Characterization of steel fiber and/or polymer concrete mixes and applications to slender rectangular and I-beams

Ashraf Ibrahim Ahmed

University of Nevada, Las Vegas
INFORMATION TO USERS

This manuscript has been reproduced from the microfilm master. UMI films the text directly from the original or copy submitted. Thus, some thesis and dissertation copies are in typewriter face, while others may be from any type of computer printer.

The quality of this reproduction is dependent upon the quality of the copy submitted. Broken or indistinct print, colored or poor quality illustrations and photographs, print bleedthrough, substandard margins, and improper alignment can adversely affect reproduction.

In the unlikely event that the author did not send UMI a complete manuscript and there are missing pages, these will be noted. Also, if unauthorized copyright material had to be removed, a note will indicate the deletion.

Oversize materials (e.g., maps, drawings, charts) are reproduced by sectioning the original, beginning at the upper left-hand corner and continuing from left to right in equal sections with small overlaps.

Photographs included in the original manuscript have been reproduced xerographically in this copy. Higher quality 6" x 9" black and white photographic prints are available for any photographs or illustrations appearing in this copy for an additional charge. Contact UMI directly to order.

ProQuest Information and Learning
300 North Zeeb Road, Ann Arbor, MI 48106-1346 USA
800-521-0600

UMI®
CHARACTERIZATION OF STEEL FIBER AND/OR POLYMER CONCRETE MIXES
AND APPLICATIONS TO SLENDER RECTANGULAR AND I- BEAMS

by

Ashraf Ahmed

Bachelor of Science in Civil Engineering
Cairo University, Egypt
1986

Master of Science in Civil Engineering
New Mexico State University, Las Cruces, NM
1995

A dissertation submitted in partial fulfillment
of the requirements for

Doctor of Philosophy Degree in Civil Engineering

Department of Civil and Environmental Engineering
Howard R. Hughes College of Engineering

Graduate College
University of Nevada, Las Vegas
December 2001
The Dissertation prepared by

Ashraf I. Ahmed

Entitled

Characterization of Steel Fiber and/or Polymer Concrete Mixes and
Applications to Slender Rectangular and I-Beams

is approved in partial fulfillment of the requirements for the degree of

Doctor of Philosophy

Examination Committee Chair

Dean of the Graduate College

Examination Committee Member

Examination Committee Member

Graduate College Faculty Representative
ABSTRACT

Characterization of Steel Fiber and/or Polymer Concrete Mixes and Applications to Slender Rectangular and I-Beams

by

Ashraf Ibrahim Ahmed

Dr. Samaan G. Ladkany; P.E., Examination Committee Chair
Professor of Civil Engineering
University of Nevada, Las Vegas

This dissertation presents results from experimental studies related to polymer modified concrete, steel fiber reinforced concrete, and steel fiber/polymer modified concrete. As a first stage of this research, the properties of different concrete mixes were characterized. These mixes were: plain concrete, steel fiber concrete with fiber volume fraction of 1%, polymer modified concrete with 1% to 7.5% solids of polymer, and steel fiber/polymer modified concrete with 1% to 7.5% polymer solids and 1% steel fiber fraction. Concrete cylinders and 4 x 4 inches beams were tested under compressive, tensile, flexural, and bar pull-out loadings.

In the second phase of this research, slender beams with a depth to width ratio of three were tested under four point loading for shear and flexure. Half I-beams, with gross aspect ratio of four and web aspect ratio of three were tested under the combined loading of bending, shear, and torsion. Lateral eccentric loads were applied transversely in the
shallow direction to the 3 x 9 inches beams and the half I-beams. Dog bone shaped reinforced and un-reinforced specimens with 3 x3 inches square sections were tested under pure torsional loading. The addition of 1% steel fibers alone or with 5% solids of polymers to concrete mixes improved their toughness and ductility. The contribution of steel fibers to bending, shear, and torsion in slender and half I-beams is presented. The ACI code methods for calculating the torsional, shear, and flexural resistance of beams are compared to the experimental results. Post crack analysis performed on the slender beams and half I-beams indicated that the tested specimens could carry 70% of the maximum applied loads after initial concrete cracking and failure. The reduction in the tensile stresses of stirrups and longitudinal reinforcing bars, due to the steel fibers and polymer, are presented. Fibers and polymers increase bending and toughness in concrete.
# TABLE OF CONTENTS

ABSTRACT ............................................................................................................................... iii

LIST OF FIGURES ...................................................................................................................... xi

LIST OF TABLES .......................................................................................................................... xiii

ACKNOWLEDGMENTS .............................................................................................................. xv

CHAPTER 1  INTRODUCTION AND SCOPE OF RESEARCH .................................................
1.1 Ratios and Non-Dimensional Constants ............................................................................. 4

CHAPTER 2 LITERATURE REVIEW FOR STEEL FIBER REINFORCED CONCRETE AND LATEX MODIFIED CONCRETE .................................................................
2.1 Steel Fiber Reinforced Concrete ....................................................................................... 5
2.1.1 Introduction .................................................................................................................. 5
2.1.2 Pseudo Elastic Behavior of Steel Fiber Reinforced Concrete ......................................... 6
   2.1.2.1 Spacing Mechanism ......................................................................................... 6
   2.1.2.2 Composite Material Mechanism ...................................................................... 7
2.1.3 Fiber Orientation Distribution ..................................................................................... 8
2.1.4 Ultimate Strength and Toughness ................................................................................ 10
2.2 Polymer Modified Concrete (PMC) ................................................................................ 14
   2.2.1 Polymer Latexes (Polymer dispersion) .................................................................... 14
   2.2.2 Polymerization-Hydration Process ........................................................................ 15
   2.2.3 Application of Polymer Modified Concrete (PMC) ................................................ 17
2.2.4 Mix proportions ......................................................................................................... 17
2.3 Fiber/Polymer Modified Concrete ..................................................................................... 19

CHAPTER 3 MATERIAL CHARACTERIZATION OF PLAIN, FIBER, LATEX, AND FIBER/latex CONCRETE .................................................................................................
3.1 Introduction ....................................................................................................................... 24
3.2 Concrete mix proportions ................................................................................................. 25
   3.2.1 Plain Concrete .......................................................................................................... 25
   3.2.2 Steel Fiber Reinforced Concrete (SFRC) ................................................................ 26
   3.2.3 Polymer Modified Concrete (PMC) ......................................................................... 26

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.
3.2.4 Steel Fiber/Polymer Modified Concrete (SFPMC) ........................................................................... 28

3.3 Concrete materials ............................................................................................................................. 29
3.3.1 Coarse aggregate ......................................................................................................................... 29
3.3.2 Fine aggregate ............................................................................................................................ 29
3.3.3 Cement ........................................................................................................................................ 29
3.3.4 Steel fiber .................................................................................................................................... 29
3.3.5 Superplasticizer .......................................................................................................................... 29
3.3.6 Polymer ....................................................................................................................................... 30

3.4 Quality control .................................................................................................................................... 32
3.4.1 Curing of concrete ....................................................................................................................... 32
3.4.2 Concrete workability and superplasticizer .................................................................................. 35
3.4.3 Coarse aggregate gradation ....................................................................................................... 37
3.4.4 Fine aggregate gradation ............................................................................................................. 38

3.5 Concrete mixing and specimens preparation .................................................................................... 38
3.5.1 Mixing procedure ......................................................................................................................... 38
3.5.2 Slump test .................................................................................................................................. 40
3.5.3 Cylinders preparation ................................................................................................................. 40
3.5.4 Curing method ............................................................................................................................. 40

3.6 Specimens testing ............................................................................................................................... 40
3.6.1 Compression test ......................................................................................................................... 41
3.6.2 Splitting test ............................................................................................................................... 41
3.6.3 Flexural strength test ................................................................................................................. 41
3.6.4 Pull-out test ............................................................................................................................... 42

3.7 Results and discussions ..................................................................................................................... 42
3.7.1 Compressive, tensile, and flexural strengths for trial mix #1 ........................................................... 42
3.7.2 Stress-strain relationship for trial mix #1 ..................................................................................... 45
3.7.2.1 Compressive stress-strain behavior ...................................................................................... 45
3.7.2.2 Tensile stress-strain behavior ............................................................................................ 45
3.7.2.3 Flexural stress-strain behavior ............................................................................................ 46
3.7.3 Trail mixes #2 .............................................................................................................................. 47
3.7.4 Compressive, tensile, and flexure strengths behavior for trial mix #2 ........................................... 48
3.7.5 Stress-Strain Relationship for trial mix #2 .................................................................................. 49
3.7.5.1 Compressive stress-strain behavior ...................................................................................... 49
3.7.5.2 Tensile stress-strain behavior ............................................................................................ 49
3.7.5.3 Flexural stress-strain behavior ............................................................................................ 49
3.7.6 Toughness and resilience .............................................................................................................. 50
3.7.7 Post crack analysis ........................................................................................................................ 51
3.7.7.1 Post crack analysis for compressive strength ...................................................................... 51
3.7.7.2 Post crack analysis for tensile (splitting) strength ................................................................ 51
3.7.7.3 Post crack analysis for flexural strength .............................................................................. 52

3.8 Shear strength of short deep beams ................................................................................................. 54
CHAPTER 4

BEHAVIOR OF CONCRETE SLENDER BEAMS IN SHEAR

4.1 Introduction .................................................................................................................. 79
  4.1.1 Scope of research ................................................................................................. 79
  4.1.2 Materials .............................................................................................................. 79

4.2 Beam Classifications .................................................................................................. 80
  4.2.1 Deep beams ......................................................................................................... 80
  4.2.2 Short beams ........................................................................................................ 80
  4.2.3 Intermediate beams ............................................................................................ 81
  4.2.4 Long beams ........................................................................................................ 81

4.3 Literature Survey .................................................................................................... 81
  4.3.1 Steel fiber reinforced concrete ........................................................................ 81
  4.3.2 Polymer concrete ............................................................................................... 88

4.4 Slender Beams ........................................................................................................ 90
  4.4.1 Slender beams without shear reinforcement (stirrups) and tension reinforcement ......................................................................................... 90
  4.4.2 Slender beams with shear reinforcement (stirrups) and single tension reinforcement ........................................................................................................ 89
  4.4.3 Slender beams with shear (stirrups) reinforcement and double tension and compression reinforcement ............................................................... 91

4.5 Strain gages configuration ..................................................................................... 91

4.6 Testing methodology ............................................................................................... 92

4.7 Shear Capacity of Short Slender Concrete Beams Without Transverse (Stirrups) Reinforcements ................................................................. 92

4.8 Diagonal Tensile and Diagonal Compressive Strains for Short Slender Beams with Longitudinal and Transverse (Stirrups) Reinforcements .......................................................................................................................... 93

4.9 Tension in Stirrups and Longitudinal Reinforcement of Short Slender Concrete Beams .......................................................................................................................... 96

4.10 Contribution of Stirrups to Beam Shear ............................................................... 99

4.11 Conclusions ............................................................................................................ 101

CHAPTER 5

I-GREDERS UNDER COMBINED BENDING, SHEAR AND TORSION ......................... 115

5.1 Introduction ............................................................................................................. 115
  5.1.1 Literature review ............................................................................................... 115
5.2 Design of I-Girder Dimensions ..............................................................116
5.3 I-Girder Reinforcement Configurations and Strain Gage Layout ..............................................................117
5.4 I-Girder Concrete Casting ........................................................................118
5.5 Test Methodology ....................................................................................118
5.6 Behavior of Diagonal Shear in Concrete I-Beams .............................119
5.7 Behavior of Transverse Reinforcement (Stirrups) in Concrete I-Beams .....................................................................................................121
5.8 Tension in Longitudinal Reinforcement of Concrete I-Beams ................123
5.9 Post Crack Initiation studies of I-beams ...............................................127
5.10 Torion Behavior Theories .......................................................................130
5.10.1 Skew bending theory ......................................................................130
5.10.2 Space truss analogy theory ...............................................................130
5.11 Combined Loading Interaction ................................................................131
5.11.1 Torsion-bending interaction ............................................................131
5.11.2 Shear-Torsion interaction ..................................................................132
5.12 Conclusions ...............................................................................................134

CHAPTER 6 PRE AND POST CRACK STUDY OF THIN AND I-GIRDER BEAMS UNDER LATERAL LOADS ...............................................152
6.1 Introduction ................................................................................................152
6.2 Thin and Wide Rectangular Beams Under Bending and Torsion Due to Lateral Eccentric Loads ............................................................152
6.3 Thin Rectangular Beams Under Two Point Loads After Crack Initiation ....................................................................................................152
6.4 I-Beams Under Torsion Due To Lateral Loads ....................................153
6.5 Torsion of Rectangular Shallow and Wide Beams due to Eccentric Lateral Loads ............................................................154
6.6 Tensile Strains in Stirrups for Sallow and Wide Beams ......................157
6.7 Tensile Strains in Longitudinal Reinforcement for shallow and Wide Beams .............................................................................................158
6.8 Concrete Tensile and Compressive Strains of Thin Beams Under Vertical Two Point Loading After Crack Due to Lateral Loading ....................................................................................................159
6.9 Longitudinal and Transverse (Stirrups) Tension Strains of Thin Beams Under Two Point Loading After Crack Due to Lateral Loading .............................................................................................163
6.10 Behavior of Torsional Shear For I-Beams Due to Lateral Loads ..............................................................................................................................165
6.11 Stirrups Behavior of Concrete I-Beams ................................................166
6.12 Longitudinal Reinforcement Behavior of Concrete I-Beams .............167
6.13 Transformed Cracked Section Analysis ..............................................168
6.14 Steel Fibers and Polymers Contribution ..............................................169
6.15 Combined Loading Interaction ............................................................... 169
6.16 Conclusions ................................................................................................ 169

CHAPTER 7  
7.1 Introduction ................................................................................................ 200
7.1.1 Literature review ........................................................................ 200
7.1.2 Justification for the pure torsion experiments ....................... 203
7.2 Pure Torsion Test Specimens ................................................................. 203
7.2.1 Beams without torsion reinforcement ...................................... 203
7.2.2 Specimens with torsion reinforcement .................................... 204
7.2.3 Strain gages ................................................................................. 204
7.2.4 Preparation of the beams .......................................................... 205
7.3 Test Methodology ..................................................................................... 205
7.4 Pure Torsional Behavior of Polymer Modified Concrete ........... 205
7.4.1 Polymer modified concrete specimens .................................... 206
7.4.2 Steel fiber reinforced concrete specimens .............................. 207
7.4.3 Steel fiber/polymer modified concrete specimens ................. 208
7.5 Torsional Behavior of Un-Reinforced and Reinforced Specimens ................................................................................................ 210
7.5.1 Torsional toughness of dog-bone specimens ......................... 210
7.5.2 Torsional ductility ....................................................................... 210
7.5.3 Torsional behavior of un-reinforced specimens .................... 210
7.5.4 Torsional behavior of reinforced specimens ......................... 211
7.6 Diagonal Strain Measurements Due to Torsion in Concrete .............. 213
7.7 Reinforcement Behavior .......................................................................... 217
7.7.1 Longitudinal reinforcement behavior ...................................... 217
7.7.2 Transverse reinforcement (stirrups) behavior ......................... 218
7.8 Comparison Between Theoretical and Experimental Results 219
7.9 Conclusions ................................................................................................ 222

CHAPTER 8  
8.1 Summary .................................................................................................... 241
8.2 Conclusions................................................................................................ 241
8.3 Recommendations ..................................................................................... 247
8.4 Suggestions for Future Research ........................................................... 247

BIBLIOGRAPHY ...................................................................................................................... 248

APPENDIX A  
A.1 Mix Proportions for One Cubic Yard Batch of concrete ............... 254
A.2 Mix Adjustment ........................................................................................ 256
A.3 New Mix Proportions for One Cubic Yard Batch of Concrete ........ 258
APPENDIX B  EQUIPMEN USED IN THE EXPERIMENTAL PROCEDURE .........................................................................................260
B.1  Description of MTS Testing System ............................................. 260
B.2  Description of Soiltest Digital Compression Tester ...................... 261
B.3  Description of Tinius Olsen Universal Testing Machine .......... 261
B.4  National Instrument Data Acquisition System ............................ 262

APPENDIX C  STRAIN GAGE INSTALLATION ......................................................266
C.1  Surface Preparation .................................................................................266
C.1.1  Solvent degreasing ..................................................................266
C.1.2  Dry abrasion .............................................................................267
C.1.3  Wet abrasion ............................................................................267
C.1.4  Gage location layout lines ......................................................267
C.1.5  Surface conditioning ...............................................................267
C.1.6  Neutralizing ..............................................................................268
C.2  Gage Installation Procedure ...................................................................268
C.2.1  Strain gage and solder terminal positioning ..........................269
C.2.2  Strain gage alignment ..............................................................269
C.2.3  Strain gage bonding ................................................................269
C.2.4  Adhesive Preparation ..............................................................269
C.2.4.1  Preparing AE-10 adhesive ...................................269
C.2.4.2  Strain gage curing .................................................270

VITA ..........................................................................................................................................271
LIST OF FIGURES

Figure 2.1 Process of latex formation (Lavelle, 1988) ..........................................................22
Figure 2.2 Simplified model of formation of polymer-cement co-matrix (Ohama, 1998) ..........................................................23
Figure 3.1 UT wavy steel fibers ........................................................................62
Figure 3.2 Styrene Butadiene Rubber (Modifier A), Dow Chemical Company ........................................................................62
Figure 3.3 Nevada Ready Mix (NRM) and Nevada Department of Transportation (NDOT) coarse aggregate gradation .......................63
Figure 3.4 Coarse aggregate sizes ........................................................................63
Figure 3.5 Nevada Ready Mix (NRM) and Nevada Department of Transportation (NDOT) fine aggregate gradation ..............................64
Figure 3.6 Two cubic foot concrete mixer ........................................................................64
Figure 3.7 Compression test and strain gages configuration ...................................................65
Figure 3.8 Splitting test and strain gages configuration ........................................................66
Figure 3.9 Flexure test and strain gages configuration ..........................................................67
Figure 3.10 S-shape steel beam for Tinius Olsen machine calibration ..................................68
Figure 3.11 Tinius Olsen machine calibration chart ...............................................................68
Figure 3.12 Pull-Out test and strain gages configuration ..........................................................69
Figure 3.13 Rebar set-up for pull-out specimen .................................................................70
Figure 3.14 Pull-out specimen concrete pouring .................................................................70
Figure 3.15 Pull-out test using MTS machine ........................................................................71
Figure 3.16 Micro-structural crack development at the interface (Michigan State University, 1999) ..........................................................71
Figures 3.17-3.25 Figures Corresponding to section 3.7 in this dissertation ........................72
Figures 3.26-3.28 Figures Corresponding to section 3.8 in this dissertation ........................76
Figures 3.29-3.30 Figures Corresponding to section 3.9 in this dissertation ........................78
Figure 4.1 Slender shear beams without transverse shear reinforcement (stirrups) ..........................................................103
Figure 4.2 Slender shear beams with transverse shear reinforcement (stirrups) and single tension reinforcement (2#4) ........................104
Figure 4.3 Shear specimen with transverse shear reinforcement (stirrups) and double reinforcement (4#4) ..........................................................105
Figure 4.4 Strain gages configuration on steel reinforcement and on concrete for slender beams ........................................................................106
Figure 4.5 Slender beam under four point loading ................................................................107
Figures 4.6-4.9 Figures Corresponding to section 4.7 in this dissertation ................................107
| Figures 4.10-4.14 | Figures Corresponding to section 4.8 in this dissertation | 109 |
| Figures 4.15-4.20 | Figures Corresponding to section 4.9 in this dissertation | 112 |
| Figure 5.1      | AASHTO geometrical dimensions of standard bridge | 136 |
| Figure 5.2      | $I$-girder reinforcement configurations | 136 |
| Figure 5.3      | $I$-girder strain gages configuration | 137 |
| Figure 5.4      | $I$-beam wood forms | 138 |
| Figure 5.5      | Electrical one inch diameter probe concrete vibrator | 138 |
| Figure 5.6      | Test set up with fixed ends and lateral supports | 139 |
| Figure 5.7      | Confining of half $I$-beams flange with steel plates | 139 |
| Figure 5.8      | Inclined web crack for polymer modified concrete half $I$-beams | 140 |
| Figures 5.9-5.14 | Figures Corresponding to section 5.6 in this dissertation | 140 |
| Figures 5.15-5.18 | Figures Corresponding to section 5.7 in this dissertation | 143 |
| Figures 5.19-5.26 | Figures Corresponding to section 5.8 in this dissertation | 145 |
| Figures 5.27-5.29 | Figures Corresponding to section 5.9 in this dissertation | 149 |
| Figures 5.30-5.31 | Figures Corresponding to section 5.11 in this dissertation | 151 |
| Figure 6.1      | Test loading configuration for thin rectangular beams | 173 |
| Figure 6.2      | Test set up for rectangular thin beams under torsion | 174 |
| Figures 6.3-6.17 | Figures Corresponding to section 6.5 in this dissertation | 174 |
| Figures 6.18-6.21 | Figures Corresponding to section 6.6 in this dissertation | 182 |
| Figures 6.22-6.25 | Figures Corresponding to section 6.7 in this dissertation | 184 |
| Figures 6.26-6.30 | Figures Corresponding to section 6.8 in this dissertation | 186 |
| Figures 6.31-6.35 | Figures Corresponding to section 6.9 in this dissertation | 188 |
| Figures 6.36-6.42 | Figures Corresponding to section 6.10 in this dissertation | 191 |
| Figures 6.43-6.46 | Figures Corresponding to section 6.11 in this dissertation | 194 |
| Figures 6.47-6.50 | Figures Corresponding to section 6.11 in this dissertation | 196 |
| Figures 6.51-6.52 | Figures Corresponding to section 6.15 in this dissertation | 198 |
| Figure 7.1      | Un-reinforced dog bone specimen | 225 |
| Figure 7.2      | Reinforced dog bone specimen | 225 |
| Figure 7.3      | Strain gages configuration for the reinforcing cage of dog bone specimen | 226 |
| Figure 7.4      | Dog bone beams form | 226 |
| Figure 7.5      | Dog bone beam before and after removing the Styrofoam | 227 |
| Figure 7.6      | Pure torsion test set-up using the MTS machine | 227 |
| Figure 7.7      | Twisted beam due to pure torsional loading | 228 |
| Figures 7.8-7.20 | Figures Corresponding to section 7.4 in this dissertation | 228 |
| Figures 7.21-7.24 | Figures Corresponding to section 7.5 in this dissertation | 235 |
| Figures 7.25-7.29 | Figures Corresponding to section 7.6 in this dissertation | 237 |
| Figures 7.30-7.31 | Figures Corresponding to section 7.7 in this dissertation | 239 |

Figures B.1-B.5  Figures Corresponding to appendix B in this Dissertation | 263
# LIST OF TABLES

| Table 3.1  | 304 RIBTEC-GR Steel fiber properties .................................................. 30 |
| Table 3.2  | Styrene Butadiene Rubber (SBR) content classification ............................ 31 |
| Table 3.3  | Compressive and Tensile strength for trial concrete mix .......................... 46 |
| Table 3.4  | Compressive, Tensile, and Flexural Strengths for trial concrete mix #2 .... 48 |
| Table 3.5  | Resilience and toughness for trial mix #1 (NDOT mix) in inch-pound/cubic inch 52 |
| Table 3.6  | Resilience and toughness for trial mix #2 in inch-pound/cubic inch ........... 53 |
| Table 3.7  | Ratio between compression strength after cracking to ultimate compression strength ........................................ 53 |
| Table 3.8  | Ratio between tension (splitting) strength after cracking to ultimate tension strength ........................................ 54 |
| Table 3.9  | Ratio between modulus of rupture after cracking to ultimate modulus of rupture ........................................ 55 |
| Table 3.10 | Shear strength capacities for trial mix #1 ........................................... 56 |
| Table 3.11 | Shear strength capacities for trial mix #2 ........................................... 57 |
| Table 3.12 | Ultimate load and bond stress for different concrete mixes ....................... 60 |
| Table 3.13 | Compression and tension strengths for PMC, SFRC, and SFPMC for trial mix #3 ........................................ 61 |
| Table 4.1  | Shear strengths for PMC, SFRC, and SFPMC beams without Stirrups........... 92 |
| Table 4.2  | Calculated strains from cracked and un-cracked models analysis versus measured strain in the longitudinal reinforcements 98 |
| Table 4.3  | Shear capacity of stirrups at maximum measured strains for PMC, SFRC, and SFPMC slender beams ......................... 100 |
| Table 4.4  | Shear capacity of stirrups at maximum predicted shear for PMC, SFRC, and SFPMC slender beams ................................. 100 |
| Table 5.1  | The difference between measured torsion and calculated torsion from ACI code ........................................ 126 |
| Table 5.2  | Calculated strains from cracked models analysis using \( n = \frac{E_s}{E_c} \) or \( 2n = \frac{2E_s}{E_c} \) versus measured strain in the compression longitudinal reinforcements 128 |
| Table 6.1  | Calculated strains from cracked models analysis using \( n = \frac{E_s}{E_c} \) versus measured strain in the longitudinal reinforcements 170 |
Table 6.2  Contribution of steel fibers in carrying forces in transverse (stirrups) and longitudinal reinforcements for all beams with different loading configurations .......................................................171

Table 7.1  Dimensions of tested beams .............................................................206

Table 7.2  Beam toughness up to the ultimate torque level (inch-pound)....211

Table 7.3  Ultimate angle of twist ($\theta_{ult}$) of PMC, SFRC, and SFPMC beams under torsional loading .........................................................212

Table 7.4  Contribution of longitudinal reinforcements, Stirrups, and steel fibers in carrying tension force with respect to PMC beams........218

Table 7.5  Experimentally measured shear modulus, G, (psi) for PMC, SFRC, and SFPMC beams..................................................219

Table 7.6  Experimental and code cracked and ultimate torque capacities (inch-pound) .................................................................223
ACKNOWLEDGEMENTS

First of all, I thank Allah (GOD) almighty who gave me the power, the knowledge, and the patience to accomplish this study.

I would like to express my deep gratitude and appreciation to my advisor and committee chair Dr. Samaan G. Ladkany for his support, guidance, and encouragement throughout my program of study and research. I thank the Nevada Department of Transportation, the Graduate College and the Civil Engineering Department for providing me with research and teaching assistantships during my Ph.D. program. Also, I appreciate the cooperation of DOW Chemical Company for donating the Styrene Butadiene Rubber polymer.

I am grateful to Dr. Brendan O'Toole for his interest in my research and his valuable help and advice during the experimental phase of this research. I am thankful to the contribution of my committee members Drs. James Cardle, William Culbreth, Gerald Frederick, and Moses Karakouzian, for their comments and advice.

I would like to thank Mr. Allen Sampson for his assistance in the workshop and help during the various stages of my research. I appreciate the help of my colleague Mohammed Hassan during the experimental stage.

I am grateful to my parents for their constant love and encouragement.

Last but not least, I would like to thank my wife Nadia Rashed for her constant encouragement, understanding, and patience throughout the course of my Ph.D. program.
CHAPTER 1

INTRODUCTION AND SCOPE OF RESEARCH

Polymer modified concrete has been widely used as overlay for bridge decks to reduce cracks which could be developed due to shrinkage. Steel fiber reinforced concrete was used to arrest crack propagation and provide more ductility and toughness to concrete up to the ultimate failure level. An experimental study of the mechanical properties of polymer modified concrete, steel fiber reinforced concrete, and steel fiber/polymer modified concrete structures has been undertaken. This research involves extensive experimental testing of different concrete specimens utilizing various concrete mixes. A literature review of steel fiber reinforced concrete and polymer modified concrete is presented in chapter 2. A literature review indicates that no significant work is reported on the combination of steel fiber and polymer concrete.

The objectives of this study are introducing a material which can also be used in structural members, describing the behavior of half I-beams under combined loading of shear, bending and torsion since research was urgently needed as requested by Precast/Prestressed Concrete Institute (PCI). The shear behavior of slender beams is investigated as well as describing the contribution of reinforcements. The behavior of slender beams under lateral loading simulating earthquake loading is also reported. The design and testing of several concrete mixes including steel fiber, polymer, steel fiber and polymer together to the ultimate failure stresses are discussed in chapter 3.
Characterization of materials under compression, tension, flexure, and pull-out loading were performed. The behavior of slender beams which have an aspect ratio (depth/width) of three in a shear failure mode is discussed in chapter 4. Steel fiber and polymer enhanced ductility to the beams. Also, the contribution of longitudinal and transverse reinforcements is presented. The length of the 4 x 4 inches beams, the slender 3 x 9 inches beams, and the half I-beams was set to 34 inch due to the practical considerations of laboratory space and physical limitations of the MTS and Tinius Olsen Universal Testing Machines (TOUTM). It was also determined that predominantly shear failure modes are desired to assess the contribution of polymer concrete, steel fiber concrete and steel fiber/polymer concrete to the beam strength. A potential reduction of shear reinforcement (stirrups) in the beams would lead to substantial saving in the cost of reinforcing steel and labor.

Half I-beams with aspect ratios corresponding to the AASHTO bridge specifications are tested under the application of the combined loadings of bending, shear, and torsion. The shear and torsion behavior of these beams is discussed in chapter 5. The experimental data are compared to theoretical bending-torsion and shear-torsion interaction models available in the literature (Wang and Salmon, 1998).

A simulation of the effect of lateral loading due to earthquake on the behavior of both slender beams and half I-beams under the combined loading of shear and torsion is presented in chapter 6.

The behavior of polymer modified concrete, steel fiber reinforced concrete, and steel fiber/polymer modified concrete beams under pure torsion is presented in chapter 7. Steel fiber increased shear strength due to torsional loading as well as providing ductility
and toughness. Steel fiber/polymer modified concrete provided enhanced elasticity and
ductility to the pure torsion specimen.

1.1 Ratios and Non-Dimensional Constants

The following non-dimensional constants and ratios were used throughout this
dissertation.

\[ \rho_s = \frac{A_s}{bd} = \text{Tensile reinforcement ratio for concrete beams.} \]

\[ \rho_p = \frac{A_p}{bd} = \text{Compressive reinforcement ratio for concrete beams.} \]

\[ V_f = \frac{\text{Volume of fiber}}{\text{Volume of concrete}} = \text{Fiber volume fraction.} \]

\[ \frac{l_f}{d_f} = \frac{\text{Fiber length}}{\text{Fiber diameter}} = \text{Fiber aspect ratio} \]

\[ \frac{\text{Weight of polymer}}{\text{Weight of cement}} = \text{Polymer percentage.} \]

\[ \frac{\text{Weight of water}}{\text{Weight of cement}} = \text{Water-cement ratio.} \]

\[ \frac{\text{Weight of coarse aggregate}}{\text{Weight of fine aggregate}} = \text{Coarse aggregate-fine aggregate ratio.} \]

\[ \frac{\text{Weight of superplasticizer}}{100 \text{ pound of cement}} = \text{Percent of superplasticizer.} \]

\[ n = \frac{E_s}{E_c} = \text{Modular ratio.} \]

\[ \frac{a}{d} = \text{Shear span to depth ratio.} \]
\[
\frac{L}{d} = \frac{\text{Beam span}}{\text{Beam depth}} = \text{Beam aspect ratio (length)}.
\]
\[
\frac{d}{b} = \frac{\text{Beam depth}}{\text{Beam width}} = \text{Beam aspect ratio (depth)}.
\]
CHAPTER 2

LITERATURE REVIEW FOR STEEL FIBER REINFORCED CONCRETE AND LATEX MODIFIED CONCRETE

In this chapter a review of steel fiber reinforced concrete (SFRC) and polymer modified concrete (PMC) is given. Literature review of other topics such as shear, bending, and torsion loading combination on slender and half I-beams are presented in following chapters.

2.1 Steel Fiber Reinforced Concrete

2.1.1 Introduction

When added to concrete mixes, steel fibers distribute randomly through the mix at much closer spacing than conventional reinforcing steel. Depending on their aspect ratio (fiber length/fiber diameter), fibers act to arrest cracking by decreasing the stress intensity at the tip of internal cracks. Steel fibers may improve the ultimate tensile strength of concrete because much energy is absorbed in de-bonding and pulling out of fibers from the matrix before complete separation and failure of concrete occurs (Hwan Oh, 1992). Ultimate tensile and shear strengths depend on volume percentages of fibers.
in the concrete mix, however the compressive and tensile strengths may decrease compared to those of plain concrete. Fibers also increase the shear-friction strength of concrete. Steel fibers have been used as web reinforcement to resist shear, arrest crack propagation, and maintaining post micro crack integrity of the surrounding concrete mix (ACI 544-IR-82).

In this research steel fiber reinforced concrete and latex modified concrete have been characterized with respect to their shear, compressive and tensile strengths. In some selected mixes; steel bar bond strength, and shear torsional strength were also studied. The following literature review concentrates only on those aspects of steel fiber concrete and latex modified concrete mentioned above.

2.1.2 Pseudo elastic behavior of steel fiber reinforced concrete

Two mechanisms were proposed to predict the first crack strength. One mechanism relates the first crack strength to the spacing between fibers, while the other mechanism relates it to the fiber volume, orientation, and aspect ratio of the fiber. The fiber spacing mechanism is based on crack arrest by the fiber, while the other mechanism is based on the laws of the composite materials (ACI 544.1R-86).

2.1.2.1 Spacing mechanism

To increase the tensile strength of concrete inherent with internal structural flaws, Romualdi and Baston (1963) tried to decrease the stress intensity at the tip of an internal crack using closely spaced wires to arrest cracking. They proposed that the fiber spacing
could be estimated as:

\[ S = 13.8d \frac{1}{\sqrt{p}} \quad (2.1) \]

where \( S \)=fiber spacing; \( d \)=fiber diameter; and \( p \)=volume percentage of fibers. Another equation for fiber spacing derived by McKee (1969) is:

\[ S = \frac{\sqrt{V}}{\sqrt{p}} \quad (2.2) \]

Where \( p \)=volume percentage of fibers and \( V \)=volume of one fiber

**2.1.2.2 Composite material mechanism**

In plain concrete, a major crack in the tensile zone which may lead to immediate failure of the beam is preceded by micro cracking growth. For fiber reinforced concrete, the behavior is linear up to the proportional limit of first crack strength. Increasing the quantity of fibers increases the load at which the load-deflection curve deviates from linearity. This is due to the high value of Young’s Modulus of fibers, compared to that of plain concrete.

The strength after cracking \( \sigma_u \) is governed by the ultimate pullout bond strength \( \tau_u \) of the fiber (Lim et. al. 1987).

\[ \sigma_u = \eta_1 \eta_2 V_d \frac{\tau_u}{2r} \quad (2.3) \]

where:
\( r \) = the ratio of the fiber cross-sectional area to its perimeter.

\( \eta_o' \) = the orientation factor at cracking. (Lim et. al. 1987).

### 2.1.3 Fiber orientation distribution

An important function of steel fibers in concrete is to arrest and deflect micro cracks developing in concrete under external load effect. The equation commonly given in the literature (Soroushian and Lee, 1990) to predict the number of fibers per unit cross-sectional area of concrete is of the form:

\[
N_f = \eta_o \frac{V_f}{A_f}
\]  \hspace{1cm} (2.4)

where:

\( N_f \) = Number of fibers per unit area

\( A_f \) = Cross-sectional area of steel fibers = \( \pi \frac{d_f^2}{4} \)

\( d_f \) = Fiber diameter

\( \eta_o \) = Orientation factor (0.41-0.82)

When uniformly dispersed in an infinitely large volume of concrete, steel fibers are expected to be randomly oriented with equal probabilities in different directions in space.

Soroushian and Lee (1990) developed theoretical expression for the number of fibers per unit cross-sectional area. Orientation of steel fibers in concrete and the number of fibers per unit area are influenced by the boundaries restricting the random orientation of
fibers and by the fact that steel fibers tend to settle down and reorient in horizontal planes when fibrous concrete is vibrated during placement. The vibration of concrete re-orients steel fiber from 3-D condition to approach 2-D condition. The type of steel fiber and the location in the cross section with respect to the casting direction did not have any statistically significant effect on the measured value of number of fibers per unit area.

Leung and Shapiro (1999) investigated the fiber pullout yield strength for different fibers at different inclination angle (0°, 30°, and 60°). The pullout loads for 0° fibers is less than those of 30° and 60° fibers. To pull out the fiber for the 0° case, energy is required to overcome bonding and friction of the interface. For 30° and 60° fibers, an additional energy is required for the fiber to bend and deform.

Mansur et. al. (1999) studied the deformation and ductility of high strength fiber concrete (HSC) with compressive strength ranging from 10000 psi to 17000 psi. The concrete mixes consist of ordinary Portland cement and silica fume with ratio of 9:1 by weight, the coarse aggregate used was crushed granite and fine aggregate was natural sand. The coarse and fine aggregates were washed and stored properly to achieve a saturated surface dry condition. Different mixes were prepared with water/cement ratios of 0.4, 0.35, 0.3, 0.25, and 0.2 with different steel fiber volume fractions of 0.5% and 1%. The effect of fiber volume fraction (Vf) of steel fibers, shape, and casting direction of specimens on strength, deformation, and ductility was investigated. Cylinders and prisms, used to study the effect of the specimen shape, are circular and square in cross section with same aspect ratio (length/least dimension of cross section). Square cross section means an additional volume of concrete surrounding the circle. Therefore,
casting and testing of a specimen in vertical position means that the fibers and coarse aggregate will be aligned in a direction perpendicular to the loading axis. However, this situation is reversed when the specimens are cast horizontally, but tested in the vertical direction. The differences in the geometry and internal configuration of fibers and coarse aggregates may influence the response of the concrete under loading. It was found that the initial tangent modulus \( E_n \) is not affected by specimen shape, but horizontally cast prisms possessed higher values than those cast vertically. Horizontally cast prisms have about the same value of \( E_n \) of plain concrete, but fiber concrete cylinders have lower values. Concrete specimens cast vertically have marginally higher compressive strength than those cast horizontally. The shape of specimen has no effect of strength provided it has same aspect ratio. An increase in \( V_f \) increases the strains at peak stress for cylindrical specimens, but does not affect this value for horizontally cast prisms. Vertically cast prisms have higher stresses at peak stress than corresponding cylinders. Cylinders (cast vertically) exhibit better ductility than horizontally cast prisms.

2.1.4 Ultimate strength and toughness

In tension, flexure and torsion, the maximum strength is controlled primarily by fibers gradually pulling out, and the stress in the fibers at ultimate load which is less than the yield strength of the fiber. The decrease in ultimate load past ultimate strength is gradual and the total energy absorbed, in depending and stretching of the fibers, before complete separation of a beam is higher for fiber reinforced concrete than for plain concrete. The ultimate strength of fiber reinforced concrete depends on the fiber volume fraction \( V_f \) and aspect ratio \( (l_f/d_f) \). There is a considerable increase in the ductility or
toughness (area under stress-strain curve or load-deflection curve) as evidenced by a larger yield like zone and a shallow descending portion of the stress-strain curve for fiber reinforced concrete as compared with plain concrete subjected to compression.

Lok and Xiao (1999) demonstrated how the first crack flexural strength ($f_{cr}$) and the ultimate flexural strength ($f_{tu}$) could be derived from a constitutive stress-strain model for SFRC. They classified the flexural moment-curvature as 1) softening, 2) idealized elastoplastic, and 3) strengthening. They also presented a prediction model for cracked moment, ultimate moment, and ultimate tensile strength for a range of $0.5\% < V_f < 2\%$, $30 \leq l/d \leq 100$, $3 \text{ N/mm}^2 \leq \tau_d$ (bond strength) $\leq 7 \text{ N/mm}^2$, and compression strength $30 \text{ N/mm}^2 \leq f'_c \leq 50 \text{ N/mm}^2$. The behavior as suggested by Ghalib (1980) that the transition point occurs when the flexural stress in the tension face equals the direct tensile strength of the concrete composite material. Beyond this stage, the stress is transferred to the steel fiber bridging the propagating crack length. Consequently, the neutral axis shifts towards the compression face. The inferior tensile strength of the composite contributed to the compression stress, remaining almost linear up to failure. Finally, complete collapse occurred when the fibers pulled out of the matrix at the ultimate load ($P_{ult}$).

Graig et. al. (1987) addressed the increased ductility of reinforced concrete sections subjected to pure bending with and without compression reinforcement, and with and without steel fibers. Four $7'' \times 15''$ beams were considered, two of them without steel fibers and the other two with steel fibers (one singly reinforced while the other beam is doubly reinforced for each group). The fiber volume fraction was $1.75\%$ by volume and had aspect ratio of 100, the beams were tested under four point loads with a simply supported
length of 108’. The theoretical moment-rotation relationship was computed for each section using a computer program developed for this study. It was found that the use of fibers improved the ductility of the section and increased the ultimate moment capacity of the beam. The section ductility increased with the addition of fiber and with higher concrete grades.

Dawrakanath and Nagaraj (1992) reported a comparative study of the full depth inclusion (steel fibers dispersed in the entire volume of the beam) and half depth inclusion (steel fibers dispersed over half the depth of the beam on the tension side) of steel fibers on reinforced concrete beams. The beams measured were 100 mm x 208 mm with a length of 1800 mm. The beams were singly reinforced with two reinforcement percentages of 0.77% and 1.28%. For each percentage of reinforcement, two fiber volume fractions ($V_f$) of 0.75% and 1.5% were used. For each of the fiber percentages, two different fiber dispersion (half-depth inclusion and full depth inclusion) were considered. Beams had a shear span of 600 mm and central flexure span of 300 mm (only tension bars in this section). The beams were provided with transverse reinforcement ($\phi$ 6 mm stirrups) on the shear span to ensure flexure failure. Beams were tested over a simply supported span of 1500 mm. The investigation was carried out to observe the deformation behavior of reinforced concrete beams and make a comparative study of the same beams with full-depth fiber and half-depth fiber inclusion. It was found that half-depth fiber inclusion is practically as effective as full-depth fiber inclusion in bringing about the desired modifications in the deformational characteristics up to failure. The improvements in these characteristics are reflected in terms of reduced...
deflections, under similar loads, reduced strains in steel and reduced curvature. Reduced steel strains are indications of reduced crack widths.

Mitchell et. al. (1996) investigated the effect of steel fibers on the behavior of reinforced concrete elements subjected to pure tension. Twelve tension specimens with cross section area of 95x172 mm and 1500 mm long were tested. Six specimens were constructed with normal-strength concrete with and without steel fiber, and another six specimens were constructed with high-strength concrete, with and without, steel fiber. The steel fiber used was hooked end with length of 30 mm and diameter of 0.5 mm. The fiber volume fraction \( (V_r) \) used was 1%. A single No. 15 bar was provided in each specimen giving a reinforcement ratio of 1.23 percent. The reinforcing bars were with no coating, 6 to 8 mil coating, and 10 to 12 mil coating and extended 250 mm outside of the ends of concrete. A Linear Voltage Differential Transducer (LVDT) was placed on two sides of the specimens and was clamped to the steel reinforcing bar just outside of the reinforced concrete. At each load stage, the cracks were measured using a crack width comparator. It was found that the steel fiber increased the tensile strength, tension stiffening, and ductility for both normal and high strength concrete. After cracking and significant deformation, reinforced concrete showed some degree of tension stiffening. After yielding of reinforcing bars, only the specimens containing steel fiber showed stiffening. Steel fiber reduced crack widths in both normal and high strength concrete, high strength concrete exhibited smaller crack widths than normal concrete specimens. Steel fiber prevented bond-splitting cracks from propagating in both normal and high strength concrete. Specimens reinforced with epoxy-coated bars showed larger crack
widths than specimens reinforced with uncoated bars and increased coating thickness resulted in larger crack widths because the roughness of the coated bars is smaller than uncoated bars which results in reducing the cavities around surface area of the bar increases the chance for the concrete and bar to act as one unit. This behavior caused a non-splitting failure mode, but lead to crack development before failure.

2.2 Polymer Modified Concrete (PMC)

2.2.1 Polymer latexes (Polymer dispersion)

Polymer latexes (Polymer dispersion) consisting of very small (0.05-5 μm in diameter) particles dispersed in water are usually produced by emulsion polymerization. Polymer latexes are generally classified into three types by the kind of electric charges on polymer particles: cationic (positively charged), anionic (negatively charged), and nonionic (uncharged). In general, the polymer latexes are copolymer systems of two or different more monomers, and their total solids including polymers, emulsifiers, and stabilizers are 40-50% by mass.

The general requirements of polymer latexes as polymer-based admixtures are as follows:

1. Very high chemical stability towards the extremely active cations such as calcium ion (\(\text{Ca}^{2+}\)) and aluminum ions (\(\text{Al}^{3+}\)) liberated during cement hydration.

2. Very high mechanical stability under severe actions, especially high shears in mortar or concrete during mixing and transfer in pumps.
3. Low air-entraining action due to the use of suitable antifoaming agent during concrete mix.

4. No adverse influence on cement hydration.

5. Excellent water resistance, alkali resistance and weatheribility of the polymer films formed in concrete.

6. Thermal stability for wide variation in temperature during transportation and strong (freeze-thaw stability) in cold climate areas or in winter or high temperature storage stability in hot climate areas or summer.

7. Formation of continuous polymer films in concrete due to a lower minimum film-forming temperature than the application temperature, and the high adhesion of the polymer film to cement hydrate and aggregate. The minimum film-forming temperature is defined as the lowest temperature at which the polymer particles of latex have sufficient mobility and flexibility to coalesce into continuous polymer films (Ohama, 1998).

2.2.2 Polymerization-hydration process

It is very important that both cement hydration and polymer film formation proceeds well to yield a monolithic matrix phase with a network structure in which the cement hydrate phase and polymer phase interpenetrate. In polymer-modified concrete, aggregates are bound by such a co-matrix phase.
Polymer latex modification of concrete is governed by both cement hydration and polymer film formation process in their binder phase.

The cement hydration process generally precedes the polymer film formation process by the coalescence of polymer particles in polymer latexes. Both cement hydration and polymer film formation processes form a co-matrix phase as shown in Figures 2.1 and 2.2 (all figures are located at the end of this chapter).

Polymer-modification of hydrating cement systems is mechanical in nature. A network of polymer adheres to hydration products and clogs the pore structure, continued formation of hydration product occurs along with formation of a polymer network, interconnecting as moisture is consumed. Hydration products continue to form and encase as a cement matrix with a polymer film matrix dispersed throughout, bridging micro cracks caused by shrinkage (Colville et. al., 1999).

Some chemical reactions may take place between the particle surfaces of reactive polymers such as polyacrylic asters (PAE) and calcium ions (Ca$^{2+}$), Ca (OH)$_2$ solid surfaces over the aggregates. Such reactions are expected to improve the bond between the cement hydrates and aggregates, and to improve the properties of hardened latex modified concrete.

In most latex-modified concretes, a large quantity of air is entrained compared with ordinary concrete because of the action of the surfaces contained as emulsifiers and stabilizers in the polymer latex (Lavelle, 1988). Some air entrainment is useful to improve workability, however an excessive amount of entrained air causes a reduction in strength because the concrete density decreases and the larger air bubbles may become
crack initiators. Air entrainment is controlled by using proper antifoam agent. Recent commercial polymers latexes contain proper antifoam agents. The air content in most latex-modified concretes is in the range of 5-20% of the concrete volume (ACI 548.1R-92).

Polymer Modified Concrete (PMC) mixtures are normal Portland cement concrete mixtures to which polymer has been added during the mixing process. Common polymers used are latex such as: Styrene-Butadiene rubber, polyvinyl acetate, acrylic, and natural rubber. Acrylic polymers latex is used in Portland cement primarily in tile adhesive, grout, floor, topping, stucco, and patching applications. Epoxy resins are a group of thermoset resins, which when reacted with curing agent, cure to a tough chemically resistant polymer and, when mixed with concrete provide freeze-thaw resistance.

2.2.3 Application of polymer modified concrete (PMC)

Polymer modified concrete with percentage ranges from 25 to 35 percent of cement weight may be used for bridge decks overlay. The most widely used latex is Styrene-Butadiene (Dow-1 and Dow -2) in parking garage floor systems and ramps, in industrial floors subjected to variety of exposures, which can cause rapid deterioration of concrete. Adding polymer to concrete to construct precast members improves concrete workability because the water-cement ratio is usually low (ACI 548.1R-92).

2.2.4 Mix proportions

Polymer levels of 15-20 percent of cement weight are required for optimum
performance of concrete such as reducing micro-cracks, increasing freeze-thaw
resistance, and resisting corrosion. The percentage is based on the weight of polymer
solids to the weight of cement. For a latex product having 50 percent solids, 30 lb of the
latex liquid would be required per 100 lb of cement to achieve a 15 percent latex level
(ACI 548.1R-92). According to (DOW-1 and DOW-2), Styrene Butadiene Latex
(Modifier A) is added by about 35 percent of cement weight to produce polymer-
modified concrete to be used in bridge overlays.

Type I, II, and III Portland cement have been successfully used in polymer modified
Concrete (PMC) mixes. Air-entraining Portland cement should not be used unless
specified by the polymer manufacturer (ACI 548.1R-92). Water-cement ratios for
workable latex modified mixes are typically 0.3 to 0.4. For pre-cast members, polymer
content could range from 10-20 percent and water-cement ratio can be held in the range
of 0.25 to 0.35 (ACI 548.1R-92).

Vipulanandan and Paul (1990) studied the compressive and tensile properties of
epoxy polymer concrete and polyester polymer concrete in terms of curing conditions,
temperature, and strain rate. The mixes basically were polymer and sand, there is no
cement since polymer acts as binder instead of cement. They also studied the influence of
various aggregate sizes and grading on the mechanical properties of epoxy polymer
concrete and polyester polymer concrete. They added epoxy by weight of cement as 15
and 20 percent and polyester by 15%. All specimens were first cured at room
temperature for one day and then cured at 22, 40, 60, and 80 C° (72, 104, 140, and 176
F°) for another day. The optimum curing temperature was 80 C° based on optimum
compressive strength. The epoxy polymer concrete compressive strength at 80 °C was 11143 psi while the polyester polymer concrete compressive strength at the same temperature value was 9714 psi. The compressive strength for both epoxy polymer concrete and polyester polymer concrete increases with increasing loading strain rate and decreased with the temperature increase beyond 80 °C. It was observed also, that there was no direct correlation between particle parameters effect (gradation and size) on compressive strength of polymer concrete and similar trend was observed for tensile strength. It was observed that the order of mixing the aggregates with the polymer influenced the performance of the concrete. It is better to add the larger particles first, this is due to the fact that when finer particles are added first to the polymer, they would require great amount of polymer to coat their large surface area and do not leave enough resin for the rest of the aggregate.

2.3. Fiber/Polymer Modified Concrete

Chen and Chung (1996) performed a comparative study of the tensile, compressive, and flexural properties for concrete reinforced with steel, carbon, and polyethylene fibers. The concrete considered was either with latex or without latex. The latex used was Styrene Butadiene Rubber with an amount of 20 percent by weight of cement along with antifoam agent in the amount of 0.5 percent by weight of latex. The steel fiber used had a length of 5 mm and diameter of 60 μm, carbon fiber had a length of 5 mm and diameter of 10 μm, and polyethylene fiber used had a length of 5 mm and diameter of 38 μm. Specimens with dimensions of 2x2x2 inches were used for compression tests, while
dog-bone-shaped specimens with a middle section of 20x30 mm were used for tension tests. Flexural tests were performed on 40x40x160 mm prisms under three-point loading and 140 mm span length. It was found that the carbon fiber increased tensile strength by about 2.45% without latex and 3.15% with latex compared to plain concrete. Steel fiber increased flexural strength by about 5.5% without latex and 9.68% with latex. Polyethylene fiber gave the higher value of flexural toughness of 1.305 Mpa.mm without latex, and 1.318 Mpa.mm with latex.

Soroushian et. al. (1991) presented material properties of latex-modified carbon fiber reinforced mortar. The carbon fiber used has a length which ranged between 1/16 inch and 1/2 inch at fiber volume fraction (V_f) of 3 percent. Styrene butadiene latex was used with solid/binder content of 10 percent by cement weight and antifoam agent was used with the latex. 1.5x1.5x6 inch specimens were subjected to flexure test under three point loading and 3x6 inch cylinders were prepared for compression tests. It was found that at 10 percent solids of latex, the compressive strength was reduced by about 18 percent and flexural strength increased by about 3 percent. The high efficiency of carbon fiber in reinforcing cement materials results partially from their small diameters, which lead to relatively close spacing of fibers in the composite materials, the closely spaced carbon fibers encounter micro cracks in the matrix, thus arresting them. The small fiber dimensions produce a relatively high fiber count per volume and typically make it difficult to disperse carbon fibers in cement mixtures. Fine aggregate with a maximum size of 1/10 inch are required for getting uniform fiber dispersion.
Zayat and Bayasi (1996) reported effects of varying amounts of styrene butadiene latex in carbon fiber cement. The carbon fiber had a length of 1/8 inch and diameter of $4 \times 10^{-4}$ inch. They used 0, 5, 10, and 15 percent of polymer binder/ratio and carbon fiber volume fraction ($V_f$) of 2 percent. 4x8 inch cylinders were tested for compression strength and 1.5x1.5x6.5 inch prisms were tested under central point loading with span of 4 inches for flexure. Tensile dog-bone specimens with a central cross section of 1x1 inch were used to establish the tension stress-strain relationship. It was found that at 5 and 10 percent of latex content, the compressive strength of carbon fiber cement might tend to decrease; while at 15 percent binder/cement ratio, the compressive strength increases. Latex addition to carbon fiber cement increases flexural strength. At 5 and 10 percent latex/binder ratio, tensile strength is unaffected while it is increased by the addition of 15 percent latex content.
Figure 2.1 Process of latex formation (Lavelle, 1988)
a) Immediately after mixing
Unhydrated Cement particles
Polymer particles
Aggregates
Entrained air

b) First step
Mixtures of Unhydrated Cement
Particles and cement gel
Entrained air

c) Second step
Mixtures of cement gel and unhydrated cement
particles enveloped with a close-packed layer
of polymer particles
Entrained air

d) Third step
Cement hydrates enveloped with
polymer films or membranes
Entrained air

Figure 2.2 Simplified model of formation of polymer-cement co-matrix (Ohama, 1998)
CHAPTER 3

MATERIAL CHARACTERIZATION OF PLAIN, FIBER, LATEX, AND FIBER/LATEX CONCRETE

3.1 Introduction

As discussed in chapter 2, concrete properties can be improved by adding chopped steel fibers to the concrete mix to produce steel fiber reinforced concrete (SFRC). Polymer modified concrete (PMC), which is conventional concrete modified with polymer, has wide applications in repair of deteriorated or damaged concrete structures, and the construction of bridge decks and overlays of ramps and other structures subject to wear and adverse weather conditions.

A combination of steel fiber and polymer, when added to concrete, produce steel fiber/polymer modified concrete (SFPMC), which provides the advantages of both steel fiber and polymer together to the strength and durability of concrete.

The mechanical characteristics of 1% steel fiber reinforced concrete (SFRC), 1% to 7.5% solids of polymer modified concrete (PMC), and 1% to 7.5% polymer solids with 1% steel fiber of steel fiber/polymer modified concrete are presented. Applications to slender rectangular beams and half I-beams subject to shear, torsion, and bending stresses are investigated in this dissertation. Therefore, it is necessary, as a first stage of this research, to characterize the properties of a specific concrete mix used by the Nevada
Department of Transportation (NDOT) and to modify it into SFRC, PMC, and SFPMC by adding 1% of steel fiber by volume and various percentages of latex polymer (styrene butadiene rubber). The final aim in this research is to recommend a modified mix that may be used in the structural elements of bridges and on the concrete slabs themselves instead of using these mixes occasionally in overlay of bridge decks.

3.2 Concrete Mix Proportions

A 3/8" size aggregate concrete mix was chosen according to Nevada Department of Transportation (NDOT) specifications as shown below. The concrete mix is designed for a 28 day compressive strength of 4000 psi and slump of 4"±1". The mix proportions are as follows:

3.2.1 Plain concrete

<table>
<thead>
<tr>
<th>Mix Components</th>
<th>Weight in pounds per cubic yard</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/8 Coarse Aggregate (SSD)</td>
<td>1735</td>
</tr>
<tr>
<td>Fine Aggregate (SSD)</td>
<td>1400</td>
</tr>
<tr>
<td>Water</td>
<td>317</td>
</tr>
<tr>
<td>Cement, Type I</td>
<td>658</td>
</tr>
<tr>
<td>Entrained Air: 2%</td>
<td></td>
</tr>
<tr>
<td>W/C = 0.48</td>
<td></td>
</tr>
<tr>
<td>Sand : Gravel = 0.45 : 0.55</td>
<td></td>
</tr>
</tbody>
</table>

This original plain concrete mix was modified with the addition of 1% steel fiber, 1% to 7.5% of polymer solids (styrene butadiene), or both steel fiber and polymer.

1 SSD means Saturated Surface Dry
3.2.2 Steel fiber reinforced concrete (SFRC)

When steel fiber is added to plain concrete, superplasticizer should be used to increase the concrete workability and water should be reduced by the same amount of added superplasticizer (ACI 548.1R-92). One percent of steel fiber by volume was added to plain concrete producing SFRC. One percent fiber volume fraction ($V_f$) of steel fiber was chosen as an average of reasonable $V_f$ limits of 0.5% to 1.5%. The concrete workability will greatly decrease if the fiber volume fraction exceeds 1.5%. The steel fiber is known to be effective when the fiber volume fraction is larger than 0.5% (ACI Committee 544-3R-84). The SFRC mix used is:

<table>
<thead>
<tr>
<th>Mix Components</th>
<th>Weight in pounds per cubic yard</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/8 Coarse Aggregate (SSD)</td>
<td>1735</td>
</tr>
<tr>
<td>Fine Aggregate (SSD)</td>
<td>1400</td>
</tr>
<tr>
<td>Water</td>
<td>303.5</td>
</tr>
<tr>
<td>Cement, Type I</td>
<td>658</td>
</tr>
<tr>
<td>Water reducer Superplasticizer</td>
<td>12.34</td>
</tr>
<tr>
<td>1% Steel Fiber</td>
<td>132.3</td>
</tr>
<tr>
<td>$W/C$ = 0.47</td>
<td></td>
</tr>
<tr>
<td>Sand : Gravel = 0.45 : 0.55</td>
<td></td>
</tr>
</tbody>
</table>

3.2.3 Polymer modified concrete

Polymer added to plain concrete, with varying ratios of polymer to cement weight, produces polymer modified concrete (PMC). For example, 10 percent polymer means 10 percent solids of polymer added. The water in the mix should be reduced by the same amount of the water contained in the polymer emulsion since polymer is not added to the concrete in a solid form, but as an emulsion.
### 3.2.3.1  PMC of 2.5 percent polymer (2.5 percent solids)

The mix was adjusted and mix proportions were set as follows:

<table>
<thead>
<tr>
<th>Mix Components</th>
<th>Weight in pounds per cubic yard</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/8 Coarse Aggregate (SSD)</td>
<td>1735</td>
</tr>
<tr>
<td>Fine Aggregate (SSD)</td>
<td>1400</td>
</tr>
<tr>
<td>Water</td>
<td>299.4</td>
</tr>
<tr>
<td>Cement, Type I</td>
<td>658</td>
</tr>
<tr>
<td>Styrene Butadiene Rubber</td>
<td>32.9</td>
</tr>
<tr>
<td>1% Steel Fiber</td>
<td>132.3</td>
</tr>
</tbody>
</table>

W/C = 0.47

Sand : Gravel = 0.45 : 0.55

### 3.2.3.2  PMC of 3.75 percent polymer (3.75 percent solids)

The mix was adjusted and mix proportions were set as follows:

<table>
<thead>
<tr>
<th>Mix Components</th>
<th>Weight in pounds per cubic yard</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/8 Coarse Aggregate (SSD)</td>
<td>1735</td>
</tr>
<tr>
<td>Fine Aggregate (SSD)</td>
<td>1400</td>
</tr>
<tr>
<td>Water</td>
<td>291</td>
</tr>
<tr>
<td>Cement, Type I</td>
<td>658</td>
</tr>
<tr>
<td>Styrene Butadiene Rubber</td>
<td>49.35</td>
</tr>
<tr>
<td>1% Steel Fiber</td>
<td>132.3</td>
</tr>
</tbody>
</table>

W/C = 0.44

Sand : Gravel = 0.45 : 0.55
3.2.3.3 PMC of 5 percent polymer (5 percent solids)

<table>
<thead>
<tr>
<th>Mix Components</th>
<th>Weight in pounds per cubic yard</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/8 Coarse Aggregate (SSD)</td>
<td>1735</td>
</tr>
<tr>
<td>Fine Aggregate (SSD)</td>
<td>1400</td>
</tr>
<tr>
<td>Water</td>
<td>283</td>
</tr>
<tr>
<td>Cement, Type I</td>
<td>658</td>
</tr>
<tr>
<td>Styrene Butadiene Rubber</td>
<td>65.8</td>
</tr>
<tr>
<td>1% Steel Fiber</td>
<td>132.3</td>
</tr>
</tbody>
</table>

W/C = 0.43 and Sand : Gravel = 0.45 : 0.55

3.2.3.4 PMC of 7.5 percent polymer (7.5 percent solids)

The mix was adjusted and mix proportions were set as follows:

<table>
<thead>
<tr>
<th>Mix Components</th>
<th>Weight in pounds per cubic yard</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/8 Coarse Aggregate (SSD)</td>
<td>1735</td>
</tr>
<tr>
<td>Fine Aggregate (SSD)</td>
<td>1400</td>
</tr>
<tr>
<td>Water</td>
<td>266.5</td>
</tr>
<tr>
<td>Cement, Type I</td>
<td>658</td>
</tr>
<tr>
<td>Styrene Butadiene Rubber</td>
<td>98.7</td>
</tr>
<tr>
<td>1% Steel Fiber</td>
<td>132.3</td>
</tr>
</tbody>
</table>

W/C = 0.405 and Sand : Gravel = 0.45 : 0.55

3.2.4 Steel Fiber/Polymer Modified Concrete (SFPMC)

The previous polymer modified concrete (PMC) mixes (2.5%, 5%, and 7.5%) were modified with the addition of 1% steel fiber by volume of the concrete mix to produce SFPMC.
3.3 Concrete Materials

3.3.1 Coarse aggregate

3/8" coarse aggregates were produced at the Nevada Ready Mix (NRM) aggregate pit in saturated surface dry (SSD) condition (the surface of aggregate particles are wet). The aggregates were stored in a steel tank and covered by wood boards to keep them in SSD condition. The size of coarse aggregate was chosen as 3/8" because the slender beams considered in this research have 3" width which is a minimum of 3-times of the steel fiber length (l_f = l_f), as specified by ASTM C1018.

3.3.2 Fine aggregate

Sand was imported from the Nevada Ready Mix pit in saturated surface dry (SSD) condition (the surfaces of aggregate particles are wet) and was stored in a steel tank and covered by a wood board to keep it in SSD condition.

3.3.3 Cement

Type I low alkali Portland cement was used.

3.3.4 Steel fiber

UT wavy steel fibers (Figure 3.1), all figures are located at the end of this chapter, purchased from Ribbon Technology Incorporation which meet ASTM A820, were used having length of 1 inch and diameter of 0.02 inch. The tensile strength of the fiber ranges between 160 ksi and 200 ksi, also 304 RIBTEC-GR was used for I-girder specimens. Other properties of steel fiber is shown in Table 3.1

3.3.5 Superplasticizer

High–range water reducing admixture superplasticizer with the trade name of RHEOBUILD 2500, manufactured by Master Builders and satisfying ASTM C494.
Table 3.1 304 RIBTEC-GR steel fiber properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Melt temperature range, °F</td>
<td>2550-2850</td>
</tr>
<tr>
<td>Thermal conductivity @ 1000 °F</td>
<td>11.6</td>
</tr>
<tr>
<td>BTU/NR/ft²/F°/°F</td>
<td></td>
</tr>
<tr>
<td>Modulus of elasticity x 10^-4 @800 °F, psi</td>
<td>18</td>
</tr>
<tr>
<td>Fiber tensile strength @ 1800 °F, psi</td>
<td>18000</td>
</tr>
<tr>
<td>Coefficient of thermal expansion x 10^6 @ 1800 °F/°F</td>
<td>1900</td>
</tr>
</tbody>
</table>

Type F, was used. The manufacturer recommended dosage was from 18-30 oz per 100 pound of cement, depending on the required slump which ranges from 3 inches to 5 inches.

3.3.6 Polymer

Styrene Butadiene Rubber (SBR) with a trade name of Modifier A (Figure 3.2), a DOW Chemical Company product, has been used as polymer additive to the concrete. Modifier A is approved by the Federal Highway Administration (FHWA) to be used in bridge overlays. DOW supplies their products in 55-gallon (208 liter) drums, 5000 gallon (19 m³) tank truck, or 20,000-gallon (75 m³) railcars. Delivery time of 2-4 days is typical (Kuhlman, 1990). Latex emulsion admixture (DOW Modifier A) is a styrene butadiene polymeric emulsion in which the polymer comprises 47-49 % of the total emulsion. Stabilizers and an antifoam agent have been added at the point of manufacture. The air content of Polymer Modified Concrete using Modifier A is about 6.5% (DOW-1)

3.3.6.1 Composition/Information on ingredients

<table>
<thead>
<tr>
<th>Component</th>
<th>Percentage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Styrene Butadiene polymer</td>
<td>40-60%</td>
</tr>
<tr>
<td>Water</td>
<td>40-59%</td>
</tr>
</tbody>
</table>

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.
Proprietary stabilizer 1 – 5%

Modifier A classification are shown in Table 3.2

3.3.6.2 Physical and chemical properties

The physical and chemical properties of modifier A can be summarized as follows:

Appearance: Milky white liquid emulsion.

Odor: slight odor

Vapor pressure: 17.5 mmHg @ 20°C

Table 3.2 Styrene Butadiene Rubber (SBR) content classification

<table>
<thead>
<tr>
<th></th>
<th>Analysis</th>
<th>Unit</th>
<th>Specification</th>
<th>Method</th>
</tr>
</thead>
<tbody>
<tr>
<td>Solids</td>
<td>47.6</td>
<td>%</td>
<td>47.0 – 49.0</td>
<td>DOWM 100008</td>
</tr>
<tr>
<td>pH</td>
<td>10.8</td>
<td></td>
<td>9.0 – 11.0</td>
<td>DOWM 100429</td>
</tr>
<tr>
<td>Residue-200 Mesh</td>
<td>0.005</td>
<td>g/900ml</td>
<td>0.500 Max.</td>
<td>DOWM 101784</td>
</tr>
<tr>
<td>Particle Size</td>
<td>2000</td>
<td>Angstroms</td>
<td>1900 – 2200</td>
<td>DOWM 100200</td>
</tr>
<tr>
<td>Viscosity</td>
<td>30</td>
<td>centipoise</td>
<td>40 Max.</td>
<td>DOWM 100317</td>
</tr>
<tr>
<td>Surface Tension</td>
<td>26</td>
<td>dyne/cm</td>
<td>22 – 31</td>
<td>DOWM 100362</td>
</tr>
<tr>
<td>Freeze/Thaw</td>
<td>&lt;0.1</td>
<td>g/400ml</td>
<td>0.100 Max.</td>
<td>LTM014</td>
</tr>
<tr>
<td>% Buta Content</td>
<td>39.4</td>
<td>%</td>
<td>30.0 – 40.0</td>
<td>LTM223</td>
</tr>
<tr>
<td>Weight per Gallon</td>
<td>8.50</td>
<td>lb/gal</td>
<td>8.40 – 8.60</td>
<td>DOWM 100364</td>
</tr>
</tbody>
</table>

Boiling point: 212°F, 100°C

Solubility in water: Latex as sold is dilutable. Polymer component is insoluble

Specific Gravity: 0.980 – 1.040
3.3.6.3 Stability and reactivity

Chemical stability: stable under recommended storage conditions, which is between temperatures of 40°F and 110°F. It may coagulate if frozen at 32°F. Material may develop bacterial odor on long-term storage.

3.4 Quality Control

3.4.1 Curing of concrete

3.4.1.1 Plain concrete curing condition control

In 2000, Plante et. al. studied the effect of field temperature on the compressive strength of concrete cylinders in summertime. Sets of 150x300 mm and 100x200 mm cylinders were molded from the same batch of concrete. Each test required three or four cylinders, which were tested under compression after 7 and 28 days (one and two cylinders) respectively.

Each set of cylinders was cured under the following conditions:

-24 hours at an ambient temperature of 23°C (control cylinders),
-24 hours exposed to exterior ambient conditions,
-24 hours in exterior shaded conditions,
-72 hours exposed to exterior ambient conditions,
-24 hours in exterior shaded conditions, then exposed to exterior ambient conditions with the shade removed for the following 48 hours,
-On site curing box, or
-Outside in water container
-In the job site, the cylinders were placed in the box 4 hours after casting and removed from the molds after 72 hours. Some of the samples were kept 24 hours unprotected outside at ambient conditions; others are kept 72 hours unprotected at ambient conditions.

- In laboratory, 24 hours at ambient temperature (23°C).

Following initial curing, all sets of cylinders were kept until 7-28 days according to Canadian Standard Association (CSA) requirement.

The water level in the container maintained 2 cm before the top surface of cylinders. The job site box was maintained at 23°C by either cooling or heating necessary.

For temperature control, each set of four cylinders one contained thermocouple to measure the internal temperature during the initial curing period. The temperature was recorded for the first 3 days.

A micro-structural analysis of concrete was carried out on a number of cylinders. This analysis comprised a study of fresh fractures and polished surfaces of the concrete samples by scanning electron microscope (SEM). The porosity of the concrete was evaluated by a mercury intrusion porosimotry (MIP).

It was found that the compressive strength after 28 days for the unprotected cylinders dropped about 15% compared to box cured cylinders. Also, the compressive strength of the unprotected cylinders is 10% less than container-cured cylinders the difference in compressive strength between cylinders shaded for first 24 hours and those in direct sun was 10 Mpa. MIP analysis shows that high initial temperature increases the concentration of pores greater than 0.3 μm. The pore volume of pores superior to 0.3 μm represent 10, 21, and 25% of the total pore volumes of the concrete cured in water.
shade, and in direct sunlight respectively. Those pores control the compressive loss in concrete strength; the development of this porosity depends on the thermal evolution of the concrete during its hydration. At an early age of concrete, temperature accelerates cement hydration and the product of hydration formed in this way blocks later cement hydration by forming a barrier between water and anhydrous grains. This explains the drop in compressive strength for those cylinders which were kept unprotected in sunlight. It was concluded that adequate protection is required for the cylinders for first 24 hours after casting and use of water container diminishes the effect of high exterior temperatures.

3.4.1.2 Polymer concrete curing condition control

The method of curing polymer modified concrete must take into consideration both cement hydration and film formation. Recommended practice is curing under plastic or burlap for the first 24h, followed by 24h of dry curing to ensure that film formation takes place. Subsequent wet curing is not necessary in most conditions due to the polymer film blocking the pores. This process allows cement hydration to proceed for the first 24h establishing the initial matrix pore structure and consolidation of latex in the pore fluid. Subsequent drying causes film formation to occur, bonding the polymer to hydration products on the pore walls. As further evaporation occurs, polymer builds up clogs the pores, trapping moisture inside. Continued hydration will eventually consume this trapped moisture, causing further film formation. The foregoing scenario depends on the occurrence of two separate processes that rely on the depletion of water. For these to occur, the polymer and the cement cannot chemically interact. Chemical interaction between polymer molecules and hydration products will cause coagulation and breaks in
the polymer molecules resulting in the release of heat and decomposition. This does not occur in typical polymer-modified concrete (Colville et. al., 1999). So, to obtain maximum physical properties, latex-modified concrete should be air-dry cured at ambient room temperature.

Folic and Radoninjanin (1998) studied the effect of latex modified concrete on the curing conditions and mechanical properties. The first phase of their research explained the curing influence on some properties of concrete to establish the most suitable curing conditions for concrete. Concrete was modified by 5, 10, 15, and 20 percent of styrene butadiene latex dispersion on the cement mass. In the second phase of the research, concrete was modified by 2.5, 5, and 7.5 percent of latex dispersion. Compression tests were performed on 6 inch cubes, and 4x4x6 inch prisms were tested for flexure. It was found that the greatest effect on physical and mechanical properties of latex modified concrete was achieved at the optimal combination of wet and dry curing, curing in high humidity conditions within a six day period followed by curing in a dry environment. Compressive strength was slightly increased by 1-7% and flexural strength increased about 40% at polymer percent of 7.5%. Water absorption decreased with an increase of the polymer-cement ratio, at 7.5 percent water absorption was 32 percent less than in the regular concrete. The reason for the improvement is that the cracks were filled with polymer and the surface was covered with a polymer membrane.

3.4.2 Concrete workability and superplasticizer

According to Faroug et. al. (1999), the definition of workability is determined by the relationship between two factors: 1) the rheological parameters of a given mix; and 2) the
dynamic forces acting on it during processing (Szwaboski, 1987). The rheological properties are determined by the reaction of the mix to the forces acting on it during transport and mechanical processing, and by the resistance of its structure of these forces. A concrete mix can be considered as a three-phase system in which shear resistance (τ) is a fundamental property of the system. Shear resistance results from a combination of cohesion; internal friction; and viscous resistance. The relative contribution of each of these three components makes the resistance dependent on the composition of the concrete mix, and on its structure. For liquid concrete with high paste content, viscous resistance is predominant. Alternatively, for low paste content mixes, internal friction is predominant. The shear stress is lower than the total value of cohesion and internal friction; the mix behaves like a viscous liquid when the stress is higher than the total of cohesion and internal friction resistance. However, plastic flow of the concrete mix occurs when the shear stress values are close to the total value of resistance of cohesion and internal friction. The use of superplasticizer causes changes in the rheological characteristics of the concrete mix, making it more liquid due to high negative value of electrokinetic’s potential of the cement-water interface which increases cement dispersion in concrete paste and facilitates the release of water by reducing adsorptive and capillary forces within the cement paste (Roy and Asaga 1980 a, b ; and Malhorta 1990). The introduction of superplasticizer into concrete mixes improves their workability by lowering the shear and flow resistance (Faroug et. Al. 1999). However, this effect gradually disappears with passage of time. The lower the w/c ratio, the more effective is the superplasticizer in increasing the mix workability when applied at constant dosage. At high w/c ratio ≥0.5, the superplasticizer becomes ineffective and
segregation of the mix may occur. Latex addition into fresh concrete causes the effect typical for admixtures like superplasticizer because latex particles act to lubricate the mix and improve its workability (Soroushian et. al., 1991). Such polymer influence enables making concrete mixtures of a required consistency with a water quantity up to 30 percent less than in traditional concrete. Better workability of modified concretes in comparison to the traditional cement concrete, results from the known “ball-bearing” influence of polymer particles and the dispersion effect of surface active substances in latex (Folic and Radonjanin, 1998).

3.4.3 Coarse aggregate gradation

Six samples of Nevada Ready Mix (NRM) coarse aggregate were sieved, the gradation did not match NDOT specifications gradation for coarse aggregate as shown in Figure 3.3.

The coarse aggregate was sieved and each size was proportioned and mixed together to get an aggregate gradation meeting NDOT specifications. The mixed aggregate was stored until the day of mixing. On the mixing day, the aggregate was wetted with a calibrated amount of water of 4.8 pounds per 2 cubic feet of aggregate about one hour before mixing to get the saturated surface dry (SSD) condition. Figure 3.4 shows the aggregate sizes of 3/8”, #4, # 8, and #16.

The Nevada Department of Transportation (NDOT) Specifications for this type of aggregate are:

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>Percent Passing by Mass</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/2”</td>
<td>100</td>
</tr>
<tr>
<td>3/8”</td>
<td>85-100</td>
</tr>
</tbody>
</table>
and the NDOT provided mix specifications are:

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>Percent Passing by Mass</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/2&quot;</td>
<td>100</td>
</tr>
<tr>
<td>3/8&quot;</td>
<td>99</td>
</tr>
<tr>
<td># 4</td>
<td>28</td>
</tr>
<tr>
<td># 8</td>
<td>4</td>
</tr>
<tr>
<td># 16</td>
<td>1</td>
</tr>
<tr>
<td># 200</td>
<td>0.7</td>
</tr>
</tbody>
</table>

3.4.4 Fine aggregate gradation

Three samples of NRM sand were sieved, the gradation of NRM sand met the NDOT specifications as shown in Figure 3.5. Sand was also wetted on the day of mixing about one hour before mixing to keep it in SSD condition.

3.5 Concrete Mixing and Specimens Preparation

A two-cubic foot concrete mixer (Figure 3.6) was used to mix concrete at Civil and Environmental Engineering Department, Howard Hughes College of Engineering at University of Nevada, Las Vegas (UNLV). A quantity of two-cubic feet of concrete was mixed for each batch forming eight standard (6" x 12") cylinders, or six cylinders and 2-4" x 4 " x 34" beams.

3.5.1 Mixing procedure

According to Vipulanandan and Paul (1990), it is better to add well-mixed aggregate rather than to add each portion separately. If the aggregates are added separately, it is
better to add the larger particles first. This may be due to the fact that when finer particles are added to the polymer first they require a great amount of polymer for coating the larger surface area, and do not leave enough resin for the rest of the aggregates.

3.5.1.1 Mixing procedure for plain concrete

The mixing procedure for plain concrete was as follows:

1. Add coarse and fine aggregate to mixer and start mixer.
2. Add 1/3 of water.
3. Add 1/3 of cement (using scoop).
4. Repeat steps 2 and 3 until all materials are added to the mixer.
5. Mix for 3 minutes.
6. Stop mixer for 2 minutes.
7. Mix for 2 more minutes.

3.5.1.2 Mixing procedure for steel fiber reinforced concrete (SFRC)

According to Tanigawa et. Al. (1980), the coarse aggregate as well as discrete fibers have a greater tendency to align horizontally, and some air and water may be trapped underneath the fibers and coarse aggregates leaving some voids. To alleviate this problem, the following mixing procedure for SFRC was used:

1. Add coarse and fine aggregate to mixer and start mixer.
2. Add 1/3 of pre-mixed water with superplasticizer.
3. Add 2/3 of cement (using scoop).
4. Add 1/3 of water + superplasticizer.
5. Add fiber through wire mesh basket.

6. Add 1/3 of cement.

7. Add 1/3 of water + superplasticizer.

8. Mix for 3 minutes.

9. Stop mixer for 2 minutes.

10. Mix for 2 more minutes.

3.5.1.3 Polymer modified concrete (PMC)

Same mixing procedure as for plain concrete was used, except that the water is premixed with polymer.

3.5.1.4 Steel fiber/polymer modified concrete (SFPMC)

Same mixing procedure as SFRC was used except that water is premixed with polymer instead of superplasticizer.

3.5.2 Slump test

Slump was performed as per ASTM C143 for plain, steel fiber reinforced concrete, polymer modified concrete, and steel fiber/polymer modified concrete.

3.5.3 Cylinders preparation

Standard cylinders (6" x 12") were prepared as per ASTM C192, which were cast vertically (Mansur et. al., 1999)

3.5.4 Curing method

As mentioned in section 3.4, both plain concrete and steel fiber reinforced concrete samples were kept at room temperature for a day followed by moist curing for 27 days (wet burlap). Both polymer modified concrete (PMC) and steel fiber/polymer modified
concrete (SFPMC) samples were kept wrapped with wet burlap for a day at room temperature followed by 27 days dry curing.

Some samples (Plain, SFRC, PMC, and SFPMC) were left exposed to the weather conditions in July 2000 for the first 24 hours followed by wet curing for 27 days (wet burlap) to study the effect of curing conditions on mechanical properties of concrete.

3.6 Specimens Testing

3.6.1 Compression test

6" x 12" cylinders were tested in compression according to ASTM C39. A Soiltest Digital Compression Machine (Figure B.1, Appendix B) was used to test the specimens. Half inch strain gages were installed (Appendix C) longitudinally in the middle of cylinders on opposite sides to measure strains under compression for the middle section of the cylinder (localized strains), as shown in Figure 3.7 to develop stress-strain relationships under compression loading.

3.6.2 Splitting test

6" x 12" cylinders were used for the splitting tension test according to ASTM C 496 to determine the tensile strength of concrete. Half inch strain gages were installed in the middle of the cylinder cross sections along the horizontal diameter on opposite sides, as shown in Figure 3.8 (Appendix C) to develop stress-strain relationship under splitting tension loading.

3.6.3 Flexural strength test

4" x 4" x 3 4/" beams were cast horizontally (Mansur et. al., 1999) and tested under three point loading in flexure using a Tinius-Olsen Universal Testing Machine.
(Appendix B) to determine the modulus of rupture of concrete. The length of the 4’ x 4’ beams was chosen according to the Tinius Olsen Testing Machines space limitation and to get bending failure rather than shear failure since the a/d ratio was 16/4=4 which falls in the intermediate beams category (Wang and Salmon, 1998).

Half inch strain gages were installed (Appendix C) as shown in Figure 3.9 on test specimen to determine stress-strain characteristic curve under flexure. The Tinius Olsen Universal Testing Machine was calibrated by comparing estimated strain values versus measured strain values using strain gages installed on S-shape steel beam (Figure 3.10). The calibrated chart is shown in Figure 3.11.

3.6.4 Pull-out test

Two 6” x 12” cylinders were prepared from PMC, SFRC, and SFPMC mixes of trial mix #3. In each cylinder, a #3 grade 60 (f_v=60 ksi) re-bar (3/8 inch diameter) was embedded in the cylinder with length of 4 inch, while #5 (5/8 re-bar) was embedded 6 inch on the other side of the cylinder. A bond breaker along a length of 1 inch was provided to force the #3 re-bar to be pulled out as shown in Figure 3.12. The re-bars were inserted inside the cylinder plastic mold and concrete was poured as shown in Figures 3.14. The 3/8 inch steel bar was pulled-out using an MTS machine (see Appendix B) to determine bond strength of concrete as shown in Figure 3.15.

3.7 Results and Discussions

3.7.1 Compressive, tensile, and flexure strengths behavior for trial mix #1

Table 3.3 shows the compression, tension (cylinder splitting), and flexural strength values for plain, steel fiber reinforced concrete (SFRC), polymer modified concrete
(PMC), and steel fiber/polymer modified concrete (SFPMC) for trial mix #1 (NDOT mix). Adding steel fiber, polymer, or steel fiber and polymer together to plain concrete reduced compressive strength of the concrete because of the tendency to increase air content due to polymer addition or steel fiber addition, which reduced concrete density and strength. Also, the presence of steel fiber increased gaps, pockets, and cavities around the surface area of the fiber, which increase air content of concrete. Addition of 5% solids of polymer to concrete to produce polymer modified concrete reduced the compressive strength ($f_c$) of trial mix #1 by 7.4% with an average value of 4053 psi compared to that of plain concrete with an average value of 4378 psi, and tensile strength ($f_t$) average value was 451 psi with strength reduction of 16.5 % compared to that of plain concrete with an average value of 540 psi. The average flexural ($f_r$) strength value of 613 psi with strength reduction of 3.2 % compared to that of plain concrete with an average value of 633 psi. Also the addition of 1% steel fiber to the plain concrete mix reduced compressive strength by 21 % with an average value of 3459 psi; however it increased average tensile strength to 601 psi with an increase of 11.3 % compared to that of plain concrete. Also, the flexural ($f_r$) strength value was 782 psi with a strength increase of 23.5 % compared to that of plain concrete. Moreover, the addition of 1% steel fiber and 5% solids of polymer together to the concrete mix to produce SFPMC reduced the average compressive strength by 10.5 % with an average value of 3917 psi compared to that of plain concrete, and increased the tensile strength value by 8.9 % with and average value of 588 psi compared to that of plain concrete. Also, flexural strength was increased to 994 psi with percentage of 57 % compared to that of plain concrete. From the previous discussion, the reduction of compressive strength with addition of polymer,
Steel fiber, or both is because the addition of the polymer increases air content in concrete, which leads to a lower density of the concrete mix. This trial mix did not provide enough cohesion between the polymerization product and coarse aggregate and hence did not improve the compressive strength property. Adding polymer to concrete reduce micro-crack formation due to cohesion between aggregate and polymerization products which was not enough in this trial mix. The addition of the steel fiber to the plain concrete mix reduced compressive strength values because the presence of steel fiber in the concrete matrix increases the cavities and gaps around the fiber surface area which increase air content. This is due to the deposit of CH crystals adjacent to the fiber surface which is not necessarily continuous and contains some pockets of very porous material. The weak link between the fiber and the matrix is not necessarily at the actual fiber-matrix interface, it can also beat the porous layer (Michigan state University Publication, 1999) as shown in Figure 3.16. However, the increase the tensile strength and flexural strength values by bridging the cracks which keeps the cracks from widening. To provide enough bond between the fiber and concrete matrix, enough cement (600-1000 pound per cubic yard) should be provided to the concrete mix (ACI 544.1R-82). Also, the amount of cement per cubic yard for latex modified concrete should be at least 658 pounds (DOW Chemical Company Publication -1). The PMC specimens with 3.75% solids of polymer (Table 3.3) were vibrated using 1-inch probe diameter concrete vibrator. The concrete was compacted to a denser level and consequently the compressive, tensile, and flexural strength values were increased. The compressive strength was 5009 psi with an increase of 14.4 % compared to that of plain concrete and tensile strength value was 580 psi with an increase of 7.4%, while the

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.
flexural strength reduced by 10.2% with an average value of 568 psi due to low cohesion between the polymerization product and coarse aggregate.

3.7.2 Stress-strain relationship for trial mix #1

3.7.2.1 Compressive stress-strain behavior

Figure 3.17 shows the compressive stress-strain relation for plain, SFRC, PMC, and SFPMC for trial mix #1 (NDOT), with strengthening trend for samples containing 1% steel fiber (Lok and Xiao, 1999). The measured ultimate strain value was 0.002 for plain concrete because strain gage broke (ACI ultimate value under compression =0.003), with the addition of polymer only to plain concrete, the ultimate strain increased to a strain value of 0.005 with the addition 5% solids of polymer. Hence, PMC may be more ductile than plain concrete. With the addition of steel fiber and polymer together, concrete exhibited a higher ultimate strain, at 1% steel fiber and 5% solids of polymer, the ultimate strain value was 0.0065 as compared with 0.0035 (Swamy and Al-Ta’an, 1981), 0.0035 (Hasson and Sahebjan, 1985), and 0.0038 (Lok and Xiao, 1999). The area under the stress-strain curve for this SFPMC mix was maximum (hence, it had the maximum toughness). The SFPMC at 5% solids of polymer and 1% steel fiber produced the best ductility and toughness behavior under compression loading.

3.7.2.2 Tensile stress-strain behavior

Figure 3.18 shows the tensile stress-strain behavior of plain, SFRC, PMC, and SFPMC. The specimen which contained 5% solids of polymer and 1% steel fiber gave ultimate strain of 0.002 and a maximum toughness. The mixes of plain and PMC linearly behaved up to the ultimate stress value and did not carry much tensile strain because the material was brittle under tension loading. The presence of steel fiber, in all
the mixes of polymer modified concrete, improved the concrete ductility since the strain value increased from about 0.0003 to 0.002.

Table 3.3 Compressive and tensile strength for trial concrete mix (NDOT mix) #1

<table>
<thead>
<tr>
<th>Concrete Mix</th>
<th>$f_c$ (psi)</th>
<th>$f_t$ (psi)</th>
<th>$f'_t$ (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plain</td>
<td>4378</td>
<td>540</td>
<td>633</td>
</tr>
<tr>
<td>1% Steel Fiber</td>
<td>3459</td>
<td>601</td>
<td>782</td>
</tr>
<tr>
<td>(2.5% solids of Polymer)</td>
<td>4047</td>
<td>409</td>
<td>694</td>
</tr>
<tr>
<td>(5% solids of Polymer)</td>
<td>4053</td>
<td>451</td>
<td>613</td>
</tr>
<tr>
<td>(7.5% solids of Polymer)</td>
<td>3344</td>
<td>439</td>
<td>596</td>
</tr>
<tr>
<td>1% Steel Fiber + 2.5% solids of Polymer</td>
<td>3479</td>
<td>550</td>
<td>800</td>
</tr>
<tr>
<td>1% Steel Fiber + 5% solids of Polymer</td>
<td>3917</td>
<td>588</td>
<td>994</td>
</tr>
<tr>
<td>1% Steel Fiber + 7.5% solids of Polymer</td>
<td>3182</td>
<td>527</td>
<td>644</td>
</tr>
</tbody>
</table>

± Increase or decrease compared to plain concrete

3.7.2.3 Flexural stress-strain behavior

Figure 3.19 shows the stress-strain behavior under flexural loading. The presence of polymer in PMC specimens improved the concrete ductility and increased the ultimate strain to 0.00017 for 5% solids of polymer. The presence of steel fiber improved the ductility and the ultimate strain to 0.00022 for SFPMC of 1% steel fiber and 5% solids of polymer; it also produced the maximum toughness. From the previous discussion, addition of 5% solids of polymer and 1% of steel fiber to the NDOT concrete mix gave the best properties, however there was insufficient bond between steel fiber and concrete.
matrix. Similarly, not enough cohesion between the polymerization product and coarse aggregate was developed. Therefore, another mix was proportioned by increasing the amount of cement to 846 pound per cubic yard instead of 658 pound per cubic yard to provide enough bond between the steel fiber and the concrete matrix. Sand to coarse aggregate ratio was 0.55:0.45, water/cement (w/c) ratio was reduced from 0.48 to 0.41 to improve concrete strength. Therefore, trial mixes of plain, SFRC of 1% fiber volume fraction, and SFPMC with 1% steel fiber and 5% solids of polymer were prepared.

3.7.3 Trial mixes #2

3.7.3.1 Plain concrete

<table>
<thead>
<tr>
<th>Mix Components</th>
<th>Weight in pounds per cubic yard</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/8 Coarse Aggregate (SSD²)</td>
<td>1400</td>
</tr>
<tr>
<td>Fine Aggregate (SSD)</td>
<td>1735</td>
</tr>
<tr>
<td>Water</td>
<td>347</td>
</tr>
<tr>
<td>Cement, Type I</td>
<td>846</td>
</tr>
<tr>
<td>Entrained Air: 2%</td>
<td></td>
</tr>
<tr>
<td>W/C= 0.41</td>
<td></td>
</tr>
<tr>
<td>Sand : Gravel = 0.55 : 0.45</td>
<td></td>
</tr>
</tbody>
</table>

3.7.3.2 Steel fiber reinforced concrete

<table>
<thead>
<tr>
<th>Mix Components</th>
<th>Weight in pounds per cubic yard</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/8 Coarse Aggregate (SSD)</td>
<td>1735</td>
</tr>
<tr>
<td>Fine Aggregate (SSD)</td>
<td>1400</td>
</tr>
<tr>
<td>Water</td>
<td>333.4</td>
</tr>
<tr>
<td>Cement, Type I</td>
<td>846</td>
</tr>
<tr>
<td>Water reducer Superplasticizer</td>
<td>15.86</td>
</tr>
<tr>
<td>1% Steel Fiber</td>
<td>132.3</td>
</tr>
<tr>
<td>W/C= 0.39 and Sand : Gravel= 0.55:0.45</td>
<td></td>
</tr>
</tbody>
</table>

² SSD means Saturated Surface Dry
3.7.3.3 Steel fiber/polymer modified concrete

<table>
<thead>
<tr>
<th>Mix Components</th>
<th>Weight in pounds per cubic yard</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/8 Coarse Aggregate (SSD)</td>
<td>1735</td>
</tr>
<tr>
<td>Fine Aggregate (SSD)</td>
<td>1400</td>
</tr>
<tr>
<td>Water</td>
<td>304.6</td>
</tr>
<tr>
<td>Cement, Type I</td>
<td>658</td>
</tr>
<tr>
<td>Styrene Butadiene Rubber</td>
<td>84.6</td>
</tr>
<tr>
<td>1% Steel Fiber</td>
<td>132.3</td>
</tr>
</tbody>
</table>

W/C= 0.36 and Sand : Gravel = 0.55:0.45

3.7.4 Compressive, tensile, and flexure strengths behavior for trial mix #2

Several specimens of plain concrete, SFRC, and SFPMC with 1% steel fiber, and both fiber and 5% solids of polymer were prepared and tested. Results are shown in Table 3.4.

Trial mix #2 improved the compressive strength of plain concrete to 8076 psi, tensile strength to 796 psi, and flexural strength to 727 psi. However, the compressive strength values of SFRC and SFPMC decreased with values of 7016 psi and 6173 psi with percentages of 13% and 23.6 % respectively. The tensile strength values were

Table 3.4 Compressive, tensile, and flexural strengths for trial concrete mix #2

<table>
<thead>
<tr>
<th>Concrete Mix</th>
<th>fc (psi)</th>
<th>ft (psi)</th>
<th>ft (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plain</td>
<td>8076</td>
<td>796</td>
<td>727</td>
</tr>
<tr>
<td>1% Steel Fiber</td>
<td>7016</td>
<td>812</td>
<td>930</td>
</tr>
<tr>
<td>1% Steel Fiber + 5% solids of Polymer</td>
<td>6173</td>
<td>819</td>
<td>862</td>
</tr>
</tbody>
</table>
improved to 812 psi and 819 with percentage increases of 2% and 2.9% for SFRC and SFPMC respectively. The flexural strength values for both SFRC and SFPMC were 930 psi and 862 psi with increase of 28% and 18.6 respectively.

3.7.5 Stress-Strain Relationship for Trial Mix #2

3.7.5.1 Compressive stress-strain behavior

Figure 3.20 shows the stress-strain trend for trial mix #2 under compression loading. The ultimate strength was higher than for trial mix #1, however the ultimate strain decreased compared to trial mix #1. For example the SFPMC showed more toughness than plain and SFRC mixes, the ultimate strain was 0.003 while it was 0.0065 for trial mix #1. The ultimate stresses for both SFRC and SFPMC mixes were less than that of plain concrete for trial mix #2 because this mix had less coarse aggregate than trial mix #1, the amount of coarse aggregate has been shown to influence the elasticity of the material.

3.7.5.2 Tensile stress-strain behavior

Figure 3.21 shows the stress-strain relation for trial mix #2 under splitting tensile loading. The ultimate strain value for SFPMC was about 0.002 with maximum toughness compared to plain and SFRC mixes. The ultimate strain values for both trial mix #1 and trial mix #2 were approximately similar, but the SFPMC for trial mix #2 gave higher toughness values.

3.7.5.3 Flexural stress-strain behavior

The beams which were tested under flexural loading for trial mix #1 had a 32 inch span length. Some beams with a shorter span of 15 inches for steel fiber reinforced
concrete and steel fiber/polymer modified concrete were tested under three point loading. In this case the shear span to depth ratio (a/d) was smaller than that of the longer beams and shear contribution took effect with the bending. The modulus of rupture for those beams was higher than those of longer beams by a factor of 1.2.

Figure 3.22 shows the stress-strain relation for trial mix #2 under flexural loading. Compared to trial mix #1, the ultimate strains for both SFRC and SFPMC were around 0.00022 which was similar to that of trial mix #1, but trial mix #2 gave higher toughness values.

3.7.6 Toughness and resilience

Coefficient of resilience is defined as the area under stress-strain curve for the linear stage only, while the toughness is defined as the area under the stress-strain curve up to ultimate values of stress and strain. Resilience and toughness were calculated for different mixes of trial mix #1 (NDOT mix) and trial mix #2 using Figures 3.17 to 3.22. Table 3.5 and Table 3.6 show the resilience and toughness values for trial mix #1 (NDOT mix) and trial mix #2 respectively. In this research toughness was defined as the area under the stress-strain diagram up to the ultimate values of stress and strain. The toughness for 5% solids of polymer and 1% steel fiber were 17, 1.15, and 0.136 inch-pound/cubic inch under compression, tension, and flexure loadings respectively for trial mix #1 (NDOT mix). The combination of 5% solids of polymer and 1% steel fiber gave the highest toughness value for trial mix #1 under different loading conditions. The toughness for 5% solids of polymer and 1% steel fiber were 12.15, 1.44, and 0.107 inch-pound/cubic inch under compression, tension, and flexure respectively for trial mix #2. The combination of 5% solids of polymer and 1% steel fiber gave the highest toughness
value under compression and tension among other mix combinations, while 1% steel fiber mix toughness was 0.122 under flexure.

3.7.7 Post crack analysis

Specimens containing steel fiber were loaded for several loading cycles after first cracking took place under compression, tension, and flexure to study the post crack behavior of SFRC and SFPMC.

3.7.7.1 Post crack analysis for compressive strength

Figure 3.23 shows the ultimate compressive strength values after one, two, and three loading cycles after the first crack developed. Table 3.7 shows the ratio of ultimate compression strength which could be resisted by steel fiber after first crack was developed. The steel fiber can resist about 78% of the compressive strength of SFRC or SFPMC after first crack, 54% after second loading cycle, and 43% after third loading cycle. SFRC or SFPMC specimens failed with warning and a structural member built of them remain in service at about 78% of its ultimate load until the deteriorated member replaced or repaired.

3.7.7.2 Post crack analysis for tension (splitting) strength

Figure 3.24 shows the ultimate tensile (splitting) strength values after one and two loading cycles after the first crack developed. Table 3.8 shows the ratio of ultimate tensile (splitting) strength which could be resisted by steel fiber after first crack was developed. The steel fiber can resist about 74% of the compressive strength of SFRC or SFPMC after first crack and 48% after second loading cycle. Structural members which
exposed to tensile stresses could withstand about 74% of ultimate tensile stresses after the first crack develops in the structural member.

3.7.7.3 Post crack analysis for flexural strength

The failure mode for polymer modified concrete beams was sudden and explosive

Table 3.5 Resilience and toughness for trial mix #1 (NDOT mix) in inch-pound/cubic inch

<table>
<thead>
<tr>
<th>Concrete Mix</th>
<th>Compression</th>
<th>Tension</th>
<th>Flexure</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Resilience</td>
<td>Toughness</td>
<td>Resilience</td>
</tr>
<tr>
<td>Plain</td>
<td>0.766</td>
<td>4.77</td>
<td>0.032</td>
</tr>
<tr>
<td>(2.5% solids of Polymer)</td>
<td>1.98</td>
<td>12.32</td>
<td>0.029</td>
</tr>
<tr>
<td>(5% solids of Polymer)</td>
<td>0.2</td>
<td>13.51</td>
<td>0.056</td>
</tr>
<tr>
<td>(7.5% solids of Polymer)</td>
<td>0.29</td>
<td>7.8</td>
<td>0.043</td>
</tr>
<tr>
<td>1% Steel Fiber + 2.5% solids of Polymer</td>
<td>1.8</td>
<td>11.3</td>
<td>0.04</td>
</tr>
<tr>
<td>1% Steel Fiber + 5% solids of Polymer</td>
<td>0.25</td>
<td>17</td>
<td>0.038</td>
</tr>
<tr>
<td>1% Steel Fiber + 7.5% solids of Polymer</td>
<td>0.29</td>
<td>8.56</td>
<td>0.016</td>
</tr>
</tbody>
</table>

like a plain concrete failure mode (brittle failure) under bending. The steel fiber reinforced concrete and steel fiber/polymer modified concrete beams failed in a ductile failure mode.
Figure 3.25 shows the flexural strength ultimate values of concrete 4'' x 4'' x 34'' beams after one, two, and three loading cycles after the first crack developed. Table 3.9 shows the ratio of ultimate flexural strength that could be resisted by steel fiber after the first crack developed. The steel fiber can resist about 72% of the compressive strength of SFRC or SFPMC after first cracking, 61% after second loading cycle, and

Table 3.6 Resilience and toughness for trial mix #2 in inch-pound/cubic inch

<table>
<thead>
<tr>
<th>Concrete Mix</th>
<th>Compression Resilience</th>
<th>Tension Resilience</th>
<th>Flexure Resilience</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Plain</td>
<td>2.45</td>
<td>7.58</td>
<td>0.06</td>
</tr>
<tr>
<td>1% steel fiber</td>
<td>2.5</td>
<td>6.35</td>
<td>0.057</td>
</tr>
<tr>
<td>1% Steel Fiber + 5% solids of Polymer</td>
<td>1.46</td>
<td>12.15</td>
<td>0.025</td>
</tr>
</tbody>
</table>

Table 3.7 Ratio between compressive strength after cracking to ultimate compressive strength

<table>
<thead>
<tr>
<th>Mix type</th>
<th>Before Crack</th>
<th>Post Crack #1</th>
<th>Post Crack #2</th>
<th>Post Crack #3</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.5% solids polymer+1% steel fiber (Trial Mix#1)</td>
<td>1</td>
<td>0.81</td>
<td>0.62</td>
<td>0.45</td>
</tr>
<tr>
<td>5% solids polymer+1% steel fiber (Trial Mix#1)</td>
<td>1</td>
<td>0.79</td>
<td>0.43</td>
<td>0.41</td>
</tr>
<tr>
<td>5% solids polymer+1% steel fiber (Trial Mix#2)</td>
<td>1</td>
<td>0.79</td>
<td>0.51</td>
<td>0.37</td>
</tr>
<tr>
<td>1% Steel fiber (Trial Mix #2)</td>
<td>1</td>
<td>0.74</td>
<td>0.58</td>
<td>0.47</td>
</tr>
<tr>
<td>Average</td>
<td>1</td>
<td>0.78</td>
<td>0.54</td>
<td>0.43</td>
</tr>
</tbody>
</table>
52% after third loading cycle. The beams tested deflected about 1.25 inch under loading before complete failure with crack width about 7/8 inches. The failure behavior was ductile like homogeneous materials (steel for example) because steel fiber bridged cracks and more energy is consumed in de-bonding and stretching of the fibers before complete failure. In some cases the crack widths and beam deflection values were 3/4 inches and 1.5 inches respectively. Structural member exposed to bending stresses could stand with about 72% of ultimate tensile stresses after first crack. From the foregoing discussion, the presence of steel fiber provides ductility and toughness to structural members which tend to behave like homogeneous material (steel for example). Also, the failure pattern is ductile and accompanied with warning. Moreover, any structural

<table>
<thead>
<tr>
<th>Mix type</th>
<th>Before Crack</th>
<th>Post Crack #1</th>
<th>Post Crack #2</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.5% solids polymer+1% steel fiber</td>
<td>1</td>
<td>0.72</td>
<td>0.46</td>
</tr>
<tr>
<td>(Trial Mix#1)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5% solids polymer+1% steel fiber</td>
<td>1</td>
<td>0.78</td>
<td>0.52</td>
</tr>
<tr>
<td>(Trial Mix#1)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7.5% solids polymer+1% steel fiber</td>
<td></td>
<td>0.65</td>
<td>0.49</td>
</tr>
<tr>
<td>(Trial Mix#1)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5% solids polymer+1% steel fiber</td>
<td></td>
<td>0.78</td>
<td>0.53</td>
</tr>
<tr>
<td>(Trial Mix#2)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1% Steel fiber (Trial Mix #2)</td>
<td>1</td>
<td>0.76</td>
<td>0.42</td>
</tr>
<tr>
<td>Average</td>
<td>1</td>
<td>0.74</td>
<td>0.48</td>
</tr>
</tbody>
</table>
member that contained steel fiber and/or polymer, under any combination of loading, could carry about 70% of the ultimate loading after first crack failure, and hence could be repaired with minimum scaffolding support.

Table 3.9 Ratio between modulus of rupture after cracking to ultimate modulus of rupture

<table>
<thead>
<tr>
<th>Mix type</th>
<th>Before Crack</th>
<th>Post Crack #1</th>
<th>Post Crack #2</th>
<th>Post Crack #3</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.5% solids polymer+1% steel fiber (Trial Mix#1)</td>
<td>1</td>
<td>0.66</td>
<td>0.60</td>
<td>0.47</td>
</tr>
<tr>
<td>5% solids polymer+1% steel fiber (Trial Mix#1)</td>
<td>1</td>
<td>0.69</td>
<td>0.58</td>
<td>0.47</td>
</tr>
<tr>
<td>7.5% solids polymer+1% steel fiber (Trial Mix#1)</td>
<td>1</td>
<td>0.72</td>
<td>0.62</td>
<td>0.54</td>
</tr>
<tr>
<td>5% solids polymer+1% steel fiber (Trial Mix#2)</td>
<td>1</td>
<td>0.80</td>
<td>0.73</td>
<td>0.62</td>
</tr>
<tr>
<td>1% Steel fiber (Trial Mix #2)</td>
<td>1</td>
<td>0.72</td>
<td>0.54</td>
<td>0.48</td>
</tr>
<tr>
<td>Average</td>
<td>1</td>
<td>0.72</td>
<td>0.61</td>
<td>0.52</td>
</tr>
</tbody>
</table>

3.8 Shear Strength of Short Deep Beams

Since the 4" x 4" x 34" beams discussed earlier failed in a flexure mode, it was necessary to reduce the beam span and the (a/d) ratio substantially in order to produce shear failure mode. Some of the specimens were tested under four point loads to determine the shear strength. The specimens tested had span lengths of 6 inches and were loaded under four point loads spaced 3 inches apart with an a/d ratio of 1.5/4=3/8. The failure mode was approximately 45 degree cracking from the point of loading.
to the beam support as shown in Figure 3.26. Figure 3.27 and Figure 3.28 show the shear strength for both trial mix #1 and trial mix #2. The SFPMC of 5% polymer and 1% steel fiber for trial mix #1 gave a shear strength value of 476 psi (Table 3.10), which was the maximum value of all the mixes. Table 3.11 shows the shear strength for trial mix #2 for SFRC and SFPMC. Again the 5% polymer and 1% steel fiber gave the highest shear strength value of 652 psi. This proved that the 5% of polymer solids and 1% steel fiber was the best combination of concrete mixes investigated.

As shown from Table 3.3 adding 5% solids to the concrete mix reduced the compression strength by 7.4%, thus 5% solids of polymer modified concrete is in effect a plain concrete with some additive to limit crack growth. The polymer modified concrete (PMC) was considered as plain concrete with an admixture of 5% solids of latex which has a chemical effect and a minor mechanical properties effect on concrete. The 5% solids of

Table 3.10 Shear strength capacities for trial mix #1

<table>
<thead>
<tr>
<th>Mix type</th>
<th>Shear Strength (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.5% solids polymer</td>
<td>317</td>
</tr>
<tr>
<td>3.75% solids polymer</td>
<td>224.5</td>
</tr>
<tr>
<td>2.5% solids polymer+1% steel fiber</td>
<td>341</td>
</tr>
<tr>
<td>5% solids polymer+1% steel fiber</td>
<td>476</td>
</tr>
<tr>
<td>7.5% solids polymer+1% steel fiber</td>
<td>303.4</td>
</tr>
</tbody>
</table>
polymer (Styrene Butadiene Rubber) added took the place of water reducing superplasticizer, therefore there was no need to construct pull-out specimens of purely plain concrete. The pull-out specimens were made of three distinct concrete mixes of polymer modified concrete (PMC), steel fiber reinforced concrete (SFRC), and steel fiber/polymer modified concrete (SFPMC).

3.9 Development Length of Reinforcing Bars and Bond Strength of Concrete

3.9.1 Basic development length in tension

The steel bar embedded in concrete was subjected to a tensile force $T$, this force is resisted by the bond stress between the steel bar and the concrete. The development length (Nawy, 1996) can be written as:

$$L_d = \frac{f \cdot d_b}{4U_u}$$ (3.1)

<table>
<thead>
<tr>
<th>Mix type</th>
<th>Shear Strength (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1% steel fiber</td>
<td>412</td>
</tr>
<tr>
<td>5% solids polymer+1% steel fiber</td>
<td>652</td>
</tr>
</tbody>
</table>

Table 3.11 Shear strength capacities for trial mix#2

where:

$L_d = \text{the minimum permissible anchorage length and is called development length.}$

$d_b = \text{steel bar diameter.}$
$f_y = \text{yield strength of the bar.}$

$U_u = \text{ultimate average bond stress.}$

According to ACI Code, the development length in tension for bars which have diameter less than #6 can be calculated as follows:

$$l_d = \frac{a\beta\lambda f'_t d_b}{25\sqrt{f'_c}}$$

Where:

$\alpha = 1$ for normal concrete,

$\beta = 1$ for uncoated steel bars,

and

$\lambda = 1$ for non-horizontal bars.

Comparing the two previous equations, the following equation could be written:

$$U = \frac{25\sqrt{f'_c}}{4}$$

Considering that the development length calculated from the code formula is not at the ultimate bond strength level, since ACI code formula includes factor of safety.

Since the compressive strengths of PMC, SFRC, SFPMC were 5863, 5842, and 5932 psi respectively, the calculated bond strengths using the ACI formula were 478.6, 477.7, and 481.4 psi for PMC, SFRC, SFPMC as shown in Table 3.10.

3.9.2 Pull-out test results

The surface bond stress between steel and concrete calculated as follows:

$$u_b = \frac{P_b}{\pi d l}$$
where:

\[ u_s = \text{average surface bond stress between concrete and steel} \]

\[ d = \text{steel bar diameter} \]

\[ l = \text{steel bar embedded length} \]

Table 3.12 shows the compression strength, the ultimate pull-out load, measured bond strength, and calculated bond strength using the ACI formula for PMC, SFRC, and SFPMC. For polymer modified concrete (PMC) specimens, the 3/8 rebar was pulled out (Figure 3.29) at an average ultimate tension load of 8146 pound and average bond strength of 1729 psi. The 3/8 inch re-bar failed for all the four steel fiber reinforced concrete (SFRC) and steel fiber/polymer modified concrete (SFPMC) specimens at ultimate load values of 10005 and 10017 pounds respectively.

The bond strength of the SFRC and FPMC concrete mixes were stronger than the ultimate strength of the steel re-bars, so the bar yielded, necked, and broke as shown in Figure 3.30. Since the steel re-bar yielded, necked and broke, the ultimate and yield strength for the reinforcing bar was calculated as follows:

\[ F_u = \frac{T_u}{A_s} \]

where:

\[ F_u = \text{the ultimate tensile strength of the re-bar} \]

\[ A_s = \text{rebar cross-sectional area} \]

The ultimate tensile stress of the re-bar was 90741 psi. For steel with 90 ksi ultimate strength, the yield strength is 70 ksi (Salmon and Johnson, 1996).
The ultimate pull-out load values for SFRC and SFPMC were higher than those two recorded values; however, if those two values were considered as ultimate pull-out values, the associated bond stress values are 2123 and 2126 psi respectively. The average tested bond strengths of polymer modified concrete, steel fiber reinforced concrete, and steel fiber/polymer modified concrete were higher by factors of 3.61, 4.44, and 4.42 than the calculated average bond stress given by the ACI code. This proves that adding steel fiber and steel fiber and polymer substantially increases the ultimate average bond strength of regular concrete.

3.10 Conclusion

Polymer modified concrete is usually used in bridge overlays to reduce cracks and enhance impermeability. The percentage of solid polymer used for this purpose is

Table 3.12 Ultimate load and bond stress for different concrete mixes

<table>
<thead>
<tr>
<th>Mix Type</th>
<th>Compression Strength (psi)</th>
<th>Ultimate Load (Pound)</th>
<th>Measured Bond Strength (psi)</th>
<th>Bond strength (psi) (ACI Code) $U_u = \frac{25\sqrt{f_c}}{4}$</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>PMC</td>
<td>5863</td>
<td>8146</td>
<td>1729</td>
<td>478.6</td>
<td>slip</td>
</tr>
<tr>
<td>SFRC</td>
<td>5842</td>
<td>10005</td>
<td>Larger than 2123</td>
<td>477.7</td>
<td>3/8 inch re-bar failed</td>
</tr>
<tr>
<td>SFPMC</td>
<td>5932</td>
<td>10017</td>
<td>Larger than 2126</td>
<td>481.4</td>
<td>3/8 inch re-bar failed</td>
</tr>
</tbody>
</table>
around 32% of cement weight. Adding 1% steel fiber by volume and 5% solids of polymers to a concrete mix improves the tensile, flexural, and shear capacities of the mix. Such a mix combination also improves the ductility of the material and converts the behavior of concrete from brittle to ductile. Also, the toughness (area under stress–strain curve) for this mix was the best compared to the other mixes. The mix could carry about 70% of ultimate loads and stresses after the initial cracking.

A third trial mix prepared and tested for compressive and tensile strengths. This trial mix is similar to trial mix #2 with the coarse aggregate to sand ratio reversed (0.55:0.45) to provide increased strength to concrete.

Table 3.13 Compressive and tensile strengths for PMC, SFRC, and SFPMC for trial mix #3

<table>
<thead>
<tr>
<th>Concrete Mix</th>
<th>fc(Psi)</th>
<th>ft(psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PMC</td>
<td>5863</td>
<td>638</td>
</tr>
<tr>
<td>SFRC</td>
<td>5842</td>
<td>773</td>
</tr>
<tr>
<td>SFPMC</td>
<td>5932</td>
<td>745</td>
</tr>
</tbody>
</table>

Table 3.13 shows the compressive and tensile (splitting) cylinders strengths for PMC, SFRC, and SFPMC for trial mix #3.

Slender beams (3”x9”), pure torsion specimens (3”x3”), and half I-beams (3”x12”) are prepared using trial mix #3. These specimens will be introduced and discussed in subsequent chapters.
Figure 3.1 UT wavy steel fibers

Figure 3.2 Styrene Butadiene Rubber (Modifier A), Dow Chemical Company
Figure 3.3 Nevada Ready Mix (NRM) and Nevada Department of Transportation (NDOT) coarse aggregate gradation

Figure 3.4 Coarse aggregate sizes
Figure 3.5 Nevada Ready Mix (NRM) and Nevada Department of Transportation (NDOT) fine aggregate gradation.

Figure 3.6 Two cubic foot concrete mixer.
Figure 3.7 Compression test and strain gage configuration
Figure 3.8 Splitting test and strain gage configuration
Figure 3.9 Flexure test and strain gage configuration
Figure 3.10 S-shape steel beam for Tinius Olsen machine calibration

Figure 3.11 Tinius Olsen machine calibration chart
PULL-OUT TEST

Figure 3.12 Pull-Out test and strain gage configuration
Figure 3.13 Rebar set-up for pull-out specimen

Figure 3.14 Pull-out specimen concrete pouring
Figure 3.15 Pull-out test using MTS machine

Figure 3.16 Micro-structural crack development at the interface (Michigan State University, 1999)
Figure 3.17 Stress-strain relationship under compression loading for trial mix #1

Figure 3.18 Stress-strain relationship under tension (splitting) loading for trial mix #1
Figure 3.19 Stress-strain relationship under flexural loading for trial mix #1

Figure 3.20 Stress-strain relationship under compression loading for trial mix #2
Figure 3.21 Stress-strain relationship under tension (splitting) loading for trial mix #2

Figure 3.22 Stress-strain relationship under flexural loading for trial mix #2
Figure 3.23 Post crack compressive strength values

Figure 3.24 Post crack tensile strength values

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.
Figure 3.25 Post crack flexural strength values

Figure 3.26 Small deep beams shear failure pattern
Figure 3.27 Shear strength capacity for trial mix #1

Figure 3.28 Shear strength capacity for trial mix #2
Figure 3.29 #3 Re-bar pull-out test for polymer modified concrete (re-bar pulled-out)

Figure 3.30 #3 Re-bar pull-out test for steel fiber reinforced concrete and steel fiber/polymer modified concrete (re-bar failed)
CHAPTER 4

BEHAVIOR OF CONCRETE SLENDER BEAMS
IN SHEAR AND FLEXURE

4.1. Introduction

Research was urgently needed as suggested by the Precast/Prestressed Concrete Institute (PCI) to study the behavior of slender beams having aspect ratios more than two. The application of slender beams may be as spandrel beams connected to slabs, or in highway bridges as rectangular or half I-beams where they may be exposed to eccentric wind and seismic forces. In most cases, these beams are exposed to combined loading conditions of bending, shear, and torsion.

4.1.1 Scope of research

In this chapter, the results of experimental research performed on slender beams with a depth to width ratio of three and length to depth ratio of 3.33 are shown. The beams were tested under four point loads with a shear span to depth ratio of about 1.1, which classify the beams as short and slender beams.

4.1.2 Materials

The beams were made of three distinct concrete mixes of polymer modified concrete (PMC), steel fiber reinforced concrete (SFRC), and steel fiber/polymer modified concrete (SFPMC). The polymer modified concrete (PMC) was considered as plain concrete with
an admixture of 5% solids of latex which has a chemical effect and a minor mechanical properties effect on concrete, as discussed in chapter 3. The 5% solids of polymer (Styrene Butadiene Rubber) added took the place of water reducing superplasticizer. Therefore, there was no need to construct beam specimens of purely plain concrete. Trial concrete mix# 3, which was discussed in chapter 3 is used to make slender beams with aspect ratio (beam depth/beam width) of three and tested under shear and flexure loading. The concrete mixes include polymer modified concrete (PMC) with 5% solids of polymer, steel fiber reinforced concrete (SFRC) with 1% fiber volume fraction, and steel fiber/polymer modified concrete (SFPMC) with 1% steel fiber volume fraction and 5% solids of polymer.

4.2 Beam Classifications

4.2.1 Deep beams

The shear span (a) is the distance between the beams support and the point of applied load, while the shear span to depth (a/d) ratio is highly influential factor in establishing shear strength and mode of failure. The beam is considered as deep beam when a/d<1 at which the shear stress has a predominant effect at which the beam tend to behave like tied-arch after cracking occurs wherein the load in carried by direct compression around the arch area and by direct tension in the longitudinal steel.

4.2.2 Short beams

For 1< a/d ≤2.5, the beams are considered as short beams at which shear strength exceeds the inclined cracking capacity. After the flexure-shear crack develops, the crack extends into the compression zone as the load increases. Failure is either shear-tension
failure, this failure mode is by anchorage failure at tension reinforcement, or shear-crushing failure, at which it happens by crushing near the compression face.

4.2.3 Intermediate beams

For $2.5 < a/d \leq 6$, the beams are intermediate length beams where flexural cracks are first to form, followed by inclined flexure-shear cracks.

4.2.4 Long beams

Long beams are those which have $a/d > 6$ where the failure starts with yielding of the tension reinforcement and ends by crushing of the concrete at the section of maximum bending moment. Before complete failure, slightly inclined cracks between the support and the section of maximum bending moment may be present (Wang and Salmon, 1998).

4.3 Literature Survey

The following literature review concentrates on beam shear and flexural behavior.

4.3.1 Steel fiber reinforced concrete beams

Batson et. al. (1972) tested $4'' \times 6'' \times 78''$ beams with same flexural reinforcing steel ratio, and steel fiber volume fractions ($V_f$) varied from 0.22-2.66 percent. Those beams were tested under four point loading to determine the effectiveness of steel fibers as web reinforcement. The test program was conducted in two phases: the first phase was at $a/d$ ratio of 4.8 and variable fiber content. The second phase was at different $a/d$ ratios and different steel fiber ratios. The boundary between the shear and the moment failure conditions occurs at a spacing of fiber of 0.3'', which is necessary for crack arrest mechanism to be effective. Also, they found that beams fail in shear for $a/d < 3$ and replacement of stirrups by steel fibers provided effective shear reinforcement.
Muhidian and Regan (1977) tested twenty-five I-beams under central concentrated load. Three beams without stirrups, four beams with stirrups, and eighteen beams with steel fiber were considered. The beam sections were 30 mm (1.2 inch), 50 mm (2 inch), and 70 mm (2.8 inch) web widths, 50 mm (2 inch) and 100 mm (4 inch) flange depths, and 300 mm (12 inch) and 400 mm (16 inch) flange breadths. The percentages of steel fiber were 0-3% by volume and a/d were 3.24 and 4.68. The specimens without shear reinforcement failed after the appearance of web shear cracks, while the beams with stirrups and those with steel fiber sustained higher loads. The shear stress was 4.3 N/mm² (626 psi) with steel fiber and 2.04 N/mm² (297 psi) without steel fiber developing similar patterns of cracks. The dominant web cracks were slightly below 45° to the horizontal in the vicinity of the load. The failure of fiber-reinforced beams involved widening of one or more shear cracks, which had an inclination a little flatter than 45°.

Mansur and Paramasivam (1985) studied the behavior and strength of steel fiber reinforced concrete beams under combined loading of torsion, bending, and shear. Tests were conducted on thirty-three rectangular beams under different combinations of torsion (T), bending (M) and shear (V). Tests were conducted on 33-4"x6" rectangular beams different lengths of 4'-7" and 6'-7". The steel fiber volume fraction was 0.75% and the beams were divided to three groups: 1) beams to fail under combined torsion and bending 2) beams to fail under combined torsion, bending, and shear, and 3) beams to fail under combined torsion and shear. It was observed that the torque-twist relationship was linear up to about 70% of the ultimate torque, afterwards the curves deviate from linearity because of microcracking development. Similar behavior was observed for load-
deflection relationship. The failure pattern of the beams were divided into two modes: 1) mode 1 at which bending is predominant over torsion, failed by the formation of compression zone on the top of the vertical face of the beam. 2) failure governs by torsion when torsion to moment ratio is high T/M > 1.98, the compression zone appeared on one of the vertical faces of the beam, the tension cracks on the opposite face were inclined at an angle of 45°. They suggested that for mode 1, failure unaffected by the amount of transverse shear, while for beams failing in mode 2 were independent of the magnitude of applied bending moment. Finally, the interaction was torsion-bending for mode 1, while the interaction for mode 2 was torsion-shear.

Niyogi and Dwarakanathan (1985) studied the effect of steel fiber on concrete beams under combined actions of moment and shear. They tested thirty plain and reinforced concrete beams under combined moment and shear under four points loading, for different shear span to depth (a/d) ratios. The beams were tested under pure moment (at the zero shear section between loads). Fiber volume fraction varied from 0-3% for each mix and the aspect ratio (l/d_f) of fibers was kept constant at 50. Test beams were 4\" x 6\" x 78\" and were tested over a simple span of 72\". There was a small increase in post-cracking strength of the beams under the combined bending and shear. The ratio of cracking load to ultimate load varied approximately between 0.95-1 for beams with V_f=1% and 0.8-0.9 with V_f=3%. Fiber reinforced beams under combined bending and shear exhibited two failure modes, one of typical shear and the other of bending. For small shear span (a/d=2), failure was characterized by formation of diagonal cracking, while for large shear span (a/d=4.5 and 6); failure was characterized by the formation of vertical flexural cracks originating from the tension face of the beam. They found also
that beams failing under combined bending and shear with larger fiber volume concentration ($V_f=3\%$) gave more significant shear strength increase than under pure flexure. The shear strength of the beam decreases as $a/d$ increases due to the contribution of bending failure mode.

Swamy and Bahia (1985) evaluated the effectiveness of steel fiber on shear strength and shear deformations. Tests were conducted on nine T-beams and two rectangular beams. All beams were 11.1 feet long, and simply supported on an effective span of 9.1 feet under a constant moment-shear ratio of 4.5. Fibers were added in different ratios from 0-1.2 %. It was found that the steel fibers reduced shear deformation at all stages of loading, fibers controlled the cracking and displacement in concrete in the dowel zone. In T-beams and rectangular beams with 1.95 % tension steel, 0.8 % fiber volume, the beams exhibited flexural failure. In T-beams with 2-4% of tension steel reinforcement and fiber volume of 0.8-1.2%, the shear strength was increased by 80%. The T-beams carried about 28 % higher load than that of the rectangular beams.

Mansur et. al. (1986) studied the effectiveness of steel fibers as web reinforcement in concrete beams containing longitudinal bar reinforcement only. They tested simply supported beams under two symmetrical point loads. The beams were of rectangular section ($6'' x 9''$) with two different lengths of $6'7''$ and $8'2''$. The fibers used were $1.25'' x 0.02''$ with fiber volume fractions ($V_f$) of 0, 0.5, 0.75, and 1% and the $a/d$ ratio was varied from 2 to 4.4 at increment of 0.8. It was found that all beams without fibers failed in shear for all $a/d$ ratios (2-4.4), while for beams containing short fibers the failure mode changed from shear to flexure for higher values of $a/d$. The presence of steel fiber increased the shear resistance more than bending strength of the beam. The critical $a/d$
ratio is the ratio at which failure mode changes from shear to flexure. It was found that the critical a/d ratio required inducing shear failure decreased with increasing fiber content and shear strength increased with increasing fiber content and lower a/d ratio since shear failure mode is in effect predominantly.

Sharma (1988) studied the effect of steel fiber on shear strength of concrete beams. Seven beams of 6"x12" were tested; three of them were singly reinforced without steel fibers, while the other four beams were doubly reinforced with steel fibers. The steel fiber used had 50 mm (2 inch) length and 0.6 mm (0.024 inch) diameter with fiber volume fraction \(V_f\) of 1%. All beams were tested under four point loads. The beams without web reinforcement failed suddenly after the appearance of diagonal cracks, while the fiber reinforced beams exhibited appreciable ductility and comparatively large post cracking strength. Steel fiber is effective in increasing the shear strength of concrete and has more ductility and significant amount of energy absorption than normal reinforced concrete beams.

Narayanan and Darwish (1988) studied shear strength in beams containing fly ash and steel fiber in the concrete mix. They tested 24 beams having cross section of 5.91"x3.35" and were cast in two lengths (35.4" and 45.7"). 20% by weight of cement was replaced by pulverized fly ash (PFA) in the mix. The fiber used was 0.016"x1.575" with fiber volume fractions \(V_f\) of 0.5% and 1% and the main tension steel ratio \(\rho = 2\% \text{ or } 5.72\%\). The beams were tested under four points loading, the shorter beams had shear/span ratio \(a/d\) of 2 and longer beams had \(a/d = 3\). They observed that the largest amount of cracks for beams failing in shear occurred along the compression path from loading point to the support. The fracture process consisted of progressive debonding of the fibers, which
show crack propagation. Final failure was due to unstable crack propagation with fibers being pulled out. Also, replacement of half the volume of stirrups by fibers does not significantly decrease the shear strength of the beam. First crack strength was higher for the beams containing fibers compared with beams provided with stirrups only. The substitution of 20% of cement by PFA increased the cubic compressive strength of concrete by up to 10%; also concrete mix workability was increased.

EL-Niema (1991) studied the role of steel fibers in improving the first-crack load and ultimate load of reinforced concrete beams under shear. Nine beams reinforced with longitudinal and transverse reinforcement had steel fibers of varying amounts and aspect ratios and one beam with the same reinforcement and had no fibers. The steel fibers used had aspect ratios of 63.83, 95.75, and 127.7 and added at fiber volume fractions of 0.4, 0.7, and 1%. The beams had cross sections of 100 mm (4 inch) x 200 mm (8 inch) with span of 1800 mm (70 inch) and were tested in a flexural mode under four point loading at a/d ratio of 3.86. Six standard cylinders were tested for crushing strength at 7 and 28 days. It was found that the percentage increase in cracking load of fibrous beam compared with a beam without fibers is as large as 65 percent, especially when the aspect ratio is high ($l/d_f=127.7$). Also, the average ultimate load is increased with a maximum value of 22.5 percent, the maximum increase for compression strength was 13 percent, especially with high aspect ratios of fibers. Longitudinal and transverse steel reinforcement strains decreased with the increase of fiber aspect ratio ($l/d_f$) because part of the load is transferred by the fibers across the cracks and resisted by debonding and stretching. This shows that the longer the fibers, the greater the share of load carried by
them. Beam deflection is smaller for beams with fibers of higher aspect ratio and large volume percentages of fibers, for a given load.

Tan et al. (1993) developed an analytical model for the shear capacity of steel fiber reinforced concrete (SFRC) beams. The model was based on principle of stress-strain relations for the cracked SFRC. Six I-beams with aspect ratio (depth/width) of 4.75 and provided with longitudinal reinforcement of 6#4 in the bottom flange were tested. Three beams had fiber volume ratio of 0, 0.5, and 0.75% and the other three had 1%. The beams were tested under four point loading with shear span (a/d) of 1.5, 2 and 2.5. The shear strength of concrete beams increased by adding small amount of steel fiber, also at a given load, the steel longitudinal strains were less for SFRC beams when compared to RC beams especially after diagonal cracking of the web. The analytical formula for SFRC was able to model the shear behavior of SFRC and the ultimate loads for SFRC beams are well predicted using the analytical procedure.

Adebar et al. (1997) tested eleven large-scale rectangular beams without stirrups. The beams had dimensions of 150 mm (6 inch) x 610 mm (24 inch) and had 6 (1/4 inch) to 20 mm (#7 re-bar) diameter bars as flexural tension reinforcement and an identical amount of compression reinforcement. In three of the specimens, axial tension was applied in addition to transverse shear and the associated bending moments. The fiber used in this study was hooked-ended steel fiber with two lengths of 30mm (1.2 inch) and 50 mm (2 inch). All fibers had a diameter of 0.5 mm (0.02 inch); therefore the aspect ratio was 60 for the shorter fiber and 100 for the longer fiber. The fiber volume fraction was 0.4, 0.6, 0.75 and 1.5% for short fibers, and 0.4 and 0.6% for long fibers. The beams were loaded in a special beam element tester developed by the first author.
Hydraulic actuators were controlled so that the bending moments applied to the two ends of the specimens were equal in magnitude and in the same direction. An opposite force couple provided by the two transverse rigid links satisfied moment equilibrium. The forces in the transverse rigid links were equal to the uniform shear force applied to the specimens. The bending moment varied linearly along the specimen and was zero at midspan. Then, the bending moment at any point along the beam was equal to shear force times the distance from midspan. It was found that there was a reduction in compressive strength with the increase of fiber amount and size; this is due to poor consolidation of fresh concrete in cylinders. Also, the beams containing 0.75% steel fiber resisted an 85% larger shear force than the specimen without fiber, and the beams containing 1.5% steel fiber resisted 117% larger shear force than beams without steel fibers and were more ductile. An equal volume of larger fiber resulted in about the same shear strength increase but considerably more ductile than shorter fiber.

4.3.2 Polymer concrete beams

Polymer concrete is plain concrete in which polymer is substituted for cement as a binder to aggregates. The following literature review focuses on polymer concrete.

Rebiz et. al. (1993) modified the ACI-ASCE shear strength more detailed equation to accommodate the reinforced polymer concrete in which polyester was used as a binder instead of cement. 4”x 6” beams were tested under four points loading with different shear span to depth ratio (a/d) and variable reinforcement ratios (ρ). The ACI-ASCE approach does not yield very good results because of poor correlation between experimental and theoretical values. A new approach based on statistical regression analysis and dimensional analysis was presented. This approach has taken into account...
the difference in behavior between short beams and long beams using interpolation function. The use of interpolation function proved to be an excellent choice of the prediction of ultimate shear for polymer concrete because of excellent correlation between experimental and theoretical values.

Rebiz et. al. (1993) studied the shear behavior of reinforced polymer (PC) using unsaturated polyester resins binders based on recycled plastics. Twenty-five beams of different shear span to depth (a/d) ratios (short beams a/d<2.5 and long beams a/d>2.5), steel reinforcement ratio (ρ), and with and without steel fiber ratios were tested under four point loading. Most of the beams fail in a shear-tension failure: other types of failure were shear-compression, diagonal tension, arch-rib with crushing along the diagonal strut, and arch-rib due to crushing of the arch crown. It was found that an increase in a/d resulted in a decrease in the shear strength of polymer concrete, since flexural stresses increased and arch-rib action diminished. It was also observed that the difference between the shear strength at failure and the shear strength at first crack formation decreased with the increase in a/d because the stresses redistribute after the formation of diagonal crack since the smaller the a/d ratio for the beams the more the stresses transfer back to the support because distance between the point of loading and the support is short. The increase of reinforcement ratio (ρ) increased the ultimate shear strength while it does not affect the shear strength at the formation of the first diagonal crack because the steel dowel action took place after the formation of diagonal crack due to stress redistribution.
4.4 Slender Beams

3" x 9" x 34" concrete slender beams with aspect ratio (depth/width) of three were performed with different configurations. No. 4 steel reinforcing bars with diameter of 0.5 inch, cross section area of 0.2 square inch, and grade 60 deformed bars were used as longitudinal reinforcement and W4 wire bars with diameter of 0.225 inch, cross section area of 0.04 square inch, and grade 60 were used as transverse reinforcements (stirrups). A minimum of ½ inch concrete cover was used for these slender members according to ACI (American Concrete Institute) Code provision 7.7.1.

4.4.1 Slender beams without shear reinforcement (stirrups) and tension reinforcement:

Two polymer modified concrete beams, two steel fiber reinforced concrete beams, and two steel fiber/polymer modified concrete beams were cast without transverse reinforcement (stirrups), but with tension reinforcement of 2#4 rebars and reinforcement ratio of 1.1% at the tension face of the beams to force the beam to break in shear rather than flexure to determine the shear strength and shear failure pattern in the slender beams for the three mixes mentioned above as shown in Figure 4.1 (all figures are located at the end of this chapter).

4.4.2 Slender beams with shear reinforcement (stirrups) and single tension reinforcement:

Three beams, one beam from each mix, were made with single reinforcement of 2#4 rebars at the tension face of the beam and with a reinforcement ratio (ρ) of 1.1%. W4 wire shear reinforcements (stirrups) were distributed at d/2 increment, ACI maximum spacing requirements, (about 4 inches) along the beam length as shown in Figure 4.2.
4.4.3. Slender beams with shear (stirrups) reinforcement and double tension and compression reinforcement:

Three beams, one beam from each mix, were made with double reinforcement of 2#4 rebars at the tension face and 2#4 rebars at the compression face of the beam. W4 wire rebars transverse shear reinforcements (stirrups) were distributed at d/2 increment (about 4 inches spacing) along the beam length which represents the ACI maximum spacing requirements as shown in Figure 4.3.

4.5 Strain Gage Configurations

1/4 inch strain gages were installed at the mid-length of the 1#4 re-bar on the tension side and at the third length of the other #4 re-bar on the tension side (under loading line). These strain gages were considered for doubly reinforced beams only, because these beams had one re-bar at each corner of the beam cross section which meets the torsional reinforcements requirements according to ACI concrete code. Other gages were placed at the middle length of two stirrups under the loads at third distance of the beam span from each support. Also 3/4 inch gages were mounted on concrete with inclination of 45 degree to the beam centerline in the mid-distance of the shear span, the two gages were at right angle to each other to form a rosette, also a 1/2 inch strain gage was installed at the mid-distance of the shear span with 45 degree angle to the beam section centerline on the other side of the beam as shown in Figure 4.4 (appendix 4).

4.6 Testing Methodology

A Tinius Olsen Universal Testing Machine with ultimate capacity of 30,000 pounds
was used for testing the slender beams under four points loading. The beam span was 30 inches long and with shear span (a) of 9 inches as shown in Figure 4.5. The shear span to depth (a/d) ratio was 1.1 which falls in the short beam category, 1< a/d <2.5. (Wang and Salmon, 1998)). Two sets of readings were taken 1) readings due to loading under gravity, four point loading perpendicular to beam section’s strong axis until first crack occurs 2) readings after the occurrence of cracks in the beam due to the application of four point loads with eccentricity in the transverse direction of the beam and flexure around the weak axis of the beam (this procedure will be presented in chapter 6). A National Instrument Data Acquisition System with 24 channels was used to read the strain values using the LABVIEW 5.1 software.

4.7 Shear Capacity of Short Slender Concrete Beams Without Transverse (Stirrups) Reinforcements

Table 4.1 shows the shear strengths for slender beams without shear reinforcement (stirrups).

<table>
<thead>
<tr>
<th>Mix type</th>
<th>Ultimate shear capacity, $v_c$ (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PMC</td>
<td>580</td>
</tr>
<tr>
<td>SFRC</td>
<td>Larger than 606 (exceeded the machine capacity)</td>
</tr>
<tr>
<td>SFPMC</td>
<td>Larger than 606 (exceeded the machine capacity)</td>
</tr>
</tbody>
</table>

The shear capacity for polymer modified concrete was 580 psi. The shear cracks were inclined connecting the loading points to the support (Figure 4.6), this type of crack
is called shear-compression crack (Rebeiz, et. al., 1993) at which failure developed because of cracks extends to the compression zone of the beam but concrete did not crush like normal concrete. After the initiation of crack, the failure mode was sudden and explosive and the beam broke in two pieces as shown in Figure 4.7. For both steel fiber reinforced concrete and steel fiber/polymer modified concrete slender beams, the Tinius Olsen Testing Machine stopped at its ultimate capacity and the ultimate shear strength for those two beams is obviously higher than the calculated value of 606 psi for both of them. Steel fibers bridge the cracks after their initial formation allowing a partial transfer of shear force to the stirrups allowing an increase in the beams shear capacity. The shear crack started as inclined hairline crack from the point of loading to the support with inclination of 39.5 degrees as shown in Figure 4.8. The beam was loaded for several cycles of loading and unloading after it was loaded to the ultimate shear strength level in which the crack width increased until the steel fibers pulled-out with ductile failure mode as shown in Figure 4.9. The steel fiber/polymer modified concrete slender beams had a similar failure mode to steel fiber reinforced concrete slender beams.

4.8 Diagonal Tensile and Diagonal Compressive Strains for Short Slender Beams with Longitudinal and Transverse (Stirrups) Reinforcements

4.8.1 Polymer modified concrete beams

Figure 4.10 and Figure 4.11 show the diagonal shear crack at 45 degree inclination for singly reinforced and doubly reinforced polymer modified concrete respectively. Figure 4.13 shows the relationship between the maximum shear versus diagonal tensile and compressive strains measured from strain gages located at the centerline of the beam.
near the support (Figure 4.4) for singly and doubly reinforced polymer modified concrete slender beams. The diagonal tensile strain exhibited a linear trend for both singly and doubly reinforced slender beams up to shear value of 10 kips, the trend changed to non-linearity beyond this point because micro-cracks started to be developed in concrete. At initiation of the diagonal shear crack, the diagonal tensile shear strain value for singly reinforced slender beams was 0.000144 at a shear load of 14.5 kips, while the diagonal tensile shear strain for doubly reinforced slender beams was 0.000153 at the same shear level with 6% increase in diagonal tensile strain due to improved ductility of the doubly reinforced beams having a higher reinforcement ratio of 2.2 % rather than 1.1 % for singly reinforced beams. The diagonal compression strain value for singly reinforced slender beams was 0.000248 at shear load of 14.5 kips, while it was 0.000306 for doubly reinforced slender beams at the same shear level with 23 % increase in concrete diagonal compression.

4.8.2 Steel fiber reinforced concrete beams

Figure 4.14 shows the relationship between the maximum shear versus diagonal tensile and compressive strains measured from strain gages located at the centerline of the beam near the supports (Figure 4.4) for singly and doubly reinforced steel fiber reinforced concrete slender beams. The diagonal tension trend was linear until development of micro-cracks; afterwards the line slope increased up to shear load of 14.375 kips for singly and doubly reinforced slender beams. At initiation of the diagonal shear crack, the diagonal tensile shear strain was 0.0002 for singly reinforced slender beams, while the diagonal tension shear strain for doubly reinforced slender beams was 0.000237 at the same shear level with 18% increase in tensile strain due to improved
ductility of the doubly reinforced beams having a higher reinforcement ratio of 2.2 \% rather than 1.1 \% of singly reinforced beams. The diagonal compression strain value for singly reinforced slender beams was 0.000165, while it was 0.000205 for doubly reinforced slender beams at the same shear level with 24 \% increase in concrete diagonal compression. The diagonal tensile strains of singly and doubly reinforced beams increased by 39\% and 55\% respectively as compared to those of polymer modified concrete. Steel fiber increased beams ductility and energy absorption than polymer modified concrete beams (this agrees with Sharma, 1988).

4.8.3 Steel fiber/polymer modified concrete beams

Figure 4.15 shows the relationship between the maximum shear versus diagonal tensile and compressive strains measured from strain gages located at the centerline of the beam near the support (Figure 4.4) for singly and doubly reinforced steel fiber/polymer modified concrete slender beams at crack initiation. The diagonal tension trend was almost linear shear load of 14.375 kips for singly and doubly reinforced slender beams, which means that micro-cracks did not develop at early loading stages. At initiation of the diagonal shear crack, the diagonal tensile shear strain was 0.000205 for singly reinforced slender beams, while the diagonal tension shear strain for doubly reinforced slender beams was 0.000248 at the same shear level with 21\% increase in tensile strain due to improved ductility of the doubly reinforced beams having a higher reinforcement ratio of 2.2 \% rather than 1.1 \% of singly reinforced beams. The diagonal compressive strain value for singly reinforced slender beams was 0.000268, while it was 0.000318 for doubly reinforced slender beams at the same shear level. The diagonal tensile strains of singly and doubly reinforced beams increased by 42\% and 62\%
respectively as compared to those of polymer modified concrete. The presence of steel fiber increased the diagonal tensile strains by 41% for doubly reinforced beams as compared to those of polymer modified concrete slender beams.

4.9 Tension in Stirrups and Longitudinal Reinforcement of Short Slender Concrete Beams

4.9.1 Polymer modified concrete slender beams

Tensile strains in the selected stirrups and longitudinal re-bars were recorded in doubly reinforced slender beams to study the contribution of both concrete and reinforcement in resisting shear and flexure. Figure 4.16 shows the relationship between the tensile strains in both re-bars and stirrups versus the maximum shear for polymer modified concrete slender beams. The maximum stirrups and re-bars tensile strains were 0.00083 and 0.001695 respectively which corresponds to shear value of 14.5 kips; these strain values were lower than the steel yield strain of 0.00207. The tensile strains of longitudinal reinforcements were almost twice as high as in the stirrups.

4.9.2 Steel fiber reinforced concrete slender beams

Figure 4.17 shows the relationship between the tensile strain in both re-bars and stirrups versus maximum shear for steel fiber reinforced concrete slender beams. The maximum stirrups and re-bars tensile strains were 0.0002975 and 0.001103 respectively (Tan et. al., 1997 found that steel reinforcement tensile strain was 0.00135 at shear load of 22.5 kips for I-beams with aspect ratio of 4.75 and tensile flexure reinforcement of 6#4).
4.9.3 Steel fiber/polymer modified concrete slender beams

Figure 4.18 shows the relationship between the tensile strain in both re-bars and stirrups versus maximum shear for steel fiber/polymer modified concrete slender beams. The stirrups and rebars tensile strains were 0.00028 and 0.001072 respectively.

4.9.4 Steel fiber and polymer effect on stirrups and re-bars behavior for concrete slender beams

Figure 4.19 and Figure 4.20 show the relationship between the tensile strain in both rebars and stirrups versus the maximum shear for all beams. The tensile strains in stirrups and rebars for SFRC and SFPMC beams were almost identical and lower than those of polymer modified concrete beams. The presence of steel fiber and polymer reduced the tensile strains, produced from the application of four point loading on slender beams, in rebars and stirrups for both SFRC and SFPMC beams as compared with PMC beams. This observation proves that the steel fiber reinforced concrete and steel fiber/polymer modified concrete have a higher tensile and shear strengths than those of polymer modified concrete or plain concrete. The steel fibers are preventing the early development of the micro-cracks in the concrete thus delaying the transfer of flexure and shear loads to the reinforcement.

The tensile strains in the rebars of SFRC beams were 3.7 times higher than those in the stirrups, also the stirrups and rebars tensile strains of polymer modified concrete slender beams were almost 2.8 and 1.5 times of those of steel fiber reinforced slender beams because steel fiber increased tensile strength of concrete and part of the loading transferred by fibers (this agrees with EL-Niema, 1991 and Tan et. al., 1993). The tensile strains of the rebars of SFPMC beams were about 3.8 times higher than those of
stirrups, while the stirrups and rebars tensile strains of polymer modified concrete slender beams were almost 3 and 1.6 times of those of steel fiber/polymer modified concrete slender beams. The presence of steel fiber and polymer together reduced the strain contribution of reinforcement since the steel fiber increased tensile strength of concrete and part of the loading transferred by fibers, while polymers provided more ductility to concrete and reduced micro-crack growth.

Table 4.2 Calculated strains from cracked and un-cracked models analysis versus measured strain in the longitudinal reinforcements

<table>
<thead>
<tr>
<th></th>
<th>PMC</th>
<th>SFRC</th>
<th>SFPMC</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Cracked Model</td>
<td>Uncracked Model</td>
<td>Cracked Model</td>
</tr>
<tr>
<td>Singly reinforced beams</td>
<td>0.001558</td>
<td>0.000963</td>
<td>0.00155</td>
</tr>
<tr>
<td>Doubly reinforced beams</td>
<td>0.00123</td>
<td>0.00062</td>
<td>0.00122</td>
</tr>
<tr>
<td>Measured for doubly reinforced beams only</td>
<td>0.001695</td>
<td>0.001103</td>
<td>0.001072</td>
</tr>
</tbody>
</table>
4.10 Contribution of Stirrups to Beam Shear

The shear carried by stirrups was calculated according to ACI code from the following formula:

\[ V_s = \frac{A_v f_s d}{s} \]  \hspace{1cm} (4.1)

Where:

- \( V_s \) = Shear force in stirrups.
- \( A_v \) = Area of stirrup (2-legs of U-stirrup)
- \( f_s \) = Tensile stress in stirrups = \( \varepsilon_{st} \times E_s \)
- \( \varepsilon_{st} \) = Strains measured in stirrups.
- \( E_s \) = Modulus of elasticity (29 x 10^6 psi).
- \( d \) = Distance between tension flexural reinforcements and the top of beam's cross section.
- \( s \) = Spacing between stirrups.

The shear (V) is simply the stirrups shear plus the un-reinforced beams' shear.

The percentages of stirrups shear (\( V_s \)) with respect to shear (V) at maximum machine capacity for PMC, SFRC, and SFPMC were 22%, 9%, and 8.5% respectively (Table 4.3). The stirrups of SFRC and SFPMC beams carried lower percentage of shear force than PMC beams because part of shear carried by steel fibers.

The shear strengths of un-reinforced PMC, SFRC, and SFPMC dog-bone specimens due to pure torsion (from chapter 7) were 373 psi, 453 psi and 414 psi respectively. The ratio of shear strengths of SFRC and SFPMC to shear strength of PMC were 1.218 and 1.113 respectively. The measured shear forces for un-reinforced concrete beams at maximum testing machine capacity were 14.36, 14.5, and 14.5 kips for PMC, SFRC, SFPMC (Table 4.3), where the testing loads reached the maximum capacity of the
Table 4.3 Shear capacity of stirrups at maximum measured strains for PMC, SFRC, and SFPMC slender beams

<table>
<thead>
<tr>
<th></th>
<th>PMC</th>
<th>SFRC</th>
<th>SFPMC</th>
</tr>
</thead>
<tbody>
<tr>
<td>$V_s$ (kips)</td>
<td>3.97</td>
<td>1.43</td>
<td>1.34</td>
</tr>
<tr>
<td>$V_c$ (kips)</td>
<td>14.36</td>
<td>14.5</td>
<td>14.5</td>
</tr>
<tr>
<td>$V$ (kips)</td>
<td>18.33</td>
<td>15.92</td>
<td>15.84</td>
</tr>
<tr>
<td>$\frac{V_s}{V} \times 100$</td>
<td>22%</td>
<td>9%</td>
<td>8.5%</td>
</tr>
</tbody>
</table>

Table 4.4 Shear capacity of stirrups at predicted shear for PMC, SFRC, and SFPMC slender beams

<table>
<thead>
<tr>
<th></th>
<th>PMC</th>
<th>SFRC</th>
<th>SFPMC</th>
</tr>
</thead>
<tbody>
<tr>
<td>$V_s$ (kips)</td>
<td>3.97</td>
<td>2.12</td>
<td>1.675</td>
</tr>
<tr>
<td>$V_c$ (kips)</td>
<td>14.36</td>
<td>17.66</td>
<td>16.14</td>
</tr>
<tr>
<td>$V$ (kips)</td>
<td>18.33</td>
<td>19.78</td>
<td>17.82</td>
</tr>
<tr>
<td>$\frac{V_s}{V} \times 100$</td>
<td>22%</td>
<td>10.7%</td>
<td>9.4%</td>
</tr>
</tbody>
</table>

testing machine for SFRC and SFPMC beams. The predicted shear values are simply multiplying the measured shear strengths of SFRC and SFPMC by 1.218 and 1.113 respectively giving predicted shear values of 19.78 and 17.82 kips (Table 4.4). The percentages of stirrups shear ($V_s$) with respect to predicted total shear ($V$) for PMC, SFRC, and SFPMC were 22%, 10.7%, and 9.4% respectively, also the contribution of steel fiber in carrying shear for SFRC is 11.3% (the difference between 22% of PMC and
10.7% of SFRC) and the contribution of steel fiber and polymer in carrying shear for SFPMC is 12.6% (the difference between 22% of PMC and 8.6% of SFPMC).

4.11 Conclusions

Shear stresses were resisted by concrete and steel reinforcements. The presence of steel fiber improves concrete diagonal tensile strains and reduces tensile strains in reinforcements due to the increase in the shear and tensile strengths. Crack growth relieves diagonal strains in concrete for polymer modified concrete beams, while steel fiber reinforced concrete beams does not crack as early as PMC beams. The presence of steel fiber and polymer together in steel fiber/polymer modified concrete reduced micro-cracks which increase the tensile strain values more than both PMC and SFRC beams.

The compressive strain values are almost similar because this phenomena do not exist, while the beams contain steel fiber have more strains because steel fibers tend to buckle under compression loadings. The presence of polymer and fiber together improve concrete ductility and reduce micro-cracks. Steel fiber increases diagonal tensile strains by about 39% and 55% for singly and doubly reinforced slender beams respectively as compared to those of polymer modified concrete beams and reduced both stirrups and rebar tensile strains by 64% and 35% respectively. Moreover, steel fiber may replace some of the stirrups in high shear beams which may result in substantial saving in materials and labor. The presence of both steel fiber and polymer together improves concrete ductility, impermeability and resistance to micro-cracking which is required for structural members that are subject to fatigue and impact loading and environmental hazards, such as highway bridges. The combination of steel fiber and polymer increased
concrete diagonal tensile strains by an average values of 42% and 62% for singly and
doubly reinforced slender beams respectively as compared to polymer modified concrete
beams and reduced both stirrups and rebar tensile strains by 66% and 37% respectively.
Again, the steel fiber and polymer concrete could replace some stirrups and act as partial
shear reinforcement. This agrees with the conclusions of Batson et. al., 1972, and
Narayanan and Darwish, 1988, while polymer reduced microcracks which is desired in
some structures in chemical plants or water retaining structures.
Figure 4.1 Slender beams without transverse shear reinforcement (stirrups)
Figure 4.2 Slender beams with transverse shear reinforcement (stirrups) and single tension reinforcement (2#4)
Figure 4.3 Shear specimen with transverse shear reinforcement (stirrups) and double reinforcement (4#4)
Figure 4.4 Strain gages configuration on steel reinforcement and on concrete for slender beams
Figure 4.5 Slender beam under four point loading

Figure 4.6 Polymer modified concrete slender beams shear diagonal crack
Figure 4.7 Polymer modified concrete failure mode

Figure 4.8 Steel fiber reinforced concrete slender beams with shear diagonal crack
Figure 4.9 Steel fiber reinforced concrete slender beams failure mode in shear

Figure 4.10 Polymer modified concrete singly reinforced slender beam with stirrups at crack initiation

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.
Figure 4.11 Polymer modified concrete doubly reinforced slender beams with stirrups at crack initiation

\[\text{Figure 4.12 Shear force and bending moment diagram for beams under four point loading}\]
Figure 4.13 Concrete shear versus diagonal tensile and compressive shear strains for polymer modified concrete with different reinforcement configuration at crack initiation.

Figure 4.14 Concrete shear versus diagonal tensile and compressive shear strains for steel fiber reinforced concrete with different reinforcement configuration at crack initiation.
Figure 4.15 Concrete shear versus diagonal tensile and compressive shear strains for steel fiber/polymer modified concrete with different reinforcement configuration at crack initiation.

Figure 4.16 Concrete shear versus tensile strains in stirrups and longitudinal steel bars for polymer modified concrete at crack initiation.
Figure 4.17 Concrete shear versus tensile strains in stirrups and longitudinal steel bars for steel fiber reinforced concrete at crack initiation.

Figure 4.18 Concrete shear versus tensile strains in stirrups and longitudinal steel bars for steel fiber/polymer modified concrete at crack initiation.
Figure 4.19 Concrete shear versus tensile strains in longitudinal steel bars for slender beams at crack initiation

Figure 4.20 Concrete shear versus tensile strains in stirrups for slender beams at crack initiation
CHAPTER 5

HALF I-BEAMS UNDER COMBINED BENDING

SHEAR, AND TORSION

5.1 Introduction

Polymer modified concrete, steel fiber reinforced concrete, and steel fiber/polymer modified concrete half I-beams with gross aspect ratio of four and web aspect ratio of three were manufactured. The behavior of the beams under combined loadings of bending, shear, and torsion was studied. These types of loading may appear in structural members such as spandrel beams and bridge girders.

The following is a brief literature review which concentrates on steel fiber reinforced concrete beams under combined loading.

5.1.1 Literature review

Mansur and Paramasivam (1985) studied the behavior and strength of steel fiber reinforced rectangular concrete beams under combined loading. Tests were conducted on thirty-three rectangular beams under different combinations of torsion (T), bending (M) and shear (V). The tested beams had cross section dimensions of 4’’ x 6’’ and with two different lengths 4’-7’’ and 6’-7’’. The steel fiber volume fraction was 0.75% and the beams were divided into three groups: 1) beams to fail under combined torsion and
bending 2) beams to fail under combined torsion, bending, and shear, and 3) beams to fail under combined torsion and shear. It was observed that the torque-angle of twist relationship was linear up to about 70% of the ultimate torque, afterwards the curves deviates from linearity because of microcracking development. Similar behavior was observed for load-deflection relationship. The failure pattern of the beams were divided into two modes: 1) mode 1 at which bending is predominant over torsion, failed by the formation of compression zone on the top of the vertical face of the beam 2) Failure governs by torsion when torsion to moment ratio is high $T/M > 1.98$, the compression zone appeared on one of the vertical faces of the beam, the tension cracks on the opposite face were inclined at an angle of 45°. They suggested that mode 1, failure of the beam is unaffected by the amount of transverse shear, while beams failing in mode 2 were independent of the magnitude of applied bending moment but failed in shear. Finally, the interaction was torsion-bending for mode 1, while the interaction for mode 2 was torsion-shear.

No other relevant paper on fiber concrete beams under combined loading were identified, while Mansur and Paramasivam (1985) studied the effect of combined loadings of bending, shear, and torsion on long beams with span to depth ratios range between 9.33 and 13.33 and cross section aspect ratio (width/depth) of 1.5. Our study focuses on short slender beams with cross section aspect ratio range from 3 to 4 and span to depth ratio of 3.33.

5.2 Design of Half I-beam Dimensions

The aspect ratios of the experiment specimens were chosen according to the Highway
bridge I-beams dimensions which are recommended by the American Association of State Highway and Transportation Officials (AASHTO) (Nawy, 1996). AASHTO I-beams dimensions (Type I to Type V) ranges are shown in Figure 5.1 (all figures are located at the end of this chapter). The aspect ratios of the web range between 1.83 and 5.25 and the flange width to beam depth ranged from 1.6 to 2.0.

In this research, half I-beams without bottom flange were manufactured with the flange width of 6", web thickness of 3", web height of 9", and total beam height of 12" as shown in Figure 5.2. The model web aspect ratio was three and total aspect ratio was four which fall in the range of the AASHTO I-section aspect ratios.

5.3 Half I-Beams Reinforcement Configurations and Strain Gages Layout

The half I-beams were designed for flexure, shear, and torsion according to the ACI concrete code. The beam was reinforced with longitudinal reinforcement of 4#4 bars (one bar at each corner) as torsional reinforcement and transverse reinforcement (stirrups) of #3 bars at 4 inch increments along beam length which is approximately a maximum allowable spacing of half the beam depth (d/2). Flexure reinforcement of one #5 re-bar was provided in the bottom of the tension face of the beam. Quarter inch strain gages were installed in the middle of #4 longitudinal reinforcement on two opposite rebars and two strain gages were installed in the mid-length of two stirrups, at third distance from the support as shown in Figure 5.3; while 3/4 inch strain gages were installed on the concrete surface at third and middle distances from the support and at 5.5 inches, which is the location of the neutral axis based on transformed cracked section, from the top of the beam, with 45 degree inclination to measure shear strains.
5.4 Half I-beams Concrete Casting

Steel reinforcement cages were placed in a wood form as shown Figure 5.4, while concrete was placed and vibrated with an internal concrete electrical vibrator, having one inch probe diameter as shown in Figure 5.5.

5.5 Test Methodology

A central point eccentric load was applied with eccentricities ranges from zero to 2 inch to apply combined loadings of bending, shear and torsion using Tinus Olsen Universal Testing Machine (TOUTM). The beam lengths were 34 inches; they supported at 30 inch span lengths. Steel fixtures were manufactured to hold the beam ends and lateral steel struts prevented the beam from flipping as shown in Figure 5.6. The flange of the beams was confined with two steel plates from both sides at mid-length of the beam with two C-clamps to prevent local failure as shown in Figure 5.7. The strain readings were collected by data acquisition system.

The shear stress due to torsion was calculated from the following formulas:

\[ \tau_t = \frac{Tt}{J} \]  \hspace{1cm} (5.1)

\[ J = \frac{1}{3} bt^3 \]  \hspace{1cm} (5.2)

Where:

\( \tau_t \) = Shear stress due to torsion (psi).

\( T \) = Applied torque (inch-pound).

\( t \) = Beam thickness (inch).

\( b \) = Beam width (inch).
The torsional constant (stiffness) is \( J \) (in.):

The direct shear stress was calculated from the following formula:

\[
\tau_v = \frac{V S}{I_{cr} b}
\]

(5.3)

Where:

\( \tau_v \) = Direct shear stress (psi).

\( V \) = Shear force (pound).

\( S \) = First moment of area about strain gages axis location (in.).

\( I_{cr} \) = Moment of inertia of cracked section about strain gages axis location (in.).

\( b \) = Web thickness (inch).

The shear (\( \tau \)) is the additions of direct shear (\( \tau_v \)) and shear due to torsion (\( \tau_t \)).

\[
\tau = \tau_v + \tau_t
\]

(5.4)

5.6 Behavior of Diagonal Shear in Concrete Half I-beams

5.6.1 Polymer modified concrete I-beams

The half I-beams were loaded with varying central point loads at different eccentricities ranging between zero and 2 inch as shown in Figure 5.6. At the maximum eccentricity of 2 inch and a gravity load of 28 kips, inclined 45° hairline cracks appeared at various places on the beam web and at the edge of the flange in a shear-torsion mode. Figure 5.9 shows the relationship between calculated combined shear stresses, due to both vertical shear and torsion, versus diagonal shear strains as measured by the inclined strain gages mounted on the concrete surface, of the polymer modified concrete half...
I-beams. Vertical shear only was present at zero eccentricity because torsional shear was zero. The ultimate total shear strain value, at eccentricity of 2 inch, was 0.000534 at 1723 psi.

5.6.2 Steel fiber reinforced concrete I-beams

Figure 5.10 shows inclined web cracks in concrete due to combined torsional and flexural loading for steel fiber reinforced concrete half I-beams. The crack slope was steeper than in the polymer modified concrete beams, since the steel fibers bridged the micro cracks and improved concrete cohesion. The shear strain value was 0.000653 at the calculated combined shear stress of 1727 psi, at 2 inch eccentricity, as shown in Figure 5.11. The presence of steel fiber provided more ductility as compared with polymer modified concrete half I-beams since the steel fibers bridged the cracks and increased beams capacity to resist shear.

5.6.3 Steel fiber/polymer modified concrete I-beams

Figure 5.12 shows inclined web cracks in concrete due to combined torsional and flexural loading for steel fiber polymer modified concrete half I-beams. The crack trend was similar to that of steel fiber reinforced concrete beams. The relation between calculated combined shear versus shear strain has the same trend as those of polymer modified concrete and steel fiber reinforced concrete half I-beams. The shear strain value was 0.00074 at calculated combined and torsional shear stress of 1813 psi at 2 inch eccentricity as shown in Figure 5.13. The presence of polymer with steel fiber provided more ductility as compared with polymer modified concrete and steel fiber reinforced concrete half I-beams, since the fibers bridged the cracks and polymer increased the bond between the fibers and the concrete matrix.
Figure 5.14 shows the relationship between calculated combined shear stresses in concrete versus shear strain, due to eccentric gravity loading conditions at eccentricity of 2 inch, for all beams and crack initiation. The maximum steel fiber reinforced concrete shear strain of 0.000653 showed an increase of 22% as compared to shear strain of 0.000534 of polymer modified concrete beams because steel fibers started to function after micro-cracks development. Also, the shear strain of steel fiber/polymer modified concrete beams was 0.00074 with 38.6% increase over that of polymer modified concrete beams.

5.7 Behavior of Transverse Reinforcement (Stirrups) in Concrete half I-beams

5.7.1 Polymer modified concrete

Tension strains in stirrups were recorded for I-beams to study the contribution of concrete, steel fiber, and transverse reinforcement in resisting shear. Figure 5.15 shows the relationship between the applied central point load, which accounts for double the shear value, versus tensile strains in the transverse reinforcement (stirrups) for polymer modified concrete half I-beams at different loading eccentricities and crack initiation. The relationship was linear up to an approximate applied load of 21,000 pounds and changed to non-linear due to the development of micro cracks in concrete. The maximum stirrups' strain was 0.00083 at an applied load of 26,750 pounds and eccentricity of 2 inch.

5.7.2 Steel fiber reinforced concrete

Figure 5.16 shows the relationship between the applied central point load versus
tensile strains in the transverse reinforcement (stirrups) for steel fiber reinforced concrete half I-beams at different loading eccentricities and crack initiation. The relationship was linear up to an approximate applied load of 20,000 pounds and changed to non-linear due to the development of micro cracks in concrete. The maximum measured strain in the stirrups was 0.000654 at an applied load of 27,125 pounds and eccentricity of 2 inches.

5.7.3 Steel fiber/polymer modified concrete

Figure 5.17 shows the relationship between the applied central point load versus tensile strains in the transverse reinforcement (stirrups) for steel fiber/polymer modified concrete half I-beams at different loading eccentricities and crack initiation. The relationship was linear up to approximately applied load of 24,000 pounds and changed to non-linear due to the development of micro cracks developed in concrete. The presence of steel fiber and polymer together delayed the micro-cracks development to a higher load as compared to polymer modified concrete beams. The maximum stirrups' strain was 0.000466 at applied load of 28,075 pounds and eccentricity of 2 inches.

The maximum stirrup strain at a loading eccentricity of 2 inches decreased by 21% and 43% respectively for SFRC and SFPMC half I-beams as compared to that of PMC half I-beams as shown in Figure 5.18, because steel fibers carried a portion of the shear load which reduced the stirrups' resistance to shear, also the presence of polymer with steel fibers together increased the ability of concrete to carry shear which reduced the contribution of stirrups in resisting the shear stresses. The contribution of steel fibers in resisting shear were 21% and 43% for SFRC and SFPMC respectively. Polymer increases the bond strength between the fibers and the concrete matrix and reduces micro cracking in concrete, thus increasing the contribution of fibers in shear and torsion.
5.8 Tension in Longitudinal Reinforcement of Concrete half I-beams

5.8.1 Polymer modified concrete

The tension strains in the top and bottom re-bar of the longitudinal reinforcement due to combined loading of bending, shear, and torsion were recorded. Figure 5.19 shows the relationship between the applied central point load, at different loading eccentricities and crack initiation, versus tensile strains in the bottom rebars of the longitudinal reinforcement for polymer modified concrete half I-beams. The top re-bars are exposed to compression force due to bending and tension force due to torsion, while the bottom re-bars experienced tension forces due to both bending and torsion. The trend of the load-strains relationship for top re-bars started with negative values of strains and changed to tension strain value at the maximum load level (Figure 5.20), because the strains due to the compressive force of bending and tension force of torsion loading subtract in the top re-bars. The strain values in the bottom re-bars were positive because both forces due to bending and torsion added together. The tension strain values were 0.000091 and 0.001214 respectively at a central load of 26,750 pounds for the top and bottom re-bars.

5.8.2 Steel fiber reinforced concrete half I-beams

Figure 5.21 shows the relationship between the applied central point load versus tensile strains in the bottom rebars of the longitudinal reinforcement for steel fiber reinforced concrete half I-beams at different load eccentricities and crack initiation. The trend of the load strain relationships for top and bottom re-bars was similar to that of the polymer modified concrete half I-beams as shown in Figure 5.22. As an example, at an
eccentricity of 2 inch, the tension strain values were 0.000127 and 0.00094 respectively at a central load of 27,125 pounds for top and bottom re-bars.

5.8.3 Steel fiber/polymer modified concrete half I-beams

Figure 5.23 show the relationship between the applied central point load versus tensile strains in the bottom rebars of the longitudinal reinforcement for steel fiber/polymer modified concrete half I-beams at different load eccentricities and crack initiation. The trend of the load-strains relationship for top and bottom re-bars was similar to that of the polymer modified concrete half I-beams as shown in Figure 5.24. As an example, at eccentricity of 2 inch, the tension strain values were 0.000174 and 0.00081 respectively at a central load of 28,000 pounds for top and bottom re-bars.

Steel fiber decreased bottom rebars tension strains by 23% as compared with that of polymer modified concrete beams. Also, the steel fiber and polymer decreased the longitudinal reinforcement’s tensile strains by 33% as compared with that of polymer modified concrete beams as shown in Figure 5.25. Steel fiber and polymer did not do much for the top rebars of the longitudinal reinforcement since the steel fibers are exposed to compression, the curves of the relationship for PMC, SFRC, and SFPMC are approximately similar for top rebars as shown in Figure 5.25.

Both longitudinal and transverse reinforcement did not reach yield strain during testing, but they experienced strain values lower than steel yield strains. The torsion carried by longitudinal reinforcement was calculated according to ACI with the actual tension stress ($f_s$) calculated from the experimentally measured strains in longitudinal reinforcements instead of the yield stress, $f_y$ as follows:

$$ T = \frac{2A_o A_t f_s}{2(x_o + y_o) \cot \theta} \quad \text{(for longitudinal reinforcement)} \quad (5.5) $$
Where:

\[ f_s = \varepsilon E \text{ (psi)} \]

\[ \varepsilon_i = \text{Measured strains in the longitudinal reinforcement.} \]

\[ E = \text{Steel reinforcement modulus of elasticity } = 29 \times 10^6 \text{ psi} \]

Also the torsion carried by stirrups was calculated according to ACI concrete code from the following formula.

\[ T = \frac{2A_o \Delta e_{f_s} \cot \theta}{s} \quad (\text{for transverse reinforcement}) \tag{5.6} \]

Where:

\[ f_s = \varepsilon_i E \text{ (psi)} \]

\[ \varepsilon_i = \text{Measured strains in the transverse (stirrups) reinforcement.} \]

\[ E = \text{Steel reinforcement modulus of elasticity } = 29 \times 10^6 \text{ psi} \]

Table 5.1 shows the difference between the measured torsion and calculated torsion for PMC, SFRC, and SFPMC half I-beams. The percentage difference between the measured torsion and calculated torsion for PMC half I-beams was 1%, this is within the accuracy of measurements. However, the difference between measured torsion and calculated torsion for SFRC half I-beams was 5% because steel fiber carried those 5% of torsion, also the difference between the measured torsion and calculated torsion for SFPMC half I-beams was 18% since the presence of polymer with steel fiber increases the bond between the fibers and the concrete matrix and decrease microcracking in concrete by increasing cohesion.

5.8.4 Transformed cracked section analysis

ACI concrete code provision A.5.5 states that in doubly reinforced flexural members, an effective modular ratio \( 2n = 2E_s/E_c \) shall be used to transform compression.
reinforcement for stress computation. Compressive stress in such reinforcement shall not exceed permissible tensile stress. The reason for considering modular ratio \((2n)\) in stress calculation for design rather than \((n)\) is because concrete under stress deforms with time (creep) and it is also subject to shrinkage over a period of time. These time dependent effect do not occur in steel, hence as concrete deforms there is a continuous transfer of load from concrete to steel. One way of approximating the effect of this transfer of load is to increase the equivalent concrete area of compression steel to more than \(n\) times the actual steel area (Wang and Salmon, 1992). In our experiment, there was no time dependent effect on the tested beams, since tests were performed shortly after full curing time of concrete. Thus, the analysis was performed twice based on two transformed cracked section methods using modular rations \(n\) and \(2n\) for transforming compression steel. Figure 5.26 shows a transformed section using modular ratio \(n\). The strains in both top and bottom rebars of longitudinal reinforcement are shown in Table 5.2. Strains of

<table>
<thead>
<tr>
<th></th>
<th>PMC half l-beams</th>
<th>SFRC half l-beams</th>
<th>SFPMC half l-beams</th>
</tr>
</thead>
<tbody>
<tr>
<td>Torsion carried by stirrups (inch-pound)</td>
<td>9244</td>
<td>9780</td>
<td>5255</td>
</tr>
<tr>
<td>Torsion carried by re-bars (inch-pound)</td>
<td>17791</td>
<td>15951</td>
<td>17467</td>
</tr>
<tr>
<td>Torsion carried by stirrups+rebars (inch-pound)</td>
<td>27035</td>
<td>25731</td>
<td>22725</td>
</tr>
<tr>
<td>Measured Torsion (inch-pound)</td>
<td>26750</td>
<td>27100</td>
<td>28000</td>
</tr>
<tr>
<td>% Difference</td>
<td>+ 1%</td>
<td>+5%</td>
<td>18%</td>
</tr>
</tbody>
</table>
longitudinal reinforcement steel bars were calculated for PMC, SFRC, and SFPMC I-beams under both bending and torsion. The measured strains in the bottom re-bars were 0.001214, 0.00094, and 0.00081 for PMC, SFRC, and SFPMC half I-beams, while the calculated strains for the bottom re-bars using a modular ratio, n were 0.001551, 0.00141, and 0.00138 respectively for PMC, SFRC, and SFPMC half I-beams. Again, the strains in the top re-bars were 0.000091, 0.000127, and 0.000174 for PMC, SFRC, and SFPMC half I-beams, while the calculated strains for top re-bars were 0.000071, 0.000127, and 0.000174 respectively for PMC, SFRC, and SFPMC half I-beams. The analysis for calculating reinforcement stresses based on modular ratio n, gave reasonably close values to the measured strain values, while the calculated strains based on 2n were much higher than the measured values as shown in Table 5.2. It could be concluded that using modular ratio of n for transforming compression steel is more realistic than using 2n for beam testing, however using 2n is recommended for design.

5.9 Post Crack Initiation studies of half I-beams

The half I-beams were tested after the initiation of cracks and readings were collected for concrete shear strain, tension strain in both stirrups and longitudinal reinforcement to study the role of steel fiber and polymer after cracking of concrete.

5.9.1 Behavior of concrete shear after crack for half I-beams

Figure 5.27 shows the relationship between calculated combined shear stresses versus shear strains at loading eccentricity of 2 inch after crack initiation. The shear strain for polymer modified concrete was 0.000471 with about 88% of that value at crack initiation which shows the softening in the beam stiffness. The shear strain for steel fiber
reinforced concrete was 0.000591 which was 91% of that value at crack initiation because steel fiber bridged cracks and more energy consumed in de-bonding and stretching of fibers which had the beam to resist torsional loading after crack initiation. The shear strain for steel fiber/polymer modified concrete beams was 0.0007104 which was 96% of that value before crack since the concrete section capacity to resist torsion after crack initiation was about the same of that at initiation of crack. The steel fibers

Table 5.2 Calculated strains from cracked models analysis using \( n = \frac{E_s}{E_c} \) or \( 2n = \frac{2E_s}{E_c} \) versus measured strain in the compression longitudinal reinforcements

<table>
<thead>
<tr>
<th></th>
<th>PMC</th>
<th>SFRC</th>
<th>SFPMC</th>
</tr>
</thead>
<tbody>
<tr>
<td>Calculated strains for bottom re-bars due to bending</td>
<td>( n ) 0.00081</td>
<td>0.00066</td>
<td>0.000619</td>
</tr>
<tr>
<td></td>
<td>( 2n ) 0.00115</td>
<td>0.00112</td>
<td>0.00106</td>
</tr>
<tr>
<td>Calculated strains for top re-bars due to bending</td>
<td>( n ) -0.000667</td>
<td>-0.000546</td>
<td>-0.000511</td>
</tr>
<tr>
<td></td>
<td>( 2n ) -0.0003389</td>
<td>-0.000295</td>
<td>-0.000288</td>
</tr>
<tr>
<td>Calculated strains in the bars due to torsion (Equation 5.5)</td>
<td>0.000741</td>
<td>0.00075</td>
<td>0.00076</td>
</tr>
<tr>
<td>Calculated strains for bottom re-bars due to bending and torsion</td>
<td>( n ) 0.001551</td>
<td>0.00141</td>
<td>0.00138</td>
</tr>
<tr>
<td></td>
<td>( 2n ) 0.001891</td>
<td>0.00187</td>
<td>0.00182</td>
</tr>
<tr>
<td>Measured strains in bottom re-bars due to bending + torsion</td>
<td>0.001214</td>
<td>0.00094</td>
<td>0.00081</td>
</tr>
<tr>
<td>Calculated strains for top re-bars due to bending and torsion</td>
<td>( n ) 0.000071</td>
<td>0.000114</td>
<td>0.000108</td>
</tr>
<tr>
<td></td>
<td>( 2n ) 0.000402</td>
<td>0.000455</td>
<td>0.000472</td>
</tr>
<tr>
<td>Measured strain in top re-bars due to bending + torsion</td>
<td>0.000091</td>
<td>0.000127</td>
<td>0.000174</td>
</tr>
</tbody>
</table>

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.
improved the beam behavior after crack initiation since the beam stiffness did not change much.

5.9.2 Behavior of tension in stirrups after crack for half I-beams

Theoretically, the reinforcements should carry all developed stresses due to combined loadings after cracks development. The maximum tensile strain in stirrups for polymer modified concrete beams was 0.00121 (Figure 5.28) after crack initiation. The ratio between the strain in stirrups after and at crack initiation was 1.46 which means that stirrups carried the loading after the initiation of cracks. Also, the maximum tension strains in stirrups for steel fiber reinforced concrete and steel fiber/polymer modified concrete beams were 0.00085 and 0.000634 respectively (Figure 5.28) after crack initiation. The ratio between those values after and at crack initiation were 1.32 and 1.36 respectively which were less than that ratio of polymer modified concrete beams because steel fiber carried part of the stresses and steel fiber and polymer increased concrete contribution in carrying stresses.

5.9.3 Behavior of tension in bottom re-bars of longitudinal reinforcement after crack for half I-beams

The maximum tensile strain in the bottom rebars of the longitudinal reinforcement for polymer modified concrete beams was 0.000154 (Figure 5.29) after crack initiation. This ratio between this value after and at crack initiation was 1.27. Moreover, the maximum tensile strain in longitudinal reinforcement for steel fiber reinforced concrete and steel fiber/polymer modified concrete beams, after crack initiation, were 0.00114 and 0.00063 respectively as shown in Figure 5.29. The ratios between those values after and at crack initiation were 1.21 and 1.18 respectively which were less than that ratio of polymer
modified concrete beams because steel fiber carried part of loads, also steel fiber and polymer increased concrete contribution in carrying loads.

5.10 Torsion Behavior Theories

5.10.1 Skew bending theory

The basic characteristics of skew bending theory are the assumptions of a skew failure surface. This surface is initiated by a helical crack on three faces of a rectangular beam, while the end of this helical crack are connected by a compression zone near the fourth face. The failure surface intersects both the longitudinal and transverse reinforcements. The forces in this reinforcement provide the internal forces and moments to resist the external forces and moments. At the beam failure, the two parts of the beam separated by the failure surface rotate against each other about a neutral axis on the inside edge of the compression zone. It is often assumed that both the longitudinal and transverse reinforcements will yield at the collapse of the beam. In 1959, Lessig first proposed the skew bending theory in connection with two modes of failure, mode 1 failure at which the compression zone is along a side face. For mode 1, the failure produced a torsion-bending interaction curve, whereas for mode 2 the failure gave a torsion-shear interaction curve (Hsu, 1984). When compared to tests, this method was found conservative in the torsion-bending interaction and to be too conservative in the torsion-shear interaction.

5.10.2 Space truss analogy theory

The concept of simulating the post-cracking act of a reinforced concrete member
subjected to shear diagonal cracks by a truss mode. The diagonal cracks separate the concrete into a series of concrete struts. It was assumed that the beam will act like a plane truss to carry the load. The top and bottom longitudinal reinforcement steel bars serve as the top and bottom chords of the truss, while the transverse reinforcements (stirrups) and the concrete struts serve as the web members. The inclination of the concrete struts was assumed to be 45° (Hsu, 1984).

5.11 Combined Loading Interaction

5.11.1 Torsion-bending interaction

When bending acts simultaneously with torsion, the bending capacity of the member is reduced and cracks occur at low loading level. According to Collins and Lampert (Wang and Salmon, 1998) there are two failure modes, the first occurs due to yielding of tension steel and stirrups and the second occurs due to yielding of compression steel and stirrups. The interaction bending-torsion equation when tension and compression reinforcement are equal can be approximated as follows:

\[
\left(\frac{T_n}{T_{no}}\right)^2 + \left(\frac{M_n}{M_{no}}\right) = 1
\]

(5.7)

Where:

\(T_n\) = Nominal torsional moment strength in presence of flexure.

\(T_{no}\) = Nominal torsional strength when member is subject to torsion alone.

\(M_n\) = Nominal flexural strength in the presence of torsion.

\(M_{no}\) = Nominal flexural strength when the member is subject to flexure alone.
In case of unequal top and bottom reinforcement at which tension reinforcement is larger than compression reinforcement, the bending torsion equation can be modified for the case of first yield in bottom steel as follows.

\[
\left( \frac{T_n}{T_{no}} \right)^2 = \frac{1}{r} \left( 1 - \frac{M_n}{M_{no}} \right)
\]  

(5.8)

Where:

\[
r = \frac{A_s' f'_y}{A_s f_y}
\]

(5.9)

and

\[
A_s' = \text{Top compression reinforcement.}
\]

\[
A_s = \text{Bottom tension reinforcement.}
\]

\[f_y \text{ and } f'_y \text{ are the yield stresses for top and bottom steel reinforcements.}
\]

For the case when the top steel yields first, the expression is:

\[
\left( \frac{T_n}{T_{no}} \right)^2 = 1 + \left( \frac{M_n}{M_{no}} \right) \frac{1}{r}
\]

(5.10)

5.11.2 Shear-Torsion interaction

The shear-torsion interaction for bars without web reinforcement follows quarter circle relationship, but the sections with web reinforcements relation curve is flatter than quarter circle (Wang and Salmon, 1998). The quarter circle expression is:

\[
\left( \frac{T_n}{T_{no}} \right)^2 + \left( \frac{V_n}{V_{no}} \right)^2 = 1
\]

(5.11)

\[V_{no} = \text{Nominal shear strength when member is subject to shear only, and this computed as follows:}
\]
\[ V_{nm} = 10\sqrt{f'_c b_d} \]  
(Nawy, 1996)  
(5.12)

\[ V_n = \text{Nominal shear strength in torsion and shear.} \]

In our experiment, the maximum loads applied to the half I-beams were limited by the machine capacity, so the ultimate loading conditions did not exist. The moment-torsion and shear-torsion interactions were plotted based on actual measured load, moment, torsion, and shear.

The moment-torsion interaction equation for unequal top and bottom reinforcement at which tension reinforcement is larger than compression reinforcement (Wang and Salmon, 1992) was modified based on the actual moment and torsion values as follows:

\[ \left( \frac{T}{T_o} \right)^2 = \frac{1}{r} \left( 1 - \frac{M}{M_o} \right) \]  
(5.13)

Where:

- \( M_o \): Maximum moment applied on the half I-beams.
- \( M \): Moment applied on the half I-beams at any loading increment.
- \( T_o \): Maximum torsion applied on the half I-beams at eccentricity of 2 inches.
- \( T \): Torsion applied on the half I-beams at ant eccentricity and loading stages.

Also, the shear-torsion equation based on actual loads was modified as follows:

\[ \left( \frac{T}{T_o} \right)^2 + \left( \frac{V}{V_o} \right)^2 = 1 \]  
(5.14)

Where:

- \( V_o \): Maximum shear applied to the half I-beams.
- \( V \): Shear applied to the half I-beams at any loading increment.
- \( T_o \): Maximum Torsion applied to the half I-beams at eccentricity of 2 inches.
Torsion applied to the half I-beams at any eccentricity and loading stages.

Figures 5.30 and 5.31 show the bending-torsion and shear-torsion interactions at maximum eccentricity of 2 inch for both external applied loads and internal strengths in the half I-beams. The beam resistance to bending-torsion mode of failure was below the maximum failure envelope, also the shear-torsion failure mode was lower than the maximum loading interaction envelope because of the steel fiber contribution in carrying part of the combined loadings of bending-torsion or shear-torsion as shown in Figures 5.30 and 5.31.

5.12 Conclusions

Polymer modified concrete, steel fiber reinforced concrete, and steel fiber/polymer modified concrete short half I-beams were tested under combined loadings of bending, shear, and torsion. Concrete shear strength due to combined vertical shear and torsional moments was investigated. The presence of steel fiber and steel fiber and polymer together improved concrete shear strengths and performance since the steel fibers provided ductility to the concrete, while adding 5% of polymer solids per weight of cement reduced micro cracking and a measure of plasticity in concrete. The steel fibers improved concrete resistance to torsion since they increased concrete shear strain by 22% at crack initiation. The contribution of steel fibers in carrying shear in the presence of stirrups were 21% and 43% for SFRC and SFPMC half I-beams respectively, while the contribution of steel fibers in carrying torsion in the presence of longitudinal reinforcement were 23% and 33% for SFRC and SFPMC half I-beams respectively. Also, the steel fiber and fibers and polymer together increased the concrete combined...
steel fiber and polymer mixes provided additional ductility and toughness under combined bending, shear, and torsional loadings. 21% of stirrups area may be replaced by 1% steel fibers, while 43% of stirrups area may be replaced by 1% steel fibers and 5% solids of polymers by cement weight. Again, 23% of longitudinal reinforcement area may be replaced by 1% steel fibers, while 33% of longitudinal reinforcement area may be replaced by 1% steel fibers and 5% solids of polymers by cement weight. The compression steel reinforcement should be transformed using a modular ratio, n to design experimental beams, however 2n may be used according to ACI code in actual design to account for long term creep and shrinkage of concrete. The failure interaction mode of our half I-beams was in combined shear and torsion. The presence of polymer with steel fiber increases the bond between the fibers and the concrete matrix which decreases microcracking. The failure mode was close to shear-torsion interaction.
The model aspect ratio falls within the actual size range:

\[ 1.83 < 3 < 5.25 \]

Figure 5.1 AASHTO geometrical dimensions of standard bridge I-sections

Figure 5.2 Half I-beams reinforcement configurations
1/4 inch strain gage

Figure 5.3 Half I-beams strain gages configuration
Figure 5.4 Half I-beam wood forms

Figure 5.5 Electrical one inch diameter probe concrete vibrator
Figure 5.6 Test set up with fixed ends and lateral supports

Figure 5.7 Confining of half I-beams flange with steel plates
Figure 5.8 Inclined web crack for polymer modified concrete I-beams

Figure 5.9 Calculated combined shear stresses versus shear strain in concrete due to eccentric gravity loading conditions for polymer modified concrete half I-beams and crack initiation

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.
Figure 5.10 Inclined web torsional crack for steel fiber reinforced concrete I-beams

Figure 5.11 Calculated combined shear stresses versus shear strain in concrete due to eccentric gravity loading conditions for steel fiber reinforced concrete half I-beams and crack initiation
Figure 5.12 Inclined web torsional crack for steel fiber/polymer modified concrete half I-beams

Figure 5.13 Calculated combined total shear stresses versus shear strain in concrete due to eccentric gravity loading conditions for steel fiber/polymer modified concrete half I-beams and crack initiation
Figure 5.14 Calculated combined shear stresses in concrete versus shear strain due to eccentric gravity loading conditions at eccentricity of 2 inches for half I-beams and crack initiation.

Figure 5.15 Applied central point load versus tensile strains in the transverse reinforcement (stirrups) for polymer modified concrete half I-beams at different loading eccentricities and crack initiation.

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.
Figure 5.16 Applied central point load versus tensile strains in the transverse reinforcement (stirrups) for steel fiber reinforced concrete half I-beams at different loading eccentricities and crack initiation.

Figure 5.17 Applied central point load versus tensile strains in the transverse reinforcement (stirrups) for steel fiber/polymer modified concrete half I-beams at different loading eccentricities and crack initiation.
Figure 5.18 Applied central point load versus tensile strains in the transverse reinforcement (stirrups) for all half I-beams at loading eccentricity of 2 inches and crack initiation.

Figure 5.19 Applied central point load versus tensile strains in the bottom rebars of the longitudinal reinforcement for polymer modified concrete half I-beams at different loading eccentricities and crack initiation.
Figure 5.20 Applied central point load versus strains in the longitudinal reinforcement for polymer modified concrete half I-beams due to eccentric gravity loading conditions at different loading eccentricities and crack initiation.

Figure 5.21 Applied central point load versus tensile strains in the bottom rebars of the longitudinal reinforcement for steel fiber reinforced concrete half I-beams at different loading eccentricities and crack initiation.
Figure 5.22 Applied central point load versus tensile strains in the longitudinal reinforcement for steel fiber/polymer modified concrete half I-beams at different loading eccentricities and crack initiation.

Figure 5.23 Applied central point load versus strains in the bottom rebars of the longitudinal reinforcement for steel fiber reinforced concrete half I-beams due to eccentric gravity loading conditions at different loading eccentricities and crack initiation.

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.
Figure 5.24 Applied central point load versus strains in the longitudinal reinforcement for steel fiber/polymer modified concrete half I-beams due to eccentric gravity loading conditions at different loading eccentricities and crack initiation.

Figure 5.25 Applied central point load versus strains in the longitudinal reinforcement for half I-beams due to eccentric gravity loading conditions at different loading eccentricities and crack initiation.
Figure 5.26 Cracked I-beam and transformed section analysis

Figure 5.27 Calculated combined shear stresses versus shear strain in concrete due to eccentric gravity loading conditions at eccentricity of 2 inches for half I-beams after crack initiation

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.
Figure 5.28 Applied central point load versus strains in the transverse reinforcement for half I-beams (stirrups) due to eccentric gravity loading conditions at different loading eccentricities after crack initiation.

Figure 5.29 Applied central point load versus strains in the longitudinal reinforcement for half I-beams (stirrups) due to eccentric gravity loading conditions at different loading eccentricities after crack initiation.
Figure 5.30 Torsion-Moment interaction diagram for SFPMC half I-beams for both the maximum experimentally external applied load and internal measured strengths at maximum eccentricity of 2 inch (Equation 5.13)

Figure 5.31 Torsion-Shear interaction diagram for SFPMC half I-beams for both the maximum experimentally external applied load and internal measured strengths at maximum eccentricity of 2 inch (Equation 5.14)
CHAPTER 6

PRE AND POST CRACK STUDY OF SLENDER, SHALLOW RECTANGULAR
AND HALF I- BEAMS UNDER ECCENTRIC LATERAL LOADS

6.1 Introduction

Slender deep beams, such as spandrel beams in hybrid structures or slender half
I-beams in bridges, may experience lateral eccentric loads due to wind and earthquakes.
These forces were simulated for both slender beams and half I-beams in our experimental
investigations. Lateral loads were applied to both the 3 x 9 inches slender beams and the
half I-beams which were tested for shear and torsion, as deep slender beams with aspect
ratios of 3 and 4 and presented in chapter 4 and chapter 5. In the case of the 3 x 9 inches
slender beams, initial failure was obtained by loading the beams laterally in combined
moment, shear, and torsion. The damaged beams were then supported on their slender
edge and loaded in deep slender beam mode until failure. The goal is to assess the
potential failure modes under the combined loading conditions of bending, shear, and
torsion in the weak axis of the beams.

6.2 Slender and Wide Rectangular Beams Under Bending and Torsion Due to Lateral
Eccentric Loads

Slender and wide beams having dimensions of 3 x 9 inches and with transverse
reinforcement and having either single reinforcement (2#4) or double reinforcement
(4#4) at their corners were tested under gravity loads in the elastic range without causing
failure, then were laid on their sides and tested under four point loads perpendicular to
their weak axis with eccentricities of 1, 2, and 3 inches from the mid-beam axis as shown
in Figure 6.1 and Figure 6.2. The shear span (a=9 inch) was kept the same as for the tests
conducted in chapter 4.

6.3 Slender Rectangular Beams Under Four Point Loads After Crack Initiation

The 3 x 9 inches slender beams were loaded again under four point loading, as
presented in chapter 4 after they were loaded under lateral eccentric loads, up to failure
initiation at maximum load eccentricity to study the post crack behavior of these beams
and to determine the role of steel fiber and polymer in helping the regular steel
reinforcement contribution in resisting the stresses and strains which were developed due
to shear and flexure in short beams.

6.4. Half I-beams Under Torsion Due to Lateral Loads

Half I-beams were loaded laterally under a three point loads with varying
eccentricities of 1, 2, 3, and 4 inches from the cross sectional center of gravity line. In
this case, the concrete strain gages measured the shear strain due to torsion and horizontal
component of strain due to bending, while the direct shear stress was zero at the top and
bottom surfaces of the beams. The concrete strains transformed at 45° due to bending
were calculated from the following formula:

\[ f_c = \frac{MC}{2I_{cr}} \]  (6.1)
Where:

\[ \varepsilon = \frac{f_c}{E_c} \]

\( f_c \) = Concrete stress at bottom fiber of the concrete (psi).

\( M \) = Bending moment at the strain gage location (inch-pound).

\( l_{cr} \) = Concrete cracked moment of inertia (in\(^4\)).

\( \varepsilon \) = Concrete strain due to bending.

\( E_c \) = Concrete young's modulus (psi).

The shear stress due to torsion was calculated from the following formulas:

\[ v = \frac{T_t}{J} \]

\[ J = \frac{1}{3} bt^3 \]

Where:

\( v \) = Shear stress due to torsion (psi).

\( T \) = Applied torque (inch-pound).

\( t \) = Beam thickness (inch).

\( b \) = Beam width (inch).

\( J \) = Torsional constant (stiffness) (in\(^4\)).

6.5 Torsion of Rectangular Shallow and Wide Beams due to Eccentric Lateral Loads

6.5.1 Polymer modified concrete beams

Figure 6.3 shows inclined spiral torsion cracks due to lateral four point loads for singly reinforced polymer modified concrete shallow and wide beams. Figure 6.4 shows the concrete torsional shear stresses versus shear strains due to eccentric lateral loads for...
singly reinforced polymer modified concrete shallow beams at different loads and eccentricities. The maximum torsional shear strain for singly reinforced shallow and wide beams was 0.000094, at an eccentricity of 3 inches and a torsional shear stress of 416 psi. The maximum torsional shear strain for doubly reinforced slender beams was 0.0001, at eccentricity of 3 inches and torsional shear stress of 416 psi as shown in Figure 6.5, while the spiral crack pattern for doubly reinforced slender beams loaded laterally is shown in Figure 6.6. Figure 6.7 shows a comparison between singly and doubly reinforced slender beams at which the shear strain value increased by 6.4 % for doubly reinforced beams as compared with singly reinforced beams. The reason for this phenomena, which was also evident in other beam and pure torsion experiments is because diagonal micro cracking started earlier in singly reinforced beams, thus relieving some of the tensile stresses at 45° from the neutral axis and parallel to the strain gage.

6.5.2 Steel fiber reinforced concrete beams

Figure 6.8 shows inclined spiral torsion cracks due to lateral four point loads steel fiber reinforced concrete shallow and wide beams. Figure 6.9 shows the concrete torsional shear stresses versus shear strains due to eccentric lateral loads for singly reinforced steel fiber reinforced concrete shallow beams at different loads and eccentricities. The maximum shear strain and torsional shear stress for singly reinforced shallow beams were 0.000146 at 527 psi and eccentricity of 3 inches. Moreover, the maximum shear strain for doubly reinforced shallow beams was 0.000163 at torsional shear stress of 555 psi as shown in Figure 6.10. Figure 6.11 shows a comparison between singly and doubly reinforced slender beams at which the shear strain value increased by 11.6 % and shear stress increased by 5.3 % for doubly reinforced beams as compared
with singly reinforced beams, because steel fibers provided more ductility and toughness to concrete.

**6.5.3 Steel fiber/polymer modified concrete slender-wide beams**

Figure 6.12 shows inclined spiral torsion cracks due to lateral four point loads steel fiber/polymer modified concrete shallow and wide beams which are similar to those of both polymer modified and steel fiber reinforced concrete slender beams. Figure 6.13 shows the concrete torsional shear stresses versus shear strains due to eccentric lateral loads for singly reinforced shallow beams at different loading eccentricities. The maximum torsional shear strain for singly reinforced shallow beams was 0.000157, at an eccentricity of 3 inches and torsional shear stress of 527 psi. The maximum shear strain for doubly reinforced shallow beams loaded laterally was 0.000179, at an eccentricity of 3 inches and torsional shear stress of 569 psi as shown in Figure 6.14. Figure 6.15 shows a comparison between singly and doubly reinforced shallow beams at which the shear strain value increased by 14% and shear stress increased by 8% for doubly reinforced beams as compared with singly reinforced beams. The presence of polymer with steel fibers increased the beams capacity to torsional loadings, also concrete shear strain increased because polymer provided more ductility to the concrete material.

Figure 6.16 compares the torsional shear stresses and shear strains for all singly reinforced shallow beams. The shear stresses for steel fiber reinforced concrete beams increased by 55% as compared with that of polymer modified concrete beams, while the shear stress increased by 27%. The presence of steel fiber bridged cracks and increased the concrete torsional shear capacity, also concrete toughness and ductility increased due to the presence of steel fiber. The presence of polymer with the steel fiber together
increased the shear strain by 67% and increased shear stress by 27%. Polymers provided more ductility to concrete. Figure 6.17 compares the shear stresses and shear strains for all doubly reinforced shallow beams. The shear stresses for steel fiber reinforced concrete beams increased by 63% as compared with that of polymer modified concrete beams while the shear stress increased by 33%. The presence of polymer with the steel fiber together increased the shear strain by 79% and increased shear stress by 37%, because polymers provided more ductility to concrete.

6.6 Tensile Strains in Stirrups for Sallow and Wide Beams

Figure 6.18 shows the relationship between tensile strains in stirrups versus concrete maximum shear for doubly reinforced polymer modified concrete shallow beams at different loading eccentricities. The maximum stirrups' strain was 0.000495, at an eccentricity of 3 inch and shear force of 3750 pound while those two values were 0.000514 and 5000 pound for steel fiber reinforced concrete beams respectively which means that steel fibers increased both shear and ductility of the beams as shown in Figure 6.19. The stirrups tensile strain was 0.000493 at shear of 5000 pound for steel fiber/polymer modified concrete beams as shown in Figure 6.20. Figure 6.21 compares the shear at the support versus tensile strain in stirrups' strains for all beams. At shear value of 3750 pound, which is the maximum shear for polymer modified concrete beams, the strain value was 0.000327 for steel fiber reinforced concrete beams with strain reduction of 36% because part of the stresses were carried by steel fibers. The presence of the steel fibers with the polymers reduced the stirrups strain to 0.000283 at 3750 pound with 43% reduction in stirrups' strains for steel fiber/polymer modified concrete beams.
because the presence of polymer increased the concrete ductility and shared in carrying part of the stresses.

6.7 Tensile Strains in Longitudinal Reinforcement for shallow and Wide Beams

Figure 6.22 shows the relationship between tensile strains in the longitudinal reinforcement versus concrete maximum shear for polymer modified concrete shallow beams at different loading eccentricities. The maximum rebars' strain was 0.001534 at eccentricity of 3 inch and shear at support of 3750 pound which is about 3 times higher than stirrups' strain for the same beams. Strains in the longitudinal reinforcement for steel fiber reinforced concrete beams was 0.001227 at shear of 5000 pound as shown in Figure 6.23 which was higher that that of stirrups by 2.5 times for the same beams. Steel fiber increased the concrete capacity to resist shear and that shared in carrying part of the stresses which reduced the longitudinal reinforcement strains. The longitudinal reinforcement's strain was 0.001165 at maximum shear of 5000 pound as shown in Figure 6.24 for steel fiber/polymer modified concrete beams and that value was higher than that of stirrups by 2.4 times. Figure 6.25 shows a comparison of tensile strain in the longitudinal reinforcement for all beams. At shear value of 3750 pound, which is the maximum shear at support for polymer modified concrete beams, the strain value was 0.00092 for steel fiber reinforced concrete beams with strain reduction of 40% as compared with that of polymer modified concrete beams because part of the stresses were carried by steel fibers. The presence of the fibers with the polymers reduced the stirrups strain to 0.000853 at shear value of 3750 pound with 44% reduction in longitudinal reinforcement's strain for steel fiber/polymer modified concrete beams because the
presence of polymer increased the concrete ductility and shared in carrying part of the stresses.

6.8 Concrete Tensile and Compressive Strains of Slender Beams Under Vertical Two Point Loading After Crack Initiation Due to Lateral Loading

6.8.1 Polymer modified concrete slender beams.

Figure 6.26 shows the relationship between shear at the support versus the diagonal tensile and compressive shear strains for singly and doubly reinforced polymer modified concrete slender beams after crack initiation. The diagonal tensile strain exhibited a linear trend for singly and doubly reinforced slender beams up to shear value of 7.5 kips, the trend changed to non-linearity beyond this point because micro-cracks started to be developed in concrete. After crack initiation, the diagonal tensile strain was 0.000045 at shear load of 10.5 kips for doubly reinforced slender beams, while it was 0.000033 at shear load of 10.5 kips for singly reinforced slender beams. The diagonal compression strain value for singly reinforced slender beams was 0.000072 at shear of 10.5 kips, while it was 0.000089 for doubly reinforced slender beams at the same shear level.

The percentage of losing diagonal tension strains for singly reinforced polymer modified concrete beams at shear of 10.5 kips after crack initiation compared to that value at crack imitation (tensile shear strains at crack initiation were 0.00006 and 0.000078 for singly and doubly reinforced beams at 10.5 kips, values were presented in chapter 4) was about 45%, while it was about 42% for doubly reinforced beams. Also, the beam shear strength decreased from 14.5 kips at crack initiation to 10.5 kips after crack initiation.
6.8.2 Steel fiber reinforced concrete slender beams.

Figure 6. 26 show the relationship between shear at the support versus the diagonal tensile and compressive shear strains for singly and doubly reinforced steel fiber concrete slender beams after crack initiation. The diagonal tensile strain exhibited a linear trend for singly and doubly reinforced slender beams up to shear value of 9 kips, the trend changed to non-linearity beyond this point because micro-cracks started to be developed in concrete. After crack initiation, the diagonal tensile strain was strain was 0.000188 for singly reinforced beams and 0.000221 for doubly reinforced beams with increase of 18% at shear of 14.4 kips. Those two values were approximately four times bigger than the diagonal tensile strains of polymer modified concrete slender beams because steel fiber bridged cracks and more energy consumed in stretching and de-bonding of fibers until they pulled-out. These phenomena increased the tensile diagonal tension of SFRC beams as compared with PMC beams. The diagonal compression strain value for singly reinforced slender beams was 0.000152 at shear of 14.4 kips, while it was 0.000201 for doubly reinforced slender beams. The maximum diagonal compressive strains for singly and doubly reinforced slender beams after crack initiation for steel fiber reinforced concrete were approximately as twice as bigger than those of polymer modified concrete slender beams.

The percentage of losing diagonal tensile strains for singly reinforced steel fiber concrete beams at and after crack initiation was 6%, while it was 6.8% for doubly reinforced slender beams.

6.8.3 Steel fiber/polymer modified concrete slender beams.

Figure 6.28 shows the relationship between shear at the support versus the diagonal
tensile and compressive shear strains for singly and doubly reinforced steel fiber/polymer modified concrete for slender beams after crack initiation. The diagonal tensile and compressive strain-shear relationship has the same trend as those of polymer modified concrete and steel fiber concrete beams. The maximum diagonal strain was 0.000161 for singly reinforced beams and 0.000215 for doubly reinforced beams. Those two values were as about 4 times bigger than the maximum diagonal tensile strains of polymer modified concrete because the steel fiber bridged the cracks and more energy consumed in stretching and de-bonding of the fibers until they pulled-out, also the presence of polymer reduced crack propagations and also reduced crack widths. The diagonal compressive strain value for singly reinforced slender beams was 0.0003348 at shear 14.4 kips, while it was 0.000368 for doubly reinforced slender beams. The presence of steel fibers and polymers increased the energy required to fail the beams, because fibers arrest cracks and polymer reduced crack propagation. These two functions increased the diagonal tension capacity of the beams. In this case, the maximum diagonal tensile strains for singly reinforced beams after crack was higher by 14.2 % as compared with that value before crack. Also, the maximum diagonal tensile strains for doubly reinforced beams after crack was higher by 2.4 % as compared with that value before crack.

6.8.4 Steel fiber and polymer effect on shear behavior for concrete slender beams

Figure 6.29 and Figure 6.30 show the relationship of concrete diagonal tensile and compressive strains versus shear at the supports for different concrete mixes with single reinforcement (2#4) and double reinforcement (4#4) configurations at and after crack initiation. The trend for both singly and doubly reinforced slender beams was similar; the
doubly reinforced beams have higher tensile and compressive diagonal strains than singly reinforced beams because the reinforcement ratio was as twice as much. The presence of steel fiber increased the diagonal tension strains as compared with polymer modified concrete beams because steel fiber increased the shear loads and ductility, also more energy is consumed in both de-bonding and stretching of the fiber until they pulled-out. The presence of polymer and fiber together increased the concrete ductility. The reduction in concrete diagonal tensile strains before and after cracks for both singly and doubly polymer modified concrete slender beams were 42% and 38% respectively. After crack initiation, concrete looses its strength to resist shear stresses which in effect carried by stirrups and re-bars.

The steel fiber improved concrete diagonal tension capacity, since reduction in concrete diagonal tensile at and after crack initiation for both singly and doubly steel fiber reinforced concrete slender beams were 6.8% and 6% respectively. It could be concluded that presence of steel fiber increased the diagonal tensile strains of polymer modified concrete slender beams by an average value of 34%. The presence of steel fiber and polymer together improved the concrete diagonal tension capacity, since the concrete diagonal tensile strains at and after crack initiation for both singly and doubly reinforced steel fiber concrete slender beams were increased by 2.4% and 14.2% respectively. It could be concluded that presence of steel fiber and polymer together increased concrete ductility and improved the diagonal tension of polymer modified concrete slender beams by an average value of 48%.
6.9 Longitudinal and Transverse (Stirrups) Tension Strains of Slender Beams Under Two Point Loading After Crack Initiation Due to Lateral Loading

Figure 6.31 shows the relationship between shear at the support versus the tensile strains in the longitudinal and transverse (stirrups) reinforcements for singly and doubly reinforced polymer modified concrete slender beams at crack initiation and after crack. The maximum shear after crack was 9 kips with stirrups’ tensile strains of 0.000696 and rebars’ tension strains of 0.001417. Concrete is responsible for resisting shear before crack, while stirrups and re-bars took over after crack and resisting the tension force developed in them. After crack, stirrups carried about 41% extra tensile strain in stirrups as compared to that value at crack initiation at same shear level of 9 kips, while re-bars carried about 38% extra tensile strain in longitudinal rebars as compared to that value at crack initiation at same shear level.

Figure 6.32 shows the relationship between shear at the support versus the tensile strains in the longitudinal and transverse (stirrups) reinforcements for singly and doubly reinforced steel fiber concrete slender beams at crack initiation and after crack. After crack, the maximum tensile strains in stirrups were 0.000372 and maximum tensile strains in re-bars were 0.0011885. The maximum tensile strains in stirrups and rebars at crack initiation were 0.0002975 and 0.001103 respectively. After crack, stirrups carried about 25% while re-bars carried about 8% of the strain values developed in both stirrups and rebars at crack initiation at the same shear value of 14.5 kips.

Figure 6.33 shows the relationship between shear at the support versus the tensile strains in the longitudinal and transverse (stirrups) reinforcements for singly and doubly reinforced steel fiber/polymer modified concrete slender beams at crack initiation and
after crack. After crack, the maximum tensile strains in stirrups were 0.000323 and maximum tensile strains in re-bars were 0.001114. The maximum tensile strains in stirrups and re-bars at crack initiation were 0.000279 and 0.001072 respectively. After crack, stirrups carried about 16% while re-bars carry about 4% of the strain values developed in both stirrups and re-bars at crack initiation at the same shear level.

Figure 6.34 and Figure 6.35 show the relationship between shear at the support versus the tensile strains in the longitudinal and transverse (stirrups) reinforcements for singly and doubly reinforced slender beams. Steel fiber/polymer modified concrete slender beams stirrups' and rebar's tensile strains were lower than those of both polymer modified concrete and steel fiber reinforced concrete beams. After crack initiation, stirrups carried about 70%, 25% of the strain values developed at crack initiation for PMC and SFRC slender beams, while re-bars carried about 38%, 8%, and 4% of the strain values developed in rebars at crack initiation for PMC, SFRC, and SFPMC slender beams respectively. The maximum tension strain values in stirrups after crack initiation for SFPMC is not consistent since it shows higher values after crack initiation as compared with SFRC slender beams (Figure 6.35), it seems that the SFPMC specimens was damaged after it was tested laterally. It could be concluded that steel fiber or steel fiber and polymer together may partially substitute the transverse reinforcement which saves labor cost and time. The presence of longitudinal reinforcement is important for the beams to behave as reinforced beams and resist flexural stresses and satisfy services conditions such as deflection control.
6.10 Behavior of Torsional Shear for Half I-beams Due to Lateral Loads

Half I-beams which tested under gravity loads and do not fail completely, as stated in chapter 4, were laid on their sides and loaded laterally with eccentric central point load with eccentricities ranges between 1 and 4 inches.

6.10.1 Polymer modified concrete half I-beams

Figure 6.36 shows inclined torsion cracks in the beam flange due to lateral eccentric central point load for polymer modified concrete I-beams. Figure 6.37 shows the concrete torsional shear stresses versus shear strains due to eccentric lateral central loads at different loading eccentricities. The maximum torsional shear strain was 0.000431 at an eccentricity of 4 inches and torsional shear stress of 711 psi. This value is about 1.7 times of that of rectangular slender wide beams because the torsional rigidity (J) of the beam and the reinforcement ratio were much higher than that of rectangular beams. The maximum shear stress due to torsion only for half I-beams tested under gravity loads at eccentricity of 2 inch was 812 psi (presented in chapter 5). The beam exposed to eccentric lateral loads could resist about 87% of the gravity maximum shear stress due to torsion.

6.10.2 Steel fiber reinforced concrete half I-beams

Figure 6.38 shows inclined torsion cracks in the beam web for steel fiber reinforced concrete I-beams. Figure 6.39 shows the concrete torsional shear stresses versus shear strains due to eccentric lateral central loads for steel fiber reinforced concrete half I-beams. The maximum shear strain and torsional shear stress at an eccentricity of 4 inch
were 0.000548 and 807 psi respectively. The maximum shear stress due to torsion only was 99% for that of gravity ultimate shear stress which is 814 psi.

6.10.3 Steel fiber/polymer modified concrete I-beams

Figure 6.40 shows inclined torsion cracks in the web due to lateral loads for steel fiber/polymer modified I-beams. Figure 6.41 shows the concrete torsional shear stresses versus shear strains. The maximum shear strain was 0.000683 at eccentricity of 4 inch and torsional shear stress of 807 psi. The maximum shear stress due to torsion only was 92% for that of gravity maximum shear stress (as presented in chapter 5) which is 814 psi.

Figure 6.42 compares the torsional shear stresses versus shear strains for all beams. The shear strains for steel fiber reinforced concrete beams increased by 26% as compared with that of polymer modified concrete beams, while the shear stress was 13.5% higher. The presence of steel fiber bridged cracks and increased the concrete torsional shear capacity, also concrete toughness and ductility increased due to the presence of steel fiber. The presence of polymer with the steel fiber together increased the shear strain by 58% and increased shear stress by 13.5%. Polymers provided more ductility to concrete.

6.11 Stirrups Behavior of Concrete Half I-beams

Tensile strains in stirrups were measured due to lateral eccentric loading. Figure 6.43 shows the relationship between concrete shear at the support versus tensile strains in stirrups for polymer modified concrete half I-beams at different loading eccentricities. The maximum stirrup’s strain was 0.000735 at eccentricity of 4 inch and shear of 5837 pound, while those two values were 0.000524 and 6625 pound respectively for steel fiber.
reinforced concrete as shown in Figure 6.44. The stirrups' strain was 0.00038 at maximum shear 6625 pound for steel fiber/polymer modified concrete beams as shown in Figure 6.45. Figure 6.46 compares the torsional shear stresses and stirrups' strains for all beams. At shear value of 5837 pound, which is the maximum shear for polymer modified concrete half I-beams, the strain value was 0.000462 for steel fiber reinforced concrete beams with strain reduction of 37% because part of the stresses were carried by steel fibers, also this reduction is higher than the strain reduction of stirrups for rectangular wide beams because I-beam torsional rigidity is higher and concrete resisted more torsional shear stress than that of rectangular beams. The presence of steel fibers with polymers together reduced the stirrups strain to 0.00038 at shear value of 6625 pound, also the stirrups strain was 0.000348 at shear value of 5837 pound with 53% reduction in stirrups strains for steel fiber/polymer modified concrete beams as compared by strain value in stirrups of polymer modified concrete half I-beams because the presence of polymer increased the concrete ductility and shared in carrying part of the stresses.

6.12 Longitudinal Reinforcement Behavior of Concrete Half I-beams

Figure 6.47 shows the relationship between concrete shear at the support versus tensile strains in longitudinal reinforcement for polymer modified concrete I-beams at different loading eccentricities. The maximum stirrups' strain was 0.000905 at eccentricity of 4 inch and shear of 5837 pound which is about 23% higher than stirrups' strain for the same beams. Strains in the longitudinal reinforcement for steel fiber reinforced concrete beams was 0.000594 at shear of 6625 pound as shown in Figure 6.48.
which was higher than that of stirrups' by 12% for the same beams. Steel fiber increased the concrete capacity to resist shear and its contribution in carrying part of the loads. The longitudinal reinforcement's strain was 0.000455 at maximum shear of 6625 pound as shown in Figure 6.49 for steel fiber/polymer modified concrete beams and that value was higher than that of stirrups by 19% for the same beam.

From Figure 6.50, at shear value of 5837 pound which is the maximum shear for polymer modified concrete beams, the strain value was 0.000458 for steel fiber reinforced concrete beams with strain reduction of 50% because part of the loads was carried by steel fibers. Also, the presence of the fibers with the polymers together reduced the stirrups strain to 0.000377, at shear value of 5387 pound, with reduction of 58% in strain of longitudinal reinforcement's as compared with that of polymer modified concrete because the presence of polymer increased the concrete ductility.

6.13 Transformed Cracked Section Analysis

Since the measured strains in the longitudinal reinforcements for wide shallow beams were below the yield strain of the grade 60 steel (εy = 0.00207), a transformed cracked section using linear variation analysis with modular ratio n=E_s/E_c for comparison with experimental results. Table 6.1 shows that the difference between the calculated and measured strains in the longitudinal reinforcements were 10.7%, 122%, and 135% for PMC, SFRC, and SFPMC wide shallow 3 x9 inches beams. The measured strain were much lower than the calculated values because the steel fibers helped in carrying moments in both longitudinal and transverse directions, while this case is not present in the case of PMC beams.
6.14 Steel Fibers and Polymers Contribution to Beam Strength

Table 6.2 shows the contribution of steel fibers and steel fibers and polymer in carrying forces developed in transverse and longitudinal reinforcements for all beams with different loading configurations as presented in chapters 4, 5, and 6. The contribution of steel fiber and steel fiber and polymers was higher for the case of lateral loading application than for the case of vertical loading application, since the beams were exposed to combined loadings of bending, shear, and torsion. In addition, the moment of inertia of the beam cross-section about the weak axis is lower than that about strong axis which lowers the beams resistance to different loading combination. Thus, the steel fibers and steel fibers and polymer helped in carrying the developed forces in both transverse and longitudinal reinforcements for the beams loaded laterally (about the weak axis of the beam) with higher ratios than those of the beams loaded vertically.

6.15 Combined Loading Interaction

Figures 6.51 and 6.52 show the bending-torsion and shear-torsion interaction diagrams at maximum eccentricity of 3 inch for both external applied loads and internal strengths in the shallow wide beams. The shallow wide beam resistances to bending-torsion and shear-torsion modes were below the maximum failure envelope, due to the contribution of steel fibers in carrying part of the combined loads of bending-torsion or shear-torsion. The contribution of steel fiber in both bending-torsion and shear-torsion failure modes is the difference between the failure envelope and the measured experimental data (Figures 6.51 and 6.52).
6.16 Conclusions

Polymer modified concrete, steel fiber reinforced concrete, and steel fiber/polymer modified concrete shallow wide and half I-beams were tested under combined loadings of bending, shear, and torsion due to eccentric applied lateral loads. The contribution of steel fiber and steel fiber and polymers was higher for the case of lateral loading application than for the case of vertical loading application in carrying part of the applied loads. Slender beams could carry about 70% of the shear loads when exposed to lateral loads. The I-beam exposed to eccentric lateral loads could resist about 87% of the gravity maximum shear stress due to torsion for polymer modified concrete beams, 99% for steel fiber concrete beams, and 92% for steel fiber/polymer modified concrete beams. Also, slender beams could resist about 70% of the applied loads after cracks.

Table 6.1 Calculated strains from cracked models analysis using $n = E_p/E_c$ versus measured strain in the longitudinal reinforcements under maximum loads and eccentricities

<table>
<thead>
<tr>
<th>3 x 9 inches shallow wide beams loaded laterally</th>
<th>PMC</th>
<th>SFRC</th>
<th>SFPMC</th>
</tr>
</thead>
<tbody>
<tr>
<td>Calculated strains for bottom re-bars due to bending</td>
<td>0.00154</td>
<td>0.002</td>
<td>0.00205</td>
</tr>
<tr>
<td>Calculated strains in the bars due to torsion (Equation 5.5)</td>
<td>0.000336</td>
<td>0.000447</td>
<td>0.000456</td>
</tr>
<tr>
<td>Calculated strains for bottom re-bars due to bending and torsion (+10.7%)</td>
<td>0.001876</td>
<td>0.00245 (+122%)</td>
<td>0.00251 (+135%)</td>
</tr>
<tr>
<td>Measured strains in bottom re-bars due to bending + torsion</td>
<td>0.001695</td>
<td>0.001103</td>
<td>0.00107</td>
</tr>
</tbody>
</table>
Steel fibers carried 36% of stirrups' forces, while steel fibers and polymer carried 43% for laterally loaded shallow and wide beams. Again, steel fibers carried 40% of re-bars forces, while steel fibers and polymers carried 44% for laterally loaded shallow and wide beams. Steel fibers carried 37% of stirrups' forces, while steel fibers and polymer carried 53% for laterally loaded I-beams. Again, steel fibers carried 50% of re-bars forces, while steel fibers and polymers carried 58% for laterally loaded half I-beams.

The presence of polymer and fiber together improves concrete ductility and reduces cracks. Steel fiber improves diagonal tension strain as compared to polymer modified.

Table 6.2 Contribution of steel fibers and polymers in carrying forces in transverse (stirrups) and longitudinal reinforcements for all beams at maximum loads and/or eccentricities

<table>
<thead>
<tr>
<th></th>
<th>SFRC beams</th>
<th>SFPMC beams</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Stirrups</td>
<td>Re-bars</td>
</tr>
<tr>
<td>Slender beams loaded</td>
<td></td>
<td></td>
</tr>
<tr>
<td>vertically with four</td>
<td>11.3%</td>
<td>35%</td>
</tr>
<tr>
<td>point loads</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wide-shallow beams</td>
<td>34%</td>
<td>40%</td>
</tr>
<tr>
<td>loaded laterally with</td>
<td></td>
<td></td>
</tr>
<tr>
<td>eccentric four point</td>
<td></td>
<td></td>
</tr>
<tr>
<td>loads</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Half I-beams Loaded</td>
<td>21%</td>
<td>23%</td>
</tr>
<tr>
<td>vertically with eccentric</td>
<td></td>
<td></td>
</tr>
<tr>
<td>central point load</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Half I-beams loaded</td>
<td>41%</td>
<td>42%</td>
</tr>
<tr>
<td>laterally with eccentric</td>
<td></td>
<td></td>
</tr>
<tr>
<td>central load</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Concrete and reduces stirrups and rebars tensile strains since adding 5% of polymer solids by cement weight increased the bond between the steel fiber and the concrete matrix. It could be concluded that steel fiber or steel fiber and polymer together may partially substitute the longitudinal and transverse reinforcement which saves labor cost and time (this agrees with Batson et. al., 1972), and (Narayanan and Darwish, 1988). The measured strain in the longitudinal reinforcements for both shallow wide beams and half I-beams loaded laterally were much lower than the calculated values because the steel fibers helped in carrying moments in both longitudinal and transverse directions.
Torsion under two point lateral loading for singly reinforced beams with stirrups

Torsion under two point lateral loading for doubly reinforced beams with stirrups

Figure 6.1 Test loading configuration for slender rectangular beams
Figure 6.2 Test set up for rectangular shallow and wide beams under torsion

Figure 6.3 Inclined spiral torsional crack for singly reinforced polymer modified concrete shallow and wide beams
Figure 6.4 Concrete torsional shear stress versus shear strain due to eccentric lateral loads for singly reinforced polymer modified concrete shallow and wide beams.

Figure 6.5 Concrete torsional shear stress versus shear strain due to eccentric lateral loads for doubly reinforced polymer modified concrete shallow and wide beams.
Figure 6.6 Inclined spiral torsional crack for doubly reinforced polymer modified concrete shallow and wide beams due to eccentric lateral loads

Figure 6.7 Concrete torsional shear stress versus shear strain due to eccentric lateral loads at an eccentricity of 3 inch for polymer modified concrete shallow and wide beams

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.
Figure 6.8 Inclined spiral torsional crack for steel fiber reinforced concrete shallow and wide beams due to eccentric lateral loads

Figure 6.9 Concrete torsional shear stress versus shear strain due to eccentric lateral loads for singly reinforced steel fiber concrete shallow and wide beams
Figure 6.10 Concrete torsional shear stress versus shear strain due to eccentric lateral loads for doubly reinforced steel fiber concrete shallow and wide beams.

Figure 6.11 Concrete torsional shear stress versus shear strain due to eccentric lateral loads at an eccentricity of 3 inch for steel fiber concrete shallow and wide beams.
Figure 6.12-inclined spiral torsional crack for steel fiber/polymer modified concrete shallow and wide beams due to eccentric lateral loads

Figure 6.13 Concrete torsional shear stress versus shear strain due to eccentric lateral loads for singly reinforced steel fiber/polymer modified concrete shallow and wide beams
Figure 6.14 Concrete torsional shear stress versus shear strain due to eccentric lateral loads for doubly reinforced steel fiber/polymer modified concrete shallow and wide beams

Figure 6.15 Concrete torsional shear stress versus shear strain due to eccentric lateral loads at an eccentricity of 3 inch for steel fiber/polymer modified concrete shallow and wide beams
Figure 6.16 Concrete torsional shear stress versus shear strain due to eccentric lateral loads at an eccentricity of 3 inch for singly reinforced shallow and wide beams

Figure 6.17 Concrete torsional shear stress versus shear strain due to eccentric lateral loads at an eccentricity of 3 inch for doubly reinforced shallow and wide beams
Figure 6.18 Measured concrete shear versus tensile strain in stirrups due to eccentric lateral loads for doubly reinforced polymer modified concrete shallow and wide beams.

Figure 6.19 Measured concrete shear versus tensile strain in stirrups due to eccentric lateral loads for doubly reinforced steel fiber concrete shallow and wide beams.
Figure 6.20 Measured concrete shear versus tensile strain in stirrups due to eccentric lateral loads for doubly reinforced steel fiber/polymer modified concrete shallow and wide beams.

Figure 6.21 Measured concrete shear versus tensile strain in stirrups due to eccentric lateral loads at an eccentricity of 3 inch for doubly reinforced concrete shallow and wide beams.
Figure 6.22 Measured concrete shear versus tensile strain in longitudinal reinforcement due to eccentric lateral loads for doubly reinforced polymer modified concrete shallow and wide beams

Figure 6.23 Measured concrete shear versus tensile strain in longitudinal reinforcement due to eccentric lateral loads for doubly reinforced steel fiber concrete shallow and wide beams
Figure 6.24 Measured concrete shear versus tensile strain in longitudinal reinforcement due to eccentric lateral loads for doubly reinforced steel fiber/polymer modified concrete shallow and wide beams.

Figure 6.25 Measured concrete shear versus tensile strain in longitudinal reinforcement due to eccentric lateral loads doubly reinforced wide and shallow beams.

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.
Figure 6.26 Measured concrete shear versus diagonal tensile and compressive strains for polymer modified concrete slender beams with different reinforcement configuration after crack initiation.

Figure 6.27 Measured concrete shear versus diagonal tensile and compressive strains for steel fiber reinforced concrete slender beams with different reinforcement configuration after crack initiation.
Figure 6.28 Measured concrete shear versus diagonal tensile and compressive strains for steel fiber/polymer modified concrete slender beams with different reinforcement configuration after crack initiation.

Figure 6.29 Measured concrete shear versus diagonal tensile and compressive strains for singly reinforced (2#4) slender beams at crack initiation and after crack initiation.
Figure 6.30 Measured concrete shear versus diagonal tensile and compressive strains for doubly reinforced (4#4) slender beams at crack initiation and after crack initiation.

Figure 6.31 Measured concrete shear at the support versus tensile strains in stirrups and longitudinal steel bars for polymer modified concrete slender beams at crack initiation and after crack initiation.
Figure 6.32 Measured concrete shear at the support versus tensile strains in stirrups and longitudinal steel bars for steel fiber reinforced concrete slender beams at crack initiation and after crack initiation.

Figure 6.33 Measured concrete shear at the support versus tensile strains in stirrups and longitudinal steel bars for steel fiber/polymer modified concrete slender beams at crack initiation and after crack initiation.
Figure 6.34 Measured concrete shear at the support versus tensile strains in longitudinal steel bars for slender beams at crack initiation and after crack initiation

Figure 6.35 Measured concrete shear at the support versus tensile strains in stirrups for slender beams at crack initiation and after crack initiation
Flange crack

Figure 6.36-Inclined torsional crack in the flange of polymer modified concrete half I-beams due to eccentric lateral loads

Figure 6.37 Concrete torsional shear stresses versus shear strain due to eccentric lateral loads for polymer modified concrete half I-beams
Figure 6.38 Inclined torsional crack in the web of steel fiber reinforced concrete half I-beams due to eccentric lateral loads

Figure 6.39 Concrete torsional shear stress versus shear strain due to eccentric lateral loads for steel fiber reinforced concrete half I-beams
Figure 6.40 Inclined torsional crack in the web and flange of steel fiber/polymer modified concrete half I-beams due to eccentric lateral loads

Figure 6.41 Concrete torsional shear stresses versus shear strain due to eccentric lateral loads for steel fiber/polymer modified concrete half I-beams

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.
Figure 6.42 Concrete torsional shear stress versus shear strain due to eccentric lateral loads for half I-beams

Figure 6.43 Measured concrete shear versus tensile strain in stirrups due to eccentric lateral loads for polymer modified concrete half I-beams
Figure 6.44 Measured concrete shear versus tensile strain in stirrups due to eccentric lateral loads for steel fiber reinforced concrete half I-beams

Figure 6.45 Measured concrete shear versus tensile strain in stirrups due to eccentric lateral loads for steel fiber/polymer modified concrete half I-beams
Figure 6.46 Measured concrete shear versus tensile strain in stirrups due to eccentric lateral loads for concrete half I-beams at an eccentricity of 4 inch

Figure 6.47 Measured concrete shear versus tensile strain longitudinal reinforcement due to eccentric lateral loads for polymer modified concrete half I-beams
Figure 6.48 Measured concrete shear versus tensile strain longitudinal reinforcement due to eccentric lateral loads for steel fiber reinforced concrete half I-beams

Figure 6.49 Measured concrete shear versus tensile strain longitudinal reinforcement due to eccentric lateral loads for steel fiber/polymer modified concrete half I-beams
Figure 6.50 Measured concrete shear versus tensile strain in longitudinal reinforcement due to eccentric lateral loads for concrete half I-beams at an eccentricity of 4 inch

Figure 6.51 Torsion-Moment interaction diagram for shallow and wide beams for both the maximum experimentally external applied load and internal measured strengths at maximum eccentricity of 3 inch (equation 5.13)

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.
Figure 6.52 Torsion-Shear interaction diagram for shallow and wide beams for both the maximum experimentally external applied load and internal measured strengths at maximum eccentricity of 3 inch (equation 5.14)
CHAPTER 7

PURE TORSION MEASUREMENTS IN CONCRETE BEAMS

7.1 Introduction

The behavior of 3x 3 inches square dog bone specimens of polymer modified concrete (PMC), steel fiber reinforced concrete (SFRC), and steel fiber/polymer modified concrete (SFPMC) under pure torsional loading are presented. The tested specimens were designed according to provision 11.6 of ACI concrete code for torsion design. The specimens cross sections were designed to satisfy the MTS testing machine collet grip limitations. In our experiments, the fiber volume fraction is 1% and the fiber aspect ratio \( \left( \frac{l_f}{d_f} \right) = 50 \).

7.1.1 Literature review

The following literature concentrates on the behavior of steel fiber reinforced concrete specimens under pure torsion.

Niyogi and Dwarakanathan (1985) studied the effect of steel fiber on concrete beams under pure torsion for two different concrete mixes. The two ends of the tested beam are fitted with detachable clamp boxes and torsion arms, and the torque is applied on one arm by a 10 ton machine while the other arm is restrained against rotation. The fiber volume fraction varied from 0-3% for each mix and the aspect ratio \( \left( \frac{l_f}{d_f} \right) \) of fibers was kept constant at 50. Test beams were \( 4'' \times 6'' \times 78'' \) and were tested over a simple span of 72''. Pure torsion specimens failed under diagonal tension with the appearance of inclined...
cracks, originating at about mid-depth on a vertical long face of the beam. The torsional strength increased with larger fiber volume concentration ($V_f=3\%$).

Graig et al. (1986) studied the behavior of fiber concrete in pure torsion. Eight beams of various fiber content proportions (from 0-2%) and dimensions of $6'' \times 12'' \times 70''$ were considered, steel reinforcing bars (4#3) were placed at the ends of the beams as longitudinal and hoop steel for the purpose of causing torsional failure in the middle region of the beam. Two steel fiber types were used, small fibers (length =1.18'', diameter=0.0197'') and large fibers (length =1.97'', diameter=0.0197''). All of the test specimens were tested with one end of the beam fixed while the other end was rotated with hydraulic ram, the loads were applied on arms perpendicular to the beams. The rotation corresponding to recorded load increment was electronically measured at beam third points. Additionally, cracking was observed and recorded at each load increment as the tests were conducted. They found that cracking was much more extensive in the beams containing steel fibers than in the plain concrete, which develop a single crack and exhibited a sudden failure, while fibers increase the torsional fracture energy. The maximum energy absorption was measured at about 1% fiber content. The smaller fibers showed less fracture energy absorption than the large fibers because the fiber pull-out load was much smaller.

EL-Niema (1993) studied the effect of steel fiber on concrete beams under torsion. Six beams with dimensions of 100 mmx 200 mm (4 x 8 ) and 1.8 m (71) span length were tested, three of them with longitudinal reinforcement of 4#10 mm and fiber volume fractions ($V_f$) of 0, 0.6, and 1.2 %. The other three beams had the fiber fraction same as the first three beams with additional transverse reinforcement (stirrups) of $\Phi 6$ mm @ 160
It was found that the addition of fiber increases both the cracking torque and the ultimate torque, for example at 1.2% fiber the cracking torque was increased by 23%, for beams without transverse reinforcement, while the ultimate torque was increased by 66% for the beams with transverse reinforcement. The theoretical ultimate torque and experimental ultimate torque are correlated. The theoretical and experimental torsional stiffness were very close.

Wafa et. al. (1995) studied the influence of various beam parameters on the torsional behavior of prestressed high strength concrete beams with different width to depth ratios equal 1, 2, and 3. Fourteen prestressed high-strength concrete beams of rectangular section were tested under pure torsion by the means of hydraulic jack pressing against free end of the beam while the other end was held against torsional rotation. The uncracked torsional stiffness \( K_{cr} \) for each beam was calculated as the ratio of cracking torque to the measured rotation of cracking \( K_{cr}/\phi_{cr} \). The uncracked torsional stiffness increases with the decrease in the aspect ratio. After cracking, the postcracking torsional stiffness as represented by the slope of the torque-twist curve is reduced and is influenced by the aspect ratio, the concrete strength, and the amount of the torsional reinforcement. All beams exhibited mode 2 failure in which compression zones were located on the side of the beams. Tension cracks formed on one vertical face and progress up and down to form spirals with tension cracks on the other vertical face. The development of compression crack on the other vertical face characterizes the failure pattern. The torque-twist relationship and ultimate torque-aspect ratio relationship were developed from testing, skew-bending theory, space truss theory with spalling of concrete cover, and space truss theory with softening of concrete. All theories used underestimated the
torsional stiffness of all tested beams. With the increase of prestressing level and concrete
strength, the experimental cracking torsional stiffness and ultimate torsional strengths
increased. The space truss theory with softening of concrete gave the best estimate of the
torsional strength of the tested beams.

7.1.2 Justification for the pure torsion experiments

Our literature review did not reveal research on un-reinforced and reinforced polymer
modified concrete and steel fiber/polymer modified concrete beams under pure torsion.
Therefore, the need for such research is justified. The test method used in this study is
pioneer compared to other researchers methods, since the other researchers applied
torque to the tested beams through applying vertical forces to moment arms. The ability
to develop pure torsion conditions in the test beam depends on the beam end connections
and their ability to turn without inducing axial tension or secondary moments. The
objective of our experiments is to determine the behavior and torsional shear capacities of
un-reinforced and reinforced polymer modified concrete, steel fiber reinforced concrete
and steel fiber/polymer modified concrete in pure torsion. The variation in concrete shear
strain due to torsional loading will be presented along with the strains in the longitudinal
and transverse (stirrups) reinforcements. The contribution of the reinforcement to the
torsional capacity of the square specimens is presented.

7.2 Pure Torsion Test Specimens

7.2.1 Beam specimens without torsion reinforcement

Dog bone shape specimens were cast with a middle cross section dimension of 3\" x 3\"
and middle length of 24\". The beam ends had dimension of 3x9 inches and reinforced
with #4 wires to prevent them from failure and to force the beam to fail in the tested section of 3 x 3 inches as shown in Figure 7.1 (all figures are located at the end of this chapter). These beams were designed to determine the concrete behavior and capacity under pure torsion. Two beams from each concrete mix were cast with a total number of six specimens.

7.2.2 Beam Specimens with torsion reinforcement

Dog bone shape specimens were cast with a middle cross section dimension of 3 x 3 inches and length of 24 inch. The beam torsion reinforcement was designed according to provision 11.6 of ACI concrete code. The reinforcement configuration was four longitudinal steel bars (one bar at each corner) and stirrups at one-inch increment along the beam length. The wide beam ends were reinforced to protect them from failure. #4 wires (diameter= 0.225 inch) were used for both longitudinal and transverse reinforcement. Two beams from each mix were cast with a total number of six beams. Figure 7.2 shows the beam dimensions and reinforcement configuration.

7.2.3 Strain gages

Strain gages 1/4 inch long were installed at mid length of three of the central stirrups in the mid length of the beams. Also, one 1/4 inch long gages were installed at the mid span of two opposite longitudinal re-bars as shown in Figure 7.3. Strain gages 1/2 inch long were mounted on three of the four surface of the specimen at the mid length of the beams at a 45 degree inclination to the horizontal line to measure the concrete diagonal shear strains. Two of the gages were designed to measure tensile strains, while the third gage was to measure compressive strains.
7.2.4 Preparation of the beam specimens

To form the dog bone shape, two pieces of Styrofoam with length of 24 inches and thickness of 3 inch with tapered end from thickness of 3 inches to zero, over 2 inches length, was formed. One piece of the Styrofoam was placed in 3x 9 inches at the bottom of wood form and 3 inches thick concrete was poured, then the other piece of Styrofoam was placed at the top of the concrete. The two Styrofoam pieces are shown in Figure 7.4. The cast beam shape is shown in Figure 7.5.

7.3 Test Methodology

The beams were installed vertically in our material testing machine (MTS). The top end of the beam was fixed while the bottom end rotated in a rotation control mode until failure as shown in Figure 7.6. The final mode of twisted beam is shown in Figure 7.7.

7.4 Pure Torsional Behavior of Dog-Bone specimens

Torque, angle of twist, and strain gage readings were recorded automatically through the data acquisition system connected to the machine during testing. The shear developed due to torsion was calculated using the following relationships:

\[ \tau = \frac{Tt}{J} \]  
\[ J = \frac{1}{3}bt^3 \]

Where:

\( \tau \) = Shear stress due to torsion (psi).

\( T \) = Applied torque (inch-pound).
t = Beam thickness (inch).

b = Beam width (inch).

J = Torsional constant (stiffness) (in^4).

The tested beams actual dimensions are shown in Table 7.1

Table 7.1 Dimensions of tested beams

<table>
<thead>
<tr>
<th></th>
<th>PMC</th>
<th>SFRC</th>
<th>SFPMC</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>UN-RFT</td>
<td>RFT</td>
<td>UN-RFT</td>
</tr>
<tr>
<td>Dim.</td>
<td>3.125 x3.50</td>
<td>3.25x3.37</td>
<td>3 x 3.13</td>
</tr>
</tbody>
</table>

The results of two specimens from each concrete mix and each reinforcement configuration differences range between 3 and 5%. The results discussed in the subsequent sections are the average of the two tested specimens of PMC, SFRC, and SFPMC. Figure 7.8 shows an example of the torque-angle of twist relationship of two different specimens of un-reinforced PMC concrete.

7.4.1 Polymer modified concrete beam specimens

Figure 7.9 shows the relationship between torque (T) and angle of twist (Φ) and Figure 7.10 shows the relationship between torsional shear stress (v) and angle of twist (Φ) for reinforced and un-reinforced polymer modified concrete beams. Torsional shear stress was calculated for all samples and shown in the following graphs and tables because the calculations of the shear stress (v) took into consideration the minor differences in the beam specimens' dimensions as shown in Table 7.1. The cracked angle
of twist ($\Phi_{cr}$) was 2.6 degree at ultimate shear stress value of 372 psi, which corresponds to ultimate torque of 4244 inch-pound for un-reinforced beams. The ultimate angle of twist ($\Phi_{ult.}$) was 11.8 degree at shear stress value of 130 psi, which corresponds to 1483 psi.

For reinforced beams, the cracked angle of twist ($\Phi_{cr}$) was 2.6 degree at shear stress value of 432 psi which corresponds to torque of 5130 inch-pound, and the ultimate shear and ultimate torque were 476 psi and 5640 inch-pound respectively at angle of 4.3 degree. The ultimate angle of twist ($\Phi_{ult.}$) was 19.8 degree at shear stress value of 283 psi, which corresponds to 3341 inch-pound. The reinforcement increased the beam ultimate torque by 33% and angle of twist by 68%, which means that reinforcement increased the beam torsional strength and ductility. The crack pattern for un-reinforced beams was single inclined crack at one face which joined similar inclined crack on another face resulting in sudden failure as shown in Figure 7.11, while the failure cracks for reinforced beams were more than one inclined spiral crack because the reinforcement changed the beams behavior from brittle to ductile. At ultimate angle of twist ($\Phi_{ult.}$), the concrete cover spalled off between two cracks as show in Figure 7.12.

7.4.2 Steel fiber reinforced concrete beam specimens

Figure 7.13 shows the relationship between torque (T) and angle of twist ($\Phi$) and Figure 7.14 shows the relationship between torsional shear stress ($\tau$) and angle of twist ($\Phi$) for reinforced and un-reinforced steel fiber reinforced concrete beams. The cracked angle of twist ($\Phi_{cr}$) was 3.1 degree at shear stress value of 419 psi and corresponding torque ($T_{cr}$) of 4769 inch-pound for un-reinforced beams. The ultimate shear stress and ultimate torque ($T_{ult.}$) were 453 psi and 5160 inch-pound at angle of twist of 4.4 degree
and the ultimate angle of twist ($\Phi_{ult.}$) was 15.2 degree at shear stress value of 292 psi at torque of 3326 inch-pound.

For reinforced beams, the cracked angle of twist ($\Phi_{cr}$) was 4 degree at shear stress value of 550 psi and cracked torque of 6745 inch-pound. The ultimate shear stress and ultimate torque were 594 psi and 7257 inch-pound at angle of 23 degree and the ultimate angle of twist ($\Phi_{ult.}$) was 30.3 degree at shear stress value of 463 psi and torque of 5652 inch-pound. The presence of steel fiber increased the concrete torsional shear strength and torque capacities as compared with polymer modified concrete un-reinforced beams. Also, the presence of steel fiber and reinforcement together increased the beams ductility and capability to twist more, hence the torsional shear stress and torque values increased.

High strength comes from both stretching and de-bonding of steel fiber as well as contribution of the reinforcement. Beyond the ultimate shear and torque values, the fibers were pulled out in which shear and torque values started to decrease until complete failure. The reinforcement increased the beam ultimate torque by 41% and angle of twist by 99%, which means that reinforcement doubled the beam ductility. The crack pattern for un-reinforced beams was inclined spiral cracks and crack width increased with ductile failure mode until complete failure (this agrees with Graig et al., 1986 and Wafa et al., 1995) as shown in Figure 7.15. The failure cracks for reinforced beams were similar to that of un-reinforced beams until fibers pulled-out. After pull-out of the fibers, the longitudinal reinforcement resisted torsion and started to bend until final failure as shown in Figure 7.16.

7.4.3 Steel fiber/polymer modified concrete beam specimens

Figure 7.17 shows the relationship between torque ($T$) and angle of twist ($\Phi$), and
Figure 7.18 shows the relationship between torsional shear stress ($\tau$) and angle of twist ($\phi$) for reinforced and un-reinforced steel fiber/polymer modified concrete beams. The cracked angle of twist ($\phi_{cr}$) was 3.1 degree at shear stress value of 383 psi and corresponding cracked torque ($T_{cr}$) of 4720 inch-pound. The ultimate shear stress and ultimate torque ($T_{ult}$) were 414 psi and 5102 inch-pound at angle of twist of 5.5 degree and the ultimate angle of twist ($\phi_{ult}$) was 18 degree at shear stress value of 350 psi and torque of 4513 inch-pound.

For reinforced beams, the cracked angle of twist ($\phi_{cr}$) was 3.3 degree at shear stress value of 405 psi and cracked torque of 5211 inch-pound. The ultimate shear stress and ultimate torque were 519 psi and 6669 inch-pound at angle of 27 degree and the ultimate angle of twist ($\phi_{ult}$) was 34 degree at shear stress value of 366 psi and torque of 4714 inch-pound. The presence of steel fiber and polymer together increased the concrete torsional shear and torque capacities as compared with polymer modified concrete. The behavior of steel fiber/polymer modified concrete beams was similar to those of steel fiber reinforced concrete, however the presence of steel fiber and polymer together increased the beam ductility and toughness under torsional loading. The reinforcement increased the beam ultimate torque by 31% and angle of twist by 89%. The crack pattern for un-reinforced beams was inclined spiral cracks as shown in Figure 7.19. The failure cracks for reinforced beams were similar to that of steel fiber reinforced concrete beams as shown in Figure 7.20.
7.5 Torsional Behavior of Un-Reinforced and Reinforced Specimens

7.5.1 Torsional toughness of beam specimens

The toughness of all the beams was calculated as the area under the torque-angle curve up to the ultimate torque level (the ascending portion of the curve). The steel fiber/polymer modified concrete beams gave the highest torsional toughness compared to both steel fiber concrete and polymer modified concrete for both un-reinforced and reinforced beams (Table 7.2).

7.5.2 Torsional ductility

The ductility of structural member under torsional loading is its ability to twist before it failed because higher ductility exhibited a larger angle of twist. Table 7.3 shows the angle of twist for un-reinforced and reinforced PMC, SFRC, and SFPMC beams under torsional loading.

7.5.3 Torsional behavior of un-reinforced beam specimens

The ultimate torque (Figure 7.21) and the ultimate shear stress (Figure 7.22) of steel fiber reinforced concrete beams were 5160 inch-pound and 453 psi respectively with increase over 22 of those of polymer modified concrete beams which were 4244 inch-pound and 372 psi. The ultimate torque (Figure 7.21) and ultimate shear stress (Figure 7.22) of steel fiber/polymer modified concrete beams were 5102 inch-pound and 414 psi with increase of 21% and 11% respectively over those of polymer modified concrete beams. The cracked angle of twist increased from 2.6 degree of polymer modified concrete beams to 3.1 degree for both steel fiber reinforced concrete and steel fiber/polymer modified concrete beams. The ultimate angle of twist improved from 11.8
degree for polymer modified concrete to 15.2 for steel fiber reinforced concrete with increase of 29% and to 18 degree steel fiber/polymer modified concrete beams with increase of 53%. Steel fiber reinforced concrete beams gave the highest torsional shear stress value while the steel fiber/polymer beams gave the highest angle of twist. The addition of steel fiber and steel fiber and polymer together increased concrete beam toughness (Table 7.2) and ductility (Table 7.3) due to the energy absorption of the gradual steel fiber pull-out from the concrete matrix.

### 7.5.4 Torsional behavior of reinforced beam specimens

The ultimate torque (Figure 7.23) and the ultimate shear stress (Figure 7.24) of steel fiber reinforced concrete beams were 7257 inch-pound and 594 psi with increase of 29%
and 25% respectively over those of reinforced polymer modified concrete beams which were 5640 inch-pound and 476 psi. Also, the ultimate torque (Figure 7.23) and the beams was lower than that of SFRC reinforced beams which is not expected, it seems that the SFRC specimen had weak point in which crack started earlier that SFRC specimens. The cracked angle of twist increased from 2.6 degree for polymer modified concrete beams to 3.1 for both steel fiber reinforced concrete and steel fiber/polymer modified concrete beams. Also the ultimate angle of twist improved from 18 degree for polymer modified concrete to 30.3 for steel fiber reinforced concrete with increase of 68 % and to 34 degree for steel fiber/polymer modified concrete beams with increase of 89 %. The

Table 7.3 Ultimate angle of twist ($\Phi_{ult}$) of PMC, SFRC, and SFPMC beams under torsional loading

<table>
<thead>
<tr>
<th></th>
<th>PMC</th>
<th>SFRC</th>
<th>SFPMC</th>
</tr>
</thead>
<tbody>
<tr>
<td>Un-reinforced beams</td>
<td>11.8° (0.206 rad.)</td>
<td>15.2° (0.265 rad.)</td>
<td>18° (0.314 rad.)</td>
</tr>
<tr>
<td>Reinforced beams</td>
<td>19.8° (0.246 rad.)</td>
<td>30.3° (0.529 rad.)</td>
<td>34° (0.593 rad.)</td>
</tr>
<tr>
<td>% increase of $\Phi_{ult}$ for U-RTF$^1$ and RFT$^2$ beams</td>
<td>68%</td>
<td>99%</td>
<td>89%</td>
</tr>
<tr>
<td>Ratio of $\Phi_{ult}$ of U-RFT beams w.r.t PMC beams</td>
<td>-</td>
<td>29%</td>
<td>53%</td>
</tr>
<tr>
<td>Ratio of $\Phi_{ult}$ of RFT beams w.r.t PMC beams</td>
<td>-</td>
<td>55%</td>
<td>72%</td>
</tr>
</tbody>
</table>

$^1$ U-RFT = Un-reinforced beams
$^2$ RFT = Reinforced beams
percentages of increase of concrete torsional shear stress was lower, for reinforced beams, than un-reinforced beams because longitudinal and transverse (stirrups) reinforcement contributed in carrying torque and torsional shear stress which will be discussed in detail later.

7.6 Diagonal Strain Measurements Due to Torsion in Concrete

7.6.1 Polymer modified concrete beam specimens

Figure 7.25 shows the relationship between the concrete shear stress due to torsional loading and diagonal shear strains for un-reinforced and reinforced polymer modified concrete beams. The strains were measured by two diagonal 1/2 inch long gages placed at the mid-side of the beam. The relation is linear up to the ultimate shear stress and ultimate angle of twist for un-reinforced beams. Increasing the angle of twist beyond the ultimate torque value, the concrete start to loose its shear strength and diagonal shear strains due to crack development until complete beam failure at which the relationship was similar to loading-unloading trend.

The ultimate diagonal shear strain for un-reinforced beams was 0.000226 at a shear stress of 372 psi, however those two values were 0.000129 and 184 psi at the end of the test. The shear modulus (G) is simply the slope of the un-reinforced shear stress-diagonal shear strain linear part of the curve with a value of 1653540 psi.

The torsional shear stress-shear stain relationship for reinforced concrete was non-linear because the reinforcement provides more ductility and toughness for the beams. The ultimate diagonal shear strain and shear stress were 0.00043 and 476 psi, while those two values were 0.00025 and 260 psi at the end of the test. The steel reinforcement
increased the ultimate diagonal shear strain by 90% and diagonal shear stress by 28% that represents their contribution in providing more strength, ductility (Table 7.3), and beam toughness as discussed before section 7.5.1.

7.6.2 Steel fiber concrete beam specimens

Figure 7.26 shows the relationship between the concrete shear stress due to torsion loading and diagonal shear strains for un-reinforced and reinforced steel fiber concrete beams. The relation is linear up to shear stress of 422 psi and diagonal shear strain of 0.000216 for un-reinforced beams at which micro-cracks started to develop, beyond this point the steel fibers start to stretch and de-bond until a shear stress of 430 psi and diagonal shear strain of 0.0002297. The steel fiber was responsible of increasing shear stress and diagonal shear strain after microcracks development to an ultimate shear stress and diagonal shear strain of 453 psi and 0.00025 respectively at which concrete cracked. By twisting the beams after the ultimate stress level had been reached, the steel fibers started to pull-out and the crack width increased which reduced both shear stress and diagonal shear strain values to 403 psi and 0.00022 respectively at the end of the test.

The linear shear modulus (G) value was 1953704 psi.

The torsional shear stress-shear stain relationship for reinforced beams was bi-linear because the reinforcement provides additional ductility and toughness for the beams beyond the development of initial cracks. The diagonal shear strain and shear stress were 0.00071 and 594 psi at ultimate level and were 0.00057 and 501 psi at the end of the test. The steel reinforcement increased the ultimate diagonal shear strain by 184% and increased torsional shear stress by 31%, which represents its contribution in providing ductility (Table 7.3) and beam toughness as discussed before (section 7.5.1).
7.6.3 Steel fiber/polymer modified concrete beam specimens

Figure 7.27 shows the relationship between the concrete shear stress due to torsional loading and diagonal shear strains for un-reinforced and reinforced steel fiber/polymer modified concrete beams. The relation was linear up to shear stress of 361 psi and diagonal shear strain of 0.0001885 for un-reinforced beams at which micro-cracks started to develop, beyond this point the steel fiber start to stretch and de-bonding until shear stress of 370 psi and diagonal shear strain of 0.000205. Steel fiber increased concrete shear stress and diagonal shear strain to 414 psi and 0.000263 respectively. By twisting the beams after the ultimate stress level had been reached, the steel fibers started to pull-out and the crack width increased which reduced both shear stress and diagonal shear strain values to 333 psi and 0.0002 respectively at the end of the test. The shear modulus (G) was 1918662 psi.

The torsional shear stress-shear strain relationship for reinforced beams was bi-linear because the reinforcement provides more ductility and toughness for the beams. The diagonal shear strain and shear stress were 0.00084 and 519 psi at ultimate level and were 0.00055 and 440 psi at the end of the test. The steel reinforcement increased the ultimate diagonal shear strain by 219 % that represents the contribution of them in providing ductility and toughness and the contribution of shear strength contribution of reinforcement was 25 % as stated before.

7.6.4 Torsional shear stress and diagonal strain of un-reinforced beam specimens

Figure 7.28 shows the relationship between torsional shear stress and diagonal strains for un-reinforced beams. Due to the condition of pure torsional loading on the square beam specimens, pure shear stress produces two equal and opposite strains in the
diagonal direction at 45 degree. The steel fiber reinforced concrete gave the highest shear stress value of 453 psi as compared with 414 psi of steel fiber/polymer modified concrete beams and 372 psi of polymer modified concrete beams. The steel fiber reinforced concrete shear stress increased by 22% as compared with polymer modified concrete beams while that of steel fiber/polymer modified concrete beams was higher by 11%. Moreover, the ultimate shear strain of steel fiber reinforced concrete beams was 0.00025, while that value was 0.000226 for polymer modified and 0.000263 for steel fiber/polymer modified concrete. The percentage of increase for diagonal shear strains were 11% and 16% for steel fiber reinforced concrete and steel fiber/polymer modified concrete beams respectively as compared with polymer modified concrete beams. It can be concluded that steel fiber increased the shear stress and diagonal shear strains since the steel fiber arrest and bridge cracks. The presence of polymer and steel fiber together increased the shear stress by a lower percentage than that of steel fiber concrete because polymer increases concrete air content, moreover the ultimate diagonal shear strain was higher than that of steel fiber reinforced concrete, which means that the presence of polymer and steel fiber together gave the highest toughness behavior (Table 7.2).

7.6.5 Torsional shear stress and diagonal strain for reinforced specimens

Figure 7.29 shows the relationship between torsional shear stress and diagonal strains for reinforced beams. The steel fiber reinforced concrete gave the highest shear stress value of 594 psi as compared with 518 psi of steel fiber/polymer modified concrete beams and 476 psi of polymer modified concrete beams. The steel fiber reinforced concrete shear stress increased by 25% as compared with reinforced polymer modified concrete beams while that of reinforced steel fiber/polymer modified concrete beams was

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.
higher by 9%. The ultimate shear strain of steel fiber reinforced concrete beams was 0.00071, while that value was 0.00043 for polymer modified and 0.00084 for steel fiber/polymer modified concrete. The increase percentage of diagonal strains were 65% and 95% for steel fiber concrete and steel fiber/polymer modified concrete beams respectively as compared with reinforced polymer modified concrete beams. It can be concluded that the presence of torsional reinforcement increased torsion capacity and steel fiber increased the shear stress and diagonal shear strains since the steel fiber arrests and bridges cracks, while the presence of polymer and steel fiber together decreases the ultimate shear stress to a lower value than that of steel fiber concrete because polymer increase concrete air content. The ultimate diagonal strain was higher than that of steel fiber reinforced concrete, which means that the presence of polymer and fiber together gave the highest toughness as discussed before section 7.5.1.

7.7 Reinforcement Behavior Due to Torsion

7.7.1 Longitudinal reinforcement behavior

Figure 7.30 shows the relationship between the applied torque and tension strains in the longitudinal steel bars. The ultimate tension strain in the longitudinal reinforcements for PMC, SFRC, and SFPMC were 0.00089, 0.0005, and 0.000388 at torque values of 5640, 7257, and 6669 inch-pound respectively. The ratio between the tension force in the longitudinal reinforcements of SFRC and PMC was 56%, while that ratio was 44% for SFPMC and PMC. Thus, 56% of the tension was carried by longitudinal reinforcement for SFRC and 44% was carried by steel fibers, also 44% of the tension force was carried by longitudinal reinforcement for SFPMC and 56% was carried by steel fibers as shown
in Table 7.4. The contribution of steel fiber in carrying the tension force for SFRC was less than that of SFPMC because of the more ductility (Table 7.3) provided by both steel fibers and polymer together.

7.7.2 Transverse reinforcement (stirrups) behavior

Figure 7.31 shows the relationship between the applied torque and tension strains in the transverse reinforcements (stirrups). The ultimate tension strain in the transverse reinforcements for PMC, SFRC, and SFPMC were 0.00025, 0.000197, and 0.000187 at torque values of 5640, 7257, and 6669 inch-pound respectively. The ratio between the tension force in the transverse reinforcements of SFRC and PMC was 62%, while that ratio was 75% for SFPMC and PMC. Thus, 79% of the tension force was carried by transverse reinforcement for SFRC and 21% was carried by steel fibers, also 75% of the tension force was carried by longitudinal reinforcement for SFPMC and 25% was carried by steel fibers as shown in Table 7.4.

Table 7.4 Contribution of longitudinal reinforcements, Stirrups, and steel fibers in carrying tension force with respect to PMC beams

<table>
<thead>
<tr>
<th></th>
<th>Ratio of Tension force with respect to PMC beams</th>
<th>Ratio of Tension force with respect to PMC beams</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Rebars</td>
<td>Steel fibers</td>
</tr>
<tr>
<td>SFRC</td>
<td>56%</td>
<td>44%</td>
</tr>
<tr>
<td>SFPMC</td>
<td>44%</td>
<td>56%</td>
</tr>
</tbody>
</table>

Table 7.6 shows the concrete experimental and theoretical shear stress values due to...
torsional loading. The shear stress values which were based on the formula \( \tau = \gamma G \) were higher than the experimental values since this formula assumes that shear-strain relation is linear up to the ultimate shear stress level, but in fact the relation is linear to a point below the ultimate values and deviate from linearity beyond this point because of the micro-cracks growth. The theoretical formula assumes that the material is homogeneous, but concrete has different behavior of non-homogeneous material because of the presence of micro-cracks.

| Table 7.5 Experimentally measured shear modulus, \( G \), (psi) for PMC, SFRC, and SFPMC beams |
|---------------------------------|----------------|----------------|----------------|
|                                | PMC            | SFRC           | SFPMC          |
| Shear Modulus (G), psi        | 1,653,540      | 1,953,704      | 1,918,662      |

7.8 Experimental Versus Predicted Concrete Code Torsion

The design for torsion according to the ACI concrete code provision 11.6 neglects the central portion of a solid beam which idealizes the beam as a tube. Torsion is resisted through a constant shear flow \( q \) (force per unit length of wall centerline) acting around the centerline of the tube. The ultimate cracked beam torque for un-reinforced beams can be calculated from the formula:

\[
T_{cr.} = 4\sqrt{f_c} \left( \frac{A_{cp}}{P_{cp}} \right) \quad \text{(ACI, 1999)}
\]

Where:
220

$T_{cr}$ = Cracked concrete torque (inch-pound) in a beam.

$A_{cp}$ = area enclosed by outside perimeter of concrete cross section (in$^2$).

$P_{cp}$ = Outside perimeter of concrete cross-section (inch).

The ultimate torque also be calculated for reinforced beams using the area of longitudinal steel which resists torsional loading as follows:

$$T = \frac{2A_o A f_{st}}{2(x_o + y_o) \cot \theta} \quad \text{(Fanella and Rabbat, 1997, ACI 1999)} \quad (7.4)$$

Where $T$=cross section torque capacity, $A_o = \frac{2A_{st}}{3}$, $A_l$= longitudinal area of all steel bars resisting torsion, $f_{st}$= yield stress of longitudinal reinforcement, $x_o$ and $y_o$ are center-to-center length of closed stirrup, and $\theta$= 45 degree is the cracks inclination for pure torsion.

Again the ultimate torque may be calculated for reinforced beams by the area of transverse steel (stirrups) using the following formula:

$$T = \frac{2A_o A t f_{st}}{s} \cot \theta \quad \text{(ACI, 1999)} \quad (7.5)$$

where $T$=cross section torque capacity, $A_o = \frac{2A_{st}}{3}$, $A_t$=Transverse area of one leg of U-stirrup, $f_{st}$= yield stress of transverse reinforcement, $s$=spacing between stirrups, and $\theta$= 45 degree is the cracks inclination for pure torsion.

The maximum closed stirrups spacing, in a concrete beam under torsion, is the smallest of $P/8$, $d/2$, or 12 inches.

Where:

$P$ = The perimeter of centerline of outermost closed transverse reinforcement.

d = Distance from extreme compression fiber to centroid of longitudinal tension reinforcement.
The tested concrete section had one longitudinal #4 wire bar at each corner and #4 wire stirrups at spacing of one inch increment. Both longitudinal and transverse reinforcement did not reach yield strain during testing, but they experienced strain values lower than steel yield strains. The beam torque capacity was calculated from the previous formulas but the actual tension stress \( f_s \) calculated from the experimentally measured strains in both longitudinal and transverse reinforcements was substituted for the yield stress, \( f_y \), of the steel reinforcement.

\[
T = \frac{2A_s A_f f_s}{2(x_o + y_o)\cot\theta} \quad \text{(for longitudinal reinforcement)} \quad (7.6)
\]

Where:

\( f_s \) = \( \varepsilon_s E \) (psi).

\( \varepsilon_s \) = Measured strains in the longitudinal reinforcement.

\( E \) = Steel reinforcement modulus of elasticity = \( 29 \times 10^6 \) psi

The calculated torque capacities of PMC, SFRC, and SFPMC were 7374, 4052, and 3191 inch-pound respectively. Also the beam torque capacity was calculated according to ACI concrete code from the following formula.

\[
T = \frac{2A_s A_f f_s}{s \cot\theta} \quad \text{(for transverse reinforcement)} \quad (7.7)
\]

Where:

\( f_s \) = \( \varepsilon_s E \) (psi).

\( \varepsilon_s \) = Measured strains in the transverse (stirrups) reinforcement.

\( E \) = Steel reinforcement modulus of elasticity = \( 29 \times 10^6 \) psi

The torque capacities of PMC, SFRC, and SFPMC calculated from this formula were 4712, 3332, and 3389 inch-pound respectively. The smallest calculated value governs.
From Table 7.6, it could be concluded that the code cracked and ultimate torque values were lower than experimental values because the presence of polymer in PMC beams reduced micro-crack growth and increased the beam crack capacity. Also the presence of steel improved the cracked and ultimate torque for the beams. The ultimate calculated torque values using ACI code equations were lower than measured values for all beams because the ACI code method for calculating ultimate torque does not include the contribution of the concrete, however the presence of polymer in PMC specimens reduced micro-crack growth which increases the concrete contribution in resisting torque. The presence of steel fibers in SFRC and SFPMC specimens increased the concrete capacity to resist torsional loading. The design method suggested by ACI code does not include the contribution of concrete or steel fibers along with the longitudinal and transverse reinforcement.

7.9 Conclusions

From the previous results and discussion, the following conclusions could be presented:

1. Steel fiber increased the diagonal shear stress by 23% for SFRC and 11% for SFPMC under torsional loading, also torque was increased by 22% for SFRC and 21% for SFPMC as compared with polymer modified concrete beams without fibers.

2. Steel fiber and polymer gave highest diagonal shear strains, because they provided ductility and toughness to beams.

3. Steel reinforcement improved the ultimate torque compared to un-reinforced beams by
Table 7.6 Experimental and code cracked and ultimate torque capacities (inch-pound)

<table>
<thead>
<tr>
<th></th>
<th>PMC $f_c = 5863$ psi</th>
<th>SFRC $f_c = 5842$ psi</th>
<th>SFPMC $f_c = 5832$ psi</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>U-RFT $^1$</td>
<td>RFT $^2$</td>
<td>U-RFT</td>
</tr>
<tr>
<td>Exp. $T_{cr}$</td>
<td>4244</td>
<td>5130</td>
<td>4769</td>
</tr>
<tr>
<td>Code $T_{cr}$</td>
<td>2765</td>
<td>-</td>
<td>2199</td>
</tr>
<tr>
<td>% increase</td>
<td>53%</td>
<td>-</td>
<td>117%</td>
</tr>
<tr>
<td>Exp. $T_{ult}$</td>
<td>4244</td>
<td>5640</td>
<td>5160</td>
</tr>
<tr>
<td>Code $T_{ult}$</td>
<td>-</td>
<td>4713</td>
<td>-</td>
</tr>
<tr>
<td>% increase</td>
<td>-</td>
<td>20%</td>
<td>-</td>
</tr>
</tbody>
</table>

$^1$ U-RFT = Un-reinforced beams  
$^2$ RFT = Reinforced beams

33%, 41%, and 31% for PMC, SFRC, and SFPMC respectively.

4. The contribution of steel fiber in carrying tension forces in both longitudinal and transverse reinforcements were 56% and 25% respectively when compared to PMC specimens without steel fibers.

5. Half of the percentages of steel fiber contribution in carrying tension forces in both stirrups and re-bars (Table 7.4) may partially replace stirrups and rebars with safety factor of 2. A percentage of 22% of stirrups area may be replaced by 1% steel fiber, or 28% of stirrups area may be replaced by 5% solids of polymer and 1% of steel fibers. Again, A percentage of 10.5% of rebars area may be replaced by 1% steel fiber, or 12.5% of
stirrups area may be replaced by 5% solids of polymer and 1% of steel fibers for beams subjected to pure torsion.

67. ACI code torsion design was conservative since the torsion design method does not take the contribution of concrete and steel fibers into consideration.
Figure 7.1 Un-reinforced dog bone specimen

Figure 7.2 Reinforced dog bone specimen

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.
Figure 7.3 Strain gages configuration for the reinforcing cage of dog bone specimen

Figure 7.4 Dog bone beams form
Figure 7.5 Dog bone beam before and after removing the Styrofoam

Figure 7.6 Pure torsion test set-up using the MTS machine
Figure 7.7 Twisted beam due to pure torsional loading

Figure 7.8 Torque versus angle of twist for Un-reinforced polymer modified concrete beams for two tested specimens
Figure 7.9 Torque versus angle of twist for polymer modified concrete beams

Figure 7.10 Apparent torsional shear stress versus angle of twist for polymer modified concrete beams
Figure 7.11 Failure pattern for un-reinforced polymer modified concrete beams

Figure 7.12 Failure pattern for reinforced polymer modified concrete beams
Figure 7.13 Torque versus angle of twist for steel fiber concrete beams

Figure 7.14 Apparent torsional shear stress versus angle of twist for steel fiber concrete beams
Figure 7.15 Failure pattern for un-reinforced steel fiber concrete beams

Figure 7.16 Failure pattern for reinforced steel fiber reinforced concrete beams
Figure 7.17 Torque versus angle of twist for steel fiber/polymer modified concrete beams

Figure 7.18 Torsional shear stress versus angle of twist for steel fiber/polymer modified concrete beams
Figure 7.19 Failure pattern for un-reinforced steel fiber/polymer modified concrete beams

Figure 7.20 Failure pattern for reinforced steel fiber/polymer modified concrete beams
Figure 7.21 Torque versus angle of twist for un-reinforced beams

Figure 7.22 Apparent torsional shear stress versus angle of twist for un-reinforced beams
Figure 7.23 Torque versus angle of twist for reinforced beams

Figure 7.24 Apparent torsional shear stress versus angle of twist for reinforced beams

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.
Figure 7.25 Apparent torsional shear stress versus concrete shear strain relationship for polymer modified concrete beams

Figure 7.26 Apparent torsional shear stress versus concrete shear strain for steel fiber reinforced concrete beams
Figure 7.27 Apparent torsional shear stress versus concrete shear strain for steel fiber/polymer modified concrete beams

Figure 7.28 Apparent torsional shear stress versus concrete shear strain for un-reinforced beams
Figure 7.29 Apparent torsional shear stress shear versus concrete shear strain for reinforced beams

Figure 7.30 Torque versus tensile strains in the longitudinal reinforcements

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.
Figure 7.31 Torque versus tensile strains in stirrups
CHAPTER 8

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

8.1 Summary

The current research is an experimental study for the behavior of polymer modified concrete, steel fiber reinforced concrete, and steel fiber/polymer modified concrete beam structures. The material properties were characterized for compression, tension, and modulus of rupture due to flexure loadings. The pull-out forces of reinforcing bars and bond stresses were investigated. Slender beams with aspect ratio of 3 were tested for shear and flexure to study the behavior PMC, SFRC, and SFPMC slender beams. Dog bone shaped reinforced and un-reinforced specimens with square sections were tested under pure torsional loading. Half I-beams were tested under combined loadings of bending, shear, and torsion. Eccentric lateral loads were applied to the slender beams and half I-beams to study their behavior under simulated wind or earthquake lateral loads. The results presented in this research are for the pre-defined dimensions and reinforcement configurations of the tested specimens, as shown in the figures and graphs presented.

8.2 Conclusions

Investigation of polymer modified concrete (PMC), steel fiber reinforced concrete (SFRC), and steel fiber/polymer modified concrete (SFPMC) were presented. The
properties of PMC, SFRC, and SPMC materials were characterized under compression, splitting tension, and flexure loading. Also, the pull-out forces of reinforcing bars and bond stresses were investigated. Adding 1% by volume steel fiber and 5% solids of polymer to regular concrete was the best concrete mix since the combination increased the ultimate compression strain from 0.003 for regular concrete to 0.0065, and ultimate tension strain from 0.0003 to 0.002 which improved concrete toughness and ductility under compressive and tensile forces. Also, the behavior under flexural loading was similar to those of compression and tension since the concrete ultimate strain was improved from 0.00017 to 0.00022. The presence of steel fiber and polymer could increase the concrete strengths under previous loading, only if water/cement ratio kept small at 0.36, the coarse aggregate to fine aggregate ratio should be at least 0.55:0.45 to provide enough strength to concrete, and cement content should be at least 9 bags (846 pounds) per cubic yard to provide enough bond between concrete and steel fibers.

Structural members with steel fibers could resist about 70% of the ultimate loading after cracks. The bond stress between concrete and reinforcing steel bars under tension was higher than that calculated theoretically by a factor ranges between 1.23 and 1.5.

The behavior of slender beams with aspect ratio of 3 under shear loading was reported. It was found that the steel fiber or steel fiber and polymer together increased ultimate shear strength from 580 psi to 606 psi as compared with polymer modified concrete beams. The reinforcement ratio has an influence on the concrete shear stress and strain. The beams which were doubly reinforced have diagonal concrete shear strains higher than singly reinforced beams by about 6% for PMC, 18% for SFRC, and 21% for SPMC.
Steel fiber and steel fiber and polymer increased beams diagonal shear stress and strains because steel fiber bridges cracks.

The maximum shear after crack was about 72% of that value at crack initiation for PMC beams, while the maximum shear after cracks were the same as that at crack initiation for both SFRC and SFPMC beams. The concrete diagonal shear strains for PMC beams were 55% and 68% of those values at crack initiation for singly and doubly. The concrete shear strains of SFRC slender beams was 94% of that value after because steel fibers carried part of the stresses and strains. Steel fiber/polymer modified concrete beams had more toughness and ductility more than other beams, at which the concrete shear strains after crack was 97% of that value before crack.

Moreover, steel fiber may replace some of the stirrups in beams which may result in substantial savings in materials and labor. Steel fiber increases diagonal tensile strains by about 39% and 55% for singly and doubly reinforced slender beams respectively as compared to those of polymer modified concrete beams and reduced both stirrups and rebar tensile strains by 64% and 35% respectively. The presence of both steel fiber and polymer together improves concrete ductility, impermeability and resistance to micro-cracking which is required for structural members that are subject to fatigue and impact loading and environmental hazards, such as highway bridges. The combination of steel fiber and polymer increased concrete diagonal tensile strains by an average values of 42% and 62% for singly and doubly reinforced slender beams respectively as compared to polymer modified concrete beams and reduced both stirrups and rebar tensile strains by 66% and 37% respectively.
Polymer modified concrete, steel fiber reinforced concrete, and steel fiber-polymer modified concrete short half I-beams were tested under combined loadings of bending, shear, and torsion. The presence of steel fiber and steel fiber and polymer together improved concrete shear strengths and performance since the steel fibers provided ductility to the concrete, while adding 5% of polymer solids per weight of cement reduced micro cracking and a measure of plasticity in concrete. The steel fiber and fibers and polymer together increased the concrete combined shear strain by 22% and 38.6% when compared to PMC half I-beams. Steel fiber and steel fiber and polymer mixes provided additional ductility and toughness under combined bending, shear, and torsional loadings. A percentage of 21% of stirrups area may be replaced by 1% steel fibers, while 1% steel fibers and 5% solids of polymers may replace 43% of stirrups area by cement weight. Again, 23% of longitudinal reinforcement area may be replaced by 1% steel fibers, while 33% of longitudinal reinforcement area may be replaced by 1% steel fibers and 5% solids of polymers by cement weight. The compression steel reinforcement should be transformed using a modular ratio, n for experimental beams, however 2n may be used according to ACI code in actual design to account for long term creep and shrinkage of concrete. The failure interaction mode of our half I-beams was in combined shear and torsion. The presence of polymer with steel fiber increases the bond between the fibers and the concrete matrix which decreases micro-cracking.

Lateral loads due to wind or earthquake were simulated by applying eccentric lateral loads to shallow and wide beams, and half I-beams were reported. The crack patterns due to combined loading were spiral cracks as proposed by skew bending theory and space truss analogy theory. Polymer modified concrete, steel fiber reinforced concrete,
and steel fiber/polymer modified concrete shallow wide and half I-beams were tested under combined loadings of bending, shear, and torsion due to eccentric lateral loads. The contribution of steel fiber and steel fiber and polymers was higher for the case of lateral loading application than for the case of vertical loading application in carrying part of the applied loads. Slender beams could carry about 70% of the shear loads when exposed to lateral loads. The half I-beams exposed to eccentric lateral loads could resist about 87% of the gravity maximum shear stress due to torsion for polymer modified concrete beams, 99% for steel fiber concrete beams, and 92% for steel fiber/polymer modified concrete beams. Also, slender beams could resist about 70% of the applied loads after cracks.

Steel fibers carried 36% of stirrups’ forces, while steel fibers and polymer carried 43% for laterally loaded shallow and wide beams. Again, steel fibers carried 40% of rebars forces, while steel fibers and polymers carried 44% for laterally loaded shallow and wide beams.

Steel fibers carried 37% of stirrups’ forces, while steel fibers and carried 53% for laterally loaded I-beams. Again, steel fibers 50% of re-bars, while steel fibers and polymers carried 58% for laterally loaded half I-beams.

The presence of polymer and fiber together improves concrete ductility and reduces cracks. Steel fiber improves diagonal tension strain as compared by polymer modified concrete and reduces stirrups and rebars tensile strains since adding 5% of polymer solids by cement weight increased the bond between the steel fiber and the concrete matrix. It could be concluded that steel fiber or steel fiber and polymer together may partially substitute the transverse reinforcement which saves labor cost and time. The measured
strain in the longitudinal reinforcements for both shallow wide beams and half I-beams loaded laterally were much lower than the calculated values because the steel fibers helped in carrying moments in both longitudinal and transverse directions.

Dog bone shaped reinforced and un-reinforced specimens with square sections were tested under pure torsional loading. Steel fiber increased the diagonal shear stress by 23% for SFRC and 11% for SFPMC under torsional loading; also torque was increased by 22% for SFRC and 21% for SFPMC as compared with polymer modified concrete beams without fibers. Steel fiber and polymer gave highest diagonal shear strains because they provided ductility and toughness to beams. Steel reinforcement increased the ultimate torque compared to un-reinforced beams by 33%, 41%, and 31% for PMC, SFRC, and SFPMC respectively. The contribution of steel fiber in carrying tension forces in both longitudinal and transverse reinforcements were 56% and 25% respectively when compared to PMC specimens without steel fibers. A percentage of 22% of stirrups area may be replaced by 1% steel fiber, or 28% of stirrups area may be replaced by 5% solids of polymer and 1% of steel fibers. Dog bone shaped reinforced and un-reinforced specimens with square sections were tested under pure torsional loading. A percentage of 10.5% of rebars area may be replaced by 1% steel fiber, or 12.5% of stirrups area may be replaced by 5% solids of polymer and 1% of steel fibers. Theoretical methods for calculating torsional shear gave close approximate values to the measured shear strengths due to torsion obtained from experiment. ACI code torsion design was conservative since the torsion design method does not take the contribution of concrete and steel fibers into consideration.
8.3 Recommendations

Steel fiber and steel fiber and polymer improved concrete properties under all possible loadings which structural members can be exposed to. Steel fibers may partially replace shear or torsion transverse reinforcements (stirrups). Polymer provides ductility and reduces micro cracks of concrete which improve concrete durability: it also increases the bond between the steel fibers and concrete matrix. The presence of steel fiber and steel fibers and polymers help structural members function after cracks.

If the specimen dimensions are scaled up, the steel fiber and aggregate should be increased in length and size. Therefore, the results for full scale structural testing may have to be validated.

8.4 Suggestions for Future Research

Experimental work had been accomplished for this study for slender beams with aspect ratio of 3. The collected data and results may be modeled by 3-D finite element analysis to predict the behavior of any other structural member which may be exposed to combinations of loadings and with varying aspect ratio. Also, the effect of steel fibers configurations and aspect ratios on the behavior of slender beams under combined loading may be studied.
BIBLIOGRAPHY

1. Building code requirements for structural concrete (ACI318-99) and commentary (ACI318R-99), American Concrete Institute, Framington Hills, MI.


8. DOW Chemical Company Publication 1 “The Use Of Dow Modifier A In Bridge Deck Overlayments.”

9. DOW Chemical Company Publication 2 “A Handbook on Portland Cement Concrete and Mortar Containing Styrene/Butadiene Latex”


15. Fiber Reinforced Concrete (1999), Michigan State University, Computer CD.


38. “Measurement of Properties of Fiber Reinforced Concrete”, Report by ACI Committee 544, ACI 544-2R-89


57. "Specifications For Structural Concrete For Buildings, ACI 301-89", Report by ACI 301.


APPENDIX A

CONSIDERATIONS FOR VARIOUS CONCRETE MIX DESIGN

A.1 Mix Proportions for One Cubic Yard Batch of concrete

A1.1. Plain Concrete

<table>
<thead>
<tr>
<th>Computed Batch Weight in Pounds</th>
<th>weight per one cubic foot in Pounds</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/8 Coarse Aggregate (SSD)</td>
<td>1735</td>
</tr>
<tr>
<td>Fine Aggregate (SSD)</td>
<td>1400</td>
</tr>
<tr>
<td>Water</td>
<td>317</td>
</tr>
<tr>
<td>Cement, Type I</td>
<td>658</td>
</tr>
<tr>
<td>Entrapped Air: 2%</td>
<td></td>
</tr>
</tbody>
</table>

W/C= 0.48

A.1.2. Steel Fiber Reinforced Concrete

<table>
<thead>
<tr>
<th>Computed Batch Weight in Pounds</th>
<th>Weight per one cubic foot in Pounds</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/8 Coarse Aggregate (SSD)</td>
<td>1735</td>
</tr>
<tr>
<td>Fine Aggregate (SSD)</td>
<td>1400</td>
</tr>
<tr>
<td>Water</td>
<td>309</td>
</tr>
<tr>
<td>Cement, Type I</td>
<td>658</td>
</tr>
<tr>
<td>Water reducer Superplasticizer</td>
<td>6.58</td>
</tr>
<tr>
<td>1% Steel Fiber</td>
<td>132.3</td>
</tr>
<tr>
<td>Entrapped Air: 2%</td>
<td></td>
</tr>
</tbody>
</table>

W/C= 0.47
### A.1.3. Steel Fiber /Polymer Modified Concrete

#### A.1.3.1. Adding 2.5 % solids of polymer

<table>
<thead>
<tr>
<th>Computed Batch Weight in Pounds</th>
<th>weight per one cubic foot in Pounds</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/8” Coarse Aggregate (SSD)</td>
<td>1735</td>
</tr>
<tr>
<td>Fine Aggregate (SSD)</td>
<td>1400</td>
</tr>
<tr>
<td>Water</td>
<td>299.4</td>
</tr>
<tr>
<td>Cement, Type I</td>
<td>658</td>
</tr>
<tr>
<td>Modifier A</td>
<td>16.45</td>
</tr>
<tr>
<td>“Styrene Butadiene Rubber” (2.5%)</td>
<td></td>
</tr>
</tbody>
</table>

W/C=0.455

#### A.1.3.2. Adding 5 % solids of polymer

<table>
<thead>
<tr>
<th>Computed Batch Weight in Pounds</th>
<th>weight per one cubic foot in Pounds</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/8” Coarse Aggregate (SSD)</td>
<td>1735</td>
</tr>
<tr>
<td>Fine Aggregate (SSD)</td>
<td>1400</td>
</tr>
<tr>
<td>Water</td>
<td>283</td>
</tr>
<tr>
<td>Cement, Type I</td>
<td>658</td>
</tr>
<tr>
<td>Modifier A</td>
<td>32.9</td>
</tr>
<tr>
<td>“Styrene Butadiene Rubber” (5%)</td>
<td></td>
</tr>
</tbody>
</table>

W/C=0.43

#### A.1.3.3. Adding 10 % solids of polymer

<table>
<thead>
<tr>
<th>Computed Batch Weight in Pounds</th>
<th>weight per one cubic foot in Pounds</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/8” Coarse Aggregate (SSD)</td>
<td>1735</td>
</tr>
<tr>
<td>Fine Aggregate (SSD)</td>
<td>1400</td>
</tr>
</tbody>
</table>
Water  250  9.26 (4.2 kg)
Cement, Type I  658  24.4 (11.065kg)
Modifier A  65.8  2.44 (1.1 kg)
“Styrene Butadiene Rubber” (5%)
Entrapped Air: 2%

W/C=0.38

A.2 Mix Adjustment

A.2.1. Addition of 1% steel fiber

\[
\frac{W + P}{C} = 0.48
\]

for super plasticizer WATER REDUCER the amount per manufacturer is:

from 10-25 oz/ 100 lb of cement

Use 16 oz=1 lb / 100 lb of cement,

P=1/100= 1%

Then;  \[ P = (1/100) \times 658 = 6.58 \text{ lb} \]

\[
\frac{W + 6.58}{658} = 0.48
\]

Then Water =309 lb

and W/C =  309/658= 0.47

For 30 oz per 100 lb cement superplasticizer

Then;  \[ P = ((30/16)/100) \times 658 = 12.34 \text{ lb} \]

\[
\frac{W + 12.34}{658} = 0.48
\]

Then Water =303.5 lb
and \( W/C = \frac{303.5}{658} = 0.46 \)

**A.2.2.** Addition of polymer by 2.5% of cement weight

then;

\[
\frac{W + P}{C} = 0.48
\]

\[
\frac{W + 0.025 \times 658}{658} = 0.48
\]

Then Water = 299.4 lb

\( w/c = 299.4/658 = 0.455 \)

**A.2.3.** Addition of polymer by 5% of cement weight

then;

\[
\frac{W + P}{C} = 0.48
\]

\[
\frac{W + 0.05 \times 658}{658} = 0.48
\]

Then Water = 283 lb

\( w/c = 283/658 = 0.43 \)

**A.2.4.** Addition of polymer by 10% of cement weight

then;

\[
\frac{W + P}{C} = 0.48
\]

\[
\frac{W + 0.1 \times 658}{658} = 0.48
\]

Then Water = 250 lb
Addition of polymer by 10% of cement weight and 30 oz per 100 pound of cement superplasticizer

then;

\[
\frac{W + P + SP}{C} = 0.48
\]

\[
W = 0.1 \times \frac{658}{658} + (\frac{30}{16}) \times \frac{658}{100} = 0.48
\]

Then Water = 237.7 lb

w/c = 237.7/658 = 0.36

A.3 New Mix Proportions for One Cubic Yard Batch of Concrete

A.3.1. Plain concrete

<table>
<thead>
<tr>
<th>Computed Batch Weight in Pounds</th>
<th>weight per one cubic foot in Pounds</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/8 Coarse Aggregate (SSD)</td>
<td>1326</td>
</tr>
<tr>
<td>Fine Aggregate (SSD)</td>
<td>1621</td>
</tr>
<tr>
<td>Water</td>
<td>282.1</td>
</tr>
<tr>
<td>Cement, Type 1 (9 BAGS)</td>
<td>846</td>
</tr>
<tr>
<td>Entrapped Air: 2%</td>
<td></td>
</tr>
</tbody>
</table>

W/C = 0.33

Cement: Sand: Gravel: by weight

1 : 1.91 : 1.57

Sand: gravel

0.55 : 0.45

The water used for wetting the coarse aggregate is 64.8 LB per cubic Yard
### A.3.2. Steel fiber reinforced concrete

<table>
<thead>
<tr>
<th>Computed Batch</th>
<th>weight per one cubic foot in Pounds</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Weight in Pounds</strong></td>
<td><strong>one cubic foot in Pounds</strong></td>
</tr>
<tr>
<td>3/8 Coarse Aggregate (SSD)</td>
<td>1326</td>
</tr>
<tr>
<td>Fine Aggregate (SSD)</td>
<td>1621</td>
</tr>
<tr>
<td>Water</td>
<td>266.24</td>
</tr>
<tr>
<td>Cement, Type I (9 BAGS)</td>
<td>846</td>
</tr>
<tr>
<td>Entrapped Air: 2%</td>
<td></td>
</tr>
<tr>
<td>Water reducer Superplasticizer (30 Oz/100 Lb cement)</td>
<td>15.86</td>
</tr>
<tr>
<td>1% Steel Fiber</td>
<td>132.3</td>
</tr>
</tbody>
</table>

W/C = 0.315

Cement: Sand: Gravel: by weight

2 : 1.91 : 1.57

Sand: gravel

0.55 : 0.45

### A.3.3. Steel fiber/polymer modified concrete

10% polymer

<table>
<thead>
<tr>
<th>Computed Batch</th>
<th>weight per one cubic foot in Pounds</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Weight in Pounds</strong></td>
<td><strong>one cubic foot in Pounds</strong></td>
</tr>
<tr>
<td>3/8” Coarse Aggregate (SSD)</td>
<td>1326</td>
</tr>
<tr>
<td>Fine Aggregate (SSD)</td>
<td>1621</td>
</tr>
<tr>
<td>Water</td>
<td>240</td>
</tr>
<tr>
<td>Cement, Type I</td>
<td>846</td>
</tr>
<tr>
<td>Modifier A</td>
<td>84.6</td>
</tr>
<tr>
<td>“Styrene Butadiene Rubber” (10%)</td>
<td></td>
</tr>
<tr>
<td>Entrapped Air: 2%</td>
<td></td>
</tr>
<tr>
<td>1% Steel Fiber</td>
<td>132.3</td>
</tr>
</tbody>
</table>

W/C = 0.284
APPENDIX B

EQUIPMENT USED IN THE EXPERIMENTAL PROCEDURE

B.1 Description of MTS Testing System

The MTS is an Axial-Torsional testing system (Figure B.1), which can be programmed using the Teststar software which works on an OS/2 operating system. The test procedures could be created using a displacement control or a force control in the axial mode and an angle control or torque control respectively. The MTS is equipped with two sets of gripping mechanisms, a wedge grip assembly and a collet grip assembly. The wedge grip assembly is used for axial mode applications only and has a capacity of 22,000 lbs in tension or compression. The collet grip assembly can be used in the axial or torsional applications and has a capacity of 55,000 lbs in axial mode and 20,000 in.lbs in the torsional mode respectively, the grips are selected based on the application. The MTS is built with a load cell and a displacement cell which are capable of reading the load and displacement to a high degree of accuracy. The Teststar software has a feature which could read up to 16 channels of input. In addition to the load and displacement readings, the MTS is connected to a signal conditioner which could handle eight channels of strain input at any given time (Figure B.5).
B.2 Description of Soiltest Digital Compression Tester

SOILTEST Digital Concrete Compression Tester CT-6000 series (Figure B.2) offer the most advanced concept in machine design, with pre-stressed steel frame construction and microprocessor technology, assuring high accuracy and dependability in test results. Model CT-6200 has an 8 inch diameter ram and a load capacity of 450,000 lbs/2000 KN.; model; CT-6500 has a 10 CT-6500 has a 10 inch diameter ram and a load capacity of 675,000 lbs/3000 KN. The maximum piston travel is 2.5 inches (63.5 mm). The frame stiffness is enhanced by tension tie bars that are pre-stresses beyond the maximum load of the machine; the tie bars are enclosed in rectangular steel tubes. The machines are supplied with a fully electronic digital control panel; an optional print-out system for automatic printed data output is available. The digital control panel provides total display of pre-test data input selections, test loads and stress values (resolution: 27-30 lbs (0.133 KN) in CT-6200 models and 50 lbs (0.222 KN.) in CT-6500 models). Pre-test data parameters are entered into the system memory through use of function keys and a data entry key board. A pace rate deviation indicator indicates the actual loading rate and allows the operator to correct for any deviation from the pre-selected rate during the test. Data acquisition input keys can be used to instruct the system to store, erase, print, or transfer data to a printer or computer.

The testers have been designed to meet ASTM C-39 and E-4 standards.

B.3 Description of Tinius Olsen Universal Testing Machine

Tinius Olsen Universal Testing Machine (TOUTM) has axial load capacity of 30,000
lbs. It is connected to output data logger (Olsen recorder) to printout charts of Load-Deflection Curve. The layout of the machine is shown in Figure B.3.

B.4 National Instrument Data Acquisition System

National instrument data acquisition (DAQ) system with 24 channels was used for strain gages readings. The DAQ chassis was SCXI-1001 includes six SCXI-1121 at which each module contains 4 channels numbered from 0 to 3. The DAQ is connected to a computer through DAQCard-16XE-50. The strain readings were collected by software LABVIEW 5.1. Figure B.4 shows the DAQ system connected to the computer.
Figure B.1 MTS Axial – Torsional testing machine
Figure B.2 SOILTEST Digital Concrete Compression Tester  CT-6000 series

Figure B.3 Tinus Olsen Universal Testing Machine

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.
Figure B.4 National Instrument Data Acquisition System connected with Computer

Figure B.5. Multi-Channel signal conditioner/amplifier 2100 system
APPENDIX C

STRAIN GAGE INSTALLATION

C.1 Surface Preparation

The purpose of surface preparation is to develop a chemically clean surface having a roughness appropriate to the gage installation requirements, a surface alkalinity corresponding to a pH of about 7, and a visible gage layout lines for locating and orienting the strain gage. The procedure outlined as per Instruction Bulletin B-129 was observed while preparing the surface for the installation of these strain gages.

C.1.1 Solvent degreasing

The first step in the installation of strain gages is degreasing. Degreasing was performed to remove oils, greases, organic contaminants, and soluble chemical residues. The first operation was always degreasing to avoid having the subsequent abrading operations drive surface contaminants into the clean surface material. The CSM-1A Degreaser with an one way aerosol sprayer was used to degrease the strain gage area because dissolved contaminants can not be carried back into the parent solvent. An area covering 4 to 6 inches on all sides of the intended gage location had to be degreased. Degreasing was always done in one direction only. This procedure must be repeated until the sponge used for wiping is clean and a new sponge was used each time.
C.1.2 Dry abrasion

Surface abrasion was done to remove any loosely bonded adherents and to develop a surface texture suitable for bonding. Abrading was started with coarse 60 grit silicon-carbide paper. Finish abrading was done with silicon-carbide paper of 120 grit. The abrasion was performed so as to form a 45° crosshatched surface in the gage area.

C.1.3 Wet abrasion

Abrading was done with M-Prep Conditioner A while keeping the surface wet. Conditioner A is a mildly acidic solution which accelerates the cleaning process.

C.1.4 Gage location layout lines

The normal method was followed for locating and orienting a strain gage on the test surface by first marking the surface with a pair of crossed reference lines at the point where the strain measurement is to be made. The lines were made perpendicular to one another, with one line oriented in the direction of strain measurement. The gage was then installed such that the triangular index marks defining the longitudinal and transverse axes of the grid were aligned with the reference lines on the test surface. The layout lines were made with a tool which burnishes, rather than scribes the surface. A scribed line may raise a burr or create a stress concentration. Such lines may be also proving to be detrimental to strain gage performance. Usually a ball-point pen was used to burnish layout lines on steel rebars.

C.1.5 Surface conditioning

After the layout was marked, Conditioner A was applied repeatedly, and the surface scrubbed with cotton-tipped application until a clean tip is no longer disclosed by the scrubbing. During this process, the surface was kept constantly wet with
Conditioner A until the cleaning was completed. When clean, the surface was dried by wiping through the cleaned area with a single stroke from one direction. Utmost care was taken to begin the stroke inside the cleaned area to avoid dragging contaminants in from the boundary of the area. Then, with a fresh sponge, a single slow stroke was made in the opposite direction. The sponge was not wiped back and forth and the Conditioner was not allowed to dry on the surface.

C.1.6 Neutralizing

The final step in surface preparation was to bring the surface condition back to an optimum alkalinity of 7.0 to 7.5 pH, which is suitable for all Micro-Measurements strain gage adhesive systems. This was done by applying M-Prep Neutralizer 5 liberally to the cleaned surface, and scrubbing the surface with a clean cotton-tipped applicator. The cleaned surface was kept completely wet with Neutralizer 5 throughout this operation. After neutralization, the surface was dried by wiping through the cleaned area with a single slow stroke of a clean gauze sponge. With a fresh sponge, a single stroke was made in the opposite direction, beginning with the cleaned area to avoid re-contamination from the un-cleaned boundary.

C.2 Gage Installation Procedure

The strain gages used in the experiments were CEA-06-240LZ-120, EA-06-500BH-120, and N2A-06-750DT-120 purchased from Measurements Group. These gages are capable reading up to 5%. The AE-10 adhesive was selected to install gages on concrete. After preparing the surface as explained in the previous section, the gages were installed as per Measurement Group Instruction Bulletin B-137-16.
C.2.1 Strain gage and solder terminal positioning

The gage was removed from the acetate envelop by grasping the edge of the gage backing with tweezers, and placing bonding side down on a chemically clean glass plate. The solder terminal was placed on the plate adjacent to the edge. A space of approximately 1/16 inch was left between the gage and the terminal. One end of cellophane tape was tacked on the glass plate behind the gage and terminal, and wiped forward onto the terminal and the gage. Carefully, the tape was lifted at a shallow angle.

C.2.2 Strain gage alignment

The gage/tape assembly was placed on the specimen, so that the triangular alignment marks on the gage were over the layout lines previously burnished on the specimen. Holding the tape at a shallow angle, the assembly was wiped onto the specimen surface. If the assembly was not properly aligned, the tape was lifted and realigned again.

C.2.3 Strain gage bonding

The tape lifted at a shallow angle until the gage and terminal are free of specimen surface. The loose end of the tape was tucked under, so that the gage lie flat with the bonding side exposed.

C.2.4 Adhesive Preparation

C.2.4.1 Preparing AE-10 adhesive

Each kit of AE-10 adhesive contains materials for mixing six batches of adhesive. One of the calibrated droppers filled with Curing Agent 10 exactly to the number 10 and the contents were dispensed into the center of the jar of Resin AE. The bottle of curing agent was immediately capped to avoid moisture absorption. The contents were mixed thoroughly for 5 minutes, using one of the plastic stirring rods. The pot life or working
time after mixing is 15 to 20 minutes. The dropper was discarded after use. AE-10 cures at 70°F in 6 hours, attaining a 6% elongation capability and essentially creep-free performance. To obtain 10% elongation capability, the curing time is extended to 24 to 48 hours at 75°F.

C.2.4.2 Strain gage curing

A rubber pad 3/32 inch thick was placed over the installed gage. This allows the clamping force to be exerted evenly over the gage. A spring clamped was used to apply a pressure of 5 to 20 psi. Care was taken to make sure that the clamping pressure is equal over the entire gage. Unequal clamping pressure may result in an irregular glue line. In some cases where a clamp was not feasible, a sand bag was place over the strain gage area that was calculated to provide approximately the same pressure. The gage was then cured following the recommendations to attain the desired strain measurements. After curing, the tape was pulled back directly over itself, peeling it slowly and steadily off the surface. Once the tape was removed, the gage was covered with another tape exposing only the solder tabs on the gage. The solder tabs were then coated with solder flux and the lead wires for the strain gages were soldered.
VITA

Graduate College
University of Nevada, Las Vegas

Ashraf Ibrahim Ahmed

Home Address:
6570 west Flamingo road #233
Las Vegas, Nevada 89103

Degrees:
Bachelor of Science, Civil Engineering, 1986
Cairo University, Egypt

Master of Science, Civil Engineering, 1995
New Mexico State University, Las Cruces, NM

Special Honors and Awards:
First Place award in AIAA Region VI Conference, CALPOLY, Pomona, CA, 1998.

Publications:


Polytechnic Institute (CALPOLY), Pomona, CA (1998). (First Place Winner).


Ph.D. Dissertation Title: Material Characterization of Steel Fiber/Polymer Concrete and Its Application to Thin And I-Beams

Dissertation Examination Committee:

Chairperson, Dr. Samaan G. Ladkany, Ph.D., P.E.
Committee Member, Dr. James A. Cardie, Ph.D., P.E.
Committee Member, Dr. Gerald R. Frederick, Ph.D., P.E.
Committee Member, Dr. Moses Karakouzian, Ph.D., P.E.
Graduate Faculty Representative, Dr. Brendan J. O'Toole, Ph.D.
and Dr. William G. Culbreth, Ph.D., P.E.