CHAPTER II
LITERATURE REVIEW

2.1: Direct pullout bond tests of FRP reinforcement

Many bond test studies have been done with FRP bars. Most of these studies used the direct pullout method. This method consists embedding rebar a specific distance into a concrete cylinder, usually a 152 mm (6 in.) x 305 mm (12 in.), or a concrete block. Once cured, the bar is pulled out using a universal testing machine or hydraulic ram, while displacements and loads are measured. Although this is a common practice for determining bond behavior, it is widely believed that this method will yield unconservative bond stress values.

Larralde and Silva-Rodriguez (1993) did a study with No. 3 bars (9.5 mm (3/8 in.)), and No. 5 (15.9 mm (5/8 in.)) FRP bars, and compared their results with steel bars of the same size. A total of six No. 3 bars at two different embedment lengths, 76 mm (3 in.), and 152 mm (6 in.), and six No. 5 bars at the same two-embedment lengths were tested. They used 152mm (6 in.) by 305mm (12 in.) concrete cylinders for the test. The procedure was repeated for the steel bars with fewer repetitions used.

Larralde and Silva-Rodriguez (1993) found that the larger embedment lengths yielded an overall smaller maximum bond stress. For the No. 3 bars at the 76.2 mm (3 in.) embedment length, the maximum average bond stress was 9.96 MPa (1.44 ksi). The No. 3 bars embedded at 152 mm (6 in.) gave a maximum average bond stress of 9.31 MPa (1.35 ksi). The No. 5 bars exuded the same behavior as the No. 3 bars with regards to the embedment length. The 76 mm (3 in.) embedment length for the No. 5 bars gave an average maximum bond stress of 6.68 MPa (0.97 ksi), and for the 152 mm (6 in.)

embedment length; the maximum average bond stress was 5.90 MPa (0.86 ksi). Larralde and Silva-Rodriguez (1993) determined that the reduction in bond stress due to an increased embedment length was due a nonlinear bond stress distribution. They compared their FRP data with their similar steel data and concluded that FRP has a lower bond strength and higher slip at failure than conventional steel reinforcement.

Brown and Bartholomew (1993) tested No. 3 bars, from two manufacturers, to try and characterize the bars' bond behavior. A total of 12 tests were completed at a concrete strength of 29 MPa (4200psi), and embedment lengths of 102 mm (4 in.) and 152 mm (6 in.) were used. The bars were cast into standard 152 x 305 mm (6 x 12 in.) concrete cylinders. They found that the bond stress increased with the longer embedment length. The average maximum bond stress for the 102 mm (4 in.) embedment length was 6.8 MPa (0.99 ksi), and for the 152 mm (6 in.) embedment length, 8.2 MPa (1.20 ksi). They also compared their bond stress values with that of steel and found that the FRP reinforcing bars had approximately two-thirds the strength of steel bars.

Chaallal, and Benmokrane (1993) performed another study using the direct pullout method. Twenty four tests were performed, using 30 MPa (4.35 ksi) concrete, on No. 4, 5, and 6 FRP bars, at embedment lengths of five and ten times the bar diameters. Four tests were done per bar size, per embedment length. The bond strength of the bars ranged from 11.1 MPa (1.61 ksi) to 15.1 MPa (2.19 ksi) with an overall average bond stress for all the bars of 12.9 MPa (1.87 ksi). Even though it goes unmentioned in the paper, it should be noted that the bond strength was lower for the larger embedment lengths. Also, through analysis of the results, Chaallal and Benmokrane (1993) found that development length required to develop the capacity of the bar is 20 bar diameters.

Ehsani, et al. (1997) conducted a study that compared bond behavior of FRP using the direct pullout method to FRP using beam tests. Eighteen pullout specimens with varying rebar sizes (No. 3, 6, and 9 FRP bars), and embedment lengths were tested, and compared to 48 beam tests. Their direct pullout specimens consisted of casting three bars in 1219 mm (48 in.) x 762 mm (30 in.) x 406 mm (16 in.) concrete block. After curing (28 days) they applied load directly to the block through a hydraulic ram and read live and free end measurements with dial gages. Upon comparison of the results, Ehsani, et al. (1997) found that the direct pullout method yielded non-conservative development lengths. Analyzing a sampling of their data, they found the ultimate bond stresses increased by an average of 13% when the direct pullout test was used. From this comparison, Ehsani, et al. (1997) concluded that beam tests should be used to accurately model the bond behavior of reinforcement in concrete.

Larralde, et al (1994) performed a series of tests using the direct pullout method with No. 4 bars in 152 mm (6 in.) x 304 mm (12 in.) cylinders. They used embedment lengths of 127 mm (5 in.), 178 mm (7in.), 229 mm (9 in.), and 279 mm (11 in.). They found that failure, for the 127 mm (5 in.) embedment length occurred through longitudinal and radial cracks formed in the cylinder allowing the bar to be pulled out. Failure occurred for the 279 mm (11 in.) embedment length by splitting of the concrete cylinder. The 178 mm (7 in.) and the 229 mm (9 in.) embedment lengths failed by either of the previously mentioned modes. Results for their pullout tests are given in tabular format in the literature. The table indicates that FRP generally exhibits a decrease in nominal bond strength for an increase in embedment length. Upon analysis of the results, Larralde, et al (1994) concluded that a nonlinear bond stress distribution exists between the concrete and the bar. According to their results, the majority of the stress is taken by the concrete surface near the loaded end of the cylinder.

Al-Zahrani, et al. (1999) conducted a study that investigated bond failure modes of axisymmetrically lugged FRP bars. Fifty-one tests were conducted using the direct pullout method. The FRP bars were cast into a 150 mm (5.9 in.) cube and embedment lengths were either five or ten times the bar diameter. They found that all of the bars failed due the machined lugs being sheared off. It was noted that the ten bar diameter embedment lengths had 25% less bond strength than the five bar diameter embedment lengths. Al-Zahrani, et al. (1999) believe that a non-linear bond stress distribution exists along the embedment length. They also investigated whether the height of the lugs affected the bond strength, and concluded that it did not. Although, it was found that the width of the lug could have significant impact on the bond strength. Lastly, they concluded that the fiber type controlled the bond strength of FRP rods.

Katz (1999) performed a study comparing five different types of FRP bars with each other and a mild steel bar for bond strength characterization, using the direct pullout method. Rods were embedded 60 mm (2.4 in.) in a 150 x 150 x 120 mm (5.9 x 5.9 x 4.7 in.) concrete block, which was cast horizontally. Katz (1999) found that the FRP bars with mechanical deformations performed the best with an average bond stress of 14.6 MPa (2.12 ksi). The next best performance was by the FRP bars with helical wrapping and sand impregnation (small particles). They had an average bond stress at 13.8 MPa (2 ksi), followed by plain helical wrapped bars at 12.2 MPa (1.77 ksi). The Helically wrapped sand impregnated bars (large particles) performed poorly, 4 MPa (0.58 ksi), because of a 0.5 to 1 mm (0.02 to 0.04 in.) gap between the concrete and the bar. The gap was “Probably the result of dissolution of some of the polymer at the surface, which prevented hydration of the cement in this region” (Katz, 1999). Also, the plain, smooth FRP bar performed poorly, 1 MPa (0.15 psi), because it had no surface enhancements at all. Lastly the steel bar performed just below the three well performing FRP bars at 12.1 MPa (1.76 ksi).

In this same study, Katz (1999) looked at the pre-peak and post-peak behavior of the bars. The bars with helical wrapping and/or fine sand impregnation showed good pre-peak and post-peak ductility. They were able to maintain all or the majority of their load while slipping after peaking. Conversely, the mechanically deformed bars (FRP and steel) showed brittle pre-peak and post-peak behavior. They lost the majority of their load after peaking with continued slip. Also higher slips were obtained with the more ductile bars before the peak was reached.

Chaallal and Benmokrane (1995) performed direct pullout according to ASTM C234, using glass fiber reinforced polymer bars (GFRP) bars, and mild steel bars. From the pullout tests, Chaallal and Benmokrane (1995) found a nonlinear tensile and bond stress distributions for both GFRP, and steel bars. They found both of the distributions to be exponential along the embedment length. The tensile stress distribution increased to the ultimate tensile strength of the bar, and then moved down the embedment length to the free end, with an increase in load. The same holds true for the bond stress, but the corresponding maximum bond stress moved further down the embedment length due to progressive failure along the bar. Also obtained from the direct pullout tests were development lengths required to reach the ultimate tensile strength of the GFRP, and steel bars. The ultimate tensile strength was set at 690 MPa (100 ksi) for the GFRP bars, and

480 MPa (69.6 ksi) for the steel. The development lengths for the No. 4 bars, were 270 mm (10.6 in.) (21 bar diameters) for the GFRP, and 130 mm (5.12 in.) (10 bar diameters) for the steel.

2.2 Beam End Bond Tests

Beam end bond tests are another bond test method, however, in this test the concrete around the bar is in tension. These tests usually have two pieces of rebar cast in opposite sides of a rectangular block. The bars run parallel to the long side of the rectangle, and have enough concrete cover on all sides, so that splitting does not become a factor. Loads are applied to one of the rebar specimens and horizontal and vertical reactions are applied at the bottom corner of the loaded sided. Another reaction is applied vertically at the top on the free end to ensure the block remains level. Loads are measured, and displacement readings are usually taken at the loaded and free end of the bars.

Clark and Johnston (1983) performed a study using beam end bond tests. They examined No. 6 steel reinforcing bars at embedment lengths of 203 mm (8 in.), 305 mm (12 in.), and 406 mm (16 in.). At each embedment length, one bar was loaded at one day, and one bar at 28 days to failure. Also, one bar was loaded to a working a stress at one day, and held for 27 days, then loaded to failure. They found an ultimate bond stress of 5.28 MPa (0.77 ksi) for the 203 mm (8 in.) embedment length, 6.34 MPa (0.92 ksi) for the 305 mm (12 in.) embedment length, and 5.65 MPa (0.82 ksi) for the 406 mm (16 in.) embedment length. Clark and Johnston (1983) explain the variation (increase, then decrease of bond stress with embedment length) of the results by their belief that high-localized stresses are encountered to a certain point and after that point, a decrease in bond stress will begin to be seen. Other results show that earlier loaded bars (load applied at one day) had higher slip values than those loaded at 28 days. Also, the working stress bars had the same ultimate strength as the 28-day bars, with higher slips at failure.

Clark and Johnston (1983) concluded that controlled early loading resulted in no detrimental effects on the ultimate concrete bond strength. Also that early loading results in higher slips at service loads

Mathey and Watstein (1961) conducted a study on high-yield-strength reinforcing bars. Eighteen pullout specimens were tested and compared to 18 beam specimens using 690 MPa (100 ksi) yield-strength deformed steel bars. The full-scale beam tests had cross-sectional dimensions of 203 mm (8 in.) x 457 mm (18 in.), and spanned 2240 mm (88 in.). Only No. 4 and 8 bars were investigated for the program, and embedment lengths ranged from 178 mm (7 in.) to 432 mm (17 in.) for the No. 4 bars, and 178 mm (7 in.) to 864 mm (34 in.) for the No. 8 bars. Maximum bond stresses for the beam end tests ranged from 4.1 MPa (0.6 ksi) to 11.3 MPa (1.64 ksi), with maximum free end slips ranging from 0.05 mm (0.002 in.) to 19.1 mm (.75 in.). Mathey and Watstein (1961) found that the bond strength decreased with an increase in embedment length for a given bar size, as well as decreasing for an increase in bar diameter. Also, they found that pullout specimens yielded higher bond strength values, than the beam specimens.

Johnston and Zia (1982) performed a series of beam end bond tests on epoxy coated reinforcing bars to compare their performance with normal mill scale bars. A total of 26 static beam end specimens were tested. No. 6 bars were used at 203 mm (8 in.), 330 mm (13 in.), and 457 mm (18 in.) embedment lengths, and No. 11 bars were used at

406 mm (16 in.), 610 mm (24 in.), and 762 mm (30 in.) embedment lengths. The average bond stress for the No. 6 bars corresponding respectively to the previously given embedment lengths are 12.2 MPa (1.76 ksi), 7.99 MPa (1.16 ksi), and 6.02 MPa (873 ksi). Likewise the No. 11 average bond stresses are respectively 9.60 MPa (1.39 ksi), 7.75 MPa (1.12 ksi), and 6.23 MPa (0.90 ksi). Johnston and Zia (1982) found that the epoxy coated rebar exhibited higher slips than mill scale bars at lower loads. Also, cracks in the concrete formed sooner and the pullouts for the epoxy-coated bars happened earlier. Johnston and Zia (1982) concluded that epoxy coated bars have less strength, slip resistance, and form cracks sooner than normal mill scale bars. Also, both the epoxy-coated and mild steel bars decreased bond strength with increased embedment length, and bars size, so the epoxy coating has no effect on those parameters. Johnston and Zia (1982) recommend a 15% increase in development length to compensate for the reduced performance of the epoxy coated bars.

Chaallal and Benmokrane (1995) conducted an extensive study on glass fiber reinforced polymer (GFRP) rebar for concrete applications. Pultrall, Canada supplied the bars tested throughout the study. Initially, Chaallal and Benmokrane (1995) performed several types of tests to characterize the many physical properties of the bar. Included in their characterization were tensile strength and modulus testing of the bars, determined according to ASTM D638, and D695. They found a mean ultimate tensile strength of 689 MPa (99.9 ksi), and mean modulus of elasticity of 42 GPa (6100 ksi)

The next phase of testing included direct pullout tests, and beam end bond tests. The beam end tests were performed on steel and GFRP bars and done in accordance with RILEM (RELIM/CEB/FIP, 1978). Chaallal and Benmokrane (1995) used No. 4, 5, 6, and

8 bars, with the embedment length held constant at ten bar diameters. Chaallal and Benmokrane (1995) found that the bond strength of the bars decreases with an increase in bar diameter. Also the maximum bond strength for GFRP bars is approximately 60- 90% of that of steel bars, depending on the bar diameter.

The final part of Chaallal and Benmokrane's (1995) study consisted of testing eight concrete beams for cracking moment, and ultimate moment capacities. All beams were 3300 mm (130 in.) long. Two beams (Series 1) were reinforced with GFRP bars and had cross-sectional dimensions of 200 x 300 mm (8 x 12 in.), and two beams (Series 2) reinforced with GFRP bars had cross-sectional dimensions of 200 x 550 mm (8 x 22 in.). The other four beams were companion beams to the four beams reinforced with GFRP. They were dimensionally the same, only the GFRP bars replaced with mild steel bars. It was found that the ultimate moments of the steel and GFRP beams were comparable. "This is attributed to the fact that the ultimate capacities are largely related to ultimate strength of reinforcement, and the ultimate strength of GFRP and steel bars are similar (690 and 600 MPa, respectively)" (Chaallal and Benmokrane, 1995). However, Chaallal and Benmokrane (1995) found that Series 1 beams showed four times the deflections as their companion steel reinforced beams. The Series 2 beams only showed three times the deflections as their companion steel reinforced beams. It was also noted that the steel reinforced beams failed due to tension in the rebar, while the GFRP reinforced beams failed due to crushing of the concrete. As a result, Chaallal and Benmokrane (1995) determined that extending the depth would help to control deflections, and develop the full strength of the GFRP bars.

Tighiouart, et al (1998) also conducted a study using GFRP rebar in beam end bond specimens. A total of 64 beam end tests were carried out in accordance to RILEM specifications. No. 4, 5, 6, and 8 bars were tested at embedment lengths of six, ten and sixteen bar diameters. They found that GFRP bars had lower bond strength values than that of steel bars, and that GFRP bars develop most of their bond strength from adhesion, and friction instead of bearing, as with steel bars. Tighiouart, et al (1998) also found that the maximum bond strength of both types of rebar decreases when the diameter of the rebar increases. The authors explain that bleeding of water in the concrete causes the larger diameter bars to have smaller bond strengths. They believe that a higher quantity of bleeding water gets trapped under the larger diameter bars creating a larger void space, thus reducing the bond strength. Tighiouart, et al (1998) also found that as the embedment length increases, the load approaches the tensile strength of the bars, and the average bond strength of the bars goes down. The authors explain the decrease in bond strength by a non-linear stress distribution along the embedment length of the bar.

2.3 Summary

Some of the major aspects of the literature reviewed from this chapter can be compiled in table form for comparison purposes. Table 2.1 is a table of bond stresses by author, bar size, and embedment length for the direct pullout bond test method.

Table 2.1: Summary of literature reviewed for direct pullout test method.

Author	Year	Bar size	Embedment Lengths (mm)	Max Bond Stress (MPa)
Larralde & Silva-Rodriguez	1993	3	76.2	9.96
			152	9.31
		5	76.2	6.68
			152	5.9
Brown & Bartholomew	1993	3	102	6.8
			152	8.2
Chaallal & Benmokrane	1993	4	62.5	15
			125	11.1
		5	75	12.5
			150	11.4
		6	90	15.1
			180	12.2
Ehsani, et al.	1997	3	38	32.3
		6	152	10.3
		9	203	10.7
Larralde, et al.	1994	4	127	5.88
			178	6.41
			229	6.04
			280	4.55

Table 2.2 shows the bond stresses by author, bar size, and embedment length for the beam end bond test method.

Table 2.2: Summary of literature reviewed for beam end test method.

Author	Year	Bar size	Embedment Lengths (mm)	Max Bond Stress (MPa)
Clark & Johnston[1]	1983	6	203	5.28
			305	6.34
			406	5.65
Mathey &Watstein[1]	1961	4	178	11.1
			267	9.34
			356	6.79
			432	6.25
		8	178	7.05
			356	4.68
			533	4.73
			711	4.6
			864	4.6
Johnston & Zia[1]	1982	6	203	12.2
			330	7.99
			457	6.02
		11	406	9.6
			610	7.75
			762	6.23
Chaallal & Benmokrane	1995	4	127	10.6
		5	159	7.3
		6	191	6.6
		8	254	6.4
Tighiouart, et al.	1998	4	76	11.3
			127	10.6
			203	8.7
		5	95	10.6
			159	7.3
		6	115	7.1
			191	6.6
			306	5.3
		8	152	7
			254	6.4
			406	5.1

1: Bars tested are steel instead of FRP

All of the previous literature varied somewhat on the details and results of their specific type of bond tests. Even so, there are some basic trends in the literature that pertain to all of the previous work done. It is apparent from previous literature that the average bond stress decreases with larger bar diameters. Different explanations exist for the decrease in bond stress due to the larger bar diameters. Another trend is the average bond stress decreases as the embedment length increases. It is generally thought that the decrease in bond stress with increase in embedment length is due to the nonlinear stress distribution that exists between the bar and the concrete. Still another trend in previous literature is the consensus that beam end bond tests yield more accurate results, than the direct pullout test method.

It can be seen from examining the previous literature that a wide range of bond stress values exist depending on the type of test method, and the type of bar tested. The range of bond stresses that exist for steel rebar is not as vast at for FRP rebar due to the fact that most mild steel bars have the same type of surface deformations and more standardized material properties. The bond stresses for the steel rebar ranged from 4.1 MPa (0.6 ksi) to 12.2 MPa (1.76 ksi). The bond stresses for the FRP rebar ranged from 1 MPa (0.15 ksi) to 15.1 MPa (2.19 ksi). The wider range of bond stresses is a direct result of non-standardized material properties, and different surface treatments to improve the bond. So, even though mild steel and FRP bars exhibit the same general bond characteristics, the FRP bars have a much wider range of bond stresses due to the surface conditions and non-standardized material properties.