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Modeling of shear deficient bridge girders externally wrapped with FRP

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ABSTRACT: Currently, numerous bridge girders worldwide are rapidly deteriorating and structural deficiency of these bridges, especially in shear, is becoming common to structural engineers. The application of externally bonded fiber reinforced polymers (FRPs) to bridge girders is an economical and promising solution. Numerous models have been proposed in technical literature to predict the FRP shear contribution as well as the ultimate shear capacity of a FRP wrapped girder; however, the reliability of these models still needs to be verified. In this paper, the codes and models used in North America to predict the fiber reinforced polymer shear contribution to reinforced concrete bridge girders are evaluated. These codes and models are compared against experimental results, consisting of more than 300 specimens, obtained from an extensive literature review. The results show that the current codes yield inaccurate predictions and are very conservative.

1 INTRODUCTION

The world's infrastructure is rapidly deteriorating and in need of drastic rehabilitation. For example, bridges in Canada have reached 49 percent of their useful life, according to Statistics Canada (2006). It is now becoming increasingly dangerous as seen in the 2006 disaster in Naval, Quebec, just north of Montreal, when a highway overpass collapsed and resulted in the death of five people (CBC News 2006). Internal corrosion of rebar in transportation infrastructure can drastically affect the structural integrity of bridges, which has increased substantially in the last three decades (Magnitude of Rebar Corrosion Problems 2010). In addition, the live loads on highway bridges are being continuously tested as more trucks are exceeding weight limitations (TRB 1997). Furthermore, earthquakes are more common as seen in the 2010 earthquake in Haiti. As a result, structures, especially bridges, are in need of strengthening and rehabilitation. Therefore, a reliable and economical solution is required (Moroz, H. 2008) for bridge rehabilitation.

The use of fiber reinforced polymers (FRPs) for repair of bridge girders is a promising solution as they have a high strength to weight ratio and are corrosion resistant. Furthermore, the rehabilitation process is not laborious or time consuming and will keep traffic disruptions to a minimum. FRP can be externally applied to deteriorating or structurally deficient reinforced concrete elements to increase their flexural or shear capacity. FRP are made of an adhesive (a resin), which bonds the fibers (carbon, glass or aramid) to the member (ACI 440.2R-08). This paper will focus on the shear strengthening of beams with externally bonded FRP.

To increase the shear strength, the externally bonded FRP wrap can be applied continuously or discontinuously (strips) to a beam in three general wrapping schemes: side bonding, "U" wrapping, or fully wrapping. Fully wrapping a beam is unrealistic for most rehabilitation purposes as the bridge deck is typically in the way. For side and U wrapping, anchorage should be provided to a premature debonding failure of the beam (CSA S608-02).

The objective of this paper is to assess the accuracy of the current design guidelines used in North America, which predict the ultimate shear capacity of reinforced concrete beams with externally applied FRP by comparing them to experimental results obtained from an extensive literature review. The American design guidelines provided by ACI 440.2R-08 and the Canadian design code provided by CSA S608-02 will be evaluated in this paper. In addition, recommendations and directions for future research and development will also be identified.

2 THEORETICAL BACKGROUND

2.1 *Concrete and Steel Shear Contributions*

The shear capacity of a reinforced concrete element is generally calculated by aggregating the concrete (V_c) and steel stirrups (V_s) contributions as done by ACI 440.2R-08 and CSA S608-02. The concrete contribution accounts for aggregate interlock between the two faces of the shear cracks, dowel action of the flexural reinforcement and the uncracked concrete in the compression zone (ACI 445 1998). The computation of the concrete contribution to the shear capacity has its basis in the Modified Compression Field Theory (ACI 445 1998). It is also assumed that once the first diagonal shear crack occurs the concrete contribution remains near constant (ACI 445 1998). Once this shear crack develops, the steel stirrups crossing this crack become active in resisting the shear force. The stirrups crossing this initial shear crack will yield first followed by the remaining stirrups. The steel stirrup shear contribution is computed using the Truss Analogy (ACI 445 1998).

2.2 *Fiber Reinforced Polymer Shear Contributions*

The effectiveness of the FRP depends on the mode of failure. Premature debonding of the FRP from the concrete surface is one of two typical failure modes which is a function of the bond properties between the concrete and the FRP. The bond between the FRP sheets and the concrete substrate depends on many factors such as the anchorage or bond length and the FRP stiffness. Shear failure or FRP rupture is the other mode of failure. Since the FRP shear contribution is difficult to quantify, various researchers have come up with different models. The FRP shear contribution was initially modeled using the Truss Analogy as internal stirrups. However, this model was not accurate enough to predict the FRP shear contribution. Consequently, various researchers developed factors and empirical formulations of effective strains to account for these inaccuracies (Triantafilluo 1998, Triantafilluo & Antonopoulus 2000, Khalifa & Nanni 2000). These models and equations used to calculate the effective strain in the FRP and thus predict the FRP shear contribution are typically empirical equations taking the FRP stiffness ($\rho_F E_F$), the concrete strength (f_c') and other variables into account.

3 DESIGN GUIDELINES

3.1 *ACI 440.2R-08*

The design guidelines provided by ACI 440.2R-08 consider several factors when predicting the FRP shear contribution such as the wrapping scheme, failure mode, beam geometry and existing concrete strength (ACI 440.2R-08). These design guidelines also predict the ultimate capacity (V_N) of a structural member by summing the concrete, steel and FRP shear contributions as shown in Equation 1.

$$V_N = V_C + V_S + V_F \tag{1}$$

The shear contribution of the FRP is calculated by predicting the tensile forces in the FRP across the initial shear crack (Equation 2).

$$V_F = \frac{A_{Fv} f_{F,e} (\sin\beta + \cos\beta) d_F}{s_F} \tag{2}$$

Where β is the fiber inclination angle, d_F is the FRP effective height, s_F is the strip spacing and A_{Fv} is the area of the FRP strip calculated using Equation 3.

$$A_{Fv} = 2 n_F t_F w_F \tag{3}$$

Where n_F is the number of FRP plies, t_F is the thickness of one FRP ply and w_F is the width of the FRP strip. The effective stress in the FRP sheets, $f_{f,e}$, can be calculated using Equation 4.

$$f_{F,e} = \varepsilon_{F,e} E_F \tag{4}$$

Where $\varepsilon_{F,e}$ is the FRP effective strain and E_F is the FRP Modulus of Elasticity.

According to ACI 440.2R-08 “The effective strain is the maximum strain that can be achieved in the FRP system at nominal strength and is governed by the failure mode of the FRP system and of the strengthened reinforced concrete member.” Therefore, for completely wrapped members, ACI assumes that the FRP system will not reach its ultimate strain before the loss of aggregate interlock of the concrete as shown in Equation 5.

$$\varepsilon_{F,e} = 0.004 \le 0.75\varepsilon_{F,u} \tag{5}$$

For structural members with FRP side bonded or U wrapped to a member, it was observed that the debonding of the FRP will occur before the loss of aggregate interlock of the concrete. For this reason, a bond reduction coefficient (k_v) is used to determine an effective strain as shown in Equation 6.

$$\varepsilon_{F,e} = k_v \varepsilon_{F,u} \le 0.004 \tag{6}$$

This bond reduction coefficient is mainly a function of the FRP stiffness and the concrete strength. Equations 7 through 10 are used to compute this value.

$$k_v = \frac{k_1 k_2 L_e}{11900\varepsilon_{F,u}} \le 0.75 \tag{7}$$

$$L_e = \frac{23300}{\left(n\ t_F E_F\right)^{0.58}} \tag{8}$$

$$k_1 = \left(\frac{f_c'}{27}\right)^{2/3} \tag{9}$$

$$k_2 = \frac{d_F - L_e}{d_F} \qquad \text{U wrap} \tag{10a}$$

$$k_2 = \frac{d_F - 2L_e}{d_F} \qquad \text{Side Bonded} \tag{10b}$$

L_e is the active bond length calculated from Equation 8 and n is the modular ratio of elasticity between FRP and concrete (E_F/E_c). k_1 and k_2 are modification factors that are used to account for the type of wrapping scheme and concrete strength.

Additional anchorage can also be provided to increase the amount of strain developed in the FRP; however in no case should the strain exceed 0.004.

3.2 *CSA S806-02*

The Canadian design code provided by CSA S806-02, calculates the ultimate shear capacity of a RC member as the summation of concrete, steel and FRP shear contribution following Equation 1 alike the ACI design equation. Although CSA S806 design code does allow for the use of different fiber angles other than 90 degrees to the longitudinal axis of the beam, it is not reflected in the FRP shear contribution Equation 11.

$$V_F = \frac{\phi_F A_F E_F \varepsilon_{F,e} d_F}{s_F} \tag{11}$$

The CSA S806 design code accounts for different wrapping schemes by limiting the effective strain. For members completely wrapped or U wrapped the effective strain is set to 0.004. For members with side bonded, the effective strain is set to 0.002. Furthermore, for members that are U wrapped with FRP or have it bonded to the side, additional anchorage or development length must be used to ensure that the premature debonding of FRP does not occur.

4 EXPERIMENTAL DATABASE

An extensive literature review was performed to collect all the reported experimental data in the literature (Norris et al. 1997, Pellegrino & Modena 2002, Chajes et al. 1995, Deniaud & Cheng 2001, Zhang & Hsu 2005, Tanarslan et al. 2008, Täljsten 2003, Triantafillou 1998, Al-Sulaimiani et al. 1994, Cao et al. 2005, Khalifa & Nanni 2000, Uji 1992, Täljsten & Elfgren 2000, Li et al. 2001, Chaallal et al. 2002, Khalifa & Nanni 2002, Diagani et al. 2002, Chaallal & Bousselham 2006, Funakawa et al. 1997, Araki et al. 1997, Umezu et al. 1997, Sato et al. 1997, Hassan et al. 2002, Ianruberta & Imbimbo 2004, Pellegrino & Modena 2008, Adhikary & Mutsuyoshi 2004, Kim et al. 2007, Dirar et al. 2007, Zhang et al. 2007).

The experimental database included a total of 305 beam specimens of which 214 were externally reinforced with FRP and the remaining were unwrapped (control). Among the tested beams, 75.1% were rectangular and the remaining 24.9% were T-beams as shown in Figure 1a. Carbon was the most commonly used type of FRP consisting of 56.1% of the database followed by Aramid at 8.2% and glass at 5.9 % (Figure 1b). Furthermore, the majority of the beams were reinforced with uni-directional FRP, consisting of 72.9% of the database, and the rest with bi-directional FRP consisting of 26.6% (Figure 1c). Figure 1d shows most of the beams were continuously wrapped along their length (67.3%) rather than in discrete strips (32.7%). Most of the beams were U wrapped (42.1%), followed by side bonded (30.8%) and completely wrapped (27.1%) (Figure 1e). The majority of the tested beams failed either due to FRP rupture (45.9%) or premature debonding of the FRP reinforcement (45.4%); however some of the beams failed in flexure (8.7%) (Figure 1f).

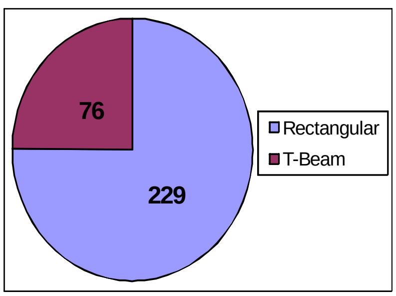

Figure a. Beam Type

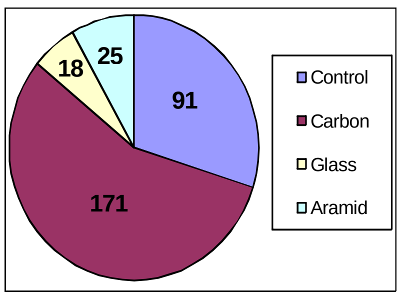

Figure b. FRP Type

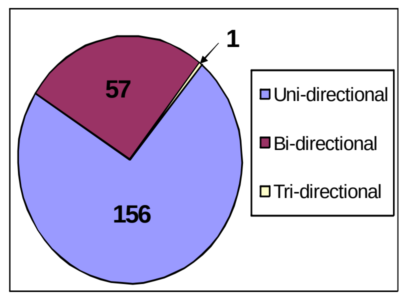

Figure c. FRP Fiber Orientation

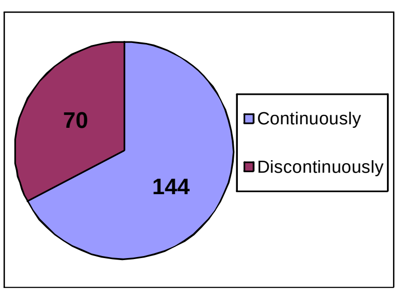

Figure d. FRP Wrapping Scheme

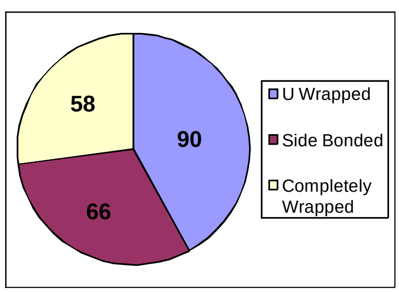

Figure e. FRP Wrapping Scheme

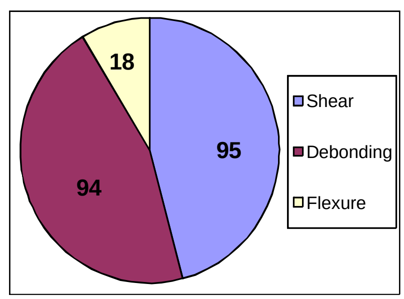

Figure f. Failure Mode

Figure 1. Detailed Breakdown of the Experimental Database

In the experimental database there is a wide range of values for many of the variables used to predict the FRP shear contribution; for example the concrete strength ranged from 17.8 MPa to 71.4 MPa. Table 1 shows some of the variables in the database and the ranges of their corresponding values.

5 RESULTS AND DISCUSSION

The experimental database was used to predict the shear strength using the design equations provided by ACI 440.2R-08 and CSA S806-02. These design equations are used worldwide and should be able to accurately predict the FRP shear contribution to the ultimate shear capacity of a structural member. Over estimating the FRP shear contribution can lead to catastrophic failures. Underestimating this contribution can lead to over reinforcing a member which may promote a less ductile failure (Teng et al. 2009). In the evaluation of these design equations, all safety and environmental reduction factors were set to unity (1).

5.1 *ACI 440.2R-08*

The design guidelines provided by ACI 440.2R-08 allow an engineer to easily design for external FRP shear reinforcement of a structural member. These design guidelines assume failure modes based on wrapping scheme. For example if the member is completely wrapped it assumes a shear failure or FRP rupture, on the other hand, if the member is U wrapped or side bonded it assumes a premature debonding of the FRP from the concrete substrate. These assumptions may not be entirely true and accurate if enough development length or additional mechanical anchorage is provided. Figure 2 shows a comparison of the experimental and predicted FRP shear contribution results from the ACI 440.2R-08 design guidelines.

Table 1: Experimental Database Material Properties

Properties	Average	Standard Deviation	Minimum	Maximum
Beam Width (*b*) (mm)	275	195	70	706.2
Beam Height (*h*) (mm)	332	115	110	600
Beam Length (*l*) (mm)	2306	1174	800	6000
Shear Span (*a*) (mm)	874	405	320	1675
Effective Depth (*d*) (mm)	272	103	50.8	510
Concrete Compressive Strength (f'_c) (MPa)	35.3	10	17.8	71.4
Tensile Reinforcement Ratio (ρ) (%)	2.74	1.33	1.03	7.54
FRP Thickness (t_F) (mm)	0.627	0.732	0.044	3.000
FRP Modulus (E_F) (GPa)	169.7	101.2	5.3	392.0
FRP Ultimate Strain ($\varepsilon_{F,u}$) (%)	1.59	0.68	0.20	3.70
FRP Ultimate Stress ($f_{F,u}$) (MPa)	254	1374	67.8	4490

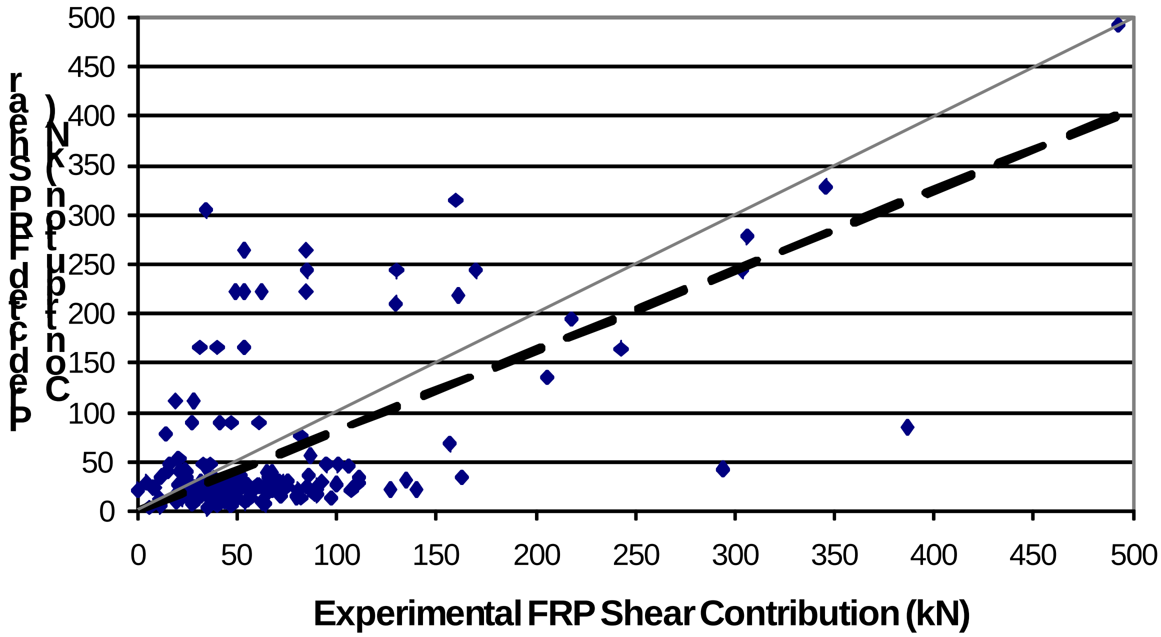

Figure 2. ACI 440.2R-08 Comparison of FRP Shear Contribution: Experimental to Predicted Results

The average ratio of the experimentally determined FRP shear contribution to that of ACI 440.2R- 08 prediction is 2.40 with a standard deviation of 2.26. This shows that not only does ACI underestimate the FRP shear contribution but also the dispersion is quite significant. Furthermore, when the database is divided into different types of failures (FRP rupture and premature debonding), then the average ratio of experimental results to that predicted by ACI for shear failure is 2.99 with a standard deviation of 2.68. When premature debonding is the failure mode, then the average ratio is 1.87 with a standard deviation of 1.70. This shows that the design guidelines are more accurate at predicting debonding failures than shear failures.

5.2 *CSA S806-02*

The design code provided by CSA S806-02 is simple and easy to implement. It assumes a constant value for the effective strain in the FRP based on the wrapping scheme and anchorage provided. These guidelines do not differentiate between different types of failures (shear or debonding). However, CSA S806-02 clearly states that for FRP side bonded or U Wrapped to a reinforced concrete member that "sufficient development length shall be provided or mechanical anchorages shall be used" (CSA S806-02 Clause 11.3.2.1). This insures that premature debonding of the FRP will not occur. Figure 3 shows a comparison between the experimental and the predicted results according to CSA S806-02 design guidelines.

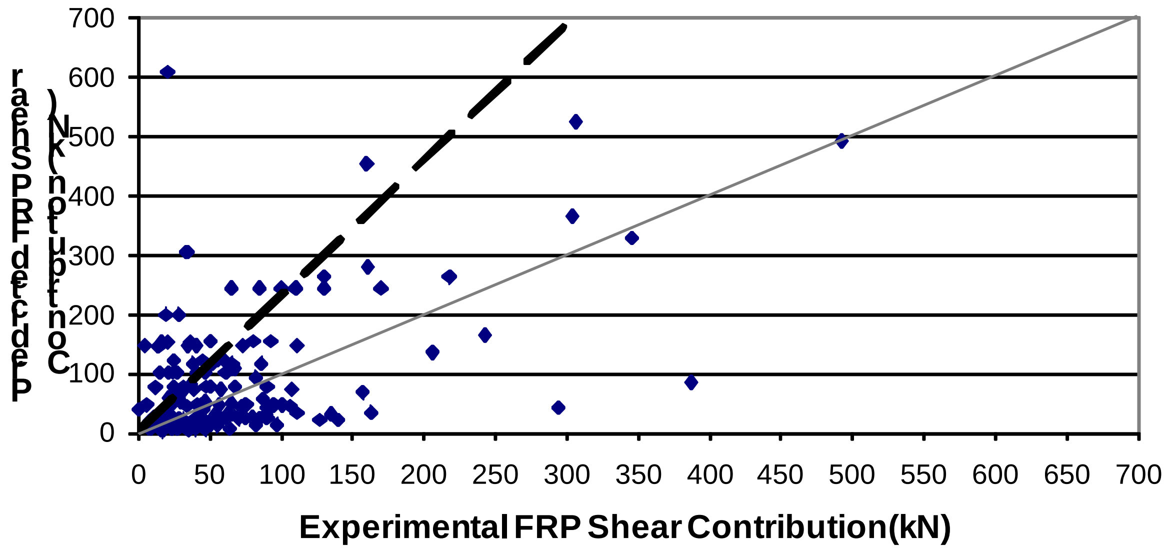

Figure 3. CSAS806-02 Comparison of FRP Shear Contribution: Experimental to Predicted Results.

As shown in Figure 3, the design equations provided by CSA S806-02 cannot predict the results accurately where the average of the ratio of the experimentally determined FRP shear contribution to that predict by CSA S806-02 is 1.73 with a standard deviation of 2.23. These inaccuracies may stem from the fact that constant values are assumed for an effective strain and different failure modes are not considered. When the database is divided into different types of failure modes, the average ratio for shear failures is 2.66 with a standard deviation of 2.69.

6 CONCLUSION

The use of externally applied FRP for shear reinforcement is gradually gaining popularity in bridge repair and rehabilitation due to its superior material properties as well as its ability to be easily installed without traffic disruptions. Based on the comparison between the experimental and predicted results the following conclusions were drawn:

- The current Canadian and American design guidelines presented and evaluated here show that the existing models cannot predict the FRP shear contribution accurately.
- The guidelines provided by ACI 440.2R-08 are more conservative than the ones provided by CSA S806-02. When the failure mode is in shear the American guidelines are more conservative than the Canadian guidelines.

- The American guidelines allow for the prediction of the FRP shear contribution when premature debonding of the FRP is the mode of failure although it is less conservative; however, the Canadian guidelines do not permit this failure mode since it is considered a premature failure. In any case, one should design for the rupture of the FRP by providing enough development length or additional mechanical anchorage. This will enable the FRP to reach its maximum allowable strain at failure thus ensuring the greatest FRP shear contribution is achieved.

Further experimentation in this field will allow for the development of more accurate design guidelines, which will greatly aid in not only the design process but the acceptance of FRP as a suitable repair and rehabilitation technique for structural deficient bridges.

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