ABSTRACT

SHAO, YUTIAN. Behavior of FRP-Concrete Beam-Columns under Cyclic Loading. (Under the direction of Amir Mirmiran)

Use of concrete-filled fiber-reinforced polymers (FRP) tubes (CFFT) for columns and piles has been studied extensively over the last decade. The focus, however, has been exclusively on the monotonic behavior of CFFT. An issue that has received little attention is the implications of using CFFT in seismic regions. Survey of damaged structures in recent earthquakes indicates that catastrophic failure of an entire structure may result from failure of few columns in a chain action. Since it may not be economical to design columns to respond to earthquake loads in their elastic range, dissipation of energy by post-elastic deformation is desired. Although, FRP materials are known for their linear elastic behavior, some FRP systems may exhibit non-linearity due to their laminate architecture and inter-laminar shear. Also, confinement of concrete core in CFFT improves its ductility. This study was carried out to evaluate the cyclic behavior of CFFT beam-columns, and determine whether non-linearity of FRP or confinement of concrete can provide seismic performance comparable to reinforced concrete (RC) columns or concrete-filled steel tubes (CFST).

The experimental work consisted of cyclic loading and unloading of FRP-wrapped concrete cylinders and FRP coupons, and reverse cyclic loading of CFFT beam-columns under constant axial load. Some measures of hysteretic performance, including cumulative energy dissipation, ductility and pinching effect were used to evaluate the cyclic response of tested CFFT beam-columns. The study resulted in a cyclic model for FRP-confined concrete in compression, and cyclic models for linear and non-linear FRP materials in tension and
compression. A fiber element model was employed to predict the cyclic behavior of CFFT beam-columns. A parametric study was carried out on the cyclic behavior of CFFT beam-columns, and to compare the hysteretic response of CFFT beam-columns with those of RC and CFST members.

The two types of CFFT beam-columns tested under this study represented two different failure modes; a brittle compression failure for the over-reinforced white tube specimens with thick FRP tube and with majority of the fibers in the longitudinal direction, and a ductile tension failure for the under-reinforced yellow tube specimens with thin FRP tubes and off-axis fibers. The study showed feasibility of designing ductile CFFT members for seismic applications comparable to RC members. Significant ductility can result from the inter-laminar shear in the FRP tube. Moderate amounts of internal steel reinforcement can further improve the performance of CFFT members. Adding internal steel can be ineffective and may lead to premature failure. Slender CFFT members have less capacity than their short stocky counterparts. However, they are less susceptible to pinching effect and premature shear failure.
BEHAVIOR OF FRP-CONCRETE
BEAM-COLUMN UNDER CYCLIC LOADING

by
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BIOGRAPHY

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TABLE OF CONTENTS

LIST OF TABLES………………………………………………………………….. viii

LIST OF FIGURES………………………………………………………………. x

CHAPTER 1 INTRODUCTION………………………………………………….. 1

1.1 Problem Statement……………………………………………………… 1

1.2 Research Objectives…………………………………………………… 2

1.3 Research Approach……………………………………………………... 3

1.4 Organization of the Dissertation……………………………………….. 5

CHAPTER 2 LITERATURE REVIEW………………………………………… 6

2.1 Introduction…………………………………………………………….. 6

2.2 Confinement Models for Concrete…………………………………….. 6

2.3 Cyclic Behavior of Plain and Confined Concrete………………….. 9

2.4 Modeling of FRP Laminates………………………………………….. 10

2.5 Applications of CFFT Beam-Columns……………………………... 15

2.6 Evaluation Criteria for Seismic Performance………………………… 17

2.6.1 Ductility………………………………………………………. 17

2.6.2 Stiffness Degradation…………………………………………. 19

2.6.3 Pinching Effect………………………………………………... 20

2.6.4 Energy Dissipation Capacity………………………………….. 21
3.4.2 Instrumentation Test Setup and Procedure.......................... 57
3.4.3 Test Observations and Failure Modes.............................. 58
3.4.4 Test Results and Discussions......................................... 59

CHAPTER 4 MODELING OF FRP-CONCRETE BEAM-COLUMNS........... 142

4.1 Introduction........................................................................... 142
4.2 Modeling for FRP-Confined Concrete in Uniaxial Compression...... 142
  4.2.1 Monotonic Model.......................................................... 142
  4.2.2 Cyclic Model............................................................... 144
      4.2.2.1 Unloading from Envelope Curve to Zero Stress......... 144
      4.2.2.2 Reloading from Zero Stress to Envelope Curve......... 147
      4.2.2.3 Unloading between Arbitrary Stress Levels............ 148
      4.2.2.4 Reloading between Arbitrary Stress Levels.......... 148
      4.2.2.5 Validation of the Cyclic Model............................. 149
  4.3 Modeling of FRP Laminates.............................................. 150
      4.3.1 Modeling of White Tubes in Monotonic Loading......... 150
      4.3.2 Modeling of White Tubes in Cyclic Loading.............. 151
      4.3.3 Modeling of Yellow Tubes in Monotonic Loading........ 151
      4.3.4 Modeling of Yellow Tubes in Cyclic Loading........... 152
  4.4 Modeling of FRP-Concrete Beam-Columns........................... 155
4.4.1 Validation for the Fiber Element Analysis Tool .......................... 157

CHAPTER 5 PARAMETRIC STUDIES AND COMPARISONS .................... 177

5.1 Parameter Selection ........................................................................ 177

5.2 Parameter Assessment .................................................................... 179

5.2.1 Hysteretic Response .................................................................... 179

5.2.2 Envelope Curve ........................................................................... 181

5.2.3 Cumulative Energy ...................................................................... 182

5.2.4 Ductility ....................................................................................... 182

5.2.5 Pinching ....................................................................................... 183

5.3 Comparison between CFFT and RC Beam-Columns ....................... 184

CHAPTER 6 SUMMARY, CONCLUSIONS AND RECOMMENDATIONS ...... 217

6.1 Summary ....................................................................................... 217

6.2 Conclusions ................................................................................... 218

6.3 Recommendations for Further Research ....................................... 220

REFERENCES ....................................................................................... 221

APPENDIX A Cyclic Stress-Strain Relation for Steel ................................. A-1

APPENDIX B Fiber Element Analysis ...................................................... A-5
# LIST OF TABLES

2.1 Mechanical Properties of Typical Thermoset Resins........................................ 24

2.2 Damage Index Classifications................................................................. 24

3.1 Test Matrix of FRP-Concrete Beam-Column Specimens............................. 62

3.2 Mechanical Properties of White FRP Tubes............................................. 63

3.3 Mechanical Properties of Yellow FRP Tubes........................................... 63

3.4 Comparison of White and Yellow Tubes.................................................. 64

3.5 Technical Properties of Electrical Resistance Strain Gages ...................... 65

3.6 Matrix of FRP-Confined Stub Tests......................................................... 66

3.7 Mechanical Properties of GFRP Laminates.............................................. 67

3.8 Mechanical Properties of CFRP Laminates............................................. 67

3.9 Confinement Ratio for FRP-Wrapped Concrete Stubs............................... 68

3.10 Test Matrix of FRP Coupons................................................................. 69

3.11 Geometric Properties of White Coupons in Tension................................ 70

3.12 Geometric Properties of White Coupons in Compression.......................... 70

3.13 Geometric Properties of Yellow Coupons in Tension................................ 71

3.14 Geometric Properties of Yellow Coupons in Compression......................... 71

3.15 Geometric Properties of Yellow Coupons in Reverse Cyclic Loading........... 72

3.16 Average Results of Tension Tests.......................................................... 73
3.17 Average Results of Compression Tests…………………………………… 74

5.1 Matrix of Parametric Study.................................................................... 187
# LIST OF FIGURES

2.1 Mechanical Properties of Polymers......................................................... 25  
2.2 Mechanical Properties of Fibers.............................................................. 25  
2.3 Modes of Failure in a Laminate................................................................. 26  
2.4 Macromechanics of FRP Composites......................................................... 26  
2.5 Helically Wound Fiber Reinforced Cylindrical Shell.................................. 27  
2.6 Positive Rotation of Principal Material Axes from Arbitrary XY Axes......... 27  
2.7 Tensile Test Result of GFRP Coupons....................................................... 28  
2.8 Compressive Test Result of GFRP Coupons............................................... 28  
2.9 Definition of Displacement-Based Ductility............................................... 29  
2.10 Definition of Energy-Based Ductility...................................................... 29  
2.11 Cumulative Ductility Factor....................................................................... 30  
2.12 Stiffness Degradation................................................................................. 31  
2.13 Pinching Effects......................................................................................... 31  
3.1 White Tube.................................................................................................. 75  
3.2 Cross-Section of the White Tube................................................................. 75  
3.3 Yellow Tube................................................................................................. 76  
3.4 Short Steel Cage.......................................................................................... 76  
3.5 Bottom View of Dowel Bars........................................................................ 77
3.6 Long Steel Cage................................................................. 77
3.7 Wooden Base Plate before Casting Concrete............................... 78
3.8 Formwork of an End Block..................................................... 78
3.9 Overview of Specimens before Casting End Blocks....................... 79
3.10 Placement of a CFFT Beam-Column into an End Block............... 79
3.11 Specimens after De-Molding End Blocks.................................. 80
3.12 Instrumentation for Large-Scale Beam-Column Tests.................. 81
3.13 Load Cell to Measure Axial Load......................................... 82
3.14 String Potentiometer.......................................................... 83
3.15 Cross Sections of CFFT Beam-Column Specimens...................... 84
3.16 Wires of Internal Strain Gages............................................. 85
3.17 Location of External Strain Gages.......................................... 85
3.18 Electrical Resistance Strain Gages........................................ 86
3.19 PI Gage............................................................................. 86
3.20 Potentiometer...................................................................... 87
3.21 Inclinometer....................................................................... 87
3.22 Monitoring of Support Movement.......................................... 88
3.23 440-kip 40-inch Stroke Actuator........................................... 88
3.24 Large-Scale Beam-Column Test Setup (Schematic Drawing)........ 89
3.25 Overview of Large-Scale CFFT Beam-Column Tests

3.26 Horizontal Frame System with Threaded Rods

3.27 Steel Plate Embedded in End Block

3.28 Steel Plates and Vertical Loading System

3.29 Loading Regime for CFFT Beam-Column Tests

3.30 Distributed Cracking at the Top of Specimen Y1

3.31 Crushing of White Tubes in Compression

3.32 Longitudinal Shear Splitting of White Tubes

3.33 Local Buckling of White FRP Tubes

3.34 Mid-Span Flexural Failure of Yellow Tubes

3.35 Normalized Load versus Deflection for Specimen W1

3.36 Normalized Load versus Deflection for Specimen W2

3.37 Normalized Load versus Deflection for Specimen W3

3.38 Normalized Load versus Deflection for Specimen Y1

3.39 Normalized Load versus Deflection for Specimen Y2

3.40 Normalized Load versus Deflection for Specimen Y3

3.41 Normalized Envelope Curves of Load versus Deflection

3.42 Axial Load versus Mid-Span Deflection for Specimen W1

3.43 Axial Load versus Mid-Span Deflection for Specimen W2
3.44 Axial Load versus Mid-Span Deflection for Specimen W3......................... 101
3.45 Axial Load versus Mod-Span Deflection for Specimen Y1....................... 101
3.46 Axial Load versus Mid-Span Deflection for Specimen Y2....................... 102
3.47 Axial Load versus Mod-Span Deflection for Specimen Y3....................... 102
3.48 Normalized Load versus Steel Strain for Specimen W2.......................... 103
3.49 Normalized Load versus Steel Strain for Specimen W3.......................... 103
3.50 Normalized Load versus Steel Strain for Specimen Y2.......................... 104
3.51 Normalized Load versus Steel Strain for Specimen Y3.......................... 104
3.52 Cyclic Variation of Depth of Neutral Axial for Specimen W1................... 105
3.53 Cyclic Variation of Depth of Neutral Axial for Specimen W2................... 105
3.54 Cyclic Variation of Depth of Neutral Axial for Specimen W3................... 106
3.55 Cyclic Variation of Depth of Neutral Axial for Specimen Y1................... 106
3.56 Cyclic Variation of Depth of Neutral Axial for Specimen Y2................... 107
3.57 Cyclic Variation of Depth of Neutral Axial for Specimen Y3................... 107
3.58 Normalized Total Moment versus End Rotations for Specimen Y1.............. 108
3.59 Normalized Total Moment versus End Rotations for Specimen Y2.............. 108
3.60 Normalized Total Moment versus End Rotations for Specimen Y3.............. 109
3.61 Normalized Total Moment versus End Rotations for Specimen W3.............. 109
3.62 Hoop Strains versus Longitudinal Strain at Top of Specimen W1.............. 110
3.63 Hoop Strains versus Longitudinal Strains at Bottom of Specimen W1............. 110
3.64 Hoop Strains versus Longitudinal Strains at Top of Specimen W2.............. 111
3.65 Hoop Strains versus Longitudinal Strains at Bottom of Specimen W2........... 111
3.66 Hoop Strains versus Longitudinal Strains at Top of Specimen W3............... 112
3.67 Hoop Strains versus Longitudinal Strains at Bottom of Specimen W3........... 112
3.68 Hoop Strains versus Longitudinal Strains at Top of Specimen Y1.............. 113
3.69 Hoop Strains versus Longitudinal Strains at Bottom of Specimen Y1.......... 113
3.70 Hoop Strains versus Longitudinal Strains at Top of Specimen Y2.............. 114
3.71 Hoop Strains versus Longitudinal Strains at Bottom of Specimen Y2.......... 114
3.72 Hoop Strains versus Longitudinal Strains at Top of Specimen Y3.............. 115
3.73 Hoop Strains versus Longitudinal Strains at Bottom of Specimen Y3.......... 115
3.74 Normalized Energy Dissipation Capacity............................................. 116
3.75 Ductility Factors for White Specimens.............................................. 116
3.76 Displacement-Based Ductility Factor for Yellow Specimens.................... 117
3.77 Pinching Effects Comparison ............................................................... 117
3.78 Normalized Load versus Deflection for Secondary Monotonic Test of Y1..... 118
3.79 Normalized Load versus Deflection for Secondary Monotonic Test of Y2..... 118
3.80 Normalized Load versus Deflection for Secondary Monotonic Test of Y3..... 119
3.81 Comparison of Secondary Monotonic Tests for Specimens Y1, Y2 and Y3... 119
3.82 Comparisons of CFFT with RC Beam-Columns................................. 120
3.83 Typical GFRP and CFRP Specimens.................................................... 121
3.84 Test Setup for Stub Specimens.......................................................... 121
3.85 Stub Test Equipment............................................................................. 122
3.86 Loading Regimes for Stub Tests.......................................................... 122
3.87 Typical Failure Mode for a Single Layer Glass Wrap........................... 123
3.88 Typical Failure Mode for Two-Layer Glass Wraps............................... 123
3.89 Typical Failure Mode for a Single-Layer Carbon Wrap....................... 124
3.90 Typical Failure Mode for a Two-Layer Glass Wrap.............................. 124
3.91 Stress-Strain Response for Single Layer Glass Wraps......................... 125
3.92 Stress-Strain Response for Two Layer Glass Wraps............................ 125
3.93 Stress-Strain Response for Single Layer Carbon Wraps..................... 126
3.94 Stress-Strain Response for Two-Layer Carbon Wraps......................... 126
3.95 Typical Unloading Curves before Bend Point..................................... 127
3.96 Typical Unloading Curves after Bend Point........................................ 127
3.97 Typical Reloading Curves from a Low Stress Level............................. 128
3.98 Typical Reloading Curves from a High Stress Level............................ 128
3.99 Axial Stresses versus Volumetric Strain for Specimen G22................ 129
3.100 Axial Stresses versus Volumetric Strain for Specimen C25.................. 129
3.101 Test Setup for Tension Coupons………………………………...……………... 130
3.102 Test Setup for Compression Coupons………………………………..………… 130
3.103 Typical Failure Modes for White Coupons in Tension......................... 131
3.104 Typical Failure Modes for Yellow Coupons in Tension........................ 131
3.105 Typical Failure Modes for White Coupons in Compression.................... 132
3.106 Typical Failure Modes for Yellow Coupons in Compression.................... 132
3.107 Monotonic Tests of White Coupons in Tension........................................ 133
3.108 Cyclic Test of White Coupon WTC1 in Tension........................................ 133
3.109 Cyclic Test of White Coupon WTC2 in Tension........................................ 134
3.110 Cyclic Test of White Coupon WTC3 in Tension........................................ 134
3.111 Monotonic Tests of White Coupons in Compression.............................. 135
3.112 Cyclic Test of White Coupon WCC1 in Compression.............................. 135
3.113 Cyclic Test of White Coupon WCC2 in Compression.............................. 136
3.114 Monotonic Tests of Yellow Coupons in Tension..................................... 137
3.115 Cyclic Test of Yellow Coupon YTC1 in Tension..................................... 137
3.116 Cyclic Test of Yellow Coupon YTC2 in Tension..................................... 138
3.117 Cyclic Test of Yellow Coupon YTC3 in Tension..................................... 138
3.118 Monotonic Tests of Yellow Coupons in Compression.......................... 139
3.119 Cyclic Test of Yellow Coupon YCC1 in Compression.......................... 139
4.16 Reloading from an Arbitrary Stress Level………………………………………. 166

4.17 Comparison of Proposed Model with Two-Layer CFRP Stub Unloaded to Zero Stress………………………………………………………………………... 167

4.18 Comparison of Proposed Model with Two-Layer CFRP Stub Unloaded to Non-Zero Stress………………………………………………………………………... 167

4.19 Comparison of Proposed Model with Three-Layer CFRP Stub Unloaded to Zero Stress………………………………………………………………………... 168

4.20 Comparison of Proposed Model with Three-Layer CFRP Stub Unloaded to zero and Non-Zero Stress………………………………………………………………………... 168

4.21 Monotonic Tension Model for White Tubes…………………………………………………... 169

4.22 Monotonic Compression Model for White Tubes…………………………………………………... 169

4.23 Monotonic Tension Model for Yellow Tubes…………………………………………………... 170

4.24 Monotonic Compression Model for Yellow Tubes…………………………………………………... 170

4.25 Proposed Cyclic Model for Yellow Tubes…………………………………………………... 171

4.26 Fiber Representation of Concrete-Filled FRP Tube…………………………………………………... 171

4.27 Degrees of Freedom for 3-Node Element Based on Kirchhoff Beam Theory….. 172

4.28 Kinematics of the Section with Slippage…………………………………………………... 173

4.29 Schematic Model for CFFT Beam-Columns…………………………………………………... 173

4.30 Comparison of Specimen Y1 with Proposed Model…………………………………………………... 174
5.14 Effect of L/D Ratio for Linear FRP with Internal Steel ............................... 195
5.15 Effect of L/D Ratio for Non-Linear FRP with Internal Steel ....................... 195
5.16 Effect of P/P₀ Ratio for Linear FRP without Internal Steel ......................... 196
5.17 Effect of P/P₀ Ratio for Non-Linear FRP without Internal Steel ................ 196
5.18 Effect of P/P₀ Ratio for Linear FRP with Internal Steel ............................ 197
5.19 Effect of P/P₀ Ratio for Non-Linear FRP with Internal Steel ..................... 197
5.20 Effect of D/t Ratio on the Envelope Curves for Linear FRP without Internal
    Steel.............................................................................................................. 198
5.21 Effect of D/t Ratio on the Envelope Curves for Non-Linear FRP without
    Internal Steel ............................................................................................... 198
5.22 Effect of D/t Ratio on the Envelope Curves for Linear FRP with Internal
    Steel.............................................................................................................. 199
5.23 Effect of D/t Ratio on the Envelope Curves for Non-Linear FRP with Internal
    Steel.............................................................................................................. 199
5.24 Effect of L/D Ratio on the Envelope Curves for Linear FRP without Internal
    Steel.............................................................................................................. 200
5.25 Effect of L/D Ratio on the Envelope Curves for Non-Linear FRP without
    Internal Steel............................................................................................... 200
5.26 Effect of L/D Ratio on the Envelope Curves for Linear FRP with Internal
    Steel.............................................................................................................. 201
5.27 Effect of L/D Ratio on the Envelope Curves for Non-Linear FRP with Internal Steel

5.28 Effect of P/P₀ Ratio on the Envelope Curves for Linear FRP without Internal Steel

5.29 Effect of P/P₀ Ratio on the Envelope Curves for Non-Linear FRP without Internal Steel

5.30 Effect of P/P₀ Ratio on the Envelope Curves for Linear FRP with Internal Steel

5.31 Effect of P/P₀ Ratio on the Envelope Curves for Non-Linear FRP with Internal Steel

5.32 Effect of D/t Ratio on Cumulative Energy for Linear FRP without Internal Steel

5.33 Effect of D/t Ratio on Cumulative Energy for Non-Linear FRP without Internal Steel

5.34 Effect of D/t Ratio on Cumulative Energy for Linear FRP with Internal Steel

5.35 Effect of D/t Ratio on Cumulative Energy for Non-Linear FRP with Internal Steel

5.36 Effect of L/D Ratio on Cumulative Energy for Linear FRP without Internal Steel

5.37 Effect of L/D Ratio on Cumulative Energy for Non-Linear FRP without Internal Steel
5.38 Effect of L/D Ratio on Cumulative Energy for Linear FRP with Internal Steel… 207

5.39 Effect of L/D Ratio on Cumulative Energy for Non-Linear FRP with Internal Steel……………………………………………………………………………….. 207

5.40 Effect of P/P₀ Ratio on Cumulative Energy for Linear FRP without Internal Steel………………………………………………………………………………….. 208

5.41 Effect of P/P₀ Ratio on Cumulative Energy for Non-Linear FRP without Internal Steel………………………………………………………………………………….. 208

5.42 Effect of P/P₀ Ratio on Cumulative Energy for Linear FRP with Internal Steel………………………………………………………………………………….. 209

5.43 Effect of P/P₀ Ratio on Cumulative Energy for Non-Linear FRP with Internal Steel………………………………………………………………………………….. 209

5.44 Effect of D/t Ratio on Ductility………………………………………………….. 210

5.45 Effect of L/D Ratio on Ductility………………………………………………….. 210

5.46 Effect of P/P₀ Ratio on Ductility………………………………………………….. 211

5.47 Effect of D/t Ratio on Pinching………………………………………………….. 211

5.48 Effect of L/D Ratio on Pinching………………………………………………….. 212

5.49 Effect of P/P₀ Ratio on Pinching………………………………………………….. 212

5.50 Sectional Equivalency of Linear FRP, Non-Linear FRP and Steel Tubes……… 213

5.51 Hysteretic Curves of Linear CFFT, CFST and RC Beam-Columns……………… 214

5.52 Hysteretic Curves of Non-Linear CFFT, CFST and RC Beam-Columns………. 214
5.53 Envelope Curves of Linear CFFT, Non-Linear CFFT, CFST and RC Beam-Columns

5.54 Cumulative Energy Dissipation of Linear CFFT, Non-Linear CFFT, CFST and RC Beam-Columns

5.55 Ductility and Pinching Behavior of Linear CFFT, Non-Linear CFFT, CFST and RC Beam-Columns

A.1 Menegotto-Pinto Steel Cyclic Model

A.2 Definition of Curvature Parameter R in Menegotto-Pinto Steel Cyclic Model
1.1 Problem Statement

Environmental effects on concrete bridge pier columns and piles could significantly deteriorate their long-term durability and structural integrity. The key problems are permeability of concrete and corrosion of the embedded steel reinforcement. The concept of using concrete-filled fiber reinforced polymer (FRP) tubes (CFFT) as columns or piles was first introduced by Mirmiran and Shahawy (1995) to address these problems for the Florida Department of Transportation. As an alternative to conventional reinforced concrete (RC) columns or prestressed concrete piles, CFFT members can increase both durability and strength of the structure. The benefits of CFFT members are twofold; protection of concrete against harsh environmental effects especially in coastal areas, and strength enhancement of concrete due to its confinement. Previous studies on confined concrete have shown that lateral confinement with FRP tube not only increases its ultimate compressive strain (Samaan et al. 1998), but also results in a compressive strength larger than the sum of the individual strengths of concrete core and the tube (Fam 2000).

Due to these advantages, applications of CFFT systems have been studied extensively in recent years. The focus, however, has been exclusively on the static and creep behavior of such hybrid systems. Survey of damaged structures in recent earthquakes indicates that catastrophic failure of an entire structure may be triggered by the failure of a few columns in a chain action. Since it is not economical to design columns to respond to earthquake loads in their elastic range, dissipation of energy by post-elastic deformation is necessary. On the
other hand, FRP materials are well known for their linear elastic behavior until failure. An issue that has received little attention is the implications of using CFFT columns and piles in seismic regions. This issue has design implications as to whether confinement alone can provide adequate ductility for this type of structural members, or they need some internal steel reinforcement, particularly in the plastic hinge regions of the member. Also some FRP structures may exhibit non-linearity due to their laminate architecture and inter-laminar shear. Another issue of great concern in FRP structures is the definitions of ductility. In RC structures, ultimate displacements are measured against a reference displacement at the first yield of steel reinforcement. However, there is no such yield criteria for FRP-RC members. Therefore, one needs to develop a better understanding of the ductility measures that lend themselves to non-yielding materials such as FRP.

The present study addresses the need for better understanding of the cyclic behavior of CFFT and its components, FRP-confined concrete and FRP tubes. This study explores the effects of FRP laminate architecture on its cyclic response to determine whether it is possible to develop sizeable ductility and dissipate adequate energy in CFFT columns without any internal steel reinforcement. Finally, the study will compare the cyclic behavior of CFFTs with their RC and concrete-filled steel tube (CFST) counterparts.

1.2 Research Objectives

The following objectives were identified for this study:

1. Investigate cyclic response of FRP-confined concrete in uniaxial compression;

2. Develop a model for the cyclic behavior of FRP-confined concrete in uniaxial compression;
3. Investigate cyclic response of FRP laminates with different levels of fiber reinforcement under uniaxial tension and compression loading;

4. Develop cyclic models for FRP laminates with different levels of fiber reinforcement under uniaxial tension and compression loading;

5. Investigate the behavior of CFFT beam-columns, with different levels of fiber reinforcement, and with or without internal steel reinforcement, under constant axial loading and reverse transverse loading;

6. Employ an analytical tool to study the cyclic response of CFFT beam-columns under different influencing factors, particularly the levels of fiber reinforcement and internal steel reinforcement; and

7. Compare cyclic response of CFFT beam-columns with those of reinforced concrete (RC) and concrete-filled steel tubes (CFST).

1.3 Research Approach

The CFFT system used in this research consists of three components, namely FRP tube, concrete core and internal steel reinforcing bars. In order to study the cyclic behavior of CFFT beam-columns, which can also cover some general conditions such as arbitrary axial load levels, fiber orientations, and concrete strengths, the following four logically correlated steps were carried out as the main elements of this research:

Firstly, the constitutive relations for each of the components were experimentally investigated under cyclic loading. This included coupons of the FRP tube under uniaxial tension and compression or reverse loading, and stubs of FRP-wrapped concrete under
uniaxial compression. Some experimental details were specially designed to help identify the phenomenological behavior in order to accurately establish the cyclic models.

Secondly, the cyclic behavior of CFFT beam-columns was experimentally investigated using a number of specimens with two different types of FRP tubes and three different levels of internal steel reinforcement, including specimens with no internal reinforcement. The specimens were loaded under constant axial loading and reverse transverse loading in four point flexure. Special attention was given to the support conditions and instrumentation to accurately capture the behavior of tested specimens.

Thirdly, the loading, unloading and reloading constitutive rules for all components of CFFT were developed based on test observations, and regression analysis of the experiment results. These models were then verified against the experimental data of the present study or those of independent investigations.

Finally, a fiber element model was adopted and modified for use in the study of the cyclic behavior of CFFT beam-columns. The cross section of CFFT was discretized into fiber elements of FRP, concrete, and reinforcing bars. The constitutive laws for each type of element was used accordingly to establish the overall cyclic response of the beam-columns. The model was verified against the experimental results of the present study. It was then used for a parametric study of different influencing factors, including type of FRP tube, reinforcement ratio of internal steel (if any), diameter-to-thickness and length-to-diameter ratios of the tube, and the level of axial loads as a fraction of the axial capacity of the column. The model was also used to compare the cyclic behavior of CFFT beam-columns against its equivalent RC and CFST counterparts.
1.4 Organization of the Dissertation

This dissertation consists of six chapters and an appendix, as follows:

Chapter 1 (this chapter) highlights the problem statement, and the significance, objectives and methodologies of the research undertaken in this study. Chapter 2 presents a literature review of the previous work on modeling of concrete confined with FRP, modeling of FRP laminates, experimental work on CFFT beam-columns, and evaluation criteria for seismic performance of RC structures in general. Chapter 3 summarizes the results of the experimental work on FRP-confined concrete stubs and the FRP coupons under cyclic uniaxial loading, and the CFFT beam-columns under constant axial loading and reverse transverse loading. Chapter 4 presents the cyclic models developed for the components of the CFFT systems, namely FRP-confined concrete and the FRP tube. It also discusses a fiber element model that was utilized to study the cyclic behavior of CFFT beam-columns. The described models are verified against experimental data. Chapter 5 reports on parametric studies of CFFT beam-columns under different factors affecting their cyclic behavior. It also compares the cyclic response of CFFT beam-columns with their equivalent RC and CFST counterparts. Chapter 6 summarizes the research undertaken in this study, and provides concluding remarks in addition to recommendations for future research. Appendix A describes details of the fiber element model used in this study, and the hysteretic model that was adopted for steel reinforcement.
CHAPTER 2
LITERATURE REVIEW

2.1 Introduction

Cyclic performance of CFFT beam-columns is a new research area that has not been explored in the past. Literature review in this chapter seeks relevant studies for various aspects of the problem, including confinement models for concrete, cyclic response of unconfined and confined concrete, behavior of FRP laminates, applications of CFFT members, and evaluation criteria for seismic performance of concrete structures.

2.2 Confinement Modeling of Concrete

Over the years, a number of confinement models have been developed for concrete. Until a few years ago, majority of the models were developed and calibrated for concrete confined by transverse steel. Only in recent years, a need has been recognized for modeling of FRP-confined concrete. In this section, a brief overview of the previous work on the subject is presented.

The most widely used model for concrete confined by steel spirals or ties is that of Mander et al. (1988), whose simple equations describe the entire stress-strain curve. It also accommodates cyclic loading and different strain rates. An energy balance approach is adopted to predict the ultimate compressive strain of concrete based on the first fracture of the transverse steel reinforcement. Fardis and Khalili (1982) made the first attempt at developing a confinement model for FRP-wrapped concrete. Ahmad et al. (1991) recalibrated an earlier model for steel-confined concrete (Ahmad and Shah 1982) to make it
applicable to concrete stubs wrapped with glass FRP wires. Priestley et al. (1992) studied the use of glass FRP wraps to enhance flexural and shear performance of seven RC bridge columns in seismic regions. Test results showed that properly designed FRP wraps could prevent lap-splice failures in plastic hinge regions, and provide sufficient shear strength for columns. They slightly modified the model of Mander et al. (1988) to make it applicable to FRP-wrapped concrete columns. On the other hand, Saadatmanesh et al. (1994) directly used the model of Mander et al. (1988) with no modification, and conducted a parametric study of circular and rectangular columns strengthened with glass or carbon FRP straps. They did not verify their analytical results with any test data.

The first tests on concrete-filled FRP tubes were carried out by Kargahi (1995), and showed significant improvement in strength and ductility of concrete. Mirmiran and Shahawy (1995) described the deficiencies of the model of Mander et al. (1988) for FRP-confined concrete, and developed an iterative strategy to ensure strain compatibility between the concrete core and the FRP wrap in the circumferential direction. The concept of enforcing strain compatibility was later adopted by other investigators (Spoelstra and Monti 1999). Later on, Harmon et al. (1998) articulated the reason why the energy balance approach of Mander et al. (1988) does not apply to FRP-confined concrete. They used fracture mechanics instead to model confined concrete. Mirmiran and Shahawy (1997a, 1997b) reported on the unique dilation characteristics of FRP-confined concrete. In an effort to combine the confining effects of internal steel hoops and external FRP wraps, Restrepol and DeVino (1996) proposed analytical expressions based on the model of Mander et al. (1988) for the analysis of RC columns wrapped with FRP.
Picher et al. (1996) examined the effect of fiber orientation on FRP-confined concrete. They concluded that different fiber orientations may affect stiffness and strength of concrete, yet its ductility remains almost the same. They also recommended rounding the corners of square or rectangular sections to improve their confinement effectiveness. Bavarian et al. (1996) studied the effect of different fiber types, including glass and aramid, and showed that the behavior of confined concrete directly depends on the stiffness and strength of the FRP jacket. Mastrapa (1997) reported that adhesive bond between FRP and concrete does not significantly affect the strength or ductility of confined concrete. El Echary (1997) assessed the effect of length-to-diameter ratio of the FRP tube on the confinement of concrete, and reported a maximum of 20% strength reduction at a length-to-diameter ratio of 5:1.

Miyauchi et al. (1997) tested a number of carbon-wrapped concrete stubs, and proposed a new model that consisted of a parabola and a tangential straight line at the end. The model was used to perform a time history response analysis for existing RC pier columns strengthened with FRP and subjected to earthquake motion. The analytical results showed that the existing pier columns with only two layers of carbon wrap could withstand the Southern Hyogo Prefecture earthquake.

Watanabe et al. (1997) and Zagers (1998) used finite elements for the analysis of FRP-confined concrete. Samaan et al. (1998) proposed a bilinear model to trace the entire stress-strain curve for the FRP-confined concrete in monotonic compression. The model correlates the dilation rate of concrete with the hoop stiffness of the FRP. The parameters of the model are directly related to the material properties of the FRP and the concrete core. In recent years, other confinement models have also been developed (Toutanji 1999, Fam and Rizkalla 2001).
2.3 Cyclic Behavior of Plain and Confined Concrete

While the ultimate strength theory was being developed as a new method to analyze RC sections under monotonic loading, Sinha et al. (1964) began their pioneering work on the cyclic behavior of concrete. Based on tests of 36 plain concrete cylinders, they suggested that the envelope curve may be considered unique for each strength level of concrete. They also defined a “shakedown limit” for the stress-strain relations of plain concrete, as the locus of points where the reloading and unloading portions of each cycle cross. While stressing above this shakedown limit would lead to additional strains, unloading and reloading at or below this limit would not develop permanent strains. Later, Karsan and Jirsa (1969) confirmed the hypothesis of unique envelope curve for concrete. They also studied the distribution of common points, where the reloading and unloading paths cross each other. They concluded that plain concrete would fail under repeated loading, if stresses exceed 63% of the compressive strength of concrete.


Desayi et al. (1979) and Shah et al. (1983) both confirmed the unique envelope curve for confined concrete. However, the latter found two exceptions, when cyclic load is applied slowly or when the number of cycles to failure is too large. In those cases, the total strain under cyclic load may exceed the envelope curve under monotonic load. However, these restrictions are not too severe for seismic applications and earthquake loading.
As stated earlier, the model of Mander et al. (1988) describes detailed rules of unloading and reloading for confined concrete. In recent years, Bahn and Hsu (1998) proposed a new model for stress-strain behavior of concrete under cyclic loading. Also, Cabrera (1996) extended the monotonic model of Samaan et al. (1998) for the FRP-confined concrete, and incorporated the unloading and reloading rules similar to those in the model of Mander et al. (1988). The model was then used to study the hysteric response of FRP-confined concrete. She noted significant pinching in the response of FRP-confined concrete. However, no experimental data was available at the time to verify her analytical results.

2.4 Modeling of FRP Laminates

Mechanical properties of polymers are highly dependent upon the magnitude, rate and frequency of loading as well as the ambient temperature. Therefore, it is not feasible to obtain a single stress-strain curve for polymers. Figure 2.1 shows a typical set of such curves for different rates of loading. As the loading rate increases, the response becomes more linear, whereas at sufficiently low rates of loading considerable plastic flow and yielding is evident. Higher temperatures generally result in a characteristic behavior similar to that for low rates of loading. Tables 2.1 provides mechanical properties for two typical thermoset resins: polyester and epoxy.

Adding fibers to the polymer matrix greatly reduces the above effects. The main function of the fibers is to carry the majority of the load applied to the composites, while the matrix distributes the loads between the fibers. Figure 2.2 compares typical tensile stress-strain curves of some of the most commonly used fibers, namely E-glass, S-glass, and carbon.
Mechanical properties of FRP materials depend on the fabrication technique, type and properties of its components, particularly the fibers, and the volume fraction of the fibers in the overall mix. Pressure or vacuum molding generally results in a higher fiber volume fraction as compared to hand lay-up. While the ultimate strength of FRP materials depends on the strength and modulus of the fibers, its in-service properties are functions of the matrix. Fibers generally exhibit linear elastic behavior, while resins are visco-elastic or visco-plastic. As such, linear elastic behavior of fibers is generally the dominant factor in the response of unidirectional FRP materials loaded in the direction of the fibers. However, a nonlinear behavior is often observed in the off-axis direction, under certain fiber orientations and fiber volume ratios, as the matrix resists the pull out of broken fibers in shear.

FRP materials are laminate structures made up of a stack of lamina with various fiber orientations. Bonding of the plies or layers of a laminate is often made with the same matrix used in the lamina. In the filament-winding of a tube, for example, each fiber orientation represents a ply, and the entire laminate is made with the same matrix in a single batch. Figure 2.3 shows the different modes of failure in laminate structures, including fiber rupture, transverse or longitudinal cracking of the matrix, debonding at the fiber-matrix interface, and delamination between different layers.

Because of their inherent heterogeneous and anisotropic nature, FRP materials are studied from two points of view: micromechanics and macro-mechanics. The former is a study of FRP at the level of its constituent materials and their interaction at a microscopic scale, whereas the latter is a study of FRP materials at a macroscopic scale, assuming homogeneity along with the average properties of the constituent materials. On the other hand, FRP materials are more advantageous than their isotropic counterparts, because they
can be engineered or tailored for optimum properties in different directions. The tailoring process includes selecting appropriate constituents, fiber volume fraction, fiber orientation, and the stacking sequence of plies.

Figure 2.4 shows fibers uniformly dispersed within a matrix in a unidirectional lamina. Perfect bond is assumed at the interface between fibers and the matrix. The lamina, therefore, has orthotropic properties with the greatest stiffness and strength in the direction of fibers. The primary modulus of elasticity $E_{11}$ can be calculated as

$$E_{11} = E_m V_m + E_f V_f$$

(2.1)

where $E$ and $V$ are the elastic modulus and volume fraction, respectively, and subscripts $f$ and $m$ denote fibers and the matrix, respectively. The above equation, known as the “law of mixture,” can be derived from the resultant axial force $P_{11}$ in the lamina, as given by

$$P_{11} = P_m + P_f$$

(2.2)

where $P_m$ and $P_f$ are the resultant forces in the matrix and the fibers, respectively. The equation can be written in terms of stresses as

$$\sigma_{11} A_{11} = \sigma_m A_m + \sigma_f A_f$$

(2.3)

where $\sigma$ and $A$ are the stress and area identified with subscripts for each phase, and therefore, in terms of volume fractions, it can be written as

$$\sigma_{11} = \sigma_m V_m + \sigma_f V_f$$

(2.4)

from which, one can derive Equation (2.1), assuming strain compatibility. Similarly, the Poisson's ratio $\nu_{12}$ of the lamina can also be written as

$$\nu_{12} = \nu_m V_m + \nu_f V_f$$

(2.5)
where $\nu_m$ and $\nu_f$ are the Poisson’s rations for the matrix and the fibers, respectively. The transverse modulus of elasticity $E_{22}$ can be written as

$$E_{22} = E_f E_m \left( E_m V_f + E_f V_m \right)$$

Finally, the shear modulus $G_{12}$ is expressed as

$$G_{12} = G_f G_m \left( G_m V_f + G_f V_m \right)$$

At the macromechanics level, the stress-strain relationship of uni-directional lamina can be sufficiently described using the generalized Hooke's law, as

$$
\begin{bmatrix}
\sigma_1 \\
\sigma_2 \\
\sigma_3 \\
\tau_{23} \\
\tau_{31} \\
\tau_{12}
\end{bmatrix} =
\begin{bmatrix}
C_{11} & C_{12} & C_{13} & 0 & 0 & 0 \\
C_{12} & C_{22} & C_{23} & 0 & 0 & 0 \\
C_{13} & C_{23} & C_{33} & 0 & 0 & 0 \\
0 & 0 & 0 & C_{44} & 0 & 0 \\
0 & 0 & 0 & 0 & C_{55} & 0 \\
0 & 0 & 0 & 0 & 0 & C_{66}
\end{bmatrix}
\begin{bmatrix}
\varepsilon_1 \\
\varepsilon_2 \\
\varepsilon_3 \\
\gamma_{23} \\
\gamma_{31} \\
\gamma_{12}
\end{bmatrix}
$$

where $\sigma_i (i = 1, 2, 3)$ and $\varepsilon_i (i = 1, 2, 3)$ are the normal stresses and strains in the three principal material directions (see Figure 2.4), respectively, and $\tau_{ij} (i,j = 1, 2, 3)$ and $\gamma_{ij} (i,j = 1, 2, 3)$ are the shear stresses and strains, respectively, and $C_{ij}$ are stiffness coefficients. For a thin orthotropic shell, transverse strains are negligible, and therefore, it can be shown that

$$
\begin{align*}
\gamma_{13} &= 0 \quad \Rightarrow \quad \tau_{13} \neq 0 \\
\gamma_{23} &= 0 \quad \Rightarrow \quad \tau_{23} \neq 0 \\
\varepsilon_3 &= 0 \quad \Rightarrow \quad \sigma_3 = 0
\end{align*}
$$

As such, the constitutive equations can be simplified in the principal material directions of the orthotropic material as

$$
\begin{bmatrix}
\sigma_1 \\
\sigma_2 \\
\tau_{12}
\end{bmatrix} =
\begin{bmatrix}
Q_{11} & Q_{12} & 0 \\
Q_{12} & Q_{22} & 0 \\
0 & 0 & Q_{66}
\end{bmatrix}
\begin{bmatrix}
\varepsilon_1 \\
\varepsilon_2 \\
\gamma_{12}
\end{bmatrix}
$$

At the macromechanics level, the stress-strain relationship of uni-directional lamina can be sufficiently described using the generalized Hooke's law, as
where $Q_{ij}$ denote the reduced stiffness of an orthotropic lamina, and are related to the engineering properties measured along the principal directions, as given by

$$
Q_{11} = \frac{E_1}{1 - \nu_{12} \nu_{21}}; \quad Q_{12} = \frac{\nu_{12} E_2}{1 - \nu_{12} \nu_{21}}; \quad Q_{11} = \frac{E_2}{1 - \nu_{12} \nu_{21}}; \quad Q_{66} = G_{12}
$$

(2.11)

The above relations were developed for the principal materials directions in an orthotropic material. However, the principal directions of orthotropy often do not coincide with the geometric coordinate system, as evident in a helically wound glass FRP tube (see Figure 2.5). Transformation from the principal materials direction to an arbitrary coordinate system can easily be done, as shown in Figure 2.6, using the following equation:

$$
\begin{bmatrix}
\sigma_x \\
\sigma_\theta \\
\tau_{x\theta}
\end{bmatrix} =
\begin{bmatrix}
cos^2 \varphi & sin^2 \varphi & -2 \sin \varphi \cos \varphi \\
\sin^2 \varphi & \cos^2 \varphi & 2 \sin \varphi \cos \varphi \\
\sin \varphi \cos \varphi & -\sin \varphi \cos \varphi & \cos^2 \varphi - \sin^2 \varphi
\end{bmatrix}
\begin{bmatrix}
\sigma_1 \\
\sigma_2 \\
\tau_{12}
\end{bmatrix}
$$

(2.12)

where $\varphi$ is the angle of rotation. Similar transformations can be applied to the strains and material properties of the shell.

As stated earlier, nonlinearity in the off-axis direction could be significant. Hahn and Tsai (1973) used a complementary energy density function to derive nonlinear relations for in-plane shear. Hahn (1973) extended the nonlinear theory of unidirectional lamina to that of laminated composites. Lifshitz (1998) studied the shear modulus of T300-934 graphite/epoxy lamina with four layers at fiber orientations of $\pm 45^\circ$. Hu (1993) reported that unidirectional FRP may exhibit severe nonlinearity in its in-plane shear stress-strain relation. Also, some deviation from linearity may be observed under in-plane transverse loading, but the degree of nonlinearity is not comparable to that of the in-plane shear.

Haj-Ali and Kilic (2002) made coupon tests to calibrate three-dimensional micromechanical models for E-glass/vinyl-ester pultruded FRP. Tension, compression, and
shear tests were performed on off-axis coupons cut at different angles of 0°, 15°, 30°, 45°, 60°, and 90°. The overall linear elastic properties were identified along with the nonlinear stress-strain behavior under in-plane multi-axial tension and compression loading. The material had a lower ultimate tensile stress and initial stiffness in tension compared to the corresponding values from compression tests, regardless of thickness and orientation. This was attributed to the existing voids and micro-cracks that are more pronounced in matrix-mode tensile loading. The difference between the compressive and tensile properties increased for off-axis angles approaching 90°.

2.5 Applications of CFFT Beam-Columns

In 1995, Mirmiran and Shahawy developed the concept of concrete-filled FRP tubes to address the corrosion problems for the Florida Department of Transportation. Cabrera (1996) conducted several shear tests on short beams with square sections. Half of the beams had shear connectors, while the other half had smooth internal surface. The beams were loaded to failure in a four-point loading test. Test results demonstrated that slippage between FRP and concrete could be completely arrested using shear connectors. She also conducted a study on the seismic performance of concrete-filled FRP tube based on a cyclic stress-strain model. Mirmiran et al. (1998b) presented the design, fabrication and testing of the new hybrid column with internal shear connector ribs in the longitudinal and transverse directions. Based on the experimental interaction diagram for the proposed column, Mirmiran et al. (1999a) showed the new system to be equivalent to RC columns with over 5% steel reinforcement.
Seible et al. (1995) studied the composite carbon shell systems for bridge columns under seismic loads. The new structural system was evaluated experimentally under simulated seismic loads using pilot test units which incorporated different design assumptions with the objective of matching or exceeding the performance of conventionally reinforced concrete columns. Consequently, a rational and comprehensive design procedure was developed to incorporate the mechanical characteristics of advanced composite materials into the design methodology conventional columns.

Mirmiran et al. (2000a) further showed that off-the-shelf tubes with no surface preparation could be comparable to reinforced or prestressed concrete columns. The CFFT system, with an equivalent reinforcement ratio of only 1.4%, performed better than a 6% reinforced concrete section or a prestressed concrete section with 33% more concrete area. Fam (2000) also studied the behavior of concrete-filled FRP tubes under axial/flexural loading conditions. The specimens ranged from 3.5 to 34 ft span length and from 3½ to 37 in diameter. Mirmiran et al. (2000b) investigated the enhancement of serviceability, strength and ductility of concrete-filled tubes under flexural loads by partial prestressing of the section. They recommended to keep the effective prestress within 10%-25% of the unconfined strength of concrete core, and to limit tendon eccentricities to the kern distance to avoid tensile stresses at the interface of the concrete core and the FRP tube, unless the tube is equipped with ribs or shear connectors.

Davol et al. (2001) extended the studies of concrete-filled tubes to carbon FRP laminates. They tested two different laminate structures. The first shell with a lay-up of \([90^\circ, \pm 10^\circ_2, 90^\circ, \pm 10^\circ_2, 90^\circ]_{\text{sym}}\) experienced a compression failure due to local buckling at a
longitudinal strain of 0.55%. The second shell with a lay-up of \([90^\circ, \pm10^\circ, 90^\circ, \pm10^\circ, 90^\circ, \pm10^\circ, 90^\circ, \pm10^\circ, 90^\circ, 90^\circ, \pm10^\circ, 90^\circ, 90^\circ, \pm10^\circ, 90^\circ]\) failed in compression at a much higher strain of 0.83%.

Fan et al. (2000) studied the seismic performance of reinforced concrete columns confined by FRP tube under low cycle reverse loading. The FRP tubes acted as formwork as well as lateral confining system. There was a 2 in. gap between FRP tubes and the footing of columns. The experimental results showed that the FRP tube did not increase the capacity of the column, but it greatly enhanced its ductility ratio up to 10.

Yuan et al. (2002) proposed a novel hybrid GFRP tube filled with concrete. They reported that the FRP tube enhanced the strength of concrete by \(2\frac{1}{2}\) times. Fiber orientations in the tubes were \(\pm45^\circ\). Coupons of the tube were cut and tested under monotonic tension and compression. Tests showed a bilinear response in tension (Figure 2.7) and compression (Figure 2.8), with similar moduli of elasticity. However, the ultimate tensile strength was lower than the ultimate compressive strength.

### 2.6 Evaluation Criteria for Seismic Performance

In search of finding appropriate measures for CFFT systems, several criteria for evaluating seismic performance of a concrete structure are discussed in this section.

#### 2.6.1 Ductility

Ductility of a member is defined as its ability to sustain inelastic deformations prior to collapse, without significant strength or stiffness degradation. A ductile system displays sufficient warning before catastrophic failure. Ductility is generally measured as the ratio of the ultimate deformation to that at the equivalent yielding of steel reinforcement. The
selected deformation component may be deflection, curvature, or rotation. However, such
definition of ductility is not directly applicable to FRP or FRP-reinforced concrete (FRP-RC)
members. Mirmiran et al. (1999b) summarized the following two ductility factors for FRP-
RC members, especially CFFT members:

1. The yield-based ductility index follows the conventional definition of ductility, as
   \[ \mu_D = \frac{\Delta_u}{\Delta_y} \]  
   where \( \Delta_u \) and \( \Delta_y \) are the ultimate and yield deflections, respectively. The ultimate
deflection is generally considered to be the deflection at the time of collapse as long
as the load drops is no more than 15% of the capacity of the member (Park and
Paulay 1975). The yield deflection is defined as that of an equivalent elasto-plastic
system with the same elastic stiffness and ultimate load as those of the real system
(Figure 2.9). This definition was first suggested by Park (1989) for use with nonlinear
materials or when various parts of a system commence their yielding at different load
levels.

2. The energy-based ductility index takes into account the portion of the total
   absorbed energy, \( E_{TOT} \) (i.e., toughness), that is elastically released at the time of
   failure, \( E_{EL} \). This definition was first suggested by Naaman and Jeong (1995) for
   concrete beams prestressed with FRP tendons, and is given by
   \[ \mu_D^* = \frac{1}{2} \left( \frac{E_{TOT}}{E_{EL}} + 1 \right) \] 
   For elasto-plastic materials such as steel, both measures of ductility result in exactly
   the same value. For nonlinear systems, Naaman and Jeong (1995) suggest that the unloading
   slope at failure \( s \) be approximated as the weighted average of the two initial slopes \( s_1 \) and \( s_2 \)
of the load-deflection curve (Figure 2.10). This has been justified for CFFTs by several uniaxial cyclic load tests (Mirmiran and Shahawy 1997a). Both ductility measures (yield-based and energy-based ratios) indicate that whereas CFFTs are not as ductile as the conventional RC columns in the tension failure region, they are quite comparable to RC sections at higher levels of axial load.

Under cyclic loading, due to opening and closing of crack and apparent stiffness degradation, performance of concrete structures depends greatly on the loading regime. Kaku and Asakusa (1991) proposed the concept of the cumulative ductility factor, which is used to evaluate the deformability and energy dissipation capacity of concrete structures. As shown in Figure 2.11, the cumulative ductility factor (CDF) may be defined as the sum of the ductility ratios at each loading cycle up to the failure of the structure, and normalized as follows:

\[
CDF = \sum \left( n \times \frac{\delta_n}{\delta_1} \right) / \delta_1
\]  

(2.15)

### 2.6.2 Stiffness Degradation

Numerous experimental studies have shown that repeated cyclic loading may cause three distinct effects in the response of an RC structure: (a) reduction in stiffness, (b) deterioration in strength, and (c) a characteristic known as pinching, which is caused primarily by bond-slip behavior of reinforcement. Mander et al. (1988) studied the cyclic behavior of steel-RC columns, and put forth a cyclic model for confined RC sections taking into account the first two of these distinct effects, as shown in Figure 2.12, and described below:

1. When RC column is completely unloaded to zero stress, there is a plastic strain, \( \varepsilon_{p_l} \),
2. When RC column is reloaded again from a certain reloading strain of $\epsilon_{ro}$ to the same unloading strain, $\epsilon_{un}$, there is a strength reduction from $f_{un}$ to $f_{new}$, which leads to a stiffness degradation for the confined concrete; and

3. After the transition region, the reloading curve finally returns to the envelope curve of the monotonic response.

### 2.6.3 Pinching Effect

When concrete is subjected to cyclic loading, the shape of its stress-strain curve is greatly influenced by the shape of the stress-strain loop for the steel, because the applied moment is carried largely by the reinforcement placed in the section, especially after the first yielding. The Bauschinger effect of steel makes the stress-strain relationship curved and accompanied by some rounding. The so-called “pinching effect” appears in the hysteretic stress-strain response of some RC members, when subjected to cyclic loading. It reflects the strength decay due to several factors, primarily the bond-slip of the reinforcement, yielding of the reinforcement, and the loss of shear resistance after formation of cracks (Monti and Spacone 2000). As shown in Figure 2.13, the pinching effect implies that there is less energy dissipation per cycle than in the generally assumed parallelogram of classical elasto-plastic behavior.

The pinching effect is often a direct result of bar slippage. This effect is especially important in structures predominantly controlled by shear. When there is considerable slippage, the bond strength could be reduced by as much as 50%, resulting in a loss of stiffness. This effect is usually accumulated with the cracking of the concrete itself.
Since there is no quantitative measure for the pinching effect, in this dissertation it is defined as the ratio between the maximum width of the hysteretic response and its width at the origin. The higher the ratio is, the larger the pinching effect would be.

### 2.6.4 Energy Absorption Capacity

Energy absorption capacity was first adopted as a measure of seismic performance by Gosain et al. (1977), who proposed a simple cumulative energy ratio, as follows:

\[
D_e = \sum_i \frac{F_i \delta_i}{F_y \delta_y} \tag{2.16}
\]

where \(F_i\) and \(\delta_i\) are the force and deflection at the \(i^{th}\) loop, respectively, and \(F_y\) and \(\delta_y\) are the force and deflection at the yield point, respectively. Only the hysteretic loops for which \(F_i/F_y > 0.75\) are included in the summation. This is based on the assumption that when the peak force has dropped below 75% of the yield force, the remaining capacity of the member may be considered negligible.

Nmai and Darwin (1986) used the concept of energy dissipation index \(D_i\) to evaluate the cyclic performance of an RC member. The energy dissipation index normalizes the total dissipated energy \(E\) with respect to the elastic energy \(E_y\) at the initial yield for an equivalent full span beam, and is given by

\[
D_i = \frac{E}{E_y} = \frac{E}{0.5 \cdot P_y \cdot \Delta_y \cdot [1 + (\frac{A_i}{A_y})^2]}
\tag{2.17}
\]

where \(P_y\) is the yielding load of top flexural reinforcement in the beam, \(\Delta_y\) is the load-point deflection at yield of top flexural reinforcement, \(A_y\) is the area of top reinforcement at the
face of the support, and \( A' \) is the area of bottom reinforcement at the face of the support. From the above equation, it is clear that the higher the value of \( D_i \), the better the performance of the member would be under seismic loads.

### 2.6.5 Damage Index

The best known and most widely used damage index is the cumulative index suggested by Park and Ang (1985) and summarized by Williams and Sexsmith (1995), which consists of a simple linear combination of normalized deformation and energy absorption:

\[
D = \frac{\delta_m}{\delta_u} + \beta_e \int \frac{dE}{F_y \delta_u}
\]

(2.18)

where \( \delta_m \) is the maximum deformation experienced, \( \delta_u \) is the ultimate deformation under monotonic loading, \( F_y \) is the calculated yield strength, \( dE \) is the dissipated energy increment, and

\[
\beta_e = (-0.447 + 0.73 \frac{l}{d} + 0.24n_0 + 0.314 \rho_t) \cdot 0.7 \rho_w
\]

(2.19)

where \( l/d \) is the ratio of shear span to beam depth, \( n_0 \) is the normalized axial force, \( \rho_w \) is the reinforcement ratio for confining steel, and \( \rho_t \) is the reinforcement ratio for the longitudinal steel.

The above damage index consists of two terms. The first term is a simple, pseudo-static displacement measure. It takes no account of the cumulative damage, which is accounted for solely by the energy term. The advantages of this model are its simplicity and the fact that it has been calibrated against a significant number of observed seismic damages,
including some instances of shear and bond failure. Park et al. (1985) suggested the use of $D = 0.4$ as a threshold value between repairable and irreparable damage, while the same authors later (Park et al. 1987) suggested a more detailed classification, as reported in Table 2.2.

A number of difficulties arise in using Equation (2.18). A major problem is the determination of the ultimate deformation $\delta_u$ and the strength deterioration parameter $\beta_e$. Park and Ang (1985) proposed regression equations for the two parameters related to the confining reinforcement ratio and material strengths. However, the equation for $\beta_e$ yields very small values, so that the energy term makes a negligible contribution to the overall index, which therefore takes virtually no account of the hysteretic effect. In this dissertation, no attempt was made at developing an equivalent damage index for the CFFT specimens, due to the small number of specimens in the test database.
Table 2.1 Mechanical Properties of Typical Thermoset Resins (Hollaway, 1990)

<table>
<thead>
<tr>
<th>Properties</th>
<th>Polyester Resin</th>
<th>Epoxy Resin</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density (lb/ft³)</td>
<td>74.9-87.4</td>
<td>68.7-84.3</td>
</tr>
<tr>
<td>Tensile Strength (ksi)</td>
<td>6.5-13.1</td>
<td>5.8-14.5</td>
</tr>
<tr>
<td>Compressive Strength (ksi)</td>
<td>14.5-36.3</td>
<td>14.5-29.0</td>
</tr>
<tr>
<td>Modulus of Elasticity in Tension (ksi)</td>
<td>363-580</td>
<td>435-798</td>
</tr>
<tr>
<td>Poisson’s Ratio</td>
<td>0.37-0.40</td>
<td>0.38-0.40</td>
</tr>
<tr>
<td>Coefficient of Thermal Expansion (10⁶/°C)</td>
<td>100-120</td>
<td>45-65</td>
</tr>
<tr>
<td>Shrinkage at Curing (%)</td>
<td>5-8</td>
<td>1-2</td>
</tr>
</tbody>
</table>

Table 2.2 Damage Index Classifications (Park and Ang 1985)

<table>
<thead>
<tr>
<th>Category</th>
<th>Damage Index</th>
<th>Damage Level</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>$D &lt; 0.1$</td>
<td>No damage or localized minor cracking</td>
</tr>
<tr>
<td>2</td>
<td>$0.1 \leq D &lt; 0.25$</td>
<td>Minor damage: light cracking throughout</td>
</tr>
<tr>
<td>3</td>
<td>$0.25 \leq D &lt; 0.4$</td>
<td>Moderate damage: severe cracking, localized spalling</td>
</tr>
<tr>
<td>4</td>
<td>$0.4 \leq D &lt; 0.8$</td>
<td>Severe damage: concrete crushing, reinforcement exposed</td>
</tr>
<tr>
<td>5</td>
<td>$D \geq 0.8$</td>
<td>Collapse</td>
</tr>
</tbody>
</table>
Figure 2.1 Mechanical Properties of Polymers (Hollaway 1990)

Figure 2.2 Mechanical Properties of Fibers
Figure 2.3 Modes of Failure in a Laminate (Berthelot 1995)

Figure 2.4 Macromechanics of FRP Composites (Hollaway 1990)
Figure 2.5 Helically Wound Fiber Reinforced Cylindrical Shell (Jones 1975)

Figure 2.6 Positive Rotation of Principal Material Axes from Arbitrary XY axes (Jones 1975)
Figure 2.7  Tensile Test Result of GFRP Coupons (Yuan et al. 2002)

Figure 2.8  Compressive Test Result of GFRP Coupons (Yuan et al. 2002)
Figure 2.9  Definition of Displacement-Based Ductility
(Park and Paulay 1975, Mirmiran et al. 1999)

Figure 2.10  Definition of Energy-Based Ductility
(Naaman and Jeong 1995, Mirmiran et al. 1999)
Figure 2.11 Cumulative Ductility Factor (Kaku and Asakusa 1991)
Figure 2.12 Stiffness Degradation

Figure 2.13 Pinching Effects (Budek et al. 2002)
3.1 Test Plan

A detailed experimental program was carried out with three different components, as follows, to evaluate the cyclic behavior of concrete-filled FRP tubes (CFFTs):

1. Beam-column tests: Six (6) large-scale CFFTs with two different types of FRP tubes and three different levels of internal steel reinforcement were tested under constant axial compression loading and reversed cycles of transverse loading in four-point flexure. The objective of these tests was to establish the cyclic response of hybrid FRP-concrete columns with and without steel reinforcement.

2. Stub tests: Twenty-four (24) short stubs of FRP-wrapped concrete were tested under cycles of loading and unloading in uniaxial compression. The objective of these tests was to develop the cyclic model for concrete confined with different amounts of FRP.

3. FRP coupons: Twenty-five (25) coupons cut from the two types of FRP tubes were tested under cycles of loading and unloading in uniaxial tension or compression, as well as reverse cyclic loading in tension and compression. The objective of these tests was to establish the material models for the FRP tubes.

A description of each of the tests is provided in the following sections.
3.2 Large-Scale CFFT Beam-Column Tests

3.2.1 Test Specimens

Six (6) large-scale beam-column specimens were prepared with two different types of FRP tubes and three different levels of internal steel reinforcement. The test matrix for the CFFT beam-columns is shown in Table 3.1. The CFFT system consisted of three materials: FRP tube, concrete, and reinforcing steel. A description of each of the constituents of the system is provided below.

**FRP Tubes:** Two types of FRP tubes were used in this study, hereafter identified as white and yellow tubes, based on their surface color. These types of tubes have in the past been tested under monotonic loading for the Florida Department of Transportation (Mirmiran 1999a, Mirmiran et al. 2000a and 2000b and El Khoury 1999). The white tubes were also used for pile driving in Florida (Mirmiran et al. 2002, Mirmiran and Shahawy 2003), as well as for the time-dependent creep and shrinkage studies (Naguib 2000, Naguib and Mirmiran 2002a and 2002b).

The white tubes were manufactured by centrifuge or spin casting. The procedure involves cutting and placing appropriate layers of fabric inside a steel mandrel, and then rotating the mandrel while injecting the resin from inside. The centrifugal force moves the resin to the inner surface of the steel mandrel, and binds it to the fabric, leaving minimal (if any) void in the laminate. The white tubes were made of E-glass fibers and epoxy resin (Figure 3.1). The tubes had an outside diameter of 12 in and a wall thickness of 0.5 in. The wall thickness, however, was not entirely made of a structural laminate, but consisted of a 0.18 in resin-rich inner layer, and a 0.02 in exterior white gel coat. The interior layer is inherent to the manufacturing process, as the excess resin to ensure proper wetting of the
fabric. It does also serve as a liner in pipe flow applications (Figure 3.2). The exterior layer provides ultra-violet (UV) protection for outdoor applications. The manufacturer’s data indicated that if the thickness of the resin-rich layer and the exterior gel coat were excluded from the cross-sectional area, the average tensile, compressive and flexural strength of the FRP tube would be 57.9 ksi, 55.7 ksi and 81.0 ksi, respectively. The tube consisted of 40 plies, with a symmetric fiber architecture of $[0^\circ/0^\circ/+45^\circ/-45^\circ]_{10}$ starting from the outside of the tube. The subscript 10 shows the repetition of the 4 ply angles ten times in the same order. Table 3.2 shows the mechanical properties of the white tubes. The coupon tests were carried out by the Owens Corning Corp. The burnout test by the same company determined the glass content of the tubes as 51.2%.

The yellow tubes were made using the filament winding process with 17 layers of E-glass fibers at $\pm 55^\circ$ angle and a thermosetting epoxy resin. The winding angle was optimized for pipe flow applications with a 2:1 pressure loading in the hoop direction versus the longitudinal direction. Figure 3.3 shows the yellow tubes with an outside diameter of 12.68 in and a wall thickness of 0.2 in. The manufacturer data includes cyclic pressure rating of 300-150 psig, static pressure rating of 450-225 psig, a maximum rated temperature of 210°F, a normal weight of 11 lb/ft, and a coefficient of thermal expansion of $8.8 \times 10^{-6}/^\circ F$. The tube has no internal lining. The material properties for this tube at 75°F temperature are listed in Table 3.3, as provided by the manufacturer and the independent coupon tests by the Owens Corning Corp. The burnout test by the same company determined the glass content of the tubes as 75.5%. Table 3.4 compares the mechanical properties of the two types of tubes. Note that the manufacturing process and the fiber architecture for the yellow tubes resulted in a
compressive strength higher than the tensile strength. Although not typical for most FRP materials, this may be attributed to the fiber orientation, as explained in Chapter 2.

**Concrete:** One batch of concrete was used for filling the tubes, and another for the end blocks. The concrete was always ordered from a local plant and arrived ready-mixed. It included type I Portland cement with no admixtures. The target slump for all batches of concrete was 4 in. The target strength for the concrete in the tubes and in the end blocks were 4 and 5 ksi at 28 days, respectively. Two (2) of the specimens (W1 and W2) were cast at the University of Cincinnati (UC), while the other four (4) specimens (Y1, Y2, Y3, and W3) were cast at the North Carolina State University (NCSU). The average 28-day compressive strength for cylinders taken form the concrete batch at the UC was 3.35 ksi. The average compressive strength for those in NCSU was 3.94 ksi.

**Reinforcing Steel:** The reinforcing steel used in the specimens was Grade 60 with manufacturer reported yield strength of 70 ksi, a yield strain of 0.43%, and an ultimate strength of 100 ksi. The details of the reinforcing steel cages inside the tubes are shown in Figure 3.4 to 3.6. The longitudinal bars inside the tubes were No. 4, 5, or 6 (i.e., 0.5, 0.625, or 0.75 in diameter), while the hoop bars were all No. 4 (0.5 in diameter) placed at 4 in on center. There was no steel reinforcement at the mid-span region of Specimens W1 and Y1. Therefore, only two short steel cages (Figure 3.4) with extended dowel bars (Figure3.5) were placed at both ends. For all other specimens, a long steel cage (Figure 3.6) and a short steel cage (Figure 3.4) were placed inside the tube with a lap splice of 30 in, which is consistent with the provisions of the ACI 318-99 for seismic applications.
3.2.2 Specimen Fabrication

The FRP tubes provided the formwork for concrete. Only a wooden plate was necessary at the bottom for the extended dowel bars. The tubes were first cut to the proper length, and were then cleaned. A length of 6 in was included for each tube at either end for embedment into the end blocks. The steel cages were first pulled inside the tubes while laid horizontally. A clear cover of 0.5 in was maintained for the longitudinal bars using plastic spacers. The tubes with the internal steel cage and the bottom wood form were then erected vertically against a frame or scaffolding. Prior to casting of concrete, the bottom edge of the tubes and the holes in the wood plate were sealed with Silicon (Figure 3.7). Concrete was placed from a hopper, and was compacted using an immersion type vibrator. Top surfaces of the specimens were then covered with polystyrene sheets to maintain a moist environment. The specimens were lifted and laid horizontally three (3) days after casting.

Formwork for the end blocks were prepared with plywood (Figure 3.8), while the concrete in the tubes was being air cured. Figure 3.9 shows the placement of the CFFT specimens into the end blocks. Positioning and leveling of the tubes and the end blocks were made using a laser pen and a leveling device. The end blocks were heavily reinforced in all directions. The reinforcement cage was made in two separate pieces to accommodate the placement of the tubes (Figure 3.10). A clear cover of 0.5 in was maintained for the reinforcement in the end blocks. The cages were tied to the dowels from the CFFT specimens and to an ASTM 193 Grade B7 alloy high strength steel rod with a smooth surface. The rod was intended to act as a hinge. After all joints in the form and around the rod were sealed with Silicon, and after applying form release to the inside of the forms, the end blocks were filled with concrete. After casting the end blocks, the top surfaces were covered with
polystyrene sheets to maintain a moist environment. Three (3) days after casting, the specimens were de-molded for air curing (Figure 3.11).

3.2.3 Instrumentation

The primary interest in the instrumentation of the CFFT beam-columns was concerned with the strain distribution and deflection at the mid-span section, which is in a region of pure flexure without any shear force. Figure 3.12 shows the schematics of the instrumentation plan. The level of axial load was monitored using a load cell fabricated from a rectangular steel tube and a full bridge of strain gages (Type CEA-06-250UW-120 by Measurements Group, Inc.), as shown in Figure 3.13. The load cell was calibrated using the 1,000-kip Baldwin machine (Baldwin-Lima-Hamilton Corp.) in the lab. Figure 3.14 shows how the mid-span deflections were monitored using a cable-extension transducer (string potentiometer) with a 15 in stroke (PT-100 Series by Celesco Tranducer Products, Inc.). The horizontal movements of the end blocks were also monitored by wire potentiometers of similar type. The difference between the two end measurements was used to find the axial shortening of the specimens.

Eight (8) post-yielding electrical resistance strain gages (Type YFLA-5-3L by Tokyo Sokki Kenkyujo Co.) were attached to the longitudinal bars of at the mid-span section of those specimens that had internal steel reinforcement (Figure 3.15). The wires from the strain gages were extended along the bars to one end of the tube. They were then bundled together around a hoop bar, and were taken outside through a hole cut in the tube (Figure 3.16).

Sixteen (16) electrical resistance strain gages (Type PFL-30-3L by Tokyo Sokki Kenkyujo Co.) were affixed to the outside surface of each tube at its mid-span section. Eight
were in the hoop direction, while the other eight were in the longitudinal direction (Figures 3.17 and 3.18). Table 3.5 shows the technical parameters for all electrical resistance strain gages used for the specimens and for the load cell.

Since strain gages may fail before the specimen reaches its ultimate capacity, additional 4 in long PI gages (Type PI-5-100 by Tokyo Sokki Kenkyujo Co., Ltd.) with a strain capacity of 0.005 in/in were used to measure the longitudinal strains at the top and bottom of each specimen at its mid-span section (Figure 3.19).

The slippage between the end blocks and the FRP tube were monitored using 3 in stroke potentiometers (BEI Duncan Electrics Division Sensors & Systems Company) (Figure 3.20). Each potentiometers was affixed to a magnetic base on the end block and would make contact with an aluminum angle that was glued to the surface of the FRP tube.

The rotations of the end blocks were monitored using inclinometers (Lucas Control Systems, Corp.) (Figure 3.21). The inclinometer was first nailed onto a wood plate. The plate was then glued onto the end block. Since the reinforced concrete end block could be regarded as a rigid body, any point on this rigid body would have the same rotation.

The vertical movements at the supports were also measured to assess the flexibility and movement of the entire test system under cyclic loading (Figure 3.22). The readings were also used to correct the mid-span deflections, if necessary.

Load and stroke data were directly taken from the MTS controller for the actuator (Figure 3.23). Readings from the transducers and the strain gages were recorded using a computer controlled high speed Megadac data acquisition system.
3.2.4 Test Setup and Procedure

Figure 3.24 shows the schematics of the test setup. Each specimen was first loaded in the axial direction to 100 kips, which was about 6%-10% of its axial capacity. The specimen was then subjected to transverse loading in four point flexure, while maintaining the axial load constant. The flexural span was 8 ft for Specimens W1 and W2 at the University of Cincinnati, and 9 ft for Specimens W3, Y1, Y2, and Y3 at the NCSU. Figure 3.25 shows the general view of the test setups in the laboratories at the UC and the NCSU.

The lateral load was applied by a hydraulic actuator reacting on a steel frame. The frame at the UC was self-reacting, while the frame at the NCSU was post-tensioned to the strong floor. The axial load was applied using an ENERPAC hydraulic jack reacting on a separate horizontal frame that was linked to the specimen using a hinge connection at either end. The frame consisted of 8-ft long ASTM 193 Grade B7 high strength alloy steel coarse threaded rods coupled together and tied to steel I-beams (Figure 3.26). An 11 in by 8 in steel plate was embedded in each of the two end blocks of the specimen to provide a smooth loading surface in the axial direction, as shown in Figure 3.27.

The pin supports for transverse loading were simulated using a 2 in diameter high strength roller, which extends outside the end blocks on both sides. At the UC, the rods were inserted in slots within 6 in x 8 in steel tubes (Figure 3.27). The tubes at the UC were attached to the self reacting frame. At the NCSU, a support system was designed to accommodate the rods by tightening top and bottom plates with four bolts (Figure 3.22). The supporting systems at the NCSU were post-tensioned to the strong floor. In both cases, the supporting systems were designed to minimize flexibility and movement in the vertical (transverse) direction. Moreover, the rods and all contact surfaces in the supporting systems
were well greased to accommodate axial shortening and horizontal movements of the specimen, i.e., roller support conditions.

The transverse cyclic loading was applied through a central loading system that consisted of a 2 in and a 2½ in thick steel plate at the top and bottom, respectively. The top plate was bolted to the actuator. The plates were tied together with four 1¼ in ASTM 193 Grade B-7 high strength alloy steel coarse threaded rods. The system would allow downward loading by direct bearing on top, while upward loading is applied by tensioning the threaded rods. The rods were tensioned sufficiently to minimize any slack in the system. Each loading point consisted of a roller, a round loading plate and a neoprene pad to ensure adequate rotation capacity and lateral friction. (Figures 3.19 and 3.28).

**Loading Regime:** While there is no universally accepted loading regime for reverse cyclic tests, typically the displacement corresponding to the yielding of steel reinforcement is sought first as the yield displacement \( \Delta_y \), and then the displacements are increased as multiples of the reference displacement \( \Delta_y \). The advantage for this method is the straightforward ductility reading for each cycle and a cumulative ductility factor. However, for FRP-concrete specimens without internal steel reinforcement, the reference displacement must be defined and may be different for different specimens. Therefore, for the purpose of consistency, it was decided that a uniform loading regime be applied to all specimens. Figure 3.29 shows the selected loading regime, during which the specimen would be first pushed down +¼ in and pulled up -¼ in for two consecutive cycles. Subsequently, two repeated cycles of larger displacements, ±½, ±¾, ±1, ±1½, ±2, ±2½, ±3, ±4, and ±5 in, would be applied, unless the specimen would fail earlier.
The 5 in stroke limit was selected to avoid significantly large end rotations which could result in lateral instability of the axial loading system. However, the chosen stroke limit should for the most part provide a reasonably large deflections (1/20 of the span length) and large end rotation (15°) to assess ductility and flexibility of the system.

If the specimen did not fail after the specified loading regime, it was then subjected to a monotonic four point flexure test without the presence of axial load. The specimen was typically pushed down to determine maximum deflection at failure. Both the cyclic and the monotonic loading tests were displacement control at a rate of 1 in/min up to 2 in displacement and 2 in/min afterwards.

The top and bottom surfaces of the tubes were typically cut at the mid-span section to examine the crack pattern in concrete and yielding of any internal steel reinforcement (Figure 3.30).

3.2.5 Test Observations and Failure Modes

The white specimens failed at a relatively low deflection as compared to the yellow specimens. Specimens W1, W2, and W3 failed at deflections of 3, 2½ and 2 in, respectively. Specimens Y1, Y2, and Y3 failed at deflections of 10, 11, and 12 in, respectively. It appears that with larger amount of internal steel, the white specimens showed less deformability, while the yellow specimens showed greater deformability. This could be attributed to the fact that the white specimens had a much greater FRP reinforcement to begin with, and acted as over-reinforced specimens (Mirmiran 1999, El Khoury 1999, and Mirmiran et al. 2000b).

The white tubes generally failed by local crushing under one of the loading points (Figure 3.31), which represents a compression-controlled failure of an over-reinforced
section. The failure was at times accompanied by a subsequent splitting or horizontal shear rupture at about 1/3 height of the specimen extending to the end block (Figure 3.32). Local buckling of the tube wall was also observed in white specimens (Figure 3.33). Autopsy of the specimens revealed crushing of concrete at the mid-span section. All cracks in concrete were flexural. Once the crushed concrete was removed, the internal reinforcing steel did not appear to have been overly stressed, as no obvious sign of necking or elongation was observed.

The yellow specimens proved to be under-reinforced, as they showed clear tension-controlled failure (Figure 3.34). Therefore, they benefited from increased amount of internal steel reinforcement, in that deformability and capacity were both increased for larger amounts of internal steel. The FRP tube did not show any sign of crushing in compression. Autopsy of the yellow specimens revealed widely distributed flexural cracks (Figure 3.30) and yielded steel bars, but no sign of concrete crushing.

3.2.6 Test Results and Discussions

In this section, test results for various aspects of the beam-column specimens are presented.

**Load-Deflection Response:** Figures 3.35-3.40 show the load-displacement hysteretic curves for all six beam-column specimens. Since the test specimens had different concrete strengths, cross-sections, and span lengths, it was decided to normalize the data for comparison. The lateral load is normalized as $\alpha P/(f'_c D^2)$, where $P$ is the total applied load in the transverse direction, $f'_c$ is the 28-day compressive strength of concrete, $D$ is the concrete core diameter or the internal diameter of the tube, and $\alpha$ is the moment arm ratio, which is
equal to 1.0 for the specimens at the NCSU and 0.925 (i.e., 37/40) for the specimens at UC. The deflection is normalized as $\delta/L$, where $\delta$ is the mid-span deflection and $L$ is the flexural span length. The envelope curves for all specimens are plotted in Figure 3.41.

The white specimens generally exhibited a linear behavior, small ultimate deflections, narrow hysteretic response, and little energy dissipation. As stated earlier, the white tubes could be considered as over-reinforced sections (Mirmiran 1999, El Khoury 1999, and Mirmiran et al. 2000b). Specimen W1 without any internal steel reinforcement showed more load cycles and higher deflections of up to 3 in, as compared to the other two. Specimen W3 with 2.55% steel reinforcement ratio, on the other hand, had a premature failure at 2 in deflection. Specimen W2 with 1.7% steel reinforcement ratio deflected about 2½ in, but showed higher strength and better energy dissipation characteristics with wider hysteretic loops than the other two specimens. From the envelope curves, it appears that internal steel reinforcement improves the stiffness of the white specimens, although it does not contribute to their strength.

The yellow specimens generally exhibited a non-linear elasto-plastic behavior, large ultimate deflections, wide hysteretic loops, and significant energy dissipation. The internal reinforcement not only contributed to the initial and secondary stiffness of the yellow specimens, but also increased their ultimate capacity. As discussed earlier, the yellow tubes could be considered as under-reinforced sections (Mirmiran 1999, El Khoury 1999, and Mirmiran et al. 2000b). Specimen Y3 with 2.55% steel reinforcement ratio had the best hysteretic response among all specimens.

**Consistency of Axial Loads:** Although the tests were designed with a constant axial load, some variation in the axial load was observed throughout the test. The primary source
of variation was the mid-span deflections and end rotations of specimens, and because the axial loading system was not servo-controlled. For the tests at the UC, a hydraulic jack was placed within the self-reacting frame to apply the axial load. The tests at the NCSU were performed by post-tensioning the threaded rods with a hydraulic jack from outside the self-reacting frame. Therefore, for the system at the UC it was possible to adjust the level of axial load as the load dropped or increased during the test. The system at the NCSU, however, did not allow that flexibility. Nonetheless, in both cases, the load level was allowed to fluctuate. The effect of these fluctuations is included in the total moment capacity, which is the sum of primary moments and the secondary P-Δ effects.

Figures 3.42-3.47 show the fluctuations of axial loads around its constant value of 100 kips. For the white specimens, the axial load varied within a range of ±20% at mid-span deflections of about 3 in. For the yellow specimens, the ±20% fluctuation occurred at the mid-span deflections of about 4 in. It is important to note that in a real column, as the mid-span deflections increase to 1/33-1/25 of its height, the axial loads are likely to be redistributed to the neighboring columns.

Strains in Internal Steel Reinforcement: Figures 3.48-3.51 show the load versus steel strain curves for Specimens W2, W3, Y2, and Y3, respectively. The loads are normalized, as discussed before. The curves are plotted for the outermost layer of internal steel reinforcement in each specimen. In each figure, the yield strain for Grade 60 steel is marked. In general, these curves resemble the load-displacement curves for the same specimens. Within the yield limits, the strains in steel reinforcement are quite stable. Specimen W3 shows little or no yielding for the most part, indicating the premature compression-controlled failure of the specimen. The steel reinforcement in Specimen W2
only reached twice the yield strains, whereas those in the yellow specimens exceeded 6 times the yield strains. The wide hysteretic loops for Specimens Y2 and Y3 (Figures 3.50 and 3.51) indicate one of the reasons for the large energy dissipation and ductility of these specimens.

**Depth of Neutral Axis:** Figures 3.52-3.57 show the load versus depth of neutral axis at the mid-span sections of each CFFT specimen. The loads are normalized, as discussed before. The depth of neutral axis is calculated from the strains at the top and bottom of the tubes. The depth is then normalized as a fraction of the concrete core diameter. A slanted Z shape hysteretic curve can be seen in each figure. This is mainly due to the presence of an axial load. Every time that the specimen cycles through the zero lateral load, the axial load causes the neutral axis to reach infinity. For pure flexure with no axial load, one would expect a slanted S shape. The hysteretic loops for the white specimens are generally dense and narrow, whereas those for the yellow specimens are wider and show larger variation of the neutral axis.

**Moment-Rotation Response:** Figures 3.58-3.61 show the hysteretic curves of the total mid-span moment versus end rotations for Specimens Y1, Y2, Y3, and W3, respectively. The end rotations were not measured for Specimens W1 and W2 that were tested at the UC. The moments are normalized as $M_{\text{total}}/(f'_cD^3)$, where $M_{\text{total}}$ is the sum of primary moment from the lateral loads and the secondary moments due to the P-Δ effects, $f'_c$ is the 28-day compressive strength of concrete, and $D$ is the concrete core diameter or the internal diameter of the tube. Specimen Y1 had the largest end rotations, since it was the most flexible specimen. Specimen W3, on the other hand, had the smallest end rotation, as it was the most stiff or rigid specimen. The shape of curves for the white specimen is quite different from those of the yellow specimens, due to the linear behavior, higher strength, and less end
rotation of the white tubes. Specimen Y2 had a smaller total mid-span moment than Specimen Y1, even though it was subjected to a higher lateral load. This was because of the larger P-Δ effects in Specimen Y1.

End Slippage of FRP Tubes: Four potentiometers were used to measure slippage of the FRP tubes out of the end blocks at top and bottom on both ends. The maximum slippage for Specimens Y1, Y2, Y3, and W3, was measured as 0.07, 0.02, 0.03, and 0.11 in, respectively. The slippage was not measured for Specimens W1 and W2 that were tested at the UC. The slippage in all cases is not very significant and could be neglected. The higher slippage of Specimen W3 could be attributed to its smooth surface as well as its higher rigidity, which could not accommodate end rotations.

Axial Shortening: The axial shortening of the specimens were measured as the difference between the horizontal displacements of the two end blocks. The shortening of Specimens Y1, Y2, Y3, and W3, was measured as 0.45, 0.26, 0.22, and 0.19 in, respectively. The axial shortening was not measured for Specimens W1 and W2 that were tested at the UC. Specimen Y1 was the most flexible specimen, and therefore, had the largest axial shortening, which was 1/240 of its span length. Specimen W3 was the most rigid specimen, and therefore, had the least axial shortening, which was 1/356 of its span length. In general, the axial shortening during the tests could be neglected.

Hoop Strains: Figures 3.62-3.73 show the hoop versus axial strains at the top and bottom of the six CFFT specimens. The hoops strains generally varied between -0.002 and +0.004. The curves show a slightly nonlinear relationship between the hoop and axial strains primarily with an opposite sign. It means that when the tube is in tension in the axial direction, its hoop strain at the same point is in compression. This is mainly due to the
Poisson’s effect for the tube. The confinement also plays a role on the compression side of the specimen. The unsymmetrical shape of the curves may be attributed to the presence of an axial compressive load at all times.

**Energy Dissipations:** Cumulative energy dissipation is an important measure of the seismic performance of a structural member. In earthquake events, structures often fail due to the accumulation of small deflections, rather than a single large deformation. Figure 3.74 shows the cumulative energy dissipation capacity for all six beam-columns. The cumulative energy is calculated as the area of the hysteretic load-deflection curves. Since both the loads and the displacements were normalized, the cumulative energy is also non-dimensionalized. The slope of the cumulative energy curves represents the energy dissipation in each individual cycle, or the rate of the dissipated energy. In all specimens, the rate of energy dissipation increased significantly at higher load cycles. The figure also shows that the yellow specimens provided much greater energy dissipation capacity than their white counterparts, both in terms of the magnitude of energy dissipated as well as the number of cycles to failure. An increase of the internal steel reinforcement generally resulted in an increase of the energy dissipation capacity, except for Specimen W3 that failed prematurely.

**Ductility:** Ductility is another important measure of the seismic performance of a structural member. As discussed in Chapter 2, there are two definitions of ductility that could be extended to FRP structures: displacement-based ductility and energy-based ductility. In this section, both types of ductility index are calculated for the CFFT beam-columns using their load-deflection curves. Figure 3.75 shows both the displacement-based ductility and the energy-based ductility for the white specimens. Both ductility factors are less than 3 for the white specimens. It is also clear that as the steel reinforcement ratio is increased, both
ductility measures are decreased for the white specimens. This may be attributed to the fact that white specimens were over-reinforced with the FRP shell itself, and the additional steel reinforcement did not improve their ductility. Both definitions of ductility factors, however, appear to be appropriate and reasonably close for the white specimens.

Figure 3.76 shows the displacement-based ductility for the yellow specimens using the secondary monotonic loading that followed the cyclic tests. Note that the yellow specimens did not fail during the cyclic loading up to 5 in stroke (about 1/20 of the span length). The energy-based ductility factor could not be calculated for the yellow specimens using the cyclic test results, since the ultimate limit state was not captured in cyclic loading. It could not be calculated using the secondary monotonic test results, either, because of prior damages that would affect the total dissipated energy. The figure shows that with the increase of internal steel reinforcement, the displacement-based ductility factor increases significantly. The ductility factors for the yellow specimens ranged between 18 and 22, which were much higher than those of the white specimens. In summary, it is recommended that a minimum ductility factor of 4 to 5 be used for CFFT members.

**Pinching Effect**: The definition of pinching effect used in this study was explained in Chapter 2. The pinching factor is defined as a non-dimensional ratio of the maximum and minimum widths of the last cycle of the load-deflection hysteretic response. The lower the pinching factor, the less pinching effect is observed. The primary reason for the pinching effect is the slippage between the yielded steel reinforcement and the surrounding concrete during the opening and closing of concrete cracks under cyclic loading. Figure 3.77 shows the variation of pinching factor for the CFFT specimens as a function of the steel reinforcement ratio. It is clear that for the same amount of internal steel reinforcement, the
pinching factors for the yellow and white specimens are almost the same. Also, as the reinforcement ratio increases, the pinching effect becomes less noticeable. The only exception in the above both statements is Specimen W3, which failed prematurely and at a slightly higher pinching factor. In the same figure, equivalent pinching factors for a few RC and CFST columns from test data (Priestley et al. 1996, Budek et al. 2002, Elremaily and Azizinamini 2002) in the literature have been shown for comparison. It can be seen that pinching in CFFT is comparable to RC beam-columns.

**Secondary Monotonic Tests:** As discussed earlier, the yellow specimens did not fail under cyclic loading up to a stroke of 5 in (1/20 of the span length). Due to excessive end rotations, it was decided to stop the cyclic tests. After removing the axial load, the specimens were monotonically loaded downward (push) until failure in the same test frame. Figures 3.78-3.80 show the load-deflection curves for the monotonic tests in comparison with the corresponding cyclic tests for Specimens Y1, Y2, and Y3, respectively. Due to significant accumulated damages, all secondary monotonic tests showed less initial stiffness than their respective cyclic tests. Figure 3.81 shows a comparison of the load-deflection curves for the three yellow specimens. It can be seen that the reserved capacity and deflection of the specimens increase with the amount of internal steel reinforcement.

**Comparison with RC Beam-Columns:** In this section, the observed hysteretic behavior of CFFT beam-columns is compared with test data of a typical RC beam-column from the literature.

Priestley and Benzoni (1996) studied the seismic performance of circular columns with low longitudinal reinforcement ratios. A column with a diameter of 24 in and a height of 72 in was tested under lateral cyclic loading at the top. The axial load was 5.7% of the
section capacity. The longitudinal steel reinforcement ratio $\rho_s$ was 0.53%, the volumetric steel reinforcement ratio $\rho_v$ was 0.78% and the strength of concrete was 4.36 ksi. The ductility factor for this specimen reached 12. The normalized hysteretic curve for this column is plotted with those of Specimens W1 (Equivalent $\rho_v=18.34\%$) and Y1 (Equivalent $\rho_v=1.14\%$) in Figures 3.82 (a) and (b), respectively. In comparison with RC, Specimen Y1 has more deformation and energy dissipation capacity, while Specimen W1 has a much higher strength.

3.3 FRP-Confined Concrete Stub Tests

3.3.1 Test Specimens

Twenty-four (24) FRP-confined concrete stubs were tested under cycles of loading and unloading in axial compression. The test matrix is shown in Table 3.6. All specimens were 6 in diameter cylinders with 12 in height, made of the same batch of concrete with a 28-day compressive strength of 5.83 ksi. Two types of wraps were used: glass and carbon. Both types of fibers were unidirectional and were wrapped only in the hoop direction. Specimens were labeled as R, C, or G, for the reference (or control), carbon-wrapped, or glass-wrapped specimens, respectively. For the wrapped specimens, the specimen label is followed by two numbers, where the first one represents the number of layers (1 or 2), and the second number indicates the loading regime (1 to 6). The loading regime is discussed in more detail in Section 3.3.4.

The same epoxy, Sikadur Hex 300 (from Sika Corp.), was used for wrapping carbon and glass fibers. It is a two-component, moisture-tolerant, high strength, and high modulus epoxy, with a clear amber color. The epoxy has a pot life of 4 hours and a tack free time of
20 hours. It has a tensile strength of 10.5 ksi, a tensile modulus of 459 ksi, and a maximum elongation of 4.8%. Its flexural strength and modulus are 17.9 ksi and 452 ksi, respectively.

Glass sheets were SikaWrap Hex 100G (from Sika Corp.). They are unidirectional E-glass fabrics with a white color, a weight of 27 oz per square yard, a tensile strength of 330 ksi, a tensile modulus of 10,500 ksi, and a density of 0.092 lbs/in³. Maximum elongation of glass fibers is 4%. The equivalent thickness of fabric per layer is 0.0226 in. The GFRP laminate with the above epoxy has a tensile strength of 88.8 ksi, a tensile modulus of 3,790 ksi, and a maximum elongation of 2.45%. The thickness of each ply of laminate is 0.04 in. Table 3.7 provides manufacturer data on the GFRP laminate and its components.

Carbon sheets were SikaWrap Hex 103C (from Sika Corp.). They are high strength, unidirectional fabric with a black color, a weight of 18 oz per square yard, a tensile strength of 550 ksi, a tensile modulus of 34,000 ksi, and a density of 0.065 lbs/in³. Maximum elongation of carbon fibers is 1.5%. The CFRP laminate with the above epoxy has a tensile strength of 123.2 ksi, a tensile modulus of 10,240 ksi, and a maximum elongation of 1.12%. The thickness of each ply of laminate is 0.04 in. Table 3.8 provides manufacturer data on the CFRP laminate and its components.

### 3.3.2 Specimen Fabrication

All concrete cylinders were compacted using a steel rod. After 14 days of air curing, surface of the specimens were cleaned with a steel brush. The fabric sheets were cut to appropriate lengths for one or two layers of wraps. A minimum of 4 in of overlap was included for each layer of fabric. The two components of epoxy were mixed for 5 minutes, using a paddle style drill mixer at a low speed of 400-600 rpm until consistency was achieved.
Using a brush, the epoxy was applied to the concrete surface until fully saturated. The fabric was also fully saturated with the same epoxy using a brush and a roller. The saturated fabric was then wrapped around the concrete stubs. Additional epoxy was applied as an overcoat to ensure full wetting of the fabric. Excess epoxy and potential voids were rolled out on the surface. After the epoxy cured for at least 72 hours, all specimens were capped with sulphur mortar (Figure 3.83). The specimens were tested at least one week after wrapping.

### 3.3.3 Instrumentation

The stub test set-up is shown in Figures 3.84 and 3.85. Two Linear Variable Differential Transformer (LVDTs) were installed to monitor the axial displacement of the specimens. Four strain gages were pasted at the mid-height of each specimen, two in the axial direction, and the other two in the hoop direction. Readings from the LVDTs, strain gages, and load and stroke signals from the test machine were recorded using a computer-controlled high speed Megadac data acquisition system. An ultrasonic pulse velocity system was also set up at the mid-height of the specimen to monitor the development of internal cracks. The ultrasonic testing is reported elsewhere (Wei 2000).

### 3.3.4 Test Setup and Procedure

A Tinius Olsen universal testing machine with a 400-kip capacity was used at the University of Cincinnati (UC) to apply the cyclic loading. The reference specimens (R1, R2, and R3) were all monotonically loaded to failure. The wrapped specimens were subjected to one of the following loading regimes (also, see Figure 3.86 and Table 3.6):

1. Monotonic loading from virgin (0%) to failure (100%);
2. Loading from virgin (0%) to 40% of capacity, unloading to 0%, and re-loading to failure (100%);

3. Loading from virgin (0%) to 60% of capacity, unloading to 0%, and re-loading to failure (100%);

4. Loading from virgin (0%) to 80% of capacity, unloading to 0%, and re-loading to failure (100%);

5. Loading from virgin (0%) to 80% of capacity, unloading to 40%, and re-loading to failure (100%); and

6. Loading from virgin (0%) to 80% of capacity, unloading to 60%, and re-loading to failure (100%).

### 3.3.5 Test Observations and Failure Modes

Figures 3.87 and 3.88 show the typical failure modes for the one and two layer glass wraps, respectively. Figures 3.89 and 3.90 show the typical failure modes for the one and two layer carbon wraps, respectively. In all cases, failure was sudden, accompanied with rupture of the fibers in the hoop direction and a large explosive sound. However, there was no advance sign of failure. No clear difference was observed between the failure of one and two layer wraps of each type. On the other hand, rupture of the GFRP wraps appeared to be more contained within the mid-height of the specimens, whereas the CFRP wraps basically “unzipped” through the entire height of the specimens.
3.3.6 Test Results and Discussions

In this section, test results for various aspects of the FRP-confined specimens are presented.

**Stress-Strain Response:** The axial stress versus axial and hoop strains for the one and two layer GFRP and CFRP specimens are shown in the same scale in Figures 3.91-3.94. The initial slope of the curves resembles that of concrete core, since there is no fiber in the axial direction. As concrete begins to dilate extensively, the response curve becomes softer. The bend point in the response is about 70% of the ultimate strength in almost all specimens. It appears that single layer of GFRP or CFRP does not provide any strength enhancement for concrete, while an increased ductility is observed. The ultimate strains for the GFRP stubs are higher than those of their CFRP counterparts with the same number of layers. Therefore, confinement with GFRP provides better ductility for concrete. On the other hand, ultimate strengths of CFRP specimens are much higher than those of GFRP specimens. The differences in strength and ductility may be attributed to the difference in strengths and elastic moduli of the CFRP and GFRP wraps.

**Confinement Efficiency:** Confinement efficiency is defined as the ratio of the strength of confined concrete to that of unconfined concrete. It is generally accepted that the confinement efficiency is a function of the confinement ratio \( CR \), as given by:

\[
CR = \frac{2f_j t_j}{f_{\text{co}} D}
\]  

(4.1)

where \( f_j \) is the hoop strength of the FRP wrap, \( t_j \) is the thickness of the wrap, \( f_{\text{co}} \) is the compressive strength of unconfined concrete, and \( D \) is the concrete core diameter. The confinement ratios \( CR \) for each of the tested stubs are listed in Table 3.9. There appears to be a threshold of \( CR \) between 0.3 and 0.4 for full confinement effect. At lower levels of \( CR \),
the wrap only contributes to ductility but not strength of concrete. This is often termed as partial confinement.

**Effects of Unloading and Reloading Levels:** Cyclic response of FRP-confined concrete depends on the level of unloading, and whether it occurs prior to or after the bend point in the envelope curve. The unloading curve can be characterized as linear elastic when unloading takes place below the bend point (see Figure 3.95). On the other hand, when concrete is loaded beyond the bend point, its unloading curve is clearly nonlinear with a distinct plastic or residual strain (see Figure 3.96).

The shape of the reloading curve is generally linear. However, the reloading stress has a pronounced effect on the strength degradation. Figure 3.97 shows a typical reloading from a point near zero stress level. The reloading curve does not return to the original unloading point, but rather a lower stress level, with a distinct strength degradation and much energy dissipation. Figure 3.98 shows a typical reloading from a point just below the unloading point. The reloading curve almost fully returns to the previous unloading point, with negligible strength degradation and little energy dissipation.

**Volumetric Strains:** Lateral expansion of concrete under compression is what activates the confinement of FRP wraps. Volumetric strain $\varepsilon_v$ is the algebraic sum of the strains along three dimensions (Mirmiran and Shahawy, 1997a), as given by

$$\varepsilon_v = \varepsilon_c + 2\varepsilon_r$$

(4.2)

Where $\varepsilon_c$ is the axial strain and $\varepsilon_r$ is the lateral (radial) strain. The axial stress versus volumetric strains for two of the stubs, G22 and C25, under different loading regimes are plotted in Figures 3.99 and 3.100, respectively. Positive values indicate volume expansion, while negative values mean contraction. In both cases, concrete is initially compressed until
about 70% of its unconfined strength. After that concrete begins to dilate, activating the FRP confinement. Since the wraps are not thick (only one or two layers), no dilation reversal occurs. Unloading and reloading portions of the volumetric strains are quite wider for the GFRP wraps, as compared to the CFRP wraps. In neither case, however, the unloading and reloading curves follow the initial compaction rate of concrete.

3.4 FRP Coupon Tests

The manufacturer data for the FRP tubes as well as independent coupon test data by Owens Corning Corp. were provided in Tables 3.2 and 3.3 earlier in this chapter. However, the information was found insufficient for modeling of cyclic behavior for two reasons: (a) no data on cyclic behavior of the FRP materials were available, and (b) the nonlinear behavior of the FRP materials, as expected from the response of yellow beam-columns, was not captured without tracing the entire stress-strain response. Therefore, coupon from both types of FRP tubes were tested under cycles of loading and unloading in axial tension or compression, as well as reverse cyclic loading in tension and compression.

3.4.1 Specimen Preparation

A total of twenty-five (25) coupons were cut from the untested sections of both types of FRP tubes. The tensile coupons were 12 in long to provide an adequate gripping length at each end. The compressive coupons were 2 in long to avoid buckling failure. Both types of coupons were 1½ in wide. The test matrix is shown in Table 3.10. For each type of FRP tubes, four series of coupon testing were carried out: monotonic tension, monotonic compression, cyclic tension, and cyclic compression. For the tensile tests of yellow coupons,
more specimens were tested to obtain at least three valid and consistent test data. Additional tests were carried out for yellow specimens with reverse cyclic loading in tension and compression. Tables 3.11-3.15 show the geometric properties of the white and yellow coupons.

3.4.2 Instrumentation, Test Setup and Procedure

For each cyclic test group, coupons were loaded up to a certain level and then unloaded to zero. The unloading levels were randomly selected in order to study the full range of cyclic behavior. Some coupons were unloaded once then reloaded to failure, while others were cyclically loaded at two or three different levels to get additional test data.

Tension Tests: Uniaxial tension tests followed the ASTM D3039 (2002). Tests were performed in MTS test machine. The specimens were instrumented with a 1¼ in stain gage in the middle. One 1.75 in stroke potentiometer was placed on either side of the coupon to measure the specimen deformations and average strains. Tensile load was applied at a displacement rate of 0.015 in/min for both monotonic and cyclic tests. The dimensions and gripping of a typical sample are shown in Figure 3.101.

Compression Tests: Uniaxial compression tests followed the ASTM D695-02a (2002). Tests were performed in Forney test machine. No strain gage was placed on the specimens. However, two 1.75 in stroke potentiometers were used to measure deformations and average strains. Test setup is shown in Figure 3.102.
3.4.3 Test Observations and Failure Modes

**Tension Tests of White Coupons:** Mode of failure for all tension coupons of the white tubes was identical, whether subjected to monotonic or cyclic loading. Prior to fiber rupture, cracking sound could be heard with increasing frequency. The sound was attributed to cracking of the resin rich layer in the construct of the FRP tube. With the increased cracking of the resin, the load was transferred to the fibers. Failure was always sudden, with a burst rupture of fibers almost always at the mid-height of the coupons. Figure 3.103 shows a typical failure with ruptured fibers and granulated resin.

**Tension Tests of Yellow Coupons:** Failure of the yellow coupons was distinctly different from the white coupons, in that there was no advance warning prior to failure (Figure 3.104). Failure in half of the specimens occurred at or around the mid-height, while for the other half, failure occurred at the grips. Moreover, failure was not accompanied with any noise nor any violent fiber rupture. Failure could only be detected initially from the load drop in the test machine. There are two primary reasons for the observed behavior. First and most important is the fiber orientation in the yellow coupons. Since they were cut along the length of the tube, the fibers were at ±55°. Therefore, for a 12 in long and 1½ in wide coupon, no continuous fiber was present along the whole length of the tube. As such, the behavior was predominantly dictated by that of the resin and interlaminar shear. Secondly, the fiber volume fraction for the yellow tubes was 75.5% versus 52% for the white tubes. Also, there was no resin lining on the interior surface. Therefore, the resin was not as strong as that in the white tubes.

**Compression Tests of White Coupons:** All compression coupons of white tubes failed identical to each other. The failure always occurred at one end. The specimen failed
just like a broom, by separating the glass plies and the resin-rich inner layer at one end (Figure 3.105). Those compression coupons without any lateral bracing failed due to local buckling. Such a failure was always accompanied with a huge sound and sudden energy release.

**Compression Tests of Yellow Coupons:** The compression coupons of yellow tubes failed at the mid-heights of the specimens, either in a horizontal direction or in the direction of the fibers. Careful observation revealed that the fracture lines on the two faces of each coupon were not at the same height. The axial compressive forces resulted in lateral tension and sheared off the resin for the most part. However, the direction of resin failure depended on the imperfections of the laminate and the alignment of the fibers with respect to the loading plates (Figure 3.106).

### 3.4.4 Test Results and Discussions

Figure 3.107 shows the monotonic stress-strain curves of white coupons in tension. The response is generally linear. The softening that occurs at about 25% of the capacity may be attributed to the fracture of resin rich inner layer. Rupture occurs at an average stress of 75 ksi and an average strain 0.023. The secant slope of the curve at the ultimate strength is 3,250 ksi. The manufacturer data and coupon tests by Owens Corning Corp. are also shown in the figure, and summarized in Table 3.16. By comparison, the secant modulus of elasticity is close to the manufacturer data, whereas the ultimate strength is slightly higher than the manufacturer data.

The cyclic stress-strain response of three white coupons in tension are shown in Figures 3.108-3.110. The response is almost linear. The unloading curves normally return to
the original point, while slight energy dissipation and stiffness degradation are observed during the unloading and reloading cycles.

Figure 3.111 shows the monotonic stress-strain curves of white coupons in compression. Similar to their tensile response, the white specimens behaved almost linearly to failure, which occurs at an average stress of 60 ksi and an average strain of 0.023. The secant slope of the curve at the ultimate strength is 2,900 ksi. The manufacturer data and coupon tests by Owens Corning Corp. are also shown in the figure, and summarized in Table 3.17. By comparison, the secant slope is lower than the manufacturer data, whereas the ultimate strength is very close to that reported by the manufacturer.

The cyclic stress-strain response of two white coupons in compression are shown in Figures 3.112 and 3.113. The response is generally linear with unloading curves returning to the origin. The energy dissipation and stiffness degradation during the unloading and reloading cycles could therefore be neglected.

Figure 3.114 shows the monotonic stress-strain curves of yellow coupons in tension. Significant non-linearity can be seen until rupture occurs at an average stress of 5.5 ksi and an average strain of 0.0045. The secant slope of the curve at the ultimate strength is 1,500 ksi. The manufacturer’s data and coupon tests by Owens Corning Corp. are also shown in the figure, and summarized in Table 3.16. By comparison, the initial slope is close to the manufacturer data, whereas the ultimate strength is only 70% of the manufacturer data.

The cyclic stress-strain response of three yellow coupons in tension are shown in Figures 3.115-3.117 The response is significantly nonlinear with considerable energy dissipation during the unloading and reloading cycles. The degradation of both stiffness and strength can be seen from the figures.
Figure 3.118 shows the monotonic stress-strain curves of yellow coupons in compression. Similar to their tension behavior, the yellow specimens showed significant non-linearity in compression. Rupture occurs at an average stress of 23 ksi and an average strain of 0.045. The secant slope of the curve at the ultimate strength is 563 ksi. The manufacturer data and coupon tests by Owens Corning Corp. are also shown in the figure, and summarized in Table 3.17. By comparison, the initial slope is about 1/3 to ½ of the manufacturer data, whereas the ultimate strength is slightly less than that reported by the manufacturer.

The cyclic stress-strain response of three yellow coupons in compression are shown in Figures 3.119-3.121. The response is significantly nonlinear with considerable energy dissipation during the unloading and reloading cycles. However, the unloading curves show no plastic or residual strains, as they almost return to the origin.

The above-mentioned tests investigated the monotonic or cyclic behavior only within compression regime or tension regime. To investigate the transition behavior of yellow coupons from tension to compression or from compression to tension, reverse cyclic tests were carried out. Table 3.15 provided the geometric properties of the two yellow coupons. Figures 3.122 and 3.123 show the reverse cyclic behavior in tension and compression. Due to limitations of the testing equipment, overall buckling always occurred in compression. However, test data at least show the conceptual outline of the entire hysteretic response, which is very helpful for cyclic modeling of the yellow tubes, as will be discussed in Chapter 4.
Table 3.1 Test Matrix of FRP-Concrete Beam-Column Specimens

<table>
<thead>
<tr>
<th>FRP Type</th>
<th>White Specimens</th>
<th>Yellow Specimens</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specimen Name</td>
<td>W1</td>
<td>W2</td>
</tr>
<tr>
<td>Test Location</td>
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<td>UC</td>
</tr>
<tr>
<td>Span Length, $L$ (in)</td>
<td>96</td>
<td>96</td>
</tr>
<tr>
<td>Core Diameter, $D$ (in)</td>
<td>11.00</td>
<td>11.00</td>
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<tr>
<td>Tube Thickness, $t$ (in)</td>
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<td>0.5</td>
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<td>Steel Reinforcement</td>
<td>None</td>
<td>8#4</td>
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<tr>
<td>Steel Reinforcement Ratio, $\rho_s$ (%)</td>
<td>0.0</td>
<td>1.65</td>
</tr>
<tr>
<td>$L/D$ Ratio</td>
<td>8.7</td>
<td>8.7</td>
</tr>
<tr>
<td>$D/t$ Ratio</td>
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<td>22.0</td>
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<tr>
<td>Reinforcement Index, $\omega$</td>
<td>3.29</td>
<td>3.59</td>
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### Table 3.2 Mechanical Properties of White Tubes

<table>
<thead>
<tr>
<th>Properties</th>
<th>Manufacturer Data</th>
<th>Coupon Tests (Owens Corning)</th>
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<tr>
<td></td>
<td>Value (ksi)</td>
<td>Value (ksi)</td>
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<tr>
<td></td>
<td>ASTM Standard</td>
<td>ASTM Standard</td>
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<tr>
<td>Axial Tension</td>
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<tr>
<td>Strength</td>
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<tr>
<td>Modulus</td>
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<td>Axial Compression</td>
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<td>65.63</td>
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<tr>
<td>Modulus</td>
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<td>Beam Flexure</td>
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<td>Strength</td>
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<tr>
<td>Modulus</td>
<td>3592.74</td>
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### Table 3.3 Mechanical Properties of Yellow Tubes

<table>
<thead>
<tr>
<th>Properties</th>
<th>Manufacturer Data</th>
<th>Coupon Tests (Owens Corning)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Value (ksi)</td>
<td>Value (ksi)</td>
</tr>
<tr>
<td></td>
<td>ASTM Standard</td>
<td>ASTM Standard</td>
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<tr>
<td>Axial Tension</td>
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<tr>
<td>Strength</td>
<td>10.3</td>
<td>9.5</td>
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<tr>
<td>Modulus</td>
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<td>1671</td>
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<td>Axial Compression</td>
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<tr>
<td>Strength</td>
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<td>Modulus</td>
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<tr>
<td>Beam Flexure</td>
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<tr>
<td>Strength</td>
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<tr>
<td>Modulus</td>
<td>2180</td>
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Table 3.4 Comparison of White and Yellow Tubes

<table>
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<tr>
<th>Description</th>
<th>White Tubes</th>
<th>Yellow Tubes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness</td>
<td>0.5 in</td>
<td>0.2 in</td>
</tr>
<tr>
<td>Axial Tensile Strength</td>
<td>57.9 ksi</td>
<td>10.3 ksi</td>
</tr>
<tr>
<td>Tensile Modulus of Elasticity</td>
<td>2256 ksi</td>
<td>1820 ksi</td>
</tr>
<tr>
<td>Axial Compressive Strength</td>
<td>55.7 ksi</td>
<td>33.3 ksi</td>
</tr>
<tr>
<td>Compressive Modulus of Elasticity</td>
<td>3373 ksi</td>
<td>1260 ksi</td>
</tr>
<tr>
<td>Flexural Strength</td>
<td>81 ksi</td>
<td>25 ksi</td>
</tr>
<tr>
<td>Flexural Modulus of Elasticity</td>
<td>3564 ksi</td>
<td>1179 ksi</td>
</tr>
<tr>
<td>Fiber Architecture</td>
<td>(0°, 0°, ±45°)₁₀</td>
<td>(±55°), Total = 17</td>
</tr>
<tr>
<td>Fabrication Method</td>
<td>Spin Casting</td>
<td>Filament Winding</td>
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<tr>
<td>Glass Content</td>
<td>51.2%</td>
<td>75.5%</td>
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</table>
Table 3.5 Technical Properties of Electrical Resistance Strain Gages

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<thead>
<tr>
<th>Gage Location</th>
<th>Reinforcing Steel</th>
<th>FRP Tube</th>
<th>Load Cell</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gage Label</td>
<td>WFLA-6-11-3LT</td>
<td>PFL-30-11-3L</td>
<td>CEA-06-250UW-120</td>
</tr>
<tr>
<td>Manufacturer</td>
<td>Tokyo Sokki Kenkyujo Co., Ltd</td>
<td>Tokyo Sokki Kenkyujo Co., Ltd</td>
<td>Measurements Group, Inc.</td>
</tr>
<tr>
<td>Length</td>
<td>¼ in</td>
<td>1¼ in</td>
<td>½ in</td>
</tr>
<tr>
<td>Gage Factor</td>
<td>2.12±1%</td>
<td>2.13±1%</td>
<td>2.065±0.5%</td>
</tr>
<tr>
<td>Temperature Compensation</td>
<td>11x10⁻⁶/°C</td>
<td>11x10⁻⁶/°C</td>
<td>0</td>
</tr>
<tr>
<td>Transverse Sensitivity</td>
<td>-0.1%</td>
<td>-0.5%</td>
<td>(0.4±0.2)%</td>
</tr>
</tbody>
</table>

* All gages were 120 Ω.
Table 3.6 Matrix of FRP-Confined Stub Tests

<table>
<thead>
<tr>
<th>Specimen Name</th>
<th>Wrapping Material</th>
<th>Layer</th>
<th>Loading Pattern</th>
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</thead>
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<tr>
<td></td>
<td></td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td>R1</td>
<td>N.A.</td>
<td></td>
<td>0% 100%</td>
</tr>
<tr>
<td>R2</td>
<td></td>
<td></td>
<td>0% 100%</td>
</tr>
<tr>
<td>R3</td>
<td></td>
<td></td>
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</tr>
<tr>
<td>C11</td>
<td></td>
<td></td>
<td>0% 100%</td>
</tr>
<tr>
<td>C12</td>
<td></td>
<td></td>
<td>0% 40% 0% 100%</td>
</tr>
<tr>
<td>C13</td>
<td></td>
<td></td>
<td>0% 60% 0% 100%</td>
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<td>C14</td>
<td></td>
<td></td>
<td>0% 80% 0% 100%</td>
</tr>
<tr>
<td>C15</td>
<td></td>
<td></td>
<td>0% 80% 40% 100%</td>
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<tr>
<td>C16</td>
<td></td>
<td></td>
<td>0% 80% 60% 100%</td>
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<td>C21</td>
<td></td>
<td></td>
<td>0% 100%</td>
</tr>
<tr>
<td>C22</td>
<td></td>
<td></td>
<td>0% 40% 0% 100%</td>
</tr>
<tr>
<td>C23</td>
<td></td>
<td></td>
<td>0% 60% 0% 100%</td>
</tr>
<tr>
<td>C24</td>
<td></td>
<td></td>
<td>0% 80% 0% 100%</td>
</tr>
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<td>C25</td>
<td></td>
<td></td>
<td>0% 80% 40% 100%</td>
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<tr>
<td>C26</td>
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<td></td>
<td>0% 80% 60% 100%</td>
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<td>G11</td>
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<td></td>
<td>0% 100%</td>
</tr>
<tr>
<td>G12</td>
<td></td>
<td></td>
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<td>G13</td>
<td></td>
<td></td>
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<td>G14</td>
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<td></td>
<td>0% 80% 0% 100%</td>
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<tr>
<td>G15</td>
<td></td>
<td></td>
<td>0% 80% 40% 100%</td>
</tr>
<tr>
<td>G16</td>
<td></td>
<td></td>
<td>0% 80% 60% 100%</td>
</tr>
<tr>
<td>G21</td>
<td></td>
<td></td>
<td>0% 100%</td>
</tr>
<tr>
<td>G22</td>
<td></td>
<td></td>
<td>0% 40% 0% 100%</td>
</tr>
<tr>
<td>G23</td>
<td></td>
<td></td>
<td>0% 60% 0% 100%</td>
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<tr>
<td>G24</td>
<td></td>
<td></td>
<td>0% 80% 0% 100%</td>
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<tr>
<td>G25</td>
<td></td>
<td></td>
<td>0% 80% 40% 100%</td>
</tr>
<tr>
<td>G26</td>
<td></td>
<td></td>
<td>0% 80% 60% 100%</td>
</tr>
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</table>
### Table 3.7 Mechanical Properties of GFRP Laminates

<table>
<thead>
<tr>
<th>Materials</th>
<th>Tensile Property</th>
<th>Value (ksi)</th>
<th>Thickness (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>E-Glass Fibers</td>
<td>Tensile Strength</td>
<td>330</td>
<td>0.0226</td>
</tr>
<tr>
<td></td>
<td>Tensile Modulus</td>
<td>10,500</td>
<td></td>
</tr>
<tr>
<td>Epoxy</td>
<td>Tensile Strength</td>
<td>10.5</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td>Tensile Modulus</td>
<td>459</td>
<td></td>
</tr>
<tr>
<td>GFRP Laminate</td>
<td>Tensile Strength</td>
<td>88.8</td>
<td>0.04</td>
</tr>
<tr>
<td></td>
<td>Tensile Modulus</td>
<td>3,790</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Compressive Strength</td>
<td>86.6</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Compressive Modulus</td>
<td>4,313</td>
<td></td>
</tr>
</tbody>
</table>

### Table 3.8 Mechanical Properties of CFRP Laminates

<table>
<thead>
<tr>
<th>Materials</th>
<th>Tensile Property</th>
<th>Value (ksi)</th>
<th>Thickness (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Carbon Fibers</td>
<td>Tensile Strength</td>
<td>550</td>
<td>0.0134</td>
</tr>
<tr>
<td></td>
<td>Tensile Modulus</td>
<td>34,000</td>
<td></td>
</tr>
<tr>
<td>Epoxy</td>
<td>Tensile Strength</td>
<td>10.5</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td>Tensile Modulus</td>
<td>459</td>
<td></td>
</tr>
<tr>
<td>CFRP Laminate</td>
<td>Tensile Strength</td>
<td>123.2</td>
<td>0.04</td>
</tr>
<tr>
<td></td>
<td>Tensile Modulus</td>
<td>10,240</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Compressive Strength</td>
<td>113</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Compressive Modulus</td>
<td>9,726</td>
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</table>
### Table 3.9 Confinement Ratio for FRP-Wrapped Concrete Stubs

<table>
<thead>
<tr>
<th>Series</th>
<th>Tensile Strength of FRP, $f_j$ (ksi)</th>
<th>Thickness of FRP, $t_j$ (in)</th>
<th>Average Strength of Specimens, $f'_{cu}$ (ksi)</th>
<th>$f'<em>{cu}/f'</em>{co}$ *</th>
<th>CR</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-Layer Glass</td>
<td>88.8</td>
<td>0.04</td>
<td>7.20</td>
<td>1.23</td>
<td>0.203</td>
</tr>
<tr>
<td>2-Layer Glass</td>
<td>88.8</td>
<td>0.08</td>
<td>10.36</td>
<td>1.78</td>
<td>0.406</td>
</tr>
<tr>
<td>1-Layer Carbon</td>
<td>123.2</td>
<td>0.04</td>
<td>8.62</td>
<td>1.48</td>
<td>0.282</td>
</tr>
<tr>
<td>2-Layer Carbon</td>
<td>123.2</td>
<td>0.08</td>
<td>13.32</td>
<td>2.28</td>
<td>0.564</td>
</tr>
</tbody>
</table>

* The unconfined concrete strength $f'_{co}$ for all specimens was 5.83 ksi.
Table 3.10 Test Matrix of FRP Coupons

<table>
<thead>
<tr>
<th>Type of Specimens</th>
<th>Yellow</th>
<th></th>
<th>White</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Compression</td>
<td>Tension</td>
<td>Compression</td>
<td>Tension</td>
</tr>
<tr>
<td>Monotonic Test</td>
<td>3</td>
<td>3</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>Cyclic Test</td>
<td>3</td>
<td>3</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>Reverse Cyclic</td>
<td>2</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td>Width (in)</td>
<td>1.5</td>
<td>1.5</td>
<td>1.5</td>
<td>1.5</td>
</tr>
<tr>
<td>Total Length (in)</td>
<td>2</td>
<td>12</td>
<td>2</td>
<td>12</td>
</tr>
<tr>
<td>Gage Length (in)</td>
<td>2</td>
<td>6</td>
<td>2</td>
<td>6</td>
</tr>
<tr>
<td>Thickness (in)</td>
<td>0.2</td>
<td>0.2</td>
<td>0.5</td>
<td>0.5</td>
</tr>
<tr>
<td>Fiber Orientation</td>
<td>±55°</td>
<td>±55°</td>
<td>0°/0°/45°/45°</td>
<td>0°/0°/45°/-45°</td>
</tr>
<tr>
<td>Failure Mode</td>
<td>Shear</td>
<td>Shear</td>
<td>Lateral Tear-off and Local Buckling</td>
<td>Fiber Rupture</td>
</tr>
</tbody>
</table>
### Table 3.11 Geometric Properties of White Coupons in Tension

<table>
<thead>
<tr>
<th>Test Method</th>
<th>Specimen Name</th>
<th>Thickness (in)</th>
<th>Width (in)</th>
<th>Gage Length (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Monotonic</td>
<td>WTM1</td>
<td>0.35</td>
<td>1.50</td>
<td>5.90</td>
</tr>
<tr>
<td></td>
<td>WTM2</td>
<td>0.41</td>
<td>1.45</td>
<td>6.20</td>
</tr>
<tr>
<td></td>
<td>WTM3</td>
<td>0.31</td>
<td>1.46</td>
<td>6.10</td>
</tr>
<tr>
<td>Cyclic</td>
<td>WTC1</td>
<td>0.40</td>
<td>1.50</td>
<td>5.80</td>
</tr>
<tr>
<td></td>
<td>WTC2</td>
<td>0.38</td>
<td>1.51</td>
<td>5.70</td>
</tr>
<tr>
<td></td>
<td>WTC3</td>
<td>0.37</td>
<td>1.50</td>
<td>6.80</td>
</tr>
</tbody>
</table>

### Table 3.12 Geometric Properties of White Coupons in Compression

<table>
<thead>
<tr>
<th>Test Method</th>
<th>Specimen Name</th>
<th>Thickness (in)</th>
<th>Width (in)</th>
<th>Gage Length (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Monotonic</td>
<td>WCM1</td>
<td>0.40</td>
<td>1.45</td>
<td>1.89</td>
</tr>
<tr>
<td></td>
<td>WCM2</td>
<td>0.40</td>
<td>1.47</td>
<td>1.91</td>
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<tr>
<td></td>
<td>WCM3</td>
<td>0.40</td>
<td>1.47</td>
<td>1.82</td>
</tr>
<tr>
<td>Cyclic</td>
<td>WCC1</td>
<td>0.33</td>
<td>1.57</td>
<td>1.70</td>
</tr>
<tr>
<td></td>
<td>WCC2</td>
<td>0.33</td>
<td>1.65</td>
<td>1.74</td>
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</table>
Table 3.13 Geometry of Yellow Coupons in Tension

<table>
<thead>
<tr>
<th>Test Method</th>
<th>Specimen Name</th>
<th>Thickness (in)</th>
<th>Width (in)</th>
<th>Gage Length (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Monotonic</td>
<td>YTM1</td>
<td>0.20</td>
<td>1.49</td>
<td>7.50</td>
</tr>
<tr>
<td></td>
<td>YTM2</td>
<td>0.21</td>
<td>1.52</td>
<td>7.15</td>
</tr>
<tr>
<td></td>
<td>YTM3</td>
<td>0.21</td>
<td>1.52</td>
<td>5.75</td>
</tr>
<tr>
<td>Cyclic</td>
<td>YTC1</td>
<td>0.21</td>
<td>1.5</td>
<td>5.75</td>
</tr>
<tr>
<td></td>
<td>YTC2</td>
<td>0.22</td>
<td>1.49</td>
<td>6.90</td>
</tr>
<tr>
<td></td>
<td>YTC3</td>
<td>0.21</td>
<td>1.54</td>
<td>5.50</td>
</tr>
</tbody>
</table>

Table 3.14 Geometric Properties of Yellow Coupons in Compression

<table>
<thead>
<tr>
<th>Test Method</th>
<th>Specimen Name</th>
<th>Thickness (in)</th>
<th>Width (in)</th>
<th>Gage Length (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Monotonic</td>
<td>YCM1</td>
<td>0.22</td>
<td>1.45</td>
<td>1.57</td>
</tr>
<tr>
<td></td>
<td>YCM2</td>
<td>0.22</td>
<td>1.46</td>
<td>1.52</td>
</tr>
<tr>
<td></td>
<td>YCM3</td>
<td>0.22</td>
<td>1.47</td>
<td>1.48</td>
</tr>
<tr>
<td>Cyclic</td>
<td>YCC1</td>
<td>0.20</td>
<td>1.43</td>
<td>1.70</td>
</tr>
<tr>
<td></td>
<td>YCC2</td>
<td>0.19</td>
<td>1.56</td>
<td>1.68</td>
</tr>
<tr>
<td></td>
<td>YCC3</td>
<td>0.20</td>
<td>1.44</td>
<td>1.79</td>
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</table>
Table 3.15 Geometric Properties of Yellow Coupons in Reverse Cyclic Loading

<table>
<thead>
<tr>
<th>Test Method</th>
<th>Specimen Name</th>
<th>Thickness (in)</th>
<th>Width (in)</th>
<th>Gage Length (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cyclic</td>
<td>YCTC1</td>
<td>0.20</td>
<td>1.53</td>
<td>6.05</td>
</tr>
<tr>
<td></td>
<td>YCTC2</td>
<td>0.20</td>
<td>1.51</td>
<td>5.95</td>
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</tbody>
</table>
Table 3.16 Average Results of Tension Tests

<table>
<thead>
<tr>
<th>Fiber Orientation</th>
<th>White Tube</th>
<th>Yellow Tube</th>
</tr>
</thead>
<tbody>
<tr>
<td>0°/0°/45°/-45°</td>
<td>10, total 40 layers</td>
<td>±55°, total 17 layers</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Source</th>
<th>Manufacturer</th>
<th>Owen Corning</th>
<th>Present Study</th>
<th>Manufacturer</th>
<th>Owens Corning</th>
<th>Present Study</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ultimate Strength (ksi)</td>
<td>57.9</td>
<td>69.8</td>
<td>75.0</td>
<td>10.3</td>
<td>9.5</td>
<td>6.5</td>
</tr>
<tr>
<td>Ultimate Strain (in/in)</td>
<td>0.0257</td>
<td>0.0195</td>
<td>0.0230</td>
<td>0.0057</td>
<td>0.0057</td>
<td>0.0045</td>
</tr>
<tr>
<td>Initial Modulus of Elasticity (ksi)</td>
<td>2,256</td>
<td>3,579</td>
<td>6,800</td>
<td>1,820</td>
<td>1,672</td>
<td>4,000</td>
</tr>
<tr>
<td>Secant Modulus of Elasticity (ksi)</td>
<td>2,256</td>
<td>3,579</td>
<td>3,250</td>
<td>1,820</td>
<td>1,672</td>
<td>1,500</td>
</tr>
</tbody>
</table>
Table 3.17 Average Results of Compression Tests

<table>
<thead>
<tr>
<th>Properties</th>
<th>Manufacturer</th>
<th>Owens Corning</th>
<th>Present Study</th>
<th>Manufacturer</th>
<th>Owens Corning</th>
<th>Present Study</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum Stress, (ksi)</td>
<td>55.7</td>
<td>65.6</td>
<td>60.0</td>
<td>33.3</td>
<td>25.0</td>
<td>23.0</td>
</tr>
<tr>
<td>Maximum Strain, (in/in)</td>
<td>0.0165</td>
<td>0.0181</td>
<td>0.023</td>
<td>0.0057</td>
<td>0.0057</td>
<td>0.0044</td>
</tr>
<tr>
<td>Initial Modulus of Elasticity, (ksi)</td>
<td>3,373</td>
<td>3,629</td>
<td>2,900</td>
<td>1,820</td>
<td>1,179</td>
<td>750</td>
</tr>
<tr>
<td>Secant Modulus of Elasticity, (ksi)</td>
<td>3,373</td>
<td>3,629</td>
<td>2,900</td>
<td>1,820</td>
<td>1,179</td>
<td>563</td>
</tr>
<tr>
<td>Orientation</td>
<td>[0°/0°/45°/-45°]×10, total 40 layers</td>
<td>±55°, total 17 layers</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Figure 3.1 White Tube

Figure 3.2 Cross-Section of the White Tube
Figure 3.3   Yellow Tube

Figure 3.4   Short Steel Cage
Figure 3.5  Bottom View of Dowel Bars

Figure 3.6  Long Steel Cage
Figure 3.7 Wooden Base Plate before Casting Concrete

Figure 3.8 Formwork of an End Block
Figure 3.9  Overview of Specimens before Casting End Blocks

Figure 3.10  Placement of a CFFT Beam-Column into an End Block
Figure 3.11 Specimens after De-Molding End Blocks
Figure 3.12 Instrumentation for Large-Scale Beam-Column Tests
Figure 3.13  Load Cell to Measure Axial Load
Figure 3.14 String Potentiometer
Figure 3.15 Cross Sections of CFFT Beam-Column Specimens

(a) Specimens without Steel Reinforcement
(b) Specimens with Steel Reinforcement

Figure 3.15 Cross Sections of CFFT Beam-Column Specimens
Figure 3.16  Wires of Internal Strain Gages

Figure 3.17 Location of External Strain Gages
Figure 3.18  Electrical Resistance Strain Gages

Figure 3.19  PI Gages
Figure 3.20  Potentiometer

Figure 3.21  Inclinometer
Figure 3.22  Monitoring of Support Movement

Figure 3.23  440-kip 40-in Stroke Actuator
Figure 3.24: Large-Scale Beam-Column Test Setup (Schematic Drawing)
Figure 3.25 Overview of Large-Scale CFFT Beam-Column Tests

(a) UC
(b) NCSU
Figure 3.26  Horizontal Frame System with Threaded Rods

Figure 3.27  Steel Plate Embedded in End Block
Figure 3.28  Steel Plates and Vertical Loading System

Figure 3.29  Loading Regime for CFFT Beam-Column Tests
Figure 3.30 Distributed Cracking at the top of Specimen Y1

Figure 3.31 Crushing of White Tubes in Compression
Figure 3.32  Longitudinal Shear Splitting of White Tubes

Figure 3.33  Local Buckling of White FRP Tubes
Figure 3.34  Mid-Span Flexural Failure of Yellow Tubes
Figure 3.35 Normalized Load versus Deflection for Specimen W1

Figure 3.36 Normalized Load versus Deflection for Specimen W2
Figure 3.37 Normalized Load versus Deflection for Specimen W3

Figure 3.38 Normalized Load versus Deflection for Specimen Y1
Figure 3.39 Normalized Load versus Deflection for Specimen Y2

Figure 3.40 Normalized Load versus Deflection for Specimen Y3
Figure 3.41 Normalized Envelope Curves of Load versus Deflection
Figure 3.42 Axial Load versus Mid-span Deflection for Specimen W1

Figure 3.43 Axial Load versus Mid-span Deflection for Specimen W2
Figure 3.44 Axial Load versus Mid-span Deflection for Specimen W3

Figure 3.45 Axial Load versus Mid-span Deflection for Specimen Y1
Figure 3.46 Axial Load versus Mid-span Deflection for Specimen Y2

Figure 3.47 Axial Load versus Mid-span Deflection for Specimen Y3
Figure 3.48 Normalized load versus Steel Strain for Specimen W2

Figure 3.49 Normalized load versus Steel Strain for Specimen W3
Figure 3.50 Normalized Load versus Steel Strain for Specimen Y2

Figure 3.51 Normalized Load versus Steel Strain for Specimen Y3
Figure 3.52 Cyclic Variation of Depth of Neutral Axis for Specimen W1

Figure 3.53 Cyclic Variation of Depth of Neutral Axis for Specimen W2
Figure 3.54 Cyclic Variation of Depth of Neutral Axis for Specimen W3

Figure 3.55 Cyclic Variation of Depth of Neutral Axis for Specimen Y1
Figure 3.56 Cyclic Variation of Depth of Neutral Axis for Specimen Y2

Figure 3.57 Cyclic Variation of Depth of Neutral Axis for Specimen Y3
Figure 3.58 Normalized Total Moment versus End Rotations for Specimen Y1

Figure 3.59 Normalized Total Moment versus End Rotations for Specimen Y2
Figure 3.60 Normalized Total Moment versus End Rotations for Specimen Y3

Figure 3.61 Normalized Total Moment versus End Rotations for Specimen W3
Figure 3.62 Top Hoop Strains versus Longitudinal Strains for Specimen W1

Figure 3.63 Bottom Hoop Strains versus Longitudinal Strains for Specimen W1
Figure 3.64 Top Hoop Strains versus Longitudinal Strains for Specimen W2

Figure 3.65 Bottom Hoop Strains versus Longitudinal Strains for Specimen W2
Figure 3.66 Top Hoop Strains versus Longitudinal Strains for Specimen W3

Figure 3.67 Bottom Hoop Strains versus Longitudinal Strains for Specimen W3
Figure 3.68 Top Hoop Strains versus Longitudinal Strains for Specimen Y1

Figure 3.69 Bottom Hoop Strains versus Longitudinal Strains for Specimen Y1
Figure 3.70 Top Hoop Strains versus Longitudinal Strains for Specimen Y2

Figure 3.71 Bottom Hoop Strains versus Longitudinal Strains for Specimen Y2
Figure 3.72 Top Hoop Strains versus Longitudinal Strains for Specimen Y3

Figure 3.73 Bottom Hoop Strains versus Longitudinal Strains for Specimen Y3
Figure 3.74 Normalized Energy Dissipation Capacity

Figure 3.75 Ductility Factors for White Specimens
Figure 3.76 Displacement-Based Ductility Factors for Yellow Specimens

Figure 3.77 Pinching Effects Comparison
Figure 3.78 Normalized Load versus Deflection for Secondary Monotonic Test of Specimen Y1

Figure 3.79 Normalized Load versus Deflection for Secondary Monotonic Test of Specimen Y2
Figure 3.80 Normalized Load versus Deflection for Secondary Monotonic Test of Specimen Y3

Figure 3.81 Comparison of Secondary Monotonic Tests for Specimens Y1, Y2 and Y3
Figure 3.82 Comparisons of CFFT with RC Beam-Columns

(a) Comparison Between W1 and RC Specimens

(b) Comparison between Y1 and RC Specimens
Figure 3.83 Typical GFRP and CFRP Specimens

Figure 3.84 Test Setup for Stub Specimens
Figure 3.85  Stub Test Equipment

Figure 3.86  Loading Regime for Stub Tests
Figure 3.87 Typical Failure Mode for a Single Layer Glass Wrap

Figure 3.88 Typical Failure Mode for Two-Layer Glass Wraps

Splitting
Figure 3.89 Typical Failure Mode for a Single Layer Carbon Wrap

Figure 3.90 Typical Failure Mode for a Two-Layer Carbon Wrap
Figure 3.91 Stress-Strain Response for Single Layer Glass Wraps

Figure 3.92 Stress-Strain Response for Two-Layer Glass Wraps
Figure 3.93 Stress-Strain Response for Single Layer Carbon Wraps

Figure 3.94 Stress-Strain Response for Two-Layer Carbon Wraps
Figure 3.95 Typical Unloading Curves before Bend Point

Figure 3.96 Typical Unloading Curves after Bend Point
Figure 3.97  Typical Reloading Curves from a Low Stress Level

Figure 3.98  Typical Reloading Curves from a High Stress Level
Figure 3.99 Axial Stress versus Volumetric Strain for Specimen G22

Figure 3.100 Axial Stress versus Volumetric Strain for Specimen C25
Figure 3.101  Test Setup for Tension Coupons

Figure 3.102  Test Setup for Compression Coupons
Figure 3.103 Typical Failure Modes for White Coupons in Tension

Figure 3.104 Typical Failure Modes for Yellow Coupon in Tension
Figure 3.105 Typical Failure Modes for White Coupons in Compression

Figure 3.106 Typical Failure Modes for Yellow Coupons in Compression
Figure 3.107  Monotonic Tests of White Coupons in Tension

Figure 3.108  Cyclic Test of White Coupon WTC1 in Tension
Figure 3.109  Cyclic Test of White Coupon WTC2 in Tension

Figure 3.110  Cyclic Test of White Coupon WTC3 in Tension
Figure 3.111 Monotonic Tests of White Coupons in Compression

Figure 3.112 Cyclic Test of White Coupon WCC1 in Compression
Figure 3.113 Cyclic Test of White Coupon WCC2 in Compression
Figure 3.114 Monotonic Tests of Yellow Coupons in Tension

Figure 3.115 Cyclic Test of Yellow Coupon YTC1 in Tension
Figure 3.116  Cyclic Test of Yellow Coupon YTC2 in Tension

Figure 3.117  Cyclic Test of Yellow Coupon YTC3 in Tension
Figure 3.118  Monotonic Tests of Yellow Coupons in Compression

Figure 3.119  Cyclic Test of Yellow Coupon YCC1 in Compression
Figure 3.120 Cyclic Test of Yellow Coupon YCC2 in Compression

Figure 3.121 Cyclic Test of Yellow Coupon YCC3 in Compression
Figure 3.122 Reverse Cyclic Test of Yellow Coupon YCTC1

Figure 3.123 Reverse Cyclic Test of Yellow Coupon YCTC2
CHAPTER 4
MODEING OF FRP-CONCRETE BEAM-COLUMNS

4.1 Introduction
Cyclic modeling of FRP-concrete beam-columns with and without internal steel reinforcement requires constitutive models in tension and compression for the confined concrete core, FRP tube, and steel reinforcement. It also requires an appropriate beam-column fiber analysis tool with accurate sectional and longitudinal discretizations of the hybrid system that allow for cycles of loading and unloading. In the following sections, first the constitutive models are described or developed for each component of the system, and then the fiber analysis tool used in this study is presented. It should be noted here that the tensile strength and tension stiffening of concrete have been neglected in this analysis.

4.2 Modeling of FRP-Confined Concrete in Uniaxial Compression

4.2.1 Monotonic Model
Constitutive model for FRP-confined concrete under monotonic compression was developed by Samaan et al. (1998). The model has been compared and verified through several independent studies (Lam and Teng 2002, and De Lorenzis, 2001). Therefore, it was used as the envelope of the cyclic model for FRP-confined concrete.

As shown in Figure 4.1, the model assumes a bilinear stress-strain curve. The initial slope is controlled by the modulus of elasticity of concrete core, as the lateral expansion of concrete is generally limited in this region. The second slope is controlled by the hoop stiffness of the FRP tube, after concrete core has significantly cracked, and the confining
action of the tube has been activated. There is a curved transition zone in between the two regions. The entire stress-strain \((f_c - \varepsilon_c)\) response is defined as

\[
f_c = \frac{\left( E_1 - E_2 \right) \varepsilon_c}{1 + \left( \frac{E_1 - E_2 \varepsilon_c}{f_0} \right)^n} + E_2 \varepsilon_c \tag{4.1}
\]

where \(E_1\) and \(E_2\) are the first and second slopes of the response, respectively, given by

\[
E_1 = 47.586 \sqrt[1000]{f_c} \text{(ksi)} \tag{4.2}
\]

\[
E_2 = 52.411 f_c^{0.2} + 1.3456 \frac{E_j t_j}{f_c D} \tag{4.3}
\]

and \(f_c^{0.2}\) is strength of unconfined concrete at 28 days, \(E_j\) is modulus of elasticity of the FRP tube, \(t_j\) is thickness of the FRP tube, \(D\) is diameter of concrete core, and \(f_0\) is the Y-intercept of the second slope, given by

\[
f_0 = 0.872 f_c^{0.2} + 0.371 f_c + 0.908 \tag{4.4}
\]

and \(f_c\) is the confining pressure exerted by the FRP tube on concrete, as

\[
f_c = \frac{2 f_j t_j}{D} \tag{4.5}
\]

and \(f_j\) is the ultimate hoop strength of the FRP tube, and finally, \(n\) is the curve shape parameter for the transition zone, usually selected as 1.5.

The ultimate strength of FRP-confined concrete is given by

\[
f_{cu} = f_c^{0.5} + 3.38 f_r^{0.7} \tag{4.6}
\]

The ultimate strain of FRP-confined concrete can then be calculated as

\[
\varepsilon_{cu} = \frac{f_{cu} - f_0}{E_2} \tag{4.7}
\]
4.2.2 Cyclic Model

The stub test results of Chapter 3 in addition to test results of Scherer (1996) and Mastrapa (1997) were used to develop a cyclic model for FRP-confined concrete. First, the conditions of unloading from the envelope curve to a zero stress level and reloading from a zero stress level back to the envelope curve are described. Then, the more generalized conditions of unloading and reloading between any two arbitrary stress levels below the envelope curve are presented. The model proposed and verified herein closely follows the general format of that by Mander et al. (1988), which has been widely referenced and compared with for steel-confined concrete.

4.2.2.1 Unloading from Envelope Curve to Zero Stress

A total of 10 sets of test data were compiled for the conditions of full unloading from the envelop curve to a zero stress level (El-Khoury 1999 and Mastrapa 1997). The unloading pattern depends on the stress level \( f_{un} \), from which unloading initiates. Figures 3.95 and 3.96 show two typical cases of unloading from the envelope curve. The first figure shows a linear unloading path from the initial slope of the envelope curve, whereas the second figure shows a curvilinear unloading path from a higher stress level on the second slope of the envelope curve. Furthermore, the unloading path from the initial slope returns to the origin, whereas the unloading path from the second slope terminates at a residual or plastic strain \( \varepsilon_{pl} \). Test results indicate that the higher the unloading stress is, the larger the plastic strain would be. The unloading path is therefore modeled by its secant modulus of elasticity \( E_{secu} \) as

\[
E_{secu} = \frac{f_{un}}{\varepsilon_{un} - \varepsilon_{pl}}
\]  \hspace{1cm} (4.8)
where $\varepsilon_{\text{un}}$ is the unloading strain on the envelope curve. Figure 4.2 shows a schematic illustration of the unloading rule. Regression analysis has shown that $E_{\text{secu}}$ is strongly correlated to the unloading stress from the envelope curve $f_{\text{un}}$. Figure 4.3 shows the normalized unloading modulus $E_{\text{secu}}/E_1$ versus the normalized unloading stress $f_{\text{un}}/f'_{\text{co}}$ for all of the 10 unloading sets in the database. The range of $f_{\text{un}}/f'_{\text{co}}$ in the database is between 1.3 and 2.2. Based on additional observations and understanding of the unloading phenomenon, a tri-linear model is proposed for $E_{\text{secu}}$ with a 70.3% goodness of the fit, as

$$
\frac{E_{\text{secu}}}{E_1} = \begin{cases} 
1, & 0 \leq \frac{f_{\text{un}}}{f'_{\text{co}}} < 1 \\
-0.44 \frac{f_{\text{un}}}{f'_{\text{co}}} + 1.44, & 1 \leq \frac{f_{\text{un}}}{f'_{\text{co}}} < 2.5 \\
0.34, & \frac{f_{\text{un}}}{f'_{\text{co}}} \geq 2.5
\end{cases} \quad (4.9)
$$

Figure 4.4 shows a comparison of the predicted and the experimental values for the unloading slope. A maximum error of $\pm 15\%$ is noted. From Equations 4.8 and 4.9, for each unloading stress on the envelope curve, there is a unique $E_{\text{secu}}$ and a unique plastic strain $\varepsilon_{\text{pl}}$. It should be noted that the plastic strain can be regarded as the intersection of the unloading slope from the envelope curve to the pivot point used by some hysteretic models for unconfined concrete or steel-confined concrete. (Mander et al. 1988, Park et al, 1985). The unloading curve starts from the unloading point ($\varepsilon_{\text{un}}, f_{\text{un}}$) to the point of plastic strain ($\varepsilon_{\text{pl}}, 0$) as

$$
\varepsilon_{\text{pl}} = \varepsilon_{\text{un}} - \frac{f_{\text{un}}}{E_{\text{secu}}}
$$

(4.10)

Figures 4.5 to Figure 4.8 show enlarged plots for four of the unloading paths from the second slope on the envelope curve. The figures clearly show that the tangent slope of the
unloading path at the point of plastic strain is almost zero. Moreover, the unloading paths may be represented by a polynomial curve, such as

\[ f_c = \frac{(1-x)^{n_1}}{(1+kx)^{n_2}} f \text{un} \]  

(4.11)

where \( x \) is the normalized strain on the unloading path, as given by

\[ x = \frac{\varepsilon_c - \varepsilon \text{un}}{\varepsilon \text{pl} - \varepsilon \text{un}}, \quad 0 < x < 1 \]  

(4.12)

and \( n_1 \), \( n_2 \) and \( k \) are shape parameters. Three constraints can be imposed on the model, as:

1. On the envelope curve, \( x = 0 \), \( \varepsilon_c = \varepsilon \text{un} \), and \( f_c = f \text{un} \);

2. At the zero stress level, \( x = 1 \), \( \varepsilon_c = \varepsilon \text{pl} \), and \( f_c = 0 \); and

3. The tangent slope of the unloading path at any point is given by

\[ \frac{\partial f_c}{\partial \varepsilon_c} = \frac{n_1(1-x)^{n_1-1}(1+kx)^{n_2} + n_2(1-x)^{n_2} k(1+kx)^{n_1-1}}{(1+kx)^{2n_2}} E_{\text{Secu}} \]  

(4.13)

and its value on the envelope curve (\( x = 0 \)) is given by

\[ E_u = (n_1 + n_2 k) E_{\text{Secu}} \]  

(4.14)

and at the point of plastic strain (\( x = 1 \)), the slope is 0, as expected.

Figure 4.9 shows the normalized stress-strain data from Figures 4.5 to 4.8. Regression analysis from all data sets found the shape parameters to be all equal to 2, simplifying the unloading path as

\[ f_c = \frac{(1-x)^2}{(1+2x)^2} f \text{un} \]  

(4.15)

As such, the tangent slope of the unloading path on the envelope curve \( E_u \) turns out be six times the secant unloading slope \( E_{\text{Secu}} \).
4.2.2.2 Reloading from Zero Stress to Envelope Curve

Figures 4.10 to 4.13 show enlarged plots for four reloading paths. Although some softening can be seen at the onset of reloading, it is typically modeled as a linear curve. Since the specimen does not return to the same unloading stress $f_{un}$ at the unloading strain $\varepsilon_{un}$, some strength and stiffness degradations are prevalent due to internal cracks. The new stress level $f_{new}$ is less than $f_{un}$.Reloading back to the initial slope on the envelope curve, however, does not constitute such degradation, as concrete cracks are not significant. Figure 4.14 shows a 10% strength degradation for the 22 reloading data sets with a 90% goodness of the fit. As such, the new stress level can be given as

$$f_{new} = 0.9 f_{un} \quad (4.16)$$

The reloading path can therefore be constructed between the point of plastic strain ($\varepsilon_{pl}$, 0) and the new stress level ($\varepsilon_{un}$, $f_{new}$). The reloading path is then extended using the same slope until its return to the envelope curve ($\varepsilon_{re}$, $f_{re}$) as

$$\frac{f_{re}}{\varepsilon_{re}} = \frac{f_{new}}{\varepsilon_{un} - \varepsilon_{pl}} \quad (4.17)$$

where the return point ($\varepsilon_{re}$, $f_{re}$) can be computed in combination with Equation 4.1 for the envelope curve. Figure 4.2 shows the schematic illustration of the reloading rule. Due to the uniqueness of the plastic strain for each unloading point on the envelope curve, the unloading point ($\varepsilon_{un}$, $f_{un}$), the new stress point ($\varepsilon_{un}$, $f_{new}$), and the point of plastic strain ($\varepsilon_{pl}$, 0) are kept constant in the hysteretic response, until a new unloading path initiates from the envelope curve.
### 4.2.2.3 Unloading between Arbitrary Stress Levels

Once $E_{secu}$ and $\varepsilon_{pl}$ are determined for an unloading point, the hysteretic response can be easily generalized. Unloading from an arbitrary point $(\varepsilon'_{un}, f'_{un})$ to an eventual reloading point $(\varepsilon_{ro}, f_{ro})$ follows a path similar to the primary unloading path, as

$$x = \frac{E_c - \varepsilon_{un}'}{\varepsilon_{pl} - \varepsilon_{un}'}$$

(4.18)

$$f_c = \frac{(1 - x)^2}{(1 + 2x)^2} f_{un}'$$

(4.19)

Figure 4.15 shows the schematic illustration of these unloading rules.

### 4.2.2.4 Reloading between Arbitrary Stress Levels

Four different cases may arise during an arbitrary reloading:

1. When the reloading stress $f_{ro}$ and the target stress $f'_{un}$ are both less than the new stress level $f_{new}$, the reloading path will follow a straight line targeting the new stress point $(\varepsilon_{un}, f_{un})$ until it reaches $f'_{un}$.

2. When the reloading stress $f_{ro}$ is lower than the new stress level $f_{new}$, but the target stress $f'_{un}$ is higher than $f_{new}$, the reloading path will first follow a straight line to the new stress point $(\varepsilon_{un}, f_{un})$, and then follows the slope from the point of plastic strain $(\varepsilon_{pl}, 0)$ to the new stress point $(\varepsilon_{un}, f_{new})$ until it reaches the target stress.

3. When the reloading stress $f_{ro}$ is higher than the new stress $f_{new}$, and the reloading strain $\varepsilon_{ro}$ is higher than $\varepsilon_{un}$, the reloading path will follow the slope from the point of plastic strain $(\varepsilon_{pl}, 0)$ to the new stress point $(\varepsilon_{un}, f_{new})$ until it reaches the target stress of $f'_{un}$, where $f'_{un} \leq f_{ro}$.
4. When the reloading stress \( f_{ro} \) is higher than the new stress \( f_{new} \), but the reloading strain \( \varepsilon_{ro} \) is lower than \( \varepsilon_{un} \), the reloading path will follow a straight line to the unloading point \((\varepsilon_{un}, f_{un})\) on the envelope curve until it reaches the target stress level of \( f'_{un} \), where \( f'_{un} \leq f_{un} \). The difference between \( f_{ro} \) and \( f_{un} \) is considered very small, and therefore, the path targets the previous unloading point \((\varepsilon_{un}, f_{un})\) on the envelope curve rather than the return point \((\varepsilon_{re}, f_{re})\).

Figure 4.16 shows the schematic illustration of these reloading rules.

### 4.2.2.5 Validation of the Cyclic Model

In this section, test data from an independent experimental investigation in China (Chen 2001), which was not included in the regression analysis, is used to verify the proposed cyclic model of FRP-confined concrete. In this independent test, four concrete stubs with 6 in diameter and 12 in height were tested under cyclic loading. The 28-day compressive strength of concrete \( f'_{co} \) was 5.74 ksi. Two different FRP jackets with two and three layers of carbon wrap were used. The carbon FRP was FTS-C1-30 from Tonen Corp. with a 0.0065 in thickness per layer, a tensile strength of 515 ksi, a tensile modulus of 34,083 ksi, and an ultimate elongation of 1.5%. An epoxy resin with a tensile strength of 3.6 ksi was used as adhesive. The actual thickness per layer of cured wrap varied between 0.024 and 0.039 in. Figures 4.17 to 4.20 compare the proposed cyclic model (solid lines) with the test data (dashed lines) under different loading patterns. A very good agreement is noted.
4.3 Modeling of FRP Laminates

If the laminate structure and the properties of resin and fibers were known, the engineering properties could be derived from the mixture rule and the classical laminate theory. Alternatively, in this section, coupon tests are used to establish the engineering properties of the laminate for the two types of tubes tested in this study.

4.3.1 Modeling of White Tubes in Monotonic Loading

Monotonic tension tests of white coupons (Figures 3.107) indicated a linear elastic response. This is expected, as over 75% of total fibers are in the longitudinal direction. Minor stiffness degradations in the coupon tests may be attributed to the resin rich layer on the inside of the tube. The tension model may be written as

\[ f_j = 3330 \cdot \varepsilon_j \]  \hspace{1cm} (4.20)

where \( f_j \) and \( \varepsilon_j \) are the stress and strain in the tube, respectively. An ultimate strain \( \varepsilon_u \) of 0.021 and an ultimate stress \( f_u \) of 70 ksi was noted from the coupon tests. Figure 4.21 shows the coupon tests (dotted lines), the proposed model with an 87.5% goodness of the fit, the manufacturer data and the independent test results from Owens Corning Corp.

Monotonic compression tests of white coupons (Figures 3.111) also indicated a linear elastic response. The compression model may be written as

\[ f_j = 2609 \cdot \varepsilon_j \]  \hspace{1cm} (4.21)

with an ultimate strain \( \varepsilon_u \) of 0.023 and an ultimate stress \( f_u \) of 60 ksi. Figure 4.22 shows the coupon tests (dotted lines), the proposed model with a 92.1% goodness of the fit, the manufacturer data and the independent test results from Owens Corning Corp.
4.3.2 Modeling of White Tubes in Cyclic Loading

Cyclic tension tests of three white coupons (Figures 3.108 to 3.110) and the cyclic compression tests of two white coupons (Figures 3.112 to 3.113) indicated little or no energy dissipation nor stiffness degradation in the hysteretic response under tension or compression. Therefore, the unloading and reloading response of white tubes in tension and compression may be regarded as linear, following Equations 4.20 and 4.21, respectively.

4.3.3 Modeling of Yellow Tubes in Monotonic Loading

Monotonic tension tests of yellow coupons (Figures 3.114) showed a clear nonlinear response. This may be attributed to the $\pm 55^\circ$ fiber orientation in the laminate structure of the tube and the viscous damping properties of the resin. As such, axial tensioning of the yellow coupons results in considerable inter-laminar shear loading. Pure shear in laminate structures is expected for $\pm 45^\circ$ angle plies. The response can be accurately modeled using a polynomial as

$$f_j = 3500 \cdot \varepsilon_j \cdot (1 - 114 \cdot \varepsilon_j)$$

with an ultimate strain $\varepsilon_u$ of 0.004386 and an ultimate stress $f_u$ of 7.68 ksi. Figure 4.23 shows the coupon tests (dotted lines), the proposed model with a 93.4% goodness of the fit, the manufacturer data and the independent test results from Owens Corning Corp.

Monotonic compression tests of yellow coupons (Figures 3.118) also showed considerable nonlinearity, again due to its fiber architecture. The response is modeled using a polynomial as

$$f_j = 1000 \cdot \varepsilon_j \cdot (1 - 10.5 \cdot \varepsilon_j)$$

(4.23)
with an ultimate strain $\varepsilon_u$ of 0.04762 and an ultimate stress $f_u$ of 23.81 ksi. Figure 4.24 shows the coupon tests (dotted lines), the proposed model with a 94.3% goodness of the fit, the manufacturer data and the independent test results from Owens Corning Corp.

### 4.3.4 Modeling of Yellow Tubes in Cyclic Loading

From the cyclic tests of three coupons in tension (Figures 3.115 to 3.117) and three others in compression (Figures 3.119 to 3.121), the following observations could be made regarding the hysteretic behavior of yellow tubes:

1. Both the unloading and reloading paths can be described as parabolic curves, similar to the envelope curve;
2. The unloading path appears to return to the origin, with little or no residual or plastic strain;
3. The reloading path from zero stress level follows the envelope curve;
4. No strength or stiffness degradation is apparent in the reloading path;
5. The reloading path from an arbitrary stress level targets the last unloading point, and asymptotes back to the envelope curve with a common tangent; and
6. Tangent slope of the unloading path at the immediate unloading point is twice the secant slope between the unloading point and the origin.

The above observations could be cast into a simple set of unloading and reloading rules. However, several important considerations must be given to the practicality and implications of the derived rules. Firstly, due to the $\pm 55^\circ$ fiber orientation in the yellow tubes and the narrow width of the coupon samples, fibers were not continuous throughout the length of the coupons and did not rupture in either tension or compression tests. As such, the
coupon tests primarily represented the inter-laminar shear and the resin properties. Note that the coupon tests in the present study were based on ASTM D695, whereas the manufacturer data is based on ASTM D2105. The latter test is carried out on the whole segment of a small tube, rather than coupons of a large tube.

Secondly, both tension and compression tests were only concerned with loading from and unloading to the zero stress level. Therefore, the behavior of the tube in stress reversal was not captured in those tests. Limited load reversal tests that were carried out in this study on similar coupons of the yellow tubes had buckling problem at low stress levels in compression. Nevertheless, two typical examples of such coupon tests with limited stress reversal were shown in Figures 3.122 and 3.123. These stress reversal tests showed the unloading path in one direction to target the envelope curve in the other direction, rather than returning to the origin. This phenomenon was also confirmed from the behavior of yellow tubes filled with concrete in the beam-column tests. Return of the unloading paths to the origin would have resulted in an overwhelming pinching effect in the response of the beam-columns. No such pinching, however, was observed in the beam-columns with or without internal steel reinforcement.

Based on the above discussion, it is justified to consider the hysteretic behavior of the yellow tubes, as shown in Figure 4.25. The proposed model is described below for the generalized conditions of unloading and reloading between any two arbitrary stress levels within the envelope curve. The following three rules govern the construct of the unloading and reloading paths between any two arbitrary stress levels:

1. The unloading and reloading paths follow a parabolic shape, as given by

   \[ f_j = k_1^{u/r} \varepsilon_{j}^2 + k_2^{u/r} \varepsilon_{j} + k_3^{u/r} \]  
   \hspace{1cm} (4.24)
where \( k_i^{ur} \) are the coefficients with superscripts \( u \) or \( r \) for unloading or reloading, respectively. Since FRP is used as reinforcement, unloading is considered from tension, whereas reloading is from compression.

2. The unloading and reloading paths target the ultimate stress level in the opposite direction.

3. Tangent slopes of the unloading and reloading paths at the immediate start point is twice that of the secant slope between the start point and the ultimate stress level in the reversed direction.

Coefficients in Equation (4.24) can be found using the above boundary conditions for the unloading and reloading paths as:

\[
k^u_1 = -\frac{f_{uc} - f_{un}}{(\epsilon_{uc} - \epsilon_{un})^2}
\]

\[
k^u_2 = -2 \cdot \frac{f_{uc} - f_{un}}{(\epsilon_{uc} - \epsilon_{un})^2} \cdot \epsilon_{uc}
\]

\[
k^u_3 = f_{uc} - k\gamma \cdot \epsilon_{uc}^2 - k_h \cdot \epsilon_{uc}
\]

\[
k^r_1 = -\frac{f_{ut} - f_{re}}{(\epsilon_{ut} - \epsilon_{re})^2}
\]

\[
k^r_2 = -2 \cdot \frac{f_{ut} - f_{re}}{(\epsilon_{ut} - \epsilon_{re})^2} \cdot \epsilon_{ut}
\]

\[
k^r_3 = f_{ut} - k\gamma \cdot \epsilon_{ut}^2 - k_h \cdot \epsilon_{ut}
\]

where \( f_{ut} \) and \( f_{uc} \) are the ultimate tensile and compressive strengths of the tube, respectively, and \( \epsilon_{ut} \) and \( \epsilon_{uc} \) are the ultimate tensile and compressive strains of the tube, respectively. In the case of yellow tubes, the manufacturer data were found more appropriate, since they followed ASTM D2105. Therefore, the ultimate tensile and compressive strengths of the tube
were taken as 10 and 33 ksi, respectively, and the ultimate tensile and compressive strains were taken as 0.0057 and 0.026, respectively.

4.4 Modeling of FRP-Concrete Beam-Columns

Once the constitutive relations for the FRP-confined concrete and the FRP tube are established, the beam-column behavior of concrete-filled FRP tubes can be studied using fiber strip analysis. Fiber strips represent longitudinal fibers, for which the force-deformation relations of the cross section are obtained by integrating their uniaxial stress-strain models. The following assumptions were used in the fiber strip analysis:

1. Plane sections of the concrete-filled FRP tube remain plane and normal to the neutral axis after bending;
2. Buckling of the tube is neglected, even though slight discrepancy may arise for tubes with low to moderate diameter-to-thickness ratios;
3. Creep and shrinkage of concrete and creep of FRP are neglected;
4. Residual stresses in concrete, FRP, and steel are neglected;
5. Internal Steel reinforcement is placed uniformly around the cross section;
6. Slippage at the interface between FRP and concrete is neglected due to lack of bond-slip data;
7. Tensile strength and tension stiffening of concrete are neglected; and
8. Warping of the section, shear and torsion are neglected.

Figure 4.26 shows the discretized cross section with different fiber strips for the concrete core, FRP tube, and internal steel reinforcement. Cyclic models for the FRP-confined concrete and the FRP tubes are those described earlier in Sections 4.2 and 4.3,
respectively. Cyclic model for the steel reinforcement is taken after Menegotto and Pinto (1973), as modified by Filippou et al. (1983). Details of this model are described in Appendix A.

In this section, the basic characteristics of the fiber strip analysis, which was adopted for this study are described. This model was initially developed and validated by Aval et al. (2002) for concrete-filled steel tubes. Figure 4.27(a) shows a three-node combined element with 13 degrees of freedom (DOF), including five DOFs for each end node and three DOFs for the middle node. The element consists of three components: two beam-column frame elements for the concrete core and the FRP tube, and a distributed bond interface element that represents the relative slippage (Figure 4.27(b)).

The model uses the Kirchhoff beam theory, higher order quartic Hermitian shape functions for transverse displacements, and quadratic Lagrangian shape functions for axial deformations.

Details of the shape functions, element kinematics, governing equations, tangent stiffness matrix, and the Gauss-Lobatto integration technique for the combined element are presented in Appendix B.

In general, transverse displacements for concrete and FRP are assumed to be the same. The slippage between FRP and concrete produces a distributed bond at the interface, based on the difference between elongation rates and curvatures of FRP and concrete (Figure 4.28). Different bond-slip models could be implemented into the analysis for the FRP-concrete interface. However, in this study, only perfect bond between FRP and concrete was considered due to lack of bond-slip data. The tangent stiffness matrix for the combined
element is derived using the virtual work method and by differentiating the internal nodal forces with respect to the nodal displacements.

### 4.4.1 Validation of the Fiber Strip Analysis Tool

The above fiber strip model was implemented in a general purpose nonlinear finite element analysis program (FEAP), which was initially developed by Professor Taylor at the University of California, Berkeley (Taylor 2002). In this section, validity of the model is demonstrated for the six beam-column tests of the present study.

Figure 4.29 shows the schematic model and the boundary conditions for the beam-column tests described in Chapter 3. Due to symmetry, only one half of the beam-column is considered. Symmetry is preserved through the use of fixed-roller support at the mid-span of the specimen. Pin-roller support is used at the end. The end blocks are not directly modeled in the analysis.

Figures 4.30 to 4.35 compare the fiber strip analysis (solid lines) with test data (dashed lines) for Specimens W1, W2, W3, Y1, Y2, and Y3, respectively. Perfect bond is assumed between FRP and concrete in all cases. It is observed that, in general, there is a good agreement between the analytical and experimental results. Further parametric study for different FRP types, steel reinforcement ratios and axial load levels will be carried out in Chapter 5.
Figure 4.1 Monotonic Stress-Strain Envelope for FRP-Confined Concrete (Samaan et al. 1998)

\[
f'_{cu} = f'_c + 3.38f_r^{0.7}
\]

\[
f_c = \frac{(E_1 - E_2)\varepsilon_c}{1 + \left(\frac{(E_1 - E_2)\varepsilon_c}{f_o}\right)^{1/n}} + E_2\varepsilon_c
\]

Figure 4.2 Unloading and Reloading between Envelope Curve and Zero Stress
Figure 4.3  Regression Analysis of Unloading Slopes

$$E_{secu}/E_1 = -0.44 f_u/f'_{co} + 1.44$$

$$R^2 = 70.32\%$$

Figure 4.4  Comparison of Predicted and Experimental Values of Unloading Slope
Figure 4.5 Unloading Curve 1

Figure 4.6 Unloading Curve 2
Figure 4.7  Unloading Curve 3

Figure 4.8  Unloading Curve 4
Figure 4.9 Normalized Unloading Curves

Figure 4.10 Cyclic Curves for G1 Series Stubs
Figure 4.11  Cyclic Curves for G2 Series Stubs

Figure 4.12  Cyclic Curve for C1 Series Stubs
Figure 4.13 Cyclic Curves for C2 Series Stubs

Figure 4.14 Calibration of Strength Degradation
Figure 4.15 Unloading from an Arbitrary Stress Level
Figure 4.16  Reloading from an Arbitrary Stress Level
Figure 4.17 Comparison of Proposed Model with Two-Layer CFRP Stub Unloaded to Zero Stress

Figure 4.18 Comparison of Proposed Model with Two-Layer CFRP Stub Unloaded to Non-Zero Stress
Figure 4.19 Comparison of Proposed Model with Three-Layer CFRP Stub Unloaded to Zero Stress

Figure 4.20 Comparison of Proposed Model with Three-Layer CFRP Stub Unloaded to Zero and Non-Zero Stress
Figure 4.21 Monotonic Tension Model for White Tubes

Figure 4.22 Monotonic Compression Model for White Tubes
Figure 4.23 Monotonic Tension Model for Yellow Tubes

$\sigma_j = -399000 \varepsilon_j^2 + 3500 \varepsilon_j$

$R^2 = 93.35\%$

Figure 4.24 Monotonic Compression Model for Yellow Tubes

$\sigma_j = -10500 \varepsilon_j^2 + 1000 \varepsilon_j$

$R^2 = 94.37\%$
Figure 4.25 Proposed Cyclic Model for Yellow Tubes

Figure 4.26 Fiber Representation of Concrete-Filled FRP Tube
(Aval et al. 2002)
Figure 4.27 Degrees of Freedom for 3-Node Element Based on Kirchhoff Beam Theory (Aval et al. 2002)
Figure 4.28 Kinematics of the Section with Slippage
(Aval et al. 2002)

\[ a = 37 \text{ in}, \quad b = 11 \text{ in (UC)} \]
\[ a = 40 \text{ in}, \quad b = 14 \text{ in (NCSU)} \]

Figure 4.29 Schematic Model for CFFT Beam-Columns

\[ a = 37 \text{ in}, \quad b = 11 \text{ in (UC)} \]
\[ a = 40 \text{ in}, \quad b = 14 \text{ in (NCSU)} \]
Figure 4.30 Comparison of Specimen Y1 with Proposed Model

Figure 4.31 Comparison of Specimen Y2 with Proposed Model
Figure 4.32  Comparison of Specimen Y3 with Proposed Model

Figure 4.33  Comparison of Specimen W1 with Proposed Model
Figure 4.34  Comparison of Specimen W2 with Proposed Model

Figure 4.35  Comparison of Specimen W3 with Proposed Model
CHAPTER 5
PARAMETRIC STUDY

5.1 Parameter Selection

Using the fiber element model described in Chapter 4, a comprehensive parametric study was carried out to better understand the hysteretic behavior of the CFFT beam-columns. In addition, a comparison was made between CFFT beam-columns and their equivalent RC and CFST counterparts.

Figure 5.1 shows the general configuration of the beam-column, for which all parametric studies were carried out. The beam-column is subjected to a constant axial load $P$, and a reverse cyclic transverse load of $Q$ applied in four point bending. The length $L$ of the beam-columns, the moment arm $a$, and the mid-span region $2b$ were chosen to be the same as the specimens tested at the NCSU. All case studies included a span length of 8 ft with a moment arm of 2 ft, a concrete core diameter of 12 in with a 28-day unconfined strength of 4 ksi, and Grade 60 steel reinforcing bars. Due to symmetry, only one half of the span was modeled, as was shown in Figure 4.29. The same displacement control regime as that shown in Figure 3.29 was used for all parametric studies.

The parameters considered in this study included the type of FRP materials, reinforcement ratio $\rho_s$ of the internal steel, diameter-to-thickness $D/t$ ratio of the tube, length-to-diameter $L/D$ ratio of the beam-column, and the axial load ratio $P/P_o$, where $P_o$ is the axial capacity of the section. Table 5.1 summarizes the range of parameters considered for the 44 case studies. The axial capacity of the section was calculated for every combination of the FRP type, tube thickness, and steel reinforcement ratio.
The mechanical properties of FRP tube depend on the fiber orientation and interaction between fibers and the matrix, as discussed in Chapters 2 and 4. Based on the observations of the white and yellow FRP tubes tested in this study, two idealized types of linear and non-linear FRP materials were considered for the parametric study. Figure 5.2 shows the monotonic stress-strain curves of the linear and non-linear FRP, as well as Grade 60 steel. Although shown in tension, it is assumed that the material properties are the same in compression. Tensile and compressive strengths for both types of FRP materials are assumed 50 ksi, to be comparable with Grade 50 steel tubes that will be discussed later. The linear FRP material has a modulus of elasticity of 2,500 ksi and an ultimate strain of 0.02. The non-linear FRP material has a parabolic stress-strain curve with an initial tangent modulus of elasticity of 2,500 ksi and an ultimate strain of 0.04. Hysteretic response of the two types of FRP materials are shown in Figure 5.3. No energy dissipation is considered for the linear FRP materials. On the other hand, the non-linear FRP material follows the same rules of loading and unloading as those described for the yellow tubes in Equations (4.24) to (4.30), except for the difference in their ultimate strengths and strains.

Reinforcement ratio $\rho_s$ of the internal steel is selected as 0% and 2% to evaluate the hysteretic response of CFFT beam-columns without or with internal reinforcement, respectively. The 2% reinforcement ratio is assumed to be a practical limit, particularly when the tube itself has sufficient flexural capacity, as was the case for the white tubes.

Diameter-to-thickness $D/t$ ratio represents slenderness of the tube as well as the confinement effectiveness in concrete core. Three different $D/t$ ratios of 25, 50 and 100 were used in this parametric study, which correspond to equivalent reinforcement ratios of 16%, 8%, and 4% for the tube, respectively. Clearly, the equivalent ratio is the sum of both
longitudinal and transverse reinforcements. The $D/t$ ratio of 50 was selected as the reference value for comparison. Columns with $D/t$ ratios of 25 and 100 in this parametric study are considered to be stocky and very slender, respectively. In concrete-filled steel tubes, the NEHRP (1994) recommends an upper bound limit on the $D/t$ ratio, as follows:

$$\frac{D}{t} < \sqrt[5]{5E_s}$$  \hspace{1cm} (5.1)$$

where $E_s$ and $F_y$ are the elastic modulus and yield strength of the steel tube, respectively. For example, for Grade 50 steel with an elastic modulus of 29,000 ksi, the maximum recommended $D/t$ ratio is 53.9.

Length-to-diameter $L/D$ ratio represents slenderness of the beam-column and the severity of the $P-\Delta$ effects. Three different $L/D$ ratios of 4, 8 and 12 were used in this parametric study. The $L/D$ ratio of 8 was selected as the reference value for comparison.

Axial load ratio $P/P_o$ depends on the service load conditions as well as span lengths and storey heights. Priestley (1996) reported that service load compression forces higher than 15% of the section capacities are uncommon for bridge columns. In buildings, however, a higher ratio may be expected. Three different $P/P_o$ ratios of 10%, 25% and 50% were used in this parametric study. The $P/P_o$ ratio of 10% was selected as the reference value for comparison, primarily to represent bridge pier columns.

5.2 Parameter Assessment

5.2.1 Hysteretic Response

Figures 5.4 and 5.5 show the effect of FRP type on the hysteretic response of CFFT beam-columns for the two cases of 0% and 2% internal steel reinforcement ratios, respectively. In each figure, a table outlines the selected parameters for the FRP type, steel
reinforcement ratio, $D/t$, $L/D$, and $P/P_o$ ratios. The slot for the parameter under investigation is shaded in gray. The effect of FRP type is more pronounced for CFFT members without any internal steel, where linear FRP provides a much higher capacity than the non-linear FRP, but with significantly lower energy dissipation and ductility. For CFFT members with 2% internal steel, the hysteretic curves of the linear and non-linear FRP merge together. However, the non-linear FRP provides much wider and more stable hysteretic loops. While non-linearity of FRP only slightly improves the pinching effects in the absence of internal steel, non-linear FRP with internal steel appears to totally shed its pinching effects.

Figures 5.6 and 5.7 show the effect of internal steel reinforcement on the hysteretic response of CFFT beam-columns for the two cases of linear and non-linear FRP, respectively. The effect of internal steel on linear FRP is not as significant, since it only improves the capacity, and not ductility, energy dissipation or pinching effects of the member. On the other hand, the effect of internal steel is quite obvious for non-linear FRP in terms of energy dissipation, ductility, pinching, as well as capacity.

Figures 5.8 and 5.9 show the effect of $D/t$ ratio on the hysteretic response of CFFT beam-columns for the two cases of linear and non-linear FRP, respectively, both without internal steel. Figures 5.10 and 5.11 show the effect for linear and non-linear FRP, respectively, both with internal steel. The lower $D/t$ ratios lead to higher capacities and slightly better energy dissipation and wider hysteretic loops. The pinching effects, however, do not seem to be affected by the $D/t$ ratio.

Figures 5.12 and 5.13 show the effect of $L/D$ ratio on the hysteretic response of CFFT beam-columns for the two cases of linear and non-linear FRP, respectively, both without internal steel. Figures 5.14 and 5.15 show the effect for linear and non-linear FRP,
respectively, both with internal steel. The lower $L/D$ ratios clearly improve capacity and stiffness of the member, regardless of the FRP type or presence of internal steel. However, pinching and energy dissipation and ductility do not seem to be affected much by the $L/D$ ratios. Also, lower $L/D$ ratios may lead to premature shear failure, hence lower ductility.

Figures 5.16 and 5.17 show the effect of $P/P_o$ ratio on the hysteretic response of CFFT beam-columns for the two cases of linear and non-linear FRP, respectively, both without internal steel. Figures 5.18 and 5.19 show the effect for linear and non-linear FRP, respectively, both with internal steel. The higher sustained axial loads clearly improve the pinching, ductility and energy dissipation of the member, irrespective of the FRP type or presence of internal steel. This may be attributed to the confinement effect due to axial loads. Furthermore, higher axial loads lead to a gradual descending branch in the envelope curve, hence better post-peak performance.

### 5.2.2 Envelope Curve

Figures 5.20-5.31 show the effect of $D/t$, $L/D$, and $P/P_o$ ratios on the envelope curves of CFFT beam-columns for linear and non-linear FRP with and without internal steel. The following observations could be made from these figures:

1. Both capacity and stiffness of the member increase with lower $D/t$ or $L/D$ ratios, whether FRP is linear or non-linear. The increase is more apparent in the absence of internal steel. The $P/P_o$ ratio does not appear to significantly affect the capacity of the member. However, higher $P/P_o$ ratios lead to higher initial stiffness for the member.
2. The maximum deflection of the member is primarily affected by the $L/D$ ratio, since short columns may fail in premature shear.

3. The shape of the envelope curve is mainly affected by the FRP type and presence of internal steel. Non-linear FRP, especially in the presence of internal steel, results in an envelope shape much similar to ductile RC columns. The higher $P/P_o$ ratio results in a clear post-peak response for the member, due to the $P$-$\Delta$ effects.

5.2.3 Cumulative Energy

Figures 5.32-5.43 show the effect of $D/t$, $L/D$, and $P/P_o$ ratios on the cumulative energy of CFFT beam-columns for linear and non-linear FRP with and without internal steel. The following observations could be made from these figures:

1. The cumulative energy dissipation of the member increases with lower $D/t$ or $L/D$ ratios or higher $P/P_o$ ratio, whether FRP is linear or non-linear. The increase is more apparent in the absence of internal steel. The effect of $P/P_o$ ratio may be attributed to the $P$-$\Delta$ effects.

2. The number of cycles to failure is primarily affected by the $L/D$ ratio, as lower ratios may result in premature shear failure.

5.2.4 Ductility

Figure 5.44 shows the effect of $D/t$ ratio on the ductility of the CFFT member with linear or non-linear FRP, and with or without internal steel. Figures 5.45 and 5.46 show similar graphs for the effects of $L/D$ and $P/P_o$ ratios, respectively. In these figures, ductility is displacement-based, as described in Chapter 2. Ductility for members with a post-peak
descending branch is calculated based on the ultimate displacement at 85% of the capacity, as suggested by Park and Paulay (1975). This particularly affects members with high levels of axial load. The following observations could be made from these figures:

1. Ductility of the member is generally higher for non-linear FRP than linear FRP. In general, the presence of internal steel increases the bend point of the curve where the equivalent yielding point for FRP members is defined in Chapter 2.

2. Ductility of the member improves with higher $D/t$ ratios. This may be attributed to the fact that the thinner tube thickness, which is higher $D/t$ ratio, makes the CFFT beam-columns more deformable, even though their capacity may be lower.

3. Ductility of the member improves with lower $L/D$ ratios, as shorter columns have a higher initial stiffness, hence lower yield deflection.

4. The $P/P_o$ ratio does not significantly influence the ductility for the most part.

5.2.5 Pinching

Figure 5.47 shows the effect of $D/t$ ratio on the pinching of the CFFT member with linear or non-linear FRP, and with or without internal steel. Figures 5.48 and 5.49 show similar graphs for the effects of $L/D$ and $P/P_o$ ratios, respectively. In these figures, pinching is measured as described in Chapter 2. The following observations could be made from these figures:

1. Pinching in the response is less for non-linear FRP rather than linear FRP. It appears that the higher deformation capacity of non-linear FRP improve crack closure in concrete.
2. Presence of internal steel significantly reduces the pinching in the response. This again may be attributed to higher deformation capacity offered by steel.

3. Pinching in the response is not significantly affected by $D/t$ ratio for the most part. Similarly, no clear trend can be seen for $L/D$ ratio.

4. The axial load generally reduces the pinching in the response. However, axial loads have no effect on the pinching of members with non-linear FRP and internal steel reinforcement, because their pinching is already quite low.

5.3 Comparisons with RC and CFST Beam-Columns

In this section, the hysteretic behavior of CFFT is compared with the equivalent RC and CFST beam-columns. The same fiber element model described in Chapter 4, in its original format (Aval et al. 2002), can be used to model RC and CFST columns. Applicability of the fiber element model to such members has been validated before (Aval et al. 2002). The cyclic model for the concrete is based on that proposed by Kent and Park (1971). The cyclic model for the steel tube is based on that proposed by Menegotto and Pinto (1973).

Figure 5.50 shows the ultimate states for hollow linear FRP tube, non-linear FRP tube and steel tube under flexure. The steel tube in the CFST beam-column is assumed to be of Grade 50 with an elastic modulus of 29,000 ksi, which is more than 10 times those of linear and non-linear FRP tubes. Due to the yielding properties of steel, the stresses along the whole section can reach the maximum strength at the ultimate state. Considering the circular shape effect of these tubes and their working compatibility with concrete as beam-columns, it is possible to approximately equate the thickness of steel tube to half that of linear or non-linear
FRP tube. In this comparison study, the thickness of steel tube is designated as 0.24 in to meet the NEHRP (1994) requirement. The volumetric steel reinforcement ratio $\rho_v$ is 6.8%. Therefore, the thickness for either linear FRP or non-linear FRP tube is 0.48 in, which is equivalent to a volumetric steel reinforcement ratio $\rho_v$ of 13.9%.

A reinforced concrete beam-column with a longitudinal steel reinforcement ratio of 4% and a volumetric steel reinforcement ratio of 5.5% is also considered for comparison. All other parameters follow the same values as the CFFT beam-columns in previous sections.

Figures 5.51 and 5.52 show the hysteretic curves for the CFFT member with linear and non-linear FRP, respectively. In each figure, the hysteretic curves for the equivalent RC and CFST beam-columns are also shown for comparison. It appears that the CFST has the widest hysteretic loops with highest capacity and ductility. The capacity of linear CFFT is very close to that of CFST. However, the shape of hysteretic curve for linear CFFT is much narrower, and its maximum deflection is quite less than that of CFST. On the other hand, deflection of the non-linear CFFT is much closer to CFST, while its capacity is almost half that of CFST.

The 4% RC beam-column has a lower capacity but wider hysteretic loops and higher deflection than those of the linear CFFT beam-column. On the other hand, it has a similar capacity and ultimate deflection capacity to those of non-linear CFFT beam-columns. However, the hysteretic loops are wider than those of non-linear CFFT, especially at the unloading portions.

The above-mentioned comparisons reveal the fact that the existence of steel in the beam-columns, either internal or external, will improve the seismic performance of the member. The linear CFFT beam-column has a similar load capacity to the CFST with half its
tube thickness. The non-linear CFFT beam-column has a similar capacity to the RC beam-column with 4% steel reinforcement ratio.

Figure 5.53 shows the envelop curves for the four types of members considered in this study. It can be found that the initial slopes for linear CFFT and CFST beam-columns are higher than that of RC specimen and non-linear CFFT. The softening of the member also occurs at a lower load level for RC and non-linear CFFT.

Figure 5.54 shows the cumulative energy dissipation of the four members. CFST has the best performance while the linear CFFT has the worst cumulative energy dissipation capacity.

Figure 5.55 shows the ductility and pinching factors of the four members. The linear CFFT beam-column has the least ductility due to its brittleness. The CFST has the best pinching behavior. The linear CFFT and non-linear CFFT have opposite behaviors in ductility and pinching.
Table 5.1 Matrix of Parametric Study

<table>
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<tr>
<th>FRP Type</th>
<th>$\rho_s$</th>
<th>$D/t$</th>
<th>$L/D$</th>
<th>$P/P_o$</th>
<th>Axial Capacity $P_o$ (kips)</th>
<th>Tube Thickness $t$ (in)</th>
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Figure 5.1 Beam-Column Model for Parametric Study
Figure 5.2  Monotonic Stress-Strain Relations for Linear FRP, Non-Linear FRP, and Grade 60 Steel

Figure 5.3  Hysteretic Stress-Strain Relations for the Linear FRP and Non-Linear FRP
Figure 5.4 Effect of FRP Type without Internal Steel

Figure 5.5 Effect of FRP Type with Internal Steel
Figure 5.6  Effect of Internal Steel Reinforcement for Members with Linear FRP

Figure 5.7  Effect of Internal Steel Reinforcement for Members with Non-Linear FRP
Figure 5.8 Effect of D/t Ratio for Linear FRP without Internal Steel

Figure 5.9 Effect of D/t Ratio for Non-Linear FRP without Internal Steel
Figure 5.10  Effect of D/t Ratio for Linear FRP with Internal Steel

Figure 5.11  Effect of D/t Ratio for Non-Linear FRP with Internal Steel
Figure 5.12 Effect of L/D Ratio for Linear FRP without Internal Steel

Figure 5.13 Effect of L/D Ratio for Non-Linear FRP without Internal Steel
Figure 5.14 Effect of L/D Ratio for Linear FRP with Internal Steel

Figure 5.15 Effect of L/D Ratio for Non-Linear FRP with Internal Steel
Figure 5.16  Effect of P/P₀ Ratio for Linear FRP without Internal Steel

Figure 5.17  Effect of P/P₀ Ratio for Non-Linear FRP without Internal Steel
Figure 5.18  Effect of P/P₀ Ratio for Linear FRP with Internal Steel

Figure 5.19  Effect of P/P₀ Ratio for Non-Linear FRP with Internal Steel
Figure 5.20  Effect of D/t Ratio on the Envelope Curves for Linear FRP without Internal Steel

Figure 5.21  Effect of D/t Ratio on the Envelope Curves for Non-Linear FRP without Internal Steel
Figure 5.22  Effect of D/t Ratio on the Envelope Curves for Linear FRP with Internal Steel

Figure 5.23  Effect of D/t Ratio on the Envelope Curves for Non-Linear FRP with Internal Steel
Figure 5.24 Effect of L/D Ratio on the Envelope Curves for Linear FRP without Internal Steel

Figure 5.25 Effect of L/D Ratio on the Envelope Curves for Non-Linear FRP without Internal Steel
Figure 5.26 Effect of L/D Ratio on the Envelope Curves for Linear FRP with Internal Steel

Figure 5.27 Effect of L/D Ratio on the Envelope Curves for Non-Linear FRP with Internal Steel
Figure 5.28 Effect of P/P_0 Ratio on the Envelope Curves for Linear FRP without Internal Steel

Figure 5.29 Effect of P/P_0 Ratio on the Envelope Curves for Non-Linear FRP without Internal Steel
Figure 5.30  Effect of P/P₀ Ratio on the Envelope Curves for Linear FRP with Internal Steel

Figure 5.31  Effect of P/P₀ Ratio on the Envelope Curves for Non-Linear FRP with Internal Steel
Figure 5.32 Effect of D/t Ratio on Cumulative Energy for Linear FRP without Internal Steel

Figure 5.33 Effect of D/t Ratio on Cumulative Energy for Non-Linear FRP without Internal Steel
Figure 5.34 Effect of D/t Ratio on Cumulative Energy for Linear FRP with Internal Steel

Figure 5.35 Effect of D/t Ratio on Cumulative Energy for Non-Linear FRP with Internal Steel
Figure 5.36 Effect of L/D Ratio on Cumulative Energy for Linear FRP without Internal Steel

Figure 5.37 Effect of L/D Ratio on Cumulative Energy for Non-Linear FRP without Internal Steel
Figure 5.38 Effect of L/D Ratio on Cumulative Energy for Linear FRP with Internal Steel

Figure 5.39 Effect of L/D Ratio on Cumulative Energy for Non-Linear FRP with Internal Steel
Figure 5.40 Effect of P/P_o Ratio on Cumulative Energy for Linear FRP without Internal Steel

Figure 5.41 Effect of P/P_o Ratio on Cumulative Energy for Non-Linear FRP without Internal Steel
Figure 5.42 Effect of $P/P_0$ Ratio on Cumulative Energy for Linear FRP with Internal Steel

Figure 5.43 Effect of $P/P_0$ Ratio on Cumulative Energy for Non-Linear FRP with Internal Steel
Figure 5.44 Effect of D/t Ratio on Ductility

Figure 5.45 Effect of L/D Ratio on Ductility
Figure 5.46 Effect of \( P/P_o \) Ratio on Ductility

Figure 5.47 Effect of \( D/t \) Ratio on Pinching
Figure 5.48 Effect of L/D Ratio on Pinching

Figure 5.49 Effect of P/Po Ratio on Pinching
Figure 5.50  Sectional Equivalency of Linear FRP, Non-Linear FRP and Steel Tubes
Figure 5.51  Hysteretic Curves of Linear CFFT, CFST and RC Beam-Columns

Figure 5.52  Hysteretic Curves of Non-Linear CFFT, CFST and RC Beam-Columns
Figure 5.53  Envelope Curves of Linear CFFT, Non-linear CFFT, CFST and RC Beam-Columns

Figure 5.54  Cumulative Energy Dissipation of Linear CFFT, Non-linear CFFT, CFST and RC Beam-Columns
Figure 5.55 Ductility and Pinching Behavior of Linear CFFT, Non-Linear CFFT, CFST and RC Beam-Columns
6.1 Summary

Use of concrete-filled fiber-reinforced polymers (FRP) tubes (CFFT) for columns and piles has been studied extensively over the last decade. The focus, however, has been exclusively on their monotonic behavior. An issue that has received little attention is the implications of using CFFT in seismic regions. Survey of damaged structures in recent earthquakes indicates that catastrophic failure of an entire structure may result from failure of few columns in a chain action. Since it may not be economical to design columns to respond to earthquake loads in their elastic range, dissipation of energy by post-elastic deformation is desired. On the other hand, FRP materials are known for their linear elastic behavior. Some FRP materials may exhibit non-linearity due to their laminate architecture and inter-laminar shear. Also, confinement of concrete core in CFFT improves its ductility. This study was carried out to evaluate the behavior of CFFT beam-columns, and determine whether non-linearity of FRP or confinement of concrete can provide seismic performance comparable to reinforced concrete (RC) columns or concrete-filled steel tubes (CFST).

The research reported in this dissertation consisted of both experimental and analytical components. The experimental component comprised of the following:

- Twenty-four short stubs of FRP-wrapped concrete under cycles of loading and unloading in uniaxial compression;
- Twenty-five coupons from the two types of FRP tubes under cycles of loading and unloading in uniaxial tension or compression; and
• Six large-scale CFFT beam-columns under a constant axial load and reverse cycles of transverse loading in four-point flexure.

The analytical component comprised of the following:
• Modeling cyclic behavior of FRP-confined concrete in compression;
• Modeling cyclic behavior of linear and non-linear FRP materials in tension and compression;
• Employing a fiber element model to predict the cyclic behavior of CFFT beam-columns;
• Carrying out a parametric study on the cyclic response of CFFT beam-columns, and evaluating their hysteretic performance measures including cumulative energy dissipation, ductility and pinching effect; and
• Comparison of the hysteretic response of CFFT beam-columns with those of reinforced concrete and concrete-filled steel tubes.

6.2 Conclusions

This study has resulted in a number of conclusions, as outlined below:
• Cyclic response of the FRP-confined concrete depends on the level of unloading in comparison to the bend point in the envelope curve. Unloading prior to the bend point is characterized as linear elastic, whereas unloading after the bend point is represented by a parabolic curve with a distinct plastic or residual strain.
• FRP tubes with fiber architecture, such as the $\pm 55^\circ$ in the yellow tubes tested for this research, exhibit significant non-linearity with considerable energy dissipation.
• The two types of CFFT beam-columns tested under this study represented two different failure modes; a brittle compression failure for the over-reinforced white tube specimens with thick FRP tube and with majority of the fibers in the longitudinal direction, and a ductile tension failure for the under-reinforced yellow tube specimens with thin FRP tubes and off-axis fibers.

• A moderate amount of internal steel reinforcement in the range of 1%-2% may improve the cyclic response of CFFT members. The improvement is more significant for the under-reinforced FRP tubes. Adding internal steel, especially for members with thick FRP tubes, can be ineffective and may result in premature failure.

• Both displacement-based ductility and energy-based ductility measured could be used to assess the hysteretic response of CFFT members. A minimum ductility factor in the range of 4-5 is recommended for CFFT members.

• The pinching effect in CFFT members depends mainly on the amount of internal steel reinforcement, rather than the type of FRP tube.

• Hysteretic performance measures of CFFT members, including energy dissipation capacity and pinching effect, are greatly enhanced at higher levels of axial load, primarily due to the confinement effects.

• Slender CFFT members have less capacity than their short stocky counterparts. However, they are less susceptible to pinching effect and premature shear failure.

• It is feasible to design CFFT beam-columns with moderate amount of internal steel reinforcement to have comparable hysteretic performance to that of RC members.
Hysteretic response of CFFT members, irrespective of the type of FRP tube or the amount of internal steel reinforcement, may not measure up to their CFST counterparts. However, selection of CFFT members may lie in their superior durability and lower weight.

6.3 Recommendations for Further Research

Through the study undertaken for this dissertation, a couple of research needs were identified that could be explored in future studies. An issue that has not been addressed is the combined effect of steel hoops and FRP jacket on the confinement of concrete core. Many investigators either ignore the effect of steel hoops for its conservative nature, or simply add their effect to that of the FRP jacket. It is clear, however, that the combined effect is not a mere summation of the two confining devices.

Another issue of importance is the off-axis non-linearity of FRP materials. In this study, the laminate response was input into the fiber element model. However, a more rigorous modeling is required to directly incorporate the laminate theory into the fiber element model. The model can then be equipped with appropriate failure criteria for the FRP composites. In this study, failure was imposed based on the engineering properties of the FRP tube.
REFERENCES


APPENDIX A

CYCLIC STRESS-STRAIN RELATION FOR STEEL

The reinforcing steel stress-strain behavior is described by the nonlinear model of Menegotto and Pinto (1973), as modified by Filippou et al. (1983) to include isotropic strain hardening. The model is computationally efficient and agrees very well with experimental results from cyclic tests on reinforcing steel bars.

The model, as presented in Menegotto and Pinto (1973) takes on the form

\[
\sigma^* = b \cdot \varepsilon^* + \frac{(1 + b) \cdot \varepsilon^*}{(1 + \varepsilon^{*R})^{1/R}}
\]  

(A.1)

where

\[
\varepsilon^* = \frac{\varepsilon - \varepsilon_r}{\varepsilon_0 - \varepsilon_r}
\]  

(A.2)

and

\[
\sigma^* = \frac{\sigma - \sigma_r}{\sigma_0 - \sigma_r}
\]

(A.3)

Equation A.1 represents a curved transition from a straight line asymptote with slope \(E_o\) to another asymptote with slope \(E_1\) (lines (a) and (b), respectively, in Figure A.1). \(\sigma_0\) and \(\varepsilon_0\) are the stress and strain at the point where the two asymptotes of the branch under consideration meet (Point B in Figure A.1); Similarly, \(\sigma_r\) and \(\varepsilon_r\) are the stress and strain where the last strain reversal with stress of equal sign took place (Point A in Figure A.1); \(b\) is the strain hardening ratio, that is the ratio between slope \(E_1\) and \(E_o\). \(R\) is a parameter that influences the shape of transition curve and allows a good representation of the Bauschinger effect. As indicated in Figure A.1, \((\varepsilon_0, \sigma_0)\)and \((\varepsilon_r, \sigma_r)\) are updated after each strain reversal.
$R$ is considered dependent on the strain difference between the current asymptote intersection point (Point A in Figure A.2) and the previous load reversal point with maximum or minimum strain depending on whether the corresponding steel stress is positive or negative (Point B in Figure A.2). The expression for $R$ takes the form suggested in Menegotto and Pinto (1973)

$$R = R_0 - \frac{a_1 \cdot \xi}{a_2 + \xi}$$ (A.4)

where $\xi$ is updated following a strain reversal. $R_o$ is the value of the parameter $R$ during first loading and $a_1$, $a_2$ are experimentally determined parameters to be defined together with $R_o$. The definition of $\xi$ remains valid in case that reloading occurs after partial unloading.

Some clarification is needed in connection with the set of rules for unloading and reloading which complement Equations (A.2) and (A.3), allowing for a generalized load history. If the analytical model had a memory extending over all previous branches of the stress-strain history, it would follow the previous reloading branch, as soon as the new reloading curve reached it. This would require that the model store all necessary information to retrace all previous reloading curves which were left incomplete. This is clearly impractical from a computational standpoint. Memory of the past stress-strain history is, therefore, limited to a predefined number of controlling curves, which in the present model include,

1. The monotonic envelope,
2. The ascending upper branch curve originating at the reversal point with smallest $\varepsilon$ value,
3. The descending lower branch curve originating at the reversal point with largest $\varepsilon$ value,
4. The current curve originating at the most recent reversal point.

Due to the above restrictions reloading after partial unloading does not rejoin the original reloading curve after reaching the point from which unloading started, but, instead, continues on the new reloading curve until reaching the envelope. However, the discrepancy between the analytical model and the actual behavior is typically very small, as discussed in detail by Filippou et al. (1983).

The above implementation of the model corresponds to its simplest form, as proposed in Menegotto and Pinto (1973): elastic and yield asymptotes are assumed to be straight lines, the position of the limiting asymptotes corresponding to the yield surface is assumed to be fixed at all times and the slope $E_o$ remains constant (Figure A.1).

In spite of the simplicity in formulation, the model is capable of reproducing well experimental results. Its major drawback stems from its failure to allow for isotropic hardening. To account for this effect Filippou et al. (1983) proposed a stress shift in the linear yield asymptote as a function of the maximum plastic strain as follows:

$$\frac{\sigma_{st}}{\sigma_y} = a_1 \left( \frac{\varepsilon_{max}}{\varepsilon_y} - a_4 \right)$$

(A.5)

where $\varepsilon_{max}$ is the absolute maximum strain at the instant of strain reversal, $\varepsilon_y$, $\sigma_y$ are, respectively, the strain and stress at yield, and $a_3$ and $a_4$ are experimentally determined parameters. The model used in this study was implemented without the isotropic strain hardening option. For this case the parameter values are: $R_o = 20$, $a_1 = 18.5$, $a_2 = 0.15$, $a_3 = 0$, $a_4 = 0$. With the exception of the last two parameters the values used are those in the original model of Menegotto and Pinto (1973).
Figure A.1 Menegotto-Pinto Steel Cyclic Model (Menegotto and Pinto 1973, Taucer et al. 1991)

Figure A.2 Definition of Curvature Parameter R in Menegotto-Pinto Steel Cyclic Model (Menegotto and Pinto 1973, Taucer et al. 1991)
APPENDIX B
FIBER STRIP ANALYSIS*

A CFFT column consists of two components, i.e., a concrete core and an FRP shell around the concrete. The stress transfer between the two components may be increased by the use of inner ribs or a studded shell that can augment the composite action in CFFTs. A practical frame element to consider the bond effect on nonlinear behavior of CFFTs under monotonic as well as cyclic loads was developed to be used in a general purpose finite-element program. It is assumed that a distributed bond is maintained between FRP shell and concrete core. There are two causes for the slippage between FRP shell and concrete core. The first is the difference between elongation of the FRP shell and the concrete core, and the second is the difference between curvatures in the cross section for the concrete core and the FRP shell. This element is planar; however, extension to a three-dimensional case is straightforward. Furthermore, the formulation can readily be extended to include the torsional degree of freedom (DOF) to model linear elastic behavior under twist.

The proposed comprehensive composite element consists of three components: two frame elements, one to model the concrete core and another to represent the FRP shell, and a distributed bond interface element that represents the relative slippage (Figure 4.40).

A 13-DOF combined element that has five DOFs per end node and three DOFs on the middle node has been chosen. This model has been verified in the steel-confined concrete tubes (Aval et al., 2002). Nonlinear behavior of materials on the cross-sectional level leads to a nonlinear relationship between the bending moment and curvature. Because of the complex deformed shape, the conventional cubic polynomial function, which causes a linear relation

* Adopted from Aval et al. (2002).
between moment and curvature, cannot represent the curvature distribution in the element. This problem may be solved by three strategies, i.e., the use of more elements for modeling a member, the use of higher order shape function, or utilizing a flexibility method. For practical proposes, it is better to model a member with only a single element. The flexibility method recently developed by Spacone et al. (1996) requires much effort and is not as straightforward as the displacement-based element to compute the resisting forces in the general-purpose finite element program. Therefore, to increase accuracy it was decided to use higher order quartic Hermitian shape function for transverse displacement. The quadratic Lagrangian shape functions for axial deformation are used. Collection of these types of shape functions for transverse and axial deformation leads to cubic order variation of bond at the interface. The explicit forms of these functions for both FRP and concrete element are:

**Shape Functions**

\[
N_u^T = \left\langle 1 - 3\xi + 2\xi^2, -\xi + 2\xi^2, 4\xi - 4\xi^2 \right\rangle \tag{B.1}
\]

\[
N_w^T = \left\langle 1 - 11\xi^2 + 18\xi^3 - 8\xi^4, \left(\xi - \xi^2 + 5\xi^3 - 2\xi^4\right)L^2, -5\xi^2 + 14\xi^3 - 8\xi^4, \left(\xi^2 - 3\xi^3 + 2\xi^4\right)L^2, 16\xi^2 - 32\xi^3 + 16\xi^4 \right\rangle \tag{B.2}
\]

**Nodal Displacement and Force Vectors**

\[
q^T = \left\langle q_u^c, q_u^f, q_w^c, q_w^f \right\rangle \tag{B.3}
\]

\[
Q^T = \left\langle Q_u^c, Q_u^f, Q_w^c, Q_w^f \right\rangle \tag{B.4}
\]

where superscripts \(c\) and \(f\) represent the concrete core and the FRP tube, respectively, and

\[
q_u^c = \left\langle u_1^c, u_2^c, u_3^c \right\rangle; \quad q_w^c = \left\langle w_1^c, \theta_1^c, w_2^c, \theta_2^c, w_3^c \right\rangle \tag{B.5}
\]
\[ Q_u^c = \{ U_1^c, U_2^c, U_3^c \}; \quad Q_w^c = \{ W_1^c, W_2^c, W_3^c \}; \]  \hspace{1cm} (B.6)

Similar vectors can be shown for the FRP tube.

**Element Kinematics**

The element deformations consist of three components: the displacements of concrete and FRP components and the interaction between them, which produces compatibility in displacements. The transverse displacement perpendicular to the axis is the same for concrete and FRP components but slippage between the two components produces a distributed bond at the interface. The displacement of two adjacent points in the concrete core and the FRP tube are calculated based on the axial and flexural deformation of their axis, as

\[ u_f = \bar{u}_f - z \cdot \theta_f; \quad u_c = \bar{u}_c - z \cdot \theta_c \]  \hspace{1cm} (B.7)

where \( \bar{u}_c \) and \( \theta \) are the axial and rotational deformation of axis, and subscripts \( c \) and \( f \) represent concrete and FRP, respectively. Therefore, the slippage is given by

\[ u_b = \bar{u}_c - \bar{u}_f - z(\theta_c - \theta_f) \]  \hspace{1cm} (B.8)

**Governing Equations**

In order to include the bond effect, in the virtual work equation the slippage should be integrated around the perimeter and added to the general expression of virtual work from concrete and FRP components. The virtual displacement work equation can be expressed as

\[ \delta V = \int_{\Omega} D^T \delta d \delta x + \int_{\Omega} \mathbf{f}^T s(x, z) \delta u_b \delta x - Q_u^c \delta q = 0 \]  \hspace{1cm} (B.9)

where the bond force denoted by \( s(x, z) \) is distributed along the interface and varies with deformation history, and \( D \) and \( d \) are generalized stress and strain, respectively, given by

234
where the prime symbols indicate the derivative with respect to $x$, and $N(x)$ and $M(x)$ are the sectional axial force and bending moment, respectively, and $u(x)$ and $w(x)$ are the displacement fields for the beam reference axis in the axial and transverse directions, respectively. The derivative of displacement field can be evaluated as

$$
\frac{du(x)}{dx} = B_u q_u,
\frac{dw(x)}{dx} = B_w q_w;
\chi = -\frac{d^2w(x)}{dx} = B_x q_w \quad (B.11)
$$

where $A_w$ is a symmetric matrix. The internal nodal forces are given as

$$
Q_{ui} = \int N^e(x)B_u\beta^Tdx + \int f_1(x)N_u^Tdx \quad (B.13)
$$

$$
Q_{ui}^t = \int N^e(x)B_u\beta^Tdx - \int f_1(x)N_u^Tdx \quad (B.14)
$$

$$
Q_{wi} = \int N^e(x)A_u q_u\beta^Tdx + \int M^e(x)B_w^Tdx - \int f_2(x)B_w^Tdx \quad (B.15)
$$

$$
Q_{wi}^t = \int N^e(x)A_u q_u\beta^Tdx + \int M^e(x)B_w^Tdx + \int f_2(x)B_w^Tdx \quad (B.16)
$$

where $f_1(x)$ and $f_2(x)$ are defined as

$$
\begin{align*}
  f_1(x) &= \int s(x,z)\beta d\rho \\
  f_2(x) &= \int zs(x,z)\beta d\rho
\end{align*}
\quad (B.17)
$$

where $d\rho$ is a differential length of the curved segment at the interface.

**Tangent Stiffness Matrix**

The tangent stiffness matrix is obtained from differentiation of internal nodal force with respect to nodal displacement
\[ K_i = \frac{\partial Q_i}{\partial q} \]  

Therefore the tangent stiffness matrix has the following form:

\[
K_i = \begin{bmatrix}
K_{u_c} u \_c & K_{u_f} u \_f & K_{u_c} w \_c & K_{u_f} w \_f \\
K_{u_f} u \_f & K_{u_f} w \_f & K_{u_f} w \_f & K_{u_f} w \_f \\
K_{w_c} w \_c & K_{w_f} w \_f & K_{w_f} w \_f & K_{w_f} w \_f \\
\text{SYM.} & K_{w_f} w \_f & & &
\end{bmatrix}
\]  

\[ K_{u_c} u \_c = \int EA^i B_u^T B_u dx + \int E_s N_u^T N_u dx \]  

(B.19a)

\[ K_{u_f} u \_f = -\int E_f N_u^T N_u dx \]  

(B.19b)

\[ K_{u_c} w \_c = \int EA^i B_u^T q_u^T A_u dx - \int EX_f N_u^T B_u dx + \int EX_f B_u^T B_u dx \]  

(B.19c)

\[ K_{u_f} u \_f = \int EX_f N_u^T B_u dx \]  

(B.19d)

\[ K_{u_f} u \_f = \int EA^i B_u^T B_u dx + \int E_f N_u^T N_u dx \]  

(B.19e)

\[ K_{u_c} w \_f = \int EX_f N_u^T B_u dx \]  

(B.19f)

\[ K_{u_f} w \_f = \int EA^i B_u^T q_u^T A_u dx - \int EX_f N_u^T B_u dx + \int EX_f B_u^T B_u dx \]  

(B.19g)

\[ K_{w_c} w \_c = \int EA^i A_u q_u^T q_u^T A_u dx + \int N^T (x) A_u dx + \int EX^i B_u^T B_u dx + \int EX^i q_u^T A_u dx \]  

\[ \int EI_f B_u^T B_u dx + \int EX^i B_u^T q_u^T A_u dx + \int EX^i A_u q_u^T B_u dx \]  

(B.19h)

\[ K_{w_c} w \_f = -\int EI^i B_u^T B_u dx \]  

(B.19i)

\[ K_{w_f} w \_f = \int EA^i A_u q_u^T q_u^T A_u dx + \int N^T (x) A_u dx + \int EI^i B_u^T B_u dx + \int EI^i A_u q_u^T B_u dx \]  

\[ \int EI_f B_u^T B_u dx + \int EX^i B_u^T q_u^T A_u dx + \int EX^i A_u q_u^T B_u dx \]  

(B.19j)

where \( EA, EX, \) and \( EI \) are the axial, cross coupling, and flexural stiffness of the section, respectively, with superscripts \( c \) and \( f \) for concrete and FRP, respectively, as given by
where $E_t$ is the tangent modulus for concrete or FRP based on the respective nonlinear stress-strain curve during the loading history. The bond also varies during deformation history, and is a function of relative displacement of concrete core and FRP shell. The relation between bond and slippage is described as

$$\delta s(x, z) = E_b \delta u_b$$  (B.21)

where $E_b$ is the tangent modulus of the bond-slip relation. The values of $E_f$, $EX_f$ and $EI_f$ are evaluated at the interface of the concrete core and the FRP shell as

$$E_f = \int E_f d\rho ; \quad \overline{EX_f} = \int z E_f d\rho ; \quad \overline{EI_f} = \int z^2 E_f d\rho$$  (B.22)

$$\overline{EA} = \sum_{i=1}^{n_{fiber}} E_{i} \cdot A_i ; \quad \overline{EX} = \sum_{i=1}^{n_{fiber}} E_{i} \cdot A_i \cdot Z_i ; \quad \overline{EI} = \sum_{i=1}^{n_{fiber}} E_{i} \cdot A_i \cdot Z_i^2$$  (B.23)