PM-1 3½"x4" PHOTOGRAPHIC MICROCOPY TARGET
NBS 1010a ANSI/ISO #2 EQUIVALENT

PRECISIONSM RESOLUTIONSM TARGETS

A New Concept in Achieving Ductility in FRP Reinforced Concrete Members: An Analytical and Experimental Study

By
Mir Mazher Ali, B.Eng.

A thesis submitted to
the Faculty of Graduate Studies and Research
in partial fulfillment of the requirements
for the degree of

Master of Engineering

© Copyright Reserved

This Master's of Engineering in Civil Engineering is a joint program with the University of
Ottawa administered by the Ottawa-Carleton Institute for Civil Engineering.

**Department of Civil and Environmental Engineering
Carleton University
Ottawa, Canada
1995**

National Library
of Canada

Acquisitions and
Bibliographic Services Branch

395 Wellington Street
Ottawa, Ontario
K1A 0N4

Bibliothèque nationale
du Canada

Direction des acquisitions et
des services bibliographiques

395, rue Wellington
Ottawa (Ontario)
K1A 0N4

Your file Votre référence

Our file Notre référence

The author has granted an irrevocable non-exclusive licence allowing the National Library of Canada to reproduce, loan, distribute or sell copies of his/her thesis by any means and in any form or format, making this thesis available to interested persons.

L'auteur a accordé une licence irrévocable et non exclusive permettant à la Bibliothèque nationale du Canada de reproduire, prêter, distribuer ou vendre des copies de sa thèse de quelque manière et sous quelque forme que ce soit pour mettre des exemplaires de cette thèse à la disposition des personnes intéressées.

The author retains ownership of the copyright in his/her thesis. Neither the thesis nor substantial extracts from it may be printed or otherwise reproduced without his/her permission.

L'auteur conserve la propriété du droit d'auteur qui protège sa thèse. Ni la thèse ni des extraits substantiels de celle-ci ne doivent être imprimés ou autrement reproduits sans son autorisation.

ISBN 0-612-13834-8

Canada

Name MIR MAZHER ALI

Dissertation Abstracts International is arranged by broad, general subject categories. Please select the one subject which most nearly describes the content of your dissertation. Enter the corresponding four-digit code in the spaces provided.

CIVIL

SUBJECT TERM

0543

SUBJECT CODE

U·M·I

Subject Categories

THE HUMANITIES AND SOCIAL SCIENCES

COMMUNICATIONS AND THE ARTS

Architecture 0729
Art History 0377
Cinema 0900
Dance 0378
Fine Arts 0357
Information Science 0723
Journalism 0391
Library Science 0399
Mass Communications 0708
Music 0413
Speech Communication 0459
Theater 0465

EDUCATION

General 0515
Administration 0514
Adult and Continuing 0516
Agricultural 0517
Art 0273
Bilingual and Multicultural 0282
Business 0688
Community College 0275
Curriculum and Instruction 0727
Early Childhood 0518
Elementary 0524
Finance 0277
Guidance and Counseling 0519
Health 0680
Higher 0745
History of 0520
Home Economics 0278
Industrial 0521
Language and Literature 0279
Mathematics 0280
Music 0522
Philosophy of 0998
Physical 0523

Psychology 0525
Reading 0535
Religious 0527
Sciences 0714
Secondary 0533
Social Sciences 0534
Sociology of 0340
Special 0529
Teacher Training 0530
Technology 0710
Tests and Measurements 0288
Vocational 0747

LANGUAGE, LITERATURE AND LINGUISTICS

Language
 General 0679
 Ancient 0289
 Linguistics 0290
 Modern 0291
Literature
 General 0401
 Classical 0294
 Comparative 0295
 Medieval 0297
 Modern 0298
 African 0316
 American 0591
 Asian 0305
 Canadian (English) 0352
 Canadian (French) 0355
 English 0593
 Germanic 0311
 Latin American 0312
 Middle Eastern 0315
 Romance 0313
 Slavic and East European 0314

PHILOSOPHY, RELIGION AND THEOLOGY

Philosophy 0422
Religion
 General 0318
 Biblical Studies 0321
 Clergy 0319
 History of 0320
 Philosophy of 0322
Theology 0469

SOCIAL SCIENCES

American Studies 0323
Anthropology
 Archaeology 0324
 Cultural 0326
 Physical 0327
Business Administration
 General 0310
 Accounting 0272
 Banking 0770
 Management 0454
 Marketing 0338
Canadian Studies 0385
Economics
 General 0501
 Agricultural 0503
 Commerce-Business 0505
 Finance 0508
 History 0509
 Labor 0510
 Theory 0511
Folklore 0358
Geography 0366
Gerontology 0351
History
 General 0578

Ancient 0579
Medieval 0581
Modern 0582
Black 0328
African 0331
Asia, Australia and Oceania 0332
Canadian 0334
European 0335
Latin American 0336
Middle Eastern 0333
United States 0337
History of Science 0585
Law 0398
Political Science
 General 0615
 International Law and Relations 0616
 Public Administration 0617
Recreation 0814
Social Work 0452
Sociology
 General 0626
 Criminology and Penology 0627
 Demography 0938
 Ethnic and Racial Studies 0631
 Individual and Family Studies 0628
 Industrial and Labor Relations 0629
 Public and Social Welfare 0630
 Social Structure and Development 0700
 Theory and Methods 0344
Transportation 0709
Urban and Regional Planning 0999
Women's Studies 0453

THE SCIENCES AND ENGINEERING

BIOLOGICAL SCIENCES

Agriculture
 General 0473
 Agronomy 0285
 Animal Culture and Nutrition 0475
 Animal Pathology 0476
 Food Science and Technology 0359
 Forestry and Wildlife 0478
 Plant Culture 0479
 Plant Pathology 0480
 Plant Physiology 0817
 Range Management 0777
 Wood Technology 0746

Biology
 General 0306
 Anatomy 0287
 Biostatistics 0308
 Botany 0309
 Cell 0379
 Ecology 0329
 Entomology 0353
 Genetics 0369
 Limnology 0793
 Microbiology 0410
 Molecular 0307
 Neuroscience 0317
 Oceanography 0416
 Physiology 0433
 Radiation 0821
 Veterinary Science 0778
 Zoology 0472

Biophysics
 General 0786
 Medical 0760

EARTH SCIENCES

Biogeochemistry 0425
Geochemistry 0996

Geodesy 0370
Geology 0372
Geophysics 0373
Hydrology 0388
Mineralogy 0411
Paleobotany 0345
Paleocology 0426
Paleontology 0418
Paleozoology 0985
Palynology 0427
Physical Geography 0368
Physical Oceanography 0415

HEALTH AND ENVIRONMENTAL SCIENCES

Environmental Sciences 0768
Health Sciences
 General 0566
 Audiology 0300
 Chemotherapy 0992
 Dentistry 0567
 Education 0350
 Hospital Management 0769
 Human Development 0758
 Immunology 0982
 Medicine and Surgery 0564
 Mental Health 0347
 Nursing 0569
 Nutrition 0570
 Obstetrics and Gynecology 0380
 Occupational Health and Therapy 0354
 Ophthalmology 0381
 Pathology 0571
 Pharmacology 0419
 Pharmacy 0572
 Physical Therapy 0382
 Public Health 0573
 Radiology 0574
 Recreation 0575

Speech Pathology 0460
Toxicology 0383
Home Economics 0386

PHYSICAL SCIENCES

Pure Sciences
 Chemistry
 General 0485
 Agricultural 0749
 Analytical 0486
 Biochemistry 0487
 Inorganic 0488
 Nuclear 0738
 Organic 0490
 Pharmaceutical 0491
 Physical 0494
 Polymer 0495
 Radiation 0754
 Mathematics 0405
 Physics
 General 0605
 Acoustics 0986
 Astronomy and Astrophysics 0606
 Atmospheric Science 0608
 Atomic 0748
 Electronics and Electricity 0607
 Elementary Particles and High Energy 0798
 Fluid and Plasma 0759
 Molecular 0609
 Nuclear 0610
 Optics 0752
 Radiation 0756
 Solid State 0611
 Statistics 0463
Applied Sciences
 Applied Mechanics 0346
 Computer Science 0984

Engineering
 General 0537
 Aerospace 0538
 Agricultural 0539
 Automotive 0540
 Biomedical 0541
 Chemical 0542
 Civil 0543
 Electronics and Electrical 0544
 Heat and Thermodynamics 0348
 Hydraulic 0545
 Industrial 0546
 Marine 0547
 Materials Science 0794
 Mechanical 0548
 Metallurgy 0743
 Mining 0551
 Nuclear 0552
 Packaging 0549
 Petroleum 0765
 Sanitary and Municipal 0554
 System Science 0790
Geotechnology 0428
Operations Research 0796
Plastics Technology 0795
Textile Technology 0994

PSYCHOLOGY

General 0621
Behavioral 0384
Clinical 0422
Developmental 0620
Experimental 0623
Industrial 0624
Personality 0625
Physiological 0989
Psychobiology 0349
Psychometrics 0632
Social 0451

The undersigned recommend to the Faculty of Graduate Studies and Research acceptance of the thesis entitled

**"A New Concept in Achieving Ductility
in FRP Reinforced Concrete Members:
An Analytical and Experimental Study"**

Submitted by
Mir Mazher Ali, B.Eng.

in partial fulfillment of the requirements for the Degree of Master of Engineering.

(Dr. A.G. Razaqpur)
Thesis Supervisor

(Dr. J.L. Humar)
Professor and Chairman
Department of Civil and
Environmental Engineering

**Carleton University
Ottawa, Ontario
1995**

ABSTRACT

Due the high strength, low elastic modulus and linear elastic behavior of FRP reinforcement, concrete structures reinforced with FRP generally undergo relatively large deflection under service loads and they exhibit a brittle failure mode. To investigate these problems in FRP reinforced concrete structures a parametric study was conducted. A hybrid reinforcement system is proposed which consists of a low modulus polymeric reinforcement combined with CFRP and which is believed to overcome the lack of ductility and stiffness in FRP reinforced concrete members. In this study two types of FRP reinforcements were utilized to study the overall flexural response and more specifically the ductility of FRP reinforced concrete beams.

The initial step in this project was to develop mathematical models to determine the feasibility of application of FRP as internal or external reinforcement to the structures using the current analytical methods. A comprehensive analytical investigation was carried out using refined methods of non-linear analysis. A computer program "Non-Linear Analysis & Design [NLAD] " consisted of over six thousand lines of code was developed to analyze and design symmetrical composite/concrete member. To understand the behavior of different FRP's, analysis was carried out using AFRP, GFRP and CFRP, but the experimental work was limited to CFRP.

In order to validate the results obtained from the analytical models and to provide the full response of the structural behavior of concrete beams reinforced externally or internally with FRP material, an extensive experimental program was carried out. The experimental program consisted of two phases.

The first phase dealt with the application of CFRP sheets for strengthening and/or retrofit of existing structures to increase and/or restore their strength and stiffness. It included testing of four 4.2 meter long simply supported beams of 200 mm x 400 mm in cross-section under monotonic loading. To simulate the effect of strengthening, two beams were not reinforced with any kind of internal steel reinforcement in flexure nor shear. For the beams externally reinforced with FRP sheets, the internal reinforcement provided will form the second component of the proposed hybrid reinforcement system. Although part of the study deals with strengthening of beams for shear resistance, the main thrust of the work was the study of the flexural behavior, with particular emphasis on ductility and

deformability. The results clearly showed that external reinforcement in the form of epoxy bonded sheets have a distinctive effect on the stiffening the concrete members. A definite trend was observed in increase in strength and stiffness of the beam with respect to number of layers of CFRP sheets. The use of existing methods for flexural analysis and computation of deflections are compatible for CFRP externally reinforced beams. It is evident from the experimental observations CFRP sheets can also be used for shear strength upgrade.

In the second phase three 4.2 m long and 400 mm deep high strength concrete inverted T-beams were designed using this new concept and were tested under four point bending. Two of the beams contained hybrid reinforcement while the third contained only CFRP reinforcement (control beam). The control beam failed suddenly, following the rupture of the CFRP reinforcement. The beams with hybrid reinforcement failed in a ductile manner at 63% higher load and they had at least three times more deflection at failure than the control beam. In the later beams, deflections substantially increased following the rupture of CFRP, but the moment of resistance of the beams decreased marginally.

Upon release of the load, all the hybrid reinforced beams rebounded and had very little deflection under self-weight.

ACKNOWLEDGMENTS

I owe great thanks to Dr. A.G. Razaqpur, my thesis supervisor, for his constant guidance, never faltering patience, and encouragement during every phase of this research.

I would like to thank Mr. Ken McMartin, the Manager of the Civil Engineering Laboratories and the structural laboratory staff for their help and enthusiasm during the preparation and testing of the test specimens and the ancillary testing of engineering materials.

This research was supported by a grant from the Natural Sciences and Engineering Research Council of Canada. The CFRP was donated by Mitsubishi Chemicals Company, Canada, the Forca Tow Sheet was provided by Autocon Composites of Toronto Canada, while the polypropylene grid was supplied by the Tensar Corporation of USA. I gratefully acknowledge the support provided by these organizations.

I greatly appreciate the enthusiasm and moral support provided by Mr. John Eccles, Mr. Bruce Mason, Mr. Brian Wadden, Project Managers and Mr. Robin Campbell of the Regional Municipality of Ottawa-Carleton, Mr. August Wilkins, Vice-President of Wyllie and Ufnal Consulting Engineers, Wojciech Remisz, President of Remisz and Associates Ltd., Mr. Gordon Mcrobie, Manager of Mcneely Engineering Consultants, Dr. Elvi of the University of Alberta, Dr. Bahadir Eke, Director, Transportation Division and Mr. Leslie Harebottle, Structures Branch Manager, of Morrison Hershfield Ltd.. To the numerous other friends and colleagues who have not been mentioned and who helped and extended moral support during each phase of this project, I express my appreciation.

Special thanks to Mr. and Mrs. Syed Imtiaz Ali of Edmonton for their encouragement.

Of course, sincerest thanks to my wife Majida for her unconditional love and patience.

*To my parents,
Mir Azher Ali and Dr. Zakia Sultana,
for their continued support and prayers
as I pursued yet another degree*

Table of Contents

I	Chapter One	Introduction	1
	1.1	General	1
	1.2	Objectives	5
II	Chapter Two	Previous Studies & Case Histories	6
	2.1	External Reinforcement	6
	2.1.1	General	6
	2.1.2	Previous Research on Strengthening Using Steel Plates	7
	2.1.3	Field Applications of Steel Plates	8
	2.1.4	Previous Research on FRP Laminates/Sheets	10
	2.1.5	Field Application of FRP Laminates/Sheets	22
	2.2	Internal Reinforcement	23
	2.3	Ductility & Deformability in FRP Reinforced Concrete Structures	25
III	Chapter Three	A New Concept for Achieving Ductility in FRP Reinforced Concrete Members	
	3.1	General	29
	3.2	Ductility Versus Deformability	30
	3.3	Basic Concept	30

3.4	A Conceptual FRP Hybrid Bar	31
3.5	Behavior of Beams Reinforced with FRP-HDPP Hybrid	34
	Figures	37
IV	Chapter Four	Experimental Program
4.1	General	44
4.2	Test Beams	45
4.2.1	Rectangular Beams (Phase - I)	45
4.2.2	Inverted T-Beams (Phase - 2)	48
4.3	Construction of Beams	49
4.4	Loading Apparatus	51
4.5	Experimental Setup and Test Procedure	51
4.6	Instrumentation & Data Acquisition	51
	Figures & Plates	53
V	Chapter Five	Experimental Results & Discussion
5.1	General	69
5.2	SRC Beams (B1 & B2)	70
5.2.1	General	70
5.2.2	Observed Behavior of SRC Beams	70
5.2.3	Discussion of SRC Beams	71
5.3	CLTRC Beams (B3 & B4)	
5.3.1	General	72
5.3.2	Observed Behavior of CLTRC Beam B3	73
5.3.3	Discussion of CLTRC Beam B3	74

5.3.4	Observed Behavior of CLTRC Beam B4	76
5.3.5	Discussion of CLTRC Beam B4	78
5.4	Comparison of SRC and CLTRC beams	80
5.5	CRTRC Beams (B5 & B6)	84
5.5.1	General	84
5.5.2	Observed Behavior of CRTRC Beams	84
5.5.3	Discussion of CLTRC Beams	86
5.6	CRRRC Beam (B7)	88
5.6.1	General	88
5.6.2	Observed Behavior and Discussion of Test Results of CRRRC Beam (B7)	89
5.7	Comparison of CRTRC and CRRRC Beams	90
	Figures	92
	Plates	128

VI Chapter Six Theoretical Correlation with Experimental Results

6.1	General	142
6.2	Ultimate Moment Capacity	142
6.3	Balanced Reinforcement Ratio	144
6.4	Moment-Curvature Relations	148
6.4.1	Moment-Curvature Relations Determination	148
6.4.2	Variation of Neutral Axis Depth	153
6.4.3	Variation of Curvature Along Beam Span	154
6.5	Deflections	154

6.5.1	Code Approach	154
6.5.1.1	Pre-Cracking	155
6.5.1.2	Post-Cracking	156
6.5.2	Modified Moment of inertia Method For FRP Reinforced Beams	158
6.5.3	Computation of Deflections From Curvatures	160
6.6	Tension Stiffening	164
6.6.1	Factors Effecting Tension Stiffening	164
6.6.2	Tension Stiffening Model - I	167
6.6.3	Tension Stiffening Model - II	168
6.6.4	Comparison of Calculated and Measured Deflections Using Three Methods	168
	Figures	170
VII	Chapter Seven Summary, Conclusions and Recommendations	
7.1	Summary	185
7.2	Conclusions	186
7.3	Recommendations	189

References

I to XIII

Appendix - A	Design Briefs	A-i to A-v
Appendix - B	Material Properties	B-i

B.1	Concrete	B-i
B.2	Reinforcing Steel	B-iii
B.3	Tensar Grids	B-iii
	B.3.1 Overview	B-iii
	B.3.2 Test Specimen	B-iv
	B.3.3 Test Equipment	B-iv
	B.3.4 Test Method	B-v
	B.3.5 Test Results	B-vi
Figures		B-xii to B-xvi

**Appendix - C Computer Program “Nonlinear Analysis &
Design [NLAD]”**

LIST OF FIGURES

<u>FIGURE No.</u>	<u>DESCRIPTION</u>
3.1	Conceptual Hybrid Reinforcing Bar
3.2	Idealized Stress-Strain Curves for HDPP, GFRP and CFRP
3.3	Stress-Strain Curves For CFRP Hybrid Reinforcing Bar
3.4	Stress-Strain Curve For GFRP Hybrid Reinforcing Bar
3.5	Conceptual Beam Details
3.6	Moment-Curvature Relationship For Beams Designed With Three Different Reinforcements and Similar Moment Capacity
3.7	Load-Deflection Curves For Three Beams With Different Reinforcements and Similar Load Capacity
<hr/>	
4.1	Reinforcement Details For SRC Beams (B1 & B2)
4.2	Reinforcement Details For CLTRC Beam (B3)
4.3	Stress-Strain Curve For Tensar UX-1700
4.4	Stress-Strain Curve For CFRP Sheet
4.5	Reinforcement Details For CLTRC Beam (B4)
4.6	Reinforcement Details For CRTRC Beams (B5 & B6)
4.7	Stress-Strain Curve For CFRP Rebar
4.8	Stress-Strain Curve For Tensar UX-1700HT
4.9	Reinforcement Details For CRRC Beam (B7)
4.10	Bonding Sequence of External CFRP Reinforcement To CLTRC Beams
4.11	Instrumentation Details of SRC Beams (B1 & B2)
4.12	Instrumentation Details of CLTRC Beam (B3)
4.13	Instrumentation Details of CLTRC Beam (B4)
4.14	Instrumentation Details of CRTRC Beams (B5 & b6)
4.15	Instrumentation Details of CRRC Beam (B7)
<hr/>	
5.1	Load-Deflection Curves For SRC Beam (B1)
5.2	Load-Deflection Curves For SRC Beam (B2)
5.3	Load v/s Mid-Span Steel Strain Curves For SRC Beam (B1)
5.4	Load v/s Mid-Span Steel Strain Curves For SRC Beam (B2)

- 5.5** Load v/s Mid-Span Concrete Strain at Top Curves For SRC Beam (B1)
- 5.6** Load v/s Mid-Span at Top Concrete Strain Curves For SRC Beam (B2)
- 5.7** Load v/s Steel Stirrup Strain For SRC Beam (B1)
- 5.8** Load v/s Steel Stirrup Strain For SRC Beam (B2)
- 5.9** Load-Deflection Curves For CLTRC Beam (B3)
- 5.10** Load v/s CFRP Laminate Strain at Mid-Span For CLTRC Beam (B3)
- 5.11** Load v/s Strain in CFRP Stirrup For CLTRC Beam (B3)
- 5.12** Load v/s Mid-Span Concrete Strain at Top For CLTRC Beam (B3)
- 5.13** Load-Deflection Curve For Repeated Load For CLTRC Beam (B3)
- 5.14** Load-Deflection Curves For CLTRC Beam (B4)
- 5.15** Load v/s CFRP Laminate Strain at Mid-Span For CLTRC Beam (B4)
- 5.16** Load v/s Mid-Span Concrete Strain at Top For CLTRC Beam (B4)
- 5.16(A)** Load v/s Mid-Span Concrete Strain at Top For CLTRC Beam (B4)
- 5.17** Load v/s Strain in CFRP Stirrup For CLTRC Beam (B4)
- 5.18** Comparison of Load-Deflection Curves For SRC Beams and CLTRC Beam (B3)
- 5.19** Comparison of Load-Deflection Curves For SRC Beams and CLTRC Beam (B4)
- 5.20** Comparison of Load-Deflection Curves For CLTRC Beams (B3 & B4)
- 5.21** Load-Deflection Curves For SRC and CLTRC Beams
- 5.22** Load-Deflection Curve For CRTRC Beam (B5)
- 5.23** Load-Deflection Curve For CRTRC Beam (B6)
- 5.24** Load v/s Mid-Span Concrete Strain at Top For CRTRC Beam (B5)
- 5.25** Load v/s Mid-Span Concrete Strain at Top For CRTRC Beam (B6)
- 5.26** Load v/s Strain In CFRP Rebar at Mid-Span For CRTRC Beam (B6)
- 5.27** Deformed Shape of CRTRC Beam (B5) at Various Incremental Loads
- 5.28** Deformed Shape of CRTRC Beam (B5) at Various Incremental Loads

5.29	Load-Deflection Curves For CRRC Beam B(7)
5.30	Load v/s Mid-Span CFRP Strain
5.31	Load v/s Concrete Strain CRRC Beam B7
5.32	Deformed Shape of CRRC Beam (B7) at Various Incremental Loads
5.33	Load-Deflection Curves for CRTRC Beams B5 & B6
5.34	Load-Concrete Strain for Beams B5 and B7
5.35	Comparison of Load-Deflection Curves of Hybrid Beams B5 & B6 with CFRP Reinforced Beam

6.1	Balanced Strain Condition
6.2	Variation of Reinforcement Ratio with Concrete Strength
6.2(A)	Behavior of Singly Reinforced Beam with Various Reinforcement Ratios
6.3	Deformed Element of A Flexural Member
6.4	Stress-Strain Relationship of A section
6.5	Assumed Concrete Stress-Strain Curve
6.6	Moment-Curvature Relationship For SRC Beams
6.7	Moment-Curvature Relationship For CLTRC Beam B3
6.8	Moment-Curvature Relationship For CLTRC Beam B4
6.9	Moment-Curvature Relationship For CRTRC Beams
6.10	Moment-Curvature Relationship For CRRC Beam
6.11	Variation of Neutral Axis v/s Load For SRC Beams
6.12	Variation of Neutral Axis v/s Load For CLTRC Beam B3
6.13	Variation of Neutral Axis v/s Concrete Strain For CRTRC Beams B5& B6
6.14	Variation of Neutral Axis Depth v/s Concrete Strain
6.15	Variation of Curvature of SRC Beams
6.16	Variation of Curvature CLTRC Beam B4
6.17	Variation of Curvature For 30 and 60 MPa Concrete Beam
6.18	Cracked Section of A Loaded Beam
6.19	Typical Moment-Curvature Relationship
6.20	Calculation of Deflections From Curvatures
6.21	Average Stress v/s Strain Relationship
6.22	Tension Stiffening Model -1
6.23	Tension Stiffening Model - 2
6.24	Load-Deflection Curves of Beam B7 By Using Three Methods

6.25 Load-Deflection Curves of Beam B6 By Using Three Methods

- B.1 Assumed Reinforcing Steel Stress-Strain Curve**
- B.2 Uniaxial Tensile Test Setup for Tensar UX1700 and 1700HT**
- B.3 Load-Strain Curves for Uniaxial Tensile Test for UX-1700HT**
- B.4 Cyclic Behavior of Load-Strain Curves for Uniaxial Test of Tensar UX1700HT**
- B.5 Load-Strain Curve Uniaxial Tensile Test of Tensar UX1700HT**
- B.6 Comparison of Load-Strain Curves for Tensar UX1700 and UX1700HT**

LIST OF TABLES

<u>TABLE No.</u>	<u>DESCRIPTION</u>
5.1	Comparison of Deflections for SRC and CLTRC Beams
5.2	CFRP Sheet Strains at Ultimate
5.3	Comparison of Strength of One & Two Ply of CFRP Sheets
A.1	Theoretical Values for SRC Beams (B1 & B2) [NLAD]
A.2	Theoretical Values for CLTRC Beam (B3) [NLAD]
A.3	Theoretical Values for CLTRC Beam (B4) [NLAD]
A.4	Theoretical Values for CRTRC Beams (B5 & B6) [NLAD]
A.5	Theoretical Values for CRRC Beam (B7) [NLAD]
B.1	Concrete Compressive Strength After 28 Days
B.2	Concrete Compressive Strength During Curing Period of 90 Days
B.3	Tensar Test Specimen Summary
B.4	Tensar UX1700 Test Data Summary
B.5	Tensar UX1700HT Test Data Summary
B.6	Mean Values of Test Results

LIST OF PLATES

<u>PLATE No</u>	<u>DESCRIPTION</u>
4-1	Arrangement of Tensar Grids in CRTRC Beams (B5 & B6)
4-2	Construction of CLTRC Beam (B4)
5-1	SRC Beam (B1) After Failure
5-2	SRC Beam (B2) After Failure
5-3	CLTRC Beam (B3) - Stretching of CFRP Sheet at Sides
5-4	CLTRC Beam (B3) - at Load of 50 KN
5-5	CLTRC Beam (B3) - at Failure
5-6	CLTRC Beam (B3) - After Failure (Note: Only Local Damage in Constant Moment Region)
5-7	CLTRC Beam (B3) - Post-mortem of Beam After Failure
5-8	CLTRC Beam (B4) - (Note: Shear Crack Intercepted By CFRP Stirrup)
5-9	CLTRC Beam (B4) - (Note: CFRP Shear Stirrup Ruptured by Shear Force)
5-10	CLTRC Beam (B4) - Sudden Delamination of Anchorage
5-11	CLTRC Beam (B4) - After Failure
5-12	CLTRC Beam (B4) - After Failure
5-13	CLTRC Beam (B4) - After Failure
5-14	CLTRC Beam (B4) - Post-mortem of Beam After Failure
5-15	CRTRC Beam (B5) - At 90 KN
5-16	CRTRC Beam (B5) - Crack Distribution At 150 KN
5-17	CRTRC Beam (B5) - At 150 KN
5-18	CRTRC Beam (B6) - At 170 KN
5-19	CRTRC Beam (B5) - Failure of The Beam
5-20	CRTRC Beam (B5) - Failure of The Beam
5-21	CRTRC Beam (B5) - Failure of The Beam
5-22	CRTRC Beam (B5) - Crack Distribution
5-23	CRTRC Beam (B7) - Crack Distribution
5-24	CRTRC Beam (B5) - Beam Under Self Weight
5-25	CRTRC Beam (B6) - Beam Under Self Weight
5-26	CRTRC Beam (B7) - After Failure

Chapter One

INTRODUCTION

1.1 GENERAL

Reinforced concrete structures such as coastal and marine structures, strait crossings, underground tunnels, chemical plants, waste water treatment facilities and nuclear reactors which are exposed to highly corrosive environments have been experiencing noticeably reduced service life. Also, the extensive use of de-icing salts is reducing life expectancy of steel reinforced concrete bridge decks and multistory parking garages.

The corrosion of reinforcement is caused by the ingress of moisture, which contains chloride ions, through the hairline cracks (micro cracks) formed due to shrinkage, thermal variation or frost attack on structures. Several measures have been implemented in the design of concrete structures to prevent the corrosion of steel reinforcement, including waterproofing on bridge decks, waterproofing admixtures in concrete, impermeable membranes, epoxy-coated rebars and the strict crack control design procedures in the codes for reinforced concrete design. However, none of these methods has proven fully effective.

Corrosion is a severe problem for exposed concrete structures because it affects both the serviceability and strength of structures. Various solutions to the problem of corrosion have been implemented but none of them has been so far entirely satisfactory. For example, the epoxy-coated steel bars were initially believed to be immune to corrosion but they have been found to be experiencing pitting corrosion [Sagues, Powers and Zayed (1992)]. Therefore efforts have to be redirected towards the search for new materials which are not susceptible to corrosion but which also have high tensile strength, good bond strength and high modulus of elasticity.

Fiber reinforced plastics (FRP) reinforcement as a substitute for steel reinforcement has been suggested as an alternative solution to the several active and passive corrosion protection methods that have been considered and used to extend the life of concrete structures. These materials are characterized by their light weight, high strength per unit weight and relatively high modulus of elasticity. Among the FRP materials, carbon fiber reinforced plastics (CFRP) are superior in strength and stiffness in comparison to glass and aramid fiber reinforced plastics (GFRP and AFRP). The other extremely useful characteristics of CFRP are: good corrosion resistance to acidic as well as alkaline environment, good damping behavior, high fatigue resistance, very little relaxation under stress [Iyer and Khubchandani (1992)].

FRP are manufactured in the form of sheets, rebars, tendons and grids. For superior bond properties, FRP rebars are available as round bars, with indented or ribbed surface. The diameters of the bars range from 3 mm to 20 mm. Based on the available manufacturers data, the tensile strength of typical CFRP tendons and wires varies from 1600 MPa to well above 3300 MPa, and their elastic modulus ranges from 130 to 250 GPa. The ultimate strain lies between 1.0% to 2%.

There has also been an increasing demand for FRP structural components for use in special construction applications such as in highly corrosive environments, in non-magnetic and electrically non-conductive environments [Erki et al (1992)]. Another important area of application of FRP is for strengthening and rehabilitation of structures. About one-half of the approximately 600,000 highway bridges in the United States are in need of replacement or rehabilitation [Saadatmanesh and Ehasani (1990)]. Many of these bridges were originally designed for the smaller vehicles, lighter loads, and lower traffic volumes than they are today. Also, the strength of reinforced concrete beams and columns in certain buildings may be unsatisfactory for a variety of reasons. In some cases, it is possible that the existing reinforced concrete members may have been originally designed for some service load and are required to support extra loads due to the construction of additional floors or increase in live load pattern in the building, resulting from change of occupancy. In the above circumstances there are generally two solutions such as

demolishing and rebuilding the structure to meet the current requirements or upgrading and strengthening the existing structural elements with a suitable methods such as externally bonded FRP laminates or sheets. In most, cases the technique of strengthening is more attractive than replacement because of financial and time constraints.

A concrete beam which is subjected to excessive deflections due to loss in flexural rigidity can be successfully strengthened to meet the current code requirements for deflection control. Also additional FRP bars or laminates may be bonded externally to the concrete in areas of high shear stress. By this method, the shear strength of a concrete beam can also be enhanced. For instance, the rehabilitation of a bridge can be carried out with minimal interruption to the traffic and negligible overhead clearance loss.

The post-strengthening of existing reinforced concrete structures through the bonding of the steel plates has been used for several years since the emergence of high strength epoxies and other structural adhesives in the early 1960's. This system has been successful but it also has several disadvantages such as in the handling of the heavy steel plates at the installation site, the corrosion in the steel plates and the loss in aesthetics in buildings due to bonding of thick steel plates.

A noticeable amount of research is being currently carried out in the area of strengthening of concrete structures using various FRP material such as carbon, fiber glass, aramid, etc. [Mufti et al (1992), Saadatmanesh and Ehsani (1990 and 1991), Tonen Corp. (1992)]. While steel has the advantage of a long yield plateau and a high modulus, FRP has its own structural advantages. FRP materials are very light. The density of a typical CFRP laminate is one-fifth of steel and thus adds negligible weight to the structure. The thickness of CFRP, GFRP, AFRP ranges from 0.15 mm to 3.0 mm.

One of the most compelling arguments in favour of the use of advanced composite materials in bridges in Canada, and certain parts of USA, is that these materials are chemically inert to the effects of common de-icing salts and marine environments [Bakth, Gilkie, Jaeger (1991)]. But FRP, including CFRP, have their own shortcomings. FRP

reinforced concrete lacks ductility and deflection control at the serviceability limit state which may be a problem.

Reinforcing steel has the ability to undergo deformations without reduction in the flexural capacity of the steel reinforced concrete member. The ductility of a concrete member depends mainly on the shape of its moment-curvature relationship. The moment-curvature relationship depends solely on the material properties, (i.e. their stress-strain relationship, modulus of elasticity, ultimate strain and on the reinforcement ratio of the section). In conventional reinforced concrete structures, the ductility of a section is quantified in terms of curvature ductility factor Φ_u/Φ_y , where Φ_y is the curvature when the tension reinforcement first reaches the yield strength and Φ_u is the ultimate curvature normally defined as when the unconfined concrete strain reaches its specified limiting value. For reinforced concrete beams with unconfined concrete, the flexural strength and ductility depend mainly on the tension reinforcement ratio ρ and the compression reinforcement ratio, ρ' . The compressive strength of concrete, f'_c , has a smaller effect on ductility.

Achievement of ductility in FRP reinforced structures requires further investigations because the stress-strain behavior of FRP materials is linearly elastic until failure. This behaviour of FRP material is of major concern in structural application of these material because the failure of FRP reinforced concrete structures is abrupt and, they lack the plastic energy dissipation characteristics of ordinary reinforced concrete. High strength can be successfully achieved by using FRP, but there will be inadequate warning or deformations in the member prior to failure. If an excessive overload or severe earthquake occurs, the failure will be sudden, brittle, and catastrophic. Therefore, it is necessary to develop design procedures that would overcome these shortcomings of FRP reinforced concrete structures and achieve an adequate level of warning and possible redistribution of stresses in statically indeterminate systems.

1.2 OBJECTIVES AND SCOPE

In this study, two types of FRP reinforcement are used to study the overall flexural response and more specifically the ductility of FRP reinforced concrete beams. The primary objectives of this analytical and experimental study are to:

1. Determine the feasibility of strengthening concrete beams using externally bonded FRP sheets.
2. Investigate the magnitude of increase in strength and stiffness of the beams bonded with external FRP sheets.
3. Investigate the possibility of using FRP sheets for increasing shear strength.
4. Investigate the feasibility of a hybrid reinforcement concept for achieving ductility and/or deformability in CFRP reinforced structures.
5. Study and compare deflection of FRP reinforced beams with conventional steel reinforced beams.
6. Develop the theoretical moment-curvature relations for concrete beams reinforced with FRP rebars and compare the results with the test data.

The scope of this study is restricted to straight, simply supported prismatic non-prestressed concrete members. The FRP reinforcement used in the experimental portion of this investigation comprises carbon fibre reinforced plastic sheets and bars only. The study is also limited to short-term flexural responses under four point bending and monotonically increasing static loads. Although part of the study deals with strengthening of beams for shear resistance, the main thrust of the work is the study of the flexural behaviour, with particular emphasis on ductility and deformability.

Chapter Two

PREVIOUS STUDIES AND CASE HISTORIES

2.1 EXTERNAL REINFORCEMENT

2.1.1 GENERAL

The problem of the repair and strengthening of concrete structures is of growing concern. Many of the highway bridges in North America are subjected to widespread deterioration because of several factors. These include the advent of the retirement age of the structures, increase in traffic volume and increase in gross vehicle weight by as much as forty percent from the time when some of these bridges were built. Furthermore, the damage caused by salt attack leads to the corrosion of the steel reinforcement which in turn diminishes the margin of safety against flexural as well as shear failure. According to the National Bridge Inventory [Dunker et al (1987)] in the United States, over 67% of bridges need replacement. By considering the factors which have contributed to the structural inadequacy of the existing structures, including such factors as increase in traffic volume, percentage of heavy vehicles, environmental conditions and degree of damage, it may be necessary in some cases to strengthen and restore the original rigidity of the structure. The strengthening and rehabilitation of existing structures by FRP material may provide a part of the solution to this urgent need. As will be described later, currently steel plates are bonded to a concrete member to increase its capacity. Due to their light weight and high strength, FRP laminates/sheets might be a good substitute for the steel plates. Of course, the wrapping of columns and piers by FRP sheets to increase their strength and

ductility is becoming popular, but the latter is not within the scope of the present investigation.

2.1.2 PREVIOUS RESEARCH ON STRENGTHENING USING STEEL PLATES

In the 1960's the emergence of high strength and high performance adhesives gave rise to the bonding of steel plates to existing structures to increase their flexural capacity. The advantage of using epoxy in this application is the formation of uniform bond transfer plane, as opposed to the studs which provide a set of discrete tie down points with the concomitant stress concentration [Ritchie (1989)]. Research on the strengthening of concrete structures through bonding of steel plates has been conducted over the past two decades.

The research focused mainly on the strengthening of deteriorated reinforced concrete members using steel plates. It included the effect of such variables as plate stiffness, environmental effects, various types of epoxies and plate thickness, and several other important structural parameters. These investigations were conducted by McDonald (1978), Calder (1979), Raithby (1980), Jones et al (1980), Toensmeyer and Cook (1981), McDonald (1982), Lyod and Calder (1982), Swamy et al (1982), Holeman et al (1984), Vanden Bosch (1985), Swamy et al (1987), Jones et al (1988), Collins and Cooper (1990), Basunbul et al (1990), Raju and Nadgir (1991), Oscheler (1992). The results of the investigations were encouraging and provided a great amount of technical data in the area of external reinforcement. The major drawback was the corrosion of exposed steel plates which led to the search for innovative, non-corrosive, high strength, and light weight materials such as FRP.

2.1.3 FIELD APPLICATIONS OF STEEL PLATES

Fleming and King (1967) reported that, in South Africa, in a concrete structure some of the reinforcement was accidentally excluded. In order to upgrade the flexural and shear capacity of the beam, thin steel plates were bonded to the bottom and sides of the beam. In the early 1970's, several buildings in Switzerland were strengthened using the same method [McKenna (1992)].

By 1975, more than 200 bridges on an elevated motorway in Japan had been strengthened by bonding of steel plates [Klaiber et al (1987)]. These bridges displayed excessive spalling, cracked slabs, slabs with insufficient thickness and/or insufficient reinforcement. Raithby (1980) reported that, in 1975, the Worcester Highways and Transportation Department had identified four bridges forming an interchange on M5 Motorway at Quinton, England as under-designed bridges. The decks of these two-lane, three span continuous bridges (17m - 27.4m - 17m) were a voided concrete slabs of variable depth. These bridges were frequently used by abnormal vehicles transporting indivisible loads. Three years after these bridges were opened to traffic, the design checks revealed that they were under-designed for the heaviest vehicles. Upon close inspection of the soffit at critical areas, structural cracks were found and load restrictions were posted on those bridges.

To strengthen them, steel plates were bonded to the curved slab soffits. It was reported that a total of 1376 steel plates (for four bridges) of dimensions 3.6 m long, 254 mm wide and 6.5 mm thick plates were needed. The size of the plates was governed by the availability and handling procedures. The bridges were opened to traffic while the plates were being attached. The cost of their strengthening was less than 12% of the estimated cost of replacement [Klaiber et al (1987)].

Before and after strengthening of the bridges, full scale loading tests were performed on one side span of one of the bridges. It was confirmed that, by adding external reinforcement, the flexural stiffness was increased by 11% and the effect of crack opening

for a given load was reduced by 35-40% after the bonding of the plates. The same technique was adopted in strengthening another two highway bridges in the U.K. Both applications involved bonding of 6.5 mm thick steel plates to the underside of the deck slabs [Dussek (1980)]. Also, in the early 1980's, a concrete-through-arch bridge in Switzerland was strengthened by bonding steel plates to the under sides of its floor beams in the positive moment region which cost less than 30% of the cost of replacement [Klaiber et al (1987)].

In Denmark [IABSE (1988)], an arch bridge (45m long, 13.5m wide and 27.5m deep), after 34 years of service, required improved load carrying capacity to meet present standards. Structural evaluation had shown that the two 7m bays on the either end of the arch bridge did not comply with present standards. Therefore, in each of the two end bays, 9 steel plates (400 x 5850 x 6 mm) were bonded to the concrete to upgrade the strength of the bridge. In Finland [AEROBES 1988], another arch bridge of overall length of 54m was strengthened at the edges and widened by one metre by bonding two metre long steel plates to it.

As reported by Klaiber et al (1987), in France, a skewed T-beam highway bridge was strengthened in the 1970's. This bridge suffered a permanent deflection of 80 mm due to fatigue within 10 years of construction. The bridge was strengthened by bonding four layers of steel plates to increase its strength in flexure and shear. In 1974 another T-beam bridge was strengthened in France by the same procedure but the bridge was closed to traffic to avoid vibrations during bonding of steel plates. In Russia, in a 60 year old continuous span bridge, up to 25% of the reinforcing steel had corroded away. Steel plates were bonded in the negative moment region and steel bolts were welded to the exposed reinforcement at the positive moment region while maintaining traffic in single lane at a time. In another application in France, an overhead crane rail in a paper mill was cracked severely near the supports and it was deflecting excessively which was repaired by bonding steel plates.

Klaiber et al (1987) reported that the Shelly bridge in Perth, Australia developed shear cracks in the cantilevered portion of the piers. Three layers of 200 mm x 6 mm steel plates

were epoxy bonded on top of each pier. Also two three-span continuous prestressed concrete bridges in Belgium were rehabilitated in the 1970's by external post tensioning and epoxy bonded steel plates.

In Poland, several reinforced and prestressed concrete bridges have been strengthened by bonding steel plates. This has been found to be the most economical and practical method for strengthening bridges [Klaiber et al (1987), Saadatmanesh and Ehsani (1990)].

The above applications clearly demonstrate the need for and the feasibility of externally bonded reinforcement in concrete structures. Since FRP plates can theoretically fulfill the same function as the steel plates in such applications, repair of deficient structures by FRP plates may be both technically and economically advantageous.

2.1.4 PREVIOUS RESEARCH ON FRP LAMINATES/SHEETS

Some research on strengthening of concrete beams using FRP material was conducted at the University of Arizona [Saadatmanesh and Ehsani (1990)]. Five small size concrete beams (90 mm x 150 mm x 1675 mm) were reinforced with one 9.5 mm rebar for flexure and with 5 mm diameter wires at 75 mm c/c for shear. Four of those beams were strengthened by 6 mm x 75 mm GFRP plates bonded to the full length on the bottom tension side of the beam. In each of the four beams a different type of epoxy was used to bond the GFRP sheet to the bottom of the beam. All beams were simply supported and subjected to four point bending. The objectives of this research were to investigate the load transfer capacities of the four different epoxies.

The first epoxy had a relatively low viscosity at the uncured stage and formed into rubbery texture after curing with a tensile lap shear strength of 13 MPa with aluminum substrate. The thickness of the epoxy applied to the control beam was 1.8 mm. It was observed that the initial stiffness was slightly higher than that of control beam, but the stiffness decreased instantly after the section cracked. The plate was completely separated from the beam

upon formation of a large tension crack but the beam failed in a ductile manner. The flexibility of the epoxy was reported to be too high to be able to adequately transfer the shear forces between the beam and the GFRP plate.

The second epoxy was tougher than the first with a tensile lap shear strength of 14 MPa. The applied thickness of this epoxy was 1.0 mm. The greater stiffness allowed the transfer of the shear forces and contributed to overall stiffness of the beam. Large shear cracks formed prior to failure which eventually led to plate separation and beam failure.

The third epoxy was rubber-toughened epoxy with a tensile lap shear strength of 14-15 MPa with a consistency similar to the cement paste. The applied bond line was 1.6 mm thick. There was no bond failure and the beam achieved an increase in ultimate strength of 100% over the control beam. There were no visible cracks up to 70% of the ultimate strength and the failure occurred by delamination of concrete at the interface of concrete-epoxy-GFRP plate along the full length of the beam. This failure mode was similar to the one reported by McDonald et al (1982). It was reported that the latter epoxy provided good shear transfer and composite action between the plate and the concrete beam.

The fourth epoxy was the most rigid of the four epoxies tested. The test was discontinued because of the formation of early tension cracks which spread immediately along the bond line and which led to the plate separation from the beam in a brittle manner. From the above tests it can be concluded that it is important to select a suitable epoxy for strengthening a beam, for improving its cracking and ultimate behaviour, and for providing it with sufficient stiffness and strength to transfer higher shear stresses without brittle failure.

Saadatmanseh and Ehsani (1991) tested five rectangular beams and a T-beam to investigate their strengthening by means of 152 mm x 6 mm x 4.260 m long GFRP plates with an average ultimate strength of 400 MPa and a modulus of elasticity of 37 GPa. The cross-sections of the rectangular beams were 205 mm x 455 mm and the flange width and thickness of the T-beam were 610 mm and 75 mm, respectively. The height of the T-section was 455 mm and the thickness of its web was 205 mm. All the beams were 4.57 m

long and reinforced with different reinforcement ratios to measure the strength of the upgraded beams.

In one beam, some camber was introduced before bonding of the GFRP plate. The objective was to simulate the effect of external prestressing. The beam was jacked up at mid-span and the GFRP plate was bonded to its bottom. The jack was removed after the epoxy had cured. The results indicated that the cracking load and ultimate moment increased by 100% and 400%, respectively, over the beam without GFRP plates and there was improvement in the cracking behaviour. The failure of this beam was due to the formation of longitudinal cracks at the level of the tensile reinforcement. The ductility of the externally reinforced beams decreased compared to the beam without GFRP plate.

To demonstrate the feasibility of using FRP plates for strengthening the structurally deteriorated concrete beams, ten reinforced concrete beams were tested by Sharif et al (1991). The beams were 150 mm x 150 mm in cross-section and 1200 mm long and were reinforced with two 10 mm diameter bars in tension and 6 mm diameter stirrups at 60 mm c/c. The beams were under-reinforced so that the additional reinforcement would not cause brittle failure. Fiber glass FRP with an ultimate strength of 170 MPa was used in the study. The research program and testing procedure by Sharif et al (1991) were similar to the tests performed by Macdonald (1982) and Jones et al (1988) who used steel plates. Eight of the beams were pre-loaded to 85% of their ultimate capacity which produced a mid span deflection of 10 mm. The beams exhibited a permanent deflection of 4.5 to 5.4 mm and 1.5 mm (average) cracks which propagated to 75% of the depth of the beams.

The damaged beams were repaired to study the effect of the plate thickness (1, 2 and 3 mm) and of the use of steel bolts to anchor the plates near their end in the vicinity of supports. A special plate was placed during construction inside the form as a one-piece I-jacket which was also glued to the tension and shear faces of the beam. Steel anchored bolts eliminated plate separation, however, the repaired beams failed due to diagonal tension cracks formed close to the steel bolts. The increase in plate thickness increased the shear and normal stresses at the plate curtailment, leading to the plate separation. Compared to the control beam, the increase in the flexural capacity for 1, 2 and 3 mm

plates was 26%, 28% and 54% and the stiffness increased by 2%, 45%, and 250% respectively. However, the ductility index was reduced from 3.5 to 1.3 for the beam with 3 mm plate. I-jacket FRP plates provided the best anchorage system to eliminate plate separation and diagonal tension cracks and developed the full flexural capacity of the beams.

An et al (1991) derived an analytical model to calculate the complete moment-curvature and deformations of reinforced concrete rectangular and T-beams strengthened by FRP plates. They also investigated the performance of such beams at yield and ultimate state, considering different variables such as reinforcement ratio, concrete strength, b/t of the plate, and the modulus of elasticity of the plate. The assumptions in their analysis were small deformations, no tensile strength of concrete, no shear deformations and finally no slip between the composite plate and the concrete beam. The results of this analytical model were compared with the results of beams tested by Sadatmanesh and Ehsani (1991) and close agreement was found between the two.

Ritchie et al (1991) conducted tests on sixteen reinforced concrete beams strengthened by pultruded fiber glass sheets/ channel, $0-90^\circ$ GFRP, $0-90^\circ$ with 65% GFRP+35% CFRP, $0^\circ - 90^\circ$ CFRP, $0-+/- 60^\circ$ CFRP, and steel plates. The beams were 182 mm x 365 mm in cross-section and 2.5 m long simply supported length reinforced with 2-13 mm diameter rebars. The increase in strength was observed to be from 40-97% while that of stiffness was from 18-188%. The failure loads of the beams were within 5% of their theoretical values.

Triantafillou and Plevris (1991) investigated the collapse mechanism of post-strengthened beams strengthened by FRP sheets. The strain compatibility method, concepts of fracture mechanics and a model of FRP peeling-off mechanism were used to develop a systematic approach to the study of the behaviour of FRP bonded beams. The assumptions in the analytical model were the same as those of An et al (1991), which are similar to the classical theories of reinforced concrete members subjected to flexure. FRP strengthened beams were assumed to fail in several flexural modes and the pertinent strength equations were developed.

Triantafillou and Plevris (1991) reported the work of Kaiser (1989) on the application of CFRP sheets for post-strengthening of concrete beams and the validity of the strain compatibility method in analysis of those members. A new method in post-strengthening with pretensioned FRP sheets was also introduced.

Triantafillou and Deskovic (1991) presented the mechanics associated with the short-term behaviour of beams bonded with pretensioned FRP sheets on the tension zones. Analytical models were developed to achieve maximum prestress level so that the FRP prestressed system does not fail near the anchor zones. Lower shear strength of the beam material or the adhesives was accounted for to predict failure. The results suggested that the level of the prestress is increased by increasing the thickness of adhesive layer and/or increasing the area of the FRP.

Triantafillou et al (1992), extended the work of Triantafillou and Plevris (1991) and Triantafillou and Deskovic (1991) and investigated application of prestressed CFRP sheets to the concrete beams for flexural strength upgrade. Five beams of 70 mm x 120 mm x 1200 mm reinforced with 4 mm or 6 mm bar for flexure and 4 mm bars at 40 mm c/c for shear were tested. All the under-reinforced beams were strengthened with CFRP sheets with different pre-stressing levels. The procedure for bonding of the CFRP sheet included, first the pre-tensioning of a uni-directional oversize CFRP sheet and then bonding it to the tension face of the beam. The two ends of the CFRP were cut once the epoxy had fully hardened and the sheet was then transformed into a prestressing element. The beams subjected to flexure indicated that the higher steel ratios and pre-stress levels were effective mainly in controlling cracks. Localized peeling off and subsequent slipping of CFRP sheet due to the diagonal shear crack initiated failure of all the prestressed beams. Finally, the beams failed due to the concrete crushing and debonding of CFRP sheets. The failure of the beams was similar to the beams tested by Meier and Kaiser (1991). The beams showed an increase in strength by 3 to 4 times over the control beam. Karam (1992) suggested that the efficiency of this method is controlled by the maximum achievable prestress level so that the FRP-prestress system does not fail near the anchorage zone due to the shear stress concentrations that arise from the discontinuity at

the end of the member when the prestressing agent is released. It was proposed to increase the efficiency by designing special anchorage zones.

Meier and Kaiser (1991) reported the success of application of CFRP sheets for the post strengthening of steel-reinforced concrete beams. Twenty-six beams of 150 mm x 250 mm x 2400 mm with a simply supported length of 2000 mm and one beam of 7000 mm were reinforced with 2-8 mm diameter steel rebars in tension and compression and 6.0 mm diameter bars in shear at 220 mm c/c. The beams were strengthened by bonding of 200 mm wide unidirectional CFRP sheets of 0.3 mm to 1.0 mm thickness. Strengthening with 0.3 mm x 200 mm CFRP laminates almost doubled the ultimate capacity and stiffness of the 2.0 m beams, while the 7.0 m beam with the same reinforcement as the 2.0 m beam but bonded with 1.0 mm x 200 mm CFRP laminate, showed only 22% increase in strength. The tensile stresses were carried by the steel and by the externally bonded CFRP laminate and upon yielding of the internal steel, the CFRP laminate was responsible for the additional load carrying capacity of the beams. Five failure modes were observed:

- Tensile failure of the CFRP laminate with a sharp explosive snap.
- Compression failure as the concrete reached its ultimate strain.
- Continuous peeling-off of the CFRP laminate due to uneven concrete surface.
- Sudden peel-off during loading due to development of shear cracks.
- Sudden peeling-off due to vertical displacement.

The difference in the coefficients of thermal expansion of concrete and CFRP could result in stresses at the bond line with changing temperatures. However after 100 frost cycles ranging from +20° C to -25° C, no negative effect on the load capacity of the post-strengthened beam was observed.

Meier and Kaiser (1991) studied the influence of bonded CFRP laminate on the development of bending cracks. It was noted that in an unstrengthened beam at a load of 19 kN, seven bending cracks with a total width of 3.85 mm, were observed. Upon unloading, the residual width of these cracks was 1.63 mm. The unloaded beam was then

strengthened with a 0.75 mm thick and 200 mm wide CFRP laminate and reloaded to a force of 30 kN. Four additional cracks formed and the total width of the eleven cracks was only 2.58 mm and they were evenly distributed over the length of the beam.

Meier et al (1992) reported the work of Kaiser on the fatigue test of a 2.0 m beam (300 mm x 250 mm) post-tensioned with a 0.3 mm x 200 mm hybrid FRP sheet. The beam was reinforced with 2-8 mm diameter rebars in tension and compression. The tensile strength of the hybrid sheet was 960 MPa, with a Young's modulus of 80 GPa and an ultimate strain of 1.15%. The goal of this test was to observe the behaviour of the beam after rupture of the tensile reinforcement. The first fatigue failure occurred in one of the rods in the tension zone after 480,000 cycles at a frequency of 4 Hz. After 610,000 cycles, a further break was observed in the first rod at a different cross-section, while after 720,000 cycles the second rod failed. The first damage appeared after 750,000 cycles in the form of fractures. The hybrid sheet finally failed after 805,000 cycles. It was mentioned that the test was executed with unrealistically high steel stresses.

A theoretical model based on classical reinforced concrete theory using rectangular stress blocks was developed by Deblois et al (1992). To verify the model, an experimental program was carried out using large and small beams. The small 125 mm x 125 mm x 1000 mm beams were strengthened by GFRP plates of 4.75 mm thick unidirectional plates and 6.35 mm bi-directional plates. A new technique of anchoring FRP sheets with adhesively bonded and bolted connections was used. The tensile reinforcement consisted of 3-11.3 mm diameter bars and shear reinforcement consisted of 26-2 mm diameter bars at 40 mm c/c. The variables in the test include uni- and bi-directional sheets epoxy bonded and bi-directional sheets epoxy bonded along with bolts at the ends. It was reported that bi-directional sheet in combination with epoxy bonding and bolting along with an additional bonded unidirectional sheet produced a maximum increase in strength of 77%. Epoxy bonded and epoxy bonded + bolted bi-directional sheets increased the load capacity by 32 % and 34%, respectively. Epoxy bonded unidirectional sheets increased the ultimate load by less than half the increase achieved by the use of bi-directional sheet. The large beams were 300 mm x 200 mm x 4100 mm reinforced with 2-16 mm diameter bars in

tension, 11.3 mm diameter stirrups at 120 mm c/c in shear. The GFRP bi-directional sheet was epoxy bonded + bolted. The ultimate load capacity was increased by 66% over the control beam. It was concluded that the ultimate load of externally bonded FRP sheets was well predicted by the analytical model.

Uji (1992) successfully attempted to improve the shear capacity of existing reinforced concrete members by externally bonding CFRP sheets in the shear zone. Eight concrete beams (100 mm x 200 mm x 1300 mm), consisting of 2-16 mm diameter bars in tension and compression, and 6 mm diameter stirrups at 70 mm c/c spacing (shear reinforcement of 0.81%) were used. Internal shear reinforcement was present in one of the beams in each of the three sets. The unidirectional CFRP external stirrups were bonded in one-ply and two-ply in vertical and diagonal arrangement. It was found that application of CFRP stirrups substantially improved the shear capacity of the member without any internal stirrups. On the other hand, strain in the internal and external CFRP stirrups was different at the same location for the members which contained both types of stirrups. Due to bond between the CFRP stirrups and the concrete, the strain in the CFRP stirrups increased drastically in the limited area near the crack. It was concluded that CFRP stirrups carry greater shear force than calculated from the ratio of the cross-sectional area of CFRP stirrups and the internal stirrups.

A series of beams were tested by Tonen Corporation (1992) to measure the basic flexural properties of reinforced concrete beams externally bonded by one-ply or three-ply of CFRP sheets. The test beams were 300 mm x 400 mm x 2500 mm, reinforced with 3-19 mm diameter bars in tension, 2-19 mm diameter bars in compression and 13 mm diameter stirrups at 20 mm c/c. For some of the beams, the CFRP sheets were bonded to the initially damaged beams. The flexural tests indicated that the strength of the initially damaged beams externally reinforced with one or three-ply of CFRP sheets increased by 34% and 48%, respectively.

Meier et al (1992) reported the work of Deuring on the fatigue test of 6.0 m long T-beams with the a flange of 160 mm x 900 mm and a web of 260 mm 340 mm. The beams were reinforced with 4-26 mm diameter bars in the tension zone, 42-10 mm diameter bars as

shear reinforcement distributed along the web over a length of 1880 mm from each support, 42-6 mm diameter stirrups were used as transverse reinforcement in the flanges and 4-10 mm diameter bars in the flange to hold the flange horizontal reinforcement. The load carrying capacity was increased by 32% after bonding of a 1.0 mm thick and 260 mm wide non-prestressed or prestressed CFRP sheet. It was observed that the increase in stiffness of the pretensioned CFRP sheet was higher than the non-prestressed bonded CFRP sheet. The beams were subjected to the design ultimate load for 10.7 million cycles, under variable humidity and temperature to determine the effects of climatic condition with simultaneous fatigue loading. After 12 and 14.09 million cycles, the first and second tensile reinforcement failed due to fretting fatigue. It was noted that there was no sign of any problem at the interface of concrete and CFRP. After failure of the third and fourth tensile rebars the CFRP sheet sheared off.

Meier et al (1992) described the observed failure modes as:

Mode	-1	Tensile failure of the CFRP sheet; in advance the cracking sounds of CFRP sheet announced the impending failure.
	-2	Classical concrete failure in the compressive zone of the beam.
	-3	Continuous peeling-off of the CFRP sheets due to uneven concrete surface.
	-3	Shearing of the concrete in tensile zone (also observed as secondary failure).
	-4	Interlaminar shear failure within CFRP sheet (observed as a secondary failure).
	-5	Failure of the reinforcing steel in the tensile zone (observed during fatigue tests).

Meier et al (1992) stated that the following failures are theoretically possible but not yet observed during tests:

	-6	Cohesive failure within the adhesive.

- 7 Adhesive failure at the interface of the CFRP sheet/adhesive and
- 8 Adhesive failure at the interface of the CFRP/concrete/adhesive.

The experimental program by McKenna (1993) consisted of externally bonding CFRP and GFRP sheets to eighteen reinforced concrete beams and thirty plain concrete beams. The reinforced concrete beams (150 x 300 x 2000 mm) were externally strengthened by 1, 2, 3 and 6 layers of unidirectional CFRP sheets on initially un-cracked or pre-cracked beams. All the reinforced concrete beams contained 700 mm² of tensile reinforcement and 200 mm² of compression reinforcement, with 11.3 mm diameter stirrups spaced at 95 mm. The plain concrete beams (150 x 150 x 1000 mm) were strengthened using CFRP and GFRP sheets with the number of layers varying from 1 to 6. Also a combination of CFRP and GFRP sheets bonded to plain concrete beams was investigated.

The plain concrete beams externally reinforced with CFRP sheets of 150 mm wide, 0.5 mm, 1.0 mm, 1.5 mm or 3.0 mm thick showed an increase in average maximum load proportional to the number of layers, i.e. 18%, 53%, 105% and 195% for 1, 2, 3, and 6 layers. Sudden and explosive failure without cracking sounds was observed in 3 and 6 layered beams.

In the plain concrete beams externally reinforced with GFRP sheets of 0.5 mm x 125 mm, the increase in load was 42%, 70% and 136% compared to the control beam with no GFRP sheet. However, there was no decrease in the corresponding deflections upon increase in GFRP sheet layers as observed in CFRP sheet layers. The GFRP beams deflected more and were more ductile than the CFRP reinforced beams. The load-deflection behaviour was more repeatable in the CFRP beams than GFRP beams.

In the reinforced concrete beams, 125 mm wide x 0.5 mm thick CFRP sheets were bonded. A definite trend was observed with increase in the number of layers of CFRP sheets. For the no pre-load beams, at maximum, the increase in load was linearly proportional to the number of sheets applied, specifically 4%, and 9% at yield, 5.6%, 10% and 15.1% at ultimate for 1, 2 and 3 layers of FRP sheets. The corresponding deflections

were reduced by 63% and 71% at yield and 61%, 63% and 67% at ultimate. A relatively symmetrical crack pattern for the various layers was observed. The height of cracks in flexure zone was reduced by 33%, and there was a substantial increase in the flexure-shear cracks in the combined moment and shear zone. The yielding of reinforcement produced secondary cracks, first from the flexure-shear cracks and then immediately before ultimate, from the flexural cracks in the constant moment region. In multi-layered beams, horizontal cracking at the level of reinforcement joined the flexural cracks in the constant moment region which would indicate loss of bond between concrete and the reinforcing bars. The failure of the beams was due to the rupture of the sheet in the constant moment region or due to sheet delamination at the concrete-epoxy resin-sheet interface. It was observed that, for both failure modes, the failure stresses in the FRP sheets were the same, which indicates that a critical crack width is required to initiate failure of bonded sheet.

Ichimasu et al (1993) conducted extensive analytical and experimental investigation to strengthen reinforced concrete slabs by bonded CFRP plates. Special attention was given to the application of this method in confined places such as box girders. Applicability test and bending resistance tests were conducted on beams and on full scale slabs. Eight reinforced concrete specimens were bonded with CFRP plates. Four different types of strengthening materials were used:

- Pre-peg (pre-impregnated) uni-directional carbon fibers
- Cloth: bi-directional carbon fiber cloth
- Cloth pre-peg: this cloth is bonded under thermosetting conditions (150° C for 30 min.)
- Hardened cloth plates: a number of pre-peg sheets are bonded together to produce hardened cloth.

The results indicated that the stress in the reinforcing bar (140 MPa) under service loads was 1.6 to 2.3 times less than in the unstrengthened beams while the rebar stress intensity

was reduced from 38 to 56% of the stress in the unstrengthened beam. The maximum load was achieved by pre-peg bi-directional sheets.

In another case [Ichimasu et al (1993)], ten reinforced concrete beams of the same properties were tested to examine the relationship between CFRP type, strengthening quality, anchoring method, strengthening area and bending resistance. The beams were preloaded until the tension steel stress at mid span reached 180 MPa (which represents damaged slabs). Two layers of CFRP plates were then bonded to the tension face. It was observed that the rebar stress under service loads was 30% less than the un-strengthened beams. The bending resistance was found to be proportional to the tensile rigidity of the CFRP plates.

Four full size reinforced concrete 2.5m square slabs [Ichimasu et al (1993)], 180 mm thick, were strengthened by 200 mm wide CFRP plates. Cyclic loading on the slabs showed up to 80% increase in the load which would induce a stress of 140 MPa in the rebars. The bending resistance was also proportional to the tensile rigidity of the CFRP up to the point when the rebar reached yield. Sacki et al (1993) conducted strengthening of damaged concrete beams by external prestressing of aramid fiber cable. The pre-damaged beams were strengthened by external post-tensioning. Static as well as fatigue tests (2 million cycles) indicated that the strength was increased by 33% over the un-strengthened beam.

Char et al (1994) conducted a parametric study of the flexural strength of 8-concrete girders with externally bonded (non-prestressed and prestressed) FRP sheets. The beams consisted of 2100 mm x 200 mm flanges and 1300 mm x 600 mm web reinforced with 3-No. 9 bars in tension and 5-No. 5 bars in compression. GFRP and CFRP laminates with ultimate strength of 1166 MPa and 2400 MPa and with modulus of elasticity of 51 GPa and 158 GPa, respectively, were used in the study. Two different areas of each type of plate was used to study the effect of the plate size on the strength and ductility of the upgraded girders. Half of the girders were cambered by jacking the girders upward while in loose contact with the epoxy-coated composite plate. The additional moment created by the tensile force in the GFRP and CFRP plate resulted in significant increase in the

moment carrying capacity of the girder. Increasing the area of the plate from 25.8 to 51.6 cm² further increased the moment carrying capacity of the girder. The failure of the upgraded beam was initiated by the GFRP and CFRP plates reaching their ultimate capacity.

To verify the strain compatibility method to determine the stresses and deformations, Char et al (1994) conducted a test on rectangular beams with dimensions of 205 mm x 455 mm and 4.57 m, reinforced with 2-No. -4 bars in tension and compression. The beams were cambered before the 6 mm x 150 mm GFRP plate was bonded to their tension face. The closeness of the theoretical values and experimental values suggested the validity of the strain compatibility method.

2.1.5 FIELD APPLICATIONS OF FRP LAMINATES

The California highway department has recently strengthened several highway bridge columns by wrapping FRP to provide increased structural strength and safety during earthquakes [Ballinger (1992)]. Rostasy et al (1992) reported strengthening of Kattenbusch bridge in Germany with bonded GFRP laminates. The Kattenbusch bridge is a continuous eleven span post-tensioned double box girders. Due to temperature restraint in summer time, wide cracks appeared at the working joints. All the working joints were strengthened using steel and GFRP plates which reduced the working stresses by 33%. The Ibach bridge in Switzerland is a multi-span box girder bridge with overall length of 228 m in which one of the main pre-stressing strands in the outer webs was accidentally severed by the core boring operation for a traffic signal. The damaged span was 39 m and 16 m wide. It was estimated that 175 kg of steel plates would have been required to strengthen the bridge to its original state compared to only 6.2 kg of CFRP plates that was used instead. The bridge was repaired by bonding 2 CFRP sheets of 150 mm x 5000 mm x 1.75 mm and one sheet 150 mm x 5000 mm x 2.0 mm on a mobile platform at night.

Many other applications of FRP sheets and laminates have been reported in the literature [ACI (1996), RILEM (1994)].

2.2 INTERNAL FRP REINFORCEMENT

Research has been conducted to determine the feasibility of reinforcing concrete structures using FRP rebars instead of steel rebars. Investigations were carried out by several researchers to determine the structural properties of the FRP rebars.

Ehsani et al (1993) presented the results of twenty of 78 specimens to be tested to determine the bond strength of GFRP rebars to ordinary concrete. The variables included mode of failure (pullout or splitting), concrete compressive strength, bar diameter and clear cover distances. For hooked bars the longer radius of bends showed higher stiffness, while smaller radii of bend failed by breaking the bar at the bend at very small load. It was also observed that the bond strength of straight FRP rebars is lower than that of steel.

Faza and Ganga Rao (1991) conducted large scale testing of the FRP reinforcement to the concrete members. They conducted twenty four cantilever and twelve straight pull out specimens to determine the bond mechanism of the FRP bars. They also investigated the response of the FRP reinforced beams for pre- and post cracking behaviour, load-deflection and stress variations, elastic and ultimate load carrying capacities, modes of failure, crack width and patterns. They concluded that, in order to take full advantage of the high strength of FRP rebars, high strength concrete should be utilized. A new method based on effective cracked section was proposed to determine the deformations of FRP reinforced beams.

Tezuka et al (1994) tested five two span continuous beams, reinforced or prestressed with FRP or conventional steel wires. The objective was to determine the moment redistribution capacity and ductility of such beams when failure was initiated by concrete in the compression zone. They reported that 14-30% moment redistribution occurred in

the FRP reinforced concrete beams versus 11% redistribution of negative support moment in the prestressed beam. Such a redistribution occurred despite the linear elastic behaviour of the reinforcement. It was noted that the beams flexural rigidity changed dramatically due to shear cracking, an event which is ignored in conventional moment curvature response of beams.

In 1981/82, the first GFRP highway bridge was constructed in Bulgaria. Also the Chinese completed a GFRP bridge in Beijing in 1982. This bridge consisted of six box girders with a depth of 1.67 m spanning 20.6 m with a total width of 11.38 m [Mufti et al (1991)]. A 12 m I-beam GRP bridge was built in Bulgaria [Meier (1992)].

In 1986 the Ulenbergstrasse bridge in Berlin was the first highway bridge to be constructed using GFRP prestressed tendons. The two spans of 21.3 m and 25.6 m were reinforced with 59 GFRP tendons each composed of 7.5 mm diameter which provided 600 KN of working load per unit. Also in 1986, two GRP pedestrian bridges were built in China. The first one is a single tower cable-stayed bridge spanning a main street. The main span is 27.5 m and consists of GRP box girder. The second bridge was built across a busy intersection. The bridge consisted of eight GRP girders of 19 m long, 4.3 m wide and 0.9 m depth. In 1987, the world's first major GFRP cellular construction system was incorporated in a composite steel-concrete bridge with its longest span being 117 m. The GFRP protection system was reported to reduce the rate of corrosion of the steel plate girders by 94% [Mufti et al (1991)].

The first Canadian CFRP prestressed bridge was completed in 1993 in Calgary. The two span continuous bridge comprises 13 bulb-tee section prestressed girders in each of 22.83 m and 19.23 m spans. The bridge has been open to traffic and has been functioning normally.

2.3 DUCTILITY AND DEFORMABILITY IN FRP REINFORCED CONCRETE MEMBERS

Extensive research has been carried out throughout the world on FRP reinforcement and its applications in concrete [JSCE (1993), ACI (1994), Mufti et al (1991 and 1992), Nanni and Dolan (1993), Taerwe (1995), Neale and Labossiere (1992)]. One of the most important issues pertains to the lack of ductility in FRP reinforced and/or prestressed concrete members. A number of methods have been suggested to improve ductility [Naaman (1993)], including provision of different types of FRP (e.g. CFRP and GFRP) within the same member, placement of FRP reinforcement in several layers within the beam, use of combined prestressed and non-prestressed FRP, use of bonded and unbonded reinforcement, confinement of concrete in the compression zone, and addition of fibres to concrete.

Although theoretically each of these methods can achieve certain level of ductility, to date only three methods have been actually tested. Naaman et al (1993) tested beams prestressed with CFRP tendons and containing non-prestressed mild steel. They concluded that the presence of steel allowed the beams to undergo large deflections before failure, and depending on the amount of steel, the behaviour of such hybrid reinforced beams could be same as beams prestressed with steel only. Although the latter system achieves ductility, it is our opinion that the presence of steel will eventually lead to corrosion and is contrary to the fundamental reason for the use of FRP reinforcement. Kato and Hayashida (1993) tested prestressed beams, with unbonded CFRP tendons and bonded CFRP reinforcement. The presence of bonded CFRP increased the ductility and prevented explosive failure. Unfortunately, such design may not be economical if an acceptable percentage of the ultimate moment capacity of the beam is to be maintained following the rupture of prestressed tendons. Taniguchi et al (1993) used FRP spiral confinement reinforcement in the compression zone of the beams to study their ductility. For a confinement reinforcement ratio of 0.6% and a main reinforcement index (not ratio) of 0.4, a curvature ductility factor of 4 was achieved. This method appears promising

although it may create reinforcement congestion and construction difficulties. Albderahman et al (1995) tested models of the CFRP prestressed bridge girders used in the Beddington Bridge in Calgary. Using a new definition of curvature ductility ratio, they reported ratios of 6.3 and 8.2 for their tested beams.

The concept of combining reinforcing material with different strength and stiffness to achieve ductility and load redistribution has been employed by some investigators in the past. Elgabroni (1989) reported that the use of polymer grids (Tensar) with the combination of high strength steel increased ductility by 75 % over the beams without polymer grids. In his tests, thirteen 150 mm x 300 mm x 2200 mm beams were reinforced with high strength steel in combination with polymer grids placed in varying configurations. The beams reinforced with four layers of Tensar UX-1600 along with the steel reinforcement in tension produced an average increase in ductility index of approximately 57% whereas with two layers of Tensar it increased by 49%.

A conceptual model was also developed [Elgabroni (1989)] for such an increase in ductility in beams reinforced with steel and polymers. It was stated that the increase in the maximum deflections (inelastic rotations) of these beams is probably due to the early deterioration of the bond between the steel bar and the surrounding concrete after the yielding of the steel. Because the bond stress between the reinforcing bar and the surrounding concrete is a critical factor in the inelastic rotations of the flexural members, a higher bond strength reduces the inelastic rotations or vice versa. The presence of Tensar grids affects the bond between the steel and the surrounding concrete and it may also shift the beam action mechanism to arch action mechanism.

Triantafillou and Meier (1992) combined different FRP materials with concrete to produce the desired strength, stiffness and pseudo-ductile characteristic in a hybrid member. The concept behind this beam was to avoid sudden catastrophic failure and to increase its stiffness (reduce deformations). The latter two characteristics need improvement because FRP are generally linearly elastic and brittle and they possess low moduli of elasticity. Triantafillou and Meier (1992) concluded that a concrete layer can substitute the GFRP compressive flange while the bottom section would be made of two materials, CFRP and

GFRP, in which one will fail in tension before than the other giving an indication of imminent collapse (pseudo-ductile response).

Kakizawa et al (1992) experimentally investigated cracking behaviour, failure properties and energy absorption of FRP reinforced concrete (RC), prestressed (PC) and partially prestressed (PPC) members. Sixteen simply supported rectangular beams with dimensions of 100 mm x 150 mm x 1700 mm using varying amounts of CFRP reinforcement were tested under monotonic loading. Results showed that, inspite of having the same amount of reinforcement, PPC members tended to indicate greater energy absorption than PC. Thus, it is possible to obtain various energy-absorption properties by controlling the reinforcing system. It was recommended that ductility be evaluated based on the energy absorbed before ultimate load.

Naaman (1993) reviewed the concept of ductility in prestressed and partially prestressed FRP reinforcement. It was stated that it is serious and dangerous to assume that because large deflections in FRP reinforced structures can be achieved, then ductility requirements would be satisfied. Large deflections may be due to low modulus of elasticity of the FRP reinforcement which may correspond to a large amount of elastic energy stored in the system. In a structural member, the reinforcement can sustain sufficient inelastic strain to allow concrete to reach its ultimate strain before failure. Before the concrete failure, reinforcement undergoes large inelastic strains that absorb large amounts of energy. For the steel tendon, the elastic energy released at failure can be separated from the inelastic energy and the ratio of inelastic energy consumed to elastic energy released is generally greater than 10. FRP tendons do not consume any inelastic energy prior to failure due to the lack of yield plateau, hence upon failure they release large amounts of elastic energy. In the case of prestressed members, even though the deflections of the beams with FRP tendons and steel tendons are of the same order, the elastic energy accumulated is two to three times larger, which could lead to sudden and catastrophic failure.

It should be pointed out that the notion that plastic energy dissipation is a sine que non for ductility is not universally accepted. While the traditional definition of ductility has been based on the ratio of plastic to elastic deformations and/or energy ratios, some experts

argue that such a definition is too restrictive and may not be needed to satisfy such structural performance requirements as early warning of impending failure and load redistribution. The concept to be proposed in the following chapter is intended to satisfy the latter requirements rather than plastic energy dissipation.

Chapter Three

A NEW CONCEPT FOR ACHIEVING DUCTILITY IN FRP- REINFORCED CONCRETE MEMBERS

3.1 General

To survive severe earthquakes, structures designed to the level of seismic loading recommended by the code need to be capable of ductile behavior at critical sections while undergoing horizontal displacements in the inelastic range [Park and Ruitong (1988)]. Also ductility is considered to be a desirable property of reinforced concrete elements; it gives the member the resilience to deal with dynamic loads, alleviates brittle failures, eliminates the need to include in the strength design such forces as those resulting from settlements, temperature, and shrinkage [Scholz (1990)]. These characteristics are generally possessed by steel reinforced concrete members.

- It is widely believed by several researchers that it is feasible to replace in some cases steel reinforcement in concrete by fiber reinforced plastic (FRP) reinforcement and that such substitution will overcome the problems arising from corrosion of steel reinforcement. However, due to the relatively low modulus, linearly elastic behavior and limited strain capacity of FRP's, concrete members reinforced with FRP reinforcement exhibit a brittle failure mode. Hence, lack of ductility in such members is one of the key issues facing researchers.

3.2 DUCTILITY VERSUS DEFORMABILITY

Ductility is commonly defined as the ability to undergo large inelastic deformations prior to failure. Although energy absorption by inelastic deformations is necessary for ductile response of structures under seismic loads, in most cases ductility can be assumed to exist if the deformations just before failure are significantly larger than those under normal service loads. This type of ductility would give sufficient warning prior to failure, but if the deformations are mostly elastic, then the failure would be non-ductile and there would be no redistribution of moments in statically indeterminate structures. Ideally, a ductile reinforced concrete member would maintain its ultimate moment capacity despite its loss of stiffness and the attendant large deformations. This is the kind of ductility that exists in conventional steel reinforced concrete, but FRP reinforced concrete to date lacks such ductility. Of course, the ability to undergo large deformations but with substantial drop in resisting moment cannot in reality be deemed as ductility.

Deformability in a reinforced or prestressed concrete flexural member generally refers to the ability to undergo large deformations without loss of strength. Deformations of flexural members depend on several variables such as the extent of cracking, amount of reinforcement, creep, shrinkage, history and type of loading and the flexural rigidity of the member. To date, most FRP reinforcing bars do not possess the modulus of elasticity approaching that of reinforcing steel, i.e., 200 GPa. Therefore, FRP reinforced members exhibit excessive deformations which are commonly misunderstood by many researchers as ductility.

3.3 BASIC CONCEPT

In this thesis, a hybrid reinforcement system is proposed which consists of a low modulus polymeric reinforcement combined with CFRP or GFRP and which is believed to

overcome the lack of ductility in FRP reinforced concrete members. The proposed concept allows the designer to judiciously select the desired level of ductility or deformability index which is given by ψ_u/ψ_r , where ψ_u is the ultimate curvature of the member while ψ_r is its curvature at rupture of the main FRP reinforcement. Since one component of this hybrid system is anticipated to be relatively cheap, it is expected that the proposed reinforcement system would marginally increase the cost of construction compared to members reinforced with FRP only. It should be emphasized that the proposed system is not intended to increase the dissipation of energy by plastic deformations but it aims to achieve large deformations and loss of stiffness without loss of resistance.

3.4 A CONCEPTUAL FRP HYBRID BAR

Consider a conceptual hybrid reinforcing bar made up of two materials as shown in Figure 3.1. Let the core be CFRP/GFRP (material 1) and the outer shell be a high strength polymeric material with low tensile modulus and large strain capacity such as high density polypropylene, HDPP, (material 2). Let us now consider CFRP, HDPP and GFRP with their attendant properties shown in Figure 3.2. Although the HDPP exhibits nonlinear behavior, for the purpose of the present analysis, we assume that it can be represented by a linear elastic curve as shown in Figure 3.2. Using the following material properties $E_{cr} = 140 \text{ GPa}$, $\epsilon_{cr \max} = 1.23\%$, $E_{gls} = 40 \text{ GPa}$ and $\epsilon_{gls \max} = 2.49\%$, $E_{pp} = 2.0 \text{ GPa}$ and $\epsilon_{pp \max} = 12\%$ where E and ϵ are the elastic modulus and the strain, respectively while the subscripts cr, gls, pp denote carbon, glass and polypropylene. If the hybrid bar is subjected to axial tension then, assuming perfect bond between the core and the shell, its stress-strain behavior can be characterized by a bilinear curves such as that shown in Figure 3.3 and 3.4. Using first principles, it can be easily shown that the stress-strain relation of the hybrid bar is given by

$$F = (E_1 A_1 + E_2 A_2) \epsilon$$

$$F = E_2 A_2 \left(\frac{E_1}{E_2} \frac{A_1}{A_2} + 1 \right) \epsilon$$

$$F = E_2 A_2 (n\rho + 1) \epsilon$$

$$\frac{F}{A_1 + A_2} = \frac{E_2 A_2}{A_1 + A_2} (n\rho + 1) \epsilon$$

$$\text{but } \rho = \frac{A_1}{A_2} \Rightarrow A_1 = \rho A_2$$

$$\frac{F}{A_1 + A_2} = \sigma = \frac{E_2 A_2}{\rho A_2 + A_2} (n\rho + 1) \epsilon$$

$$\sigma = E_2 \left(\frac{1 + n\rho}{1 + \rho} \right) \epsilon \quad \dots 3.1$$

But if

$$\epsilon_{u_2} \geq \epsilon > \epsilon_{u_1}$$

$$\sigma = E_2 \epsilon \quad \dots 3.2$$

Where σ = Nominal bar stress = $F / A_1 + A_2$

ϵ = Bar strain

F = Applied axial tension force

$$\rho = A_1 / A_2$$

A_1 = Cross-sectional area of material 1

A_2 = Cross-sectional area of material 2

E_1 = Modulus of elasticity of material 1

E_2 = Modulus of elasticity of material 2

n = Modular ratio E_1/E_2

ϵ_{u_1} = Ultimate strain of material 1

ϵ_{u_2} = Ultimate strain of material 2

Note that the bar will not have a single stress corresponding to any single strain because the stress in material 1 will not be same as in material 2. The nominal stress may be fictitious stress but it is expected to be close to the stress of the higher modulus material before the rupture of that material.

It is clear that the stress-strain relation and strength of the hybrid bar depends on the modular ratio and the relative cross-sectional areas of the core and the shell. By selecting proper values of n and ρ , one can tailor the stress-strain behavior of the hybrid bar to one's need.

Considering the material properties in Figure 3.2 and assuming $\rho = 0.1$, $n = 73.5$, and using Equations (3.1) and (3.2), one can show that the stress-strain curve of hybrid bars with these properties will be as illustrated in Figure 3.3 and 3.4. Note that, upon rupture of the FRP core, such a bar will exhibit a temporary drop in its capacity. In Figure 3.3 and 3.4 the point A corresponds to the rupture of the FRP core which leads to a dramatic increase in strain (point C) and a significant reduction in stiffness (slope of BD). The drop from A to B is of academic interest because in reality the jump from A to C occurs almost instantaneously. From a practical point of view the bar can be considered to follow the curve OACD. Clearly, this bar has the stress-strain characteristics of an elasto-plastic material with a significant reduction in stiffness following the rupture of the core material. Hence, it is possible to produce an elasto-plastic FRP bar which can undergo large

deformations and a change of stiffness well before rupture. The word plastic here does not connote dissipation of energy through permanent deformations.

To date, no such bar exists. Some attempts have been made by FRP manufacturers to combine different fibers in the same bar. For example, NEFMAC CG grids are made of bars combining glass and carbon fibers within the same matrix. However, the stress-strain behavior of the latter hardly differs from the pure carbon fibre bars. Since such a bar does not currently exist, in the present investigation, the two materials will be placed adjacent to each other within the concrete and they will be bonded by the surrounding concrete. A high density polypropylene grid (HDPP) was combined with CFRP to produce the hybrid reinforcement system. Subsequently, large scale concrete beams will be constructed and reinforced with this bar system.

3.5 BEHAVIOUR OF BEAMS REINFORCED WITH CFRP-HDPP HYBRID SYSTEM

Consider a typical concrete beam with dimensions shown in Figure 3.5, and let us assume that it is reinforced with conventional steel with $\rho = 0.75 \rho_b$, where ρ_b is the so-called balanced reinforcement ratio. The concept of balanced reinforcement ratio for conventional steel and FRP bars is described in Chapter Six. Using the conventional methods of analysis and assuming $f'_c = 40$ MPa and $f_y = 400$ MPa, the nominal ultimate strength of this beam can be calculated and would be equal to 299.51 kN-m (ultimate load = 449.38 kN under four point bending).

Now let us design for the same ultimate strength beams with identical dimensions and other properties except that the reinforcement would be either CFRP or the proposed carbon fibre polypropylene hybrid bar. Using the strain-compatibility method and the stress-strain relations of CFRP bar in Figure 3.2 and of the CFRP hybrid bar in Figure 3.3,

we can determine the required amount of A_{cr} or A_{hb} , where subscript hb designates the hybrid. These will equal 546.6 mm^2 of CFRP only or $450 \text{ mm}^2 : 6250 \text{ mm}^2$ of CFRP : HDPP respectively.

Let us now determine the moment-curvature response of the three beams using the procedures described in Chapter Six. The moment-curvature relations for the three beams are plotted in Figure 3.6. We notice that the beam with CFRP reinforcement shows an essentially bilinear and brittle response while the hybrid reinforced and steel reinforced systems show a more ductile response. Notice also that the addition of the HDPP shell increases strength, stiffness and the deformability index of the hybrid reinforced section. The load-deflection curves of these three beams are shown in Figure 3.7. The increase in stiffness (rigidity) is quite noticeable and it can be estimated by calculating the average slope of the second part of each curve in Figure 3.6.

$$(EI)_{CFRP} = \left(\frac{m}{\psi} \right)_{CFRP} = 1.1 \times 10^{13} \text{ N.mm}^2$$

$$(EI)_{hybrid} = \left(\frac{m}{\psi} \right)_{hybrid} = 1.69 \times 10^{13} \text{ N.mm}^2$$

$$(EI)_{steel} = \left(\frac{m}{\psi} \right)_{steel} = 3.71 \times 10^{13} \text{ N.mm}^2$$

The deformability index, ψ_u / ψ_r , for the FRP reinforced beams is

$$(Deformability \text{ Index})_{CFRP} = 0.0$$

$$(Deformability \text{ Index})_{hybrid} = 2.36$$

For the steel reinforced beam, the ductility index is the ratio of the curvature at ultimate to the curvature at yield which for the current beam is 3.0.

Hence the stiffness of the hybrid reinforced beam is almost 50% higher than the CFRP reinforced beam and it will undergo large deflections before it reaches its ultimate load. It will give adequate warning of impending failure contrary to the CFRP reinforced beam. This concept is experimentally tested as described in the following chapters.

Figure 3.1 Conceptual Hybrid Composite Reinforcing Bar

Figure 3.2 Stress-Strain Relationship of CFRP, GFRP and HDPP

Figure 3.3 Stress-Strain Relationship of CFRP Hybrid Reinforcing Bar

Figure 3.4 Stress-Strain Relationship of GFRP Hybrid Reinforcing Bar

SECTIONAL ELEVATION
1:25

SECTION 1
1:5

FIGURE 3.5: CONCEPTUAL BEAM DETAILS

Figure 3.6 Moment-Curvature Relationship of Three Identical Beams with Three Different Types of Reinforcement Designed To Carry Equal Moment.

Figure 3.7 Load-Deflection Curves of Identical Size Beams With Three Different Types of Reinforcement Designed To Carry Similar Loads.

Chapter Four

EXPERIMENTAL PROGRAM

4.1 GENERAL

The proposed concept of achieving ductility in FRP reinforced beams is experimentally investigated. This chapter provides the details of the test program.

For the beams externally reinforced with FRP sheets, the internal reinforcement provided will form the second component of the hybrid reinforcement system. Since these beams do not contain any steel reinforcement, they also serve a few other purposes as follows:

- To study the behavior of beams externally reinforced with FRP sheets.
- To determine the increase in strength and stiffness using one ply and two plies of FRP sheets.

For the beams internally reinforced with FRP rebars, the main objective will be to study these beams for the strength, stiffness and ductility. The experimental program consisted of mainly two phases:

The first phase dealt with the application of CFRP sheets for strengthening and/or retrofit of existing structures to increase and/or restore their strength and stiffness. It included testing of four simply supported beams under monotonic loading. To simulate the effect of strengthening, two beams were not reinforced with any kind of internal steel reinforcement in flexure or shear. The beams are denoted on the basis of their reinforcement configuration. Beams reinforced with conventional reinforcing steel are denoted by SRC (Steel Reinforced Concrete) beams, and the beams externally reinforced with CFRP sheet

and internally reinforced with high density polypropylene (HDPP - Tensar) grids are denoted by CLTRC (Carbon Laminate Tensar Reinforced Concrete). The two SRC beams (B1 and B2), will serve as control beams for the first phase.

The second phase involved the application of CFRP reinforcing rebars instead of steel reinforcement in new structures. It included testing of three inverted T-beams under monotonic loading. These beams are denoted as CRTRC (Carbon Rebar and Tensar Reinforced Concrete) beams. Two of CRTRC beams (B5 and B6) were internally reinforced with Tensar and CFRP rebars while one beam CRRC (Carbon Rebar Reinforced Concrete), B7, was internally reinforced with only CFRP rebars.

In addition, auxiliary tests were conducted to determine the required mechanical properties of concrete, steel and Tensar reinforcement. This chapter will describe the full details of the testing program, including the design, construction, instrumentation and loading of the test specimens. The results of the tests will be described and discussed in the following chapter.

4.2 TEST BEAMS

4.2.1 RECTANGULAR BEAMS (Phase I)

Four rectangular beams with cross-sectional dimensions of 200 x 400 mm and a length of 4.2 m were tested on a simply supported span of 4.0 m under four-point bending. Different reinforcement configurations for flexure and shear were used in the test program. All the specimens in this phase had an average concrete strength of 35 MPa.

- **SRC Beams B1 and B2:** Figure 4.1 shows the dimensions and reinforcement details of Beams B1 and B2. These beams were reinforced with 2-M15 (16 mm) steel deformed bars with a total cross-sectional area of 400 mm^2 , and M10 (11.3 mm) stirrups at 175 mm c/c distance for shear. A clear cover of 25 mm was provided throughout. The

design yield strength of both bar sizes was 400 MPa. The yield moment of the beams was 58.64 kN-m. The ultimate load when the compressive strain in concrete reached 0.0035 was 103.29 kN giving an ultimate moment capacity of 68.86 kN-m.

- **CLTRC Beam B3:** Figure 4.2 shows the dimensions, and the internal and external reinforcement details of Beam B3. The reinforcement configuration of this beam was as follows:

Flexural reinforcement :

Internal reinforcement: 8 layers of Tensar UX-1700 grids were placed at an effective depth of 365 mm. This reinforcement was designed to act as part of the main flexural reinforcement. The total width of the Tensar grid used was 1330 mm, i.e. 8 layers of 166.25 mm each. The ultimate strain of HDPP was 10.31% with an ultimate tensile strength of approximately 125 kN/m. The load-strain curve of Tensar grid is shown in Figure 4.3.

External reinforcement : External reinforcement was 1-ply of 0.165 mm thick and 200 mm wide CFRP sheets bonded to the bottom of the beam, and extending up to 175 mm from the bottom on each side of the beam, which gave a total cross sectional area of 90.75 mm^2 . The ultimate strength of CFRP sheet was 1470 MPa with a modulus of elasticity of 130 GPa at an ultimate strain of 1.2% (Figure 4.4). This CFRP sheet is available under the trademark name of FORCA TOW sheet.

Shear Reinforcement :

Internal reinforcement : Internal shear reinforcement was not provided so shear stresses were resisted by the external CFRP strips. The objective was to determine the feasibility of shear strengthening of beams.

External reinforcement : The external reinforcement consisted of 100 mm wide and 0.165 mm thick CFRP sheets bonded to the bottom and sides of

the beam in the form of U stirrups (Figure 4.2). This external shear reinforcement was designed to provide 100% of the required shear resistance. The properties of these CFRP stirrups were the same as those of flexural reinforcement. Note that the possible reduction in the strength of the CFRP U sheets at the corners of the stirrups was not considered in the design.

This beam was designed to carry half the load of the SRC beams i.e. approximately 57.31 kN. The nominal ultimate moment capacity of the beam was 38.21 kN-m. It was expected that it would fail in flexure upon rupture of the CFRP sheet in the constant moment region.

- **CLTRC Beam B4:** Figure 4.5 shows the Beam B4 which is similar to Beam B3 except it has 2 plies of CFRP sheet as flexural reinforcement which gives a sectional area of 132 mm^2 . Also it has variable sizes of external CFRP shear strips, 100 mm wide stirrups were up to a distance of 1100 mm from each support and 50 mm wide stirrups thereafter. No shear stirrups were used within a distance of 550 mm on either side of the centerline of the beam. This beam was designed to carry the same load as the Beams B1 and B2, i.e. twice the amount of load of Beam B3.

To avoid the problem of premature failure due to sheet separation at the concrete-epoxy-CFRP sheet interface, CFRP sheets used as flexural reinforcement were bent and epoxied to the end cross-section up to a height of 175 mm in Beam B3 and 100 mm in Beam B4.

To simulate the effect of strengthening, beams were not reinforced with any kind of internal steel reinforcement in flexure or shear. To measure the approximate increase in strength of these beams, the observed ultimate load (at fracture of CFRP sheet) can be subtracted from the ultimate load of plain concrete beam (first crack load). Since no plain concrete beam was tested, the first crack load can be approximated from the load-deflection curves for the beams externally reinforced with CFRP sheets.

4.2.2 INVERTED T-BEAMS (Phase II)

By reference to Figure 4.6, three inverted T-beams were designed and constructed. Each beam had an overall depth of 400 mm, web width of 200 mm, and a 400 mm x 100 mm flange. The overall length of the beams was 4.2 m simply supported over a 4.0 m span. The average compressive strength of concrete used in these beams was 80.2 MPa. The modulus of elasticity, the ultimate strength and the ultimate strain of CFRP reinforcing rebar were 147 GPa, 1813 MPa and 1.23%, respectively as shown in Figure 4.7. This bar was produced by Mitsubishi Chemicals Company under the trademark name of Leadline. The ultimate strain of HDPP - Tensar was 11.4% with an ultimate load of 165 kN/m. Figure 4.8 shows the load-strain curve of the Tensar-UX1700HT used as ductility reinforcement. The ratio of the area of CFRP to HDPP was 98.6 mm^2 :3300 mm wide. The reinforcement provided in all the beams was $0.346 \rho_b$, where ρ_b is the reinforcement ratio needed to produce simultaneous rupture of CFRP rebars and crushing of concrete. To avoid premature shear failure, all the beams in this phase (Phase - II) were over designed in shear, by providing 10 mm closed stirrups at 150 mm spacing. The inverted T-beam profile was chosen to accommodate the HDPP which is currently available in the form of grids. The beams were subjected to four point bending under monotonically increasing static loading.

- **CRTRC Beams B5 and B6:** Figure 4.6 shows the dimensions and reinforcement details of Beams B5 and B6. The flexural reinforcement consists of 2-8 mm diameter CFRP rebars with a total cross-sectional area of 98.6 mm^2 and of 3300 mm wide Tensar UX-1700 HT grid with an approximate cross-sectional area of 1847 mm^2 . The arrangement of Tensar grids in the beams is shown in Plate 4-1. The 3300 mm width was distributed into 3 sets of 9 layers each (118 mm to 123 mm wide). One set was placed in each flange overhang and one set within the steel stirrups in the web. The CFRP rebar was placed between the Tensar layers.

Using first principles and the constitutive properties of the concrete and the reinforcement used, it was determined theoretically that the CFRP will fail when the extreme compressive fibre strain $\epsilon_c = 0.0013$, the total load = 192.8 kN, and the maximum deflection (at midspan) = 51.2 mm. The ultimate load corresponding to $\epsilon_c = 0.003$ was calculated to be 181 kN with the corresponding maximum deflection of over 135 mm. The procedures used to determine those quantities will be discussed later in this thesis.

- **CRRC Beam B7:** Figure 4.9 shows Beam B7 which is similar to the CRTRC Beams B5 and B6, except that it does not have any Tensar reinforcement. The designed ultimate strength of the beam was calculated to be 104.84 kN with the corresponding maximum midspan deflection of 60.8 mm.

4.3 CONSTRUCTION OF THE TEST BEAMS

- **SRC Beams B1 and B2 and CRRC Beam B7:**

Strain gauges were attached to the longitudinal reinforcement at three locations (see Sec. 4.6). Plastic chairs were used to maintain the constant cover which was required on all sides. Since these were regular reinforced concrete beams, no special construction procedure was required.

- **CLTRC Beams B3 and B4:**

Tensar grids were cut to the required size and tied together using steel tie wires. Since these beams were cast upside down, these layers were placed on the concrete by providing sufficient cover.

Prior to the surface preparation of the concrete beams, CFRP sheets were cut into required sizes. The beams were sand blasted, an epoxy primer was evenly applied and was allowed to cure for 24 hours. The primer formed the substrate for the epoxy application. The two part epoxy was mixed per manufactures specifications and applied to the

concrete beams using 75 mm wide sponge rollers. The bonding of the sheets to the CLTRC Beam B4 was modified from the CLTRC Beam B3 which possessed some air pockets beneath the bonded CFRP sheet. Instead of applying epoxy only to the beam, epoxy was also applied as a thin layer to CFRP sheet by laying it on wax paper prior to bonding it to the beam. After a few minutes, the CFRP bonded beam was covered with wax paper and a heavy wooden plank was placed on top of it (Plate - 4.2) for even bonding and prevention of air pockets. This method of bonding produced more desirable results than the manufacturers bonding technique. The sequence of bonding external CFRP sheets is shown in Figure 4.10.

- **CLTRC Beams B5 and B6:**

Three layers of Tensar grids were bundled together and three such bundles were built. Steel stirrups were tied to the CFRP rebars at required spacing to form a cage, then one of the Tensar bundles was inserted through the stirrups. This reinforcement cage was lowered into the form work and tied at various places to avoid any lateral movement. The other two sets of Tensar were placed in the flange overhangs.

For all the beams, concrete was poured in three lifts, using proper vibration of the concrete with electric vibrator and tamping with a steel rod. The beams were finished to a smooth level surface and covered with plastic sheets. After the initial set of the concrete, all the beams were covered with wet burlap and cured for over 28 days. The designed concrete strength was 65 MPa. This concrete was produced by a local concrete plant and it contained a superplasticizer for greater workability. The actual strength from the standard cylinders was found to be 80.2 Mpa.

4.4 LOADING APPARATUS

The loading apparatus consisted of an MTS closed-loop servo-controlled, electro-hydraulic structural testing system with 650 kN capacity in tension and compression and a total stroke length of 152.4 mm. The actuator was attached to an automatic data acquisition system which recorded the load, strain gauge, and LVDT readings.

4.5 EXPERIMENTAL SETUP AND TEST PROCEDURE

The test beams were rolled into the testing frame. The loading points were marked on the beam to induce two point loads located at a distance of 666.66 mm on either side of the beam centerline. A 20 mm steel loading plate was placed on top of the beam at the load points. In order to have complete contact between the plate and the upper face of the beam, a thin layer of hydrostone was applied between them. It was ensured that the loading was applied in a symmetrical manner. Strain gauges and LVDT's were connected to the *HP Data Acquisition System*.

LVDT's were positioned at the respective places and to proof run the instrumentation, load was increased with an increment of 3 kN to a maximum of about 12 kN, which was less than the estimated cracking load, and the beams were unloaded. Thereafter, the beams were loaded at an increment of 5 kN until failure or until the machine ran out of stroke.

4.6 INSTRUMENTATION AND DATA ACQUISITION

Measurement of strains in the reinforcing steel, CFRP rebars, CFRP sheets, Tensar grids and concrete were made using electrical resistance strain gauges, consisting of 5 mm to 30

mm gauge length. The strain gauges were sensitive to 0.01 micro strain with a standard deviation of 2%.

Figure 4.11 shows the overall instrumentation of Beams B1 and B2. Three strain gauges were attached to each of the reinforcing bars located in the tension zone. Strain in the stirrups was also measured at two locations, one on each side halfway between the load point and the support. The midspan concrete strain (ϵ_c) at the top of the beam was measured at the center of the top face and sides of the beam.

Figures 4.12 and 4.13 show the instrumentation of Beams B3 and B4 to measure strains in different materials at various locations of interest. Strain was measured in the CFRP sheets at the mid-span of the beam. The strains in the external CFRP stirrup were recorded to measure the shear force carried by CFRP shear strips. The mid-span concrete strain was measured at the center of the top face and sides of the beam.

For Beams B5, B6 and B7, strain was measured in each CFRP rebar at three locations as shown in Figures 4.14 and 4.15. Strain gauges were also bonded to the Tensar grids to measure the force contributed by the Tensar grids. Also the strain in the concrete at the midspan was measured at the center of the top face and sides of the beam.

The displacements of the beams were measured using 10 LVDT's for each beam. Five LVDT's were placed on each side of the beam centerline at various locations as shown in Figures 4.11 to 4.15.

All the strain gauges were bonded with extreme care and a thin layer of silicon coating was applied to protect them against moisture and other damages during casting. Prior to actual loading of the beams, all the gauges were tested for proper functioning with the digital strain gauge tester.

Both the strain gauges and the LVDT's were controlled automatically. The readings were recorded using the *HP Microcomputer* which displayed the strain and displacements during testing, and which also revealed the workability of the individual strain gauges and LVDT's. The photographs in the next chapter will show some of the constructed beams

SECTIONAL ELEVATION
1:25

SECTION 1
1:5

FIGURE 4.1: REINFORCEMENT DETAILS FOR SRC BEAMS B1 AND B2

SECTIONAL ELEVATION

1:25

NOTE:

NO INTERNAL SHEAR STIRRUPS.
ALL DIMENSIONS ARE IN MILLIMETERS.

FIGURE 4.2: REINFORCEMENT DETAILS FOR CLTRC BEAM B3

Figure 4.3 Load-Strain Curve for Uniaxial Tensile Tests For Tensar UX-1700 (8-Strands Typical Used In The Test Beams)

**Figure 4.4 Stress-Strain Relationship For CFRP Reinforcing Sheet
(Curtosy: Forca Tow Sheet, Autocon Corp.)**

SECTIONAL ELEVATION

1:25

NOTE-1. CFRP EXTERNAL SHEAR STIRRUPS

- 4-100 mm WIDE
- 3-50 mm WIDE &
- 2-25 mm WIDE

FIGURE 4.5: REINFORCEMENT DETAILS FOR CLTRC BEAM B4

FIGURE 4.6: REINFORCEMENT DETAILS FOR
CRTRC BEAMS B5 & B6

Figure 4.7 Stress-Strain Relationship For CFRP Reinforcing Bar
(Curtosy: LEADLINE, Mistubishi Chemical Co.)

**Figure 4.8 Load-Strain Curve for Tensar UX-1700HT
(8-Strands Typical Used In The Test Beams)**

SECTIONAL ELEVATION

1:25

TYPICAL SECTION

1:5

FIGURE 4.9: REINFORCEMENT DETAILS FOR CRRC BEAM B7

SHEET 400x4200 LONG
(FLEXURAL REINFORCEMENT)

STEP-1

BEAM WAS PLACED UPSIDE-DOWN
CFRP SHEET WAS BONDED TO THE
TENSION FACE OF THE BEAM AS
SHOWN IN THIS FIGURE.

STEP-2

CFRP SHEAR STIRRUP
IS BONDED OVER THE CFRP
FLEXURAL EXTERNAL
REINFORCEMENT

THICKNESS EXAGGERATED

BEAM EXTERNALL BONDED
WITH CFRP SHEETS FOR
FLEXURAL AND SHEAR
STRENGTH UPGRADE

TYPICAL INVERTED SECTION

1:5

FIGURE 4.10: BONDING SEQUENCE OF EXTERNAL
CFRP REINFORCEMENT TO CLTRC BEAMS

SECTIONAL ELEVATION

1:25

TYPICAL SECTION

1:5

STRAIN GAUGES & LVDT'S	
SYMBOL	LOCATED ON:
	CONCRETE
	STEEL REBARS
	STEEL STIRRUP
	LVDT'S

FIGURE 4.11: INSTRUMENTATION DETAILS OF SRC BEAMS B1 & B2

SECTIONAL ELEVATION

1:25

ON FLEXURAL
REINFORCEMENT

1:5

ON SHEAR
REINFORCEMENT

1:5

STRAIN GUAGES & LVDT'S	
SYMBOL	LOCATED ON:
	CONCRETE
	TENSAR GRIDS
	CFRP SHEET - EXTERNAL
	CFRP STIRRUP - EXTERNAL
	LVDT'S

FIGURE 4.12: INSTRUMENTATION DETAILS OF
CLTRC BEAM B3

SIZE OF STRAIN GUAGES
AND LVDT'S EXAGGERATED

SECTIONAL ELEVATION

1:25

FIGURE 4.13: INSTRUMENTATION DETAILS OF CLTRC BEAM B4

SECTIONAL ELEVATION

1:25

SIZE OF STRAIN GUAGES
AND LVDT'S EXAGGERATED

TYPICAL SECTION

1:5

STRAIN GUAGES & LVDT'S	
SYMBOL	LOCATED ON:
	CONCRETE
	TENSAR GRIDS
	CFRP REBARS
	STEEL STIRRUP
	LVDT'S

FIGURE 4.14: INSTRUMENTATION DETAILS OF
CRTRC BEAMS B5 & B6

SECTIONAL ELEVATION
1:25

SIZE OF STRAIN GUAGES
AND LVDT'S EXAGGERATED

TYPICAL SECTION
1:5

STRAIN GUAGES & LVDT'S	
SYMBOL	LOCATED ON:
	CONCRETE
	TENSAR GRIDS
	CFRP REBARS
	STEEL STIRRUP
	LVDT'S

**FIGURE 4.15: INSTRUMENTATION DETAILS OF
CRRC BEAM B7**

Plate 4-1 Arrangement of Tensar Grids In CRTRC Beams B5 & B6

Plate 4-2 Construction of CRTRC Beam - B4

Chapter Five

EXPERIMENTAL RESULTS AND DISCUSSION

5.1 *GENERAL*

All the beams were tested under four point bending using quasi-static monotonic loading. Results of the tests on SRC (B1 & B2), CLTRC (B3 & B4), CRTRC (B5 & B6) and CRRC (B7) beams are reported and discussed in this chapter. Strength and stiffness comparisons are made among all the beams of Phase - I (SRC and CLTRC) and Phase - II (CRTRC and CRRC beams). Some of the experimentally measured values of strains in the various materials and the deformations of the beams are shown together with their theoretical values to study the applicability of the current methods of analysis of concrete beams. Results for the achievement of ductility in FRP structures are discussed. This chapter also includes several pictures to aid in visualizing the experimental behaviour of the beams.

Ancillary material testing was carried out to determine the properties of the materials utilized in the test beams. Three standard concrete cylinders were tested from the Phase-I concrete and six from the Phase-II after 28 and 90 days respectively which revealed the average concrete strength of 37.5 and 80.2 MPa, respectively. Three tensile coupon tests of reinforcing steel confirmed the yield strength of 400 MPa. Several specimens were used to determine the tensile strength of Tensar UX-1700 and UX-1700HT using the steel wedge jaws. The load-strain curves of the Tensar UX-1700 and UX-1700HT were shown in the previous chapter. Additional data obtained from the ancillary tests on the various materials are provided in Appendix B. Note that, due to equipment limitations, particularly due to lack of suitable grips, the CFRP sheets and bars could not be tested. Hence, the CFRP manufacturers recommended values were used.

5.2 SRC BEAMS B1 & B2

5.2.1 GENERAL

Specimen in this set were designed as control beams. These under-reinforced beams contained 2-M15 steel bars with an equivalent area of 400 mm^2 in tension and M10 bars at 175 mm c/c in shear. To proof run the instrumentation, the load was increased in increments of 3 kN to a maximum of about 15 kN, which was less than the estimated cracking load. Later these beams were loaded in increments of 5 kN until failure. These beams were also used to verify the computer program which was developed to analyze and design concrete beams with various kinds of reinforcement.

Both the beams behaved nearly the same up to complete failure. The observed ultimate failure load and deflections were within 5% of the calculated values. Even after the spalling of a large amount of concrete in the compression zone, the beams did not collapse completely, they carried the load with large deformations until the end of the stroke in the loading machine. The fracture of the reinforcing steel did not occur.

5.2.2 OBSERVED BEHAVIOUR OF BEAMS B1 & B2

Concrete reached its modulus of rupture at 25.7 kN when a vertical crack appeared in the constant moment region and propagated up by 40 mm. At 30 kN, three more cracks formed in the vicinity of one of the load plates. All these vertical cracks propagated to a maximum distance of 115 mm from the bottom of the beam. When the load reached 35 kN, two more cracks formed in the other half of the beam symmetrical with respect to the previous cracks. The latter cracks also propagated vertically upwards by 55 mm. As the load increased, vertical cracks developed at other locations along the beam. After an average load of 64 kN, the vertical cracks gradually became inclined toward the loading point, typically known as shear cracks, forming at an angle of approximately 45° to the

longitudinal axis of the beam. The reinforcing steel started to yield as branching of the cracks was observed in the compression zone, and as well the formation of new inclined cracks formed in the tension zone along much of the length of the beam.

All cracks can be considered to be medium to large cracks with an approximate crack width of 0.9 to over 3.0 mm in the tensile zone, which became gradually narrower in the compression zone. It was also observed on the data acquisition system that at approximately 70 kN, the concrete strain had not even reached 20% of its capacity whereas steel was approaching the yield strain, but the maximum deflection was less than 8 mm. At a load of 84 kN, the tension steel yielded and the deformations started to increase without much increase in the load capacity. The strain in the concrete started to increase with the increasing load. At the yielding of the reinforcement, there were 10 major cracks present in the beam which propagated into the compression zone up to 60 to 65 mm from top surface of the beam. At 109.7 kN the concrete in the compression zone failed at mid-span, giving rise to the compression failure of the beam(s) after yielding of the flexural reinforcement. Even after failure, the beam did not collapse completely and the reinforcing bars were not ruptured. The deflections were unable to increase because of the end of the stroke in the loading jack. A careful examination of the beam after failure revealed that the concrete in the compression zone was completely crushed. Plates 5.1 and 5.2 show the failed beams which clearly reveal the crushed concrete zone.

5.2.3 DISCUSSION OF THE TEST RESULTS OF BEAMS B1 & B2

Figures 5.1 and 5.2 show the load-midspan deflection of Beams B1 and B2 respectively. As the precracking behaviour of any reinforced concrete beam is linear, the initial segments of the load-deflection curves are straight lines indicating the full elastic behaviour of the beam. As the tensile stress reached the tensile strength of concrete (first cracking load), at a load of 28.5 kN the section cracked, inducing a sudden jump in the reinforcing steel strains as shown in Figure 5.3 and 5.4 for the SRC Beams B1 & B2.

From the slope of the load-deflection curves (Figure 5.1 & 5.2) for Beam B1 & B2, it can be observed that upon cracking, the flexural rigidity of the beam constantly decreased, as indicated by the reduced slope of curve compared to the pre-cracking stage. In Figures 5.5 and 5.6, it can be seen that the increase in concrete strain from the first crack load up to the yield of reinforcement is relatively small. This behaviour is typical of under reinforced concrete beams. Note the strain from the two strain gauges at midspan, one on the top surface and the other on the top of the side face, are not same but the theoretical value is basically the average of the two.

The failure loads of these two beam were within 5% of their theoretical values (provided in Appendix - A). The load-deflection curves for Beam B1 and B2 are shown in Figure 5.1 and 5.2, which show good agreement between the theoretical and the experimental curves. The maximum deflection at the first crack, at reinforcement yielding and at ultimate are 1.9 mm, 11 mm and 96 mm, respectively. Ductility index of the Beams B1 and B2 was calculated to be 10.22 and 10.06 respectively. Figures 5.7 and 5.8 show the strain at mid-height of the stirrup located half-way between the loading point and the support. Since shear behaviour is not of primary concern in these beams, the stirrups behaviour will not be further discussed.

5.3 CLTRC BEAMS B3 & B4

5.3.1 GENERAL

The primary objective of testing these beams was to determine the feasibility of externally reinforcing existing deteriorated reinforced concrete beams with bonded FRP sheets. Also, it was intended to investigate the effect of single ply and two plies of CFRP bonded sheets on strength and stiffness of the beams and on the delamination of the sheet from the concrete. The second objective was to look at the possibility of using FRP sheets for shear strength upgrade. Finally, it was intended to test the feasibility of the proposed hybrid

reinforcement concept for achieving ductility in FRP reinforced beams. Beam B3 was designed to carry approximately half the amount of load carried by Beams B1, B2 and B4 and as noted earlier, Beam B3 had only one layer of Forca tow sheet epoxied to its tensile face while Beam B4 had essentially two plies. Beam B4 was designed to carry approximately the same amount of load as Beams B1 & B2.

The following section presents the test data for Beams B3 and B4. Observations on the hairline and narrow cracks could not be made due to the presence of the externally bonded CFRP sheet in the tension and the shear zones. However, the cracks in the unbonded areas and visible cracks through the CFRP sheet were observed and are reported here.

5.3.2 OBSERVED BEHAVIOUR OF BEAM B3

This beam was loaded monotonically up to the rupture of the CFRP sheet bonded to the tension face. Figure 5.9 illustrates its load-midspan deflection curve which shows the failure load to be 52.8 kN. Some stretching noise was heard at the initial stages of loading. The section first cracked near the centerline with a thumping, cracking noise at a load of 25.1 kN. The first crack was a vertical crack and it propagated up to 135 mm from the bottom. As the load was increased in increments of 5 kN, louder cracking and stretching noises were heard. Right after the first crack, the beam started to deflect noticeably. Slightly after 33 kN some stretching of the CFRP sheets was observed close to the mid span of the beam. Another crack formed, accompanied by a loud noise, in the constant moment region at 350 mm to 400 mm from the centerline of the beam. At 40 kN the beam underwent more than 25 mm deflection; also some of the externally bonded shear stirrups started to show visible shear deformations. Stretching and squeaking noises were observed during the entire duration of the test. Immediately following a load of 43 kN, the shear crack intercepted by the CFRP stirrup started to travel vertically upward along the stirrup. The stretching of the CFRP sheet at the ends (anchor zone) of the beam can be seen in Plate-5.3.

With the next increment of the load one of the external stirrups started to show a vertical crack and split. Some minor cracks were scattered all along the beam in the visible areas. As the load reached 48.5 kN, the beam started to produce louder stretching noises. Some cracks were observed at the under side of the beam in the constant moment region. Plate-5.4 shows the beam at 50 kN. It is evident in this plate that both the longitudinal and transverse CFRP sheets have undergone extensive deformations at the specified load level. Eventually at a load of 52.8 kN, tensile rupture of the CFRP sheet occurred, with a sharp and explosive snap, along with some delamination. Plate-5.5 shows the beam right after CFRP failure.

The Tensar grids in this beam achieved their intended objective right after the failure of the CFRP sheets. As the CFRP sheet ruptured completely, it could no longer hold the beam intact and transferred all the loads to the Tensar grids. The beam did not collapse completely, but experienced large deformations and a reduction in its load carrying capacity. After the failure of the CFRP sheet, as the beam was unloaded, it essentially regained its original shape. Subsequently, the damaged beam was reloaded, and it carried almost 48 kN of load with large deformations. The complete load-deflection curve of the beam will be discussed later. The loading was discontinued at this load because the beam was touching the floor of the laboratory. Plate-5.6 shows the beam after failure, which suffered only local damage in the constant moment region. It is interesting to mention that as the beam was unloaded again, it essentially regained its original shape.

5.3.3 DISCUSSION OF THE TEST RESULTS OF BEAM B3

The experimentally measured load-deflection curve of this beam together with the predicted curve are shown in Figure 5.9. Note that the cracking load is almost the same as those of Beams B1 and B2 which shows the negligible effect of reinforcement on the first cracking load. After the section cracked there was a sudden jump in the strain of the CFRP laminate bonded to the tension flange. The theoretical and observed load versus

mid-span CFRP sheet strain are shown in Figure 5.10. The CFRP sheet bonded to the bottom of the beam reached 84% of its ultimate strength which shows the lack of full composite action between the CFRP sheet and the concrete surface. The sheets bonded to the sides developed 72% of their ultimate capacity.

Figure 5.11 shows the strain at mid-height of the beam in the CFRP shear stirrup located half-way between the load point and the support. Due to a vertical crack in the same stirrup, the strain gauges showed large fluctuations in the readings. The anchoring of the CFRP sheets not only prevented the delamination or peeling off of the sheet but also contributed to the strength of the beam. At the failure of the CFRP sheet, the maximum strain in concrete was less than 0.0015. This indicates that less than half of the strain capacity of the concrete was actually utilized. The theoretical and observed loads versus concrete strain at the top of the mid-span are shown in Figure 5.12.

The load versus deflection curve for the reloaded beam is shown in Figure 5.13. The points marked on the graph indicate the following:

From point	A to B :	Load is carried by uncracked concrete beam.
	at B :	Section cracked (first crack load).
	B to C :	Load is primarily carried by CFRP and some by the Tensar grid.
	At C :	CFRP ruptures.
	C to D :	Load capacity temporarily drops for the neutral axis to adjust.
	D to E :	Beam regains its stiffness and strength. Compare the slope of lines BC and DE.
	E to F, and F to A:	Beam was unloaded. Surprisingly there are no residual deformations upon full unloading. The lack of residual deformations is difficult to fully explain. Of course, the

Tensar will return to its original shape, but still some permanent deflection is expected.

A to G : The beam is reloaded at A until G. At G the stroke of the actuator was exhausted, and the beam was touching the floor of the laboratory. It is quite likely that the beam would have carried even more load.

The reloading curve from A to G clearly shows the role of the HDPP reinforcement and how it allowed the beam to maintain a large part of its moment capacity after the rupture of its main flexural reinforcement. Furthermore, the beam underwent significantly large deformations after the rupture of CFRP and its stiffness increased upon reloading. This increase from the unloading to reloading may be due to the work hardening of the HDPP; it is well known that strength and modulus of HDPP can be increased by means of work hardening. The behaviour of this beam after the rupture of the CFRP demonstrates the utility of the secondary HDPP reinforcement and its role in achieving ductility.

After completing all the observations on the beam, some part of the CFRP sheet was pulled out near the point of rupture and at the vertical crack in the shear stirrup to observe the conditions of cracks behind the sheet (Plate-5.7). It was observed that the bond between the CFRP sheet and the concrete interface was not good as the sheets tended to peel-out easily by hand. Major damage was noticed only in the constant moment region, as may be observed in Plate 5-7.

5.3.4 OBSERVED BEHAVIOUR OF BEAM B4

This beam was subjected to cyclic loading. Its load-deflection curve is shown in Figure 5.14.

The first cycle (25 kN): The beam was subjected to 25 kN and then unloaded. The precracking behaviour of this beam was similar to CLTRC beam B3.

The second cycle (30 kN): The beam was reloaded to 30 kN. Stretching and cracking noises started at 24 kN. At 25.6 kN concrete reached its tensile strength in the mid-span giving rise to narrow vertical cracks on both sides of the beam. These first cracks traveled 140 mm upwards. Upon unloading the beam essentially followed its loading path.

The third cycle (50 kN): In the third cycle the load was increased to 50 kN. In the first 30 kN some new hairline cracks were observed. The first cracks propagated up by 25 mm. At 36 kN, the section cracked with a loud noise. Stretching noises of the Tensar and CFRP sheets were heard. Cracks grew more prominent as the load increased. At 40.3 kN, the stirrup under the load point partially ruptured due to the applied shear force. Flexural cracks started to emerge from the CFRP sheet on the sides of the beam. At 50 kN some spalling of the concrete occurred mid way between the load point and the support and two new large shear cracks appeared at 1.2 m from the support. The beam was then unloaded and 8 mm of permanent deflection was observed at mid-span.

The fourth cycle (90.2 kN): During this cycle the load was increased to 90.22 kN and once again the reloading curve clearly followed the unloading curve. A diagonal shear crack, intercepted by CFRP stirrups, started to travel vertically along the stirrup as shown in Plate-5.8. At 60 kN, the load was held constant to observe the beam more carefully. It was observed that the beam was displaying noticeable deflection (more than 30 mm) but there was no spalling. At 70 kN a shear stirrup ruptured as shown in Plate-5.9 (picture was taken after complete failure of the beam). At 73.6 kN a CFRP stirrup peeled off due to large movements of the shear crack intercepting this stirrup, and at 80.6 kN another CFRP stirrup similarly peeled off. At 81.8 kN a large cracking noise was heard from inside the beam. At this point, peeling off of the CFRP sheet on the sides began. Immediately after 83 kN, the CFRP sheets on the sides used as anchorage began to delaminate over half of the span as shown in Plate-5.10. As the load reached 90 kN, the CFRP reinforcement in the tension zone delaminated and was severely stressed at many points. Also the CFRP shear stirrups were completely ruptured, and all the shear cracks traveled into the compression zone. At 90.2 kN, the CFRP sheet completely ruptured at 1/3 of the span. The failure was a combination of shear failure and tensile failure of the CFRP sheets

as shown in Plates-5.11 through -5.13. Immediately after failure of the sheets, the deflection jumped from 37 mm to 83 mm, and the load dropped.

After the failure of the CFRP sheet, the beam was reloaded until the stroke of the jack was exhausted. Again it was observed that upon removal of the load the beam nearly returned to its original position.

Due to the modification of bonding technique very good bond i.e full composite action between the CFRP sheet and the concrete surface was observed. To investigate the condition of the damaged beam, the CFRP sheet was removed (peeled-off) from the beam. It was observed that most of the major damage had occurred in the constant moment region as in Plate 5-14.

5.3.5 DISCUSSION OF TEST RESULTS OF BEAM B4

Beam B4 was designed to fail at 102.28 kN, while its observed failure load was 90.2 kN, a difference of 12.08 kN. This may be because of the improper placement of the Tensar grids. After the test was completed, the beam was saw-cut at various locations, and it was observed that the Tensar reinforcement was placed 60 mm higher on one side of the beam and the overall effective depth was consequently reduced by 25 mm, thereby reducing the moment capacity of the section. The beam failed in flexure as expected, and the failure was initiated by the rupture of the CFRP reinforcement.

Figure 5.14 shows the complete load-deflection curve of the beam. We notice that the first crack occurred when the load reached 25.6 kN, which was slightly higher than that of Beam B3, at which point the slope of the curve noticeably decreased, indicating a reduction in the flexural rigidity of the beam. From that point on, the beam followed essentially a linear response up to a load of 90.2 kN, which caused the rupture of the CFRP reinforcement. Thereafter, the load dropped to approximately 50 kN, and the beam experienced large deflections (over 75 mm). Subsequently, the beam was unloaded. Figure

2

PM-1 3½"x4" PHOTOGRAPHIC MICROCOPY TARGET
NBS 1010a ANSI/ISO #2 EQUIVALENT

5.14 shows the load-deflection curve for the reloaded beam which reached a maximum load of almost 30 kN with significantly large deflections. It is important to note that the ultimate capacity of the reloaded beam was reached at a maximum deflection of less than 60 mm, after which it maintained its load capacity with increasing deformations but without reduction in strength as expected. The ultimate load capacity of this beam with Tensor alone is less than that of Beam B3 which carried almost 50 kN when it was reloaded after the rupture of CFRP. Theoretically Tensor was expected to carry higher load as shown by the dotted line in Figure 5.14. While the Tensor reinforcement could still possibly carry more load by catenary action, the beam deflections were becoming excessive and the test was stopped. Due to the presence of two-ply of CFRP sheet, this beam possessed significant increase in stiffness compared to Beam B3. The increase in maximum load was 74% compared to Beam B3. This increased load also led to the rupture of the CFRP stirrups which reduced the shear capacity of the beam substantially and it did not allow the HDPP to fully realize its strength.

The load versus carbon sheet strain at midspan of the beam are plotted in Figure 5.15. Notice the dramatic increase in the strain following the first cracking of the beam. CFRP sheet reached 100% of its capacity at the time of rupture which shows full composite action or bonding of CFRP sheet to the beam. Figure 5.16 shows the theoretical and observed load versus midspan concrete strain at extreme compressive fiber. The strain in the concrete at the rupture of the carbon sheet was 0.0018 which shows the full strength of concrete was not utilized. This is true for the lower ultimate strain of the reinforcing material. Figure 5.16(A) shows the load vs. midspan strain of the reloaded beam after the rupture of CFRP sheets with Tensor reinforcement alone. Note (in Figure 5.16(A)) that after the rupture of CFRP sheet there was a sudden drop in the load carrying capacity and an increase in the concrete strain at which point the beam was unloaded.

From Figures 5.15 and 5.16, it can be assumed that without repeated loading, strains in the material would have followed a different path, as indicated by the dotted lines. This difference is due to the residual strain remaining in the material upon unloading. It can also be noticed that *"If the concrete is unloaded prior to reaching its peak stress, the unloading*

response will be approximately linear and upon reloading, the stress-strain response will be approximately follow the unloading lines until the original curve is joined" [Collins and Mitchell (1991)]. Figure 5.17 shows the load versus strain in the carbon shear stirrup located mid-way between the support and the loading point. In the particular shear stirrup, CFRP reached only up to 30% of its ultimate capacity. However, the large deflections and wide cracks caused some of the CFRP stirrups to debond and fail. The failure of those stirrups was in part responsible for the inability of the beam to carry higher load upon reloading.

5.4 COMPARISON OF SRC AND CLTRC BEAMS

First Crack Load: First crack in the beam occurs when concrete reaches its modulus of rupture. The kind of reinforcement and its modulus of elasticity has little effect on the first crack load. However, the amount of reinforcement in tension will affect the cracking moment. There was a marginal difference between the loads which produced the first crack in SRC and CLTRC beams. The first crack between loads increased for two-ply of bonded sheets by almost 1.5% over the single sheet.

Deflection: Beam B3 was designed to carry approximately half the load capacity of Beams B1 and B2. Deflections for Beam B1 & B3 at ultimate are 51 mm and 8 mm, respectively, as shown in Figure 5.18. Beam B1, B2 and B4 were designed to carry approximately similar loads, but the deflection of Beam B4 at rupture of the CFRP sheet was 53 mm compared to 8 mm deflection of B1 at the yield of steel reinforcement as shown in Figure 5.19. As can be observed in Figure 5.20, the addition of the second ply of CFRP sheet significantly increased the stiffness of the beam compared to the single ply beam. The ultimate deflection of the two beams was practically the same despite the fact that the ultimate load of the two ply was almost 75% higher than the ultimate load of the single ply beam. Comparing the response of beam B3 and B4 with B1 in Figure 5.21, we notice that while the ultimate load of B4 is almost 90% of the ultimate load of B1, its stiffness is

significantly less than that of B1. One can therefore conclude that while CFRP sheets can be used to increase the strength of a beam, it may be difficult to utilize that increased strength without further increase in deflections. Also application of CFRP sheets for shear strength upgrade will need careful consideration of crack width and slippage. The present results demonstrate that the widening of cracks, or other crack movements, may cause premature failure of the CFRP stirrups. However, in a beam which already has internal steel stirrups, wide cracks of the kind observed in Beam B4 may not be observed due to the presence of the steel. Nevertheless, further investigation would be needed to determine the influence of shear cracks and their movements on the strengthening effects of CFRP or other sheet stirrups.

Crack Pattern: The formation of cracks for CLTRC beams in the tension face, could not be observed due to the presence of external reinforcement. However, the extension of cracks beyond the external reinforcement was studied. The formation of cracks for SRC beams was very even and gradual. Branching of cracks into inclined cracks was observed in SRC beams whereas very few cracks branched in CLTRC beams. Excessive cracking could be seen in SRC beams. The number of cracks present in the SRC beams at ultimate was higher than in the CLTRC beams.

Mode of Failure: The failure of SRC beams was initiated by yielding of reinforcement, followed by compression failure of the concrete. A clear indication of failure of the SRC beams was given by excessive cracking and deflection. CLTRC Beam B3 failure was due to the tensile failure of the CFRP sheets, followed by local delamination. CLTRC Beam B4 failure was a combination of shear and tensile failure of CFRP sheets.

Plate Strains: In CLTRC Beam B3, the CFRP sheet reached 84% of its ultimate strain and the CLTRC Beam B4 reached 100% of its ultimate strain before failure. This may be due to modifications introduced in the bonding procedures of CFRP to the CLTRC Beam B4.

Tables 5.1 to 5.3 summarize some of the preceding observations and make further comparisons among the steel reinforced and CFRP reinforced beams.

Table 5-1 Comparison of Deflections For SRC and CLTRC Beams

Load (kN)	Deflection (mm)			Remarks
	SRC Beam 1	CLTRC Beam 3	CLTRC Beam 4	
30	2.45	15.933	6.632	
40	4.825	28.551	14.17	
50	7.16	43.67	21.88	
54.62	7.956	50.298	27.53	CLTRC Beam B3 ruptured
60	9.03	-	31.460	
70	11.23	-	37.293	
80	13.10	-	46.502	SRC beams starts to yield
90	28.06	-	52.613	CLTRC Beam B4 just prior to rupture
100	39.00	-	57.125	
101	40.1	-	-	SRC beams reached ultimate.

Table 5-2 CFRP Sheet Strains at Ultimate

	CFRP Sheet Strains		
Rupture Load (kN)	CLTRC Beam B3	CLTRC Beam B4	Remarks
54.62	84%	-	sheet ruptured at tension face
90.35	-	100%	sheet ruptured at tension face

Table 5-3 Comparison of Strength of One and Two Ply of CFRP Sheet

	Ultimate Load (kN)	Increase in Strength (kN)
Plain concrete beam	20 (approximated)	-
CLTRC Beam B3 - One ply of CFRP sheet	54.62	2.73 times over the plain concrete beam
CLTRC Beam B4 - Two ply of CFRP sheet	90.35	4.5 times over the plain concrete beam and almost 75% over one ply

5.5 CRTRC BEAMS (B5 & B6)

5.5.1 GENERAL

A new method of achieving strength and ductility of FRP reinforced beams was proposed and described in Chapter Three of this thesis. Beams B5 & B6 were tested to experimentally verify the proposed method. The results of these beams will be compared with those of Beam B7 to determine the increase in strength, stiffness and ductility. The reinforcement details for these beams were provided in Section 4.2.2. The sectional arrangement of the Tensar grids are shown in Figure 4.6. The test results of the beams reported here include the ultimate moment capacity (strength), ductility, deformations at various load levels, variation of neutral axis depth, mode of failure, and strain variation in concrete and in CFRP rebars. The strain gauges attached to the Tensar grids were not functional during loading of the beams therefore strain in the Tensar could not be measured. Results of two tension stiffening models and three methods of computing deflections are also reported. The experimental data are compared with predicted values, based on the analytical procedures developed and on standard procedures for analysis of concrete beams.

5.5.2 OBSERVED BEHAVIOUR OF CRTRC BEAMS (B5 & B6)

Beams B5 and B6 behaved identically throughout their loading stages up to failure. Their precracking behaviour was similar to other beams described earlier, indicating elastic behaviour. The maximum tensile stress in concrete was reached at an average of 46.5 kN, giving rise to two vertical cracks adjacent to the centerline of the beam. The section cracked with an audible noise and the crack propagated toward the compression zone. After a few seconds, at the same load, cracks appeared at other locations close to the loading point. The new cracks were located symmetrically on the two sides of the loading

line. At a load of 50.22 kN, a few minor cracks formed in the flange. Squeaking and stretching noises started to occur, indicating possible stretching of Tensar inside the beam. At 55.7 kN two more major cracks formed symmetrically with respect to the centerline of the beam. The formation of these cracks was sudden and the cracks propagated to an average height of 255 mm from the bottom of the beam. At 60 kN some of the cracks started to propagate toward the load point. The inclined cracks indicated the formation of flexural-shear cracks. From 65 to 80 kN three new diagonal shear cracks formed, and existing cracks widened, propagating horizontally between the vertical and inclined cracks. At 89.67 kN a loud noise was heard, and it continued for nearly two minutes. However, despite the cracks and the noise, there were no noticeable deflections as can be seen in Plate-5.15. At 110 kN the section cracked right under the load point and the beam sank down by almost 10 mm. From 110 to 140 kN, three more cracks formed and the loud noises continued. Plate-5.16 shows the distribution of cracks under 140 kN of load. From 140 to 190 kN no new cracks formed, but all existing cracks traveled vertically upward or joined other cracks in the vicinity. Plate-5.17 shows Beam B5 at 150 kN and Plate-5.18 shows Beam 6 at 170 kN at which instant it had a maximum deflection of 70.56 mm. Some spalling of concrete cover in the shear zone was also observed. At 191 kN another major shear crack was formed and finally at 195 kN, the CFRP rebars ruptured with a loud noise, as shown in Plates-5.19 through -5.21. The failure occurred in the region between the two load points i.e. in the constant moment region.

To avoid damage to the instrumentation, after the failure of the CFRP, the beam was unloaded, the LVDT's were removed and then the beam was reloaded. The beam continued to carry load until the jack ran out of stroke. The load was measured during the reloading process but the deflections were not. However, at the end of the stroke, the beam midspan deflection was manually measured with reference to its distance from the laboratory floor. The end point of the load-deflection curve in Figure 5.22 is based on that manual measurement. In Figure 5.22 notice that upon reloading the beam reached almost 90% of its moment capacity at the time of the rupture of the CFRP.

5.5.3

DISCUSSION OF TEST RESULTS OF BEAMS B5 & B6

Beams B5 and B6 were designed to fail at 192.79 kN, while their observed failure loads were 195.04 kN and 188.56 kN, a difference of 2.25 kN and 4.23 kN, respectively. The beams failed in flexure as expected, and the failure was initiated by the rupture of the CFRP rebars. Figures 5.22 and 5.23 shows their complete load-midspan deflection curves. We notice that the first crack occurred when the load reached 46.5 kN, at which point the slope of curve noticeably decreased, indicating a reduction in the flexural rigidity of the beam. From that point on the beam followed essentially a linear response up to a load of 195.04 kN for B5, and up to 188.56 kN for B6, which caused the rupture of the CFRP rebars. Thereafter, the load dropped to 147 kN and 170 kN for B5 and B6 respectively, and the beams experienced large deflections, which made it necessary to remove the LVDT's. Hence deflections could not be recorded automatically from that point on, but the load was recorded and the deflections were measured manually at the end of the stroke in the loading jack. At this stage, the beams adjusted their neutral axis and once again picked up load, albeit with a lower rate due to their lower flexural rigidity. They eventually failed due to crushing of concrete at loads of 170 kN and 175 kN with the corresponding maximum deflection being 165 and 162 mm, respectively. While the Tensar reinforcement could still possibly carry more load by catenary action, the beam deflections were becoming excessive and the test was stopped.

Figures 5.22 and 5.23 also show the theoretical values of deflections computed based on moment-curvature relationship. It can be seen that the theoretical values show good agreement with the experimental values. The agreement suggests that CFRP posses good bond characteristics and tension stiffening appears to have little effect.

Figure 5.24 shows the load versus midspan concrete strain at extreme compressive fiber for Beam B5. It can be seen from this figure that upon rupture of CFRP rebars the load dropped instantly without much change in the concrete strain. Upon continuation of load, the concrete strain gradually increased. Figure 5.25 shows the experimental and the theoretical mid-span concrete strain for Beam B6. The theoretical values are very close to

the experimental values. It can also be noted that upon rupture of the CFRP rebars one of the strain gauges was snapped or damaged due to sudden increase in deflection of the beam.

Similarly, Figures 5.26 show the load versus midspan rebar strain. It can be seen in this figure that CFRP rebars reached 100% of its capacity at failure as expected based on the assumption of full and uniform bond and the strain-compatibility analysis using the tension stiffening effect. The deflected shapes of the two beams at various increments of load are shown in Figures 5.27 and 5.28.

The deflected shapes in the later figures have the expected form. We note that the deflection corresponding to the peak or ultimate load is approximately 80 mm in both beams. If we were to assume that the service loads for these beams were 50% of the ultimate loads, i.e. $\frac{195}{2}$ kN and $\frac{175}{2}$ kN for B5 and B6, respectively, then their maximum service load deflections would be approximately 23 mm and 20 mm, respectively. Since

the beams have a spans of 4.0 m each, their $\frac{\text{deflection}}{\text{span}}$ ratio would be $\frac{23}{4000}$ and $\frac{20}{4000}$

which are more than double the deflection of the steel reinforced beams B1 and B2, and are larger than the usual acceptable limits specified in the code. However, it should be mentioned that, although the HDPP reinforcement significantly increased the ultimate strength of the beams with hybrid reinforcement compared to only CFRP reinforcement (Beam B7), it is envisaged that the design strength of the hybrid reinforced beams will be based on the main CFRP reinforcement only and that the HDPP will be ignored in strength calculations. With that provision in mind, the nominal ultimate strength of B5 and B6 would be equal to the strength of B7 which failed at approximately 120 kN. Hence the corresponding service load would be approximately 60 kN. Using Figures 5.22 and 5.23, the deflections corresponding to 60 kN in Beams B5 and B6 are approximately 12 and 8

mm, respectively. Hence, the corresponding $\frac{\text{deflection}}{\text{span}}$ ratios would be $\frac{12}{4000}$ and $\frac{8}{4000}$,

which are 1.44 and 0.96 times $\frac{L}{480}$ where L is the span length. Depending on the type of

construction and the function of the member under consideration, the code allows deflections ranging from $\frac{L}{480}$ to $\frac{L}{180}$. We can see that B6 satisfies the most stringent deflection requirements while B5's deflection is approximately $\frac{L}{333} < \frac{L}{360}$, with $\frac{L}{360}$ being the second deflection limit specified in the code.

With the latter considerations in mind, it can be argued that CFRP reinforced concrete members, especially those with the proposed hybrid reinforcement system, would be able to meet the deflection requirements of the CSA Code (1989 and 1994).

5.6 *CRRC BEAM (B7)*

5.6.1 GENERAL

This beam was reinforced with two 8 mm diameter deformed carbon rebars having an equivalent cross sectional area of 98.6 mm². This amount of reinforcement is near the lower limit of the under reinforced section and is equal to 0.267 ρ_b . The beam dimensions and concrete strength are the same as those of beams B5 and B6. The lack of HDPP reinforcement in this beam would enable us to determine the role of HDPP and its ability to endow beams B5 and B6 with ductility and strength after the rupture of main CFRP flexural reinforcement.

This beam was loaded similarly to beams B5 and B6 and the load was increased in increments of 5 kN. The beam failed within 1.2 times of its predicted failure load, and the failure was brittle and sudden. Observed and predicted load-deflection curves are shown in Figure 5.29.

5.6.2 OBSERVED BEHAVIOUR AND DISCUSSION OF TEST RESULTS OF BEAM B7

The section first cracked with a loud noise at 50.2 kN and close to the centerline of the beam. The formation of this crack was sudden, and it propagated towards the compression zone. It should be noted that this cracking behaviour of this beam was significantly different from those of steel reinforced beams.

It can be seen in Figures 5.30 and 5.31, that upon cracking of the section there was a sudden jump in the strains of concrete and carbon rebar. The maximum deflection immediately after cracking of the section was found to be 5.67 mm, which was significantly higher than the corresponding values for steel reinforced beam. The formation of cracks was symmetrical on both sides of the centerline of the beam. All the cracks were considered to be medium size (1.6 mm to 2.5 mm) vertical cracks which quickly propagated to the compression zone. The crack spacing was very close to the spacing of the steel stirrups inside the beam which was 150 mm c/c.

There was no branching of vertical cracks into horizontal cracks in the tension zone but they propagated into compression zone, approximately 30 to 40 mm from the extreme compression fiber. At 65 kN, it was noted that some of the cracks extended by an average of 27 mm upwards. As the load was increased, cracks became wider and/or new cracks were formed but no branching was observed. The formation of new cracks was observed almost up to the failure of the beam. There were only two cracks which traveled horizontally, one was about 332 mm away from center line and the other under the load point. It was noted that there were no shear cracks present in the beam and there was no spalling of concrete cover.

The beam failed at 125.7 kN which was nearly 1.2 % higher than the predicted failure load. The failure was sudden and brittle. The higher strength may be because of the ultimate strength of CFRP rebars is higher (almost 20%) than the manufacturers suggested guaranteed minimum strength of 1813 MPa. The measured deflections at 100 kN and just

prior to failure at 122.5 kN were 50.218 and 68.76 respectively. Since the beam was under-reinforced, the failure was pure tension failure. From the stress-strain relationship of carbon rebar, it can be seen that there is no yield plateau for FRP reinforcement. Therefore, the increase in load capacity was linear and the failure type was brittle. When the beam failed, it disintegrated into three pieces (Plate-5.22 and 5.23). The deflected shape of beam B7 at various increments of load is shown in Figure 5.32.

5.7 COMPARISON OF CRTRC & CRRC BEAMS

The control beam (Beam B7) failed at 125.7 kN with a maximum deflection of 60 mm. The failure was brittle and was initiated by the rupture of FRP at two locations within the constant moment region. On the other hand, Beams B5 and B6 with hybrid reinforcement failed at 195.04 kN and 188.56 kN (Figure 5.33), respectively, which were 69.34 kN and 62.86 kN higher than the failure load of the control beam. The higher strength was due to the contribution of the HDPP to their load carrying capacity. The hybrid and CFRP reinforced beams reached similar strains in the concrete before the rupture of CFRP rebars as shown in Figure 5.34. The Beams B5 and B6 exhibited slightly higher deformations at failure, but their failure loads were much higher than that of B7. As can be observed in Figure 5.35, the deflection of B5 and B6 at the load equal to the failure load of B7 was approximately 38 mm. Hence the hybrid reinforcement substantially decreased the deflections. The deformability index (as defined in Chapter 3) of the beams was as follows:

$$(\text{Deformability Index})_{\text{hybrid}} = 2.5 \quad (\text{Beam B5})$$

$$(\text{Deformability Index})_{\text{hybrid}} = 2.5 \quad (\text{Beam B6})$$

$$(\text{Deformability Index})_{\text{CFRP}} = 0.0 \quad (\text{Beam B7})$$

The cracks in beam B7 propagated immediately upon their formation towards the compression zone. In Beam B5 and B6, the crack pattern was similar to that in a steel

reinforced beam. However, due to the presence of Tensar grids, the branching and horizontal extension of cracks was observed at increasing loads in the Beam B5 and B6. Plates 5-24 and 5-25 illustrate the of cracks in the CRRC and the CRTRC beams. The load-deflection curves of Beams B5, B6 and B7 (Figure 5.35) show the increase in stiffness and strength of the hybrid reinforced beams compared to the control beam. Figure 5.37 also shows the transformation of the brittle failure mode of Beam B7 to a ductile failure mode of Beams B5 and B6.

The results of this study show that it is possible to achieve a completely ductile failure for FRP reinforced concrete beams under flexure. The key to achieving large deformations before failure without substantial loss in strength is the combination of stiff and strong reinforcement with a relatively flexible but strong reinforcement which is capable of undergoing substantial strain before rupture. The stiff material will be used to achieve strength and deflection control under service loads while the flexible material will impart strength and ductility after the rupture of the primary reinforcement. Further work is needed to determine whether such a system will allow moment redistribution in indeterminate beams and frames.

Fig. 5.1 Load-Deflection Curve of SRC Beam B1

**Fig. 5.2 Load-Deflection Curves SRC Beam B2
Using Moment-Curvature Approach
Including Tension Stiffening Models**

Fig. 5.3 Load v/s Midspan Steel Strain in SRC Beam B1

Fig. 5.4 Load v/s Mid-Span Steel Strain in SRC Beam B2

**Fig. 5.5 Load v/s Midspan Concrete Strains
At The Top of SRC Beam B1**

Fig. 5.6 Load v/s Mid-Span Concrete Strain At Top in SRC Beam B2

**Fig. 5.7 Load v/s Steel Stirrup Strain
of SRC Beam B1**

Fig. 5.8 Load v/s Steel Stirrup Strain SRC beam 2

**Fig. 5.9 Load-Deflection Curve of CLTRC Beam B3
(Upto Rupture of CFRP Rebar - See Also Fig. 5.13)**

Fig. 5.10 Load v/s Carbon Laminate Midspan Strain in CLTRC Beam B3

Fig. 5.11 Strain at Mid-height of Stirrup Located Mid-way between Load Point and Support in Beam B3

Fig. 5.12 Load v/s Maximum Concrete Strain at Midspan of CLTRC Beam B3

Fig. 5.13 Load-Deflection Curve of Reloaded Beam B3

Fig. 5.14 Load-Deflection Curve of CLTRC Beam B4

Fig. 5.15 Load v/s Carbon Sheet Midspan Strain in CLTRC Beam B4

Fig. 5.16 Load v/s Mid-Span Concrete Strain at The Top of CLTRC Beam B4 (See also Fig. 5.16(A))

Fig. 5.16(A) Load v/s Midspan Concrete Strain at Top of Reloaded Beam B4

Fig. 5.17 Load v/s CFRP Shear Stirrup Strain

Fig. 5.18 Experimental Load-Deflection Curves of Beam B1 and Beam B3 (B3 is Designed 50% Capacity of B1)

Fig. 5.19 Experimental Load-Deflection Curves of Beam B1 and Beam B4 (Designed Ultimate Load Capacity of B1=B4)

**Fig. 5.20 Load-Deflection Curves of CLTRC Beams B3 & B4
(Designed load Capacity of B3=0.5xB4)**

Fig. 5.21 Experimental Load-Deflection Curves of SRC (Beam B1) And CLTRC Beams (B3 & B4)

Fig. 5.22 Load-Deflection Curves of Beam B5
(Hybrid Reinforcing Concept)

Fig. 5.23 Load-Deflection Curves of Beam B6
(Hybrid Reinforcing Concept)

Fig. 5.24 Load v/s Midspan Concrete Strains at Top of Beam B5

Figure 5.25 Load v/s Midspan Concrete Strain of Beam B6

Fig. 5.26 Load v/s Midspan CFRP Strain in Beam B6

Figure 5.27 Beam B5 Observed Deformation at Various Loads

P2* - There was no increase in load

Figure 5.28 Beam B6 Observed Deformation at Various Loads

**Fig. 5.29 Load-Deflection Curve of Beam B7
(CFRP Rebars Only)**

Figure 5.30 Load v/s Midspan Carbon Rebar Strain In Beam - B7

Fig. 5.31 Load v/s Midspan Concrete Strain at Top of Beam B7

Figure 5.32 Beam B7 Observed Deformed Shape of Beam At Various Loads

Fig. 5.33 Comparison of Load-Deflection Curves of Hybrid Beams B5 & B6

**Fig. 5.34 Observed Midspan Concrete Strains
at Top of Beam B5 and B6**

Fig. 5.35 Comparison of Load-Deflection Curves of Hybrid Beams B5 & B6 with Control Beam B7

Plate 5-1 SRC Beam B1 After Failure

Plate 5-2 SRC Beam B2 After Failure

Plate 5-3 CLTRC beam B3 Stretching of CFRP Sheet at Sides

Plate 5-4 CLTRC Beam B3 At 50 KN

Plate 5-5 CLTRC Beam B3 After Failure

Plate 5-6 CLTRC Beam B3 After Failure (Only Local Damage)

Plate 5-7 CLTRC Beam B3 Post-Mortem of Beam After Failure

Plate 5-8 CLTRC Beam B4 (Shear Crack Intercepted by CFRP Stirrup)

Plate 5-9 CLTRC Beam B4 (CFRP Stirrup Ruptured Due To Shear Force)

Plate 5-10 CLTRC Beam B4 (Sudden Delamination of Anchored Sheet)

Plate 5-11 CLTRC Beam B4 After Failure

Plate 5-12 CLTRC Beam B4 After Failure

Plate 5-13 CLTRC Beam B4 After Failure

Plate 5-14 CLTRC Beam B4 (Post-Mortem)

Plate 5-15 CRTRC beam B5 At 90 KN

Plate 5-16 CRTRC Beam B5 Distribution of Cracks At 150 KN

Plate 5-17 CRTRC Beam B5 At 150 KN

Plate 5-18 CRTRC Beam B6 At 170 KN

Plate 5-19 CRTRC Beam After Failure

Plate 5-20 CRTRC Beam After Failure

Plate 5-21 Hybrid Beam After Failure

Plate 5-22 Control Beam B7 - At Failure

Plate 5-23 Control Beam B7 - At Failure

Plate 5-24 Crack Distribution of Hybrid Beam

Plate 5-25 Crack Distribution of CFRP Beam B7

Chapter Six

THEORETICAL CORRELATION WITH EXPERIMENTAL RESULTS

6.1 GENERAL

Refined analysis of the non-linear behavior of concrete beams is presented in this chapter. The existing design and analysis procedures in accordance with CSA A23.3-M84 (Canadian Standards Association, 1984) are applied to various CFRP (external - sheets) reinforced, Tensar and CFRP (internal - rebars) reinforced beams. The effect of tension stiffening is considered in determining the moment-curvature response of the beams. No material and resistance factors were applied to any theoretical material strength values. The deflections under service loads are computed using three methods : the Code approach, the modified EI approach and the integrated moment-curvature relationships, including or excluding tension stiffening effects.

6.2 ULTIMATE MOMENT CAPACITY

The ultimate moment capacity of a section is computed by making the following assumptions:

- Plane sections remain plane after bending
- The tensile strength of concrete is : a) included b) neglected
- The stress-strain curve for concrete is known

- Full composite action between the CFRP bonded sheets and the concrete face exists.

To facilitate the computation process, the stress distribution in the compression zone was found by using the equivalent rectangular stress block. Using the standard methods of analysis [Pillai and Kirk (1991)] for the singly reinforced beams the ultimate moment is given by

$$M_u = bd^2 f_c \omega (1 - 0.59\omega) \quad \text{for tension failure} \quad 6.1$$

$$M_u = 0.85 f_c ab(a - 0.5a) \quad \text{for compression failure} \quad 6.2$$

$$\text{where } \rho = \frac{A_r}{bd} \quad \text{and} \quad \omega = \frac{\rho f^*}{f_c}$$

f_c = Concrete Strength

f^* = f_y for yield strength of reinforcing steel

f^* = f_u for the ultimate strength for CFRP rebars or sheets

A_r = area of tensile reinforcement

a = Depth of the compression block

b = Beam (web) width

As usual the depth of the compression block, a , is determined by equating the tensile and compression forces on the section. For the beams internally reinforced with multi-layered Tensor grids and externally reinforced with the CFRP sheets (CLTRC Beams B3 and B4), or the CFRP rebars, an iterative procedure was adopted. The equilibrium of forces and compatibility of strains, as described in the next section, was employed for the latter beams.

6.3 Balanced Reinforcement Ratio

In a beam with balanced reinforcement ratio, the tensile reinforcement would reach its yield strength (ultimate strength in case of FRP rebars) while the concrete would simultaneously reach its maximum compressive strain of 0.003 (or 0.0035 according to CSA A23.3-M94), Figure 6.1. For any given section geometry, the balanced reinforcement ratio is a function of concrete strength, the modulus of elasticity and the yield strength of the reinforcement. Hence, using the strain distribution in a singly reinforced beam at balanced condition as shown in Figure 6.1.

Figure 6.1: Balanced Strain Condition in a Concrete Beam

$$\frac{f_y' / E_s}{0.003} = \frac{d - c_b}{c_b} \quad 6.3$$

Where c_b = neutral axis depth for balanced condition

For steel reinforcement Eq. 6.3 leads to

$$c_b = \frac{0.003 E_s}{0.003 E_s + f_y} d \quad 6.4(a)$$

While for CFRP reinforced beams

$$c_b = \frac{0.003E_{cr}}{0.003E_{cr} + f_u} d \quad 6.4(b)$$

Where E_{cr} = elastic modulus of CFRP

f_u = ultimate strength of CFRP

f_y = yield strength of steel reinforcement

By equating the tensile and compressive forces on the section and using the values of c_b from Eqs. 6.4(a) and (b), one can obtain the balanced reinforcement ratios:

$$\rho_b = \frac{0.85f'_c \beta_1}{f_y} \left[\frac{0.003E_s}{0.003E_s + f_y} \right] \quad \text{for steel reinforcement} \quad 6.5(a)$$

$$\rho_b = \frac{0.85f'_c \beta_1}{f_u} \left[\frac{0.003E_{cr}}{0.003E_{cr} + f_u} \right] \quad \text{for CFRP rebars/sheets} \quad 6.5(b)$$

Assuming concrete strengths of 20 to 100 MPa, $E_{cr} = 144$ GPa, $f_u = 1872$ MPa, $f_y = 400$ MPa, we plot in Figure 6.2 the variation of ρ_b against concrete strength. We observe that ρ_b for carbon is much smaller than the corresponding ρ_b for steel for any concrete strength. This means that it may not be practical to use such small carbon fibre reinforcement ratios. Although in ordinary reinforced concrete, over reinforced sections are considered undesirable due to their brittle failure mode, CFRP reinforced beams will be invariably over-reinforced. In CFRP reinforced concrete over-reinforced sections are more desirable than the under-reinforced sections because rupture of CFRP will lead to the complete disintegration of a beam, while failure of concrete would be considered relatively ductile. To demonstrate this behavior, a typical steel reinforced beam with reinforcement ratios ranging from ρ_{min} to $1.25 \rho_{max}$ is shown in Figure 6.2(A). This figure clearly shows the reduction in beam ductility as manifested by the reduced steel strain at ultimate - when the reinforcement ratio is increased. Notice that for a reinforcement ratio $\rho = \rho_b$, the steel strain at ultimate is less than 2%. Hence, despite the large strain capacity of steel, for realistic reinforcement ratios, the actual strain in

steel reinforcement rarely exceeds 3%. Of course, if a hybrid reinforcement system is used, then the ρ_b would involve both reinforcement types and the carbon fibre ratio could be smaller than ρ_b .

Fig. 6.2 Variation of Reinforcement Ratios with Concrete Strength

Figure 6.2(A) General Behaviour of A Singly Reinforced Beam With Various Steel Ratio's

6.4 MOMENT-CURVATURE RELATIONS

6.4.1 MOMENT-CURVATURE DETERMINATION

For an element dx with equal end moments and axial forces as shown in Figure 6.3, the radius of curvature R , the neutral axis depth c , the concrete strain in the extreme compression fiber ϵ_c , and the strain in the tension steel ϵ_s , are illustrated below. The rotation of the element dx is given by

Figure 6.3: Deformed Element of A Flexural Member With Strain Distribution

$$\frac{dx}{R} = \frac{\epsilon_c dx}{c} = \frac{\epsilon_s dx}{d - c} \quad 6.6$$

$$\therefore \frac{1}{R} = \frac{\epsilon_c}{c} = \frac{\epsilon_s}{d - c} \quad 6.7$$

The curvature $1/R$ is the rotation per unit length of the member and is denoted by ϕ in Figure 6.3.

The theoretical determination of curvature associated with the range of moments ϵ the test beams will be based on the same assumptions as those stated in Sec 6.2.1., using the strain compatibility and equilibrium conditions in the section.

The stress-strain relations for the materials used in the test beams are presented in Chapter Four, Experimental Program. To determine the complete moment curvature response, the top concrete strain ϵ_c , is increased in predetermined increments until the ultimate concrete strain, ϵ_{cu} , or the CFRP sheet failure strain ϵ_{cr}^* , or the Tensar grid ultimate strain ϵ_{tg}^* is reached. By reference to Figure 6.4 the basic steps are as follows:

- **Step - 1** Increment the top concrete strain (ϵ_c)
say from 0.0001 to 0.004 in increments of 0.0001
- **Step - 2** Draw the strain profile and let c be the depth of the neutral axis. Assume a reasonable value for the depth c .

Fig. 6.4 Stress-Strain Relationship of a Section

- **Step-3** Determine the reinforcement strain in terms of the top concrete strain and the neutral axis c . Hence

$$\epsilon_{ig}^* = \frac{\epsilon_c (d_1 - c)}{c} \quad 6.8$$

$$\epsilon_{cr}^* = \frac{\epsilon_c (d - c)}{c} \quad 6.9$$

Where ϵ_{ig} and ϵ_{cr} are the strain in the Tensar and carbon fibre reinforcement, and d_1 and d are their respective positions measured from the extreme compression fibre.

- **Step - 4** Using the stress-strain relation of the materials involved, determine the stresses corresponding to the concrete strain ϵ_c and the reinforcement strains ϵ_{ig} and ϵ_{cr} .

- **Step - 5** Find the net compressive force C . using Figure 6.4

Integrate the concrete stresses over the compression zone of the section to obtain the compressive force, C .

$$C = b \int_0^c f_c dy \quad 6.10$$

Where y is the distance of a compression fibre from the neutral axis depth, f_c is the Concrete stress and b is the beam width. Since

$$y = \frac{c}{\epsilon_c} \epsilon \quad 6.11$$

or

$$dy = \frac{c}{\epsilon_c} d\epsilon \quad 6.12$$

then

$$C = \frac{bc}{\epsilon_c} \int_0^{\epsilon_c} f_c d\epsilon \quad 6.13$$

For the ascending portion of the stress-strain curve of concrete (Figure 6.5)

$$f_c = f_c'' \left[\frac{2\epsilon}{\epsilon_o} - \left(\frac{\epsilon}{\epsilon_o} \right)^2 \right] \quad 6.14$$

Where f_c'' is the compressive cylinder strength of concrete and ϵ_o is the strain corresponding to f_c'' .

Substitute Eq. 6.14 in 6.13 and carry-out integration

$$C = \frac{bcf_c''}{\epsilon_c} \int_0^{\epsilon_c} \left[\frac{2\epsilon}{\epsilon_o} - \left(\frac{\epsilon}{\epsilon_o} \right)^2 \right] d\epsilon \quad 6.15$$

$$= bcf_c'' \left[\frac{\epsilon_c}{\epsilon_o} - \frac{1}{3} \left(\frac{\epsilon_c}{\epsilon_o} \right)^2 \right] \quad 6.16$$

For the descending portion of the curve

$$f_c = f_c'' [1 - 100(\epsilon - \epsilon_o)] \quad 6.17$$

Hence when $\epsilon_c > \epsilon_o$, we split the integral in Eq. 6.11 into two parts, from 0 (zero) to ϵ_o and from ϵ_o to ϵ . In the second part we substitute for f_c from Eq. 6.17.

$$C_2 = \frac{bcf_c''}{\epsilon_c} \int_{\epsilon_o}^{\epsilon_c} [1 - 100(\epsilon - \epsilon_o)] d\epsilon \quad 6.18$$

$$= \frac{bcf_c''}{\epsilon_c} \left[\epsilon - 100 \left(\frac{\epsilon^2}{2} - \epsilon_o \epsilon \right) \right]_{\epsilon_o}^{\epsilon_c} \quad 6.19$$

While

$$C_1 = \frac{bcf_c''}{\epsilon_c} \left[\frac{\epsilon_o^2}{\epsilon_o} - \frac{\epsilon_o^3}{3\epsilon_o^2} \right]$$

$$= \frac{bcf_c''}{\epsilon_c} \left[\epsilon_o - \frac{1}{3}\epsilon_o \right] \quad 6.20$$

$$= \frac{2}{3} bcf_c'' \frac{\epsilon_o}{\epsilon_c} \quad 6.21$$

$$C = C_1 + C_2 \quad 6.22$$

- **Step - 6** Find the total tensile force , T, including tension stiffening

Stresses corresponding to the reinforcement strains can be found from the stress-strain relationship of CFRP sheet and Tensar grids.

$$T = T_{tg} + T_{cr} + T_c \quad 6.23$$

Where T = Total tensile force

$$T_{tg} = \text{Tensile force due to Tensar grids} = E_{tg} \times A_{tg} \times \epsilon_{tg}$$

or T_{tg} can also be computed by using the total width of Tensar provided in mm and its corresponding load vs. strain curve if the exact area of Tensar is not known.

$$T_{cs} = \text{Tensile force due to CFRP sheet} = E_{cr} \times A_{cr} \times \epsilon_{cr}$$

$$T_c = \text{Tension stiffening (described later in a separate section)}$$

- **Step - 7** Use the condition of equilibrium of forces, to check if $C \neq T$, revise estimate of c and repeat steps 1 to 7, until $C = T$.

Step - 8 Knowing c , the forces and their locations, we take the moment about the neutral axis to determine the moment M corresponding to the assumed strain profile with the corresponding curvature being $\frac{\epsilon_c}{c}$.

For the inverted T-beams the same procedure is applied, except in the pre-cracking stage one must account for the gross area of the flange.

Using the procedures described herein, a computer program was developed to perform the necessary calculations of the moment-curvature response. The program is called Non-Linear Analysis and Design [NLAD] and some details about it are given in Appendix C. The moment-curvature response of the tested beams was calculated using the program NLAD, and are plotted in Figures 6.6 to 6.10. Note that the material properties and the dimensions of all the beams tested are described in Chapter Four.

6.4.2 Variation of Neutral Axis Depth

Due to the high strength and low modulus of FRP reinforcement, the variation of neutral axis depth is very small (Beams B5 & B6 and Beam B7) compared to that of conventional steel reinforced concrete beams (Beams B1 & B2). The depth of compression block depends on the depth of neutral axis at any given load or strain. The variation of neutral axis depth for SRC beams is non-linear, varying from 103 mm at very small loads to 36 mm at ultimate concrete strain of 0.0035 as shown in Figure 6.11. The variation of neutral axis depth for Beams B3, B5 & B6 are shown in Figures 6.12 to 6.14 respectively. Notice rather abrupt change in the neutral axis depth of B3 after the rupture of the CFRP reinforcement. Beams B5 and B6 on the other hand show a less dramatic change in the position of the neutral axis after the rupture of the CFRP as shown in Figure 6.13. Figure 6.14 shows more clearly how the neutral axis position changes with increased loading (point A to B) and the increase and subsequent decrease in the depth of the compression zone after rupture of the CFRP (point C).

6.4.3 Variation of Curvature Along Beam Span

The distribution of curvature along the span for the steel reinforced beam is shown in Figure 6.15. From Figure 6.15 it can be seen that there is increase in rotations after the yielding of the tension steel, which is a very important factor contributing to the ductility of the section. The variation in curvature along the length of Beam B4 is shown in Figure 6.16. The variation of curvature for CRTRC Beams B5 and B6 are shown in Figure 6.17. Comparison of Figure 6.15, 6.16 and 6.17 reveals that the curvature variation along the span is similar in the three beams. Also one can see that for the 30 MPa concrete in Figure 6.17, the curvature diagram is basically the same as that of the steel reinforced beam (Figure 6.15). Hence the ductility of the two beams may be expected to be the same.

6.5 DEFLECTIONS

6.5.1 Code Approach

Due to the emergence of high strength low modulus materials, reinforced concrete members may not satisfy all the current serviceability limit state (SLS) requirements for reinforced concrete structures even though adequate strength may be provided. Prediction of deflection of reinforced concrete structures in general is complicated due to the effect of cracking moment of inertia (I) of the member, variation of modulus of elasticity (E) with stress and the time-dependent non-linear response of the material. Building codes of have dealt with deflection control by specifying minimum depth-to-span ratios based on practical experience [ACI Committee 435 (1974)]. Various methods have been developed to accurately predict the deflection of concrete structures. The difference among most of the methods is the manner in which the flexural rigidity EI is calculated. Experience has shown that the computed deflections are within $\pm 20\%$ of the observed values for SLS which may be considered reasonable for design purposes. The question arises whether

existing methods of deflection calculations can be applied to FRP reinforced concrete structures. In this section we will present those methods, and the more detailed method based on first principles. The results of the calculations will be compared with the experimental data for the tested beams.

6.5.1.1 Pre-Cracking Stage

During the pre-cracking stage the maximum tensile stress in a concrete beam is less than the tensile strength or modulus of rupture of concrete. Due to the linear elastic part of the stress-strain curve of concrete, the pre-cracking stage of the load-deflection curves in all the specimens tested were essentially straight lines. In calculating the deflections of the beams in the pre-cracking stage, elastic theory and the transformed section method was used. The transformed section approach in the pre-cracking and post-cracking states was considered because tension and/or compression reinforcement can increase the moment of inertia of the section by as much as 30%, particularly in the post-cracking stage. For low modulus FRP reinforcement such as GFRP, in computing the deflections for the loads below the cracking load, gross concrete section can be utilized since it makes little difference whether or not the reinforcement is transformed.

Since deflections depend on the value of the elastic modulus, in the pre-cracked stage the elastic modulus of concrete, E_c , was calculated (CSA/CAN3-A23.3-M84) as follows:

$$E_c = 5000 \sqrt{f'_c} \quad 6.23$$

Where

f'_c = compressive strength of concrete in MPa

When the tensile stress of concrete exceeds the modulus of rupture, f_r , of concrete, first flexural cracks will form. The modulus of rupture of concrete according to CSA/CAN3-A23.3-M84 is given by

$$f_r = 0.6\lambda\sqrt{f'_c}$$

6.24

Where

$\lambda = 1$, for normal density concrete

6.5.1.2 Post-cracking Stage

When the tensile stress of concrete exceeds the modulus of rupture of concrete, cracks will form at discrete intervals along the beam. Upon cracking of the section, the flexural rigidity of the beam will begin to diminish and the slope of the load-deflection curve will become less steep. The moment of inertia of a cracked section, I_{cr} , can be obtained by taking moment of inertia of the cracked section about the neutral axis, c , and by neglecting the concrete section below the neutral axis:

$$I_{cr} = \frac{bc^3}{3} + [n-1]A_r[c-d']^2 + nA_r[d-c]^2 \quad 6.25$$

where

$$n = \text{modular ratio} = \frac{E_r}{E_c}$$

A_r = area of compression reinforcement

A_r = area of tension reinforcement

b = beam width

$d(d')$ = distance of the centroid of the tension (compression) reinforcement from the extreme compressive fibre

Note that FRP is rarely used as compressive reinforcement due to its relatively small compressive strength. Hence, we shall ignore the presence of FRP reinforcement in the

compression zone. Of-course, in the present testing program no compressive FRP was used.

Use of a so-called fully cracked section and its corresponding moment of inertia, I_{cr} , as given by Eq. 6.25 overestimates beam deflections. The reason is that concrete between two cracked sections still carries tension and this tends to decrease the curvature compared to the values based on fully cracked section. This phenomenon of tension transfer in cracked region is referred to as tension stiffening. Hence, accurate prediction of deflections requires considerations of tension stiffening. Accordingly, the CSA and ACI codes recommend the following expression for the moment of inertia of cracked reinforced concrete beams

$$I_e = \left(\frac{M_{cr}}{M_a} \right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a} \right)^3 \right] I_{cr} \quad 6.26$$

where

I_g = Moment of inertia of the gross un-cracked section.

I_{cr} = Moment of inertia of cracked section transformed to concrete (Eq. 6.25).

M_a = Resisting moment at the section of interest due to the load for which deflections are calculated.

M_{cr} = Cracking moment = $\frac{f_r I_g}{Y_t}$

Y_t = distance from the neutral axis to the extreme tensile fibre in the un-cracked section.

Note that Eq. 6.26 is empirical and was derived from the test data of ordinary reinforced concrete beams. Its applicability to FRP reinforced concrete is questionable.

6.5.2 Modified Moment of Inertia for FRP Reinforced Beams

It has been observed in several experimental investigations, including the one undertaken in the present study that:

- The range for the variation of the height of neutral axis is very small for FRP reinforced beams (CRTRC Beams B5 & B6 and CRRC Beam B7) compared to conventional steel reinforced concrete beams (SRC Beams B1 & B2) and
- The neutral axis shifts upward rather dramatically, following the first flexural cracks.

Accordingly, fully cracked sections devoid of tension stiffening occur at relatively small loads within the serviceability limit state. Hence, for beams subjected to four point bending, Faza and Ganga Rao proposed the moment of inertia variation as indicated in Figure 6.18(a). We notice that the region subjected to the constant moment is assumed to be fully cracked with its $I_c = I_{cr}$. Figure 6.18(b) shows the moment diagram while Figure 6.18(c) illustrates the corresponding curvature diagram of the beam.

Fig 6.18: a) Loaded Beam; b) Moment Diagram; c) M/EI Diagram

Using the moment-curvature diagram in Figure 6.18(c), they calculated the maximum deflection of simply supported beams under four point bending as follows:

The maximum deflection of the beam can be found by using the second moment area theorem, which gives reference to Figure 6.18(c).

Taking moments about the support gives,

$$\delta_{\max} = \left[\frac{Pa^2}{2EI_c} \left(\frac{2a}{3} \right) \right] + \left[\frac{Pa^2}{2EI_{cr}} \left(\frac{5a}{4} \right) \right] \quad 6.27$$

Simplifying Eq. (6.27)

$$\delta_{\max} = \frac{(8Pa^3I_{cr} + 15Pa^3I_c)}{(24EI_{cr}I_c)} \quad 6.28$$

Substituting $a = L/3$ for four-point bending gives

$$\delta_{\max} = \frac{8pL^3I_{cr} + 15pL^3I_c}{648EI_{cr}I_c} \quad 6.29$$

rewriting Eq. (6.29)

$$\delta_{\max} = \frac{23PL^3}{648E_c I_m} \quad 6.29A$$

Where

$$I_m = \frac{23I_{cr}I_c}{8I_{cr} + 15I_c} \quad 6.30$$

6.5.3 Computation of Deflections from Moment-Curvature Relation

From first principles the moment-curvature relationships can be used to find the curvatures along a beam corresponding to a given distribution of moments, then deflections can be found from these curvatures [Collins and Mitchell (1991)].

Computation of rotations and deflections of a beam comprises basically two steps:

- Calculation of moment-curvature relationship as described earlier
- Integration of curvatures along the length of the beam to compute beam rotations and deflections

To implement the above procedure, we begin by writing the governing equation of bending of a beam

$$\frac{d^2 w}{dx^2} = \frac{M}{EI} = \psi \quad 6.31$$

Where w is the deflection of the beam, x is the axial coordinate and ψ is the curvature.

Integration of Eq. (6.31) will lead to

$$\theta = \frac{dw}{dx} = \int \frac{M}{EI} dx \quad 6.32$$

or

$$\theta = \int \psi dx \quad 6.33$$

Where θ is the beam rotation. Integration of Eq. (6.32) another time will lead to [Popove, (1990)]

$$w = \int \psi x dx \quad 6.34$$

Of course Eqs. (6.33) and (6.34) are the mathematical expressions of the first and second moment area theorems. They can be used to calculate the beam rotations and deflections, respectively, at any point on the beam. If written as definite integral,

$$w = \int_a^b \psi x dx$$

6.35

then w denotes the distance of a point on the so-called elastic curve from the tangent at point b on the same curve, Figure 6.20(d). For the beams tested in the current investigation, due to symmetry, the maximum deflection can be determined by letting point a be at the support and point b at mid-span.

In the case of cracked concrete sections, ψ is not a simple analytical function, hence the integration in Eq. (6.35) needs to be performed using numerical methods.

Fig. 6.19: Typical Moment-Curvature Relationship

By referring to Figure 6.20(b), we divide the beam into a number of segments of length $\Delta X_j = \Delta X$ ($j=1, n$), and determine the average moment $M_j = \frac{M_{j+1} + M_j}{2}$ at the middle of each segment. Similarly, the distance X_j of point a from the centroid of each segment j can be determined.

Next we go to the moment curvature diagram of section j to determine the curvature ψ_j corresponding to the moment M_j (see Figure 6.19). In this fashion the curvature diagram of the beam is plotted as shown in Figure 6.20(c).

Now Eq. (6.35) can be written approximately as

$$w = \sum_{j=1}^n (\psi_j \Delta X_j) X_j \quad 6.36$$

Where n is the total number of segments from point a to b. Observe that $(\psi_j \Delta X_j)$ is the area of the segment in the curvature diagram while X_j is its moment arm with respect to the point a. To determine the complete deflected shape of the beam one must first calculate the rotation of the beam at the support using the approximate form of Eq. (6.36)

$$\theta = \sum_{j=1}^n \psi_j \Delta X_j \quad 6.37$$

Next, as indicated in Figure 6.20

$$\delta = \theta X_j - r_j \quad 6.38$$

Where r_j is the distance of point j on the elastic curve from the tangent at point a.

Hence

$$\delta_j = \theta X_j - \sum (\psi_j \Delta X_j) X_j \quad 6.39$$

Where the summation runs from one to j . Notice that X_j in this case is the distance of the centroid of a segment from the point where deflection is being calculated.

a) Beam with Two Point Loads; b) Moment Diagram; c) Curvatures; d) Deflected Shape

Fig. 6.20: Calculating Deflections From Curvatures

6.6 TENSION STIFFENING

The primary cracks in concrete form when the tensile stress in concrete exceeds the cracking stress. At cracked sections all tension is carried by the reinforcement; however, there will be some tensile stress present in concrete pieces lying between the cracks. Some of the tension carried by the reinforcement is transferred to the concrete through the bond stresses between the cracks. These pieces of concrete decrease the stretch of reinforcement to some extent; this effect is known as tension stiffening. Additional cracks will form at higher moments until the reinforcement can no longer transfer the tensile stresses to the concrete due to loss of bond between the reinforcement and the concrete. Assuming fully cracked section between adjacent cracks overestimates beam deflections. Hence, it is important to consider this phenomenon when calculating beam deflections under service loads.

6.6.1 Factors Affecting Tension Stiffening

Tension-stiffening effect of reinforced concrete is dependent on:

- Reinforcement ratio
- Diameter of reinforcement
- Distribution of reinforcement
- Bond characteristics of reinforcement
- Type of loading (sustained or repeated)
- Concrete Strength

It has been reported that the post-cracking behavior of high strength concrete is different from that of the low strength concrete. The difference between the two materials is not

only related to the material properties such as f_c' , f_t , and E [Marzouk and Chen (1993), Prakhya and Morley (1990), Gilbert et al (1978), Carreiar and Domingo et al (1986)] but also on the bond strength, μ , tension stiffening (tensile stress) and on the fracture energy of the material.

Prior to formation of the first crack in the beam the average stress in concrete, f_c , is less than the cracking stress, f_{cr} , which implies $\epsilon_t < \epsilon_{cr}$. Therefore no tension stiffening effect arises. When the first crack forms in the beam, the average stress in concrete is equal to the cracking stress ($f_c = f_{cr}$), which implies $\epsilon_t = \epsilon_{cr}$.

In the post-cracking state the average stress in concrete starts to decrease with the increase in the number of cracks in the beams and the stress reduction follows the relationship shown in Figure 6.21. The average stress in concrete can be obtained from the following equation [Collins and Mitchell (1991)]:

$$f_c = \frac{\alpha_B \alpha_L f_{cr}}{(1 + \sqrt{500 \epsilon_t})}$$

Where α_B = factor accounting for bond characteristics of reinforcement

$\alpha_B = 1$ for deformed reinforcing bars

$\alpha_B = 0.7$ for plain bars, wires, or bonded strands

$\alpha_B = 0$ for unbonded reinforcement

α_L = factor accounting for sustained or repeated loading

$\alpha_L = 1.0$ for short-duration monotonic loading

$\alpha_L = 0.7$ for sustained or repeated loads

ϵ_t = tensile strain in the concrete at the location of the reinforcement,
obtained from strain compatibility relation.

Figure 6.21: Average Stress v/s Average Strain Relationship for Concrete in Tension [Adopted Collins and Mitchell (1992)]

One of the primary variables which affect tension stiffening is the effective area of concrete in tension (or effective embedment zone) which depends on the number of reinforcing bars present in the tension face of a section, and on the side and bottom cover of the section. Determination of the effective area of concrete in tension adds to the complexity of the problem. In this study two models were selected based on the effective area of concrete in tension zone.

6.6.2 Tension Stiffening Model - 1 (TSM - 1)

This model is based on the method suggested by Collins and Mitchell (1992). Figure 6.22 shows the forces exerted by concrete in tension and compression. The variables include the effective depth of concrete in tension, d_c , which depends on the diameter of the reinforcing bar. It has been suggested that d_c is equal to 7.5 times the diameter of reinforcing bars, d_b . The concrete tensile stress can be accounted for in computation as follows:

- Actual parabolic distribution with reduced stress corresponding to increase in concrete strain as shown in Figure 6.22(c).
- Approximate uniform concrete stress between the cracks equal to half the concrete tensile strength acting over the effective embedded zone in tension as shown in Figure 6.22 (d).

The computer program NLAD developed in the present thesis can compute the concrete stress in tension according to both methods. It was observed in this study that there is not much difference between the two methods as the contribution of tensile stress is relatively small.

a) Cross-section with Effective Area of Concrete in Tension; b) Concrete Strains; c) Actual Concrete Stresses; d) Equivalent Uniform Stresses; e) Concrete Forces

Figure 6.22: Tension Stiffening Model - 1

6.6.3 Tension Stiffening Model - 2 (TSM - 2)

This model is similar to method described in TSM - 1. In this case the variable is the effective area of concrete in tension. The effective area of concrete is taken from Park and Pauley (1975). The effective area, $d_c = 2d_e$, where d_e is the distance from the extreme tensile fibre to the centroid of the main flexural reinforcement, is shown in Figure 6.23.

a) Cross-section with Effective Area of Concrete in Tension; b) Concrete Strains; c) Actual Concrete Stresses; d) Equivalent Uniform Stresses e) Location of Forces

Figure 6.23: Tension Stiffening Model - 2

6.6.4 Comparison of Calculated and Measured Deflections

Using the procedures discussed in the preceding sections, the load-midspan deflection of a series of beams were calculated and compared with the corresponding experimental data. The analyzed beams included the beams tested in the current investigation and some other beams tested by others, which had reinforcement and dimensions different from the ones tested here.

The load-mid-span deflection curves of the beams tested in the current investigation are shown in Figures 5.1, 5.2, 5.9, 5.13, 5.14, 5.22, 5.23, and 5.31. These figures show the experimental and calculated values, according to the moment-curvature models described earlier. The calculated curves, as indicated, may include or preclude the tension stiffening

effect. We notice that the inclusion or preclusion of the tension stiffening effect generally does not make a significant difference, and it would appear, as in Figures 5.22 and 5.23 that they constitute the upper and lower bounds of the measured values.

The comparison for the ordinary reinforced concrete beams (B1 and B2) in Figures 5.1 and 5.2 is close throughout the loading stages. Similarly, for the FRP reinforced beams the comparison up to the rupture of the FRP is good, but with the hybrid reinforced beams (Figure 5.9, 5.13, 5.14, 5.22 and 5.23) it is difficult to make any comparison beyond the rupture point of the CFRP because after the rupture of CFRP the beam deflection increased dramatically.

The load-deflection curves of Beams B5 and B7 computed by all three methods are shown along with the experimental values in Figures 6.24 and 6.25. It can be seen that the modified MI method highly over-estimates the beam deflections. This is due to the fact that this method under-estimates the effective moment of inertia or the contribution of concrete below the neutral axis of the beam. The rigidity of the beam depends upon the extent of cracking and the depth of neutral axis. Figure 6.24 also shows the deflections computed by the code's method which under-estimate the deflections because it over-estimate the moment of inertia of the beam after the first crack. For hybrid reinforced beams the code method estimate of moment of inertia was lower to higher as shown in Figure 6.25. The code's method developed for beams reinforced by steel is basically empirical. For FRP reinforced beams, it is proposed to compute deflections of the beam from the actual rotations and curvatures which give very close values with the experimental values as shown in Figures 5.1, 5.2, 5.9, 5.13, 5.14, 5.22, 5.23, 5.31, 6.24 and 6.25.

Figure 6.5 Assumed Concrete Stress-Strain Curve

Fig. 6.6 Moment-Curvature Relationship of SRC Beams B1 & B2

Fig. 6.7 Moment-Curvature Relationship of CLTRC Beam B3

Fig. 6.8 Moment-Curvature Relationship of CLTRC Beam B4

Fig. 6.9 Moment Curvature Relationship of Hybrid Reinforced Beams (B5 & B6)

Fig. 6.10 Moment-Curvature Relationship of CRRC Beam B7

3 of/de 3

PM-1 3½"x4" PHOTOGRAPHIC MICROCOPY TARGET
NBS 1010a ANSI/ISO #2 EQUIVALENT

PRECISIONSM RESOLUTION TARGETS

Fig. 6.11 Variation of Neutral Axis Depth SRC Beams B1 & B2

Fig. 6.12 Variation of N.A. Depth With Increase in Midspan Concrete Strain at Top of Beam B3

Fig. 6.13 Variation of Concrete Strain with Neutral Axis Depth - Beams B5 & B6

Fig. 6.14 Plot Showing Variation of N. A. Depth With Increase In Load for Beams B5 & B6

Fig. 6.15 Variation of Curvature Along The Span of SRC Beams B1 & B2

Fig. 6.16 Variation of Curvature Along The Span of Beam B4

Fig. 6.17 Variation of Curvature Along The Span of Beam Using Different Strengths of Concrete

Fig. 6.24 Load-Deflection Curves of Beam B7 By Using Three Methods

Figure 6.25 Load-Deflection Curves of Beam B5 By Using Three Methods

Chapter Seven

SUMMARY, CONCLUSION AND RECOMMENDATIONS

7.1 SUMMARY

Corrosion is the severe problem for exposed concrete structures because it affects both the serviceability and strength of structures. Various solutions to the problem of corrosion have been implemented but none has been so far entirely satisfactory. Due to their light weight, good corrosion resistance and high strength FRP reinforcement can be considered as an alternative solution to the several active and passive corrosion protection methods.

In this study application of FRP materials in the form of sheets for strengthening of existing structures and reinforcing bars for the design of new structures was investigated. Seven 4.2 m long concrete beams were tested. Two beams were reinforced with mild steel reinforcement, two beams were internally reinforced with high density polypropylene (HDPP) and externally with CFRP two sheet, one beam was reinforced with CFRP rebars and finally two beams were reinforced with CFRP rebars and HDPP grids. The shear reinforcement in all beams, except two, were conventional steel stirrups while in two beams with CFRP tow sheets, the stirrups were also CFRP and adhesively bonded to the beam surfaces. The beams with the CFRP and HDPP hybrid reinforcement were designed according to a new concept in order to achieve ductility.

The beams were tested in four point bending and they all achieved their expected capacity before failure. It was observed that while the CFRP reinforced beams achieved their strength, the accompanying deflections were large compared to those of steel reinforced beams. Also, the beam that contained only CFRP, the failure was sudden and brittle. On the other hand, the hybrid reinforced beams failed in a relatively ductile fashion and their deflections were smaller than the CFRP reinforced beam.

The beam with external sheet reinforcement were able to reach their expected strength, but the shear cracks were quite wide and the deformations of the CFRP stirrups were very noticeable. Following the rupture of the CFRP sheets, these beams were unloaded and reloaded. Upon reloading they carried a significant amount of load, accompanied by large deflections, before complete failure.

Using first principles, the flexural behavior, strength and load deflection behavior of the beams were predicted very well. Deflection calculation by the code and another effective inertia expression proposed in literature did not gave accurate results. On the other hand, the flexural capacity was accurately predicted by the equivalent rectangular stress block method. A computer program was written based on the first principles and was used to perform the required calculations.

7.2 CONCLUSIONS

The following conclusions are based on the results obtained from the experimental program and the theoretical correlation of experimental results.

- From the observations made on the test Beams B3 and B4, it can be concluded that it is feasible to strengthen for flexure and shear existing reinforced concrete structures by using externally bonded FRP sheets. Successful results requires careful surface preparation, selection of proper adhesive, bonding procedure and FRP sheet.
- The increase in strength using one-ply (Beam B3) and two-ply (Beam B4) of bonded FRP sheets to the tension face was observed to be 2.73 and 4.5 times the ultimate strength of plain concrete beam. The increase

in ultimate load and the stiffness of the beam externally reinforced with two-ply of FRP sheet was almost 85% greater than those of a similar beam (Beam B3) reinforced with only one-ply of FRP sheet.

- Externally bonded FRP shear stirrup can be used for shear strength upgrade of the existing structures. The amount of shear reinforcement can be found by using existing general method for shear design. However, due to large width of shear cracks, one may not be able fully utilize the tensile strength of the CFRP sheet.
- The beam (Beam B7) reinforced with FRP only exhibited a brittle failure mode and disintegrated into three pieces upon rupture of FRP reinforcing bars. The use of FRP reinforcement in a structure should limit the service load to no more than 50% of its ultimate capacity. The option of working stress design for FRP reinforced structures should be investigated.
- The moment-curvature relationship of the beams reinforced externally with CFRP and internally with HDPP can be determined from the strain compatibility and classical reinforced concrete beam analysis.
- The beams (Beams B5 & B6) reinforced with hybrid reinforcement had almost 60% (average of Beams B5 and B6) increase in stiffness and strength with respect to the control beam (Beam B7).
- The proposed hybrid reinforcing concept (used in beams B5 & B6) prevented the brittle failure mode observed in the CFRP reinforced beam (Beam B7).

- There was a marginal decrease (average 11%) in load carrying capacity of hybrid reinforced beams after the rupture of CFRP rebars.
- Computation of deflections and balanced reinforcement ratio according to the existing code is not applicable to FRP reinforced concrete due to the low modulus and high strength of FRP reinforcing materials. Considerations should be given to the computation of deflections using moment-curvature method. The use of modified moment of inertia method is also not considered appropriate for the beams tested in this study. The good bond characteristics for the deformed CFRP rebars used is borne out by the good agreement of the theoretical and experimental load-deflections curves. The inclusion or preclusion of tension stiffening had no great impact on the calculated deflections.
- The deformability index of hybrid reinforced beams (Beams B5 & B6) was found to be 2.4 compared to null for CFRP reinforced beam.
- The result of this study show that it is possible to achieve a completely ductile failure for FRP reinforced concrete beams under flexure. The key to achieving large deformations before failure without substantial loss in strength is the combination of a stiff and strong reinforcement with a relatively flexible reinforcement which is capable of undergoing substantial strain before rupture. The stiff material will be used to achieve primarily strength and deflection control under service loads while the flexible material will impart strength, stiffness and ductility. Further work is needed to determine whether such a system will allow moment redistribution in indeterminate beams and frames.
- While the overall concept of hybrid reinforcement was found to be valid, it may be difficult to construct such beams due to large amount

of HDPP needed to achieve an acceptable level of moment capacity after the rupture of the CFRP.

7.3 RECOMMENDATIONS

Based on this study the following recommendations can be made:

- Test hybrid reinforced beams with smaller amount of HDPP or with other types of secondary reinforcement which has a higher elastic modulus than the HDPP used in this study.
- Test beams with a combination of HDPP and GFRP. Since GFRP has a much lower modulus, the ratio of the moduli of GFRP and HDPP would not be as large that of CFRP and HDPP.
- Test continuous beams with this hybrid reinforcement to determine whether moment distribution can occur after the rupture of the CFRP at the section of maximum moment.
- Investigate the applicability of the hybrid system to prestressed concrete structures where deflection control is not a problem but brittle failure is far more likely.

References

1. ACI Committee 440 Report 1990. "State-of-the Art Report on Applications of FRP Composites".
2. An Wei, Saadatmanesh, H., Ehsani, M.R., 1991. "Beams Strengthened with FRP Plates, Analysis and Parametric study", ASCE Journal of Structural Engineering, Vol. 117. Nov. 1991.
3. Bakht Baider, Gilkie Ronald C., Jaeger Leslie G., "Design Criteria". Advanced Composite Materials with Application To Bridges, CSCE May 1991. pp. 217.
4. Ballinger, C.A., "Development of Fibre-Reinforced Plastic Products for The Construction Market - How Has And Can Be Done". Advanced Composite Material Application To Bridges and Structures, CSCE, pp. 3-14.
5. Basunbul, I.A., Gubati, A.A., Sulaimani, G.J. and Baluch, M.H. 1990. "Repaired Reinforced Concrete Beams", ACI Materials Journal July/Aug. 1990 pp. 348-354.
6. Benmokrane, B., 1994. "Grouted Anchorage's for Aramid Fibre Reinforced Plastic Prestressing Tendons: Discussion". Canadian Journal of Civil Engineering, Vol. 21(4), August 1994. pp. 713.
7. Calder, A.J.J. (1979). "Exposure Tests on Externally Reinforced Concrete Beams - First Two Years". TRRL, Supplementary. Report No. 529.

8. Char, Mohan S., Saadatmanesh, H., and Ehsani, M.R., 1994. "Concrete Girders Externally Prestressed with Composite Plates". *PCI Journal* , Vol. 39 (3), May/June 1994 pp. 40-51.
9. Clear, Kenneth C., Hartt, William H., 1993. "Performance of Epoxy-Coated Reinforcing Steel In Highway Bridges". NCHRP, Summary of Progress Dec. 1993, TRB.
10. Carreira, Domingo, J., and Chu, Kuang-Han, 1986. "Stress-Strain Relationship of Reinforced Concrete In Tension. *ACI Journal, Proceedings* Vol. 83, No. 1, Jan-Feb. , pp. 21-28.
11. Collins, F., and Roper, H. 1990. "Laboratory Investigation of Shear Repair of Reinforced Concrete Beams Loaded In Flexure". *ACI Materials Journal*. March-April 1990.
12. Deblois, M., Picard, A., and Beaulieu, D. 1992. "Reinforcement De Poutres en Beton Arme A L'Aide De Matériaux Composites : études Théorique Et Expérimentale". *Proceedings of First Conference on Advanced Composite Material in Bridges and Structures, Sherbrooke*. pp. 265-276.
13. Dunker, Kenneth F., Klaiber, Wayne D., and Sanders Wallace Jr. "Bridge Strengthening Needs In United States:.. *Transportation Research Record* 1118. TRB Washington pp. 1-9.

14. Dussek, Ian 1980. "Strengthening of Bridge Beams and Similar Structures By Means of Epoxy-Resin-Bonded External Reinforcement". Transportation Research Board, Crowthorne. pp. 21-24.
15. Ehasani, M.R., Saadatmanesh, H., and Tao, S. 1993. "Bond of GFRP Rebars to Ordinary-Strength Concrete", FRP'93, Kyoto, Japan, Oct. 17-20. - International Symposium, pp. 333- 346.
16. Elgabroni, Ali A. 1989. "Structural Behavior of Concrete Beams Reinforced With Polymer Grids and High Strength Steel". Masters Dissertation, Department of Civil Engineering , Carleton University, Ottawa, Ontario.
17. Erki M. A., Johnson G.E., Roll F., Marran K., Sleeth D. (1992). "Creep Relaxation Characteristics of A Prestressed Kevlar Cable/FRP Tube Structural System". Advanced Composite Materials In Bridges and Structures (p-445).
18. Faza Salem Samir 1992. "Bending and Bond Behavior and Design of Concrete Beams Reinforced with Fiber Reinforced Plastic Rebars". Ph.D. discretion, West Virginia University.
19. Faza, S. S., and Ganga Rao, H.V.S. 1992. "Pre- and Post-Cracking Deflection Behavior of Concrete Beams Reinforced With Fibre-Reinforced Plastic Rebars". Proceedings of First International Conference on Advanced Composites Materials in Bridges and Structures (ACMBS-1), Sherbrooke, Oct. , pp. 2 : 3-251., pp. 151-160.

20. Faza, S. S., and Ganga Rao, H.V.S. 1993. "Theoretical and Experimental Correlation of Behavior of Concrete Beams Reinforced with Fiber Reinforced Plastic Rebars". FRP '93, Kyoto, Japan, Oct. 17-20. International Symposium. pp. 599-614
21. Fleming, C. J., and King, G.E.M. 1967. "The Development of Structural Adhesives for Three Original Uses in South Africa". Bulletin RILEM No. 37, Dec. 1967, pp. 241-251
22. Holman, J.W. and Cook, J.P. 1984. "Steel Plates for Torsion Repair of Concrete Beams". Journal of Structural Division, ASCE, VOL. 110, No. 1, JAN. 1984, pp. 10-18.
23. Gilbert, R. Ian, and Warner, Robert, F., 1978. "Tension-Stiffening in Reinforced Concrete Slabs. Proceedings, ASCE, Vol. 104, ST 12, Dec., pp. 1885-1900.
24. Ichimasu, H., Maruyama, M., Watanabe, H., and Hirose, T., 1993. "RC Slabs Strengthened By Bonded Carbon FRP Plates Part 2 - Applications". Fiber Reinforced-Plastic Reinforcement for Concrete Structures, FRP'93, Kyoto, Japan, Oct. 17-20. International Symposium 1993. pp. 957-970
25. Ichimasu, H., Maruyama, M., Watanabe, H., and Hirose, T., 1993. "RC Slabs Strengthened By Bonded Carbon FRP Plates Part 1- laboratory Study". Fiber Reinforced-Plastic Reinforcement for Concrete Structures, FRP'93, Kyoto, Japan, Oct. 17-20. International Symposium 1993. pp. 933-956.
26. International Association for Bridge and Structural Engineering (IABSE) C-46/88, Repair and Rehabilitation of Bridges - Case Studies 1, Aug. 1988.

27. Irwin, C.A.A. 1975. "Strengthening of Concrete Beams By Bonded Steel Plates".
TRRL Supplementary Report No. 160, Transport and Road Research Laboratory
Crowthorne, Berkshire, 1975. 8 pp.
28. Iyer, S. L. And Khubchandani, A. (1992). "Evaluation of Graphite Composite Cables
for Prestressing Concrete". Proceedings of First International Conference Advanced
Composite Material in Bridges and Structures Sherbrooke. pp. 73-79.
29. Jones, R., Swamy, R.N., Bloxham, J. and Blouderbalah, A. 1980. "Composite
Behavior of Concrete Beams with Epoxy-Bonded External Reinforcement".
International Journal of Cement Composites and Lightweight Concrete, V.2, No. 2,
May. pp. 91-107.
30. Jones, R., Swamy, R.N., Charif, A. 1988. "Plate Separation and Anchorage of
Reinforced Concrete Beams Strengthened By Epoxy-Bonded Steel Plates". Journal of
the Institute of Structural Engineers. The Structural Engineering Journal Vol. 66(5).
31. Koerner, R.M. 1985. "Designing with Geosynthetics". Parentice-Hall, New Jersey. pp.
15.
32. Kakizawa, T., Ohno, S. and Yonezawa, T. 1992. "Fiber-Reinforced-Plastic
Reinforcement ". International Symposium. pp. 585-598.
33. Kaliber, F.W., Dunker, K.F., Wipf and Sanders, W.W. Jr. (1987), "Methods of
Strengthening Existing Highway Bridges". Transportation Research Record 293, TRB,

34. Karam, G.N. 1992. "Optimal Design for Prestressing with FRP Sheets in Structural Members". *Advanced Composite Materials in Bridges and Structures*, pp. 277-286.
35. Kato, T. and Hayashida, N. 1993. "Flexural Characteristics of prestressed Concrete Beams with CFRP Tendons". *ACI SP-138*, Nanni and Dolan (Editors) pp. 419-440.
36. Kent, Dudley Charles, and Park, Robert, "Flexural Members with Confined Concrete," *Proceedings, ASCE*, V. 97, ST7, July 1971, pp. 1969-1990.
37. Lloyd, G.O., Calder, A.J.J. 1982. "The Microstructure of Epoxy Bonded Steel-to Concrete Joints". *TRRL, Supplementary. Report No. 705*.
38. Macdonald, M.D. 1978. "The Flexural Behavior of Concrete Beams with Bonded External Reinforcement". *Transportation and Road Research Laboratory, Crowthorne, Berkshire. Supplementary Report No. 415*.
39. Macdonald, M.D. 1982. "The Flexural Performance of 3.5 m Concrete Beams with Various Bonded External Reinforcements". *TRRL, Sup. Report No. 728*.
40. Makitani, E., Irisawa, I., and Nishiura, N. 1993. "Investigation of Bond in Concrete Members with Fiber Reinforced Plastic Bars". *FRP '93, Kyoto, Japan, Oct. 17-20. International Symposium*. pp. 315-332.
41. Meier, U., and Kaiser, H. 1991. "Strengthening of Structures with CFRP Laminates", *Advanced Composite Materials in Civil Engineering Structures, Proceedings of Specialty Conference, Las Vegas, Nevada, Jan. 31-Feb. 1, 1991, ASCE, New York*.

- 42.Meier, U., Deuring, M., Meier, H., and Schwegler, G. 1992. "Strengthening of Structures with CFRP Laminates: Research and Applications in Switzerland". Proceedings of First International Conference on Advanced Composites Materials in Bridges and Structures (ACMBS-1), Sherbrooke, Oct. , pp. 243-251.
- 43.Maruyama, T., Honma, M., and Okamura, H., 1993. "Experimental Study on Tensile Strength of Bent Portion of FRP Rods", FRP '93, Kyoto, Japan, Oct. 17-20. First International Symposium, pp. 163-176.
- 44.Meier, U. 1992. "Case Histories", Advanced Composite Materials with Application To Bridges. Edited by Mufti, A. A., Erki, M. A., and Jaeger, L. Canadian Society of Civil Engineers. pp. 274.
45. Mitsubishi Kasei Corporation 1992., 5-2, Mirunouchi 2-chome, Chiyoda-ku, Tokyo 100, Japan.
- 46.Mochizuki, S., Matsuzaki, Y., and Sugita, M., "Evaluation Items and Methods of FRP Reinforcement as Structural Elements", FRP '93, Kyoto, Japan, Oct. 17-20. International Symposium, pp. 117-132
- 47.Mckay, K.S. and Erki, M.A., 1994. "Grouted Anchorage for Aramid Fibre Reinforced Plastic Prestressing Tendons: Reply". Canadian Journal of Civil Engineering, Vol. 21(4), august 1994. pp. 715.
- 48.Marzouk, H., and Chen, Z.W., 1993. Nonlinear Analysis of Normal- and High-Strength Concrete Slabs. Canadian Journal of Civil Engineering, Vol. 20, 1993.

49. Mufti, A. A., Erki, M. A., and Jaeger, L. 1991. "Advanced Composite Materials with Application to Bridges". Canadian Society of Civil Engineers. pp. 10.
50. Machida, A. (Editor), 1993. "State-of-the-Art Report on Continuous Reinforcing Materials". Japan Society of Civil Engineers, Concrete Engineering Series, , Tokyo, pp. 164.
51. Nanni, A. and Dolan, C.W. (Editors) 1992. "Fiber-Reinforced-Plastic reinforcement for Concrete Structures". ACI SP-138, pp. 976.
52. Naaman, A.E., Tan, K.H., Joeng, S.M. and Alkhairi, F.M. 1993. "Partially Prestressed Beams with Carbon Fibre Composite Strands: Preliminary Test Evaluations". Fiber-Reinforced-Plastic Reinforcement for Concrete Structures". ACI SP-138, Nanni and Dolan (Editors) pp. 441-464.
53. Naaman, A. E., 1993. Ductility Implications for Prestressed and Partially Prestressed Concrete Structures Using Fiber Reinforced Plastic Reinforcements. FRP Symposium '93, Kyoto, Japan, Oct. 17-20.
54. National Cooperative Highway Research Program (NCHRP), Research Results Digest, Digest 143 - February 1984 .
55. Ochlers, Deric John, "Reinforced Concrete Beams with Plates Glued to Their Soffits", ASCE, Journal of Structural Engineering Vol. 118(8). P- 2023-2038.
56. Park, R., and Pauley, T. 1975 Reinforced Concrete Structures, John Wiley & Sons, New York, NY.

57. Razaqpur, A.G. and Kashef, A.H., 1993. "State-of-The-Art on Fibre Reinforced Plastics for Buildings". Carleton University, Ottawa, Canada.
58. Razaqpur, A.G. and Halim, A.O.A., 1991. "Determination of Tensile Strength and Maximum Strain of Tensar UX1500 and UX1600 Using Three Different Sets of Jaws", Final Report Submitted to Tensar Corporation, Georgia, USA
59. Raithby, K.D. 1980. "External Strengthening of Concrete Bridges with Bonded Steel Plates". TRRL Supplementary. Report No. 612.
60. Ritchie, P.A., Thomas, D.A., Lu, Le-Wu, and Connelly, G.M. 1991. "External Reinforcement of Concrete Beams Using Fiber Reinforced Plastics," ACI Structural Journal, V. 88, No. 4, July-August 1991, p. 490-499.
61. Ritchie, Philip 1989. "External Reinforcement of Concrete Beams using Fiber Reinforced Plastics". Masters Thesis.
62. Rostasy, F.S., Hankers, C., Ranisch, E.H., 1992. "Strengthening of R/C- and P/C- Structures with Bonded FRP Plates". Proceedings of First International Conference on Advanced Composites Materials in Bridges and Structures (ACMBS-1), Sherbrooke, Oct., pp. 253-264.
63. Prakhya, G.K.V, and Morley, C.T., 1990, "Tension-Stiffening and Moment-Curvature Relations of Reinforced Concrete Elements. ACI Structural Journal, Sept.-Oct., pp. 597-605.

64. Park, R and Ruitong, Dai, 1988, "Ductility of Doubly Reinforced Concrete Beam Sections". ACI Structural Journal, March-April, pp. 217-225.
65. Razaqpur, A.G., 1991. "Analysis and Computer Programs". Advanced Composite Materials with Application to Bridges, CSCE, May, pp. 179-216.
66. Scolz, H., 1990, "Ductility, Redistribution, and Hyperstatic Moments in Partially prestressed members. ACI Structural Journal, May-June, pp. 341-349.
67. Saadatmanesh, H, and Ehsani, M.R., 1990. "Fiber Composite Plates Can Strengthen beams. Concrete International March 1990, Vol. 12(3)
68. Saadatmanesh, H., and Ehsani, M.R., 1991. "RC Beams Strengthened with GFRP Plates I: Experimental Study". ASCE Journal of Structural engineering, Vol. 117. Nov. 1991.
69. Saeki, N., Horiguchi, T., Inomata, M., Hata, S., Ikeda, T. 1993 (b). "Strengthening of Damaged Concrete Beams By External Prestressing of Aramid Fiber Cable". Fiber Reinforced-Plastic Reinforcement for Concrete Structures, FRP'93, Kyoto, Japan, Oct. 17-20. International Symposium 1993. pp. 913-932.
70. Sagues, A., Powers, R. And Zayed, A. (1990). Marine Environment Corrosion of Epoxy-Coated Reinforcing Steel, Corrosion of Reinforcement in Concrete (p. 539-549) as reported in ACMBS pp. - 136.

71. Sharif Alfarabi, al-Sulaimani, Basunbul. I.A., Baluch. M.H., Ghaleb, B.N. 1994.
"Strengthening of Initially Loaded Reinforced Concrete Beams using FRP Plates".
ACI Structural Journal, March/April 1994.
72. Swamy, R.N., Jones, R. and and, T.H. 1982. "Under-and Over-Reinforced Concrete
Beams with Glued Steel Plates". The International Journal of Cement Composites and
Light Weight Concrete, No. 1, Feb. pp. 19-32.
73. Swamy, R.N., and Jones, R. 1980. "Behavior of Plated Reinforced Concrete Beams
Subjected To Cyclic Loading During Glue Hardening". The International Journal of
Cement Composites, No. 4, Nov. pp. 233-234.
74. Swamy, R.N., Jones, R., Bloxham, J.W. 1987. "Structural Behavior of Reinforced
Concrete Beams Strengthened By Epoxy-Bonded Steel Plates". The Structural
Engineer. Vol. 65A(2).
75. Toensmeyer, Michael R., and Cook. John P. (1981), "Repair of Torsonally Inadequate
Concrete Beams By Use of Adhesively Bonded Plates". Transportation Research
Record 821, TRB, Washington.
76. Tonen Corporation (1992). "Flexural Performance of Steel Reinforced Concrete
Beams Reinforced with Carbon Fiber sheet". Presented at 47th Annual Meeting for
The Japanese Civil Engineering Society, Sept. 1992.
77. Triantafillou, T. C., and Deskovic, N., 1991. "Innovative Prestressing with FRP
Sheets Mechanics of Short-Term Behavior". Journal of Engineering Mechanics, Vol.
117(7), July 1991, pp. 1652-1671.

78. Triantafillou, T. C., Deskovic, N., and Deuring, M. 1992. "Strengthening of Concrete Structures with Prestressed Fiber Reinforced Plastic Sheets". *ACI Structural Journal*, 89(3). pp. 235-244.
79. Triantafillou, T. C., Plevris, N., (1991). "Post-Strengthening of R/C Beams with Epoxy-Bonded Fiber Composite Materials". *Advanced Composite Materials In Civil Engineering Structures, Proceedings of Specialty Conference, Las Vegas, Nevada, Jan. 31-Feb. 1, 1991, ASCE, New York*, pp. 245-256.
80. Triantafillou, T.C. and Meier, U. 1992. "Innovative Design of FRP Combined with Concrete. Proceedings of First International Conference on Advanced Composites Materials in Bridges and Structures (ACMBS-1), Sherbrooke, Oct. , pp. 491-500.
81. Taniguchi, H Mutsuyoshi, H., Kita, T. and Machida, A 1993. "Ductile Behavior of beams Using FRP as Tendons and Transverse Reinforcement". *ACI SP-138, Nanni and Dolan (Editors)* pp. 651-670.
82. Taerwe, L. 1995. "Non-Metallic (FRP) Reinforcement for Concrete Structures". *Proceedings of Second International RILEM Symposium (FRPRCS-2), U.K., pp. 714.*
83. Uji, Kimitaka, 1992. "Improving Shear Capacity of Existing Reinforced Concrete Member By Applying Carbon Fiber Sheet". *Transactions of the Japan Concrete Institute. Vol. 14.*
84. VanGermert, D.A., and VandenBosch, M.C.J. 1985. "Repair and Strengthening of Reinforced Concrete Structures By Means of Epoxy Bonded Steel Plate." *Int.*

Conference. On Deterioration, Bahrain. 181-192 [Reported by Saadatmanesh and Ehsani (1991)].

Appendix A

Table A1: Theoretical Values For SRC Beams B1 & B2 [14LAD]

$f_c = 35.00$	MPa	$L = 4000.00$	mm
$C_{fr} = 3.50$	MPa	$b = 200.00$	mm
$E_c = 29580.40$	MPa	$h = 400.00$	mm
$I_{tr} = 1.10167E^{+09}$	mm ⁴	$A_{st} = 400.00$	mm ²
$M_{cr} = 19.35$	kN-m	$\psi_{cr} = 5.9107^{-07}$	rad/mm

Deflections by M- ψ

Trilinear Steel Curve

Tension Stiffening Model -1

S. No.	Load (kN)	Moment (kN-m)	Concrete Strain	Tension Steel Strain	N. A. (mm)	Curvature (rad/mm)	Deflection (mm)
1	22.00	14.66	0.0001	2.14E ⁻⁰⁴	103.56	9.66E ⁻⁰⁷	0.520
2	34.96	23.31	0.0002	5.69E ⁻⁰⁴	84.56	2.37E ⁻⁰⁶	0.920
3	47.83	31.89	0.0003	9.27E ⁻⁰⁴	79.44	3.78E ⁻⁰⁶	2.582
4	60.40	40.27	0.0004	1.28E ⁻⁰³	77.44	5.17E ⁻⁰⁶	5.177
5	72.60	48.40	0.0005	1.62E ⁻⁰³	76.60	6.53E ⁻⁰⁶	7.425
6	84.37	56.25	0.0006	1.96E ⁻⁰³	76.32	7.86E ⁻⁰⁶	11.188
7	87.52	58.35	0.0007	2.59E ⁻⁰³	69.16	1.01E ⁻⁰⁵	12.051
8	88.45	58.97	0.0008	3.37E ⁻⁰³	62.36	1.28E ⁻⁰⁵	14.063
9	89.22	59.48	0.0009	4.22E ⁻⁰³	57.16	1.57E ⁻⁰⁵	16.908
10	89.93	59.95	0.001	5.12E ⁻⁰³	53.12	1.88E ⁻⁰⁵	17.512
11	90.59	60.39	0.0011	6.06E ⁻⁰³	49.92	2.20E ⁻⁰⁵	25.733
12	91.14	60.76	0.0012	7.04E ⁻⁰³	47.32	2.54E ⁻⁰⁵	29.515
13	91.71	61.14	0.0013	8.04E ⁻⁰³	45.24	2.87E ⁻⁰⁵	33.403
14	92.25	61.50	0.0014	9.05E ⁻⁰³	43.56	3.21E ⁻⁰⁵	34.383
15	92.70	61.80	0.0015	1.01E ⁻⁰²	42.16	3.56E ⁻⁰⁵	35.927
16	93.20	62.13	0.0016	1.11E ⁻⁰²	40.92	3.91E ⁻⁰⁵	39.247
17	93.61	62.41	0.0017	1.21E ⁻⁰²	39.92	4.26E ⁻⁰⁵	42.516
18	94.01	62.67	0.0018	1.32E ⁻⁰²	39.12	4.60E ⁻⁰⁵	45.743
19	94.40	62.93	0.0019	1.41E ⁻⁰²	38.48	4.94E ⁻⁰⁵	48.929
20	95.44	63.63	0.002	1.50E ⁻⁰²	38.24	5.23E ⁻⁰⁵	52.150
21	97.66	65.11	0.0021	1.56E ⁻⁰²	38.60	5.44E ⁻⁰⁵	55.399
22	98.20	65.47	0.0022	1.65E ⁻⁰²	38.32	5.74E ⁻⁰⁵	58.385
23	98.67	65.78	0.0023	1.73E ⁻⁰²	38.08	6.04E ⁻⁰⁵	61.323
24	99.09	66.06	0.0024	1.82E ⁻⁰²	37.88	6.34E ⁻⁰⁵	64.217
25	99.58	66.39	0.0025	1.90E ⁻⁰²	37.86	6.62E ⁻⁰⁵	67.084
26	99.97	66.64	0.0026	1.98E ⁻⁰²	37.64	6.91E ⁻⁰⁵	69.898
27	100.36	66.91	0.0027	2.07E ⁻⁰²	37.56	7.19E ⁻⁰⁵	72.678
28	100.77	67.18	0.0028	2.15E ⁻⁰²	37.56	7.64E ⁻⁰⁵	75.425
29	101.21	64.47	0.0029	2.22E ⁻⁰²	37.52	7.73E ⁻⁰⁵	78.140
30	101.58	67.72	0.003	2.30E ⁻⁰²	37.52	8.00E ⁻⁰⁵	80.810
31	101.90	67.93	0.0031	2.38E ⁻⁰²	37.52	8.26E ⁻⁰⁵	83.435
32	102.26	68.17	0.0032	2.45E ⁻⁰²	37.56	8.52E ⁻⁰⁵	94.660
33	102.68	68.45	0.0033	2.52E ⁻⁰²	37.64	8.77E ⁻⁰⁵	97.410
34	102.95	68.64	0.0034	2.59E ⁻⁰²	37.68	9.02E ⁻⁰⁵	100.260
35	103.29	68.86	0.0035	2.66E ⁻⁰²	37.76	9.27E ⁻⁰⁵	102.990

Table A2: Theoretical Values For CLTRC Beam B3 [NLAD]

$f_c =$	35.00	MPa	$L =$	4000.00	mm
$C_{fr} =$	3.50	MPa	$b =$	200.00	mm
$E_c =$	29580.40	MPa	$h =$	400.00	mm
$A_{cs1} =$	33.00	mm ²	$A_{cs2} =$	57.75	mm ²
$E_{cs} =$	130000	MPa	(A _{cs2} on sides 2x175x0.165)		
$A_{ge} =$	1330.00	mm (wide)			
$M_{cr} =$	18.66	kN-m	$\psi_{cr} =$	5.91 ⁻⁰⁷	rad/mm
Deflections by M- ψ			No Tension Stiffening		

S. No.	Load (kN)	Moment (kN-m)	Concrete Strain	Curvature (rad/mm)	Deflection (mm)
1	6.50	4.34	0.0001	3.48E ⁻⁰⁶	0.52
2	12.85	8.56	0.0002	6.88E ⁻⁰⁶	0.81
3	19.02	12.68	0.0003	1.02E ⁻⁰⁵	3.42
4	25.01	16.67	0.0004	1.35E ⁻⁰⁵	11.20
5	30.85	20.57	0.0005	1.66E ⁻⁰⁵	16.32
6	26.51	24.34	0.0006	1.97E ⁻⁰⁵	22.05
7	42.02	28.01	0.0007	2.27E ⁻⁰⁵	27.16
8	47.33	31.55	0.0008	2.56E ⁻⁰⁵	36.31
9	52.43	34.95	0.0009	2.084E ⁻⁰⁵	39.87
10	57.31	38.21	0.001	3.12E ⁻⁰⁵	48.21

The above table show the analysis up to the rupture of CFRP sheet at tension face.

Note HDPP is treated as mm wide in analysis not mm².

The experimental load-strain values of Tensar UX-1700 were best curve fitted (polynomial) and the obtained equation is used in NLAD for material modeling of Tensar.

Table A3: Theoretical Values For CLTRC Beam B4 [NLAD]

$f_c =$	35.00	MPa	$L =$	4000.00	mm
$C_{fr} =$	3.50	MPa	$b =$	200.00	mm
$E_c =$	29580.40	MPa	$h =$	400.00	mm
$A_{cs1} =$	66.00	mm ²	$A_{cs2} =$	66.00	mm ²
$E_{cs} =$	130000	MPa			
$A_{ge} =$	1330.00	mm (wide)			
$M_{cr} =$	18.66	kN-m	$\psi_{cr} =$	5.91^{-07}	rad/mm
Deflections by M- ψ			No Tension Stiffening		

S. No.	Load (kN)	Moment (kN-m)	Concrete Strain	Tension Face CFRP Strain	Curvature (rad/mm)	Deflection (mm)
1	9.40	6.27	0.0001	$1.02E^{-03}$	$2.79E^{-06}$	0.47
2	18.57	12.38	0.0002	$2.01E^{-03}$	$5.52E^{-06}$	0.65
3	27.49	18.33	0.0003	$2.98E^{-03}$	$8.19E^{-06}$	9.95
4	36.15	24.10	0.0004	$3.92E^{-03}$	$1.08E^{-05}$	14.92
5	44.58	29.72	0.0005	$4.84E^{-03}$	$1.33E^{-05}$	19.11
6	52.79	35.19	0.0006	$5.72E^{-03}$	$1.58E^{-05}$	26.20
7	60.66	40.44	0.0007	$6.59E^{-03}$	$1.82E^{-05}$	28.72
8	68.29	45.52	0.0008	$7.43E^{-03}$	$2.06E^{-05}$	35.71
9	75.68	50.45	0.0009	$8.24E^{-03}$	$2.28E^{-05}$	32.76
10	82.76	55.17	0.001	$9.02E^{-03}$	$2.51E^{-05}$	35.74
11	89.58	59.72	0.0011	$9.77E^{-03}$	$2.72E^{-05}$	48.71
12	96.07	64.05	0.0012	$1.05E^{-02}$	$2.92E^{-05}$	52.28
13	102.28	68.19	0.0013	$1.12E^{-02}$	$3.12E^{-05}$	53.19

The above table show the analysis up to the rupture of CFRP sheet at tension face.

Note HDPP is treated as mm wide in analysis not mm².

The experimental load-strain values of Tensar UX-1700 were best curve fitted (polynomial) and the obtained equation is used in NLAD for material modeling of Tensar.

Table A4: Theoretical Values For CRRC Beams B5 & B6 [NLAD]

$f_c =$	80.30	MPa	$L =$	4000.00	mm
$C_{fr} =$	5.37	MPa	$b =$	200.00	mm
$E_c =$	44805.13	MPa	$h =$	400.00	mm
$A_{ge} =$	3330	mm (wide)	$A_c =$	98.60	mm ²
$M_{cr} =$	28.64	kN-m	$\psi_{cr} =$	5.99E ⁻⁰⁷	rad/mm
Deflections by M- ψ			Tension Stiffening Model -1		

S. No.	Load (kN)	Moment (kN-m)	Concrete Strain	N. A. (mm)	Deflection (mm)
1	27.43	18.29	0.0001	46.9	2.37
2	42.75	28.50	0.0002	36.4	5.09
3	58.34	38.89	0.0003	33.6	9.91
4	73.71	49.14	0.0004	32.4	13.6
5	88.73	59.15	0.0005	31.9	20.9
6	103.37	68.91	0.0006	31.79	24.5
7	117.50	78.34	0.0007	31.71	27.1
8	131.35	87.57	0.0008	31.80	31.2
9	144.57	96.38	0.0009	31.90	34.4
10	157.38	104.92	0.001	31.71	37.5
11	169.64	113.09	0.0011	30.51	40.6
12	181.36	120.91	0.0012	30.10	43.4
13	192.79	128.52	0.0013	29.85	51.2

Above table show the customized output of the analysis up to the rupture of CFRP rebars.

Note HDPP is treated as mm wide in analysis not mm².

The experimental load-strain values of Tensar UX-1700HT were best curve fitted (polynomial) and the obtained equation is used in NLAD for material modeling of Tensar.

Table A5: Theoretical Values For CRRC Beam B7 [NLAD]

$f_c =$	80.30	MPa	$L =$	4000.00	mm
$C_{fr} =$	5.37	MPa	$b =$	200.00	mm
$E_c =$	44722.00	MPa	$h =$	400.00	mm
$A_{gc} =$	0.0	mm	$A_c =$	98.6	mm ²
$M_{cr} =$	28.64	kN-m	$\psi_{cr} =$	5.99E ⁻⁰⁷	rad/mm
Deflections by M- ψ			Tension Stiffening Model -1		

S. No.	Load (kN)	Moment (kN-m)	Concrete Strain	N. A. (mm)
1	12.03	8.02	0.0001	31.76
2	23.63	15.72	0.0002	31.28
3	34.91	23.27	0.0003	30.84
4	45.86	30.57	0.0004	30.44
5	56.54	37.69	0.0005	30.04
6	66.80	44.53	0.0006	29.64
7	76.78	51.19	0.0007	29.24
8	86.51	57.68	0.0008	28.88
9	95.87	63.91	0.0009	28.56
10	104.84	69.89	0.001	28.20

Note without the inclusion of tension stiffening the ultimate load capacity of the beam is almost 98.16 kN which suggests that CFRP rebars posses higher ultimate strength than the manufacturer's guaranteed minimum strength of 1813 MPa. Therefore, if we go backwards and find the ultimate strength of CFRP rebar using the ultimate load of 125.40 kN (experimental) we get approximately 2316 MPa, which is almost 21% higher than the manufacturers suggested value.

Appendix B

MATERIAL PROPERTIES

B.1 CONCRETE

Two batches of ready mixed concrete were used for casting the concrete beams. The first batch, concrete set 1, was used to construct the SRC and CLTRC beams. The second batch, concrete set 2, was used to construct the CRTRC and CRRC beams. To ensure that concrete is able to pass through the smallest spacing of Tensar grids, which is approximately 15 mm, a maximum aggregate size of 10 mm was used in both concrete sets. The ready mix concrete was mixed with 1.0 liter of superplasticizer for every 100 kg of cement and mixed vigorously in concrete truck before casting the beams. In concrete set 2, along with superplasticizer water reducing agent PD25XL was also mixed. The water cement ratio for both sets of concrete was 0.32. Twelve cylinders were taken in each concrete set at the beginning, middle and end of the casting of the beams. All the cylinders were finished to smooth and level surface and left in their mold for twenty four hours. After the cylinders were demolded and transferred to the curing room.

After 28 days of minimum curing, uniaxial compression tests on 152 mm x 305 mm concrete cylinders were completed in accordance with CAN/CSA A23.2-9C-M90, Compressive Strength of Cylindrical Concrete Specimens (Canadian Standards Association, 1990) on a *Tinius Olsen* 60,000 lb capacity universal testing machine. Prior to testing all the cylinders were capped on both sides with sulphur. The cylinders in concrete set 2 were attached with clip on gauge with a 25 mm base length, which is controlled by *MIS 458-20 Micro Console* operated on an IBM 486 compatible.

Table B-1 summarize the results of the concrete set 1. The average compressive strength of concrete, f_c is ≈ 35 MPa. Usually the tensile strength of concrete is approximately 10% of the ultimate compressive strength of concrete. According to CSA A23.2-M77, the modulus of rupture, f_r of normal density concrete can be taken as

$$\begin{aligned} f_r &= 0.6 \sqrt{f'_c} && \text{MPa (for 35 MPa concrete)} \\ &= 3.54 && \text{MPa (which is almost 10\% of } f_c) \end{aligned}$$

Table B-2 summarizes the results of the concrete set 2 which was tested at various period of curing time by Francon Lafarge Ltd. (cylinders were tested under load control). The

average compressive strength of concrete f_c at 90 days $\approx$ 80 MPa. After 81 days of curing cylinders were also tested under displacement control.

$\therefore$ the modulus of rupture for 80 MPa concrete, $f_r = 5.36$ Mpa.

Table B-1: Concrete Compressive Strength after 28 Days

Cylinder No.	Diameter D	Length L	Area $A = \frac{\pi \times D^2}{4}$	Total Load P_u	Ultimate Stress $f_c = \frac{P_u}{A}$
	mm	mm	mm ²	N	MPa
1	152.0	305.0	18146.00	625311.16	34.46
2	148.6	305.0	17342.06	610440.51	35.20
3	150.2	305.0	17117.52	536413.58	35.92
4	147.8	305.0	17155.84	617781.79	36.01

Average f_c for concrete set 1 = 35.39 MPa

**Table B-2: Concrete Compressive Strength During The Curing Period
(Load Control Testing)**

Cylinder No.	After No. of Days Air Cured (AC); Wet (WC)	f_c (MPa)	E_c (MPa)
1	4 (AC)	48.3	38224.01
2	7 (4 AC & 3 WC)	60.6	42815.30
3	7 (4 AC & 3 WC)	62.6	43516.08
4	28 (4 AC & 24 WC)	78.9	48854.11
5	28 (4 AC & 24 WC)	72.9	46959.32
6	56 (4 AC & 52 WC)	75.7	47853.16
7	81 (4 AC & 52 WC)	80.2	49524.94
8	90 (4 AC & 86 WC)	87.7	51506.55

(Displacement Control Testing)

No. of Specimen	f_c (MPa)	E_c (MPa)	Ultimate Strain
3	69.9	43688	0.187%

B.2 STEEL REINFORCEMENT

The tensile and shear reinforcement consisted of 15M and 10M (16.0 mm and 11.3 mm diameter) deformed bars with a nominal yield strength of 400 MPa. To confirm the yield strength of reinforcing bars four coupons were taken from the reinforcing bars used in the construction of reinforced concrete beams and tested on a Tinius Olsen 60,000 lb universal testing machine. The tensile strains were measured using an extensometer with a 25 mm base length.

The steel bars exhibited initial linear elastic portion and a yield plateau followed by strain hardening until ultimate strain is reached. The observed yield strength exceeded the specified guaranteed minimum strength of 400 MPa by an average of 19 MPa. Since reinforcing steel has very standard characteristics the stress-strain curve was not plotted. Tri-linear stress-strain relation for steel was assumed (Figure B1) in the analysis of test beams.

B.3 TENSAR GRIDS

B.3.1 OVERVIEW

The highest uniaxial strength polymer grids available so far are Tensar UX-1700 and UX-1700HT, which were used as reinforcing material in this experiments. Tensar is the trade name for the high strength polymer grids. Initially, polymer grids are of heavy gauge and in sheet form. Holes are then punched into the sheeting on a regular pattern while the sheet is drawn uniaxially or biaxially [Koerner (1985)]. Figure B.2 shows a typical mesh. Tensar grids come in 1.5 m x 20 m rolls, required size can be obtained by cutting through the junctions. Sectional tensile strength of the Tensar grids depends on the number of

strands available in the section. The strand dimensions for UX-1700 and UX-1700HT is approximately 145 mm in length, 2.2 mm thick and the width varies from 12 mm at the junctions to 6.0 mm at the center of the rib. The strength of polymer grids are measured in terms of their width per meter run. Therefore in the computer program the best fitted curve was used.

B.3.2 TEST SPECIMEN

To determine the tensile strength and maximum strain of Tensar UX-1700 and UX-1700HT a series of specimen were tested as shown in Table B-3. The test specimen consisted of nominally identical width with a mean value of 180 mm. All the specimen consisted of one to eight longitudinal strands with the length varying from 300 mm to 311. Specimen were tested in axial tension (machine direction).

Table B-3: Tensar Test Specimen Summary

Tensar Type	No. Of Specimen	No. of Ribs in each Specimen
UX-1700	12	8
UX-1700HT	6	8
"	3	7
"	2	6
"	2	4
"	3	2
"	3	1

B.3.3 TEST EQUIPMENT

A 100 kN Instron Universal Testing Machine, connected to IBM compatible computer to induce constant strain throughout the loading. Displacements were measured using a clip-on extensometer with a gauge length of 50.8 mm connected to an automatic load displacement plotter. Wedge jaws were used to clamp the mesh at top and bottom junctions while loading. It was reported [Razaqpur and Halim (1991)] that the mean failure strength for the wedge jaws was lowest of liquid metal, concrete and wedge jaws.

B.3.4 TEST METHOD

The specimens were tested at a constant rate of 25 mm/min., which gives strain rates of approximately 9% at a temperature of 20° C with a relative humidity of 60% to 70%. The clamp-mesh were monitored carefully for any slippage in the clamps. The load-displacement responses were displayed on the monitor through the duration of test.

Table B-5 and B-6 summarizes the observed strains of various specimens and their corresponding load. Also the load corresponding to 5% and 10% strain are given which will be used to derive the secant modulus of the specimen. Table B-5 and B-6 illustrates the mean tensile strength with the corresponding strains for the UX-1700 and UX-1700HT. Figures B.3 to B.6 represents the load-strain curves for all specimen tested. Each curve illustrates the complete response of the specimen, including the maximum load and its corresponding strain. It can be concluded that the observed values exceed the manufactures reported values for strength and maximum elongation at ultimate.

Table B-4 Tensar UX1700 Test Data Summary

Specimen (8 Strands)	Load at 5% Strain (kN/m)	Load at 10% Strain (kN/m)	Max. Load (kN/m)	Strain at Max. Load (%)
A	81.93	131.00	132.49	10.13
B	85.70	-	134.03	9.9
C	78.28	120.91	122.66	10.55
D	80.39	126.27	129.32	10.07
E	84.10	-	130.20	9.98
F	83.21	124.39	126.31	13.10
G	81.20	129.31	131.69	10.63
H	84.71	-	133.30	9.96
I	82.01	-	140.21	9.95
J	81.06	-	138.20	9.69
K	83.59	-	130.20	9.99
L	82.56	-	131.50	9.98

Table B-5 Tensar UX-1700HT Test Data Summary

Specimen (8 Strands)	Load at 5% Strain (kN/m)	Load at 10% Strain (kN/m)	Max. Load (kN/m)	Strain at Max. Load (%)
A	85.37	142.08	142.56	10.20
B	90.37	149.27	150.01	11.10
C	88.39	157.41	157.41	10.00
D	89.01	135.81	135.81	10.00
E	89.27	145.55	145.55	10.00
F	85.21	141.39	147.29	10.34

Table B-6 Mean Values of Test Results**Tensar UX1700 (8 Strands)**

	Tensile Strength in kN/m	Strain Percent
Mean (M)	131.675	10.323

Tensar UX1700-HT (8 Strands)

	Tensile Strength in kN/m	Strain Percent
Mean (M)	146.438	10.270

The factor to convert test result to kN/m was:

$$\frac{1b * 1 kN * 45 strands}{8 strands * 224.82 m} = 0.025$$

Steps to convert displacement into strain was:

Pitch of clip-on-extensometer = 2 inches (50.8 mm)

For example observed maximum strain at peak load = 0.198 inch

$$= 0.198 \times 25.4$$

$$= 5.02 \text{ mm}$$

$$\text{strain} = 5.02/50.8 = 0.099$$

$$= 9.9\%$$

**Figure B.1 Assumed Reinforcing Steel Stress-Strain Curve
(Yield Strength = 400 MPa)**

**Fig. B.2 Uniaxial Tensile Test Setup for
Tensar UX - 1700 And UX-1700HT
(8 Strands Specimen Shown)**

Figure B.3 Load-Strain Curves For Uniaxial Tensile Tests For Tensar UX-1700HT

Figure B.4 Cyclic Behaviour of Load-Strain Curves for Uni-Axial Tensile Test of Tensar UX-1700HT

Figure B.5 Load-Strain Curve for Uniaxial Tensile Tests For Tensar UX-1700HT (8-Strands Typical Used In The Test Beams)

Figure B.6 Comparison of Load-Strain Curves for Uniaxial Tensile Tests For Tensar UX-1700 And UX-1700HT (8-Strands Typical Used In The Test Beams)

Appendix C

COMPUTER PROGRAM

NON-LINEAR ANALYSIS AND DESIGN [NLAD]

C.1 Salient Features

As the laboratory testing is always limited in quantity and scope, due to the high costs and physical scaling problems, a computer program is necessary for accurate non-linear modeling and analysis of beams which will provide the means of investigating the relevant variables, and the adequacy of design problems. The computer program "Non-Liner Analysis and Design [NLAD]" developed in this study consists of over six-thousand lines of code written mostly in FORTRAN and some c with non-interactive graphics capabilities. NLAD runs only on UNIX Sun Stations because of its compatibility with other available graphic packages.

The salient features of NLAD are to compute moments, curvatures, stress, strains and deflections by using several different methods described in earlier this thesis. NLAD can be used to analyze the effect of strengthening of existing members. In analysis user has several option to chose for the analysis. The program has the capacity to analyze any symmetrical section for any given reinforcement ratio or type or it can compute properties of section for any given load or strain in any of the materials used in this investigation. NLAD is supported with two sub-routines for tension stiffening models and three for deflection computation. NLAD can also compute the SLS deflections which include the creep, shrinkage and duration of loading.

- **Section Geometry:**

The member can have any symmetrical cross-section.

- **Support Conditions:**

At present it is limited to simply-supported and cantilever members.

- **Loading:**

The load pattern can be UDL or two concentrated loads located symmetrically at any place on the beam. The dead load of the member is computed by NLAD.

- **Factors:**

Material and load factors can be specified. The default load and material factor is unity.

- **Reinforcement:**

A member can have any number of layers of reinforcement including compression steel or external reinforcement such as FRP sheets. Type of reinforcement at each layer (can be steel, CFRP, GFRP rebars and sheets or Hybrid) has to be specified in the input file. Many FRP materials which are commercially available are included in the materials library of NLAD.

- **Material Modeling:**

Concrete: Non-linear parabolic stress-strain behavior is assumed. If the concrete strength is over 30 MPa, then NLAD automatically reduces the β value as per code's requirements. The rectangular block of the code is used for ultimate strength.

Reinforcing Steel: For reinforcing steel the stress-strain shape can be specified as bi-linear or tri-linear with some strain hardening. Strain hardening initiation value has to be provided.

CFRP Sheets and Rebars: Linear elastic.

Tensor Grids: as shown in Figures of Appendix-B. The load-strain behavior of Tensor UX1700 and 1700HT are in the form of equations which are obtained from the curve fitting.

- **Analysis:**

Analysis is carried out as described in respective sections of this thesis.

- **Input:**

Section Dimension, length of the member, type of loading - if two point loads at symmetrical location then distance between them, concrete strength, cracking strength (optional), cover, type of reinforcement, number of layers of reinforcement, reinforcement details such as their location, ultimate strength, strain, strain hardening, in case of Tensor only width and effective depth is required, if tension stiffening is required then diameter of reinforcing bar and the tension stiffening factors (optional), tension stiffening method, type of analysis, only ultimate state or complete response including deflections, choose the method of computation of deflections.

- **Output**

⇒ **Data Files:**

Output.M = Consists echo of input parameters, behavior of beams for the full response, including deflections.

Strains = Consists of strain in concrete, compression steel, tension steel (layers 1 to ..n) and CFRP sheet.

⇒ **Plots:**

- * Load-deflection plot based on specified method
- * Moment-curvature plot
- * Moment/Load v/s strain in concrete
- * Moment/Load v/s neutral axis depth
- * Moment/Load v/s curvature distribution along the beam
- * Moment/Load v/s strain in compression steel
- * Moment/Load v/s strain in tension steel
- * Moment/load v/s strain in CFRP sheet

END

2 1 - 1 1 - 9 6

FIN