First Middle East Conference on Smart Monitoring,
Assessment and Rehabilitation of Civil Structures
8 – 10 February 2011, Dubai, UAE

SMAR 2011

Externally Bonded FRPs for Shear Strengthening of Prestressed Concrete Girders

M. S. Murphy[1], A. Belarbi[2], and S. Bae[3]

[1] Missouri University of Science & Technology, Missouri, USA
[2] University of Houston, Texas, USA
[3] Texas Tech University, Texas, USA

ABSTRACT: Fiber reinforced polymers (FRP) have gained increased popularity for use in strengthening or repair of concrete structures. Externally bonded applications of these advanced composites have been shown to effectively increase the shear resistance of concrete beams. The present knowledge for externally bonded FRPs used to strengthen concrete beams in shear has been developed from experimental investigations which have primarily involved only small-scale tests on reinforced concrete (RC) beams. Thus, the behavior and effectiveness of this strengthening method at the full-scale and, moreover for prestressed concrete (PC) members, has not been satisfactorily investigated. At Missouri University of Science & Technology, a series of 16 full-scale tests were conducted to investigate the effectiveness of externally bonded CFRP for strengthening AASHTO type PC girders. The results of this study showed the ultimate shear resistance of such girders to be characterized by complex failure mechanisms which are strongly influenced by the cross-sectional shape of the girders. The use of externally bonded CFRP did not provide significant increases in the ultimate shear resistance of the girders, and in some cases, a decrease in the ultimate shear resistance was observed in comparison to the reference (un-strengthened) girders.

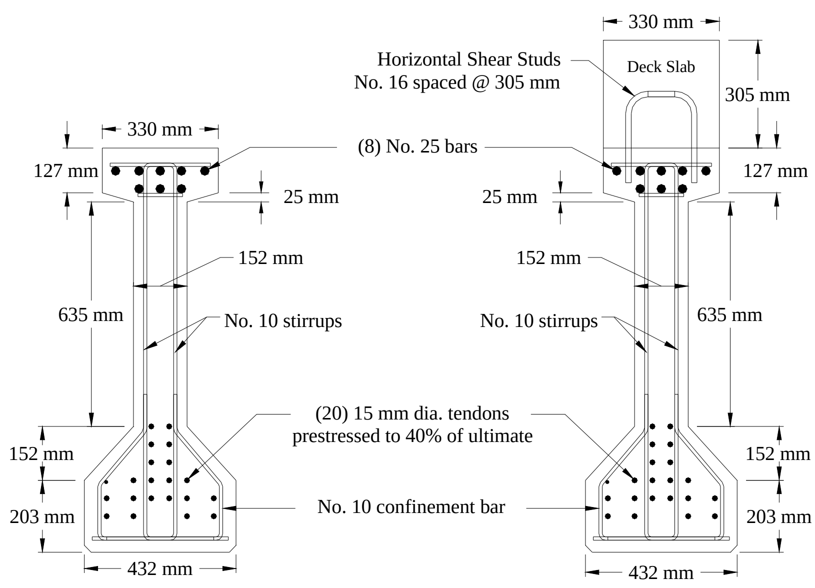
1 INTRODUCTION

The use of externally bonded FRP composites has developed as a cost-effective solution for strengthening the growing number of structurally deficient concrete structures. FRPs developed for such applications generally consist of carbon (CFRP), glass (GFRP), or aramid (AFRP) fibers in the form of sheets or thin plates. These composites offer a number of advantageous qualities such as versatility in conforming to various cross-sectional shapes, a high strength-to-weight ratio, corrosion resistance (i.e., in the case of carbon and aramid fibers), and a fast and relatively simple application process. Many experimental investigations have shown externally bonded FRP composites to be effective in improving axial, flexural, or shear resistance of concrete members. However, it is the use of these materials for shear strengthening that is still a subject of much debate. Typical shear strengthening applications are side bonding, U-wrapping, or complete wrapping of the cross-section and may consist of continuous sheets or discrete strips of FRP along the critical shear span. The present knowledge for shear strengthening of concrete beams with externally bonded FRPs has been developed from experimental investigations which have primarily involved small-scale tests on reinforced concrete (RC) beams. Thus, the behavior and effectiveness of this strengthening method at the full-scale and for prestressed concrete (PC) members has not been satisfactorily investigated. A series of full-scale tests have been conducted at Missouri University of Science & Technology to investigate the shear strengthening effectiveness of externally bonded FRPs applied to PC girders.

2 EXPERIMENTAL PROGRAM

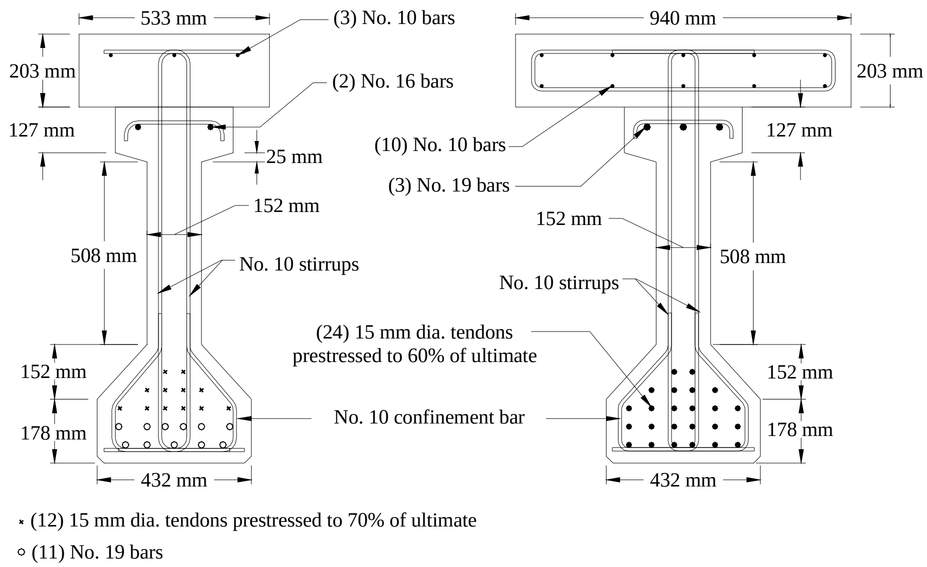
2.1 Test specimens

A total of eight full-scale PC girders were constructed to investigate the effectiveness of externally bonded FRPs for shear strengthening. Each girder was designed to provide two distinct test regions (i.e., a designated test region on each end of the girder) resulting in a series of 16 experimental test specimens. The parameters of interest were: cross-sectional configuration of the girders, transverse steel (stirrup) reinforcement ratio, FRP strengthening scheme, FRP anchorage, and effects of pre-existing shear cracks prior to FRP application. The girder cross-sections consisted of AASHTO Type 4 (Figure 1) and Type 3 (Figure 2) geometries specified by the Missouri Department of Transportation (2005). The deck slab of the girders was also a variant among the cross-sectional configurations as shown in Figures 1 and 2. As a consequence, the flexural steel reinforcement scheme also varies among the cross-sectional configurations based on design requirements for producing shear deficient girders and consideration for ACI 318 (2008) and AASHTO LRFD (2008) design specifications. The transverse steel reinforcement consisted of No. 10 double-legged stirrups with two different stirrup spacings. The stirrup spacing was either 305 mm representing a moderately reinforced member (i.e., satisfying ACI 318 and AASHTO LRFD requirements) or 457 mm representing a member with low reinforcement (i.e., not satisfying ACI 318 and AASHTO LRFD requirements).



(a) Cross-section Type I (b) Cross-section Type II

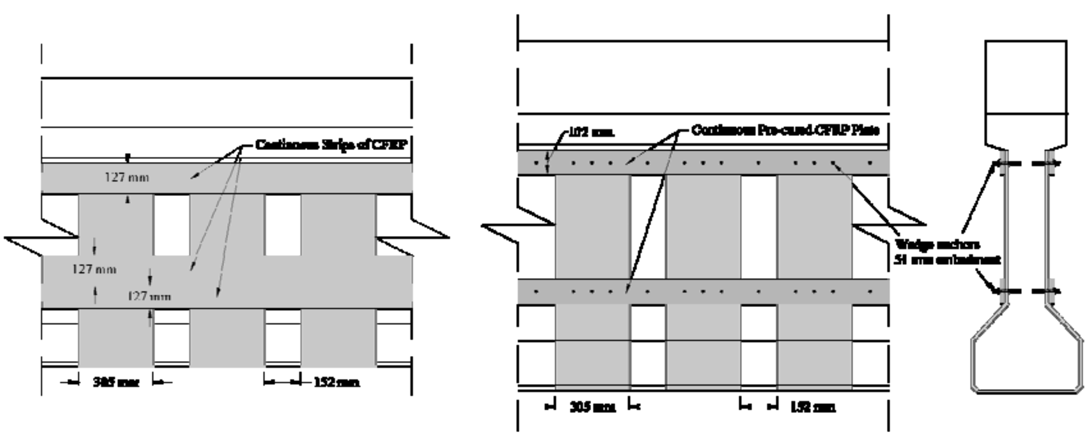
Figure 1. Cross-sectional configurations of MoDOT Type 4 experimental PC girders.



(a) Cross-section Type III (b) Cross-section Type IV

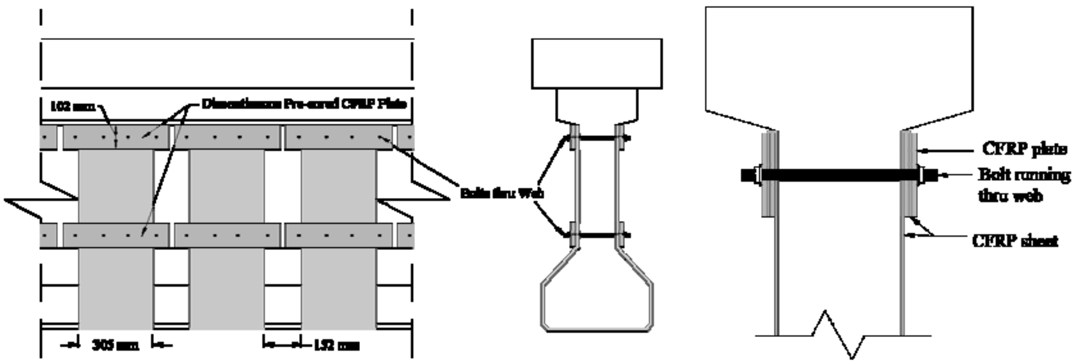
Figure 2. Cross-sectional configurations of MoDOT Type 3 experimental PC girders.

FRP strengthening schemes consisted of single-ply CFRP strips of 305 mm width applied in a U-wrap configuration. The CFRP strips were made of a uni-directional fiber composite, and they were applied with fibers oriented at either 90 or 45 degrees relative to the longitudinal axis of the girders. The strips were spaced so as to provide a 152 mm gap between each strip for the observation of crack propagation. As a consequence, it is noted that the CFRP reinforcement ratio also varies between the two different fiber orientations investigated. Anchorage of the CFRP strips to help reduce debonding issues was investigated using four different methods as shown in Figures 3 and 4.



(a) Horizontal CFRP Strips (b) Continuous Pre-cured CFRP Plates

Figure 3. CFRP anchorage systems from experimental program.



(a) Discontinuous Pre-cured CFRP Plates

(b) Discontinuous Pre-cured CFRP Plates with Sandwiched Ends

Figure 4. CFRP anchorage systems from experimental program.

The effects of pre-existing cracks were investigated in one specimen by applying load to the girder until the formation of significant shear cracking prior to the application of FRP strengthening. It is also noted that the shear span of the test region was increased after completing the first three tests in order to preclude the unfavorable failure modes observed which were believed to be contributed to arch action (deep beam) behavior. As a consequence of such changes in the shear span and cross-sectional depth of the section, the shear span-to-depth ratio is also a variable of the experimental program. Tables 1 and 2 show the experimental test matrices for the MoDOT Type 4 and MoDOT Type 3 specimens, respectively.

Table 1. Experimental test matrix for MoDOT Type 4 girders

Test I.D.	Test Parameters					
	Cross-Section Type	Shear Reinforcement		Strengthening Scheme	Anchorage Type	a/d
		Steel (ρ_v)	FRP (ρ_f)			
T4-12-Control	I	0.0031	0	None	None	2.9
T4-18-Control	I	0.0020	0	None	None	2.9
T4-18-S90-NA	I	0.0020	0.0014	Strips/90	None	2.9
T4-18-S90-CMA	II	0.0020	0.0014	Strips/90	Continuous CFRP Plates	2.9
T4-18-S90-DMA	II	0.0020	0.0014	Strips/90	Discontinuous CFRP Plates	2.9
T4-18-S45-DMA	II	0.0020	0.0010	Strips/45	Discontinuous CFRP Plates	2.9
T4-12-Control-Deck	II	0.0031	0	None	None	2.9
T4-12-S90-SDMA	II	0.0031	0.0014	Strips/90	Discontinuous Sandwich CFRP Plates	2.9

Table 2. Experimental test matrix for MoDOT Type 3 girders

Test I.D.	Test Parameters					
	Cross-Section Type	Shear Reinforcement		Strengthening Scheme	Anchorage Type	a/d
		Steel (ρ_v)	FRP (ρ_f)			
T3-12-Control	III	0.0031	0	None	None	3.4
T3-12-S90-NA	III	0.0031	0.0014	Strips/90	None	3.4
T3-12-S90-NA-PC*	III	0.0031	0.0014	Strips/90	None	3.4
T3-12-S90-DMA	III	0.0031	0.0014	Strips/90	Discontinuous CFRP Plates	3.4
T3-18-Control	IV	0.0020	0	None	None	3.4
T3-18-S90-NA	IV	0.0020	0.0014	Strips/90	None	3.4
T3-18-S90-HS	IV	0.0020	0.0014	Strips/90	Horizontal CFRP Strips	3.4
T3-18-S90-SDMA	IV	0.0020	0.0014	Strips/90	Discontinuous Sandwich CFRP Plates	3.4

* Specimen tested to investigate effects of pre-existing cracks prior to FRP strengthening

2.2 *Test set-up*

The test set-up was a three-point loading configuration as shown in Figure 5. The shorter shear span is the designated test region since it experiences the highest shear forces. The test region was 2.74 m for the first three test specimens but was increased to 3.66 m for the remaining specimens to preclude unfavorable failure modes as previously discussed. The test setup also consisted of an additional external strengthening system composed of a series of HSS hollow steel sections and No. 36 Dywidag bars (Figure 5). This system was intended to prevent failure from occurring outside the designated test region and to protect the second test region from premature damage during testing of the first test region. The opposite end of the girder is tested as a second test region by simply repositioning the support, reaction frame, and loading frame.

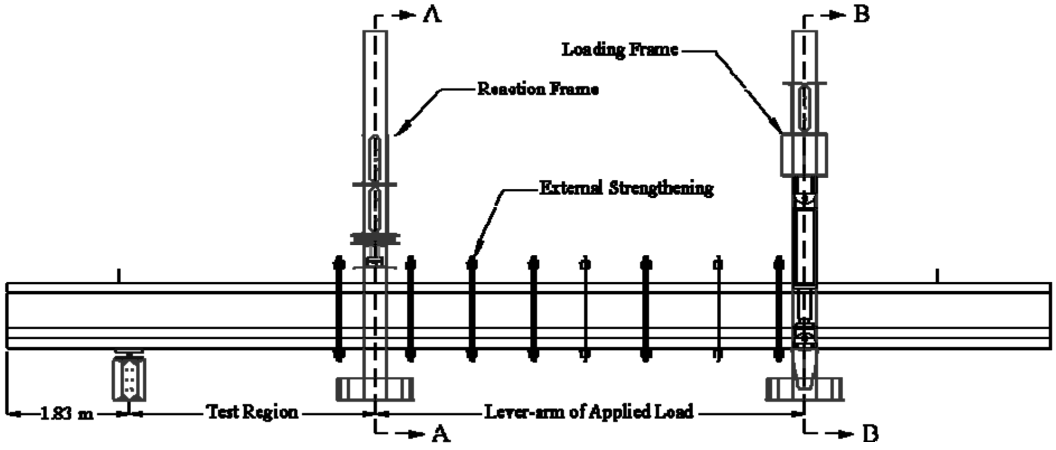


Figure 5. General test set-up for PC girders.

3 RESULTS AND DISCUSSION

The mode of failure and maximum shear resistance of each specimen is summarized in Table 3. It is important to remember that because the cross-sectional configuration varies among the experimental specimens, direct comparison of the maximum shear force and corresponding failure modes should be limited to those specimens sharing a common cross-sectional geometry. Such comparisons show that the application of CFRP for shear strengthening did not always yield an increase in the maximum shear resistance of the girders. It is believed that the thin webbed geometry typical of these types of PC girders has a significant effect on the performance of externally bonded FRPs for shear strengthening. However, within the experimental results for cross-section type II, the effectiveness of various mechanical anchorage schemes and fiber orientations is observed by comparison of the maximum shear forces measured at failure.

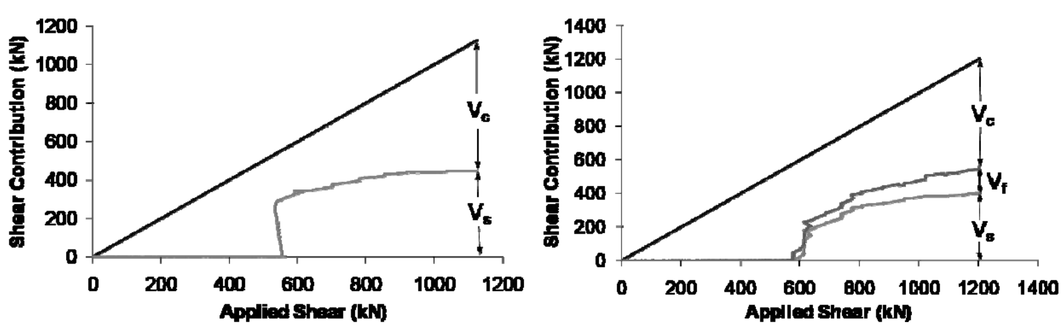
Table 3. Experimental results

Cross-Section Type	Test I.D.	Concrete Strength of Girder (MPa)	Shear Force at Failure (kN)	Failure Mode	Shear Crack Angle (deg.)
I	T4-12-Control	69	899	TF	32
	T4-18-Control	68	916	TF	26
	T4-18-S90-NA	69	859	D + TF	21
II	T4-18-S90-CMA	70	1019	D + MA + TF	25
	T4-18-S90-DMA	70	1085	D + LR + TF	24
	T4-18-S45-DMA	70	1134	D + TF	32
	T4-12-Control-Deck	73	1090	TF	26
	T4-12-S90-SDMA	71	1148	TF	30
III	T3-12-Control	61	1125	SC	23
	T3-12-S90-NA	61	1205	D + WC	22
	T3-12-S90-NA-PC	65	1063	D + WC	21
	T3-12-S90-DMA	72	1108	SC	25
IV	T3-18-Control	66	1121	DT	21
	T3-18-S90-NA	70	961	D + DT	15
	T3-18-S90-HS	70	983	D + DT	26
	T3-18-S90-SDMA	72	1045	D + DT	33

Note: TF = horizontal failure along the top flange, D = debonding of CFRP, LR = localized rupture of CFRP, DT = diagonal shear-tension failure, WC = web crushing failure, MA = mechanical anchorage failure, SC = stress concentration failure at reaction point

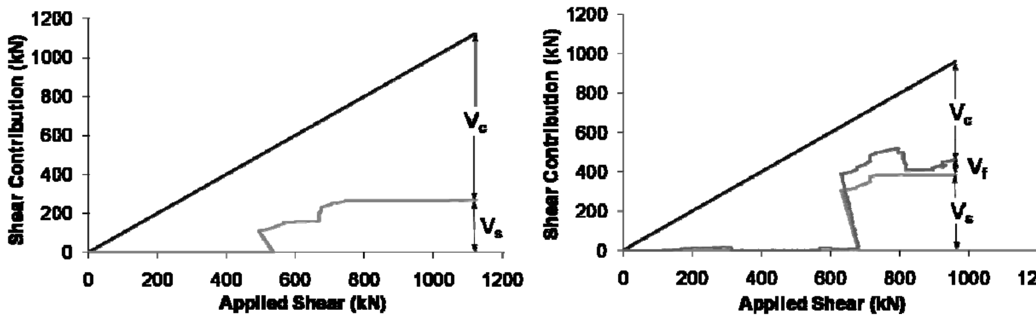
As seen in Table 3, a variety of failure modes were observed among the test specimens. Some of the test specimens exhibited multiple failure modes either at the same time or in a sequential manner. For the MoDOT Type 4 specimens (cross-sectional types I and II) shear cracks in the web were observed to propagate toward the top flange at which point they turned and ran horizontally along the longitudinal compression reinforcement. The maximum shear force carried by these specimens was ultimately governed by a failure plane created by the horizontal cracks along the top flange (failure mode - TF). This failure mode is caused by the moderate amount of longitudinal steel in the top flange designed to give the girders adequate flexural capacity. These longitudinal compression reinforcements were observed to buckle under the large compression stresses in the top flange resulting in significant horizontal cracking. For the MoDOT Type 3 specimens with moderate transverse steel reinforcement (cross-sectional type III), failure due to web crushing (failure mode – WC) or high stress concentrations near the reaction point (failure mode – SC) were observed. Web crushing was observed as a result of peeling of the concrete cover associated with debonding of the CFRP resulting in a significant reduction in the width of the concrete compressive struts within the thin webs. For the Type 3 specimens with low transverse steel reinforcement (cross-sectional type IV), failure was always characterized by diagonal shear-tension failure (failure mode – DT) preceded by some level of debonding (failure mode – D) when CFRP shear reinforcement was present.

Since an increase in shear capacity was not observed between the strengthened and un-strengthened specimens, the FRP contribution to the total resistance of the girders was determined from a shear component analysis. For this analysis, a free-body diagram of the specimens along the critical shear crack was considered. Strain gage measurements taken along the transverse steel and CFRP reinforcements were used to evaluate their respective contributions. The remaining concrete contribution was evaluated considering equilibrium of the internal forces and externally applied shear force. Figure 6 shows a sample of the results of this analysis for both strengthened and un-strengthened specimens of the MoDOT Type 3 section. As expected, the steel stirrup and CFRP strengthening contributions are nil prior to concrete cracking. After cracking, however, there is a sudden jump in their contributions followed by a gradual rise up to yielding of the steel stirrups or debonding in the case of CFRP. Comparison of the strengthened and un-strengthened results shows that the CFRP contribution is coming from a reduction in either the concrete or transverse steel contributions. Such analysis, demonstrates the strong interdependence of the three components of shear resistance (e.g., concrete, steel stirrups, and CFRP strengthening).



(a) T3-12-Control

(b) T3-12-S90-NA



(c) T3-18-Control (d) T3-18-S90-NA

Figure 6. Shear component diagrams.

4 CONCLUSIONS

The results of this experimental program indicate that the use of externally bonded FRPs may not be beneficial for shear strengthening of thin webbed members such as the AASHTO type PC girders. The cause for this behavior cannot be fully explained by the results obtained from this study; however, it is believed to be strongly influenced by the thin nature of the web of such beams and the triaxial stress state induced by debonding of the FRP. It is noted that previous studies on prestressed beams by Hutchinson and Rizkalla (1999) and Reed and Peterman (2004) were able to show improvements in shear capacity of up to 36% and 28% respectively. However, the specimens tested by Hutchinson and Rizkalla were 1:3.5 scale models, measuring only 475 mm in depth while those tested by Reed and Peterman consisted of 585 mm deep double-tee beams taken from an existing structure. These specimens are much shallower than those tested as part of this study and hence the size effect could be responsible for the lack of FRP effectiveness observed. Moreover, the shape of a double-tee beam is vastly different from the I-shape of typical AASHTO girders for which the lower flange has been shown to initiate an outward pulling force in the FRP strengthening and cause premature debonding failures. It has been demonstrated for moderately reinforced beams that debonding of FRP, which is generally accompanied by peeling of the concrete cover, can result in a break down in the diagonal compressive struts in such thin webbed members. A shear component analysis reveals the strong interdependence among the concrete, transverse steel, and FRP strengthening contributions. It is shown that the FRP contribution causes a reduction in the concrete or transverse steel reinforcement contributions. As such, the use of such strengthening approaches should be further investigated before implementing them in practice.

5 REFERENCES

AASHTO (2008), *AASHTO LRFD Bridge Design Specifications*, 4th edition, Washington, DC, USA.

ACI Committee 318 (2008), *Building Code Requirements for Structural Concrete and Commentary (ACI 318-08)*, Farmington Hills, MI, USA.

Hutchinson, R. L., and Rizkalla, S. H. (1999), "Shear Strengthening of AASHTO Bridge Girders Using Carbon Fiber Reinforced Polymer Sheets," ACI Special Publications (SP-188), Vol. 188, pp. 945-958.

MoDOT (2005), *MoDOT LRFD Bridge Design Guidelines*, Missouri Department of Transportation, Missouri, USA.

Reed, C. E., and Peterman, R. J. (2004), "Evaluation of Prestressed Concrete Girders Strengthened with Carbon Fiber Reinforced Polymer Sheets," Journal of Bridge Engineering, Vol. 9, No. 2, pp. 185-192.