CHAPTER V
CONCLUSIONS AND RECOMENDATIONS

5.1: Introduction

This thesis has two primary objectives. The first is to quantify material and bond properties, and determine their effect on the design of a bridge deck. The second objective is to compare the direct pullout method with the beam end bond test method, and determine if there is a significant difference in maximum bond stress. Recommendations, and conclusions are given in each section, with final overall conclusive remarks given at the end of the chapter

5.2: Material and Bond Properties and Their Effect on Design

5.2.1: Introduction

Several GFRP material properties were investigated, which have significant impacts on the design of a bridge deck with GFRP bars. The properties most closely evaluated were the ultimate tensile strength, the modulus of elasticity, the peak bond stress, and bond behavior of the GFRP bars. These properties are discussed and evaluated, and recommendations presented individually.

5.2.2: Ultimate Tensile Strength

The ultimate tensile strength was determined for each tensile test performed. This data was averaged by bar size and manufacturer. The tested average tensile strength, as well as the standard deviation, the manufacturer's published tensile strength data, the guaranteed tensile strength, and the design tensile strength are shown in Table 5.1.

Table 5.1: Compiled tensile strength data.

Manufacturer	Bar Size	Tested Average Tensile Strength ($f_{u,ave}$) (MPa)	Standard Deviation (σ) (MPa)	Manufacturer's Published Tensile Strength (f_{fu}) (MPa)	Guaranteed Tensile Strength[1] (f^*_{fu}) (MPa)	Design Tensile Strength According to ACI 440 Code[2] (f_{fu}) (MPa)
Hughes Bros.	#4	818	26	690	741	519
	#5	752	38	655	639	447
	#6	674	24	620	602	421
Marshall	#4	751	24	800	678	475
	#5	737	59	780	560	392
	#6	749	53	720	589	413
Pultrall	#4	606	46	617	467	327
	#5	576	15	674	531	371
	#6	554	27	719	474	332

1: ($f^*_{fu} = f_{u,ave} - 3\sigma$)   2: ($f_{fu} = f^*_{fu}$*Ce, where Ce = 0.7)

The guaranteed tensile strength, f^*_{fu}, is defined by the ACI 440.1 (2001) design guide as the tested average or mean tensile strength minus three times the standard deviation ($f^*_{fu} = f_{u,ave} - 3\sigma$). Also, the ACI 440.1 (2001) design guide states that the design tensile strength shall be the guaranteed ultimate tensile strength, f^*_{fu}, multiplied by an environmental reduction factor, C_E. C_E is taken as 0.7 for GFRP bars exposed to earth and weather.

Examination of Table 5.1 shows that Hughes Brothers publishes tensile strength values below the average tensile strength. One of these values is also below the guaranteed tensile strength as defined by ACI 440.1 (2001). The other manufacturers publish values that are above the tested average tensile strength values. Some of the tested averages are close to the published values, and the difference may be the result of the small sample size (five bars per bar size per manufacturer) for the tested averages.

Also Pultrall is the only manufacturer to publish an increase in tensile strength with bar size. This is not consistent with the tested results, which showed a drop in tensile strength with increasing bar size.

There is no clear relationship that can be established between the published values and the tested average, for all of the manufacturers. This may be due in part to the fact that the ACI 440.1 (2001) design guide was released in April of 2001, and most of the manufacturers' literature was published before that time. The published literature does not necessarily reflect the recommendations of the ACI 440.1 (2001). Therefore, for the design of a bridge deck, a designer first must determine design strength values.

One design alternative would be to use the manufacturers' published tensile strength data, which assumes that the manufacturer has established their own confidence limit that their bars will meet or exceed their published values. This would require that the designer specify tighter quality control in construction using FRP bars. Random sampling of the manufacturers' product would ensure that the FRP bars placed in construction would meet the criteria used in the design. Specifying to a manufacturer that excess material be shipped to a construction sight would allow for random samples to be gathered and tested. The testing would ensure that the bars used in construction meet or exceed what is published in the manufacturers' literature. This type of specification is analogous to cylinder testing of concrete placed during construction.

Some consideration should be given to the ACI 440.1's definition of guaranteed tensile strength. The guaranteed tensile strength, f^*_{fu}, as defined by the ACI 440.1 (2001) design guide, yields a 99.87% probability (assuming a normal distribution) that any given FRP bar will meet or exceed f^*_{fu}. This is a very high confidence limit, which in turn leads to conservative design values for tensile strength. The definition set for f^*_{fu} ensures that there is only a 0.13% chance of a tested bar being below the guaranteed tensile strength. The ACI 440.1 (2001) design guide assumes almost no risk with regards to tensile strength. This is an extremely safe approach although it does tend to negate one of the advantages of FRP bars, which is its high tensile strength. One way to utilize more of the tensile strength of an FRP bar is to consider redefining the guaranteed tensile strength. The ACI 318-99 (1999) building code in section 5.3.2.1 uses a 99% probability of meeting or exceeding strength when defining the required compressive strength of concrete. This is assuming a 1% risk of not meeting specified strength. A 1% probability of failure for FRP bars would reduce the definition of guaranteed tensile strength from its current definition to the mean minus 2.33 standard deviations ($f^*_{fu} = f_{u,ave} - 2.33\sigma$). Defining the guaranteed tensile strength in this manner will allow a designer to use more of the tensile strength of an FRP. This would also increase the appeal of using FRP bars by allowing designers to utilize more of the high tensile strength behavior that these bars exhibit.

Another design alternative would be to require that the manufacturer submit test results from their FRP bars. Using these test results, the ACI 440.1 (2001) design guide can be utilized, and the design tensile strength could be found according to the set procedures. This would produce conservative results

In conclusion, there are several options for a designer to choose with regards to the tensile strength of FRP bars. The least conservative approach would be to use the manufacturers' published data to design for tensile strength, and ensure that these design criteria are met with random sample testing. A slightly more conservative option would be to alter the definition of guaranteed tensile strength to the mean minus 2.33 standard deviations ($f^*_{fu} = f_{u,ave} - 2.33\sigma$). This would increase the usable tensile strength of the FRP bars, but would require the manufacturer to submit test results to establish the mean and standard deviation. Finally, the most conservative option is to require the manufacturer to supply test results, and design according the ACI 440.1 (2001) design guide.

5.2.3: Modulus of Elasticity

Stress-strain data was collected from all of the tensile and all of the bond tests, in order to perform modulus calculations. From the data collected, the modulus was calculated as the slope of the longest continuous (linear with no jumps or breaks) portion of the stress-strain diagram. The modulus was then compiled and averaged by bar size and manufacturer. The average tested tensile modulus, the guaranteed tensile modulus, the manufacturers' published tensile modulus, and average modulus minus three standard deviations is shown in Table 5.2.

Table 5.2: Tensile modulus data for all manufacturers by bar size.

Manufacturer	Bar Size	Tested Average Tensile Modulus ($E_{f,ave}$) (MPa)	Standard Deviation (σ)	Design Tensile Modulus[1] (E_f) (MPa)	Manufacturer's Published Tensile Modulus (E_f) (MPa)	Modulus Mean Minus 3 Standard Devs. ($E^*_f = E_{f,ave} - 3\sigma$) (MPa)
Hughes Bros.	#4	43800	2600	43800	40800	36000
	#5	44300	3620	44300	40800	33500
	#6	43000	1460	43000	40800	38600
Marshall	#4	38900	1700	38900	42000	33700
	#5	41600	2140	41600	40000	35200
	#6	40000	3370	40000	40000	29900
Pultrall	#4	39800	2910	39800	42000	31100
	#5	40700	1630	40700	42000	35800
	#6	40500	1250	40500	42000	36700

1: ($E_f = E_{f,ave}$)

The design tensile modulus is defined by ACI 440.1 (2001) design guide as the average tensile modulus. Table 5.2 shows that Hughes Brothers Inc. publishes a lower than tested tensile modulus. Marshall’s No. 4 bars tested much lower than what they publish. The published data on the No. 5, and No. 6 bars are considered the same or just slightly lower that the tested results. All of Pultrall’s bars tested lower than what is published in their literature. Even so, observing the general trend of this data results in the conclusion that manufacturers are publishing the average tensile modulus.

ACI 440.1 (2001) specifies that the average tensile modulus is equal to the design tensile modulus. The code does not require the subtraction of three standard deviations as with the tensile strength. As a result, there is not a 99.87% probability that all similar bars will meet or exceed the design tensile modulus. This may seem to be an unconservative approach to design, but investigating the effects the modulus has on design may provide insight into the rationale behind this decision.

One area influenced by the modulus is the crack width calculations. The ACI 440.1 (2001) design guide specifies the equation used to determine crack widths, and it is shown in Equation 5.1

$$w = \frac{2.2}{E_f} \beta k_b f_f \sqrt[3]{d_c A} \tag{5.1}$$

Where:

w = Crack width (mm)
E_f = Modulus of elasticity (MPa)
b = ratio of the distance from the neutral axis to the extreme tension fiber to the distance from the neutral axis to the center of the tensile reinforcement
k_b = Bond dependent factor
f_f = Tensile stress in FRP bar (MPa)
d_c = distance from the center of the bar to extreme tension fiber of the effected area, A. (mm)
A = Effective tension area of concrete, defined as the bar spacing multiplied by twice the cover distance (mm^2)

From equation 5.1, it can be seen that the crack width will increase with a decrease in the modulus of elasticity. ACI 440.1 (2001) sets the limit for crack widths at 0.5 mm (0.02 in) for FRP in concrete with exterior exposure. The effect of the modulus can be seen by examining the change in crack width for a typical bridge deck application. The crack width comparison is made assuming a slab depth of 191 mm (7.5 in.), a depth to bar of 38.1 mm (1.5 in.), a factored service moment of 18 kN-m (159 kip-in.), and a service moment of 6.2 kN-mm (54.9 kip-in.). Using the design tensile modulus in Table 5.2 for a Hughes Brothers No. 5 bar, it can be determined that the crack width would increase 0.18 mm (0.007 in.) if the mean modulus minus three standard deviations was used over just the mean. This is a small difference in crack width, especially considering the inherent inaccuracy of the crack width calculations. However, this increase would cause a significant increase in the number of FRP bars needed to control the crack width, and the bar spacing would become impractical. This is also a serviceability issue and not a strength issue, so no failure would occur if the crack limit was exceed due to a bar that did not meet the specified design modulus.

Another area affected by the modulus is the strength reduction factor (ϕ) for flexure. The strength reduction factor (ϕ) is determined by the ratio of the provided reinforcement ratio (ρ_f) compared to the balanced reinforcement ratio (ρ_{fb}). The modulus affects the calculation of the balanced reinforcement ratio (ρ_{fb}). The ACI 440.1 (2001) design guide specifies the balanced reinforcement ratio, and it is given in Equation 5.2.

$$\rho_{fb} = 0.85\,\beta_1 \frac{f'_c}{f_{fu}} \frac{E_f \varepsilon_{cu}}{E_f \varepsilon_{cu} + f_{fu}} \tag{5.2}$$

Where:

β_1 = Concrete strength dependent factor (range 0.65 to 0.85)
f'_c = Concrete strength (MPa)
f_{fu} = Design tensile strength of FRP bar (MPa)
E_f = Design modulus of FRP bar (MPa)
ε_{cu} = Ultimate concrete strain (0.003)

ACI 440.1 (2001) also specifies the criteria for the strength reduction factor (ϕ) for flexure. The strength reduction factor (ϕ) is 0.5 for cases where ρ_f is less than or equal to ρ_{fb} ($\rho_f \leq \rho_{fb}$, $\phi = 0.5$), since an under-reinforced beam would fail due to bar rupture, which is considered to be a very sudden and brittle failure. For cases where the ρ_f is greater than or equal to 1.4 times ρ_{fb} ACI 440.1 (2001) recommends a ϕ factor of 0.70 ($\rho_f > 1.4\rho_{fb}$, $\phi = 0.7$). This failure would be controlled by concrete crushing, which is considered to be somewhat less brittle. For cases where ρ_f is between ρ_{fb} and 1.4

times ρ_{fb}, the ϕ factor recommended by ACI 440.1 (2001) is defined as the ratio of ρ_f to twice ρ_{fb} ($\rho_{fb} < \rho_f \leq 1.4\rho_{fb}$, $\phi = \rho_f/2\rho_{fb}$).

Evaluating Equation 5.2, it can be seen that a larger design modulus will cause a larger balanced reinforcement ration (ρ_{fb}). From the criterion set by ACI 440.1 (2001), a larger ρ_{fb} will result in the use of a smaller ϕ factor, which will cause a greater reduction in the calculated moment capacity of section reinforced with FRP bars. So indirectly, a larger design modulus of elasticity will result in smaller moment capacities for a given cross-section, and reinforcement arrangement.

The ACI 440.1 (2001) design guide recommends the use of the average tensile modulus as the design modulus for concrete reinforced with FRP bars. This specification is slightly unconservative with respect to serviceability issues. Even so, if the mean minus three standard deviations was used for the modulus, in design, the number of bars required would increase significantly while the bar spacing would become so small as to be impractical for use. Using the average modulus for the design modulus has a more conservative effect on strength issues. A larger modulus results in a greater reduction in the calculated moment capacity. Understanding that only serviceability aspects are adversely affected by the use of a larger design modulus of elasticity, and that the structural integrity is not compromised, the conclusion is that using the average modulus for the design modulus is safe and acceptable for use in design.

5.2.4: Bond Behavior

5.2.4.1: Characterization

Each of the manufacturers' bars in the testing program exhibited a different type of bond behavior. Hughes Brothers' bars exhibited a great deal of slip, approximately 4 to 6 mm (0.16 to 0.24 in.) before reaching the peak load, and continued to hold around 80% to 90% of the load after reaching the peak. Marshall's bars reached their peak load within the first 2 mm (0.08 in.) of slip, and then lost the majority (approximately 75%) of the load. Pultrall also reached the peak load with small slips (approximately 1 to 2 mm (0.04 to 0.08 in.)), then continued to hold 60% to 70% of the load as the bar continued to slip. Figure 5.1 shows the typical behavior of all the of the manufacturers' bars.

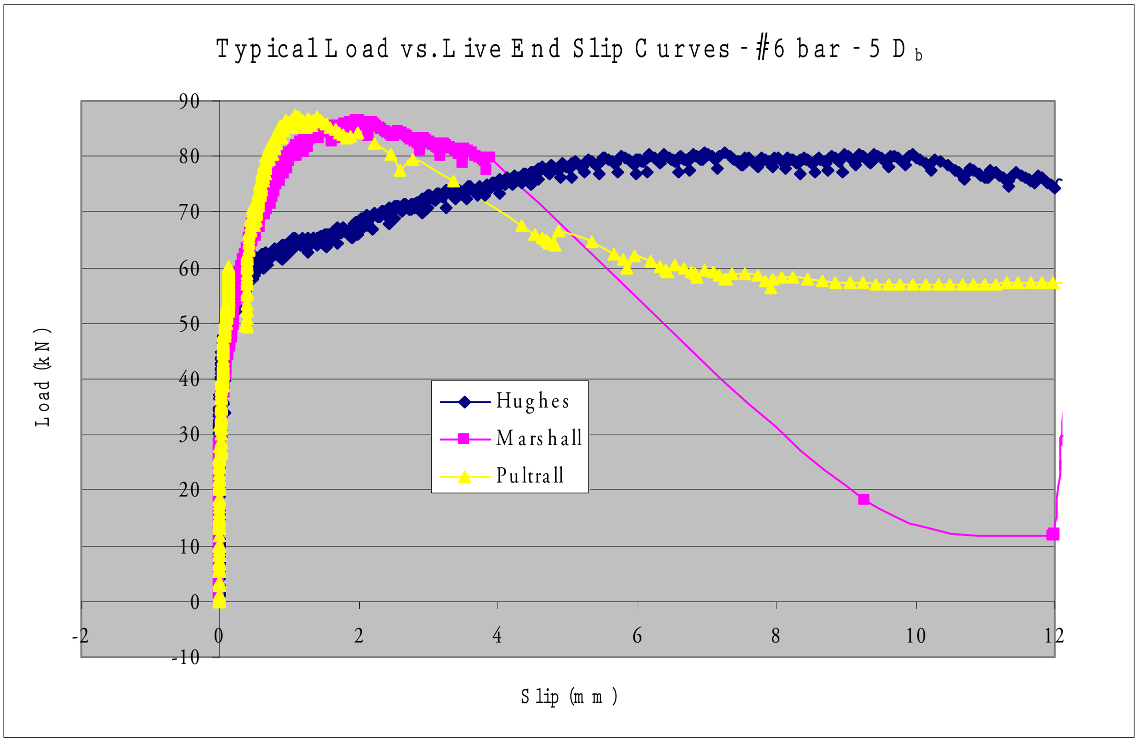

Figure 5.1: Typical load vs. live end slip curves for all manufacturers' bars.

Each of the manufacturers' bars exhibits a distinct peak and post-peak behavior, which will influence the overall performance of the bars. In a failure situation such as a vehicle impact on the barrier rail of a bridge, which could result in bond failure of the FRP bars, Hughes Brothers' bars would excel. Since the bars allow a large amount of slip before the peak is reached, and then continue to hold up to 90% of the load after peak, they could exhibit some degree of ductility in the case of a bond failure. The bars could slip and still hold load, yielding large deflections, and crack widths, giving some warning before impending failure. The same would not hold true for the Pultrall bars. Their bars have a higher peak bond stress than Hughes Brothers' bars, but only hold up to 70% of the load after the peak has been exceeded. In a case were large deflections or bond failures could occur, such as vehicular impact on the barrier rail of a bridge, the Pultrall bars may not have enough post-peak strength to hold the structure together after the damage is incurred. Marshall's bars exhibit the largest drop in load held after the peak load is attained. Since Marshall's bars only maintain approximately 25% of the load after the peak is reached, they would not do well in the case of a vehicle impacting the barrier rail of a bridge. Once the peak load is exceeded the bars would shed the load and failure of the barrier rail and bridge overhang would be quick, and with little warning.

In a typical service situation all of the manufacturers bars perform well. The ACI 440.1 (2001) design guide states that all GFRP bars under sustained or cyclic load shall be limited to 20% of the design tensile strength, to prevent a creep rupture failure. Taking the lowest possible value for the design tensile strength from Table 5.1, which is the Pultrall #6 bar, it can be found that the bar can be stressed to 66 MPa (9.6 ksi). This equates to a 19 kN (4.3 kip) load in the bar. Referring to Figure 5.1 it can be seen that all the bars have similar slips at this load level.

By examining the load-slip behavior in Figure 5.1, different types of failure modes for the three bar types can be inferred. The high peak stress with low slips indicates a shear type of failure in the Marshall and Pultrall bars. This type of failure is indicative of the polymer in the FRP bar failing, not the bar slipping through the concrete. This is antipodal to the behavior of the Hughes Brother's bar in Figure 5.1. Their bar shows a large slip before the peak stress is reached, and then a high post peak stress retention under continued slips. This is indicative of the bar losing bond and slipping through the concrete. This can be confirmed by examining the bars after testing. Figure 5.2 shows all three bars after being pulled out of the block.

Figure 5.2: Post bond test pictures of three manufacturers' bars.

Figure 5.2 shows that sand impregnation of the gray Pultrall bars was lost in the test, and that the polymer holding the sand to the bar failed. The Marshall bars in the middle of the figure show that lugged deformations failed, allowing the bar to slip. This is also a failure in the polymer of the bar. The Hughes Brothers bar shown in the top of Figure 5.2 shows has a small amount of damage to the actual bar, and indicates a failure at the concrete FRP bar interface. This confirms the failure mode inferred from Figure 5.1.

5.2.4.2: Design Effect

Bond behavior, and bond stress impact the design of concrete reinforced with FRP bars in several ways. The bond behavior of FRP bars is taken into account in the control of deflections in the ACI 440.1 (2001) design guide. ACI 440.1 (2001) specifies the formula for the effective moment of inertia (I_e). It is given here in Equation 5.3.

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 \beta_d I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \le I_g \qquad (5.3)$$

Where:

M_{cr} = Cracking moment
M_a = Applied moment
I_g = Gross moment of Inertia
I_{cr} = The cracked moment of Inertia

The factor β_d is also specified by ACI 440.1 and is given here in Equation 5.4

$$\beta_d = \alpha_b \left[\frac{E_f}{E_s} + 1\right] \qquad (5.4)$$

Where:

E_f = The design modulus for FRP
E_s = The modulus for steel

The α_b term given in Equation 5.4 is a bond dependent factor. ACI 440.1 (2001) specifies this number to be 0.5, which is the same for steel. There is no guidance given in the code as to why or how this was chosen. It would seem that there are definite differences in the bond characteristics of steel bars and FRP bars that are not deformed.

Even so, with guidance provided by ACI 440.1 (2001), no recommendations can be given to determine the α_b factors for the bars tested in the program. Until further research can provide greater understanding, designers are left to rely on the judgment of the 440 committee.

The crack width calculations are another area affected by the bond. The crack width equation is specified by ACI 440.1 (2001) is given in Equation 5.1. The bond behavior is accounted for in the bond dependent coefficient, k_b. Little guidance is given as to how to determine k_b for a given bar. The factor, k_b, is taken as one for steel bars according to the ACI 440.1 (2001) design guide. Also dictated by ACI 440.1 (2001) is that bars with superior bond performance to that of steel shall have a k_b value less than one while bars having inferior bond performance shall have a k_b value greater than one. The only recommendation given by ACI 440.1 (2001) is that deformed FRP bars shall have a k_b value of 1.2. With this design code specifications, an obvious recommended k_b value for Marshall's bars, which are deformed, is 1.2. Using specifications given by ACI 440.1 (2001), recommendations can be made on k_b values for the other bars. Referring to the results in Appendix B and Figure 5.1, it can be seen that Marshall and Pultrall exhibit similar bond behavior. On this basis, a k_b of 1.2 is recommended for bars manufactured by Pultrall. The bars produced by Hughes Brother exhibit a different bond behavior than Marshall or Pultrall, although, at lower loads all of the bars behave similarly. Even so, it is recommended that the bars produced by Hughes Brothers use a k_b value of 1.3.

Another area that is largely dependent on the bond behavior is the development length calculations for FRP bars. ACI 440.1 (2001) specifies the equation for the basic development length (l_{bf}), in mm, that comes from equilibrium. This is shown in Equation 5.5.

$$\ell_{bf} = \frac{d_b f_{fu}}{4\mu_f} \tag{5.5}$$

Where:
d_b = diameter of reinforcing bar (mm)
f_{fu} = Design tensile strength of the bar (MPa)
μ_f = Average bond stress (MPa)

Since the average bond stress is a not known in most cases, ACI 440.1 (2001) develops an equation to use for design purposes. The design equation for development length is shown in Equation 5.6.

$$\ell_{bf} = \frac{d_b f_{fu}}{18.5} \tag{5.6}$$

The two equations can be compared using the tested average maximum bond stresses obtained in the testing program. The results of this comparison are shown in Table 5.3.

Table 5.3: A development length comparison using equation 5.5, and 5.6.

Manufacturer	Bar Diameter (D_b) (mm)	Tested Average Max Bond Stress (MPa)	Design Tensile Strength (f_{fu}) (MPa)	l_{bf} from Tested Values (Eqn 5.5) (mm)	l_{bf} from Tested Values (Eqn 5.6) (mm)
Hughes Brothers	12.7	17.1	519	97	356
	15.875	19.1	447	93	384
	19.05	15.4	421	130	434
Marshall	12.7	19.2	475	78	326
	15.875	18.3	392	85	336
	19.05	15.5	413	126	425
Pultrall	12.7	17.2	327	60	225
	15.875	16.2	371	91	319
	19.05	15.4	332	103	342

It can be seen in Table 5.3 that Equation 5.6 yields a development length three to four times larger that of Equation 5.5. This occurs for several reasons. It occurs in part due to the fact that the ACI 440.1 (2001) design guide ensures that the bar will fail due to slipping of the bars vs. cracking or splitting of the concrete. Another reason is the tested average maximum bond strength values were determined from specimens with a short embedment length, and due to the non-linear bond stress distribution found along the bar (Larralde and Silva-Rodriguez, 1993), the values would decrease with an increase in embedment length. Also, the ACI 440.1 (2001) design guide incorporates a factor of safety into Equation 5.6. Regardless of the reasons for the differences, Equation 5.6 presents a very conservative estimate for the development length.

Some manufacturers publish a maximum bond stress in their literature. These numbers could be used in conjunction with equation 5.5 to produce a development length. Table 5.4 shows the development length calculated with Equation 5.5 using the manufacturers published literature. All of the manufacturers' bond stress results came from direct pullout tests, and it is not specified whether the results are averages or lower bounds. If bond stress values are not available (NA) from the manufacturers literature, it will be reflected in the table.

Table 5.4: Development length calculations using published data.

	Bar	Manufacturers	Design Tensile	l_{bf} from

Manufacturer	Diameter (D_b) (mm)	Published Bond Stress (MPa)	Strength (f_{fu}) (MPa)	Published Values (Eqn 5.5) (mm)
Hughes Brothers	12.7	11.6	519	142
	15.875	11.6	447	153
	19.05	11.6	421	173
Marshall	12.7	18.8	475	80
	15.875	14.5	392	107
	19.05	NA	413	NA
Pultrall	12.7	NA	327	NA
	15.875	NA	371	NA
	19.05	NA	332	NA

Upon comparison of Table 5.3, and Table 5.4, it can be seen that the manufacturers' published bond stresses are lower in all cases than the tested maximum average bond stress. This results in longer development lengths, using Equation 5.5. The lower maximum bond stress values published in the manufacturers literature are likely the result of using a longer embedment length, and gathering data from a larger sample size. Even though the development lengths are longer using the published data over the tested data, the design equation for the development length (Equation 5.6, Table 5.3) is still two to three times larger than the development length values obtained using equilibrium, and the published maximum bond stress values (Equation 5.5, Table 5.4). The published maximum bond stress values also show the conservatism of the design equation (Equation 5.6) for the development length. Although it is conservative, the use of Equation 5.6 should bring confidence to any design that the development length criteria is satisfied. Therefore, it is recommended that the design equation (Equation 5.6) be used when determining the development length.

5.2.5: Conclusions

Throughout this chapter the different physical and bond properties were discussed, design values obtained, and recommendations were given based on the conducted tests. The properties investigated included the tensile strength, modulus of elasticity, peak bond stress, and general bond behavior of the bars. These properties influenced several different areas of design including design tensile stress, design modulus, crack width calculation, deflection control, and development length estimation. Each of the investigated properties was related to the relevant area of design. From the developed relationships between the properties, and their design influence, the following general conclusions can be made.

- The design tensile stress specified by the ACI 440.1 (2001) design guide was found to be somewhat conservative, and alternative design approaches were proposed.
- The design modulus specified by ACI 440.1 (2001) was found to be adequate for design.
- No recommendations were given with respect to the bond coefficient for deflection calculations because not enough guidance is given in the ACI 440.1 (2001) guide.
- Recommendations were given for the k_b factor for crack width calculations for each of the tested manufacturers.
- The design development length calculation given in ACI 440.1 (2001) was found to be conservative, and is recommended for use.

5.3: Comparison of Beam End and Direct Pullout Test Methods

5.3.1: Introduction

The ACI 440K (1999) document specifies the standard test method for FRP rod and sheet. One of the standard test methods outlined in this document is the procedure for determining the bond strength of FRP bar. In this procedure, the ACI 440K (1999) document specifies the use of the direct pullout test method, which is explained in section 1.1.2, to determine the bond strength of the bar. The problem with using this test method to determine the bond characteristics is that the concrete around the bar is in compression. This is antipodal to what actually happens in a flexural situation. The concrete around the bar in a realistic bond situation is in tension. For this reason, it is the belief of many researchers that the testing procedure specified by the ACI 440 K (1999) document is not accurate for the true behavior of an FRP rod's strength in bond. Even so, the procedure is adequate for making comparisons or determining the relative bond strengths of different types of bars. A question is: is there enough difference to warrant a more difficult test method. A solution to the problem is changing the test method in the ACI 440K (1999) to specify the used of beam end bond tests, in which the concrete around a bar being tested for bond strength is in tension. This section will compare the maximum average bond stress results found from performing the beam end bond tests in the testing program to other researchers' literature where the direct pullout method was used. From the comparison, conclusions will be drawn on whether the difference is significant enough to warrant a change in the standard testing procedure.

5.3.2: Comparison

Larralde, and Silva-Rodriguez (1993) performed direct pullout tests on one type of No. 5 FRP bar. The bars in the study were glass fiber bars that used a helical wrapping of fiber to produce a spiral indentation to improve the bond. Although there is no mention of the manufacturer of the bars used in Larralde and Silva-Rodriguez's (1993) testing program, the physical description of the bars is similar to those manufactured by Hughes Brothers. Larralde and Silva-Rodriguez (1993) used an embedment length of 76 mm (3 in.). From the testing, they found an average maximum bond stress of 6.68 MPa (0.97 ksi). Referring to Table 4.7, it can be seen that the average maximum bond stress for the Hughes Brothers' No. 5 bars at the 5 bar diameter (D_b) embedment length is approximately 3 times larger than what was found by Larralde and Silva-Rodriguez (1993). One reason for the difference in bond strength could be the sand impregnation that the Hughes Brothers' bars have. There was no mention of sand impregnation for the bars that Larralde and Silva-Rodriguez (1993) tested. Also the amount of indentation was not specified, and this can have an effect the bond performance as well. Even though the surface conditions were similar, they might not have been the same, and this could cause the disparity in the results.

Larralde et al (1994) performed pullout tests using No. 4 FRP bars. The bars used were glass fiber bars with helical wrapping to produce spiral indentations. Although no mention of sand impregnation was made by Larralde et al (1994), the bars described most closely resemble Hughes Brothers bars. The average maximum bond stress of the No. 4 bars tested by Larralde et al (1997) at a 127 mm (5 in) embedment length was 5.9 MPa (0.85 ksi). The closest comparable value in Table 4.7 is the No. 4 Hughes Brothers bar at 7.5 D_b. The value in Table 4.7 is two to three times larger than that reported by Larralde et al (1994). One reason for this difference in performance is the lack of sand impregnation. Another reason is the shorter embedment length of the bars in Table 4.7. Because of the non-linear stress distribution, the shorter embedment length will yield higher bond stress values.

Ehsani et al (1997) performed pullout tests using FRP bars. The bars used in their testing program were made of glass fibers. Their bars were wrapped with a strand of fibers to create a spiral indentation. There was no mention of sand impregnation in the bars. Ehsani et al (1997) tested No. 6 bars at a 152 mm (6 in.) embedment length, and found an average maximum bond stress of 10.3 MPa (1.49 ksi). Comparing this value with that of the Hughes Brothers No. 6 bars at 7.5 D_b in Table 4.7, it can be seen that the values are in closer agreement, although the value from Table 4.7 is still higher. The difference between Ehsani et al (1997), and the results in Table 4.7 can be attributed to the lack of sand impregnation on the bars tested by Ehsani et al (1997). Also, there is some increased bond stress effect from the shorter embedment length.

Chaallal and Benmokrane (1993) performed direct pullout tests using deformed GFRP bars. The deformations to the bars were created by double wrapping glass fibers around the bar, and setting them in a polyester resin. These fibers mimicked deformations, similar to the bars that Marshall manufactures. The bars tested by Chaallal and Benmokrane (1993) were also impregnated with sand to further enhance the bond. Embedment lengths of 5 and 10 D_b were used in their study. Chaallal and Benmokrane (1993) found the average maximum bond stress at 5 D_b to be 15 MPa (2176 psi), 12.5 MPa (1.81 ksi), and 15.1 MPa (2.19 ksi) for the No. 4, 5, and 6 bars respectively. Referring to Marshall's No. 4, 5, and 6 bar bond strengths at 5 D_b in Table 4.7 it can be seen that the values in Chaallal and Benmokrane's (1993) study are lower. It can also be seen in comparison that the values follow the same trend of decreasing in bond strength between the No. 4, and No.5 bars, and then increasing again between the No. 5, and No.6 bars. The lower bond strength in Chaallal and Benmokrane's (1993) study may be a result of the adding of the deformations to the bar instead of molding them in the bar. In their study the deformations were added to the bar after it was made, by bonding glass fibers to a smooth bar. The deformations on the Marshall bars are actually formed into the bar as it is being made. This would intuitively seem stronger, even without the addition of sand impregnation.

Katz (1999) performed a study on multiple types of No. 4 FRP bars. Two of the types of bars are comparable to the bars in Table 4.7. One of the bars tested by Katz (1999) is impregnated with sand, and has a helical wrapping of fibers to improve the bond. These are similar to Hughes Brothers bars, and are designated R1. The other has deformation molded into the bar. This is very similar to Marshall's bars, and are designated R4, following Katz's (1999) nomenclature. Katz (1999) used 60 mm (≈2.5 in.) as an embedment length. He found maximum bond stresses of 13.8 MPa (2.0 ksi) for the R1 bars, and 14.6 MPa (2.12 ksi) for the R4 bars. Referring to Table 4.7 at the Hughes brothers, and Marshall No. 4 bars at 5 D_b, it can be seen both manufacturers' bars exhibited a higher maximum bond stress. Even though these bar types are similar, the manufacturer was not given by Katz (1999), and it cannot be concluded that the bars are an exact match.

5.3.3: Conclusions

The values for maximum (or peak) bond stress measured in this study are higher than the values in all of the literature reviewed for this comparison. Although some of the bars reviewed may not have been an exact match to the bars tested in this program, they were comparable. Generally, it is thought that concrete in compression around the rebar will hinder natural crack development, thus resulting in larger bond strengths. That was not found to be the case upon comparison of the results in Table 4.7 with the given literature. As a result, no clear conclusion can be drawn as to which testing procedure is better. However, practitioners should recognize that although the direct pullout method can be used for general comparisons of bond slip behavior, beam end bond tests more accurately represent actual flexural behavior.

5.4: Conclusions and Recommendations for Further Research

It is clear that material and bond properties of FRP reinforcement have a significant effect on the design of a bridge deck. The tensile strength, modulus, peak bond stress and bond behavior have impacts in several areas. These areas include the design tensile strength, the design modulus, deflection control, crack width calculations, and development length estimations. Each area was given considerable review and evaluation, and recommendations were made based on the results from the testing program, and the ACI 440.1(2001) design guide.

A comparison was made between literature that contained bond tests using the direct pullout method, which is currently specified in the ACI 440K (1999) document, and bond tests performed in the testing program (beam end bond test method) to see if there were significant differences in results from the two testing methods. No clear conclusions can be made from the comparison because all of the values from the testing program were higher than all of the values in the reviewed literature.

From the evaluation of the results, and the specifications set by the ACI 440.1 (2001) design guide, further research is needed to better understand crack width and deflection behavior. No guidance is given by ACI 440.1 (2001) design guide in relation to the α_b (bond dependent) factor used in deflection calculations, and very little guidance is given for the k_b (bond dependent) factor used in crack width calculations. More beam end bond test should be performed using different types of FRP bars at different embedment lengths, and these results should be compared to those of normal mild steel bars under the same conditions. The results from these tests would aid in determining criteria to more accurately define α_b, and k_b bond dependent values for FRP bars.

It became clear when reviewing the literature for the comparison between the direct pullout method, and the tested results from the beam end bond test method, that it was difficult to find comparable results. To accurately determine whether there is a need to reconsider the standard testing procedure outlined in the ACI 440K (1999) document, further research is needed. Bond strength testing should be conducted using both the direct pullout method, and the beam end bond test method, in the same testing program. Both types of specimens should use bars from the same manufacturer, and the same lot of production. Also, both types of specimens should use the same embedment lengths, the same concrete strength, and be cast at same time. Uniformity should be emphasized in all possible factors, to ensure that the only variable is the testing procedure.