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Bond and Low Cycle Fatigue Behavior of Thermoset Composite
Reinforcing for the Concrete Industry

by

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PREFACE

This thesis encompasses two separate research projects. The first project, described in Chapter 2 was a project investigating the fatigue behavior of thermoset Fiber Composite (FC) sandwich wall ties. This project was funded by Dow Chemical and Composite Technologies Corporation (CTC). CTC supplied the Styrofoam[®] insulation board used and the glass-fiber ties. This work was performed from August 1989 to June 1990.

The second research project detailed in this thesis was a project studying the bond and tensile properties of FC rod and FC fibers. This project was funded by the United States Department of Commerce. All of the FC materials used in this project were supplied by CTC. Work on this project started officially in July of 1989. However, several pilot tests were performed prior to this date.

1 BACKGROUND

The research documented in this thesis encompasses low-cycle fatigue behavior of Fiber-Composite (FC) sandwich wall ties exposed to low temperatures and the bond behavior of FC reinforcing rod. The fatigue work on the FC ties was performed to document the resistance of this wall system to thermal induced loads (thermocycle loading). The bond experiments were performed to document the flexural-bond and tensile behavior of the FC material when used as primary tensile reinforcing in concrete members. Additional work was performed on Random Dispersed Composite Reinforcing (RDCR) to document the bond strength of the FC material when added to concrete in a short chopped form.

Previous investigations into the bond characteristics of FC materials were studied at the beginning of this project. Since this project is on the leading edge of research in this field, few applicable references were available to the author at the time of this thesis.

Most of the testing was conducted on a pilot level with the exception of the thermocycle work. This project was a test program investigating a specific aspect of the behavior of the FC ties. Time did not permit a full literature survey for this project. The references used in the thermocycle work were References 2, 18, and 20. Reference 2 was used to

determine the maximum temperature differential for the thermocycle testing. Reference 18 applied to the thermocouple calibration. Previous work at Iowa State University (ISU) provided the material properties pertaining to strength and corrosion resistance (20). The ISU corrosion test results showed that the material in these FC rods was not seriously affected by aging or severe alkalinity (20).

References cited for the bond study are somewhat dated since the majority of fundamental bond studies were prior to 1960. The literature found which directly pertains to the FC's bonding properties (References 7, 13, and 14) were not available to the author at the time of this thesis. These papers were in the process of being published and should be obtained as soon as possible since they deal directly with the determination of the bond between concrete and FC rods.

All other references were used as a base line study for the pilot tests. Some of these references were used to study previous work performed in bond-stress distribution along reinforcing rods (References 6, 15, 17, and 19). These references documented the internal bond stress distribution and enhanced this author's understanding of the test specimen behavior by documenting the relationship between crack spacings and bond stress.

Furthermore, some of these references were used to determine a bond test configuration and setup (References 5,

6, 11, 12, and 16). The use of these articles in determining a potential bond test scheme is discussed in full at the end of Chapter 5. All of these articles were used to gain some insight into the bond studies of the past and to determine an appropriate testing procedure for new materials.

Literature applying to the tensile testing portion of this thesis included References 8 and 20. These references were used as guide in the tensile testing portion of our study. These references documented different types of grips for the FC material and the problems associated with tensile testing.

2 THERMOCYCLE TESTING

This research study was undertaken to document the behavior of the Composite Technologies Corporation and Dow Chemical Company sandwich wall system when exposed to blast freezer temperatures. Some of the constructed wall panels had lengths up to 42 feet. The combination of this length coupled with extreme temperatures inside and a range of temperatures outside prompted Composite Technologies Corporation and Dow Chemical Company to investigate the performance of their fiber composite sandwich wall tie system under shear loads.

2.1 Test Objectives

The object of this study was to determine the effects of cold temperatures on the low-cycle fatigue strength of fiber composite ties subjected to in-plane shear loads. This project encompassed twelve tests designed to document any potential change in low-cycle fatigue strength of the Composite Technologies and Dow Chemical wall tie system. Data collected included a temperature distribution across the cross section, load, deflection, tie orientation, and number of cycles at each data point. Important variables included temperature, deflection, and load frequency.

2.2 Selection of Parameters

The selection of the test parameters was based upon the relative deflection of the outer and inner wythes, the number of thermocycles expected throughout the building's life, and the face temperatures on the interior surface and the exterior surface.

Two types of construction procedures were considered in the determination of the test parameters. These methods, precast, which is tilted up into place, and cast in place, induce different loads on the walls as they are placed as shown in Figure 2.1. These loads, however, would not affect the performance of the wall system after it was placed.

Figure 2.1. Construction methods

The temperature parameters were investigated to determine if the difference in temperatures between casting

and placement of the panel had any effect on the wall system's performance. Since the temperature variation between casting and placement usually induce equal displacements in the panel, the effect of this parameter was ruled out. However, the variation of the temperature of the outside wythe could not be ruled out since it produces a linear expansion or contraction of the outer surface. This thermal induced movement causes the top of the outside wythe to deflect up or down relative to the inner wythe. This relative deflection coupled with the freezer temperatures was the focus of this project.

In order to determine the relative deflection between wythes, a temperature range had to be established. This range of temperatures was the difference between summer highs and winter lows. Locations around the world were investigated and the results are in Table 2.1. The data displayed were obtained from the ASHRE Handbook & Product Directory (2). The data in this book were much more complete for the summer than for the winter. A mean daily range and dry bulb temperatures were listed in the handbook for the summer months while just a dry bulb temperature was listed for the winter. The mean daily range represents the average difference between the daily high and low. The temperatures were split into winter and summer groups which were then separated by the percentage chance that the daily highs

Table 2.1. Weather data for temperature range (2)

Location	Temperature Difference ^a F°
Winnipeg, Manitoba Canada	113
Calgary, Alberta Canada	104
Ottawa, Ontario Canada	100
Ames, Iowa USA	96
Toronto, Ontario Canada	88
Moscow, USSR	87
Anchorage, Alaska USA	86
Lyon, France	75
Phoenix, Arizona USA	73
Berlin, Germany	69
Riyadh, Saudi Arabia	68
Vancouver, B.C. Canada	58
Los Angeles, California USA	49
San Francisco, California USA	39
San Diego, California USA	36

^aBased upon the number of hours per year that the average daily temperature could be expected to be above or below the 97.5%-2.5% interval for average daily temperatures.

and lows would fall above or below the listed temperature. In addition to this, the handbook also gave the number of hours that the temperatures above or below the percentage would be expected to occur during the season. From this information, a temperature range and a number of cycles were computed. The number of cycles was based on the assumption that the temperature remains at the maximum or minimum temperature for three hours. From this, a number of cycles per year was calculated. The data in Table 2.1 represent the temperature range and its associated maximum number of cycles over a 50-year building life.

The final values for the deflection and number of load cycles were determined from the data in Table 2.1. The temperature range was determined to be 160 degrees Fahrenheit by algebraically adding the 113-degree number in Table 2.1 to the minimum freezer temperature of -40 degrees Fahrenheit and rounding conservatively upward to 160 degrees Fahrenheit. This 160-degree temperature was applied to a tie at the top of the wall panel. This tie represents the worst case for deflection and undergoes a 0.45-inch deflection based upon Equation 2.1 and the assumed values shown.

$$\Delta L = \alpha (\Delta T) L$$

Eq. 2.1

Equation 2.1 is based upon a value for α of 5.5×10^{-6} , a value of ΔT of 160°F , and a length of 42 feet. The number of cycles was determined to be 2000 by taking the 900 cycles indicated for the data in Table 2.1, times a factor of safety of two and rounding conservatively upward to 2000 cycles. The face temperatures were selected to be -40° degrees Fahrenheit on the outside surfaces and room temperature on the inside sandwich portions (Figure 2.2). The extra wythe of concrete shown in this figure was added for ease of testing.

Figure 2.2. Temperature distribution

2.3 Materials

Each specimen for this test was composed of concrete, STYROFOAM[®] insulation board, and fiber composite ties. The material properties and engineering properties of the insulation board and the fiber composite ties were obtained from previous work on these materials by G. Thomas Wade, Max L. Porter and Daniel R. Jacobs in a report titled Glass-fiber Composite Connectors for Insulated Concrete Sandwich Walls (20). A complete listing of the engineering properties of these two materials can be found in Tables 2.2 and 2.3. The concrete for this test had an average compressive strength of 3,512 psi as determined by eight compression tests on six-inch diameter cylinders that were twelve inches tall. This concrete was supplied by a local ready-mix plant.

Table 2.2. Engineering properties for rod stock (20)

Property	Rod Stock
Width (inch)	0.354
Depth (inch)	0.187
Area (inch ²)	0.0654
Moment of Inertia (inch ⁴ ×10 ⁻⁴)	
Strong Axis	5.49
Weak Axis	0.61
Tensile Modulus (ksi)	5800
Tensile Strength (ksi)	122
Poisson's Ratio	0.26-0.29

Table 2.3. Engineering properties for Styrofoam insulation
(20)

Property	Insulation Board
Apparent Density (pcf)	2.11
R-Value (per inch, 2 inch thick)	5.00
Initial Compressive Modulus (psi)	1700
Post-yield Compressive Modulus (psi)	166
Compressive Strength at 5% Deformation (psi)	35.9
Compressive Strength at 10% Deformation (psi)	44.2
Yield Stress (psi)	30.6
Shear Modulus (psi)	1100
At Stress Level (psi)	20.8
Shear Strength (psi)	34.6

Figure 2.3. Displaced specimen

2.4 Test Specimens

In order to provide thermodynamic symmetry and a simple testing method, the specimens for this test had three wythes of concrete. The central wythe was at room temperature while the two outer wythes were at -40 degrees Fahrenheit. The center wythe was also cycled to the 0.45-inch relative deflection with the outer two wythes restrained as shown in Figure 3. This test setup provided a sample which when loaded into the MTS 110-Kip testing frame, had two accessible faces for refrigeration and two wythes for support points.

The specimens were composed of three wythes of concrete each measuring 16x32x4 inches. These wythes were separated by two layers of three-inch STYROFOAM[®] brand insulation board and were connected with two fiber composite ties per wythe. The ties were located as shown in Figure 2.4 and had dimensions as shown in Figure 2.5. Once loaded, the specimen's two exposed faces were cooled to -40 degrees with an aziotropic mixture of Chlorofluoromethane R22 and Chloropenta-fluoroethane R115 refrigerant. This mixture, commonly called R502, was compressed and circulated by a portable one horsepower compressor. The maximum possible low temperature achieved by the refrigerated cold plates without a thermal load was determined by surrounding the plates with six inches of foam insulation and recording the lowest

Figure 2.4. Typical specimen dimensions

Figure 2.5. Typical tie dimensions

temperature obtained over a three-day period with T type thermocouples. The temperatures obtained were calibrated and verified by comparing the thermocouple data with a calibrated meter supplied by the refrigeration specialist who designed the system. Under these conditions, these plates were capable of producing -65 degrees Fahrenheit without a thermal load. When placed against the concrete faces in the test configuration, these plates were capable of producing -40 degrees Fahrenheit at the outer surface of the concrete in a 24-hour period. Since the plate to concrete interface was not smooth and the heat transfer mechanism was conduction, a thermoplastic putty was used to even out the surfaces and provide an adequate contact area for the heat transfer.

2.5 Instrumentation

The data collected in this project by a central computer required the use of different types of instrumentation. The temperature data were recorded by thermocouples while the load-deflection data were recorded from the MTS machine.

Thermocouples were implanted in the concrete at the time of casting to measure the temperature distribution over the sample cross section. These thermocouples were Type T thermocouples supplied by Omega Measurements Company. The thermocouple wire was encased in a teflon sheath to protect

the wire from damage and corrosion. Eight thermocouples were placed along the ties at the ends and at the concrete/insulation interface. In addition to these thermocouples, two more thermocouples were added before the test by cutting a shallow slot in the face of the concrete and plastering in the new thermocouple. These thermocouples were added so that when the face reached the required -40 degrees fahrenheit the test could begin.

The thermocouples which were placed along the ties were connected to the central computer. The temperature data from these ties were corrected for the ambient temperature by subtracting a constant. This constant was obtained from Table IV on Page T-45 in Volume 26 of Omega Temperature (18) book supplied by Omega Measurements and applied to the input signal before the final temperature was calculated by the following equation:

$$T = a_0 + a_1x + a_2x^2 + a_3x^3 \quad \text{Eq. 2.2}$$

where a_0 , a_1 , a_2 , and a_3 are from Table 4 Page T-12 of the Omega Temperature book and x is the adjusted input signal.

The remaining surface thermocouple data was collected by an Omega Data Logger, Model OM301. This data logger was sent to Omega Measurements and calibrated prior to use. The data

recorded by this logger were used to verify the temperature data recorded by the central computer.

The load-deflection data were collected from two different sources. A load cell in the test frame supplied the load information while the deflection data were collected from the control console of the MTS testing machine.

The displacement data collection procedure was verified by cycling a test specimen through several load cycles and measuring the relative displacement between wythes with dial gages. When the desired 0.45-inch deflection was obtained, the corresponding difference in signal input was recorded from the MTS load console. The displacement data were then based on the input signal voltage.

All of this incoming data were routed through a central buss location to a Hewlett Packard Data Acquisition System. The data were stored on disk as well as on a paper print out. The data were printed after each load point was read.

2.6 Test Procedure

After attaining the desired temperature the test was performed by applying the cyclic load to the center wythe and collecting the load-deflection data and temperatures at selected intervals. The data were collected by a Hewlett Packard data acquisition system at 0, 100, 250, 500, 1000,

1500 and 2000 cycles. At these points in the test, the cyclic loading was stopped and the test specimen was displaced slowly to allow the computer to read the data. The load-deflection data were read at 0.05 inch increments up to the maximum of 0.45 inch. An Omega data logger was used to determine the temperature of the concrete faces. The sample was pushed to failure after the 2000 cycle point with data being collected at 0.05-inch intervals.

Nine specimens were tested in this manner and are tests TTS1-1 through TTS1-9. The remaining three samples were tested as control specimens at room temperature. These specimens were tests TTS1-10 through TTS1-12. The TTS1-prefix in these names represents the first thermocycle test series.

2.7 Results

The ultimate goal of these tests was to verify that the wall panel system incorporating the fiber-composite ties could withstand extreme temperature ranges over the expected life of the structure. All 12 specimens tested withstood the extreme temperatures and 0.45-inch deflection over the 2000 cycles. Table 2.4 lists the ultimate loads and corresponding deflections for each sample obtained after 2000 cycles. The data for the failure test on TTS1-1 were lost due to an error

in the data acquisition program. Insufficient data were available to reconstruct the lost curve.

Table 2.4. Ultimate load and deflection data

Specimen	Ultimate Load (pounds)	Ultimate Deflection (inches)
TTS1-1	Disk Error	Disk Error
TTS1-2	1,976	1.55
TTS1-3	3,358	1.30
TTS1-4	2,861	1.96
TTS1-5	3,986	1.40
TTS1-6	3,489	1.96
TTS1-7	3,226	1.30
TTS1-8	4,011	1.91
TTS1-9	3,882	2.00
TTS1-10	2,557	2.01
TTS1-11	3,801	1.71
TTS1-12	4,226	1.65

The performance of each specimen has been summarized graphically in Chapter 8. A typical load-deflection curve for the tests is shown in Figure 2.6. In this figure the load required to deflect 0.45-inch decreases with the number of cycles. The large difference between the load-deflection curve at 0 cycles and the load-deflection curve at 100 cycles

Figure 2.6. Typical load versus deflection curve

is probably caused by spalling of the concrete around the tie ends (as discussed below). At zero cycles the concrete surrounding the ties was undisturbed. After the first 100 cycles the concrete around the tie ends was disrupted by a combination of spalling and local crushing. This crushing produces an effective longer span between support points which in turn produces a less stiff system and therefore a system which requires a smaller load to deflect the 0.45 inches. A smaller amount of spalling and local crushing is experienced after the 100 cycle point due to the loss in rigidity of the connection. This phenomenon was evident in all of the test samples including the control specimens.

Figure 2.6 also displays a system which gains stiffness with increasing load. This increase in stiffness is possibly due a compression of the insulation material surrounding the tie as the tie is displaced. This compression will densify the material and increase the stiffness of the system.

The load-deflection curves for the entire displacement of a typical failure test (Figure 2.7) displays a curve with five distinct zones of behavior. These zones are labeled A, B, C, D, and E. Region A is a region, which when enlarged and compared to the adjacent Region B, has a larger slope and therefore a larger stiffness (see Figure 2.6). This difference was attributed to the force required to overcome the static frictional forces generated by the two layers of

Figure 2.7. Load versus deflection for full test range

foam rubbing against each other. Region B, is a region which also possesses a curve with increasing stiffness. This increase in stiffness could be attributed to the change in density of the insulation material surrounding the tie ends as the tie is displaced. Region C could be considered a zone with a linear curve. In this region, the concrete surrounding the tie ends has not suffered additional damage and the tie itself is still within its elastic range. Region D, however, is a zone where the concrete is undergoing further spalling and the tie has started to enter into a plastic behavior. The final zone, Region E, is a post-failure zone. In this zone, individual ties have failed and the remaining ties have taken up the load. The large discontinuities in this zone are a result of the loading method. A hydraulic load frame was used to apply the forces needed to displace these specimens. This system was based upon a constant strain rate and not a constant load rate. Therefore, when ties failed, the instantaneous displacement remained the same and the load was allowed to drop off. Further displacement of the specimen produced areas similar to Regions C and D. The behavior of these zones was based upon the behavior of the remaining ties. All of the test specimens failed by fracturing of the connector as shown in Figure 2.8. This fracture was a brittle failure of the FC material at the narrowest cross-section of the tie. Each tie appeared to

have suffered a delamination of the individual fibers on a plane projected back into the tie from the narrow section (see Figure 2.8). This failure is consistent with previous work done by Wade, Porter, and Jacobs (20).

Figure 2.8. Typical tie failure

Direct load and displacement comparisons between the individual tests was not possible since the angular orientation of the ties in the wythes was discovered to be varied upon dismantling of the wythes from the system after testing. This variation produced different specimen stiffnesses and therefore different behavior. The overall characteristic behavior was, however, the same as displayed in the shape and distribution of the load-deflection curves shown in Chapter 8.

In order to make a comparison between tests, the data

were reduced and normalized to total specimen stiffness by dividing the load data by the corresponding specimen stiffness based upon the individual tie stiffnesses. The specimen stiffness was calculated by measuring the angle of rotation of the ties in the concrete (Figure 2.9), calculating the appropriate rotated moment of inertia (Equation 2.3) and then calculating the total stiffness of the system by summing the individual stiffnesses of the ties. The stiffness calculations were based upon a calculated effective length of seven inches and moments of inertia of the full cross-section.

Figure 2.9. Tie orientation

Figure 2.10. Normalized load versus cycles

Figure 2.11. Normalized load versus cycles

$$I_{\theta} = \frac{I_x + I_y}{2} + \frac{I_x - I_y}{2} \cos 2\theta \quad \text{Eq. 2.3}$$

$$\text{Stiffness} = \frac{12EI_{\theta}}{L_{\theta}^3} \quad \text{Eq. 2.4}$$

These data were then plotted, using a power regression method, for all tests using the load to deflect 0.45 inch versus the number of cycles (Figures 2.10 and 2.11). The power regression model is based upon a curve which best fits Equation 2.5.

$$y = ax^{-b} \quad \text{Eq. 2.5}$$

Each curve, shown in Figures 2.10 and 2.11, displays a characteristic behavior. This behavior consists of a comparatively large load to deflect the first 0.45 inch and then slightly decreasing values after this. The difference between the first data point and the following data points can be attributed to spalling of the concrete surrounding the tie ends. The order of appearance on the graphs in Figure 2.10 and 2.11 is based on the system stiffness with TTS1-10 being the most stiff and TTS1-8 being the least stiff.

After completion of the tests, the specimens were torn apart and inspected. The internal random tie orientations

were recorded. A complete listing of the tie orientation, associated system stiffness, and stiffness ranking can be found in Table 2.5. The angles listed in Table 2.5 are angles measured counterclockwise from the weak axis. An angle of zero degrees represents strong axis orientation and an angle of 90 degrees represents weak axis orientation as shown previously in Figure 2.9.

The graphs in Figures 2.10 and 2.11 show that the specimens with the largest specimen stiffness carry the smallest load (after 2000 cycles) to deflect the required 0.45 inch as shown below. This phenomenon may be the result of a bearing failure in the concrete surrounding the tie ends. Additional analyses and a tie orientation parameter study may be helpful to determine the complete effects of tie length and orientation on the resulting system stiffness. A system with a large stiffness represents a system in which the ties are approaching the strong axis orientation. At this orientation the bearing area of the tie is an area which is only one-eighth inch wide. On the other hand, the weak axis orientation has a bearing area three-eighths inch wide. The smaller bearing area (strong axis orientation) will cause the concrete to crush and spall around the tie end and therefore produce a longer tie span as can be seen in Figure 2.12. This increased span will reduce the stiffness of the overall system as can be seen in Equation 2.4. Some evidence of this

Table 2.5. Sample stiffness

Sample	Tie Orientation by Tie # (degrees)				Stiffness (lb/in)	Rank
	#1	#2	#3	#4		
TTS1-1	0	50	70	80	278.89	3
TTS1-2	0	10	40	80	173.39	8
TTS1-3	19	0	23	25	118.67	9
TTS1-4	73	45	40	47	283.62	2
TTS1-5	0	20	5	70	248.45	6
TTS1-6	30	83	5	36	206.90	7
TTS1-7	6	20	5	0	62.97	12
TTS1-8	12	16	7	6	63.90	11
TTS1-9	52	58	32	50	268.29	5
TTS1-10	63	75	57	55	287.14	1
TTS1-11	65	42	60	33	277.29	4
TTS1-12	40	23	0	0	105.60	10

of this crushing and spalling was found in the samples after they were broken apart. All of the specimens experienced spalling to some degree. Specimens TTS1-10, TTS1-4 and TTS1-1, however, experienced the worst spalling. The diameter of

the spalled areas in these three samples ranged from one and three-quarters of an inch to one-half inch. These three test specimens were the three most stiff in the test series with respect to tie orientation. Minor spalling (less than one-quarter of an inch) in all the other test samples was found. The depth of the spalled areas was approximately one-quarter of an inch. This depth of spalling was used to determine the effective length of seven inches in the previous stiffness calculations. One-half of an inch was added to the clear span of six inches to get the seven-inch effective length.

Figure 2.12. Spalling around ties

2.8 Thermocycle Conclusions

The results of these tests indicated that the fiber-composite tie wall system performed adequately under the

expected low-cycle fatigue loads subjected to cold (-40°F) temperatures and in-plane shear. Elemental tests on the nine low-temperature and three room-temperature samples showed that temperature had little or no effect on the low-cycle, in-plane shear fatigue strength of the tie system. For example, Specimens TTS1-4 (with a total calculated connector stiffness of 283.6 lb/in.) and TTS1-11 (with a total calculated connector stiffness of 277.3 lb/in.) had approximately the same stiffness and yet the load after 2000 cycles required to achieve the needed displacement of 0.45 inch was also approximately equal (i.e., TTS1-4 had a load of 275 lbs and TTS1-11 had a load of 289 lbs). However, Specimen TTS1-4 was tested at the cold temperature and TTS1-11 was tested at room temperature. The angular orientation of the ties on the other hand reduced the overall load-carrying capacity of the wall system under low-cycle fatigue shear loadings as shown in Figures 2.10 and 2.11.

In addition to the tie orientation effects, the failure mode of these ties subjected to low-cycle fatigue loads was found to be a brittle failure of the tie material. The behavior of these ties up to failure could be categorized into five distinct regions. Each of these regions possessed different characteristics based upon the amount of strain.

2.9 Recommendations

Further studies of the behavior of this tie under low-cycle fatigue loads should be performed. These studies should include the effects of tie orientation and should fully document the internal stiffnesses of these orientations. These tests should also document the effects of the fatigue loads on the pullout strength of the tie.

3 TENSILE TESTING

The use of glass fiber composites in the construction industry has increased dramatically in the past five years. This material has been produced in a variety of products, most of which are ideally suited to corrosive and temperature sensitive environments.

Current research at Iowa State University has shown that the Fiber Composite (FC) material can be embedded into concrete and serve efficiently as tensile reinforcement under flexural loads. The need to develop a standard tensile testing procedure to accompany this research prompted this study. Calculation of the theoretical tensile strength was performed. In addition to this, experimental methods were used to determine the tensile capacity of these rods. The experimental tests performed included beam tests and traditional axial loading tests.

The material subjected to these tests was composed of E-glass fibers embedded in a vinyl-ester resin matrix. The majority of the tensile capacity is derived from the fibers. Since these fibers are sensitive to alkaline environments such as concrete, a protective coating was required. This coating must not only protect the fibers, but it must also bind the fibers together. A vinyl-ester resin was selected for this protection by CTC. This resin bonds with the fibers

and is highly resistant to corrosion.

Tensile testing of FC rods is different from testing of steel rods. The primary difference between the two lies in the poor resistance of the FC material to lateral loads. Glass-fiber composites can possess up to three sets of material properties. One set for each direction. The composite being studied here has a low resistance to lateral compressive forces. Because of this, traditional wedge action grips will not work due to the fact that the grips crush the rod. Some work on new grips was performed by Henry (8). In his work, Henry developed circular grips for testing of smooth rods. These grips are based upon wedge action but differ from traditional wedge grips in that they bear all around the specimen. These grips would not work on the rod being studied in this research since the rod is not smooth. Each strand would have one point which would bear on the grip. These points would suffer a crushing failure and the results would be tainted. Because of this, new grips were studied.

3.1 Test Specimens

Each test specimen was produced by CTC in two stages. The first stage involved the pultruding of strands. These strands were then combined and wrapped around a center

strand. A total of seven strands were used to make up a single test specimen. When finished, this rod had a diameter of either one-half inch or three-eighths inch and was similar to a prestressing cable as shown in Figure 3.1.

Figure 3.1. Test specimen configuration

3.2 Research Program

The ultimate tensile capacity of the FC rod stock was determined theoretically and experimentally in order to verify the behavior of the FC material in the concrete. By determining the tensile capacity of the rod, the effects of dowel shear and deflection on the tensile capacity could be more readily understood. Theoretical tensile capacity is based upon the rod's composition and does not account for the effects of the wrapped shape nor the effects of dowel shear and curvature of the rod caused by deflections. Experimental tensile testing was undertaken in order to document the

effects of these parameters. Tensile testing of the rod by means of applying an axial load to the rod through wedge action grips helped to isolate the effects of the wrapped shape. Beam tests were used to help separate the effects of dowel shear and curvature on the rod's ultimate tensile capacity.

3.3 Theoretical Determination

Theoretical determination of the rod's ultimate tensile capacity is based upon the amount of glass fibers in the vinyl-ester resin matrix and the volume of voids. By knowing the percentage fraction of fibers, matrix, and voids, the tensile capacity of the composite can be solved for by the rule of mixtures shown in Equations 3.1 and 3.2.

$$1 = v_f + v_m \quad \text{Eq. 3.1}$$

$$\epsilon_c = \epsilon_f v_f + \epsilon_m v_m \quad \text{Eq. 3.2}$$

where:

v_f = volume fraction of fibers

v_m = volume fraction of matrix

ϵ_c = ultimate composite strain

ϵ_f = ultimate fiber strain

ϵ_m = ultimate matrix strain

The use of these equations requires some knowledge of the final product's composition. Since the rod subjected to the testing was from an experimental production process and the consistency of the results depends upon the consistency of the production, the data obtained could only be used as a benchmark when comparing test results. At the time of the tests, the volume fraction of the fibers and matrix were being altered in order to achieve optimum production rates at the lowest cost. This volume fraction data were not recorded and, therefore, only general calculations for the theoretical ultimate tensile capacity were possible. Table 3.1 contains the data for these calculations.

Table 3.1. Data for theoretical determination of ultimate tensile strength

Fiber Volume (%)	E-Glass (ksi)	Resin (ksi)	Composite (ksi)
0.75	250	11.5	190.38
0.80	250	11.5	202.30

3.4 Experimental Tensile Strength From Flexural Loads

The ultimate tensile capacity of the rod was determined by the use of statics based upon the data collected for the BTS and Anchor Test Series. These tests were conducted to document the flexural and anchorage behavior of the FC rod on a pilot test basis. For each test, the ultimate load was recorded along with the span and cross section dimensions. A typical test configuration can be seen in Figure 3.2.

Figure 3.2. Typical test setup

Load and deflection data were collected for this simply supported test. The ultimate load was used to determine the ultimate moment. This moment was then used to calculate the

internal resistive moment based upon an assumed Whitney stress block. The actual distribution of stress may lie between the Whitney distribution and a triangular distribution. Further research is required to determine the appropriate model to fully consider the general strain compatibility distribution and equilibrium of forces across the cross section.

In order for this cross section to be in equilibrium, the internal compressive force must equal the internal tensile force and the internal moment must equal the external applied moment (Figure 3.3). Shear forces were neglected for these calculations. For the given cross section to be in equilibrium, the summation of the forces in the X direction must be equal to zero (Equation 3.3) and the summation of the internal and external moments must be zero (Equation 3.5).

Figure 3.3. Internal forces

$$A_s F_y = 0.85 f'_c b a \quad \text{Eq. 3.3}$$

gives:

$$a = \frac{A_s F_u}{0.85 f'_c b} \quad \text{Eq. 3.4}$$

$$\sum M = 0 = A_s F_u \left(d - \frac{a}{2} \right) - M_{\max} \quad \text{Eq. 3.5}$$

In these equations the variable b is equal to the width of the beam, six inches, d is equal to the depth to the reinforcing, f'_c is equal to the compressive strength of the concrete, A_s equals the area of reinforcing and F_u equals the ultimate strength. In order to solve for the ultimate strength, Equation 3.4 was substituted into Equation 3.5 and then Equation 3.5 was solved for F_u . The area of reinforcing used in these calculations was based upon summing up the area of all of the individual strands composing the rod. Each strand's diameter was measured using a micrometer. This diameter was used to calculate a circular area for each strand. This area was then multiplied by seven to get the total area for the shape. A three-eighths-inch diameter rod has an area of reinforcing equal to 0.073 square inches and a one-half-inch diameter rod has an area of 0.129 square inches. Table 3.2 displays the data used in the

determination of F_u by this method for the three-eighths inch rod. This table is based upon data collected from Beam Test Series (BTS) 1 and the anchorage test series (Anchor).

Table 3.2. Ultimate tensile strength data

Test	f'_c (psi)	M_{ext} (in-lb)	F_u (psi)
BTS1-1	3,990	61,710	76
Anchor 5	3,310	85,302	116
Anchor 6	3,310	85,218	116
Anchor 7	3,310	86,840	119
Anchor 11	4,018	Shear Failure	-
Anchor 12	4,018	100,310	137
Anchor 13	4,018	100,106	137
Anchor 14	4,018	79,725	108
Anchor 15	4,018	98,505	135
Anchor 16	4,018	90,090	123
Anchor 17	4,018	93,245	126
Anchor 18	4,018	89,602	122
Anchor 19	4,018	84,835	115
Anchor 20	4,018	98,325	134

The data in Table 3.2 are two different test series. Although the procedure for the test and the analysis were the same, these tests were performed on different production runs of the rod stock. The difference in the two production runs can be seen in the variance in the ultimate strength of the rod.

In spite of the fact that this method includes the effects of dowel shear and flexural deformations on the rod's

ultimate tensile capacity, no comparisons with the axial loading tests were possible. This type of comparison required test samples from the same rod. At the time of the BTS and Anchor test series, the manufacturer was capable of supplying rod in eight-foot lengths. Six feet of the eight were required for the flexural testing. This left two feet for the axial loading test. Because of gripping problems, a minimum length of three feet was required for the axial test.

In addition to this lack of material, the gripping problem was still under investigation. At the time of the tests, all of the axial test specimens had to be manufactured on a special basis as discussed below.

3.5 Tensile Capacity Based Upon Axial Loading

Tensile testing of the rod was also performed in a more traditional test format. In these tests, the rod was placed in wedge action grips and the tensile load was applied to the rod by a hydraulic testing frame. Because glass-fiber-based composites possess a low capacity to loads applied normal to the fiber direction, the portion of the rod in the grip had to be protected from crushing. The following test program was aimed at the development of grips that prevent the crushing of the rod.

Several different gripping devices for the rod were

tested in conjunction with the wedge grips. These devices included bullets of vinyl-ester resin cast on the ends of the rod by CTC, rubber linings for the wedge grips, and encasing the rod in copper pipe.

The first method used for the protection of the rod in the grip was a gripping bullet of vinyl-ester resin on each end of the rod. The vinyl-ester was cast onto the rod in a cylindrical shape as shown in Figure 3.4.

Figure 3.4. Typical vinyl-ester resin bullet

No special preparation was performed on the rod ends other than leaving the rod's end frayed out. Each end was inserted into an aluminum mold and the resin was poured around the end. The bullets of hardened resin were then placed into the mechanical grips of the load frame and loaded normal to the surface to supply the gripping forces. During the test, the bullets of resin experienced some cracking. In

spite of the cracks, the ultimate capacity of the rod was achieved with a minimum of slip due to the cracks. Fracture of this rod occurred at approximately one inch inside of the grip. Failure at this location was due to the anticipated rotational restraint of the forces generated by the rod's elongation and resulting unwrapping. Follow-up testing on this system was designed to prevent the cracking and potential slipping of the rod in the bullet.

The next grip configuration tested included glass reinforcing to prevent cracking and a frayed end of the rod to prevent slip. The rod ends were left frayed out and inserted into a mold with a cylindrical top and a transitional zone between the cylinder diameter and the rod diameter. This transitional zone was designed with the intent of reducing the bond and rotational stresses at the end of the grip. By providing a smooth transition for these forces, failure of the rod at the grip could be avoided. Two of these types of tests were performed on the rod. The first test experienced fracturing of the gripping material. The rod, however, did fail in tension at the grip. The second test failed inside of the grip.

The vinyl-ester resin for this test was extremely porous after hardening. This porous structure lead to a crushing failure of the vinyl-ester bullet. In both of these tests, the bullets of vinyl-ester cracked. This cracking was

believed to have been caused by curing problems.

In order for the vinyl-ester to cure properly, an even temperature must be maintained throughout the specimen. The large diameter bullet made this temperature control difficult. Extra catalyzing agents were added to help solve the curing problem. The extra catalyzing agents sped up the reaction too much. Large voids were created when the gases produced by curing tried to escape. In light of these problems, and the fact that the rod had to be specially produced for these tests, this method was abandoned.

The final method used to grip the FC rod was a method used by Homer Kalsem of the Iowa Department of Transportation (10). Mr. Kalsem suggested that the rod be epoxied into a copper pipe. The bearing stress of the wedge action grips could then be transferred from the copper pipe to the rod without crushing the rod.

Sikadure 31 epoxy was used for the first test on a one-half-inch-diameter rod. The epoxy was mixed by hand, placed in the copper pipe, compacted, and the rod was then inserted into the pipe and epoxy. Note should be made of the fact that the rod ends were unprepared with the exception of the fact that the rod ends were cleaned with a thinner to remove any grease from the bonding area. No special manufacturing was performed for this test. For this test, a length of pipe of eight inches was used. This length corresponded with the

length of the wedge grips in the hydraulic testing frame. Type K hard copper pipe with a seven-eighths-inch outside diameter and a three-quarter-inch inside diameter was also used. Figure 3.5 shows the configuration of this grip.

Figure 3.5. Copper pipe tensile grip

This specimen was test at the full strength of the epoxy. Load was applied in the same manner as the vinyl-ester bullet tests. During this test, one strand delaminated from the rod. This delamination caused a 5,000-pound drop in the applied load. The load was reapplied and the rod tested to failure at an ultimate load of 12,530 pounds (97 ksi). Slip between the epoxy and the rod was encountered at both ends.

A new epoxy, longer grips, and roughened ends were tried for the next test. This test used Sikadure 33 epoxy, a nine-inch embedment, and a three-eighths-inch diameter rod. The Sikadure 33 epoxy was selected based upon its ability to be dispensed from an air gun. This pneumatic gun mixes the components in the proper proportions and allows for a more efficient use of the epoxy. Sikadure 33 also possesses higher shear and bond strengths. The pipe length was increased to nine inches for this test to provide for one-half inch overlap outside of the wedge grips. This overlap was used to reduce the stresses at the grip. A three-eighths inch diameter rod was used based upon the fact that the one-half-inch material was unavailable at the time.

As before, this specimen was tested by applying the axial load through wedge action grips. A load was applied to the rod until it failed. Failure occurred just inside the grip at a load of 10,380 pounds. This load translated into an ultimate stress of 143 ksi. Some end slip was encountered. A new test with longer grips and prepared ends was performed next.

The next test performed on the rod stock used the Sikadure 33 epoxy, a three-eighths-inch diameter rod, and an eleven-inch embedment in the pipe. In addition to these two items, the rod ends were sandblasted with a fine silica sand in order to expose the glass fibers. By exposing the glass

fibers, bonding exposure zones for the epoxy were made available, thereby increasing the capacity of the grip. This specimen was tested to failure without visual end slip at a load of 9,670 pounds (133 ksi). Failure of this rod occurred at the center of the specimen.

3.6 Conclusions

The tensile testing study of this material has produced a reasonable procedure for the testing of FC rod stock. By embedding the specimen in a copper pipe filled with epoxy, these rods can be tested in traditional wedge action grips. Some preparation of the ends, namely cleaning and lightly sandblasting, is necessary in order to provide proper bonding.

Further work should now be performed in order to document the modulus of elasticity of rod by either strain gages or by measuring the amount of rod elongation. In addition to this work, the benefits and drawbacks of this gripping procedure should be studied in depth. Finally, this procedure should be submitted to the appropriate agencies in order to establish it as a standard test.

4 FLEXURAL TESTING

Flexural testing of the FC material was performed for the purposes of a "proof of concept", and, if successful, to determine the interaction of the rod inside of the concrete and the flexural properties of the shapes tested.

Pilot testing of the FC rod stock was performed in order to determine if the E-Glass and vinyl-ester resin materials would bond with the concrete, be resistant to corrosion, and would behave similarly to steel. Rectangular, round and wrapped shapes were tested along with grade 40 steel reinforcing rod.

The object of these tests was to determine if the FC material would bond with the concrete and resist a pullout failure. Parameters studied included the shape of the reinforcing and material composing the reinforcing.

4.1 Materials and Controls

The materials used in these tests were a nominal 3,000 psi concrete supplied by a local ready-mix company, FC rod material provided by Composite Technologies Corporation, and steel reinforcing rod from the Iowa State University Structural Engineering Lab.

The concrete supplied by a local ready-mix company was

delivered to the lab and checked for a three-inch slump. After attaining the desired slump the concrete was deposited into an overhead bucket and placed into the formwork. Each test beam was vibrated to assure that all of the voids were filled. A total of six compressive test cylinders measuring six inches in diameter by twelve inches long were cast along side of the beams. These cylinders were cast according to ASTM C192, "Making and Curing Test Specimens in the Laboratory" (3). In addition to the test specimens and the compressive test cylinders, one 36-inch modulus of rupture beam was cast. This beam was also cast according to ASTM C192 (3). All of these test specimens were covered with plastic after casting and were cured at room temperature. Each test specimen was sprayed with water daily for the first week of curing. During the remainder of the 28-day cure, these specimens were left undisturbed and covered.

Testing of these control specimens commenced after one month of curing. The compressive test cylinders were tested according to ASTM C39, "Compressive Strength of Cylindrical Concrete Specimens" (3). Ends of the specimens were placed into Aluminum caps with rubberized liners to account for uneven surfaces. Load was applied at a constant rate of 1,500 psi per minute to the test cylinder by means of the 400 kip hydraulic testing frame. The ultimate load was displayed digitally by the electronic control console for the load

frame.

The modulus of rupture beams were tested according to ASTM C78, "Flexural Strength of Concrete-Using Simple Beam with Third Point Loading" (3). The load was applied at a constant rate of 150 psi per minute by a small testing machine manufactured by American Beam Tester Company.

4.2 Test Specimens

The flexural test specimens were beams on simple supports. Each beam was six inches wide, twelve inches deep, and seventy-two inches long. The first test series, the Beam series, had five beams. These beams were reinforced with different materials and shapes. Beam 1 was reinforced with two one-half-inch diameter FC rods. These rods were shaped like a prestressing cable as discussed in Chapter 3 and shown in Figure 3.1. Beam 2 was reinforced with two similar three-eighths-inch diameter FC rods. The remaining FC reinforced beam was reinforced with two rectangular bars. These bars were cut from a section of the stock used for the ties discussed in Chapter 1. The last two beams, Beam 4 and Beam 5 were reinforced with standard steel rods. Beam 4 was reinforced with one #4 and Beam 5 was reinforced with two #6 bars.

The formwork used for the casting of these specimens was

steel formwork supplied by Economy Forms Corporation of Des Moines, Iowa. Chairs were used to support the reinforcing material during casting. These chairs were one and one-half inches tall. All sides of the steel formwork were lightly oiled prior to the pour.

4.3 Results From Beam Series

Tests on these specimens were performed in the 400 kip hydraulic testing frame. The load was applied at the midpoint on a 69-inch span. This span provided for one and one-half inch of overhang on each end of the test specimen. A typical test setup can be seen in Figure 4.1.

Figure 4.1. Typical flexural test setup

Data collected included third-point deflection and load. Crack patterns were also recorded along with some crack widths. Dial gages were used to measure the deflection at the one-third point and the hydraulic control console was used to gather the load data. The crack pattern was sketched by hand and the crack widths were measured by an optical scope. All of these data were collected and recorded at intervals of 500 pounds. The data were then transferred to a spread sheet, reduced and plotted as a load versus deflection curve. A typical Load Deflection curve can be seen in Figure 4.2.

This figure shows different zones of linear elastic behavior offset by discontinuities caused by the progression of cracks toward the loaded side. Region A is a zone characterized by an uncracked concrete section. In this region, the reinforcing has not begun to take up the full tensile load. Upon cracking of the concrete, Region B, the reinforcing takes up the tensile load which the concrete was carrying. In this zone, the cracks begin to progress upward. This progression of cracks reduces the area available for internal compressive forces and therefore reduces the stiffness of the beam. This reduction in stiffness can be seen in the reduced slope of an elastic zone between Regions A and C. Finally, Region C was characterized by sharp drop in the load carrying capacity of the beam. This drop in the

Figure 4.2. Typical load versus deflection curve

Figure 4.3. Comparison of Beam 1 and Beam 4

capacity was a result of a tensile failure of the rod.

Test specimens Beam 4 and Beam 5, the steel reinforced samples, failed in different ways. Beam 4, reinforced with one #4, experienced a primary mode of failure by yielding of the steel followed by a secondary compressive failure in the concrete. This secondary failure mode was expected. Beam 5, on the other hand, failed in a shear-bond mode. This mode occurs when a flexural-shear crack progress upward at an angle and suddenly propagates back along the reinforcing steel.

All three of the composite reinforced specimens failed by exceeding the ultimate strength of the reinforcing material. When the ultimate strength of this composite was exceeded, catastrophic failure occurs. There was little if any plastic behavior. This is evident when comparing the behavior of specimens Beam 1 and Beam 4 at failure as shown in Figure 4.3. Beam 1 had two one-half-inch diameter FC rods. Beam 4, on the other hand, was reinforced with one #4 steel reinforcing rod. The difference between the two behaviors is evident in the higher load regions. In these regions, the steel begins to yield and its curve flattens out, whereas the composite suffers a brittle failure. This brittle failure can be seen in the sudden loss of all load-carrying capacity.

Another difference between these two materials is the

tensile modulus. The FC material has a tensile modulus (E) of 5.8×10^6 psi (20) where the steel has a tensile modulus of 29×10^6 psi (1). This difference can be seen in the slope of the two curves. Beam 1 exhibits a behavior that produces more deflection per kip of load than Beam 4. This difference produces a smaller slope and therefore a smaller stiffness. The difference in the two slopes was directly attributed to differences in the span, moment of inertia, I, (indirectly the depth of the cracks), and the modulus of elasticity. Equation 4.1 shows the relationship between these variables.

$$S = \frac{KEI}{I^3} \quad \text{Eq 4.1}$$

The variable K in this equation represents a constant. This constant is 48 for the mid-span deflection used in this calculation. The span for both specimens was fixed at 69 inches. Therefore, the difference must be in the moment of inertia and the modulus of elasticity. The modulus of elasticity was assumed to be constant for each rod. This parameter depends upon the amount of glass fibers and matrix present in the composite and is determined by the rule of mixtures as discussed in the previous chapter. Equation 4.2 shows the relationship between the properties of the individual components and properties of the composite.

$$E_c = E_f V_f + E_m V_m$$

Eq 4.2

where:

E_c = tensile modulus of composite

E_f = tensile modulus of glass fibers

E_m = tensile modulus of matrix

V_f = volume fraction of fibers (%)

V_m = volume fraction of matrix (%)

Since this number is a constant for each beam, and is less than the corresponding number for steel, the behavior attributed to the difference in the slopes is a result of this difference in the moduli or the moment of inertia. Of these two variables, the modulus is the independent variable and the moment of inertia is the dependent variable. In addition to these items, the modulus of elasticity and the moment of inertia are related by the transformation constant n . This variable, n , is a ratio of the modulus of the reinforcing to the modulus of the concrete.

Both specimens experienced initial cracks at approximately the same load. This load represents the tensile strength of the concrete. Post-cracking behavior is therefore dependent upon the strength of the reinforcing. Since the modulus of elasticity is a representation of the

amount of deformation produced by a given load, and the modulus for the FC rod is lower than that for the steel rod, the FC reinforcing will elongate more than the steel reinforcing. This extra elongation produces wider and deeper cracks. The deeper the crack the smaller the beam's cross-section stiffness and the larger the deflection.

Since the lower modulus produces wider cracks that progress deeper and reduce the cross-sectional area available for compressive forces, this modulus indirectly produces a less stiff beam. Therefore, the difference in the tensile modulus accounts for the difference in the behavior between the FC reinforced specimen and the steel reinforced specimen.

Figure 4.4 graphically displays the relationship between load and crack width. This figure provides a graphical verification of the fact that the FC rod experiences larger cracks in the region before the steel has begun to yield.

This test series demonstrated that the FC rod material could be placed in a concrete member and serve efficiently as tension reinforcing under flexural loads. However, direct comparison with steel reinforcing was not possible since the area of reinforcing provided in the cross-sections was not the same between the FC material and the steel material. The next test series was designed to make this comparison.

Figure 4.4. Load versus crack width

4.4 BTS Test Series

Beam Test Series 1 (BTS1) was designed to compare the behavior of two similar beams subjected to flexural loadings. Each beam had the same dimensions as the previous pilot tests. The areas of reinforcing were, however, selected to be as close as possible between the FC and steel test specimens. The principal parameters studied in this test series were reinforcing material and area of reinforcing. This test series used the same testing procedure and ASTM references as the previous test series. The data were also collected in the same manner.

Six different test specimens were cast and cured according to the procedure used in the previous Beam test series. These specimens were labeled BTS1-1 through BTS1-6. The BTS1 represents the first beam test series and the -1 represents the specimen number. Table 4.1 lists the test specimen, type of reinforcing, and area of reinforcing.

Testing of these specimens began one month after casting. Test specimen BTS1-1 carried the point loading until a tensile failure of the FC rod occurred. BTS1-2 failed in shear-bond as Beam 5 did in the previous test series. All of the beams reinforced with steel failed by a yielding of the steel.

From Table 4.1, BTS1-1 and BTS1-5 have similar areas of

reinforcing. Since BTS1-2 failed in shear-bond, specimens 1

Table 4.1. BTS test series layout

Specimen	Reinforcing	Area (in ²)
BTS1-1	2 3/8" FC	0.15
BTS1-2	2 1/2" FC	0.25
BTS1-3	2 #3 Steel	0.22
BTS1-4	2 #4 Steel	0.40
BTS1-5	1 #3 Steel	0.11
BTS1-6	1 #4 Steel	0.20

and 5 provide the best comparison between the two materials. Figure 4.5 shows the behavior of these two test samples together.

The behavior found in Figure 4.5 is similar to the behavior displayed in Figure 4.3. The smaller slope of the FC material in the post-cracking region is a direct result of the smaller tensile modulus as in the previous series. The ultimate strength of the FC material in this test was determined to be 76 ksi by solution of the internal statics equations discussed in Chapter 3, Equations 3.6, 3.7, 3.8. This low ultimate strength was attributed to an error in the production of this rod. The yield strength of the steel was determined to be 37 ksi by the same method. This yield was

Figure 4.5. Load versus deflection for BTS1-1 and BTS1-5

also low since Grade 40 steel was used. A possible cause for this discrepancy lies in the load application. The point load was applied through a rubber bearing pad. This pad could have moved the effective depth of the compression zone up into the rubber. This would increase the internal moment arm and would produce a larger yield stress.

4.5 Conclusions

The final results of these two test series were that the FC rod material could be embedded in concrete and serve as tensile reinforcing. This material did bond with the concrete and did not appear to suffer short term degradation due to the alkalies in the concrete. All of the series objectives were met by the BTS test series.

Further studies were planned at this point to investigate the bond behavior of this material with concrete. These tests will be designed to investigate the development lengths on a pilot test scale. These tests are the subject of the following chapter.

5 ANCHORAGE TESTING

The next obvious test series was an anchorage study of the FC material in concrete. Reinforcing, to be efficient, must not slip in the concrete. Slippage produces wide cracks which exposes the reinforcing material to corrosive agents. When reinforcing rods slip, they have failed to transfer the tensile loading to the concrete. Since a pullout failure usually follows slippage, slip is considered undesirable.

The anchor test series was a study of the end slip of the reinforcing rod in a concrete beam. These beam tests were designed to provide different test lengths. The primary objective of this test series was to document the amount of end slip occurring throughout the test.

5.1 Test Specimens

As in the previous BTS series, these tests were based upon a beam test format. The specimens used in this test included beams, compressive test cylinders, and modulus of rupture beams. The test beams were six inches wide, twelve inches deep and seventy-two inches long. The beams were subjected to flexural loads as in the BTS test series. These loads were applied at two points. Each point was located over embedded bond-breakers. These bond-breakers were placed

along the reinforcing rod at discrete distances to provide for the desired test length. Each breaker was composed of slices of Styrofoam[®] insulation board. These slices were one inch thick and six inches square. In order to prevent a shear failure of the concrete the load points were located outside of the bond breakers. A typical test setup can be seen in Figure 5.1. This Figure shows the location of the bond breakers relative to the load points and labels the embedment lengths as either l_e or l_w for either the East or West side.

Figure 5.1. Typical test setup for anchor test series

A total of 35 test specimens were cast in this configuration. Thirteen of these specimens were based upon the three-eighths-inch diameter rod and the remaining specimens were reinforced with the one-half-inch diameter rod. The specimens were cast using a nominal 2500 psi

concrete mix supplied by a local ready mix plant. A three-inch slump was specified. Upon delivery, the concrete was checked for slump, placed into the forms, vibrated and then covered with plastic sheets. These specimens were left covered for the one-month cure period. Thirty-seven compressive test cylinders measuring six inches in diameter and twelve inches in height were cast along side the beams according to ASTM C192 (3). In addition to the compressive test cylinders, modulus of rupture beams were cast. These beams measured six by six by thirty-six inches and were cast according to ASTM C192 (3). All of the samples were watered daily for the first week to ensure adequate curing conditions. The test specimens were allowed to cure for one month before testing. During this time all of the specimens were not disturbed.

Testing of the anchor specimens was performed by applying two point loads as shown previously. These point loads were applied by the 400-kip hydraulic testing frame. Data collected included load and end slip. The load data were collected from the hydraulic load console and the slip data were collected by dial gages mounted on the ends of the beams. These data were recorded at 500-pound increments up to failure. The compressive test cylinders were tested according to ASTM C39 (3). Aluminum end caps with rubber liners were used to account for the rough ends. These

specimens were loaded at a constant load rate of 1,500 psi per minute until failure. The ultimate load was displayed by the hydraulic frame's load console.

5.2 Test Results

The results from this test series indicate that the rod experiences slip, but is able resist a pullout failure. All of the test specimens experienced a tensile failure of the rod. A typical load versus slip curve can be seen in Figure 5.2.

From this figure, one can see that this rod configuration, the wrapped shape with no deformations shown in Chapter 3 Figure 3.1, possessed some mechanical anchorage properties. The vertical region of this curve, where the unloaded end has not slipped, is a region where the chemical bond has not broken over the full length of the rod. When this chemical bond is broken, the first slip occurs and the mechanical anchorage takes over in full and continues to resist higher loads. The amount of mechanical anchorage appears to vary from specimen to specimen. This could be a result of the different configurations of the rod, different amounts of surface resin, and different depths of the grooves. Each rod specimen is handmade and possesses a different angle of twist for the outer strands. A smaller

Figure 5.2. Typical load versus slip

angle of twist would increase the amount of slip incurred because the area of the strand normal to the direction of loading is smaller as seen in Figure 5.3. Consolidation of the concrete around the rod did not appear to be a factor in these results.

Figure 5.3. Angle of twist and bearing length

The results for the three-eighths-inch diameter rod were scattered as seen in Figure 5.4. Of the thirteen tests only one embedment length, twenty-three inches, showed zero slip. All other lengths experienced some amount of slip. This length coupled with an f'_c of 4,500 psi provide a preliminary anchorage length. This value is based upon one test only and should serve as a starting point for future testing.

Of the twenty-two one-half-inch rod tests performed, four had zero end slip. All of the other tests suffered some slip on at least one side. The amount of slip and the test length which slipped first had no apparent pattern. Some of the test samples had longer embedment lengths slip first or suffer a larger amount of slip than corresponding lengths in

Figure 5.4. Load at first slip versus embedment for three-eighths-inch diameter rod

Figure 5.5. Load at first slip versus embedment for one-half-inch diameter rod

other samples. Figures 5.4 and 5.5 show the relationship between rod stress at first slip and embedment length for the one-half-inch and three-eighths-inch diameter rods, respectively.

The scatter in these results could be attributed to poor rod surfaces, different rod strengths, different amounts of epoxy binding the strands together, and different angles of twist.

Of the four which had no slip at all, all had embedment lengths greater than thirty-one inches. This length along with an f'_c of 3,400 psi seem to indicate that this length is sufficient for anchorage for the one-half-inch diameter rod under these loads and these support conditions.

One important fact to be recognized is the effects of the supports and load points on the anchorage data. The support points in this test were located one and one-half inch away from the rod. The large normal forces caused by the reaction will skew the data towards shorter lengths. This normal force will help the mechanical anchorage of the rods by increasing the frictional forces. Any further anchorage testing should make use of a test which minimizes the effects of the support reactions. The point loads in this test series had little if any effects on the data since they were located on the opposite side of the beam from the reinforcing.

Five of the test specimens had the rod cut out and inspected. Each of the slices contained a rod surrounded by undisturbed concrete. The fact that the concrete was undisturbed was important because this means that although the rod is unwrapping as it slips, the amount of expansion due to this unwrapping is not sufficient to produce any bond cracks visual to the naked eye. A Poisson's ratio effect could also be partially responsible for the lack of apparent bond cracks. As this rod unwraps, the angle of twist decreases and the area of bearing with the concrete decreases. The decrease in bearing area could also reduce the bearing stresses on the concrete as the rod slips.

This behavior is only a small part of what is happening along the rod. As is evidenced by Figure 5.2, there is a large amount of scatter in the results. Some of this scatter could be due to different textures in the rod, different angles of twist, and different conditions of the surface for the chemical bonding. A rough epoxy surface will provide small deformations and will increase the point of first slip. Different angles of twist will also increase this point. An oily surface will cause the chemical bonds to break down sooner and decrease the load at first slip. All of these parameters need to be studied further.

Several test specimens with the epoxy coated in sand, no angle of twist, and greased surfaces should be studied in

order to determine the effects of these variables. In addition to these studies, a more heavily instrumented test series should be performed to determine the bond stress distribution along the rod. This test series should incorporate the effects of constant moments, and minimize the effects of the supports and load points.

5.3 Discussion of Recommended Test

Some of the tests under review for further testing are based upon the work done by Ferguson (5, 6), Kemp (11), Kemp and Wilhelm (12), Mathey and Watstein (16), Perry and Thompson (19), and Warren (21). All of these tests are beam tests with the exception of Kemp's work. The beam test format is favored over the pullout test format because the effects of curvature and shear forces are included in this format.

Different test formats incorporate different parameters. Some of the important parameters yet to be studied include spacing, concrete strength, stirrups, and load distribution. The last effect, load distribution, is an important variable since the effect of developing a rod in a constant moment region is a severe condition. Tests with different load conditions such as Warren's work (21) make use of point loads and an overhang to produce a moment diagram which is

constantly changing (Figure 5.6).

Figure 5.6. Test configuration based upon Warren's work (21)

Figure 5.7. Test setup based upon Ferguson's work (5, 6)

Figure 5.8. Recommended test configuration

This test format along with a beam configured as in Ferguson's work (6) (Figure 5.7), a dog-bone shape, should be used to create the new bond test configuration. All of these variables will ultimately be incorporated in a new test scheme.

By combining these two test configurations, a new test, which minimizes the effects of the load and support forces and provides a test length in a region of changing moment and therefore shear forces, can be designed.

The instrumentation to be used in this test will include strain gages, slip wires to measure the internal differential

slip, and dial gages to measure the external deflection data. The load data will be collected by a calibrated load cell.

This test format should yield data which will lead to the development of design anchorage values. The parameters discussed above can also be investigated using this test format.

6 RANDOM DISPERSED COMPOSITE REINFORCING

Random Dispersed Composite Reinforcing, RDCR, is a product produced for the purpose of crack control in concrete members. This product is designed to stop cracks at the microscopic level. These cracks are commonly known as micro-cracks. The RDCR fibers are placed into the wet concrete mix which in turn is placed normally. Upon hardening, this concrete will have a higher resistance to thermal and plastic cracking due to the random reinforcing provided by the RDCR.

The objective of this study was to determine if the RDCR fiber would bond with the concrete and to determine if these fibers had an adverse effect on the concrete's tensile capacity. The major variable in these studies was the amount of fibers added to the concrete.

Pilot tests on RDCR were conducted at Iowa State University. These tests were performed on RDCR fibers composed of the same materials as the FC rod. These fibers were part of an experimental production line. Each fiber was two inches long, one-quarter of an inch wide and was shaped like a wave. A random sampling of ten fibers were measured and the thickness was found to vary from 0.015 inches to 0.020 inches. In addition to these dimensions the amplitude of the wave like shape was determined to vary from 0.000 inches to 0.044 inches. A typical RDCR fiber can be

seen in Figure 6.1.

Figure 6.1. Typical RDCR specimen

6.1. Test Procedure

Testing of these fibers was performed on a split cylinder test basis. Although this test does not explore the fibers capacity for crack control, it does, however, document the fibers ability to bond with the concrete. In addition to this, the split cylinder test determined the effects of the fibers on the concrete's tensile capacity.

The test procedure followed for this study was ASTM C496-71 "Splitting Tensile Strength of Cylindrical Concrete Specimens" (3). This procedure dictates that the test specimens be cylinders with a diameter of six inches and a height of twelve inches. These cylinders were laid on their side and the load is applied to the long edge. A typical

test setup can be seen in Figure 6.2.

Figure 6.2. RDCR test setup

As prescribed in ASTM C496-71, plywood bearing strips and steel plates were placed between the cylinder and the reaction and loading points. These bearing strips served to spread out the load evenly. The cylinders were loaded by the 400-kip hydraulic load frame at a constant rate of 150 psi per minute until a tensile failure occurred.

The procedure followed for the casting of these specimens was different from the average pour. The concrete was supplied by a local ready-mix company. Upon delivery a slump test was performed. An overhead bucket was then filled with 650 pounds of concrete, enough for the casting of twenty

test specimens. The concrete was then poured into a drum mixer. The mixer was started and the fibers and 326 milliliters of plasticizer were added. This mixer was allowed to run until the fibers were dispersed throughout the concrete, approximately 75 revolutions. The concrete and RDCR mixture was then transferred into wheelbarrows and placed into the cylinder molds in one lift. Each specimen was then vibrated on a shaker table for five to ten seconds. The specimen was then placed on the floor and covered with plastic and left undisturbed for the curing period.

A total of 160 test specimens were cast in this manner. Two pours of eighty each were studied. In each pour, the eighty specimens were broken down into four groups of twenty. These groups had different proportions of fibers. The proportions studied were zero, one and one-half, three, and six pounds of fibers per cubic yard of concrete. The different amounts of fibers were added at the drum mixer. All other parameters were the same.

Testing of these specimens commenced at different ages. The first eighty specimens were tested at a desired strength of 3,500 psi. A high-early strength concrete was used for this pour and therefore, the test strength was obtained at one day. Because of the early test and the fact that the concrete was gaining strength rapidly, the specimens were tested in groups of three from each proportional set.

6.2 Results

The results from this series of testing showed that the fibers could bond to the concrete. The fibers appeared to have little effect on the concrete's tensile capacity. This can be seen in the data in Table 6.1.

Table 6.1. First RDCR pour test results

Test Proportion, lb/yd ³	Average Tensile Strength, psi	Standard Deviation, psi
0	378	46
1.5	350	31
3.0	382	45
6.0	370	35

Because of the early age at testing, another series identical to the previous series was cast. This series used a 3,500 psi mix and was cast using the same procedure only this series was allowed to cure for twenty-eight days. The results from this series of tests can be seen in Table 6.2.

Table 6.2. Second RDCR test results

Test Proportion, lb/yd ³	Average Tensile Strength, psi	Standard Deviation, psi
0	423	46
1.5	437	37
3.0	421	44
6.0	434	50

The data in Tables 6.1 and 6.2 indicate that the fibers have little effect on the concrete's tensile strength.

Fibers which were oriented parallel to the failure plane could have negated any increase of the tensile capacity based upon the fibers as discussed below. The difference between the values can be attributed to the experimental error due to the concrete. What these tables do not show, however, is that the fibers bonded with the concrete. After the individual specimens were failed, each was inspected. The number of fibers crossing the failure plane were counted and studied. Since these fibers were dispersed randomly, some of the fibers were parallel to the failure plane and some were perpendicular. The fibers which were perpendicular failed in tension. Fibers which were parallel suffered a pullout failure. Orientations which fell in between these two failed by and large in tension. This failure in tension proves that the fibers are able to bond with the concrete and resist the pullout failure. One important fact to note is that when a specimen had several fibers parallel to the failure plane, the tensile capacity of this specimen was reduced. This reduction was due to the fiber itself. The fibers parallel to the failure plane reduce the amount of concrete area available for the tensile load and create a localized weak point. Since, the fiber's deformations are designed to resist longitudinal loads and not transverse loads, the fibers contributed little to the tensile load carrying capacity of the sample. Therefore, the tensile

capacity was reduced. Because of this reduction and the fact the fiber's orientation is truly random, the fiber width parameter should be studied to determine its effect on the tensile capacity of the concrete.

Further testing should be performed on the fibers to determine their crack control capacity. This capacity could be determined through slab and elemental bond tests. These slabs, if exposed to a wide temperature range and restrained on all sides, would provide an ideal format for the study of the RDCR's crack control capacity. Elemental bond tests should be performed on precracked specimens to determine the bond capacity of these fibers.

7 GENERAL SUMMARY

The work encompassed by this thesis included thermocycle work on FC sandwich wall ties, tensile testing of FC rod, pilot beam tests on the FC rod, anchorage studies on the FC rod, and pilot studies on RDCR.

The thermocycle work determined that the FC tie material performed adequately under the given loads. However, the orientation of these ties was determined to effect the low-cycle fatigue behavior more than was expected. Ties oriented in the strong direction were capable of carrying less load than the ties oriented in the weak direction. The mode of failure for these ties was a brittle fracture of the material at the smallest cross-section. Further studies were recommended in order to determine the complete effects of the tie orientation on the load carrying capacity of the wall system.

Tensile testing of the FC material concluded with the development of a suitable grip for axial loading. This grip consisted of a copper pipe which was epoxied to the FC rod. This grip produced a failure at the center of the rod and suffered no visual end slip.

The pilot flexural tests determined that the FC rod can be placed into concrete, resist short-term degradation, and serve adequately as tensile reinforcing under flexural loads.

These tests provided information required in the design of the anchorage studies.

The anchorage studies investigated the load-slip relationship of the FC rod under flexural loads. These tests were performed on both the one-half inch diameter and three-eighths inch diameter rods. From these studies, the preliminary anchorage length required for the one-half inch diameter rod under flexural loads and embedded into 3,500 psi concrete was found to be approximately thirty-one inches. This same length, under the same conditions, for the three-eighths inch diameter rods was found to be about twenty-three inches. These values, coupled with the literature review were used to design a new anchorage test.

Preliminary testing of the RDCR product determined that this fiber had little effect on the tensile capacity of the concrete. Further tests investigating this product's crack control capacity were recommended.

8 APPENDIX A

This appendix contains the data, in graphical form, for the thermocycle testing described in Chapter 2. These graphs are organized into two sections.

The first section, Figure 8.1 to Figure 8.12, displays the load-deflection data recorded at the 0, 100, 250, 500, 1000 and 1,500 cycle points. These data represent the behavior of the wall panel test specimens for loads applied as shown in Figure 2.3, "Displaced specimen".

The behavior in these graphs follows a similar pattern. This pattern can be characterized best by Figure 8.3, "Load versus deflection for TTS1-3". In this figure, the initial load-deflection curve starts at zero load and zero deflection. From this point, the curve progresses upward at an increasing rate. This increasing rate describes a curve which has an increasing stiffness. Since this is not possible, this behavior must be attributed to some variables. The cause of this increasing stiffness is theorized to be a result of the compressibility of the insulation surrounding the ties. As the tie is displaced, it compresses the insulation. This insulation densifies as it compresses and picks up stiffness. This densification of the insulation is the most likely cause for this increase in stiffness. Specimen TTS1-2, shown in Figure 8.2, did not

start at zero load. This was due to a software error. The behavior of this specimen did, however, fit the general patterns described above.

After the first load-deflection curve, the general behavior remained the same. Each curve, was, however, lower than the previous curve. This drop in stiffness was attributed to a loss of rigidity in the connection at the tie end. The largest drop in stiffness followed the initial displacement. Smaller drops were evident through out the testing.

Some curves possessed a region at the start of the load-deflection curve which had a steep slope followed by a lower, more typical slope. The steep slope could have been a result of friction forces generated by the foam rubbing against each other. Once the frictional forces were overcome, the specimen followed the same general patterns.

The second section, Figures 8.13 to 8.23, contain the load-deflection data for the 2,000th cycle. At this point in the test, the specimens were displaced to failure. This load-deflection curve, therefore, shows the full range behavior. Figure 8.20 contains the load-deflection data for TTS1-9. In this figure, five regions are evident. Figure 2.7 shows the break down of these regions. Region A is a region with a relatively large slope compared to Region B. This large slope was attributed to the force required to

overcome the static friction forces. Region B could be characterized as a zone with increasing stiffness. This increase in stiffness was possibly caused by the compression of the insulation surrounding the tie as the tie deflects. The next zone, Region C, contains a relatively straight curve. This portion of the load-deflection curve is in a range where the tie is performing elastically. The adjacent region, Region D, has a smaller slope. This reduction in slope could have been caused by a loss of rigidity at the tie ends. The final area, Region E, is an area typified by large discontinuities. These interruptions of the load-deflection curve are caused by the failure of ties. As each tie fails the load carrying capacity of the specimen decreases until all of the ties have failed. The overall behavior of these curves was summarized best by these five zones.

Figure 8.1. Load versus deflection for TTS1-1

Figure 8.2. Load versus deflection for TTS1-2

Figure 8.3. Load versus deflection for TTS1-3

Figure 8.4. Load versus deflection for TTS1-4

Figure 8.5. Load versus deflection for TTS1-5

Figure 8.6. Load versus deflection for TTS1-6

Figure 8.7. Load versus deflection for TTS1-7

Figure 8.8. Load versus deflection for TTS1-8

Figure 8.9. Load versus deflection for TTS1-9

Figure 8.10. Load versus deflection for TTS1-10

Figure 8.11. Load versus deflection for TTS1-11

Figure 8.12. Load versus deflection for TTS1-12

Figure 8.13. Load versus deflection at 2,000 cycles for TTS1-2

Figure 8.14. Load versus deflection at 2,000 cycles for TTS1-3

Figure 8.15. Load versus deflection at 2,000 cycles for TTS1-4

Figure 8.16. Load versus deflection at 2,000 cycles for TTS1-5

Figure 8.17. Load versus deflection at 2,000 cycles for TTS1-6

Figure 8.18. Load versus deflection at 2,000 cycles for TTS1-7

Figure 8.19. Load versus deflection at 2,000 cycles for TTS1-8

Figure 8.20. Load versus deflection at 2,000 cycles for TTS1-9

Figure 8.21. Load versus deflection at 2,000 cycles for TTS1-10

Figure 8.22. Load versus deflection at 2,000 cycles for TTS1-11

Figure 8.23. Load versus deflection at 2,000 cycles for TTS1-12

9 APPENDIX B

The material contained in this appendix represents the data collected from the flexural testing studies. These figures are the result of two different testing series: the Beam series and the BTS series.

Figures 9.1 through 9.5 contain the data collected from the Beam test series. These figures can not be characterized by a single behavior as the failure modes for these tests were different. In Figure 9.1, the behavior exhibited by test specimen Beam 1 is similar to the behavior described in Chapter 4, Figure 4.2. The only major difference between the behavior described in Chapter 4 and the behavior of Beam 1 was the failure mode. The specimens in Chapter 4 suffered a tensile failure whereas Beam 1 experienced a dowel-shear failure. Specimens Beam 2 and Beam 3, shown in Figures 9.2 and 9.3, did follow the Chapter 4 behavior. These specimens experienced a tensile failure of FC reinforcing rod as the primary mode of failure. The remaining test samples were reinforced with steel. The performance exhibited by Beam 4 in Figure 9.4 is consistent with the discussion in Chapter 4. The modes of failure for this beam were a tensile yielding of the reinforcement as the primary mode and a secondary mode of a compression failure. Beam 5 did not fit these behavioral patterns. This sample suffered a dowel-shear failure before

any major cracks had formed.

The second group of figures contained in this appendix are from the BTS study. The behavior of some of these specimens fits the patterns established in Chapter 4. Test specimens BTS1-1 and BTS1-2, shown in Figures 9.6 and 9.7, respectively, fit the Chapter 4 patterns. Both of these samples experienced a tensile failure of the FC reinforcing rods. Test samples BTS1-3, BTS1-5, and BTS1-6 were all reinforced with steel bars. These specimens possessed a tensile yielding as their primary failure mode. This yielding was followed by a secondary compressive failure in the concrete. The last specimen, BTS1-4 shown in Figure 9.9, experienced a yielding of the tension reinforcing. This test was halted at this point since a shear-bond failure was imminent.

Figure 9.1. Load versus deflection for Beam 1

Figure 9.2. Load versus deflection for Beam 2

Figure 9.3. Load versus deflection for Beam 3

Figure 9.4. Load versus deflection for Beam 4

Figure 9.5. Load versus deflection for Beam 5

Figure 9.6. Load versus deflection for BTS1-1

Figure 9.7. Load versus deflection for BTS1-2

Figure 9.8. Load versus deflection for BTS1-3

Figure 9.9. Load versus deflection for BTS1-4

Figure 9.10. Load versus deflection for BTS1-5

Figure 9.11. Load versus deflection for BTS1-6

10 APPENDIX C

The graphs and figures found in this appendix represent a compilation of the data collected during the anchorage studies. These figures take the form of load-slip curves. The curves in this appendix can be divided into two groups. These groups follow the same patterns but are distinct in that they have different sizes of reinforcing. Anchorage tests Anchor5 through Anchor7 and Anchor11 through Anchor20 were reinforced with the three-eighths-inch diameter FC rod. All of the remaining tests were reinforced with the one-half-inch diameter rod.

A typical load versus slip curve can be seen in Figure 10.29, "Load versus slip for Anchor33". The data in these graphs is based upon a beam test format loaded as shown in Figure 5.1, "Typical test configuration for anchor test series". In Figure 10.29, the curve can be divided into two regions. These two regions are; a vertical region where no slip has occurred, and a positive sloped region undergoing slip. All of the figures in this appendix follow this general pattern.

Some exceptions to the general pattern are Figures 10.3, 10.7, 10.11, 10.21, 10.25, 10.28, and 10.31. Figure 10.3 shows a curve for the East side which becomes vertical after undergoing slip. This vertical region could be due to a

binding of the rod in the concrete, or an error in the dial gage. Figure 10.7 has no curves on it. This was due to the fact that this specimen suffered a shear failure before any slipping occurred. Figure 10.11 displays two curves with large discontinuities in them. Each of these discontinuities are alike and occurred at the load point where a large crack progression happened. This crack progression caused the stiffness of the beam to drop and the load carried by the beam dropped off due to this. The amount of slip at this point remained constant and continued to build up after the progression. In Figure 10.21, the West curve has a vertical region which is followed by a small amount of slip followed by another vertical region which is again followed by slip. The second vertical region could be due to the same factors as in Figure 10.3. The most likely cause of this vertical region, since it is small and occurs after a small amount of slip, is an error in the dial gage reading. The curve in Figure 10.25 has two vertical regions offset by slip. These regions are most likely the result of dial gage errors since the total amount of slip is less than 0.002 inches and the East side had no slip. Figure 10.28's East curve has a discontinuity in it. This interruption of the curve could be due to the slip occurring on the West side. The discontinuity occurs at the same time as the slip on the West side. The possibility exists that the West end of the

reinforcing is absorbing all of the slip energy and is allowing the East side to remain fixed. Figure 10.31 possesses an irregular curve. This curve has a negative region in it. Since it is not possible for the reinforcing to move away from the center of the beam and the amount of slip is small, this error can be attributed to dial gage error.

Figure 10.1. Load versus slip for Anchor5

Figure 10.2. Load versus slip for Anchor 6

Figure 10.3. Load versus slip for Anchor 7

Figure 10.4. Load versus slip for Anchor8

Figure 10.5. Load versus slip for Anchor9

Figure 10.6. Load versus slip for Anchor10

Figure 10.7. Load versus slip for Anchor11

Figure 10.8. Load versus slip for Anchor12

Figure 10.9. Load versus slip for Anchor13

Figure 10.10. Load versus slip for Anchor 14

Figure 10.11. Load versus slip for Anchor15

Figure 10.12. Load versus slip for Anchor16

Figure 10.13. Load versus slip for Anchor17

Figure 10.14. Load versus slip for Anchor18

Figure 10.15. Load versus slip for Anchor19

Figure 10.16. Load versus slip for Anchor20

Figure 10.17. Load versus slip for Anchor21

Figure 10.18. Load versus slip for Anchor22

Figure 10.19. Load versus slip for Anchor23

Figure 10.20. Load versus slip for Anchor24

Figure 10.21. Load versus slip for Anchor25

Figure 10.22. Load versus slip for Anchor 26

Figure 10.23. Load versus slip for Anchor27

Figure 10.24. Load versus slip for Anchor28

Figure 10.25. Load versus slip for Anchor 29

Figure 10.26. Load versus slip for Anchor30

Figure 10.27. Load versus slip for Anchor31

Figure 10.28. Load versus slip for Anchor32

Figure 10.29. Load versus slip for Anchor33

Figure 10.30. Load versus slip for Anchor34

Figure 10.31. Load versus slip for Anchor35

Figure 10.32. Load versus slip for Anchor36

Figure 10.33. Load versus slip for Anchor37

Figure 10.34. Load versus slip for Anchor38

Figure 10.35. Load versus slip for Anchor39

Figure 10.36. Load versus slip for Anchor40

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