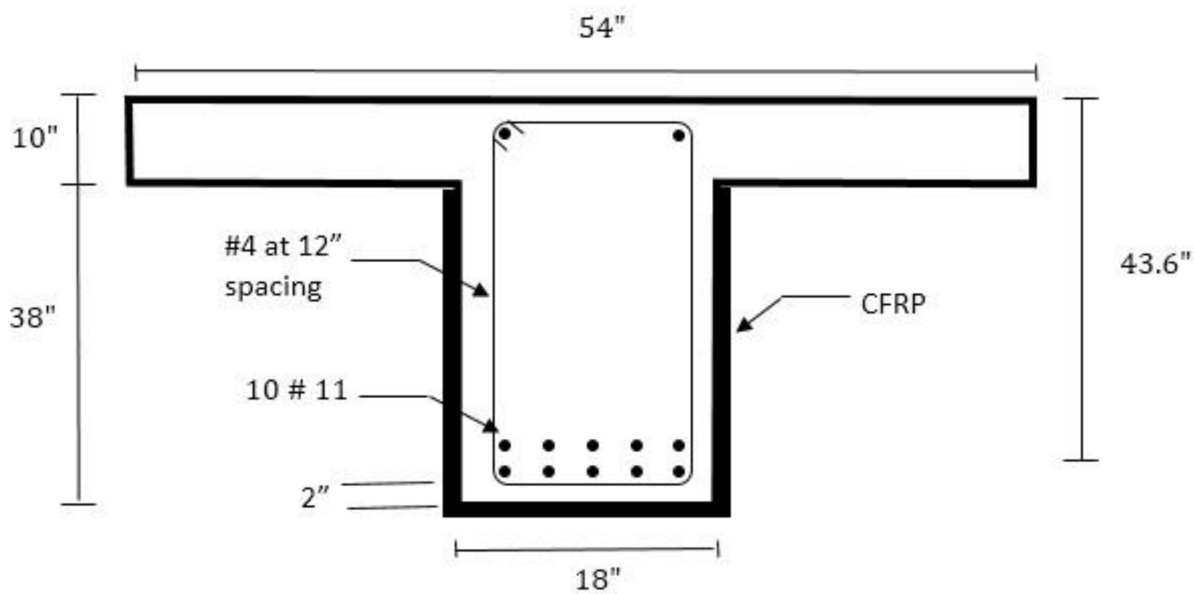

Figure B4 – Beam section

Step 1:

Determine the nominal shear resistance.

The effective shear depth, dv, is taken as the distance, measured perpendicular to the neutral axis, between the resultants of the tensile and compressive forces due to flexure; it need not be taken to be less than the greater of 0.9de or 0.72h (LRFD Article5.8.2.9).

Neglecting the possible contribution of steel in the compression zone to flexural strength, the depth of the neutral axis is:

$c = \frac{(As\ fy)}{(0.85\ \beta\ f'c\ b)} = \frac{(15.60 \times 60)}{(0.85 \times 0.85 \times 3 \times 54)} = 8.00$ in

$a = \beta_1\ c$

$a = 0.85 \times 8.00 = 6.80$ in

Check governing case of d_v:

$d_{v1} = d - \frac{a}{2} = 43.60 - \frac{6.80}{2} = 40.20$ in

$d_{v2} = 0.9d = 0.9 \times 43.60 = 39.24$ in

$d_{v3} = 0.72\ h_T = 0.72 \times 48 = 34.56$ in

$d_v = \max(d_{v1}, d_{v2}, d_{v3}) = \max(40.20, 39.24, 34.56) = 40.20$ in

The nominal shear resistance provided by the concrete, V_c, is calculated in accordance with LRFD Eqn. 5.8.3.3-3 as:

$V_c = 0.0316\ \beta \sqrt{f'_c}\ b_v\ d_v$ (LRFD 5.8.3.3-3)

For this example, the simplified method is followed: (θ = 45 degree) and (β=2). However, the iterative AASHTO LRFD Sectional Method could also be used if desired.

$$V_c = 0.0316\ (2)\left(\sqrt{3}\right)(18)(40.20) = 79 \text{ kips}$$

The nominal shear resistance provided by the internal steel reinforcement is :

$$V_s = \frac{A_v\ f_y\ d_v\ (\cot\theta + \cot\alpha)\sin\alpha}{s} = \qquad \text{(LRFD 5.8.3.3-4)}$$

$$\frac{(0.40)(60)(40.20)(1)(1)}{12} = 80 \text{ kips}$$

The nominal shear resistance provided by the vertical component of prestressing strands is V_p=0 ((this example assumes straight strands; harped or draped strands will have a V_p component).

The nominal shear resistance of the member is:

$$V_n = V_c + V_s + V_p = 79 + 80 + 0 = 159 \text{ kips}$$

Step 2:

Check whether strengthening is required.

Strength reduction factor for shear ϕ = 0.9

$$\phi V_n = 0.9(159) = 143 \text{ kip}$$

$\phi\ V_n$ = 143 kips < $V_{u\text{-}crit}$ = 200 kips. Therefore, the beam requires strengthening.

Step 3:

Selection of FRP strengthening scheme.

A U-wrap (continuous) configuration is used without an anchorage system at the ends of the sheets. The FRP sheets will be applied at 90 degrees with respect to the longitudinal axis of the girder.

Note that the following calculation to determine the FRP shear resistance for the U-Wrap without anchorage is identical for a side bonding scheme without anchorage.

Assume one layer of FRP is sufficient	n_f = 1
Orientation of FRP sheets	α_f = 90 degrees
Effective depth of FRP sheets	d_f = d - h_f = 33.60 inch

Step 4:

Determine FRP shear resistance, V_{frp}

The FRP reinforcement ratio is:

$\rho_f = \frac{2\ n_f t_f}{b_v}$ (AASHTO 4.3.2-2)

$= \frac{2 \times 1 \times 0.0065}{18} = 7.22 \times 10^{-4}$

FRP strain reduction factor for U-Wrap without anchorage or side bonding is:

$0.088 \leq R_f = 3\ (\rho f \times Ef)^{-0.67} \leq 1.0$ (AASHTO 4.3.2-5)

$4 \times (7.22 \times 10^{-4} \times 33000)^{-0.67} = 0.36$

The effective strain $\varepsilon_{fe} = R_f\ \varepsilon_{fu}$

$\varepsilon_{fe} = 0.36 \times 0.0167 = 0.006$

$V_{frp} = \rho_f\ \ E_f\ \ \varepsilon_{Fe}\ \ b_v\ d_f\ (Sin(\alpha_f) + Cos\ (\alpha_f))$ (AASHTO 4.3.2-1)

$V_{frp} = 7.22 \times 10^{-4} \times 33000 \times 0.006 \times 18 \times 33.60\ (Sin\ (90) + Cos\ (90)) = 86$ kips

Step 5:

Determine the design shear resistance of the member:

$\phi_{\ Vn\text{-}total} = \phi\ (V_c + V_p + V_s) + \phi_{\ frp}\ V_{frp}$ (AASHTO 4.3.1-1)

$= 0.9\ (79 + 0 + 80) + (0.85 \times 86) = 216$ kips

$\phi_{\ Vn\text{-}total} = 216$ kips $> V_{u\text{-}crit} = 200$ kips. Thus, one layer of FRP is sufficient.

Step 6:

Check maximum FRP shear reinforcement limitations

$V_n \leq 0.25\ f'_c\ .\ \ b_v\ \ .\ d_v + V_p$ (AASHTO 5.8.3.3-2)

$V_n = V_c + V_s + V_{frp} = 79 + 80 + 86 = 245$ kips $\leq 0.25 \times 3 \times 18 \times 40.20 + 0 = 542$ kips

Thus, the web crushing failure limit is O.K.

B5 Axial Strengthening of a confined circular column

This example illustrates the confinement strengthening of a circular column.

Column properties:

Column height = 24 ft

Column diameter= 28 in

Compressive concrete strength, $f'_c = 4$ ksi

Spiral spacing= 12 in

Vertical reinforcement, $A_{st} = 12$ # 8

$P_u = 1800$ kips

FRP Reinforcement:

Thickness, $t_f = 0.013$ in

Failure strength, $f_{fu} = 550$ ksi

Modulus of elasticity, $E_f = 33000$ ksi

Failure strain, $\varepsilon_{fu} = 0.0167$ inch/inch

Tensile strength of a single layer FRP reinforcement at 1.67% strain, $P_{frp}= 7.14$ kips/in/ply

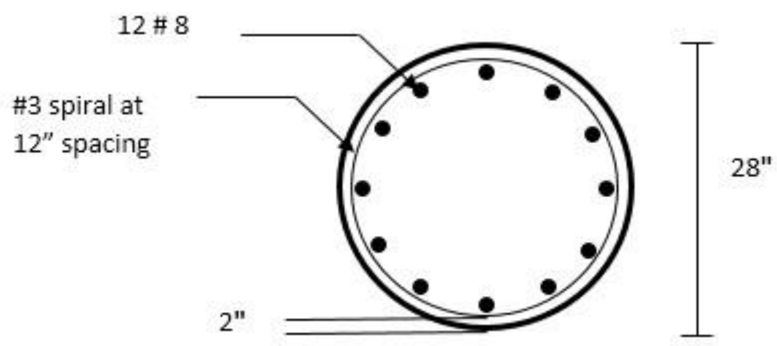

Figure B5 – Column section

Step 1:

Determine the axial strength of the column and check if it requires strengthening.

$A_g = \frac{\pi D^2}{4} = \frac{\pi \times 28^2}{4} = 615 \text{ in}^2$

$P_n = 0.85\,[0.85\, f'c\,(A_g - A_{st} - A_{ps}) + f_y A_{st} - A_{ps}\,(f_{pe} - E_p\, \varepsilon_{cu})]$ (LRFD 5.7.4.4-2)

P_n= 0.85 [0.85 × 4 (615- 9.48 - 0) + 60 × 9.48 - 0] = 2233 kips

P_r= ϕ P_n = 0.75 × 2233 = 1675 kips (LRFD 5.7.4.4-1)

P_r = 1675 kips < P_u= 1800 kips. Therefore, the column requires strengthening.

Step 2:

Compute the FRP reinforcement strength at a strain of 0.004. Note that the limit for confinement in AASHTO is different from that for flexure (see section 3.4.2.2.2 of this report).

$N_{frp}= \frac{0.004 \times 7.14}{0.0167} = 1.71$ kip/in

Step 3:

Determine the required confined concrete strength.

$P_r = 0.85\, \phi\, [0.85\, f'_{cc}\, (A_g - A_{st}) + f_y A_{st}] \geq P_u$ (AASHTO 5.3.1-1)

From which:

$$f'_{cc} \geq \frac{[(\frac{pu}{0.85\,\phi}) - \text{fyAst}]}{[0.85\,(\text{Ag} - \text{Ast})]} = \frac{[(\frac{1800}{0.85 \times 0.75}) - 60 \times 9.48]}{[0.85\,(615 - 9.48)]} = 4.38 \text{ ksi}$$

$f'_{cc} = f'c\,(1 + 2\,\frac{f_l}{f'c}) \geq 4.38$ ksi (AASHTO 5.3.2.2-1)

$= 4\,(1 + 2\,\frac{f_l}{4}) \geq 4.38$, therefore $f_l \geq 0.19$ ksi

As per article 5.3.2.2 of AASHTO, the confinement pressure shall be greater or equal to 600 psi but less than that specified in equation 5.3.3.3-2 as follows:

$f_l = 0.6 \text{ ksi} \leq (\frac{f'c}{2})(\frac{1}{k_e\ \phi} - 1) = (\frac{4}{2})(\frac{1}{0.85 \times 0.75} - 1) = 1.14$ ksi O.K.

$N_{frp} = \frac{f_l\, D}{2\,\phi_{frp}} = \frac{0.6 \times 28}{2 \times 0.65} = 12.92$ ksi/in (AASHTO 5.3.2.2-2)

Required number of plies: $n = \frac{N_{frp}}{N_{frpo}} = \frac{12.92}{1.71} = 7.56$

Try 8 layers. The column axial strength is computed as follows:

$f_l = \phi_{frp}\ \frac{2\,\text{Nfrp}}{\text{D}} = 0.65\,\frac{2\,(8)(1.71)}{28} = 0.64$ ksi

$f_l = 0.64 \leq (\frac{f'c}{2})(\frac{1}{k_e . \phi} - 1) = 1.137$ ksi O.K.

$f'_{cc} = f'c\,(1 + 2\,\frac{f_l}{f'c}) = 4\,(1 + 2\,\frac{0.64}{4}) = 5.28$ ksi

$P_r = 0.85 \, [\, 0.85 \, f'_{cc} \, (A_g - A_{st}) + f_y \, A_{st}] \geq P_u$

$= 0.85 \, [\, 0.85 \times 5.28 \, (615 - 9.48) + 60 \times 9.48 \,] = 2793$ ksi.

$P_r = \phi \, P_n = 0.75 \times 2793 = 2095$ kips > Pu = 1800 kips.

Thus, the selected number of layers are sufficient.

B6 Axial Strengthening of a square column

Column properties:

Column height = 24 ft

Column dimension = 28’’ by 28’’

Compressive concrete strength = 4 ksi

Transverse reinforcement = #3 bars at 12 in

Vertical reinforcement = 12 # 8 bars

P_u= 2100 kips

FRP Reinforcement:

Thickness, t_f = 0.013 in

Failure strength, f_{fu} = 550 ksi

Modulus of elasticity, E_f = 33000 ksi

Failure strain, ε_{fu} = 0.0167 inch/inch

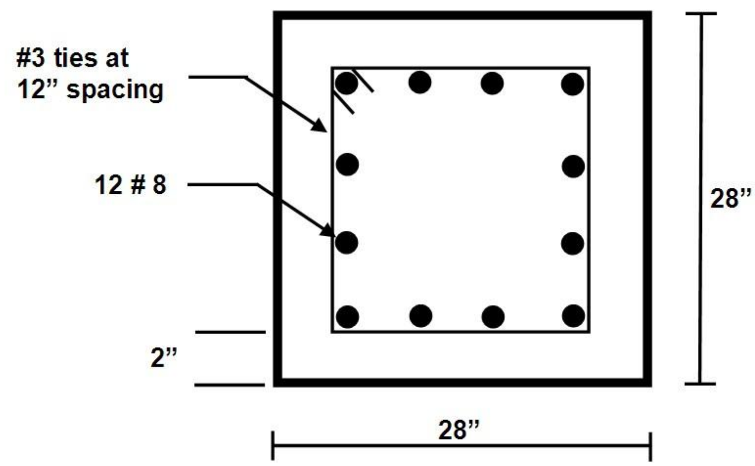

Figure B6 – Column section

Step 1:

Compute the axial strength of the column.

$P_n = 0.80\ [0.85\ f'c\ (A_g - A_{st} - A_{ps}) + f_y A_{st} - A_{ps}\ (f_{pe} - E_p\ \varepsilon_{cu})]$ (LRFD 5.7.4.4-3)

$P_n = 0.80\ [0.85 \times 4 \times (784 - 9.48 - 0) + 60 \times 9.48 - 0] = 2562$ kips

$P_r = \phi\ P_n = 0.75 \times 2562 = 1922$ kips (LRFD 5.7.4.4-1)

Step 2:

Compute the FRP reinforcement strength at a strain of 0.004. Note that the limit for confinement in AASHTO is different from that for flexure (see section 3.4.2.2.2 of this report).

$$N_{frp} = \frac{0.004 \times 7.14}{0.0167} = 1.71 \text{ kip/in}$$

Step 3:

Compute the required confined concrete strength.

$P_r = 0.80\,\phi\,[0.85\,f'_{cc}\,(A_g - A_{st}) + f_y A_{st}] \geq P_u$ (AASHTO 5.3.1-2)

From which:

$$f'_{cc} \geq \frac{[(\frac{pu}{0.80\,\phi}) - \text{fyAst}]}{[0.85\,(\text{Ag} - \text{Ast})]} = \frac{[(\frac{2100}{0.8 \times 0.75}) - 60 \times 9.48]}{[0.85\,(784 - 9.48)]} = 4.45 \text{ ksi}$$

$f'_{cc} = f'c\,(1 + 2\frac{f\,l}{f'c}) \geq 4.45$ ksi (AASHTO 5.3.2.2-1)

$= 4\,(1 + 2\frac{f\,l}{4}) \geq 4.45$, therefore $f_l \geq 0.23$ ksi

As per article 5.3.2.2 of AASHTO, the confinement pressure shall be greater or equal to 600 psi but less than that specified in equation 5.3.3.3-2 as follows:

$f\,l = 0.60 \text{ ksi} \leq (\frac{f'c}{2})(\frac{1}{k_{e\,.}\,\phi} - 1) = (\frac{4}{2})(\frac{1}{0.8 \times 0.75} - 1)) = 1.33$ ksi O.K.

$$N_{frp} = \frac{f_l\,D}{2\,\phi_{frp}} = \frac{0.6 \times 28}{2 \times 0.65} = 12.92 \text{ kip/ in}$$

Required number of plies: $n = \frac{N_{frp}}{N_{frpo}} = \frac{12.92}{1.71} = 7.56$

Try 8 layers. The column axial strength is computed as follows:

$$f_l = \phi_{frp}\,\frac{2\,\text{Nfrp}}{D} = 0.65\,\frac{2\,(8)(1.71)}{28} = 0.64 \text{ ksi}$$

$$f'_{cc} = f'c\,(1 + 2\frac{f\,l}{f'c}) = 4\,(1 + 2\frac{0.64}{4}) = 5.27 \text{ ksi}$$

$P_n = 0.8\,[0.85\,f'_{cc}\,(A_g - A_{st}) + f_y A_{st}] \geq P_u$

$= 0.8\,[0.85 \times 5.27\,(784 - 9.48) + 60 \times 9.48] = 3231$ ksi.

$P_r = \phi\,P_n = 0.75 \times 3231 = 2423 \text{ kips} > Pu = 2100$ kips

Thus, the selected number of layers are sufficient.

B7 Axial Strengthening of a confined circular column (ACI procedure)

Column properties:

Column height = 24 ft

Column diameter= 28 in

Compressive concrete strength, $f'_c = 4$ ksi

Spiral spacing= 12 in

Vertical reinforcement, $A_{st} = 12$ # 8

$P_u = 1800$ kips

FRP Reinforcement:

Thickness, $t_f = 0.013$ in

Failure strength, $f_{fu} = 550$ ksi

Modulus of elasticity, $E_f = 33000$ ksi

Failure strain, $\varepsilon_{fu} = 0.0167$ inch/inch

Tensile strength of a single layer FRP reinforcement at 1.67% strain, $P_{frp}= 7.14$ kips/in/ply

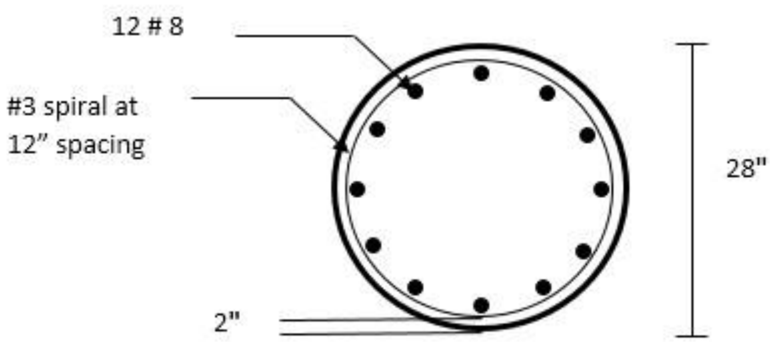

Figure B7 - Dimensions of the column

Step 1:

Determine the axial strength of the column.

$P_n= 0.85\ [0.85\ f'c\ (A_g - A_{st}) + f_y\ A_{st}]$

$P_n= 0.85\ [0.85 \times 4\ (615 - 9.48) + 60 \times 9.48] = 2233$ kips

$P_r= \phi\ P_n = 0.75 \times 2562 = 1675$ kips

$P_r = 1675$ kips $< P_u= 1800$ kips. Therefore, the column requires strengthening.

Step 2:

Compute the design FRP material properties.

$f_{fu} = C_E f_{fu}^*$

C_E for CFRP equals 0.85 for exterior exposure (bridges and piers).

f_{fu} = 0.85 × 550 = 468 ksi

ε_{fu} = $C_E \varepsilon_{fu}^*$ = 0.85 × 0.0167 = 0.0142

Step 3:

Determine the required maximum compressive strength of confined concrete, f'_{cc}.

$f'_{cc} = \frac{1}{0.85(\text{Ag}-\text{Ast})}\left(\frac{\phi\, P_{n,req}}{0.85\,\phi} - \text{fy Ast}\right)$

$f'_{cc} = \frac{1}{0.85(615-9.48)}\left(\frac{1800}{0.85 \times 0.75} - 60 \times 9.48\right)$ = 4.38 ksi

Step 4:

Determine the maximum confining pressure due to FRP jacket, f_l .

$f_l = \frac{f'_{cc} - f'_c}{0.95 \times 3.3 \times \kappa_a}$

κ_a and κ_b for circular cross section can be taken as 1.0.

$f_l = \frac{4.38-4}{0.95 \times 3.3 \times 1}$ = 0.12 ksi

Step 5:

Determine the required number of plies.

n = $\frac{f_l\, D}{2E_f t_f \varepsilon_{fe}}$

$\varepsilon_{fe} = \kappa_\varepsilon \varepsilon_{fu}$ = 0.55 × 0.0142 = 0.0078

n = $\frac{0.12 \times 28}{2 \times 33000 \times 0.013 \times 0.0078}$ = 0.50

Try 1 layer.

$f_l = \frac{2\,\text{n}E_f t_f \varepsilon_{fe}}{D} = \frac{2 \times 1 \times 33000 \times 0.013 \times 0.0078}{28}$ = 0.24 ksi

Checking the minimum confinement ratio:

$\frac{f_l}{f'_c} = \frac{0.24}{4}$ = 0.06 which is not equals or greater than 0.08.

Thus, try 2 layers:

$f_l = \frac{2\ nE_f t_f \varepsilon_{fe}}{D} = \frac{2\times2\times33000\times0.013\times0.0078}{28} = 0.48$

$\frac{f_l}{f'_c} = \frac{0.48}{4} = 0.12 \geq 0.08$ O.K.

Step 6:

Verify that the ultimate axial strain of the confined concrete, $\varepsilon_{ccu} \leq 0.01$.

$\varepsilon_{ccu} = \varepsilon'_c(1.5 + 12\ \kappa_b \frac{f_l}{f'_c} (\frac{\varepsilon_{fe}}{\varepsilon'_c})^{0.45}$

Where:

$\varepsilon'_c = \frac{1.71\ f'_c}{E_c} = \frac{1.71\times4}{3605} = 0.0019$

$\varepsilon_{ccu} = 0.0019\ (1.5 + 12 \times 1 \times 0.12\ (\frac{0.0078}{0.0019})^{0.45}\ ) = 0.008 < 0.01$ O.K.

Step 7:

Determine the column axial strength.

$f'_{cc} = f'_c + (0.95 \times\ 3.3\ \times\ \kappa_a \times f_l) = 4 + (0.95 \times 3.3 \times\ 1 \times\ 0.48) = 5.50$ ksi

$P_n = 0.8\ [0.85\ f'_{cc}\ (A_g - A_{st}) + f_y\ A_{st}]$

$= 0.85\ [0.85 \times 5.50\ (615 - 9.48) + 60 \times 9.48] = 2890$ kips

$P_r = \phi\ P_n = 0.75 \times 3289 = 2167$ kips ≥ 1800 kips

Thus, the selected number of layers are sufficient.

B8 Axial Strengthening of a confined square column (ACI procedure)

Column properties:

Column height = 24 ft

Column dimension = 28'' by 28''

Compressive concrete strength = 4 ksi

Transverse reinforcement = #3 bars at 12 in

Vertical reinforcement = 12 # 8 bars

P_u= 2100 kips

FRP Reinforcement:

Thickness, t_f = 0.013 in

Failure strength, f_{fu} = 550 ksi

Modulus of elasticity, E_f = 33000 ksi

Failure strain, ε_{fu} = 0.0167 inch/inch

Tensile strength of a single layer FRP reinforcement at 1.67% strain, P_{frp}= 7.14 kips/in/ply

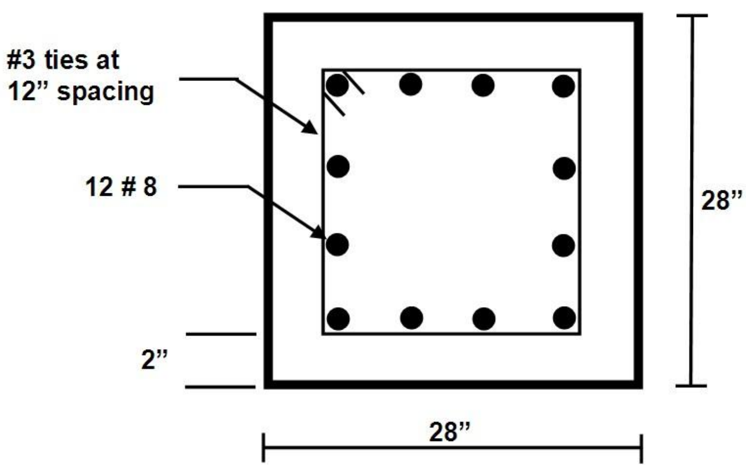

Figure B8 – Column section

Step 1:

Determine the axial strength of the column.

P_n= 0.80 [0.85 f 'c (A_g- A_{st}) + f_y A_{st}]

P_n= 0.80 [0.85 × 4 (784 – 9.48) + 60 × 9.48] = 2562 kips

P_r= ϕ P_n = 0.65 × 2562 = 1665 kips

P_r = 1665 kips < P_u= 2100 kips. Therefore, the column requires strengthening.

Step 2:

Compute the design FRP material properties.

$f_{fu} = C_E f^*_{fu}$

C_E = 0.85 for exterior exposure (bridges and piers).

f_{fu} = 0.85 × 550 = 468 ksi

$\varepsilon_{fu} = C_E \varepsilon^*_{fu}$ = 0.85 × 0.0167 = 0.0142

Step 3:

Determine the required maximum compressive strength of confined concrete, f'_{cc}.

$$f'_{cc} = \frac{1}{0.85(\text{Ag} - \text{Ast})}\left(\frac{\phi\, P_{n,req}}{0.80\,\phi} - \text{fy Ast}\right)$$

$$f'_{cc} = \frac{1}{0.85(784 - 9.48)}\left(\frac{2100}{0.80 \times 0.65} - 60 \times 9.48\right) = 5.27 \text{ ksi}$$

Step 4:

Determine the maximum confining pressure due to FRP jacket, f_l .

$$f_l = \frac{f'_{cc} - f'_c}{0.95 \times 3.3 \times k_a}$$

$$\kappa_a = \frac{A_e}{A_c}\left(\frac{b}{h}\right)^2$$

$$\frac{A_e}{A_c} = \frac{1 - \frac{\left[\left(\frac{b}{h}\right)(h-2r_c)^2 + \left(\frac{h}{b}\right)(b-2r_c)^2\right]}{3\,A_g} - \rho_g}{1-\rho_g} = \frac{1 - \frac{\left[\left(\frac{28}{28}\right)(28-2\times0.5)^2 + \left(\frac{28}{28}\right)(28-2\times0.5)^2\right]}{3\times784} - 0.012}{1-0.012} = 0.373$$

$$\kappa_a = \frac{A_e}{A_c}\left(\frac{b}{h}\right)^2 = 0.373\left(\frac{28}{28}\right)^2 = 0.373$$

$$f_l = \frac{5.27 - 4}{0.95 \times 3.3 \times 0.373} = 1.09$$

Step 5:

Determine the required number of plies.

$$n = \frac{f_l\sqrt{b^2+h^2}}{2E_f t_f \varepsilon_{fe}}$$

$$\varepsilon_{fe} = \kappa_\varepsilon \varepsilon_{fu} = 0.55 \times 0.0142 = 0.0078$$

$$n = \frac{1.09 \times \sqrt{28^2+28^2}}{2\times33000\times0.013\times0.0078} = 6.45$$

Try 7 layers.

$f_l = \frac{2\,nE_f t_f \varepsilon_{fe}}{\sqrt{b^2+h^2}} = \frac{2\times 7\times 33000\times 0.013\times 0.0078}{\sqrt{28^2+28^2}}$ = 1.18 ksi

Checking the minimum confinement ratio:

$\frac{f_l}{f'_c} = \frac{1.18}{4}$ = 0.295 ≥ 0.08 O.K.

Step 6:

Verify that the ultimate axial strain of the confined concrete, $\varepsilon_{ccu} \leq 0.01$.

$\varepsilon_{ccu} = \varepsilon'_c(1.5 + 12\ \kappa_b \frac{f_l}{f'_c} (\frac{\varepsilon_{fe}}{\varepsilon'_c})^{0.45}$

Where:

$\varepsilon'_c = \frac{1.71\, f'_c}{E_c} = \frac{1.71\times 4}{3605} = 0.0019$

$\kappa_b = \frac{A_e}{A_c}\ (\frac{h}{b})^{0.5} = 0.373\ (\frac{28}{28})^{0.5} = 0.373$

$\varepsilon_{ccu} = 0.0019\ (1.5 + 12 \times 0.373 \times 0.295\ (\frac{0.0078}{0.0019})^{0.45}\ ) = 0.0076 < 0.01$ O.K.

Step 7:

Determine the column axial strength.

$f'_{cc} = f'_c + (0.95 \times\ 3.3\ \times\ \kappa_a \times f_l) = 4 + (0.95 \times 3.3 \times\ 0.373 \times\ 1.18) = 5.38$ ksi

$P_n = 0.8\ [0.85\ f'_{cc}\ (A_g - A_{st}) + f_y\, A_{st}]$

$= 0.8\ [0.85 \times 5.38\ (784 – 9.48) + 60 \times 9.48] = 3289$ kips

$P_r = \phi\ P_n = 0.65 \times 3289 = 2137$ kips ≥ 2100 kips

Thus, the selected number of layers are sufficient.

APPENDIX C: INSPECTION CHECKLIST

General

- Are “As-Built” plans being updated to reflect field revisions?

Contractor submittal and training

- Has Contractor submitted Quality Control Plan and SDSs?
- Has Contractor submitted installing crew qualifications?
- Has Contractor submitted documentation of measuring and testing equipment current calibration?
- Are the site staff members properly trained and informed regarding technical inspection and testing requirements?
- Are the site staff members properly trained on emergency and accident procedures?

Materials qualifications/acceptance submittals

Are the following items completed:?

- Material properties test results based on a minimum of 10 samples per test.
- Fiber properties including tensile strength (determined per ASTM D3039 and not be less that 1%), modulus, and strain.
- The mean and coefficient of variation of the moisture equilibrium content, determined per ASTM D 5229, and are not greater than 2% and 10%, respectively.
- Epoxy properties to be evaluated include epoxy tensile strength, modulus, infrared spectrum analysis, glass transition temperature, gel time, pot life, and adhesive shear strength.
- The glass transition temperature is determined per ASTM D4065 and is at least 40^{o}F higher than the maximum design temperature (see Section 3.12.2.2 of the *AASHTO LRFD Bridge Design Specifications*).
- After conditioning as specified below, the glass transition temperature and tensile strain, is to retain 85% of the required values. The conditioning environments are:

    - ♦ Water: Samples are immersed in distilled water having a temperature of 100 ± 3°Fand tested after 1,000 hours of exposure.
    - ♦ Alternating UV and humidity: Samples are conditioned under Cycle 1-UV exposure per ASTM G154, and tested within two hours after removal.
    - ♦ Alkali: Samples are immersed in calcium hydroxide (pH ~11) at ambient temperature for 1000 hours prior to testing.
    - ♦ Freeze-thaw: Samples are exposed to 100 repeated cycles of freezing and thawing per ASTM C666.
- If impact tolerance is stipulated by the engineer, it is determined per ASTM D7136.

- Daily inspection during installation should include the following test measurements:
    - ♦ Ambient temperature; relative humidity, and general weather observations;

- ♦ Surface temperature of concrete;
- ♦ Surface dryness per ACI 503.4;
- ♦ Level of resin curing in accordance with ASTM D2582;
- ♦ Adhesion strength to exceed 200 psi;

- Are the submitted sample results from certified mill analyses and third-party labs?
- Have all materials met acceptance requirements?
- Have all materials not meeting acceptance requirements properly disposed?
- Specific minimum limits required by MDOT are included in Table 1:

  (Note: the recommendations in this report do not require any of the specific values given in Table 1, as long as the values listed above and all design requirements are met)

Table 1 – Strengthening system components minimum properties

Properties	Minimum Values	ASTM Test Method
Fiber		
Fiber tensile modulus	33,000,000 psi	D3039
Tensile strain at failure	1.3%	D4018
Composite sheet		
Ultimate tensile strength	3,300 lbs/in	D3039
Tensile strain at failure	1.3%	
Sheet stiffness	189,000 lbs/in	
Resin		
Tensile strength	9,000 psi	D 638
Elongation at break	4.0%	D 638
Modulus of elasticity 7 days	390,000 psi	D 638
Flexural strength	6,700 psi	D 790
Shear strength 14 days	3,500 psi	D 732
Deflection temperature	47 C	D 648
Water absorption	0.03%	

- Are the following design parameters clearly listed on the structural engineering submittals for a non-prestressed beam flexural strengthening project? Note: depending on specific design application, not all of the following parameters may be needed:

Concrete compressive strength of precast beam,
Modulus of elasticity of concrete,
Reinforcing steel yield strength,
Steel reinforcement area,
Modulus of elasticity of steel,
Internal shear reinforcement and its spacing, FRP Plate/sheet thickness,
Ultimate tensile strain in the FRP at failure,
Tensile strength in the FRP at ultimate strain,
Glass transition temperature,
Shear modulus of the adhesive,
Modulus of elasticity of fiber,
Total height of beam including deck slab,
Flange thickness,
Effective width of the flange,
Bridge span length,
Distance from extreme comp. fiber to steel centroid,
New nominal loads for fatigue limit state,
Shear force at reinforcement end-termination,
New load capacity needed

- In the case of prestressed concrete beam designed for flexural strengthening, the following parameters may be added to the above list:

Concrete compressive strength of the deck,
Area of prestress tendons,
Diameter of tendons,
Initial pre-tensioning at service limit state,
Yield strength,
Ultimate stress,
Type of strands,
Dist. from extreme comp. fiber to strand centroid.

- When designing for shear strengthening of a nonprestressed member, the following parameters may be needed:

Concrete compressive strength of precast beam,
Modulus of elasticity of concrete,
Reinforcing steel yield strength,
Steel reinforcement area,
Modulus of elasticity of steel,
Internal shear reinforcement and its spacing,
FRP plate/sheet thickness,
Ultimate tensile strain in the FRP at failure,
Tensile strength in the FRP at ultimate strain,
Modulus of elasticity of FRP,
Tensile strength of FRP at ultimate strain,
Orientation of FRP sheets,
Width of FRP sheets,
Center-to-center spacing of FRP sheets,
Total height of beam including deck slab,
Flange thickness,
Effective width of the Flange,
Bridge span length,
Dist. from extreme comp. fiber to steel centroid,
Effective depth of FRP sheets,
Width of the web,
New shear capacity needed

- When designing for shear strengthening of a prestressed member, the following parameters may be added to the above list:

Concrete compressive strength of the deck,
Area of prestress tendons,
Diameter of tendons,
Initial pre-tensioning at service limit state,
Yield strength of tendons,
Ultimate stress of tendons,
Type of strands,
Effective depth from extreme comp. fiber to centroid of reinforcement.

- Design parameters required for column wrapping (confinement) include:

Concrete compressive strength,
Reinforcing steel yield strength,
Steel reinforcement area,
Tie or spiral spacing,
Area of strands,
Initial pre-tensioning at service limit state,
Modulus of elasticity of strands,
FRP sheet thickness,
Ultimate tensile strain of the FRP
Modulus of elasticity of FRP
Tensile strength of FRP at ultimate strain,
Column geometry,
Column height,
Ultimate axial load to resist.

FRP Shipping, storage, and handling

- Do packages received include SDS?
- Were packages inspected upon delivery for damage?
- Are FRP systems stored in accordance with manufacturer's guidelines?
- Are catalysts and initiators stored separately?
- Are chemical components securely sealed per OSHA?
- Are flammable resins stored in accordance with fire regulations?
- Are expired materials disposed of in accordance with environmental control regulations?
- Are hazardous materials such as thermosetting resins properly labeled?
- Are SDSs accessible to all at project site? Are they read and understood by personnel handling hazardous materials?
- Is proper gear (suits, gloves, dust masks, respirators, etc.) available for handling resins, solvents and fiber materials?
- Is the resin mixing area well ventilated?
- Are waste materials disposed in accordance with environmental regulations?

Removal and restoration of defective surfaces prior to concrete placement

- Have the perimeters of existing spalls been identified and saw cut to a minimum depth of 0.75 in to prevent feathered edges?
- Are cracks within concrete greater than 0.01 in spaced closer than 1.5 in, and cracks wider than 1/32 in epoxy injected?
- After removal of all defective areas, did Contractor inspect and clean the substrate from any dust, laitance, grease, oil, curing compounds, wax, impregnations, foreign particles and other bond-inhibiting materials?
- Has all exposed steel been sandblasted clean prior to concrete placement?
- Did the Contractor apply a bonding and reinforcement protection to all exposed reinforcement and concrete surface prior to concrete placement?

Inspection of surface preparation prior to FRP application

- Have all inside and outside corners and sharp edges been rounded or chamfered to a minimum radius of 1/2 in?
- Is restored concrete surface smooth and uniform with a maximum out of plane deviation < 1/32 in?
- Are all voids with diameters larger than 1/2 in and depressions greater than 1/16 in filled and cured?
- Have all cracks in the concrete wider than 0.01 in spaced closer than 1.5 in and cracks wider than 1/32 in been filled using pressure injection of epoxy?
- Was the surface checked and cleaned of any dust, laitance, grease, oil, curing compounds, wax, impregnations, surface lubricants, paint coatings, stains, foreign particles, weathered layers and any other bond-inhibiting materials?

- Has the substrate concrete compressive strength checked to be 2.2 ksi or greater, and tensile strength to be 220 psi or greater?

Application conditions

- Is the ambient temperature and temperature of concrete surface within the range of 50°F-90°F?
- Are the contact surfaces completely dry at the time of installation of FRP system?
- Does the weather forecast predict dry conditions or potential rain? If rain is imminent, stop application till dry conditions are assured.

Installation of wet lay-up systems

- Are field data including temperature, surface condition, and relevant field observations being documented?
- Are witness panels prepared with a size of at least 300 – 775 in^2, but not less than 0.5% of the overall area to be strengthened?
- Is the resin mixed in quantities sufficiently small to ensure its use within manufacturer recommended pot life?
- Is the excess resin disposed of when exceeded its pot life, or when it begans to generate heat or show signs of increased viscosity?
- Are the ambient and concrete surface temperatures as specified in the contract drawings and recommended by the manufacturer?
- Is the excess primer disposed of when it exceeds its pot life?
- Is the putty, if necessary, applied as soon as the primer becomes tack-free or until non-sticky to the fingers?
- Are the surfaces of primer and putty protected from dust, moisture and other contaminants before applying the FRP system?
- Is the fiber sheet placed properly and pressed gently onto the wet saturant?
- Is any entrapped air between fiber sheet and concrete released?
- Is rolling conducted in the fiber direction for unidirectional fiber sheets?
- Is sufficient saturant applied on top of the fiber sheet as overcoat to fully saturate the fibers?
- Are lap splice lengths as specified in the contract drawings, but at least 8 in?
- Is there any deviation in fiber alignment more than 5°?
- Is the FRP system protected as necessary until it is fully cured?

Identification of defective work

- Are voids or air encapsulation (pockets) found between the concrete and the layers of primer, resin and/or adhesive, and within the composite itself? (per existing MDOT guidelines, voids with a size of 0.25 – 2.5 in^2 must be repaired).
- Are delaminations larger than 2 in^2 (1300 mm^2) detected (using acoustic sounding, ultrasonic, or thermography)? If more than 10 such delaminations were detected in 10 ft^2, were they repaired?

- Are there any wrinkling or buckling of fiber, fiber tows, or discontinuities due to fracture of the fibers?
- Are there any resin-starved areas or areas with non-uniform impregnation/wet-out?
- Are there any cracks, blisters or peeling of the surface coating?
- Is there any under-cured or incompletely cured polymer?
- Are there any incorrectly placed reinforcement configurations?

Post-installation quality control tests

Were the following QC tests completed?:

- Perform surface inspection for any swelling, bubbles, voids or delaminations after at least 24 hours of initial resin cure. If advanced equipment is unavailable, perform an acoustic tap test with a hard object to identify delaminated areas by sound. Mark all voids and assess the size of each to determine if repair is needed.
- Perform direct pull-off test according to ASTM D4541, ASTM D7234, or the method described by ACI 440.3R (2004), Test Method L.1. Successful tension adhesion strengths should exceed the greatest of 200 psi or 0.065 $\sqrt{f_c'}$ ksi, and exhibit failure of the concrete substrate.
- Repair areas after bonding tests according to the procedures established in the contract drawings and specifications.

Long-term maintenance inspections

Were the following inspections completed?:

- An annual general inspection, primarily visual in nature. In the annual inspection, the inspector looks for changes in color, signs of crazing, cracking, delamination/debonding, peeling, blistering, deflection, or evidence of other deterioration, in addition to local damage due to impact or surface abrasion.
- A detailed inspection at least once every 6 years, where the inspection attempts to more accurately quantify the performance and condition of the FRP system. Pull-off testing and test evaluation for debonding and FRP degradation are performed as part of detailed testing.

APPENDIX D: MODIFICATIONS TO EXISTING MDOT SPECIFICATIONS

The following text provides recommended changes to the MDOT Special Provisions for FRP Flexural and Shear Strengthening Systems and Column Wrapping. The recommendations that follow are not meant to be a specification. Rather, the purpose is to identify conflicts, if any, between the existing provisions and the recommendations presented in Chapter 8 of this report, as well as to fill any gaps in the existing provisions that may be addressed in this report. Issues in the existing specifications that are not directly addressed or contradicted by the recommendations of Chapter 8 are not altered.

Therefore, the first recommendation that applies to all existing documents is to incorporate the recommendations of Chapter 8. As inspection is not discussed in detail in the provisions, Chapter 8.2.4 of the report may be particularly useful. Some specific points are provided below.

D.1 MDOT Special Provision for Column Wrapping with FRP Sheets

1) Article b.1 (page 2): Revise as follows (refer to Report Section 8.2.1.1 for additional detail): *List of all materials and manufacturers, with material safety data sheets and* ***shipping****, storage and handling requirements.*

2) Article b.8 (page 2): Add the following (see Report Sections 8.2.4.1 and 8.2.5): *The contractor (installer) is required to submit a QA/QC plan. Manufacturer and contractor qualifications are to be verified in accordance with the provisions in NCHRP Report 609.*

3) Article c. (page 2). Revise as follows: *Apply FRP to the columns shown on the plans, in accordance to the number of layers specified. Each individual layer of FRP must meet the material requirements.*

4) Article c (page 3): The construction details provided can be supplemented by the description in Section 8.2.3 of the Report, especially with Article 8.2.3.1.

5) Article c (page 3): Replace the clause: "Repair delaminations larger than 0.25 square inches" with the Report recommendation in Section 8.2.5.4, which states the following: *For wet layup systems, the need for delamination repair depends on the size and number of delaminations. Small delaminations less than 2 in^2 (1300 mm^2) are permissible as long as the delaminated area is less than 5% of the total laminate area and there are no more than 10 such delaminations per 10 ft^2 (1 m^2). Delaminations exceeding these limits are to be repaired by either resin injection or ply replacement, depending on delamination size. Large delaminations, greater than 25 in^2 (16,000 mm^2), should be repaired by selectively cutting away the affected sheet and applying an overlapping sheet patch of equivalent plies with appropriate overlap length. Delaminations less than 25 in^2 (16,000 mm^2) may be repaired by either resin injection or ply replacement.*

D.2 MDOT Special Provision for FRP Flexural Strengthening System

1) Repeat numbers (1); (2); (4); (5) from Section D.1.

2) Additional detail on pull-off testing and required minimums are provided in Section 8.2.5.1 of the report.

D.3 MDOT Special Provision for FRP Shear Strengthening System

1) Repeat numbers (1); (2); (4); (5) from Section D.1.

2) Article c (page 2): The first paragraph states a maximum gap of one inch between FRP shear strips is allowed. This is in contradiction to the recommended AASHTO FRP design provisions, which are as follows:

$$\text{For } v_u < 0.125f'_c\,,\ S_{max} = 0.8d_v \leq 24\text{in}$$
$$\text{For } v_u > 0.125f'_c\,,\ S_{max} = 0.4d_v \leq 12\text{in}$$

where d_v is the effective shear depth (in) and v_u the shear stress (ksi), as defined in AASHTO LRFD section 5.8.2.9.