INFORMATION TO USERS

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

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

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

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

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

UMI[®]

Development of a New FRP Anchor for Externally Bonded CFRP Sheet/Laminate to Beams

By

Ahmed Abo El-Khair B. Mostafa

A Thesis presented to
The Faculty of Graduate Studies and Research
in partial fulfillment of the requirements
for the degree of

Master of Applied Science in Engineering*

Department of Civil and Environmental Engineering
Carleton University
Ottawa, Ontario, Canada

September 2005

© 2005, Ahmed Abo El-Khair B. Mostafa

*** The Master of Applied Science in Civil Engineering is a joint program with the University of Ottawa, administrated by the Ottawa-Carleton Institute for Civil Engineering**

Library and
Archives Canada

Bibliothèque et
Archives Canada

0-494-08381-6

Published Heritage
Branch

Direction du
Patrimoine de l'édition

395 Wellington Street
Ottawa ON K1A 0N4
Canada

395, rue Wellington
Ottawa ON K1A 0N4
Canada

Your file *Votre référence*

ISBN:

Our file *Notre référence*

ISBN:

NOTICE:

The author has granted a non-exclusive license allowing Library and Archives Canada to reproduce, publish, archive, preserve, conserve, communicate to the public by telecommunication or on the Internet, loan, distribute and sell theses worldwide, for commercial or non-commercial purposes, in microform, paper, electronic and/or any other formats.

The author retains copyright ownership and moral rights in this thesis. Neither the thesis nor substantial extracts from it may be printed or otherwise reproduced without the author's permission.

AVIS:

L'auteur a accordé une licence non exclusive permettant à la Bibliothèque et Archives Canada de reproduire, publier, archiver, sauvegarder, conserver, transmettre au public par télécommunication ou par l'Internet, prêter, distribuer et vendre des thèses partout dans le monde, à des fins commerciales ou autres, sur support microforme, papier, électronique et/ou autres formats.

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

In compliance with the Canadian Privacy Act some supporting forms may have been removed from this thesis.

Conformément à la loi canadienne sur la protection de la vie privée, quelques formulaires secondaires ont été enlevés de cette thèse.

While these forms may be included in the document page count, their removal does not represent any loss of content from the thesis.

Bien que ces formulaires aient inclus dans la pagination, il n'y aura aucun contenu manquant.

Canada

The undersigned recommend to the Faculty of Graduate Studies and Research acceptance of the thesis entitled **Development of a New FRP Anchor for Externally Bonded CFRP Sheet/Laminate to Beams**, submitted by Ahmed Abo El-Khair B. Mostafa in partial fulfillment of the requirements for the Degree of Master of Applied Science

Professor A.G.Razaqpur, Thesis Supervisor

Professor A. O. Abd El Halim, Chair of
Department of Civil and Environmental Engineering

Carleton University

Abstract

To delay the delamination of the externally bonded FRP sheet/laminate from the concrete surface and to ensure that after initial delamination the FRP continues to carry some load, a new CFRP mechanical anchor system is developed and tested in this study. The comb-shape anchor is made using an existing CFRP grid known as NEFMAC. The comb teeth are inserted into drilled holes in the concrete while the comb spine rests on the surface of the FRP sheet and is adhesively bonded to it.

A total of nine simply supported reinforced concrete beams are tested in bending to study the effectiveness of this anchor system. Four beams are strengthened with a CFRP sheet, four with a CFRP laminate and one is unstrengthened and used as control beam.

Results of the study reveal that when the anchors are spaced relatively closely, they can delay delamination and after initiation of delamination, the FRP can still carry load, while without the anchor, the FRP begins to separate from the beam as soon as delamination occurs. The developed anchor system shows promise, but further investigation is needed to determine the exact number, size and location of such anchors in order for the bonded FRP to reliably achieve its full design strength.

Acknowledgments

I would like to thank my supervisor, ***Prof. A.G.Razaqpur*** of the Civil and Environmental Department, Carleton University, for his encouragement, patience, and endless support throughout the research steps. I believe this research would not have been completed in this way without his sincere guidance and continuous support. Also, I would like to thank him for the funding provided throughout this research.

Many thanks to, ***Prof. Heng-Aik Khoo*** of the Civil and Environmental Department, Carleton University, for his help with respect to the FE program (ABAQUS)

Also I would like to thank the Civil and Environmental Laboratory team represented by Messrs ***Ken McMartin***, the laboratory manager, ***Pierre Trudel*** the civil technologist and ***Stanly Conley***, Instrumentation and Computer Specialist for their help and guidance during the experimental work.

I would like to thank the following organizations for their material/financial assistance: the Ontario centre of Excellence, Manufacturing and Material Ontario (MMO) and AUTOCON composites of Toronto for funding this project, Silca Canada for donating the CFRP laminate system and Degussa Inc. for donating the CFRP sheets used in the current study.

I would like to give my colleagues in the Civil and Environmental Department warm thanks for their support and help whenever asked for.

Words are not enough to thank my family, especially my parents, brother and sister and my uncle for their wonderful encouragement, support, guidance and spiritual fellowship.

Table of Contents

Abstract.....	iii
Acknowledgments	iv
Table of Contents	vi
List of Tables	x
List of Figures.....	xii
Chapter 1: Introduction	1
1.1 General.....	1
1.2 Objectives and Scope of the Study.....	2
Chapter 2: Literature Review.....	4
2.1 General.....	4
2.2 Previous Work.....	5
Chapter 3: Experimental Program	19
3.1 General.....	19
3.2 Test Specimens	19
3.3 Material Properties.....	20
3.3.1 Concrete	20
3.3.2 Longitudinal Steel Reinforcement.....	22
3.3.3 Transverse Steel Reinforcement.....	25
3.3.4 CFRP Laminate/Sheet	25
3.3.5 Epoxy.....	26

3.4 The CFRP Anchors.....	29
3.4.1 Stud Anchors	29
3.4.2 NEFMAC Anchors.....	33
3.5 Beams Design.....	35
3.5.1 Beams Geometry and Reinforcement	36
3.5.2 Design of Strengthened Test Beams	38
3.5.3 Details of FRP Reinforcement	41
3.5.4 Anchor Details.....	43
3.5.5 External Shear Reinforcement	46
3.6 Fabrication of the Tested Beams	48
3.7 Test Equipments and Instrumentations.....	58
3.8 Test Setup	64
Chapter 4: Experimental Results	66
4.1 General.....	66
4.2 Beams Behaviour and Observations.....	66
4.2.1 Control Beam (CB)	66
4.2.2 Beam LN90	69
4.2.3 Beam LN90B.....	73
4.2.4 Beam LA90C	80
4.2.5 Beam LA90	86
4.2.6 Beam SN90.....	93
4.2.7 Beam SA90.....	98
4.2.8 Beam LN901	103

4.2.9 Beam LA901	108
4.2.10 Beam SN901.....	112
4.2.11 Beam SA901.....	116
4.3 Summary.....	120
 Chapter 5: Analysis and Discussion of Test Result.....	122
5.1 General.....	122
5.2 Analysis of Test Results.....	122
5.2.1 Ultimate Flexural Capacity	125
5.2.2 Delamination Load.....	128
5.2.3 Load Drop after Delamination	129
5.2.4 Load - Deflection Behaviour	131
5.2.5 Strain Profiles.....	142
5.3 Interfacial Stresses.....	153
5.3.1 Interfacial Shear Stress Calculations.....	154
5.3.2 Interfacial Shear Stress Diagrams.....	155
5.4 Further Analysis of Failure Load.....	170
 Chapter 6: Summary, Conclusions, and Recommendations for Future Work	
.....	178
6.1 Summary.....	178
6.2 Conclusions.....	179
6.3 Recommendations for Future Work.....	182
References.....	184

Appendix A: Design of Test Beams.....	189
A1: Steel Reinforced Concrete Beam I	189
A2: Steel Reinforced Concrete Beam II	192
A3: Beam with CFRP (Three Laminates).....	194
A4: Beam with CFRP (Two Laminates).....	195
A5: Beam with CFRP (One Laminate)	196
A6: Beam with CFRP Sheet (SN90).....	198
A7: Beam with CFRP Sheet (SN901).....	199
A8: Choosing the Unsheeted Length.....	201

List of Tables

Table 3-1: Concrete cylinder test results at 28-days.....	21
Table 3-2: Concrete cylinder test results at age 48-days, the day of testing the first beam.....	21
Table 3-3: Summary of longitudinal tension steel reinforcement properties.....	25
Table 3-4: CFRP composite material properties provided by the manufacturers... 	26
Table 3-5: Sikadur 30 epoxy resin properties used for CFRP laminate.....	27
Table 3-6: Wabo MBrace primer and saturant properties used for the CFRP sheet	28
Table 3-7: Details of beams reinforcement, CFRP arrangement and length	40
Table 3-8: Theoretical flexural and shear capacity	40
Table 4-1: Summary of the delamination, drop and failure loads for the tested beams.....	121
Table 5-1: Summary of the delamination and failure moments of the tested beams	124
Table 5-2: Summery of delamination, drop and failure loads for the tested beams	128
Table 5-3: Magnitude of strain values (micro-strain) plotted in Fig.5-12	144
Table 5-4: Magnitude of strain values (micro-strain) plotted in Fig.5-13	145
Table 5-5: Magnitude of strain values (micro-strain) plotted in Fig.5-14	146
Table 5-6: Magnitude of strain values (micro-strain) plotted in Fig.5-15	147
Table 5-7: Magnitude of strain values (micro-strain) plotted in Fig.5-16	148
Table 5-8: Magnitude of strain values (micro-strain) plotted in Fig.5-17	149

Table 5-9: Magnitude of strain values (micro-strain) plotted in Fig.5-18	150
Table 5-10: Magnitude of strain values (micro-strain) plotted in Fig.5-19	151
Table 5-11: Magnitude of strain values (micro-strain) plotted in Fig.5-20	152
Table 5-12: Full details of beams used in analysing failure load of FRP strengthened beams.....	172
Table 5-12: Full details of beams used in analysing failure load of FRP strengthened beams (cont.).....	173
Table 5-13: Experimental versus theoretical predicted failure load of FRP strengthened beams	174

List of Figures

Figure 3-1: Tensile stress-strain relationship of steel bar-1 ($\phi=20\text{mm}$)	23
Figure 3-2: Tensile stress-strain relationship of steel bar-2 ($\phi=20\text{mm}$)	23
Figure 3-3: Tensile stress-strain relationship of steel bar-3 ($\phi=20\text{mm}$)	24
Figure 3-4: Tensile stress-strain relationship of steel bar-4 ($\phi=20\text{mm}$)	24
Figure 3-5: Anchors type-I manufactured from carbon fiber tow	30
Figure 3-6: Anchor type-II manufactured using a mould with large end circles.....	30
Figure 3-7: Anchor mould geometry and dimensions	31
Figure 3-8: Anchor mould used to manufacture stud anchors.....	32
Figure 3-9: Another view for the anchor mould	32
Figure 3-10: Typical NEFMAC CFRP grid	33
Figure 3-11: Extraction of π-shape anchor from CFRP grid	34
Figure 3-12: NEFMAC CFRP anchor type A	34
Figure 3-13: NEFMAC CFRP anchor type B	35
Figure 3-14: Details of steel reinforcement in test beams.....	37
Figure 3-15: General details for the anchor and the external FRP [Beam shown is upside down].....	42
Figure 3-16: Enlarged view of segment A-A showing the anchor and the external FRP sheet/laminate	42
Figure 3-17: Plan view of CFRP laminate/sheet and anchor arrangement for beams LN90, LA90, SN90 and SA90.....	44

Figure 3-18: Plan view of CFRP laminate/sheet and anchor arrangement for beams LN901, LA901, SN901 and SA901.	45
Figure 3-19: Typical external shear reinforcement used in the test beams.....	47
Figure 3-20: Formwork and the reinforcement cages inside them	48
Figure 3-21: Formwork for the nine beams	49
Figure 3-22: Concrete casting operation.....	50
Figure 3-23: Concrete vibration	50
Figure 3-24: Finishing of concrete surface	51
Figure 3-25: 16 standard concrete cylinders	51
Figure 3-26: Beams LN90 and LA90 after application of the epoxy.....	53
Figure 3-27: Beams LN90 and LA90 after the laminate installation	53
Figure 3-28: Beam after Wabo MBrace primer application	55
Figure 3-29: Wabo MBrace saturant layer and the sheet installation.....	55
Figure 3-30: Anchor arrangement in beam LA90	56
Figure 3-31: Anchor arrangement in beam LA901	57
Figure 3-32: Anchor arrangement in beam SA90.....	57
Figure 3-33: Anchor arrangement in beam SA901.....	58
Figure 3-34: Typical LVDT locations on test beams	59
Figure 3-35: Position of LVDT's, Supports and Actuator	60
Figure 3-36: Typical strain gauges position on the longitudinal tension steel	60
Figure 3-37: Arrangement and spacing of the strain gauges on CFRP laminate/sheet of beams LN90, LA90, SN90 and SA90.....	61

Figure 3-38: The arrangement and spacing of the strain gauges on CFRP	
laminate/sheet in beams LN901, LA901, SN901 and SA901	62
Figure 3-39: Arrangement of the strain gauges on the test beams.....	63
Figure 3-40: Location of the concrete strain gauge for beams CB, LN90, LA90, SN90	
 and SA90.	64
Figure 3-41: Location of the concrete strain gauge for beams LN901, LA901, SN901	
 and SA901.	64
Figure 4-1: Shear cracks in and compression failure of control beam CB.....	67
Figure 4-2: Load mid-span deflection of Beam CB	68
Figure 4-3: Deflected shapes at various load levels of Beam CB.....	69
Figure 4-4: Beam LN90 under four point bending	70
Figure 4-5: Shear cracks in Beam LN90 just before stopping the test	71
Figure 4-6: Load mid-span deflection of Beam LN90	71
Figure 4-7: Deflected shapes at various load levels of Beam LN90.....	72
Figure 4-8: Strain Profiles in CFRP of Beam LN90 at various load levels	73
Figure 4-9: Beam LN90B after the modifications made to Beam LN90	75
Figure 4-10: Longitudinal and transverse cut made for Beam LN90B	75
Figure 4-11: Failure in the concrete surface caused delamination in Beam LN90B	77
Figure 4-12: Load mid-span deflection of Beam LN90B.....	77
Figure 4-13: Strain profiles in CFRP laminate along Beam LN90B at various load	
 levels	79
Figure 4-14: Deflected shapes at various load levels of Beam LN90B	80
Figure 4-15: Beam LN90C after re-gluing and anchors installation	81

Figure 4-16: Full plastic behaviour of Beam LN90C at load P=362 kN	82
Figure 4-17: Load mid-span deflection of Beam LA90C	83
Figure 4-18: Deflected shapes of Beam LA90C at various load levels.....	84
Figure 4-19: Strain variation in CFRP along Beam LN90C at various load levels ..	84
Figure 4-20: FRP end slippage in Beam LA90C at failure	85
Figure 4-21: The anchor preventing the laminate from complete delamination along the length of Beam LA90C	86
Figure 4-22: Anchor arrangement and the cuts in laminate for Beam LA90	87
Figure 4-23: Beam LA90 at the commencement of the test	87
Figure 4-24: Load mid-span deflection of Beam LA90	89
Figure 4-25: After failure and the removal of the load	89
Figure 4-26: Slippage happened in Beam LA90.....	90
Figure 4-27: Deflected shapes at various load levels of Beam LA90.....	91
Figure 4-28: Strain profiles of CFRP laminate in Beam LA90 at various load levels	92
Figure 4-29: Strain profiles close to the failure load of Beam LA90.....	92
Figure 4-30: Load deflection curve of Beam SN90 at 1960 mm from the left support	94
Figure 4-31: Fully plastic behaviour of Beam SN90	95
Figure 4-32: Failure in concrete surface caused the delamination in Beam SN90 ...	95
Figure 4-33: Another view of delamination in Beam SN90.....	96
Figure 4-34: Deflected shapes at various load levels of Beam SN90	97
Figure 4-35: Strain profiles of CFRP in Beam SN90 at various load levels.....	97

Figure 4-36: Setup for Beam SA90.....	98
Figure 4-37: Anchor arrangement, longitudinal and transverse cut made to the sheet for Beam SA90.....	99
Figure 4-38: Load mid-span deflection of Beam SA90.....	100
Figure 4-39: Fully plastic behaviour of Beam SA90 at failure	101
Figure 4-40: sheet end slippage in Beam SA90	101
Figure 4-41: Deflected shapes at various load levels of Beam SA90	102
Figure 4-42: Strain profiles of CFRP in Beam SA90 at different load levels	103
Figure 4-43: Fully plastic behaviour of Beam LN901.....	104
Figure 4-44: Concrete failure in the interface between the laminate and the concrete interface of Beam LN901.....	104
Figure 4-45: Load deflection curve at 1170mm from the left support of Beam LN901	105
Figure 4-46: Deflected shapes of Beam LN901 at various load levels.....	107
Figure 4-47: Strain profiles of CFRP in Beam LN901 at various load levels.....	107
Figure 4-48: Complete plastic behaviour of Beam LA901	109
Figure 4-49: Tension steel ruptured in Beam LA901	109
Figure 4-50: Slippage happened in Beam LA901.....	110
Figure 4-51: Load mid-span deflection of Beam LA901	111
Figure 4-52: Deflected shapes of Beam LA901 at various load levels.....	111
Figure 4-53: Strain profiles of CFRP in Beam LA901 at various load levels.....	112
Figure 4-54: Fully plastic behaviour of Beam SN901	113
Figure 4-55: Load mid-span deflection of Beam SN901.....	114

Figure 4-56: Deflected shapes of Beam SN901 at various load levels	115
Figure 4-57: Strain profiles of CFRP in Beam SN901 at various load levels	115
Figure 4-58: Load mid-span deflection of Beam SA901	117
Figure 4-59: Slippage in Beam SA901	118
Figure 4-60: Deflected shapes of Beam SA901 at various load levels	119
Figure 4-61: Strain profiles of CFRP in Beam SA901 at various load levels	119
Figure 5-1: Comparison of the experimental and theoretical load mid-span deflection curves of control Beam CB	132
Figure 5-2: Comparison of the experimental and theoretical load mid-span deflection curves of Beam LN90	133
Figure 5-3: Comparison of the experimental and theoretical load mid-span deflection curves of Beam LN90B	134
Figure 5-4: Comparison of the experimental and theoretical load mid-span deflection curves of Beam LA90C	135
Figure 5-5: Comparison of the experimental and theoretical load mid-span deflection curves of Beam LA90	136
Figure 5-6: Comparison of the experimental and theoretical load mid-span deflection curves of Beam SN90	137
Figure 5-7: Comparison of the experimental and theoretical load mid-span deflection curves of Beam SA90	138
Figure 5-8: Comparison of the experimental and theoretical load mid-span deflection curves of Beam LN901 [Deflection at 1770mm from left support].	139

Figure 5-9: Comparison of the experimental and theoretical load mid-span	
deflection curves of Beam LA901	140
Figure 5-10: Comparison of the experimental and theoretical load mid-span	
deflection curves of Beam SN901.....	141
Figure 5-11: Comparison of the experimental and theoretical load mid-span	
deflection curves of Beam SA901.....	142
Figure 5-12: Stain profile of Beam CB through its depth at different load levels ..	143
Figure 5-13: Stain profile of Beam LN90 through its depth at different load levels	
.....	144
Figure 5-14: Stain profile of Beam LN90B through its depth at different load levels	
.....	145
Figure 5-15: Stain profile of Beam SA90 through its depth at different load levels	
.....	146
Figure 5-16: Stain profile of Beam LA90 through its depth at different load levels	
.....	147
Figure 5-17: Stain profile of Beam LA901 through its depth at different load levels	
.....	149
Figure 5-18: Stain profile of Beam LN901 through its depth at different load levels	
.....	150
Figure 5-19: Stain profile of Beam SN901 through its depth at different load levels	
.....	151
Figure 5-20: Stain profile of Beam SA901 through its depth at different load levels	
.....	152

Figure 5-21: Normal and shear stresses at the FRP-concrete interface (El-Mihilmy and Tedesco 2001).....	153
Figure 5-22: Schematic variation of normal and shear stresses distribution at the FRP-concrete interface (El- Mihilmy and Tedesco 2001)	154
Figure 5-23: Free body diagram of an infinitesimal element of FRP.....	154
Figure 5-24: Shear stress along the CFRP laminate in Beam LN90	156
Figure 5-25: Shear stress along the CFRP laminate in Beam LN90 at failure	156
Figure 5-26: Shear stress along the CFRP laminate in Beam LN90B	157
Figure 5-27: Shear stress along the CFRP laminate in Beam LN90B at failure.....	158
Figure 5-28: Shear stress along the CFRP laminate in Beam LA90	159
Figure 5-29: Shear stress along the CFRP laminate in Beam LA90 at failure	159
Figure 5-30: Shear stress along the CFRP laminate in Beam SN90.....	161
Figure 5-31: Shear stress along the CFRP laminate in Beam SN90 at failure.....	161
Figure 5-32: Shear stress along the CFRP laminate in Beam SA90.....	162
Figure 5-33: Shear stress along the CFRP laminate in Beam SA90 at failure.....	163
Figure 5-34: Shear stress along the CFRP laminate in Beam LN901	164
Figure 5-35: Shear stress along the CFRP laminate in Beam LN901 at failure	164
Figure 5-36: Shear stress along the CFRP laminate in Beam LA901	165
Figure 5-37: Shear stress along the CFRP laminate in Beam LA901 at failure	166
Figure 5-38: Shear stress along the CFRP laminate in Beam SN901.....	167
Figure 5-39: Shear stress along the CFRP laminate in Beam SN901 at failure.....	167
Figure 5-40: Shear stress along the CFRP laminate in Beam SA901.....	168
Figure 5-41: Shear stress along the CFRP laminate in Beam SA901 at failure.....	169

Figure 5-42: Predicted/experimental failure load ratios according to (El-Mihilmy and Tedesco model).....	175
Figure 5-43: Comparison of the predicted failure loads (kN) according to El-Mihilmy and Tedesco model with their experimental values	176
Figure 5-44: Predict anchorage failure loads (kN) according to Smith and Teng model versus the experimental results	176

Chapter 1:

Introduction

1.1 General

Over the last decade a new technique has evolved for the rapid repair and strengthening of RC structures. This technique involves the use of fibre reinforced polymers (FRP), with the fibres being principally carbon or glass. The numerous advantages of using FRP include ease of application, high ultimate strength, which greatly exceeds that of steel rebars, and lack of corrosion. The high strength and stiffness of FRP make it possible to use it as reinforcement for concrete members. Unlike steel rebars, FRP materials, particularly carbon FRP (CFRP), are unaffected by electrochemical deterioration and they can resist acids, salts, and similar aggressive chemicals under a wide range of temperatures and other severe conditions. The specific gravity of the FRP is generally one-fourth that of steel which allows reduction in the transportation costs and easier handling on construction site.

When FRP is used for post-strengthening and repair of deficient reinforced concrete members, it is generally in the form of sheets/laminates adhesively bonded to the concrete surface. The adhesives are generally epoxy-based while the sheets/laminates comprise either carbon or glass fibres. The effectiveness of this retrofit technique depends mainly on the stress transfer performance and strength of the FRP-concrete interface. Premature failure at the interface causes delamination at the epoxy- concrete, or

epoxy-FRP interface, and in both cases once delamination initiates, the FRP separates from the concrete and becomes ineffective. To prevent, or rather delay, delamination, currently FRP U-shape straps, akin to U-stirrups in conventional reinforced concrete, are glued to the tension face and sides of beams. Such a system works relatively well, provided the surface concrete is undamaged and is able to transfer the resulting shear and normal stresses.

For members where the surface concrete is weathered and/or damaged, this method may not be feasible because the interface between the concrete and the epoxy will fail at relatively low loads. In the latter case, it may be more appropriate to use mechanical anchors which can be embedded deep inside the beam in the more sound concrete. Steel stud anchors have already been effectively used by some researchers for this purpose, but the disadvantage of steel studs is their vulnerability to corrosion in severe environments. Therefore, an alternative is to develop FRP mechanical anchors, which would be immune from corrosion and would be compatible with the FRP sheet/laminate.

1.2 Objectives and Scope of the Study

In the light of the above introduction, the objectives of this study are:

1. To develop a new FRP mechanical anchor for anchoring FRP sheets/laminates onto concrete.
2. To investigate, via laboratory tests, the effectiveness of the new anchor system.

The effectiveness shall be investigated by applying it to beams externally retrofitted with adhesively bonded sheets/laminates for increased flexural capacity.

3. Provided the system is demonstrated to be effective in delaying delamination, to make suggestions/recommendations for further improving the developed system.
4. To perform some preliminary theoretical analysis of the strengthened beams in order to find their expected ultimate strength and to investigate the distribution of the stresses at the epoxy-concrete interface.

The scope of this study is limited to reinforced concrete beams strengthened externally with epoxy-bonded CFRP sheets and laminates for the purpose of increased flexural capacity. The beams would be under-reinforced before application of FRP, but after applying the FRP, they would be over-reinforced. However, the steel would still yield before the beams reach their ultimate strength. Also, only one type of anchor would be tested and due to time constraint, only the effect of a limited number of parameters would be investigated. The studied parameter includes presence of anchor, type of FRP used for strengthening, i.e. CFRP sheet versus laminate, and number of anchors. It is important to mention that this is the first study on such an anchor system; therefore, the principle objective is a proof of concept.

Chapter 2:

Literature Review

2.1 General

In recent years, the use of FRP as a material for strengthening and rehabilitation of RC structures has been rapidly growing. One of its major applications in RC structures is as external reinforcement, bonded to the surface of structural elements such as beams, slabs, and columns. Different factors must be considered when strengthening structural elements with FRP, such as bond strength, relative increase in flexural/shear strengths and ductility requirements. In addition, the ability of the FRP-concrete interface to resist the induced stresses, without premature failure, is key to the successful performance of this strengthening technique.

One of the key requirements for the effectiveness of FRP in flexural and shear strengthening is the maintenance of bond between the FRP sheet/laminate and the concrete surface. Since the FRP is bonded to the concrete surface by high strength adhesive, the governing parameter for maintaining full bond is the strength of the interface between the concrete and the adhesive. Failure at the interface normally leads to delamination and renders the FRP ineffective. The problem of delamination, its cause and procedures for delaying/preventing it have been studied by a number of researchers over the past decade. The following literature review will briefly describe the nature of these

investigations and their results/conclusions. The objective of the review is mainly to demonstrate the need for the present research.

2.2 Previous Work

Extensive research has been performed by a large number of investigators on the performance and behaviour of reinforced concrete beams strengthened with FRP sheets/laminates (e.g. Ritchie et al.1991, Saadatmansesh and Ehsani 1991, Triantafillou and Plevris 1992, Chajes et al. 1994, Sharif et al.1994, Meier 1995, Heffernan and Erki 1996, Takeda et al. 1996, Arduini and Nanni 1997, Garden et al.1998, Varastehpour and Hamelin 1997, Ross et al. 1999, Juvandes et al. 1998, Taljsten 1999, Assih et al. 2001, Bonfiglioli et al.2003). In all of these investigations either carbon or glass sheets /laminates were adhesively bonded to the soffit of simply supported beams loaded either in three point or four point bending until the beam failed either due to : (a) normal flexural failure due to crushing of concrete in the compression zone, (b) flexural tension failure due to rupture of FRP , (c) failure initiated by separation of the concrete cover to the tension steel reinforcement, (d) delamination of FRP at the concrete adhesive interface near the section of maximum moment, and (e) delamination of the FRP at the plates ends due to high interfacial shear and normal stresses. Whereas the FRP rupture and concrete crushing failure modes can be predicted using conventional methods of reinforced concrete analysis, the delamination and cover separation cannot be accurately predicted by the available methods. Premature failure due to delamination makes the task of ascertaining the increase in a RC beam flexure capacity via externally bonded FRP plates rather difficult.

To render this system of repair reliable and to ensure that the expected capacity increase is realized, two essential requirements must be satisfied. First, effective methods for preventing delamination must be developed and secondly, accurate methods of analysis should be developed for calculating the distribution and magnitude of the stresses at the concrete-FRP interface, and for determining the concrete strength under the relevant stress combinations. Knowing the preceding parameters, the delamination load can be determined by comparing the maximum value of the interfacial stresses to the concrete strength using a relevant failure criterion.

To prevent premature delamination, basically two methods have been used in the past. Swamy et al. (1987) used a steel plate near the ends of the FRP and sandwiched the FRP between the beam soffit and the steel plate, with the steel plate being anchored into the concrete by steel bolts. They noted that this anchor system allowed for a more ductile failure of the strengthened beam albeit failure was initiated by debonding of the FRP sheet. However, due to the end anchors, the FRP continued to resist higher load despite delamination. It was not stated by these authors whether slippage occurred at the FRP-concrete interface after initial debonding.

A more popular method of anchorage has been the use of FRP U-jackets Takahashi et al. (1997). The U-jackets are made of FRP sheets and applied to the bottom and sides of the beam in the anchorage zone. According Takahashi et al., the presence of U-jackets increases the FRP plates resistance to peeling, and by appropriate selection of the anchor system, the failure mode can be shifted to the rupture of the FRP rather than plate separation. They also noted that the application of the U-jacket did not increase the ultimate moment capacity of the beam compared to a similarly strengthened beam

without U-jacket, but it increased its ability to undergo up to 50% more deformation before failure, which confirms the earlier findings of Swamy et al. (1987) with respect to the benefit of steel plate anchorage. However, they did not provide any guidance about how to design such an anchor system.

Some design guidelines and standards such as CSA 5806-02 (2002) deal with the problem of premature delamination by limiting the ultimate strain in the FRP sheet/laminate. Oller et al. (2001) noted that the recommended longitudinal strain limits of 0.6-0.8% (Sika 1997, Rostasy 1998, Seim et al. 1999, Vasquez 1999, S&P 2000) are unconservative and that none of their tested beams could exceed 0.5 % strain before failure. Therefore, they recommended the use of externally applied anchors to delay the debonding of the plate. They used equally spaced U-jackets throughout the length of the beam. It was reported that in the case of the beam with U-jacket, despite delamination and some slippage, the beam failed at 34.6% higher load than the control beam without strengthening and at 9% higher load than similarly strengthened beams without anchors.

Duthinh and Starnes (2001) tested seven concrete beams using two different end anchorage systems. In one case steel plates were bolted on to the ends of the FRP plate; in another case FRP U-jackets were used. They noted that six layers of wrapping were almost as effective as the steel clamp anchor. The latter allowed the FRP laminate to reach a maximum strain of 1.14 % (60% of its rupture strain). Their conclusion was that both anchoring systems were effective. An alternative method of anchoring FRP laminate to concrete beams was proposed by Lamanna et al. (2001).

Whereas in all other strengthening methods an adhesive was always used to bond the FRP plate to the concrete surface, these investigators used only mechanical fasteners

to attach CFRP strips to the bottom of the beam. A commercially available power-actuated fastener system was used to drive the fasteners into the concrete. The small size beams in which one or two rows of fasteners were used along the beam soffit achieved from 65 to 70 % of the increase in capacity of similarly strengthened beams using only epoxy adhesive. They noted that the failure mode of the beams with the fastener system was more ductile than those with epoxy adhesive. Once again the advantage of anchorage being the ability of the strengthened beam to undergo large deformations before failure.

Other methods of anchorage, including transverse strips bonded to the beam soffit and to the FRP plate, as well as anchoring of FRP at its ends in pre-drilled holes Meier (1998) are discussed by Poulsen et al. (2001). These methods are either ineffective or too complicated.

To date there is no agreement among researchers as to which anchor system should be exclusively used. Although U-jackets are easy to apply, there are no design methods available for such a system. On the other hand, steel plate anchor systems are complicated, costly and susceptible to corrosion and deterioration in severe environments. Hence, the search for both an effective anchor system and a practical design method continues.

One of the keys to developing an effective anchor system and a practical design method is to achieve greater understanding of the factors that affect delamination in externally strengthened concrete beams. This means that the distribution and magnitude of the stresses at the FRP plate-concrete interface must be relatively accurately known at various levels of load. Although linear elastic analysis, using various assumptions and theoretical rigour (Swamy 1987, Roberts 1989, Roberts and Haji-Kazemi 1989, Oehler's

and Moran 1990, Oehler's 1992, Ziraba 1993, Ziraba et al. 1994, Zhang et al. 1995, Quantrill et al. 1996, Raoof and Zhang 1997, Jansze 1997, Varastehpour and Hamelin 1997, Taljsten 1997, Malek and Saadatmansesh 1998, Wang and Ling 1998, Rabinovitch and Frostig 1999, Tumialan et al. 1999, Raoof and Hassaanen 2000, El-Mihilmy and Tedesco 2001, Smith and Teng 2001) can be used to predict the longitudinal shear and normal (peeling) stresses distribution at the FRP-concrete interface, these solutions are not always able to provide an accurate picture of the actual stresses due to the fact that cracking and nonlinearities, as well as stress concentration at cracks, render the assumption of linear elasticity invalid. Smith and Teng (2001) review all the major available methods for predicting the delamination load of RC beams externally strengthened with FRP plates. Their conclusion is that nearly all the methods fail to predict the experimentally observed strength of tested beams. They discuss in detail the strength and weakness of each of the available methods and subsequently propose a method based on a modified form of Oehlers and Moran method (1990). They contend that their proposed method yields a more consistent prediction of the delamination load and is more accurate than the other available methods. In fact the method provides a reasonable lower bound of the experimentally observed failure loads, but is quite conservative in many cases.

The Oehler and Moran (1990) method is based on the so-called shear capacity model. In this model the debonding failure load is related to the shear strength of the concrete with no or partial contribution of the steel shear reinforcement. The ultimate debonding shear and moment at the plate ends are given by $V_{db,s}$ and $M_{db,f}$ as follows:

$$V_{db,s} = V_c = [1.4 - (d/2000)]b_c d [\rho_s f_c']^{1/3} \quad (2-1)$$

and,

$$M_{dbf} = \frac{E_c I_{tr,c} f_{ct}}{0.901 E_{frp} t_{frp}} \quad (2-2)$$

where $\rho_s = \frac{A_s}{b_c d}$ = the ratio of tension steel reinforcement, A_s is the area of tension steel reinforcement, b_c = the section width and d = the effective depth of the section. E_c and E_{frp} = the moduli of elasticity of the concrete and the FRP plate, respectively, $I_{tr,c}$ = the second moment of area of the plated cracked section transformed to concrete, f_{ct} = the cylinder splitting tensile strength of concrete and t_{frp} = the plate thickness. The above expression for V_c is given by the Australian concrete code and one of the requirement is that $[1.4-(d/2000)] \geq 1.1$. Eq.(2-1) applies to FRP sheets/laminates (plates) terminating near simple supports while Eq.(2-2) applies to plates terminating in a constant moment region. For the general case where there is shear and moment at the plate end, the following interaction equation was suggested based on experimental work:

$$\frac{M_{db,end}}{M_{db,f}} + \frac{V_{db,end}}{V_{db,s}} \leq 1.17$$

and

$$M_{db,end} \leq M_{db,f} \quad , \quad V_{db,end} \leq V_{db,s} \quad (2-3)$$

Eq. (2-3) can be re-arranged to determine the debonding shear, $V_{db,end}$, at the plate end under combined shear- moment condition:

$$V_{db,end} = \left[\frac{1.17}{\frac{a}{M_{db,f}} + \frac{1}{V_{db,s}}} \right] \quad (2-4)$$

$$V_{db,end} \leq M_{db,f} \quad , \quad V_{db,end} \leq V_{db,s}$$

where a = the unsheathed length, i.e. from the simple support to the plate end. If f_{ct} is not determined from tests, it can be taken as $0.5\sqrt{f'_c}$ where f'_c is the concrete cylinder compressive strength. Note that the terms in the denominators in Eq. (2-3) are calculated using Eqs. (2-1) and (2-2), while $M_{db,end}$ is the moment acting concurrently with $V_{db,end}$.

In applying this method to beams strengthened with FRP plates, Smith and Teng (2001) noted that the shear force corresponding to delamination of FRP plates can be calculated with the following minor modification:

$$V_{db,FRP} = \eta V_{db,s} \quad (2-5)$$

where $\eta = 1.5$ and $V_{db,s}$ is given by Eq.(2-1). This provides a 95.6% lower bound prediction to the cover separation data while when $\eta = 1.4$, it provides a 94% lower bound prediction. In practical design, Smith and Teng recommended $\eta = 1.4$ to design against all forms of plate end debonding failures.

Jones et al. (1988) suggested that the shear stress at the plate-concrete interface can be calculated using conventional beam theory and elastic analysis. They noted that multiplying these elastic shear stresses by a factor of 2.0 yields a reasonable estimate of the debonding shear at the plate ends. El-Mihilmy and Tedesco (2001) compared experimental results from several research works with the different methods suggested by Jones et al. (1988), Roberts (1989), Ziraba (1993), Quantrill et al. (1996), Taljsten (1997)

and Malek et al. (1998), and they concluded that none of these methods yields accurate results.

To obtain a more accurate method, El-Mihilmy and Tedesco (2001) modified the linear elastic method of Roberts (1989) to determine the longitudinal normal stress, the transverse peeling stress and the longitudinal shear stress near the ends of the plate. Then using a failure criterion for concrete under combined biaxial stresses, they determined the magnitude of the three stress components that would cause delamination. The method is described in detail below because it will be used later to check its applicability to the beams tested in the present investigation. The maximum interfacial shear stress at the plate end, $\tau_{\max}$, can be calculated from the following expression:

$$\tau_{\max} = V_o \frac{t_f(d_f - c)}{I} + \left[\frac{G_a b_a t_f^2}{E_f b_f t_f t_a} \right]^{0.5} \frac{M_o}{I} (d_f - c) \quad (2-6)$$

or

$$\tau_{\max} = V_o \frac{t_f(d_f - c)}{I} + \left[\frac{G_a b_a}{E_f t_a} \right]^{0.5} \frac{M_o}{I} (d_f - c) \quad (2-7)$$

where

$\tau_{\max}$ = maximum shear stress at plate curtailment location, MPa

V_o = Shear force at the plate curtailment location, N

M_o = bending moment at the plate curtailment location in N-mm

t_f = thickness of the plate, mm

I = cracked transferred moment of inertia of beam cross section in terms of FRP plate, mm^4

d_f = effective depth of the FRP plate, mm

c = the depth of the neutral axis for cracked section, mm

G_a = the adhesive shear modules, MPa, given by $G_a = \frac{E_a}{2(1 + \nu_a)}$, the assumed value of

the Poisson's ratio $\nu_a = 0.3$, therefore, $G_a = \frac{E_a}{2.6}$

b_a = width of the adhesive layer, mm

E_f = modules of elasticity of the FRP plate, MPa

b_f = width of the FRP plate, mm

t_a = thickness of the adhesive layer, mm

The shear and normal stress at the plate level, based on conventional bending theory, can be expressed, respectively, as

$$\tau = \frac{V_o Q_f}{I b_f} = \frac{V_o b_f t_f (d_f - c)}{I b_f} = \frac{V_o t_f (d_f - c)}{I} \quad (2-8)$$

$$\sigma_x = \frac{M_o}{I} (d_f - c) \quad (2-9)$$

where Q_f = the first moment of area. Let α_f defined as follows, denote the so-called shear factor:

$$\alpha_f = \sqrt{\frac{G_a t_f}{E_f t_a}} \quad (2-10)$$

or alternatively,

$$\alpha_f = \sqrt{\frac{E_a t_f}{1.5(2.6)E_f}} = c_1 \sqrt{\frac{E_a t_f}{E_f}} \quad (2-11)$$

where c_1 is an empirical constant determined from experimental tests to account for the nonlinearities that exist in the anchorage zone. Substituting Eq. (2-8), (2-9), and (2-10) into Eq. (2-7) yields

$$\tau_{\max} = \tau + \alpha_f \sigma_x \quad (2-12)$$

The expression for maximum peeling stress for the plate is given by

$$\sigma_{z,\max} = \tau_{\max} \left[\frac{t_f^4 k_n}{4E_f I_f} \right]^{0.25} = \xi_f \tau_{\max} \quad (2-13)$$

where ξ_f is defined by

$$\xi_f = \left[\frac{t_n^4 k_n}{4E_f I_f} \right]^{0.25} \quad (2-14)$$

Noting that $I_f = \frac{b f t_f^3}{12}$ and $k_n = \frac{E_a b a}{t_a}$ and substituting these values into Eq. (2-14)

yields

$$\xi_f = \left[\frac{12 t_f^4 k_n}{4 E_f b f t_f^3} \right]^{0.25} = \left[\frac{3 t_f^4 E_a b a}{E_f b f t_f^3 t_a} \right]^{0.25} = \left[\frac{3 E_a t_f}{E_f t_a} \right]^{0.25} = c_2 \sqrt{\alpha_f} \quad (2-15)$$

where c_2 is a constant determined from experimental results. They suggested $c_1 = 0.28$, $c_2 = 1.3$. Eq.(2-13) now becomes

$$\sigma_{z,\max} = 1.3\sqrt{\alpha_f \tau_{\max}} \quad (2-16)$$

in which

$$\alpha_f = 0.28\sqrt{\frac{E_a t_f}{E_f}} \quad (2-17)$$

and

$$\xi_f = 1.3\sqrt{\alpha_f} \quad (2-18)$$

Roberts (1989) replaced the bending moments M_o at the plate curtailment with M^* . This modification yielded reasonable results for plates terminated at a short distance from the support; however, it is very conservative for beams with plates terminated at large distance from the support. Therefore, a modified expression based on a statistical analysis was presented to calculate M^* from the following expression:

$$M^* = M_o \psi_f^2 \quad (2-19)$$

in which

$$\psi_f = 1.35 - 12.5 \frac{L_f}{L}, \frac{L_f}{L} \leq 0.1 \quad (2-20)$$

where L_f and L are the distances of the plate curtailment from the support and the total span length, respectively. Thus the M_o is replaced by M^* in Eq.(2-9), yielding the expression

$$\sigma_x = \frac{M^*}{I} (d_f - c) = \frac{M_o \psi_f^2}{I} (d_f - c) \quad (2-21)$$

The failure criterion for tension-tension state of stresses was applied provided both principal stresses are tensile and assuming the major principal tensile stress σ_1 to be greater than the ultimate tensile strength of concrete f_{tu} defined as

$$f_{tu} = f_t = k\sqrt{f'_c} \quad (2-22)$$

where f_t is the splitting tensile strength of concrete, and the factor k is set to 0.53 for normal-strength concrete ($f'_c < 55$ MPa) . For a higher strength concrete, a recommended value of 0.59 was adapted. For compression- tension state failure to occur, the minor principal stress need be compression, and σ_1 need be greater than f_{tu} given by

$$f_{tu} = f_t \left(1 + \frac{\sigma_2}{f'_c} \right) \quad (2-23)$$

In summary, El-Mihilmy and Tedesco proposed the following procedure for calculating the shear and normal stresses at the interface between the concrete and the end of the FRP laminate and for determining the delamination load.

- 1- Design the FRP-strengthened beam for flexure using strain compatibility method.
- 2- Check whether or not the beam can support this load without exhibiting an anchorage failure using the following steps:
- 3- Calculate the cracked section moment of inertial, I , in terms of FRP plate.
- 4- Calculate α_f and ψ_f from Eq. (2-17) and (2-20) respectively.
- 5- Calculate the elastic shear and normal stresses τ and σ_x from Eq. (2-8) and (2-21) respectively.
- 6- Calculate τ_{max} and $\sigma_{z,max}$ from Eq.(2-12) and (2-16) , respectively

7- Calculate the principal stresses (σ_1, σ_2) using

$$\sigma_{1,2} = \frac{\sigma_{z,\max}}{2} \pm \sqrt{\left(\frac{\sigma_{z,\max}}{2}\right)^2 + \tau_{\max}^2} \quad (2-24)$$

8- Compare σ_1 with f_{tu} according to the existing stress state using either Eq.(2-22) or (2-23). If $\sigma_1 > f_{tu}$, failure will occur due to delamination. To prevent delamination, reduce the applied load and repeat the procedure until $\sigma_1 \approx f_{tu}$. The load corresponding to this condition is the delamination load.

El-Mihilmy and Tedesco (2001) also proposed a simple equation to calculate the anchorage failure load which can be calculated by equating σ_1 to $0.95f_t$. From this, the anchorage failure load P_u for a beam subjected to two concentrated loads (four point bending) can be approximated by

$$P_u = \frac{3.8f_t I}{(\xi_f + 2)(d_f - c)(t_f + \psi_f^2 \alpha_f L_f)} \quad (2-25)$$

P_u = sum of the concentrated loads.

For the case of an FRP strengthened beam subjected to a uniformly distributed load, the uniform load, w_u , inducing anchorage failure can be approximated by:

$$w_u = \frac{3.8f_t I}{(\xi_f + 2)(d_f - c) \left[t_f (L - 2L_f) + \psi_f^2 \alpha_f L_f (L - L_f) \right]} \quad (2-26)$$

If the distance of the plate curtailment from the support L_f is very small with respect to the total span length L , then the uniform failure load w_u can be approximated by

$$w_u = \frac{P_u}{(L - L_f)} \quad (2-27)$$

The writer has reviewed the other methods and has determined that this method gives the most reliable prediction of the delamination failure load. This method is not intended for failures initiated by the concrete cover separation at the level of the tension steel. For the latter and other available methods of delamination analysis, the reader is referred to the detailed review papers of Smith and Teng (2001).

Chapter 3:

Experimental Program

3.1 General

The main objective of the current experimental work is to investigate a new anchor system for delaying delamination and to find the most appropriate anchor arrangement which can be used to realize this objective. Nine CFRP strengthened beams are tested to failure to investigate the new CFRP anchor used to prevent the delamination in beams strengthened externally with FRP laminate/sheet. The beams are divided into two main groups; the main parameter being the CFRP type and the amount of CFRP laminate/sheet used. In the following sections, the full details of the experimental program are presented and its results are discussed.

3.2 Test Specimens

Nine nominally identical simply supported beams were designed according to the CSA Standard A 23.3-94(CSA 1994). Detailed design calculations are given in Appendix A. All the beams were designed as under-reinforced with a reinforcement ratio of $0.3 \rho_b$. The chosen reinforcement ratio is in compliance with general design practice and the application of external FRP reinforcement to increase the flexural capacity of this kind of beams is practically feasible. Four No.20 ($\varnothing=20\text{mm}$) deformed bars ($f_y = 400 \text{ MPa}$) were used in the tension zone and two No.10 deformed bars ($f_y = 400 \text{ MPa}$) in the compression

zone. The beams were design for shear to prevent shear failure before flexural failure and the shear reinforcement consisted of No.10 steel U-stirrups ($f_y = 400$ MPa) spaced at 250 mm centre to centre. Additional external shear reinforcement was used after the casting of the beams as a precautionary measure against premature shear failure, which will be discussed in 3.5.5.

3.3 Material Properties

3.3.1 Concrete

Ready mixed concrete was ordered from the plant and was delivered to the laboratory. Separate concrete cylinders were tested at 28 days and on the day of testing the first beam to obtain the concrete properties.

Concrete with specified compressive strength $f'_c = 40$ MPa, slump of 100 mm and maximum aggregate size of 20 mm was ordered from the ready mix plant. The delivered concrete was made with super plasticizer and no silica fume. The measured slump was 91 mm. During casting, a total of 16 standard concrete cylinders (150x150x300mm) were cast and cured under the same conditions as the test beams. The curing consisted of moist curing for seven days and air-curing subsequently.

Eight cylinders were tested after 28-days; five cylinders were tested under compression and three cylinders in tension (splitting test). Another eight cylinders were tested on the day of the testing of the beams commenced (age 48-days), again with five under compression and three in splitting tension. Table 3-1 and 3-2 summarize the results for the tested specimens after 28-days and on the day of testing the first beam. The

Young's modulus E_c in these tables was calculated based on the following expression from the CSA Standard A 23.3-94:

$$E_c = 4500\sqrt{f'_c} \quad 3-1$$

Table 3-1: Concrete cylinder test results at 28-days

Age (days)	Specimen	Compression test		Tension test
		f'_c (MPa)	E_c (MPa)	f_t (MPa)
28	1	50.2	31883.38	3.33
	2	53.3	32853.08	4.4
	3	53.3	32853.08	4.4
	4	52.5	32605.6	-
	5	51.2	32199.38	-
	Average	52.1	32478.9	4.04

Table 3-2: Concrete cylinder test results at age 48-days, the day of testing the first beam

Age (days)	Specimen	Compression test		Tension test
		f'_c (MPa)	E_c (MPa)	f_t (MPa)
48	6	53.0	32761	4.6
	7	52.4	32575	4.5
	8	53.8	33007	3.1
	9	56.1	33705	-
	10	51.1	32168	-
	Average	53.3	32853	4.1

As has become customary, the actual strength of the concrete is more than 20% higher than its specified strength. This will not, however, negatively affect the results of the test or its conclusions; it will however affect the initial design of the beams as will be explained later. In the analysis, the 28-days strength values are used because the differences between the 28-days and 48-days strength values are small.

3.3.2 Longitudinal Steel Reinforcement

As stated earlier, longitudinal steel bars were used as tension and compression reinforcement in all the nine beams. Four straight No. 20 deformed bars were used in each beam with a length of 3300 mm in the tension zone; two similar No.10 bars were used in the compression zone. Four samples were taken from the end region of the tested beam to obtain the reinforcement properties in the form of tensile stress-strain relationship. Two strain gauges were used in the middle of each sample and the samples were tested in tension using the 400 Kip universal testing machine.

For each of the tension reinforcement samples, the stress-strain relationships are shown in Figs.3-1 to 3-4. The recorded strain values of the two strain gauges on each sample and their average are plotted. A good agreement may be noticed between the two strain gauges reading; however, sometime one of the strain gauges became faulty in which case the results of only one strain gauge was used. It is important to mention that the strain gauges were not always able to measure the strain after yielding of the bars. Hence, the maximum elongation of the bars was measured manually after each test.

The tensile test results for the four bars are presented in Table 3-3. Shown in the table are f_y = yield stress (MPa), ϵ_y = strain at yield, E_s = modulus of elasticity of steel (MPa), f_u = ultimate strength (MPa), ϵ_u = ultimate strain and ϵ_{sh} = strain corresponding to beginning of strain hardening.

Figure 3-1: Tensile stress-strain relationship of steel bar-1 ($\phi= 20\text{mm}$)

Figure 3-2: Tensile stress-strain relationship of steel bar-2 ($\phi= 20\text{mm}$)

Figure 3-3: Tensile stress-strain relationship of steel bar-3 ($\phi=20\text{mm}$)

Figure 3-4: Tensile stress-strain relationship of steel bar-4 ($\phi=20\text{mm}$)

Table 3-3: Summary of longitudinal tension steel reinforcement properties

Sample	f_y (MPa)	ϵ_y	ϵ_{sh}	ϵ_u	E_s (MPa)	f_u (MPa)
1	472	0.0025	0.0097	-	188800	684
2	463	0.0027	0.011	0.1105	171482	691
3	463	0.0030	0.0094	0.1003	154333	696
4	464	0.0026	0.0083	0.0786	178462	689
Average	466	0.0027	0.0096	0.0965	172593	690

The elastic modulus values in Table 3-3 seem unusually low which may indicate that the measured strain values may not be as accurate as they seem or it may be due to the reduction in the cross sectional area of the bars where the strain gauges were installed. We will use a value of 190 GPa later in our calculations, albeit $E_s = 200$ GPa is often the standard value.

3.3.3 Transverse Steel Reinforcement

Deformed No.10 bars were used as U- stirrups in all of the nine beams. As the focus of the current investigation is the flexural behaviour of beams externally strengthened with CFRP sheets/laminates, the stirrups were not tested to find their yield strength, but the specified $f_y = 400$ MPa for yield stress and $E_s = 200$ GPa were assumed in design calculations.

3.3.4 CFRP Laminate / Sheet

Two types of CFRP were used in this investigation. CFRP laminate strips known as Sika CarboDur, with each strip being 50 mm wide and 1.2 mm thick, with a cross-

sectional area of 60 mm^2 . According to the manufacturer's specifications, the tensile strength, modulus of elasticity and elongation at failure of CarboDur are 2800 MPa, 165 GPa and 1.9%, respectively (Sika 30, 2003). The second type of the CFRP used in this investigation was unidirectional high strength carbon fiber fabric, known as Wabo Mbrace CF 130 (Mbrace CF130, 1998). As reported by the manufacturer of the material, its nominal thickness, modulus of elasticity, tensile strength, and ultimate rupture strain are 0.165 mm/ply, 227 GPa, 3800 MPa and 1.67%, respectively. A summary of the CFRP material properties is given in Table 3-4.

Table 3-4: CFRP composite material properties provided by the manufacturers

Properties	Sika Carbo Dur Type S-512	Wabo MBrace CF130
Tensile strength	2800 MPa	3800 MPa
Modulus of elasticity	165 GPa	227GPa
Ultimate rupture strain	1.90%	1.67%
Thickness	1.2 mm	0.165 mm/ply
Width	50 mm	610 mm
Area	60 mm^2	-
Fiber volumetric content	> 68%	-

Note that Sika CarboDur 30 is available in 50 mm width and any lengths up to 250 m (820 ft), while the MBrace CF 130 is available in 82 m (270 ft) length and 610 mm (24 in) width.

3.3.5 Epoxy

Two types of epoxy were used, one for the sheet and the other for the laminate. In beams strengthened with laminate, Part A and Part B of the epoxy were mixed together

with a mixed ratio A: B = 3:1 by volume. Table 3-5 shows the properties for Sikadur 30 epoxy resin used in conjunction with the CFRP laminate.

Table 3-5: Sikadur 30 epoxy resin properties used for CFRP laminate

Properties	Sikadur 30	
Tensile properties ASTM D638 7 days	Yield strength	24.8 MPa
	Elastic modulus	4.5 GPa
	Rupture strain	1%
Flexural properties ASTM D790 14 days	Modulus of rupture	46.8 MPa
	Tangent modulus of elasticity in bending	11.7GPa
Shear strength ASTM C882 14 days	-	24.8MPa
Water absorption ASTM D570 24 h	-	24 h- 0.03 %
Color	Part A	White
	Part B	Black
	Mixed	Light grey
Mix	Mixed weight	1103 g/L
	Density (A+B)	2 kg/L
	Mixed ratio	3.:1 by weight and by volume

For the Wabo MBrace CF130 CFRP sheet, Wabo MBrace Primer and Wabo MBrace Saturant were used. Each has Part A and Part B. Table 3-6 includes all the material propertied for both Wabo MBrace Primer and Wabo MBrace Saturant.

Table 3-6: Wabo MBrace primer and saturant properties used for the CFRP sheet

Properties		Wabo MBrace Primer	Wabo MBrace Saturant
Tensile properties	Yield strength	14.5 MPa	54 MPa
	Strain at yield	2.00%	2.50%
	Elastic modulus	717 MPa	3034 MPa
	Ultimate strength	17.2 MPa	55.2 MPa
	Rupture strain	40%	3.50%
	Poisson's ratio	0.48	0.4
Compressive properties	Yield strength	26.2 MPa	85.2 MPa
	Strain at yield	4.00%	5%
	Elastic modulus	670 MPa	2620 MPa
	Ultimate strength	28.3 MPa	86.2 MPa
	Rupture strain	10%	5%
Flexural properties	Yield strength	24.1 MPa	138 MPa
	Strain at yield	4.00%	3.80%
	Elastic modulus	595 MPa	3724 MPa
	Ultimate strength	24.1 MPa	138 MPa
	Rupture strain	Large deformation-no rupture	5%
Color	Part A	Amber	Blue
	Part B	Clear	Clear
	Mixed	Amber	Blue
Mix	Mixed weight	1103 g/L	984 g/L
	Density	1102 kg/m ³	983 kg/m ³
	Mixed ratio	3:1 (part A :part B) by volume	3:1 (part A: part B) by volume
		100:30(part A: part B) by weight	100:34(part A: part B) by weight

3.4 The CFRP Anchors

3.4.1 Stud Anchors

In this study, initially it was intended to design and fabricate CFRP stud anchors and use them to anchor the sheets and laminates to the concrete in the tension zone of the beam. A CFRP anchor, similar in form to available steel studs, was designed and manufactured. Figs.3-5 and 3-6 show some typical anchors that were manufactured. Each anchor is made from a carbon fiber tow immersed in the Wabo Mbrace epoxy and then placed in a specially fabricated aluminum mould as shown in Figs.3-7, 3-8 and 3-9, which illustrates the geometry and dimension of the mould. The fibers were layed in the longitudinal grooves that were machined in both the male and female halves of the mould. One end of the fibers was then spread in a radial fashion inside the semi-circular groove machined in one end of each of the two halves of the mould. Although prototype anchors were successfully manufactured, due to lack of an efficient manufacturing technique, it was discovered that it would not be practical to manufacture a sufficient number of these anchors within a reasonable time period to test its performance in beams. Therefore, it was decided to adapt an existing CFRP product and use it as anchor. The latter anchors will be referred to as NEFMAC anchors because they are produced using the NEFMAC grids produced by AUTOCON composites in Toronto.

Figure 3-5: Anchor type-I manufactured from carbon fiber tow

Figure 3-6: Anchor type-II manufactured using a mould with larger end circles

Figure 3-7: Anchor mould geometry and dimensions

Figure 3-8: Anchor mould used to manufacture stud anchors

Figure 3-9: Another view of the anchor mould

3.4.2 NEFMAC Anchors

The NEFMAC anchors are π shaped and are made by cutting the ribs of a NEFMAC grid. The NEFMAC grids have square opening, with ribs of equal dimensions running in two orthogonal directions. Fig.3-10 illustrates a typical grid and as schematically illustrated in Fig.3-11, the π shape anchors are made by cutting the longitudinal and transverse ribs adjacent to one aperture. Figs.3-12 and 2-13 show two typical types of NEFMAC anchors used in the current study. The NEFMAC grids have been extensively tested by both its manufacturer and by many investigations (Zaghloul 2002). The guaranteed tensile strength of CFRP NEFMAC is 1200 MPa and its elastic modulus is 100 GPa. Due to the rough finish of its surface, it bonds very well with both concrete and the epoxy adhesive.

Figure 3-10: Typical NEFMAC CFRP grid

Figure 3-11: Extraction of π -shape anchor from CFRP grid

Figure 3-12: NEFMAC CFRP anchor type A

Figure 3-13: NEFMAC CFRP anchor type B

3.5 Beams Design

As stated earlier, nine beams were designed, fabricated and tested. All the beams had the same dimensions and internal steel reinforcement, with the main test parameters being:

1. Type of CFRP external reinforcement, i.e. either sheet or laminate.
2. Amount of CFRP reinforcement, using two FRP reinforcement ratios.
3. Presence or absence of mechanical CFRP anchors.
4. Number/spacing of anchors.

For easy identification of the beams with different characteristics, the following notation will be used: CB designates control beam, L designates beams with laminate, S designates beams with sheet, N indicates lack of anchor and A indicates presence of anchor in a beam. The number 90 refers to the distance from the theoretical cut-off point

of the sheet/laminate to its end point. This is commonly referred to as the development length. The number 901 refers to the group of beams with the smaller CFRP reinforcement ratio. For example **LN90** refers to a beam reinforced with the larger laminate reinforcement ratio, containing no anchor, with a development length of 90 mm.

3.5.1 Beams Geometry and Reinforcement

Fig. 3-14 shows the geometry and dimensions of a typical beam. All the beams have the same size and internal steel reinforcement. Each beam is 3400 mm long, with a 400 mm x 250 mm cross-section. The reinforcement consists of 4 No.20 bars as tension steel and 2 No.10 bars as hanger bars in the compression zone. The stirrups are No.10 bars at 250 mm centre to centre throughout the length of the beam. The beams are divided into groups as follows:

- One control beam (**CB**), without CFRP reinforcement.
- Two beams strengthened with two CFRP laminate strips, (each strip being 60mm² in cross section). One of the two beams, Beam **LN90**, did not contain anchors while the other, Beam **LA90**, did.
- Two beams strengthened with CFRP sheet and designed to have the same strength as the corresponding beams strengthened with two laminate strips. One beam was without anchor, the other with anchor, i.e. **SN90** and **SA90**, respectively.
- Two beams strengthened with one CFRP laminate strip (60mm² cross section area), one beam without and the other one with anchor, **LN901** and **LA901**, respectively.

- Two beams strengthened with CFRP sheet designed to give the same strength as the beams with one laminate strip, one without anchor and the other with anchor, SN901 and SA901, respectively.

Note: All dimensions are in mm

Figure 3-14: Details of steel reinforcement in test beams

3.5.2 Design of Strengthened Test Beams

It may be recalled that originally the specified concrete strength was 40MPa, but the actual strength was found to be 53 MPa. Therefore, the flexural and shear strength of these beams had to be recalculated. A slightly conservative concrete strength of 52.2 MPa was used in the calculations. The beams were analyzed according to the CSA Standard A23.3-94(CSA 1994), using the following concrete and steel properties:

* Concrete

Concrete strength $f'_c = 52.2$ MPa for all the beams and ultimate concrete strain $\epsilon_{cu} = 0.0035$ for Beam **CB** and 0.005 for the rest of the beams. The higher strain for the strengthened beams was used because these beams are over-reinforced after strengthening and previous experience(Svecove and Razaqpur 2001) has shown that in the case of over-reinforced beams, the assumption of $\epsilon_{cu} = 0.0035$ may be conservative.

* Longitudinal steel reinforcement

1- Tension steel:

$$f_y = 400 \text{ MPa}, E_s = 190 \text{ GPa}$$

2- Compression steel:

$$f_y = 400 \text{ MPa}, E_s = 200 \text{ GPa}$$

* Transverse steel reinforcement (Stirrups)

$$f_y = 400 \text{ MPa}, E_s = 200 \text{ GPa}$$

* CFRP Laminate

$$f_{fu} = 2800 \text{ MPa}, E_f = 165 \text{ GPa}, \epsilon_u = 1.9\%$$

*** CFRP Sheet**

$$f_{fu} = 3800 \text{ MPa} , E_f = 227 \text{ GPa} , \varepsilon_u = 1.67\%$$

Strain compatibility was used in analysis, assuming full bond between the steel and the concrete and between the FRP and the concrete up to failure, but strain hardening was ignored and the steel was assumed to be elasto-plastic. The shear strength of the beams was calculated using the simplified shear design method in the CSA Standard A23.3-94. Table 3-7 shows the amount of steel and external FRP reinforcement in each beam. In addition, the table indicates the axial rigidity of each type of reinforcement and the ratio of the axial rigidity of the FRP to that of the steel. This ratio indicates the relative contribution of each reinforcement type to the flexural resistance of the beam before steel yielding, after yielding, the contribution of the steel remains constant while the FRP contribution continues to increase until the FRP either delaminates or ruptures. The designed moment and shear capacities are given in Table 3-8 and the full design of the test beams is provided in Appendix A. Table 3-8 also gives the ultimate load corresponding to flexural and shear failure, assuming either four point or three point bending. These are designated as P_{uf} and P_{us} , respectively.

Table 3-7: Details of beams reinforcement, CFRP arrangement and length

Beam		Internal reinforcement		External reinforcement	Axial rigidity		
		Tension reinforcement	Compression reinforcement		$E_s A_s \times 10^3$	$E_f A_f \times 10^3$	$E_f A_f / E_s A_s$
		$A_s \text{ mm}^2$	$A_s' \text{ mm}^2$		kN	kN	
CB		1200	200	-	-	-	0
Group A	LN90	1200	200	120	228	19.8	0.09
	LA90	1200	200	120	228	19.8	0.09
	LN901	1200	200	60	228	9.9	0.04
	LA901	1200	200	60	228	9.9	0.04
Group B	SN90	1200	200	82.5	228	18.7	0.08
	SA90	1200	200	82.5	228	18.7	0.08
	SN901	1200	200	41.25	228	9.36	0.04
	SA901	1200	200	41.25	228	9.36	0.04

Table 3-8: Theoretical flexural and shear capacity

Beam	M_u (kN.m)	V_u kN	P_{uf} kN	P_{us} kN
CB	165.1	165.1	330.4	220
LN90	259.9	173.3	519.8	346.6
LA90	259.9	173.3	519.8	346.6
LN901	224.7	149.8	449.4	299.6
LA901	224.7	149.8	449.4	299.6
SN90	257.12	171.41	514.24	342.82
SA90	257.12	171.41	514.24	342.82
SN901	222.31	148.21	444.62	296.41
SA901	222.31	148.21	444.62	296.41

* Note: The material resistance factor ϕ is assumed equal to one. The control beam was tested under four point bending; all other beams were tested in three point bending.

3.5.3 Details of FRP Reinforcement

Figs.3-15 and 3-16 show the details of external FRP reinforcement and anchors. The FRP laminates used in this study are available in 50 mm wide strips. Hence in this study either one or two strips were used to strengthen the beams. The FRP sheets are available in wider sizes, but in this study either 166.7 mm or 83.3 mm wide swatches were cut from the wider rolls and bonded to the concrete surface. To achieve the same level of flexural strength as the beams strengthened with the laminate, three layers of the sheet were applied to each beam. It may be noticed in the latter figures that the external reinforcement is not extended to the ends of the beams, which has been normally the practice. The reason is that extension to ends may retard delamination, in which case the effect of the anchor cannot be investigated. Therefore, only a nominal development length was provided in order to encourage delamination. The development length is measured from the theoretical cut-off point to the end of the FRP sheet/laminate. The theoretical cut-off point is the point at which the unstrengthened beam capacity is equal to the applied moment at failure.

Figure 3-15: General details for the anchor and the external FRP

[Beam shown is upside down]

Note: All dimensions are in mm.

Figure 3-16: Enlarged view of segment A-A showing the anchor and the external FRP sheet/laminate

3.5.4 Anchor Details

The π - shape anchor were inserted into drilled holes on the tension face of each beam. Figs. 3-17 and 3-18 show the location of the anchors in the four beams which contain anchors. Figs.3-15 and 3-16 showed the position of a typical anchor in the cross-section. Note that the holes were drilled into the concrete adjacent to the FRP but not into the FRP because the holes through FRP would damage the fibers and reduce the FRP cross-sectional area. Since the strengthened beams would be tested in three-point bending, based on conventional design, the anchor spacing should be uniform because the shear is constant within the beam. However, at FRP-concrete interface, the stresses causing delamination also involve normal stresses and stress concentration occurs near the ends of the FRP laminates/sheets. Due to the preliminary nature of the current investigation and the fact that concrete cracking and nonlinearity can lead to more complex stress distribution than that predicted by elastic theory, it is not feasible to arrive at precise spacing for the anchors. Furthermore, as mentioned earlier, the anchor spacing is also dictated by the presence of the internal steel reinforcement because an anchor cannot be placed where a stirrup exists or where a longitudinal steel bar lies. Accordingly, the spacing used was based mainly on practical considerations.

Note: All dimensions are in mm.

Figure 3-17: Plan view of CFRP laminate/sheet and anchor arrangement for beams LN90, LA90, SN90 and SA90

Note: All dimensions are in mm.

Figure 3-18: Plan view of CFRP laminate/sheet and anchor arrangement for beams

LN901, LA901, SN901 and SA901

3.5.5 External Shear Reinforcement

The beams were originally designed for 40 MPa concrete and both their flexural and shear strength and their reinforcement were accordingly determined. However, the actual concrete that was delivered had a 28-day compressive strength in excess of 50MPa. Since the strengthened beams were over-reinforced, and since the flexural capacity of an over-reinforced beam is directly proportional to the concrete strength, the actual flexural capacity surpassed its design capacity by a wide margin. On the other hand, the shear capacity of a beam is proportional to the square root of concrete strength and therefore the shear capacity is not increased to the same extent as its flexural capacity. This change in the two capacities created the possibility of shear failure in the beams prior to flexural failure. To prevent this from happening, it was decided to use external steel bars as shear reinforcement. It is believed that the presence of this reinforcement will not affect the flexural behaviour and strength of the beams or the contribution of the FRP external reinforcement. In placing the external shear reinforcement, it was made certain that these bars did not come in contact with FRP sheet/ laminate. Fig. 3-19 shows the manner in which the external bars were fixed to the cross-section. Note that the bars and the top and bottom steel strips were symmetrically disposed with respect to the cross-section centreline.

Note: All dimensions are in mm.

Figure 3-19: Typical external shear reinforcement used in the test beams

3.6 Fabrication of the Tested Beams

The test beams were constructed in one phase, the construction started with making the plywood forms and attaching the strain gauges to the longitudinal steel. A thin layer of silicone coating was applied to each strain gauge to protect it against impact, moisture and possible damage during the casting processes. The reinforcement cages were constructed with high accuracy and placed in the formworks. All the forms were brushed with oil from inside to facilitate their removal after the casting and curing of concrete. Plastic chairs were placed under the reinforcement cages to maintain a clear cover of 35 mm at the bottom of the beam. Fig.3-20 and 3-21 show the concrete forms and the reinforcement cages for the nine beams

Figure 3-20: Formwork and the reinforcement cages inside them

Figure 3-21: Formwork for the nine beams

The concrete was supplied by a local concrete plant. The specified maximum aggregate size and slump were 20 mm 100 mm, respectively. Super plasticizer was used to obtain good workability without increasing the water cement ratio. All of the beams were vibrated properly using an electric vibrator. After casting, the exposed concrete surface was smoothed and covered with a plastic sheet. During casting, the slump was measured and was determined to be 91 mm. The concrete was at room temperature and the surface was kept moist for at least 10 days.

The forms were removed after 3 weeks and the specimens were cured in the normal laboratory conditions. Although the specified concrete strength was 40 MPa, the actual strength from standard cylinders was found to be 52.2 MPa at 28 days and 53.3 MPa on the day of the commencement of beams test (age 48-days). Figs.3-22 to 3-25 show the various stages of the concrete casting operation.

Figure 3-22: Concrete casting operation

Figure 3-23: Concrete vibration

Figure 3-24: Finishing of concrete surface

Figure 3-25: 16 standard concrete cylinders

After curing and the removal of the formwork, eight beams were flipped over upside down to facilitate the installation of the CFRP laminate/sheet and the drilling of holes for the anchors. The processes of applying the CFRP laminate/sheet started using air pressure to clean and dry the surface, remove dust, laitance, grease, foreign particles and other bond inhibiting materials from the surface. For beams with laminate, the laminate surface was cleaned using acetone before applying the epoxy adhesive. In applying the epoxy adhesive and the laminate, the following procedure was followed according to the manufacturer's instructions:

1. Apply the mixed Sikadur 30 adhesive onto the concrete using a spatula.
2. Apply the mixed Sikadur 30 onto the CarboDur laminate with a wedge-shaped spatula.
3. Place CarboDur laminate onto the concrete surface within the epoxy open time.

A rubber roller was used to press the laminate into the epoxy resin until the adhesive under the epoxy oozed out. Excess adhesive was removed and the epoxy was allowed to cure undisturbed for at least 24 hours. Fig.3-26 shows the beams after the application of the epoxy; Fig.3-27 shows the beams after the installation of the laminate. Subsequently, the width of the laminates was reduced using a grinder. This was needed to increase the level of the interfacial stresses and encourage delamination.

Figure 3-26: Beams LN90 and LA90 after application of the epoxy

Figure 3-27: Beams LN90 and LA90 after the laminate installation

In beams strengthened with sheet, two layers of epoxy were needed according to the manufacturing instructions. Using MBrace instructions (MBrace, 1998) the following steps were followed:

1. Wabo MBrace primer was used as first layer to penetrate the pore structures and to provide high bond base coat and strong adhesion for the Wabo MBrace epoxy.
2. Two days after primer application, the MBrace Saturant was applied using a normal painting roller.
3. The first ply of the CFRP sheet was placed onto the first layer of wet saturant using the roller. The sheet was pressed down until the adhesive exuded from under the sheet.
4. The second layer of the saturant was applied using a roller, and then steps 2, 3 were repeated to install the next two plies of the sheet. After the installation of the last ply, the epoxy was allowed to cure undisturbed for at least 24 hours.

After curing, strain gauges were attached to the CFRP surface along its length. The strain gauges were bonded after the surface of the CFRP was cleaned well using extreme care. A thin layer of silicone coating was applied on each strain gauge to ensure protection against impact, moisture and possible damage during handling. Figs.3-28 and Fig. 3-29 show various stages of applying the Wabo MBrace CF130 sheet to the beams.

Figure 3-28: Beam after Wabo MBrace primer application

Figure 3-29: Wabo MBrace saturant layer and the sheet installation

Holes were drilled in beams LA90, LA901, SA90 and SA901 using a concrete drill with a 12.5 mm (0.5") bit. Extreme care was taken not to drill through the CFRP or to hit the longitudinal or the transverse steel reinforcement. All the holes were cleaned well using air pressure to ensure a clean surface. To install the anchors, Wabo MBrace primer and saturant were used and the adhesive curing process was the same as discussed earlier for the sheet. Note that the horizontal spine of the anchor was epoxied to both the concrete surface and to the FRP sheet/laminate.

After the anchor installation and curing of the adhesive, the beams were white washed and (50x50 mm) a grid was drawn on one of its side faces to facilitate observation of crack formation and propagation during testing. Figs.3-30 to 3-33 show the four beams after anchors installations.

Figure 3-30: Anchor arrangement in beam LA90

Figure 3-31: Anchor arrangement in beam LA901

Figure 3-32: Anchor arrangement in beam SA90

Figure 3-33: Anchor arrangement in beam SA901

3.7 Test Equipments and Instrumentations

The following equipments and instrumentations were used in the testing program:

1. One MTS closed- loop servo- controlled electro-hydraulic jack as actuator, with the capacity of 110 kip (489 KN) and stroke length of 6 in (150 mm).
2. Knife-edge bearing plates with roller supports.
3. MTS data acquisition system attached to a microcomputer.
4. Ten linear variable differential transducer (LVDT).
5. Portable hand microscope with 10 micron accuracy.

Beam deflection was measured using 10 LVDT's, five on each side as shown in Fig 3-34. The distance between the LVDT's varies in each beam, depending on the position of the external shear bandages, strain gauges and anchors.

Note: All dimensions are in mm.

Figure 3-34: Typical LVDT locations on test beams

Fig. 3-35 shows a typical beam and the LVDT's installed on one face of the beam. For a beam not experiencing twisting, each LVDT pair equally spaced along the beam, should record nearly the same deflection and the actual beam deflection would be the average of the two readings.

Figure 3-35: Position of LVDT's, Supports and Actuator

Measurement of strains in the longitudinal reinforcing steel, concrete and CFRP laminate/sheet was taken using electrical resistance strain gauges with gauge length of 5 mm for longitudinal steel and CFRP laminate/sheet and 30 mm for concrete. The strain gauges were sensitive to 0.01 micro strains with a standard deviation of 2%. Fig.3-36 shows the typical position of all the strain gauges on the longitudinal steel, while Figs.3-37 and 3-38 show the arrangement of the strain gauges on the CFRP laminate/sheet. Fig.3-39 shows some typical strain gauges after their installation on beams.

Figure 3-36: Typical strain gauges position on the longitudinal tension steel

Note: All dimensions are in mm.

Figure 3-37: Arrangement and spacing of the strain gauges on CFRP laminate/sheet in beams LN90, LA90, SN90 and SA90

Note: All dimensions are in mm.

Figure 3-38: The arrangement and spacing of the strain gauges on CFRP laminate/sheet in beams LN901, LA901, SN901 and SA901

Figure 3-39: Arrangement of the strain gauges on the test beams

For each of the test beams, three strain gauges were installed on the concrete surface in the compression zone. In beams **LN90**, **LA90**, **SN90** and **SA90**, the strain gauges positions were exactly the same as the longitudinal steel strain gauges, one in the middle and the other 2 were 693mm away from the beam end on each face. In the rest of the beams, i.e. **LN901**, **LA901**, **SN901** and **SA901**, the position of the strain gauges was changed to one at mid-span and the other two at 300 mm from mid-span with one on either side. The reason is that the highest compressive strain in concrete occurs at mid-span under three point bending, and due to the large magnitude of this strain, it can be more accurately measured. In Figs.3-40 and 3-41, the typical locations of the concrete strain gauges are shown.

Figure 3-40: Location of the concrete strain gauge for beams CB, LN90, LA90, SN90 and SA90

Note: All dimensions are in mm.

Figure 3-41: Location of the concrete strain gauge for beams LN901, LA901, SN901 and SA901

3.8 Test Setup

The tests were conducted in the John Adjeleian Laboratory of the Civil and Environmental Engineering Department at Carleton University. The steel frame supporting the actuator was fixed to the 1.0 m thick laboratory floor. The roller supports setting directly on the strong floor. All the beams were simply supported, one end resting on a roller and the other on a hinge support. With the exception of the control Beam CB and Beam LN90, which were loaded in four point bending with a shear span of 1000 mm,

all the remaining eight beams were tested in three point bending, and with the load acting at mid-span. The load was applied using a hydraulic jack and displacement control. The loading and support plates were levelled using hydrostone compound. Figure 3-35 shows a typical beam and the corresponding loading jack and support. The loading rate was relatively slow to simulate static loading condition. The load was applied in increments of 10 or 20 kN, and each time after the full increment was reached, crack pattern was checked and the cracks were traced. Due to the dramatic reduction in the stiffness of the beams after yielding of tension steel, it often took much longer time to realize the full load increment.

Chapter 4:

Experimental Results

4.1 General

All the strengthened beams were subjected to a concentrated mid-span load applied monotonically. During the test, the complete behaviour of each beam was monitored, including the strains in the steel reinforcement, in the CFRP laminate/sheet and in the concrete. In addition, the displacements at five locations along the beam were recorded by an automatic data acquisition system. The crack patterns were manually traced. In this chapter, the complete behaviour of all of the beams during the tests will be discussed. Furthermore, the load-deflection curves, the strain along the CFRP laminate/sheet and the deflected shapes at different load levels will be shown. The results will be discussed and analyzed in the following chapter.

4.2 Beams Behaviour and Observations

4.2.1 Control Beam (CB)

This beam was designed as the control beam, and it may be recalled as under-reinforced beam, with a reinforcement ratio of $0.3\rho_b$, where ρ_b is the balance reinforcement ratio. The beam was the first of the nine beams to be tested and it was subjected to four point bending.

The test started by applying the load in increments of 10 kN. It was observed that the first flexural crack occurred near mid-span at 70 kN. Upon increased loading, other flexural cracks appeared in other locations between mid-span and the support, included shear cracks which developed at 260 kN near the supports and later propagated towards the loading point. At 350 kN, the shear cracks had penetrated the compression zone and it appeared that the beam might fail in shear. In fact, as can be observed in Fig.4-1, the beam failed in flexural at 375.6 kN, but due to the deep penetration of the shear cracks into the compression zone, it became necessary to increase the shear capacity of the strengthened beams and to increase the moment to shear ratio by changing the strengthened beams loading from four point bending to three point bending. This change in loading will not alter the fundamental objectives of the current investigation.

Figure 4-1: Shear cracks in and compression failure of control beam CB

The load mid-span deflection curves for Beam CB are presented in Fig.4-2. The plotted curves are for the pair of LVDT's at mid-span and for the average of the two. Notice that the readings of the two LVDT's are very close. The load-deflection curve reveals a ductile behaviour and the failure moment is 187.8 kN.m, while the theoretical calculated strength is 165.1 kN.m. The lower theoretically calculated capacity of this beam indicates that the code's approach is conservative because as we will see later more advanced computer models predict a much higher strength. These results will also serve as reference for discussing the strength and behaviour of the strengthened beams. The deflected shapes of the beam at various load levels are presented in Fig.4-3. The deflected shapes show good symmetry, indicating that the load was centrally placed and that the two halves of the beam on either sides of its centreline behaved nearly the same.

Figure 4-2: Load mid-span deflection of Beam CB

Figure 4-3: Deflected shapes at various load levels of Beam CB

4.2.2 Beam LN90

Beam LN90 was reinforced with 180 mm^2 of laminate without mechanical anchors. This beam was tested under four point bending as shown in Fig.4-4 and the test began by applying the load in increments of 10 kN. It was observed that the first flexural crack developed in the tension zone at mid-span at a load of 50 kN. Upon increased loading, the cracks propagated upward to the compression zone and new cracks appeared along the beam. At 220 kN new inclined cracks appeared near the supports and propagated towards the loading points. At 380 kN, the shear cracks were rapidly propagating towards the compression zone and it appeared that the beam may fail in shear. At a load level of 391.1 kN, the test was stopped before shear failure might occur.

Fig.4-5 shows the extent to which the shear cracks had propagated along the height of the beam when the test was stopped. As stated in the previous chapter, this

problem arose in part due to the actual concrete strength being significantly higher than the specified strength and the relatively small amount of shear reinforcement. The load mid-span deflection curve for Beam LN90 after stopping the test and before complete failure is shown in Fig.4-6. The plotted curves are based on data from the two LVDT's at mid-span and their average value. In this case we see a greater difference between the readings of the two LVDT's. It is possible that the beam was experiencing some twist in which case the average of the two LVDT's would indicate the vertical deflection of the centreline of the beam.

Figure 4-4: Beam LN90 under four point bending

Figure 4-5: Shear cracks in Beam LN90 just before stopping the test

Figure 4-6: Load mid-span deflection of Beam LN90

Fig.4-7 shows the deflected shape of the beam at various load levels. Unfortunately, some LVDT's on one side of the cross-section did not function properly; therefore, the data from the functioning LVDT's only are plotted. The deflected shape should be theoretically symmetrical, but in some cases, it is quite unsymmetrical. This lack of symmetry could be due to the formation of cracks at different locations or due to some small amount of twist. Since the deflection curves are plotted for the purpose of checking the over-all behaviour of the beam and whether it follows the theoretically expected behaviour, the failure of some LVDT's to accurately record deflections will not affect the behaviour of the FRP reinforcement or the anchors.

Figure 4-7: Deflected shapes at various load levels of Beam LN90

Fig.4-8 shows the variation of the longitudinal strains in the FRP laminate for various load levels. Notice that the strain between the two load points is essentially constant because this is a constant moment zone. The maximum strain in the FRP

reached 0.25% when the load was stopped. This strain is substantially less than the maximum strain capacity of the laminate, and of its maximum expected strain of 1.72% at failure (assuming full bond and elasto-plastic steel behaviour). The important point to observe is the sharp increase in FRP strain over a relatively short distance. The rather steep slope of the strain curves between 500 mm and 1000 mm from each support indicates high shear stresses at the FRP-concrete interface. These high shear stresses are often responsible for delamination. In the next chapter, these strain values will be used to estimate the interfacial shear stresses.

Figure 4-8: Strain profiles in CFRP of Beam LN90 at various load levels

4.2.3 Beam LN90B

To prevent possible premature shear failure, similar of that of Beam LN90, as mentioned in the previous chapter, special bandages were designed and fabricated and used as external shear reinforcement in the strengthened beams. A small amount of

pretension was applied to the external shear reinforcement to close the existing shear cracks. Furthermore, the loading position was changed from four point bending to three point bending to induce flexural failure. The total width of the three CFRP strips was reduced by 1/3. The latter was achieved by cutting each of the two 50 mm wide strips located on either side of the vertical centreline of the cross-section into 25 mm wide strips longitudinally and then one of the two 25 mm wide strips on either side was cut transversely at several locations along its length to render it ineffective as flexural reinforcement. In this case the interfacial shear stress will increase and will likely lead to early delamination. The removal of 1/3 of the laminate width reduced the total width to 100 mm, ($A_f = 120 \text{ mm}^2$). All the other laminate strengthened beams had the same two strips as external reinforcement while the amount of sheet was also reduced to give the same ultimate flexural capacity as the laminate reinforced beams.

This beam with reduced laminate strips, referred to as **LN90B** was tested in three point bending, with the load acting at mid-span. Fig.4-9 and Fig.4-10 show the new test setup as applied to Beam **LN90B**.

Figure 4-9: Beam LN90B after the modifications made to Beam LN90

Figure 4-10: Longitudinal and transverse cut made for Beam LN90B

The test was again started by applying the load in increments of 10 kN. Due to the previous loading, most of the strain gauges were inoperative. It is important to mention that due to the presence of the bandages, the shear cracks remained tight and shear failure did not appear impending.

The load was increased monotonically until delamination occurred at mid-span at 346.8 kN and propagated towards the end of the beam with a big bang, which signified release of elastic energy. It appears that the energy release generates a stress wave that travels along the FRP-concrete interface and cause further delamination. After delamination the load dropped substantially and the test was stopped. Fig.4-11 shows the delaminated sheet.

Fig.4-12 shows the load mid-span deflection of the beam up to delamination. The plotted deflection is based on the average of the recorded values from the two LVDT's at mid-span. The two major changes in the slope of this curve signify the advent of cracking and yielding of steel reinforcement in the beam. It appears from the load-deflection curves that the beam might have experienced some twisting. This is unexpected because every effort was made to place the hydraulic jack and the loading plate symmetrically on the beam.

Figure 4-11: Failure in the concrete surface caused delamination in Beam LN90B

Figure 4-12: Load mid-span deflection of Beam LN90B

The longitudinal strain variation along the FRP laminate at various load levels is plotted in Fig.4-13. Notice that due to the change in the loading from four point bending to three point bending, the strain profile is not the same as in Beam LN90. However, once again the strain profile follows the shape of the bending moment diagram. One can observe that the slope of these strain diagrams is quite steep near the ends of the sheet, but they tend to become less steep as one moves away from the ends. This means that the interfacial shear stresses near the ends of the laminate are quite high, but as one moves away from the ends it becomes smaller and almost constant. This issue will be further discussed in the next chapter. Notice that in this beam the maximum strain in the FRP reached nearly 0.6%.

It is interesting to observe that the slope of the strain diagram at $P = 346 \text{ kN}$ remains practically constant, even though within the first 500 mm from the ends of the sheet there is a reduction in slope, which may indicate some delamination or micro-cracks, causing reduction in stiffness.

Figure 4-13: Strain profiles in CFRP laminate along Beam LN90B at various load levels

Fig.4-14 shows the deflected shape of the beam at various load levels. The plotted data is from the LVDT's from one side of the beam because the LVDT's on the other side did not work properly. The deflected shape is close to the expected shape based on conventional beam theory but it shows greater curvature at mid-span than expected due to twisting. The maximum deflection of 14 mm is within the small deflection range, but as we will see later that some of the other beams reached significantly higher deflection at failure.

Figure 4-14: Deflected shapes at various load levels of Beam LN90B

4.2.4 Beam LA90C

The testing of Beam LN90B was stopped after delamination occurred at 346.8 kN. However, the concrete in the compression zone did not fail after delamination, which means that the beam could be repaired and reloaded. Hence, it was decided to re-glue the laminate to the concrete and use the CFRP mechanical anchors to further strengthen the beam against delamination. This repaired beam is referred to as Beam LA90C. Fig.4-15 shows a partial view of the beam bottom, including the FRP mechanical anchors. Notice that only half of each of the two side laminate strips in fact function as flexural reinforcement.

Upon reloading the beam, it delaminated at 328.8 kN, which is lower than the earlier delamination load. This was not entirely unexpected because it was not possible to re-glue the partially delaminated strips along its entire delaminated length. However, due

to the presence of the anchors, the advent of delamination did not render the laminate completely ineffective because it continued to carry load until it reached a maximum load of 367.5 kN and the beam failed due to crushing of concrete at mid-span. Fig.4-16 shows the deflected beam under an applied load of 362 kN, i.e. at 98.5% of its failure load.

Figure 4-15: Beam LA90C after re-gluing and anchors installation

Figure 4-16: Full plastic behaviour of Beam LA90C at load $P=362$ kN

Fig.4-17 shows the load mid-span deflection of beam **LA90C**. Notice the small drops in load each time further delamination occurred, but each time the beam regained its stiffness and continued to carry increased load. This beam failed at 6% higher load than Beam **LN90B**, which is the same beam before the installation of anchors. It is important to observe that the maximum deflection of this beam was over 100 mm and the beam continued to carry increased load until failure. The continuously positive slope of the load-deflection curve in Fig. 4-17 indicates that thanks to the anchors the FRP was able to contribute to both the strength and the stiffness of the beam up to failure.

Figure 4-17: Load mid-span deflection of Beam LA90C

Fig.4-18 shows the deflected shape of the beam at various load levels. Since once again some of the LVDT's were not functioning properly, only the data from the functioning LVDT's were used. The deflected shapes, as expected, follow their theoretical form based on conventional beam theory.

Fig.4-19 shows the variation of the longitudinal strains along the FRP laminate at different load levels. Some of the strain gauges after so many loadings and unloading peeled off and were not functioning, but the data from the functioning strain gauges up to a load of 320.25 kN clearly show that the laminate is contributing significantly to the flexural resistance of the beam. Notice that in this case the maximum recorded strain reached nearly 0.35 %. Beyond 321 kN, all the strain gauges on the FRP peeled off and the strains could not be recorded. The rather steep slope of the strain diagrams in

Fig. 4-19 indicates that high interfacial shear stresses existed at the FRP ends, which was made possible by the presence of the anchors.

Figure 4-18: Deflected shapes of Beam LA90C at various load levels

Figure 4-19: Strain variation in CFRP along Beam LA90C at various load levels

Figs 4-20 and 4-21 show the tension face of the beam after failure. In Fig. 4-20 we observed that the laminate has experienced a large amount of slip at failure ($\approx 55\text{mm}$). This means that the anchors were not able to prevent slippage, but it also means that the slippage must have been caused by the shear stresses resisted by the FRP. Hence, the anchors appear to have been partially effective in enabling the FRP to resist shear stresses after the initiation of delamination. Fig. 4-21 shows one of the anchors near the mid-span after failure. The anchor is fully embedded in the concrete, but the concrete cover has experienced substantial damage. Since the anchor is embedded deeper inside the concrete, the damage to the cover has not caused anchor pullout. In the next chapter, further comments will be made about the effectiveness of these anchors.

Figure 4-20: FRP end slippage in Beam LA90C at failure

Figure 4-21: The anchor preventing the laminate from complete delamination along the length of Beam LA90C

4.2.5 Beam LA90

Beam LA90 is reinforced with two laminate strips and is retrofitted with anchors, the beam was tested under three point bending and bandages were used as external shear reinforcement. Fig.4.22 shows the anchors used for this beam and the cuts made to the laminate to reduce it from three strips to two strips. Fig.4-23 shows the beam and its test set-up. This beam was loaded incrementally similar to the previous two beams.

Figure 4-22: Anchor arrangement and the cuts in laminate for Beam LA90

Figure 4-23: Beam LA90 at the commencement of the test

Fig.4-24 shows the load mid-span deflection curve of Beam LA90 based on the data obtained from the two LVDT's at mid-span. The figure shows a noticeable drop in load at almost 350 kN. The drop is due to delamination of the FRP strips at mid-span. Although the strips delaminated from the concrete, they were still fully bonded to the anchor. Therefore, upon increased loading, the beam continued to carry load until it failed at 372.8 kN. The beam failed due to crushing of the concrete in the compression zone (Fig.4-25), but as Fig 4-26 shows the FRP laminate had undergone almost 100 mm slip. Since the beam was too close to the laboratory floor, it was not possible to observe the slip during the loading process. However, the continuous positive slope of the load-deflection curve indicates that the laminate continued to resist load after delamination. This load carrying capacity is made possible by the ability of the anchors to transfer the load from the concrete to the laminate. The beam also experienced large deformations at failure (120 mm), as Fig.4-25 shows, and exhibited good ductility as is evident from Fig.4-24.

Figure 4-24: Load mid-span deflection of Beam LA90

Figure 4-25: After failure and the removal of the load

Figure 4-26: Slippage happened in Beam LA90

Fig.4-27 shows typical deflected shape curves for Beam **LA90** at various load levels, and they are as expected. However, in this and the other beams some twist was observed, which means that the load may have been slightly off the longitudinal centreline of the beam. It is believed that the twist will not have any significant effect on the flexural behaviour of the beam.

Figure 4-27: Deflected shapes at various load levels of Beam LA90

Figs.4-28 and 4-29 show the variation of the longitudinal strain along the laminate at different load levels. It should be pointed out that after delamination, in almost every case, some or all of the strain gauges applied to the FRP surface peeled off and became inoperative. The strain profiles indicate a rather steep slope near the ends of the laminate, which is indicative of high shear stresses at the FRP-concrete interface. The slope tends to decrease as one moves away from the end regions, gradually diminishing as one approaches the mid-span. This implies that the critical shear stresses occur near the end regions although the greatest stress in the laminate occurs at mid-span. From the anchor design point of view, the end regions would need to be examined closely. Since the failure load for this beam was almost 8.8 % higher than the delamination load, one can attribute the higher failure load to the presence of the anchors. However, as will be discussed in the next chapter, none of the beams with two FRP laminates, or equivalent

amount of FRP sheet, achieved their full capacity based on perfect bond and strain compatibility theory.

Figure 4-28: Strain profiles of CFRP laminate in Beam LA90 at various load levels

Figure 4-29: Strain profiles close to the failure load of Beam LA90

4.2.6 Beam SN90

Beam SN90 stands for the beam reinforced with CFRP sheet but with no anchors. The total cross-sectional area of the sheet in this case was equal to 82.5 mm^2 . To speed up the testing, the test was started by applying 20 kN load increments. During the test, the hydraulic jack malfunctioned and the beam was unloaded after it reached a load of 228.6 kN. It was subsequently reloaded but the jack malfunction recurred at a load of 231.89 kN, hence again the beam was unloaded completely and then reloaded.

Due to the unloadings and reloadings, the LVDT's did not return to zero and they indicated some residual deflection upon full unloading. After the third loading trial, the beam reached a load of 346.7 kN and then the sheet delaminated, generating a thunderous noise. The delamination began at mid-span, but it did not propagate all the way to the supports. Fig.4-30 shows the load mid-span deflection curve of this beam, where after initial delamination the load dropped from 346.7 kN to 245.5 kN. This a rather substantial drop compared to the beams with anchors.

.

Figure 4-30: Load deflection curve of Beam SN90 at 1960 mm from the left support

The load was then increased until it reached 346.1 kN and at this load once again a loud sound could be heard as another portion of the sheet delaminated. Once again, as could be observed in Fig.4-30, the load dropped, but not to the same extent as before. The load dropped from 346.1 kN to 317.5 kN. The load was further increased after the second delamination from 317.5 kN to 326.2 kN, and at the latter load, the beam failed due to the crushing of the concrete in the compression zone. Once again, as Fig.4-31 shows, the beam experienced a large amount of deflection before failure. At failure the sheet delaminated over a large portion of its length as can be seen in Figs.4-32 and 4-33.

Figure 4-31: Fully plastic behaviour of Beam SN90

Figure 4-32: Failure in concrete surface caused the delamination in Beam SN90

Figure 4-33: Another view of delamination in Beam SN90

Fig.4-34 shows the deflection profile of the beam at various load levels. The symmetric shape of the curves shows that despite the repeated loading and unloading, the LVDT's remained functioning and the beam had not suffered unusual damage. Fig.4-35 shows the variation of the strain profile along the sheet at various load levels. Since one of the strain gauges on the left side of the beam did not function, and therefore, only the information from the right-hand side strain gauges is complete. It is interesting to observe that the strain profile for the sheet is noticeably different from that in the laminate. Whereas the laminate strain profiles have steep slope near the laminate ends, the sheets show a more gradual increase in slope. This confirms the well-known observation made by others that due to their higher thickness, the laminates are more prone to sudden delamination. In the next chapter, this issue will be further discussed. The important point

to observe in the case of this beam is that unlike the beams with anchor, the failure load for this beam is less than the load which initiated delamination.

Figure 4-34: Deflected shapes at various load levels of Beam SN90

Figure 4-35: Strain profiles of CFRP in Beam SN90 at various load levels

4.2.7 Beam SA90

Beam SA90 is the beam reinforced with CFRP sheet and retrofitted with anchors. This is the companion beam of Beam SN90, with the difference between the two being the presence of anchors in one and not the other. This beam was tested in three point bending. Fig.4-36 shows the beam setup while Fig.4-37 shows the anchors arrangement at the bottom of the beam.

Figure 4-36: Setup for Beam SA90

Figure 4-37: Anchor arrangement, longitudinal and transverse cut made to the sheet for
Beam SA90

The load was applied in increments of 20 kN and the first crack appeared at mid-span at 60 kN. Upon increase in load, more cracks appeared and the beam behaved as expected until a load of 384.8 kN was reached, at which point a loud noise could be heard, signalling delamination, followed by the load dropping to 319.9 kN. This can be inferred from the load mid-span deflection curve plotted in Fig. 4-38.

The load was subsequently increased until it reached 373.1 kN at which time the beam failed due to the crushing of the concrete in the compression zone. Fig. 4-39 shows the beam after failure. After failure the FRP sheet and anchors were inspected and it was discovered, as Fig. 4-40 shows, that significant slip had occurred (≈ 101.5 mm) at one end of the sheet. Notice that in comparison to the companion beam without anchors, the delamination load for this beam is about 11% higher, which could be attributed to the presence of the anchors.

Another observation that was made was the recurring low level noise of delamination after the initial loud noise, and it continued until almost failure. The continuing noise was due to subsequent delamination, both at the interface of the sheet and the concrete and at the interface of the anchor and the sheet.

Figure 4-38: Load mid-span deflection of Beam SA90

Figure 4-39: Fully plastic behaviour of Beam SA90 at failure

Figure 4-40: Sheet end slippage in Beam SA90

Fig.4-41 shows the deflected shape of this beam at different load levels. The deflected shapes are in accordance with the expected behaviour of the beam and good symmetry can be observed. As mentioned earlier, the deflected shape was plotted mainly to look for any abnormality during the test. The letter D in the legend of Fig.4-41 and any subsequent figure signifies the loads applied after delamination.

Figure 4-41: Deflected shapes at various load levels of Beam SA90

Fig.4-42 shows the variation of the longitudinal strain in the sheet along the beam. We can observe that in contrast to the corresponding strain profiles of the companion beam without anchors, the slope near the ends is steep and remains steep until the failure load. The present strain profiles are closer to those of the laminates prior to delamination. The steep slope is indicative of high shear stresses, particularly at the end regions. The nearly symmetric shape of the strain diagrams indicates that the strain gauges were

functioning and the results reflect the true behaviour of the FRP sheet. Observe that a maximum strain of almost 9000μ was reached in the sheet.

Figure 4-42: Strain profiles of CFRP in Beam SA90 at different load levels

4.2.8 Beam LN901

Beam LN901 was reinforced with a single 50 mm wide laminate, without anchors. The beam was loaded similar to the previous beams using load increments of 20 kN. The first crack appeared at mid-span at 60 kN. Upon increased loading, more cracks appeared and no abnormal behaviour was observed until the load reached 298.5 kN at which time delamination occurred, accompanied by the customary loud noise.

As delamination occurred, the load dropped to 262.6 kN, but subsequently it increased and reached 347.6 kN. The beam failed at the latter load due to crushing of concrete in the compression zone at mid-span. Fig.4-43 shows the beam after failure

while Fig.4-44 shows the delamination strip and the failure of the concrete at its interface with the strip.

Figure 4-43: Fully plastic behaviour of Beam LN901

Figure 4-44: Concrete failure in the interface between the laminate and the concrete interface of Beam LN901

It is important to note that the theoretical ultimate capacity of this beam without external reinforcement is almost 15 % higher than its delamination load, but the experimental failure load is about 1.5% higher than its predicted strength assuming a maximum concrete strain of 0.0035 and strain hardening. Thus one can argue that despite premature delamination, the application of external CFRP reinforcement did not increase the ultimate capacity of this beam.

The load-deflection curve of Beam LN901 is shown in Fig.4-45. Since in this case the LVDT's could not be placed exactly at the centre of the beam, the deflections are plotted for a point 1770 mm from the left-support. From this figure we can see only a small drop in load after delamination, which implies that the delamination was confined to a small region and the laminate was still bonded at its ends.

Figure 4-45: Load deflection curve at 1170mm from the left support of Beam LN901

Fig.4-46 shows the deflected shape of the beam at various load levels. The shape is almost symmetric and follows the expected pattern. Observe that these curves are plotted based on the average reading of the two LVDT's at each location along the span. Generally the pair of LVDT's gave relatively close readings, but where one or the other seems to be completely outside the expected range, the reading of only one of the two was used to plot these figures. Observe that at failure the beam maximum deflection reached nearly 100 mm, which is $1/30$ of the span. Due to such large deflection, the steel reinforcement experienced strain hardening and the higher failure load could be attributed partially to this phenomenon.

The FRP longitudinal strain variation along the laminate for various load levels is shown in Fig.4-47. We notice that it has an almost parabolic profile up to almost 200 kN. At very high load levels, a plateau is developing near the mid-span, which indicates the occurrence of delamination. The maximum strain reached is only 0.6 %, which is only 40% of the maximum strain capacity of this laminate.

Figure 4-46: Deflected shapes of Beam LN901 at various load levels

Figure 4-47: Strain profiles of CFRP in Beam LN901 at various load levels

4.2.9 Beam LA901

This beam is the companion beam to Beam LN901 and was therefore fitted with anchors. As in the case of the previous beam, the load was applied incrementally and the first crack appeared at mid-span at 60 kN. Thereafter, the load was increased and more cracks appeared along the span. Once the load reached 321.2 kN a loud noise was heard and the FRP began to delaminate and the load dropped to 273.98 kN. The load was subsequently increased until it reached 362.5 kN at which point the beam failed due to the crushing of the concrete at mid-span. However, while after crushing of concrete, the load was dropping, at 345 kN, another loud noise could be heard. This noise was generated by the rupture of one of the internal tensile reinforcement bars. Another bar ruptured subsequently at 170 kN. Clearly, the rupture of these bars was due to the large deformations experienced by the test beam. Fig.4-48 shows the beam after failure while Fig.4-49 shows the ruptured steel bars at the bottom of the beam. The large deformation of this beam was accompanied by a large amount of slippage at one end of the laminate. This can be seen in Fig.4-50 where the end slippage is almost 85 mm. It is important to observe that despite delamination, the beam continued to carry increased load. Hence, the slippage did not prevent the CFRP from resisting the applied load.

Figure 4-48: Complete plastic behaviour of Beam LA901

Figure 4-49: Tension steel ruptured in Beam LA901

Figure 4-50: Slippage happened in Beam LA901

A close examination of the load mid-span deflection curve in Fig.4-51 indicates that the delamination did not affect the stiffness of the beam because if we examine the slope of the curve before and after delamination, it is basically the same, i.e. if we ignore the dropping in the load, the curve appears almost continuous. Fig. 4-52 shows the deflected shape of the beam at various load levels. We observe a sharp curvature in the mid-span region but a much smoother and smaller curvature near the ends. The large curvature at mid-span is due to the higher moment as well as the more extensive cracking and delamination.

Figure 4-51: Load mid-span deflection of Beam LA901

Figure 4-52: Deflected shapes of Beam LA901 at various load levels

Fig.4-53 shows the variation of longitudinal strain in the FRP laminate. Here again, as in the case of previous anchored laminate, the strain diagrams have a relatively steep slope near the ends, with a much gentler slope near mid-span. However, the plateau region for this beam seems shorter, which would signify a shorter debonding region, and this may be attributed to the presence of the anchors.

Figure 4-53: Strain profiles of CFRP in Beam LA901 at various load levels

4.2.10 Beam SN901

Beam SN901 was strengthened with three layers of sheet, each layer being 83.3 mm wide and 0.165 mm thick. Notice that the thickness refers to the sheet and not the composite, which comprises the sheet and the epoxy. The total cross-sectional area of the sheet was 41.25 mm^2 . The beam did not have anchors and was loaded in three point bending similar to the other beams.

The first crack in this beam appeared at almost 40 kN and as the load was increased at 316.9 kN the loud noise of delamination could be heard, and the load dropped to 273.4 kN. Upon further increasing the load, scratching noises could be heard, indicating additional delamination at various locations. The load was eventually increased to 334.4 kN after delamination and then it failed due to crushing of concrete in the compression zone. Fig. 4-54 shows the beam after failure.

Figure 4-54: Fully plastic behaviour of Beam SN901

Fig.4-55 shows the load mid-span deflection of this beam. Although the beam could carry a higher load than the delamination load, we will see in the next chapter whether delamination occurred too prematurely and the failure load corresponds to the ultimate capacity of the unstrengthened beam assuming strain hardening.

Figure 4-55: Load mid-span deflection of Beam SN901

Fig.4-56 shows the deflected shape of the beam at various load levels. The deflected shape of this beam is quite similar to that of the previous beam and in agreement with the expected theoretical response.

Fig.4.57 shows the variation of longitudinal strain along the sheet. Notice the presence of a gentler slope near the ends, but the rather steep slope at almost 1000 mm from the left support. This high strain may be due to lack of proper bond between the FRP and the concrete at this location. The inadequate bond could be due to improper curing and/or application of the adhesive or other fabrication problems. Upon approaching the delamination load, we observe the development of a plateau, which is indicative of loss of bond.

Figure 4-56: Deflected shapes of Beam SN901 at various load levels

Figure 4-57: Strain profiles of CFRP in Beam SN901 at various load levels

4.2.11 Beam SA901

Beam SA901 is the companion beam to Beam SN901 and was retrofitted with anchors. However, as shown earlier in Fig. 3-18, the number of anchors in the mid-span region and near the ends was nearly doubled, compared to the other beams. The purpose was to investigate whether the increased number would further delay delamination and/or prevent slippage.

The beam was loaded and tested the same as the previous beam and the first crack appeared at nearly 60 kN. As the load was increased, it caused further cracking until at 335.1 kN delamination occurred. This delamination load is only about 5% higher than that of the companion beam without anchors. After delamination, the load dropped to 298.5 kN, which is comparable to load drop in the companion beam.

Subsequently, the load was further increased and once it reached 334.6 kN, another loud noise due to delamination in another region occurred, and the load dropped to 316.7 kN, which is greater than the level to which the load dropped after the first delamination. This clearly shows that the increased number of anchors had an effect on the beam behaviour and that the FRP was effectively carrying load after the first delamination at the FRP-concrete interface.

After the second delamination, the load was continued to increase until it reached 353.7 kN and once again another loud noise was heard, signalling further delamination, and the load dropped to 319.7 kN. The load continued to increase until it reached 363.8 kN and the beam failed due to concrete crushing.

Fig.4-58 shows the load mid-span deflection curve of this beam, where the drops in the load due to various delamination stages could be observed. Notice the very large

deflections and high ductility of this beam compared to the companion beam. This beam had almost 10% higher failure load compared to the companion beam and the advent of three delamination stages confirms the effectiveness of the anchors in enabling the sheet

Figure 4-58: Load mid-span deflection of Beam SA901

to resist the applied loads after initial delamination. Fig.4-59 shows the slip at one end of the sheet at failure. Fig.4-60 shows the deflected shape of the beam at various load levels. It appears that the central region of the beam has nearly equal deflection. This is somewhat unexpected but may signify extensive cracking in this region. The FRP longitudinal strain profiles are shown in Fig.4-61 at various load levels. It is rather interesting to observe the high strain values in the mid-span region and the smaller strain values near the ends. Due to the effectiveness of the anchors, this beam appears to be following the expect curvature profile of a cracked reinforced concrete beam. The high strains in the mid-span region are associated with basically the formation of a plastic

hinge, albeit the hinge may not have fully formed before failure. The sharp slope of the curve in the centre region indicates the presence of high shear stresses there.

Figure 4-59: Slippage in Beam SA901

Figure 4-60: Deflected shapes of Beam SA901 at various load levels

Figure 4-61: Strain profiles of CFRP in Beam SA901 at various load levels

4.3 Summary

The test data in this chapter will be further analyzed in the next chapter and the observed strength and behaviour of these beams will be compared with their expected behaviour based on theoretical analysis. It would then be possible to better gauge the performance and effectiveness of the proposed anchor system. Table 4-1 summarize the results from the experimental work. We notice that in each case if we compare the beam with anchors to its companion beam without anchors, both the delamination load and the failure load of the beam with anchors are higher than the corresponding values in the beam without anchors. Hence, the anchors were effective in increasing the flexural capacity of the beam.

Table 4-1: Summary of the delamination, drop and failure loads for the tested beams

Beam	Cracking load (kN)	Load at delamination (kN)	Load drop (kN)	Load at failure (kN)	Type of failure
BC	70	-	-	375.8	Yield of steel
LN90	50	-	-	391.1	stop the test
LN90B	-	346.8	293.8	-	stop the test
LA90C	-	328.8	316.16	367.46	Compression failure *
		329.23	309.33		
		341.02	321.0		
LA90	60	342.62	276.34	372.8	Compression failure
SN90	50	346.7	245.53	326.21	Compression failure
		346.07	317.52		
SA90	60	384.8	319.9	373.13	Compression failure
LN901	60	298.5	262.61	347.7	Compression failure
LA901	60	321.22	273.98	362.5	Compression failure
SN901	60	316.94	273.43	334.4	Compression failure
SA901	60	335.13	298.5	363.8	Compression failure
		337.6	319.7		
		353.7	348.6		

* The steel yielded in all the beams even though the failure was due to crushing of concrete.

Chapter 5:

Analysis and Discussion of Test Results

5.1 General

In this chapter the results of the beam tests will be analyzed and discussed. The objective of the analysis portion of the chapter is to determine the extent to which selective available analysis methods are able to predict the delamination load and the corresponding ultimate capacity of the beams with or without anchors. As stated in the literature review chapter, many methods of analysis are available, but only a selected few have been verified by experimental results. Among these, the methods of Smith and Teng (2001) and of El-Mihilmy and Tedesco (2001) have been compared extensively with both other available methods of analysis and with experimental results from a relatively large number of beams. Hence, we will use these methods to predict the delamination load for the beams tested in this study. By knowing the delamination load, one can establish a lower bound for the ultimate capacity of a strengthened RC beam. However, no attempt will be made to quantify the effect of the anchors on the delamination load or on the beams ultimate capacity.

5.2 Analysis of Test Results

Table 5-1 summarizes the delamination and failure moments of the tested beams. To be able to compare the behaviour of the control beam, which was loaded in four point

bending, with those of the other beams which were loaded in three point bending, the results are expressed in terms of moment rather than load. The table also shows the ultimate moment capacity of these beams calculated by two nonlinear analysis computer programs and by manual calculations using the CSA S806-02 (2002) recommendations. The computer programs are known as **WIZARD** and Response, both of which use the strain compatibility method and assume full bond until failure. In the manual calculations also full bond is assumed up to failure.

The program **WIZARD** was written by Abushoglin (1997) as part of his master's thesis at Carleton University, while Response was written by Benz et al. at the University of Toronto (<http://www.ecf.utoronto.ca/~bentz/r2k.htm>). With respect to the current analysis, there are two main differences between the two programs: **WIZARD** allows the user to define any maximum concrete strain at failure and then calculates the moment capacity of the beam accordingly. Program Response sets the maximum concrete strain at the default value of 0.003, in accordance with the American practice. The other difference between the two programs is in the type of stress-strain relationship used for concrete. However, it is well-known that any reasonable stress-strain relation will give relatively accurate results, particularly for under-reinforced beams. Both programs allow for strain-hardening in steel and therefore the Response results include this effect. For the program **WIZARD**, ultimate moment capacities including or excluding this effect are shown for a wide range of maximum concrete strain values, ranging from 0.003 to 0.005. In the manual calculations $\epsilon_{cu} = 0.0035$ in compliance with the CSA Standard, but the material resistance factors are set equal to 1.0.

Table 5-1: Summary of the delamination and failure moments of the tested beams

Beam	Experimental Moment (kN.m)		WIZARD										Hand calculation kN.m (0.0035)	Response (0.003)
			M _u with strain hardening (kN.m)					M _u without strain hardening (kN.m)						
	Delamination	Failure	0.003	0.0035	0.004	0.0045	0.005	0.003	0.0035	0.004	0.0045	0.005		
CB		187.9	201.70	206.09	210.19	213.79	216.85	191.63	191.63	191.63	191.63	191.63	165.1	209
LN90E	260.07	277.7	281.74	286.8	307.9	313.9	313.0	279.1	283.0	304.1	306.8	307.1	238.0	272
LA90	266.97	279.8												
SN90	230	243.76												
SA90	269.34	279.8												
LN901	223.67	260.75	248.6	252.8	253.2	254.2	252.9	244.6	249.7	249.8	250.2	249.67	222.7	243
LA901	240.92	271.66												
SN901	237.71	250.8	246.4	252.8	253.2	254.2	252.9	242.3	245.8	245.9	245.8	245.82	222.31	243
SA901	251.34	272.85												
LN90		195.53*	307.98	324.92	337.63	348.78	356.94	307.57	323.55	335.50	346.09	353.81	259.9	296
LA90C	246.59	275.593	281.4	296.8	307.9	313.9	313.7	279.1	293.9	304.1	306.8	307.1	259.9	272

- Test was stopped

It should be remarked that in over-reinforced beams, such as the FRP strengthened beams in this study, the value of ϵ_{cu} will affect their ultimate moment capacity; higher ϵ_{cu} values will result in higher moments. Also, strain hardening will increase the ultimate capacity of such beams, but the CSA method does not include strain hardening. Consequently, it is expected that the results of the manual calculations would be conservative.

5.2.1 Ultimate Flexural Capacity

The experimental failure moments of the test beams are presented in column 3 of Table 5-1. We will compare these moments with their corresponding theoretical values based on program **WIZARD**, assuming $\epsilon_{cu} = 0.0035$ and strain hardening. It is contended that the latter theoretical values are the ideal capacity of these beams under the assumption of full bond. Thus, if a beam achieves this capacity, it is assumed that it has not been negatively affected by delamination insofar as its ultimate moment capacity is concerned.

It is clear that the control beam did not achieve its theoretical flexural capacity because it failed at 187.9 kN.m versus its theoretical strength of 206.1 kN.m. As mentioned in the previous chapter, this beam may have failed in flexural-shear mode, i.e. high shear stresses may have contributed to its premature failure.

Beam LN90B delaminated at 260.7 kN.m, but the test was stopped immediately after delamination. It is, however, doubtful that it would have been able to carry a higher load because complete delamination occurred from the support to mid-span, rendering the FRP totally ineffective. The theoretical ultimate moment capacity of this beam is 296.8 kN.m, thus it achieved only 87.8% of its theoretical capacity. By contrast, after the

FRP was re-bonded in this beam and it was retrofitted with anchors, it failed at 275.6 kN.m, or 92.8% of its theoretical capacity. The reason for not achieving its full theoretical strength may be partly the lack of full bond at mid-span since the adhesive could not be easily applied to this region.

Beam LA90, which is the companion to LN90B failed at 279.6 kN.m or 94.2% of its theoretical strength. Hence in comparison to LN90B, which had no anchors, LA90 with anchors, achieved 6.4% higher moment. Similarly if we compare Beams SN90 and SA90, they failed at 244.66kN.m and 279.85 kN.m, respectively. Thus they achieved 83.4% and 95.4% of their respective theoretical capacities. It is clear that the higher capacity of SA90 is due to the anchors because the two beams are otherwise nominally identical. However, detailed statistical analysis may be required to substantiate this claim.

In comparing beams LN901 and LA901; they failed at 260.75 kN.m and 271.86 kN.m, respectively. These are 101.5% and 105.9% of their corresponding theoretical values. Clearly, both values are greater than the selected theoretical values, but the anchors in Beam LA901 did contribute to the 4.4% higher failure moment.

Finally, Beams SN901 and SA901 achieved failure loads of 250.8 kN.m and 272.85 kN.m, respectively. These are 99.2% and 107.9% of their respective theoretical values. In this case the beam with anchors reached 22.05 kN.m higher moments, which is 8.7% of its theoretical ultimate capacity.

From this discussion it seems that in each case the anchors allowed the beams to achieve higher strength, but in the case of the beams with the higher ratio of external reinforcement, the anchors did not enable them to achieve their full theoretical strength.

In the case of the beams with the smaller reinforcement ratio, the actual failure load in all cases exceeds its corresponding theoretical value.

In comparing the actual failure moments of these beams with their corresponding theoretical values based on the CSA S806-02 (CSA 2002) method, with the exception of the control Beam **CB**, all the other beams surpassed their theoretical values. Of course, the design capacities of these beams based on the CSA Standard would be less than the values shown in column 14 of Table 5-1 because the values in table are nominal strength values assuming the material resistance factor to be 1.0.

As for the two computer programs, their results are comparable, assuming the same maximum concrete strain and strain-hardening. For example, for $\epsilon_{cu} = 0.003$, **WIZARD** predict for Beam **CB** an ultimate moment capacity $M_u = 206$ kN.m vis-à-vis 209 kN.m predicted by Response. These are both higher than the observed failure moment of 187.9 kN.m. The reason may be the high shear stresses, as mentioned in the preceding chapter, and their contribution to the early failure of the beam. It may be recalled that after testing of this beam, the loading was changed from four point bending to three point bending to reduce the shear/moment ratio and the remaining beams were retrofitted with external stirrups to guard against premature shear failure.

For Beam **LN90B** the theoretical ultimate load, based on the two computer models, is approximately 272-296.8 kN.m, but delamination occurred at 260 kN.m. Hence, the beam did not reach its theoretical ultimate capacity, but once it was retrofitted with anchors, and denoted as Beam **LA90C**, it reached an ultimate moment of 275.6 kN.m, which is well within its expected failure load. This means that the anchors enabled the beam to essentially achieve its theoretical capacity. If we assume the correct

theoretical moment capacity of this beam to be 296.8 kN.m, as predicted by program **WIZARD** for an assumed $\epsilon_{cu} = 0.0035$ and strain hardening, the beam reached only 92.85% of its theoretical strength. The latter value may be reasonable because after Beam **LN90B** delaminated, it was re-bonded, but it was not possible to apply the adhesive along the entire debonded length. Thus, Beam **LA90C**, which is the re-bonded beam retrofitted with anchors, was in fact partially debonded near the mid-span. This may explain why it did not achieve its full theoretical strength.

5.2.2 Delamination Load

The delamination loads, P_{del} , of the test beams are shown in column 2 of Table 5-2. In each case, we will compare the delamination load of the beam without anchor to that of its companion beam with anchors.

Table 5-2: Summary of delamination, drop and failure loads for the tested beams

Beam	P_{del} (kN)	P_{drop} (kN)	$P_{failure}$ (kN)	ΔP_{drop} (kN)	$\Delta P_{drop}/P_{del}$ %	$P_{failure}/P_{del}$ %
CB	-	-	375.8	-	-	-
LN90B	346.8*	293.8	-	53	15.28	-
LN90	341.7	276.2	371.3	40	11.71	108.31
SN90	346.8	245.5	326.2	101.3	29.21	94.06
SN91	334.3	279.4	371.1	40	11.97	109.07
LN901	298.5	262.6	347.7	35.9	12.03	116.48
LA90C	321.2	273.9	362.5	47.22	14.70	112.86
SN901	316.9	273.4	334.4	43.5	13.73	105.52
SA901	315.1	278.5	363.4	50.4	16.01	113.55
LA90C	328.8	316.2	367.5	12.6	3.8321	111.77

* Test was stopped

Beam **LN90B** delaminated at 346.8 kN while the companion beam with anchors, i.e. **LA90**, delaminated at 342.6 kN. Hence in this case the presence of anchors did not increase the delamination capacity of the beam. These values are practically equal even though the beam with anchors has a slightly (1.1%) lower delamination moment.

As for Beams **SN90** and **SA90**, they delaminated at 346.8 kN and 384.8 kN, respectively. In this case the anchors increased the delamination capacity of the beam by 11%. Similarly, **LN901** and **LA901** delaminated at 298.5 kN and 321.2 kN, respectively. Thus, the beam with anchors had 7.6 % higher delamination load. Finally, comparing the delamination moments of **SN901** and **SA901**, i.e. 316.9 kN and 335.1 kN, respectively, we observe that the beam with anchors delaminated at 5.7% higher load. It can, therefore, be stated that in general the anchors were able to delay delamination increase in delamination load. As stated earlier, proper statistical analysis would be needed to verify the extent of the effect of the anchors on the strength and deformations of these beams.

5.2.3 Load Drop after Delamination

The drop in load after delamination signifies the loss in the flexural stiffness of the beam caused by debonding. The magnitude of this drop is indicative of the extent of delamination and of the relative contribution of the external FRP to the flexural stiffness of the beam. The magnitude of this drop, ΔP_{drop} , for all the beams is shown in column 5 of Table 5-2

In comparing Beams **LN90B** and **LA90**, we observe from column 5 of Table 5-2 that their loads dropped 53 kN and 66.3 kN, respectively. Hence, as remarked earlier, in the case of Beam **LA90**, the anchors did not contribute either to a higher delamination

load or to a greater stiffness. It is believed that there might have been some flaw in the application of the laminate to this beam.

As for Beams SN90 and SA90, their loads dropped 101.3 kN and 64.9 kN, respectively. Hence due to the anchors, as column 6 of Table 5-2 indicates, the drop in load relative to the delamination load, i.e. $\frac{\Delta P_{drop}}{P_{del}}$, for these beams were 29.21% and 16.87%, which indicates that the extent of delamination in the beam with anchors was smaller. In other words, the delamination might have been over a smaller length of the FRP-concrete interface because the released stresses were absorbed by the FRP-anchor interface.

Next let us examine Beams LN901 and LA901 which had delamination loads of 298.5 kN and 321.2 kN, respectively. Once again the anchor delayed delamination and increased the delamination load by 7.6%. If we look at the extent of the load drop for the two beams (column 5 of Table 5-2), they are 35.9 kN and 47.2 kN, respectively. Hence, in this case the beam with anchors delaminated over a slightly higher length, and the anchors were not able to prevent the propagation of delamination.

Finally, we compare Beams SN901 and SA901, which have delamination loads of 316.9 kN and 335.1 kN, respectively. We can observe that the anchors, in Beams SA901 delayed delamination and increased the delamination load by 5.7%. The drops in load for these beams were 43.5 kN and 36.6 kN, respectively; therefore, the anchors only marginally limited the propagation of delamination. From the preceding discussion it is clear that generally the anchors delayed delamination and increased the delamination load, but they did not substantially limit the propagation of the delaminated zone along the concrete- FRP interface.

If we compare the failure load of each beam to its delamination load (see column 7 of Table 5-2), then it is clear that the beams with higher ratio of FRP sheet reinforcement had lower failure loads than their delamination loads, irrespective of the presence of anchors. This behaviour is unexpected because the width of the interface between the FRP and the concrete in these beams is greater than in the corresponding beams reinforced with the laminate. The beam with laminate, i.e. LA90 had a failure load which is 8.8% higher than its delamination load.

In the beams with lower amount of external sheet or laminate reinforcement, the failure load was always higher than the delamination load, irrespective of the presence of anchors. This indicates that in these beams the delamination gradually spread from the centre of the beam towards the supports. Thus the amount of external reinforcement has a major influence on the delamination load and on the ultimate moment capacity of beams externally strengthened with CFRP. Consequently, any method of analysis which is intended to predict the delamination load must include this parameter.

5.2.4 Load - Deflection Behaviour

The load mid-span deflection curves of the test beams are shown in this section. In these figures are also shown the predicted deflection by the computer program **WIZARD**. The **WIZARD** results are presented as reference because they are based on the assumption of full bond and strain compatibility. If an experimental curve agrees with the corresponding **WIZARD** prediction, it may be concluded that partial delamination did not have a noticeable effect on the over-all behaviour of the beam.

Fig.5-1 shows the load mid-span deflection curves of the control Beam CB at different load levels. Recall that this beam did not reach its full theoretical strength, but

the predicted and measured responses up to the point where the load begins to drop agree rather well. This means that program **WIZARD** predicts the flexural behaviour of this particular concrete beams well.

Figure 5-1: Comparison of the experimental and theoretical load mid-span deflection curves of control Beam CB

Fig.5-2 shows the load mid-span deflection curve of Beam LN90. The testing of this beam was stopped before the failure of the beam due to possible shear failure. We can observe that the calculated response by **WIZARD** agrees reasonably well with the corresponding experimental response. The small difference between the two at high load levels may be due to the effect of shear, which **WIZARD** does not account for.

Figure 5-2: Comparison of the experimental and theoretical load mid-span deflection curves of Beam LN90

Figs.5-3 shows the predicted and experimental load mid-span deflection curves of Beam LN90B. It may be recalled that this beam experienced delamination at 347 kN and the test was stopped at this load. The two curves are almost identical up to almost 280 kN, which corresponds to yielding of the tension steel according to program WIZARD. From this point onward the two curves do not fully agree. It appears that yielding in the experiment did not commence until almost 315 kN, and after yielding, due to tension- stiffening, the beam does not exhibit a plateau as predicted by the computer program. However, at delamination the two curves practically coincide, which implies that tension- stiffening might have been exhausted at that load level. Note that the predicted strength is much higher than the experimental strength because the former assumes the FRP to be fully bonded up to failure.

Figure 5-3: Comparison of the experimental and theoretical load mid-span deflection curves of Beam LN90B

The predicted and experimental load mid-span deflection curves of Beam LA90C are shown in Fig.5-4. Recall that this is actually the same beam as Beam LN90B, but with its FRP rebonded and being retrofitted with anchors. Due to the incomplete rebonding of the FRP, its stiffness and strength could not be fully restored; therefore, the theoretical strength and stiffness are higher after yielding. It is, however, important to observe the rather ductile behaviour of this beam. This behaviour is quite different than the behaviour of a typical reinforced concrete beam, such as the control beam in this study, because the load-deflection curve does not exhibit a descending branch. We must also remember that after applying the FRP, this beam is over-reinforced, which is theoretically expected to fail in a brittle manner. In fact, their response is highly ductile and consequently the fear of brittle failure in such beams may not be warranted.

Figure 5-4: Comparison of the experimental and theoretical load mid-span deflection curves of Beam LA90C

Fig.5-5 shows the load-deflection curves of Beam LA90. In this case the theoretical yield load is lower than the actual yield load, but up to the yield point the two curves agree very well. However, after delamination, the two curves differ from each other because the theoretical curve is based on full bond, but in the experiment, the FRP debonded in the mid-span region. Notice again the excellent ductility of this beam, with a maximum deflection of 100 mm, which is 1/30 of the span. The gap in the experimental curve is due to the mal-function of the LVDT 's or the data acquisition system.

Figure 5-5: Comparison of the experimental and theoretical load mid-span deflection curves of Beam LA90

Fig.5-6 shows the load mid-span deflection curves of Beam SN90. In this case up to the delamination load the predicted and experimental curves agree quite well. Notice that , the experimental curve shows a sudden drop in load but then it increases under increased displacement. This means that the initial delamination was only partial, albeit over a relatively long length in the mid-span region. The positive slope of the curve after delamination is due to the partial bonding of the FRP and some strain- hardening in the steel. Notice that the maximum deflection before it begins to unload is only 90 mm, which is comparable to the previous beam. This beam exhibits a descending branch, which signifies complete loss of stiffness and full debonding.

Figure 5-6: Comparison of the experimental and theoretical load mid-span deflection curves of Beam SN90

Fig.5-7 shows the load-deflection curves of Beam SA90. In this case up to delamination there is close agreement between the calculated and the observed responses. Since this beam had anchors, it means that the FRP remained fully bonded up to delamination. It appears that the delamination load is very close to the theoretical strength of the beam, which shows the effectiveness of the anchors. After delamination, as expected, the two curves differ because the theoretical analysis is based on the assumption of full bond. This beam reached a maximum deflection of almost 70 mm, thus exhibiting good ductility.

Figure 5-7: Comparison of the experimental and theoretical load mid-span deflection curves of Beam SA90

Fig.5-8 shows the load deflection curves of a section 1770 mm from the left-support of Beam LN901. Recall that the LVDT's could not be placed at mid-span in the case of this beam due to the presence of the external shear reinforcement. Here again there is good agreement between the theoretical and experimental curves up to delamination, but thereafter the two do not agree. Notice that despite the lack of anchors, *this beam failed in a ductile fashion*. This may be attributed to the incomplete debonding and the ability of the laminate to resist load at its end region after delamination at the mid-span region. It also implies that in the case of this beam anchors might not have been necessary.

Figure 5-8: Comparison of the experimental and theoretical load deflection curves of Beam LN901 [Deflection at 1770mm from left support]

The load mid-span deflection curves of Beam LA901, the companion of Beam LN901, are shown in Fig.5-9. Notice the very close agreement between the theoretical and experimental curves up to delamination. This implies that the anchors contributed to full bond, and the beam basically achieved its theoretical strength at delamination. However, the effectiveness of the anchors is better demonstrated by the high failure load and positive stiffness of the beam after delamination. Notice also the rather large mid-span deflection of this beam at failure and its ductile failure mode.

Figure 5-9: Comparison of the experimental and theoretical load mid-span deflection curves of Beam LA901

The load mid-span deflection curves of Beam SN901 are shown in Fig.5-10. The two curves agree well up to delamination, but after that they are different. Notice, however, that the delamination in this beam did not lead to a significant drop in load. This implies that the delamination was confined to a small region at mid-span. After initial delamination, the beam continued to carry higher load and eventually reached its theoretical failure load at a maximum deflation of 80 mm. The results of this beam and those of Beam SN90, both of which are reinforced with a CFRP sheet, seem to indicate that delamination in FRP sheet reinforced beams propagates more gradually compared to beams reinforced with laminates.

Figure 5-10: Comparison of the experimental and theoretical load mid-span deflection curves of Beam SN901

Finally, the load mid-span deflection curves of Beam SA901 are shown in Fig.5-11. It may be recalled that this beam was retrofitted with more anchors than the other beams. The effect of these extra anchors can be seen in the multiple delamination stages, each represented by a load drops in Fig.5-11. We observe that the magnitude of the initial load drop is smaller in this beam compared to the previous beams. Furthermore, the beam exhibits a highly ductile response and it experienced over 120 mm of mid-span deflection at failure. It also surpassed its theoretical failure load. The desirable behaviour of this beam indicates the need for a greater number of anchors compared to the numbers used in the other beams. However, determining the correct number of anchors is a task which requires further investigation. At this juncture it could

be stated that the proposed anchor system would be effective, provided a proper number and spacing for them could be selected

Figure 5-11: Comparison of the experimental and theoretical load mid-span deflection curves of Beam SA901

Since the program **WIZARD** assumes full bond and it is not able to delaminate the loss of stiffness caused by delamination, it analyzes the beams as over-reinforced members predicting the typical brittle failure mode.

5.2.5 Strain Profiles

In order to gauge the amount of strain experienced by the FRP at various load levels, the strain profiles of the test beams at their mid-span sections are plotted in this section. In addition, to obtain the precise value of the strain at different load levels, the values are shown in a table for each beam under its strain profile diagram. Since all the

strain gauges did not always function at every load level, the strain data are plotted based on the recorded strains in concrete, steel and FRP. The dashed lines in the strain profile diagram indicate that the plotted profile is based on one or two measured strains only. In the case of one point, it is assumed that the line passes through the same neutral axis position as the other lines.

Fig.5-12 shows the strain profile of Beam CB at mid-span. This beam failed at 375.8 kN, therefore, the strains could be measured up to 90% of the failure load. Notice that the measured linear profiles agree with the well-known assumption of plane-section remains plane after bending. The maximum strains in the concrete and the tensile steel reached 0.00107 and 0.00239, respectively, which means that the steel had yielded but the measured concrete strain was not large compared to the expected failure strain of 0.0035. The preceding low concrete strain further corroborates the observation that this beam may have actually failed in shear.

Figure 5-12: Strain profile of Beam CB through its depth at different load levels

Table 5-3: Magnitude of strain values (micro-strain) plotted in Fig.5-12

Depth (mm)	Load (kN)		
	40	80	200
0	122	362	905
40	99	271	674
365	-85	-465	-1201
400	-104	-544	-1403

The strain profiles for Beam LN90 are shown in Fig.5-13. This beam was loaded up to 391.1kN, but the test was stopped because it appeared that it might prematurely fail in shear. In this case, the maximum strains in the concrete, the tension steel and the FRP laminate are 0.00117, 0.00285 and 0.00324, respectively. Hence, both the concrete and the FRP strains are relatively small compared to their failure strains, but the steel strain is well above its yield strain. This implies that provided shear failure could be prevented, the beam would be able to carry higher load unless the FRP delaminates.

Figure 5-13: Strain profile of Beam LN90 through its depth at different load levels

Table 5-4: Magnitude of strain values (micro-strain) plotted in Fig.5-13

Depth (mm)	Load (kN)			
	40	80	200	391
0	87	228	600	1176
40	66	153	386	734
365	-102	-449	-1356	-2855
400	-120	-515	-1543	-3242

The strain profiles in Fig.5-14 are for Beam LN90B, which is the same Beam as LN90, but after being retrofitted with external stirrups. After the retrofit, delamination occurred at 346.8 kN, which is lower than the maximum load achieved in Beam LN90. The maximum concrete, tension steel and FRP strains at delamination were 0.00107, 0.0024 and 0.00272, respectively. The premature delamination of this beam may have been caused by the damage due to the previous loading and unloading. Notice that the FRP strain at delamination is well below the value of .004 specified by the CSA Standard S806-02.

Figure 5-14: Strain profile of Beam LN90B through its depth at different load levels

Table 5-5: Magnitude of strain values (micro-strain) plotted in Fig.5-14

Depth (mm)	Load (kN)			
	40	80	200	346.8
0	108	223	608	1069
35	75	152	414	738
365	-240	-522	-1416	-2391
400	-274	-594	-1610	-2723

The strain profiles for Beam SA90 are plotted in Fig. 5-15. This beam delaminated at 384.8 kN and the concrete, steel and FRP strains at 379.3 kN are 0.00281, 0.00194 and 0.0024, respectively. It appears that the beam had not even reached yield when delamination occurred, and once again the FRP strain is far from the limit of 0.004 specified by the CSA Standard.

Figure 5-15: Strain profile of Beam SA90 through its depth at different load levels

Table 5-6: Magnitude of strain values (micro-strain) plotted in Fig.5-15

Depth (mm)	Load (kN)			
	40	80	200	379.29
0	102	239	701	2807
30	82	184	591	2416
365	-147	-474	-630	-1945
400	-170	-543	-757	-2401

Fig.5-16 shows the strain profiles of Beam LA90, which delaminated at 342.6 kN. The concrete, steel and FRP strains corresponding to delamination are approximately 0.0024, 0.0047 and 0.0053, which are high compared to the corresponding value for Beam SA90. It is more interesting to examine the same strains after delamination for a load slightly higher than the delamination load. They are 0.004, 0.0078 and 0.0089, respectively. These strains are significantly higher than the corresponding strain just before delamination; even though the difference between the corresponding loads is less than 4.0 kN. Hence, the loss of stiffness caused by delamination could be very significant.

Figure 5-16: Strain profile of Beam LA90 through its depth at different load levels

Table 5-7: Magnitude of strain values (micro-strain) plotted in Fig.5-16

Depth (mm)	Load (kN)					
	40	80	200	324.9	341.6	345.37
0	121	237	711	1719	2404	4029
30	94	182	539	1304	1824	3057
365	-201	-436	-1391	-3328	-4655	-7801
400	-230	-499	-1592	-3812	-5331	-8935

Next let us consider the strain profiles of Beam LA901 as plotted in Fig.5-17. This beam delaminated at 321.2 kN and the concrete, steel and FRP strains at this load are 0.0028, 0.0066 and 0.0075, respectively. Notice that the steel has well passed its yield strain while the FRP has reached almost 50 % of its maximum capacity. After delamination, the load was further increased in the test. At 311.245 kN after delamination, the preceding strains reached 0.0057, 0.0135 and 0.0153. Observe that the concrete strain is significantly greater than the maximum value of 0.0035 specified by many design standards, while the FRP strain is close to its rupture strain. The letter D in the legend of Fig.5-16 and any subsequent figure signifies the loads applied after delamination.

Figure 5-17: Stain profile of Beam LA901 through its depth at different load levels

Table 5-8: Magnitude of strain values (micro-strain) plotted in Fig.5-17

Depth (mm)	Load (kN)				
	40	80	200	321.22	311.245
0	132	287	777	2788	5725
35	91	195	551	1891	3883
365	-293	-677	-1588	-6567	-13485
400	-334	-769	-1814	-7464	-15328

In the companion Beam to LA901, i.e. LN901, which had no anchors, delamination occurred at 298.5 kN and the corresponding concrete, steel and FRP strains were 0.0023, 0.005 and 0.0063. Notice that the delamination load of LA901 is higher, therefore, it experienced greater strain compared to LN901. These results are not unexpected because as long as full bond is maintained, the beam will experience higher strain under higher load. It can be concluded from these strains that the anchors were able to delay delamination. After delamination, the strains in the two beams for the same load

level are comparable. It means that in the delaminated region, the anchors do not reduce strain significantly.

Figure 5-18: Strain profile of Beam LN901 through its depth at different load levels

Table 5-9: Magnitude of strain values (micro-strain) plotted in Fig.5-18

Depth (mm)	Load (kN)					
	40	80	200	279.83	298.5	316.017
0	116	297	891	1520	2350	5967
30	86	215	669	1183	1701	4319
365	-246	-699	-1797	-2579	-5544	-14077
400	-279	-795	-2055	-2972	-6301	-15999

The strain profiles for Beam SN901 are shown in Fig.5-19. This beam began to delimitate at 316.9 kN and the corresponding concrete, steel and FRP strains are 0.0026, 0.0062 and 0.007. If we examine the same strains in the companion Beam SA901 (Fig.5-20) at 313.2 kN, we will observe that they are 0.0026, 0.0027 and 0.0032, which shows that the steel and FRP strains are significantly lower, but after delamination these strains

increased to 0.005, 0.0071 and 0.0083, respectively. Again it appears that the anchors do not have much influence on the strain profile in the delaminated region.

Figure 5-19: Strain profile of Beam SN901 through its depth at different load levels

Table 5-10: Magnitude of strain values (micro-strain) plotted in Fig.5-19

Depth (mm)	Load (kN)					
	40.923	80.58	200.2674	283.1151	316.94	321.29
0	120	284	780	1426	2648.8	4850
30	91	198	566	1035	1922.3	3520
365	-237	-760	-1823	-3333	-6190	-11334
400	-272	-861	-207	-3789	-7037	-12885

Figure 5-20: Stain profile of Beam SA901 through its depth at different load levels

Table 5-11: Magnitude of strain values (micro-strain) plotted in Fig.5-20

Depth (mm)	Load (kN)					
	40.03	79.16	199.513	313.22	335.125	315.22
0	94	283	868	2642	5013	5926
30	79	227	704	2202	4017	4749
365	-85	-401	-1128	-2720	-7105	-8399
400	102	-467	-1319	-3234	-8267	-9774

From the strain profiles, it can be concluded that once delamination occurs at given point, the presence of the anchors does not substantially affect the longitudinal strains or curvature of the beam at that location. This is not unexpected because the anchors do not contribute to the flexural stiffness of the beam. Hence, for an anchor to be effective, it must delay delamination limit from propagation.

5.3 Interfacial Stresses

It is well-known that at the interface of the FRP laminate/sheet and concrete both normal (peeling) stresses σ_z and longitudinal shear stresses τ_{xz} exist as illustrated in Fig.5-21. Fig.5-22 shows schematically how these stresses vary along the interface as reported by El-Mihilmy and Tedesco. Experimentally it is difficult to assess σ_z , but τ_{xz} can be evaluated. Although delamination is caused by the combined effects of shear and normal stresses, the shear stress tends to dominate this phenomenon.

In this study strain gauges were installed on the FRP along its length and the FRP longitudinal strains were measured as reported in the previous chapter. As follows, we can use the measured strain values to determine the variation of shear stresses at the FRP-concrete interface.

Figure 5-21: Normal and shear stresses acting at the FRP-concrete interface
(El-Mihilmy and Tedesco 2001)

Figure 5-22: Schematic variation of normal and shear stresses distribution at the FRP-concrete interface (El-Mihilmy and Tedesco 2001)

5.3.1 Interfacial Shear Stress Calculations

To relate the shear stresses to the measured strains, consider the free-body diagram of an infinitesimal element dx of the FRP as shown in Fig.5-23. Let us assume the FRP to have a unit width and a thickness t_f . If we consider the equilibrium of the longitudinal forces acting on this element, we obtain

Figure 5-23: Free body diagram of an infinitesimal element of FRP

$$(N_f + dN_f) - N_f - \tau_{xz} dx = 0 \quad (5-1)$$

or

$$\tau_{xz} = \frac{dN_f}{dx} \quad (5-2)$$

where N_f is the longitudinal force resisted by the FRP element. Since FRP is a linear elastic material.

$$N_f = E_f \varepsilon_f t_f \quad (5-3)$$

where ε_f is the longitudinal strain, E_f = the elastic modulus and t_f = the thickness of the FRP laminate.

Substituting for N_f from Eq.5-3 into Eq.5-2, we obtain

$$\tau_{xz} = E_f t_f \frac{d\varepsilon_f}{dx} \quad (5-4)$$

We can see in Eq.5-4 that the derivative term is the slope of the longitudinal strain diagram. We will estimate the slope by assuming a linear variation between any two consecutive measured strains on the FRP, and use Eq.5-4 to calculate the shear stress. Notice that in the case of delamination, the slope of the longitudinal strain diagram would be theoretically zero because the FRP strain along any delamination region would be constant. Using the described procedure, the shear stress diagrams for the test beams were plotted and are discussed in the following section.

5.3.2 Interfacial Shear Stress Diagrams

Figs.5-24 and 5-25 show the variation of τ_{xz} in Beam LN90 at various load levels.

For clarity the figure is divided into two parts, where we can clearly see the relatively

high shear stresses near the ends of the sheet and the much smaller values in the central region. The preceding variation is in general agreement with the theoretical shape shown earlier. The maximum shear stress at $P = 391.1 \text{ kN}$ is approximately 2.2 MPa , but this stress was not sufficiently high to cause delamination.

Figure 5-24: Shear stress along the CFRP laminate in Beam LN90

Figure 5-25: Shear stress along the CFRP laminate in Beam LN90 at failure

Figs 5-26 and 5-27 show the variation of the shear stress in Beam LN90B. We notice that initially the distribution is as theoretically expected but under $P=346$ kN, which is the delamination load, the shear stress at one end exceeds 3.5 MPa, while in other regions there is a noticeable increase in this stress in the regions away from the FRP ends. The higher stresses may be due to the fact that delamination in any region would increase the stresses in the remaining bonded regions in order compensate for the stresses released by delamination. We will later use a theoretical model to predict the delamination load for this and the other beams. Due to the failure of most strains gauges on the FRP after delamination of LN90B, the strains in Beam LA90C could not be reliably recorded; therefore, the results for this beam are not shown.

Figure 5-26: Shear stress along the CFRP laminate in Beam LN90B

Figure 5-27: Shear stress along the CFRP laminate in Beam LN90B at failure

The shear stress variation along the laminate in Beam LA90 is shown in Figs 5-28 and 5-29. It is clear from Fig.5-29 that up to $P=320$ kN, or 93% of $P_{\text{delamination}}$, the shear stress varies in accordance with the linear elastic theory prediction, but thereafter, it deviates from the elastic theory. In particular, we observe that on the left side of Fig.5-28, the stress in the central region drops as the load increases from 320 kN to 342 kN., which may indicate partial declamation in this region. We also notice high shear stress away from the right end of the sheet, which may also be due the release of stresses caused by delamination. Notice that in this case the maximum shear stress exceeds 6.5 MPa, which is rather high. However, the entire 6.5 MPa may not be in fact resisted by the FRP-concrete interface because the FRP-anchor interface will also resist some of the shear. The higher shear strength of the adhesive, compared to that of concrete, will enable the FRP-anchor interface to resist higher shear stress than the FRP-concrete interface. At this

junction no theoretical method exists for the apportioning of the stress to the two interfaces.

Figure 5-28: Shear stress along the CFRP laminate in Beam LA90

Figure 5-29: Shear stress along the CFRP laminate in Beam LA90 at failure

Figs. 5-30 and 5-31 show the τ_x variation for Beam SN90. Here we observe a dramatically different type of variation, with the high shear stresses not being just confined to the end regions of the sheet. Since this pattern is noticeable at the early as well as the advanced stages of loading, at first glance it may indicate poor bonding at the FRP-concrete interface, but the graphs are anti-symmetric and therefore it may be reflective of the actual behaviour of the sheet. Further analysis is needed to fully investigate this behaviour. The high shear stresses in the central region may be caused by early delamination or lack of proper bond in the adjacent zones. Notice, however, that the maximum shear stress in this case is only 1.4 MPa at $P = 320$ kN, which is almost 92.5 % of the delamination load. This stress is relatively low in comparison to the maximum shear stress of over 6.0 MPa recorded in the previous beam. Since the delamination shear stresses of the latter two beams are significantly different in magnitude, it may be difficult to develop a delamination model purely based on the magnitude of the maximum longitudinal shear. To arrive at a rational model, the role of the transverse normal stresses must also be considered.

Figure 5-30: Shear stress along the CFRP laminate in Beam SN90

Figure 5-31: Shear stress along the CFRP laminate in Beam SN90 at failure

Figs. 5-32 and 5-33 show the shear stress variation along the FRP interface in Beam SA90. In this case the variation is close to what is expected, particularly under higher loads. At $P=384$ kN, which is practically the delamination load, we notice on the right side of the sheet an increase in shear stress away from the end region. This signals the initiation of delamination and in fact in the experiment, delamination was observed only on the right-hand side of the sheet. The maximum shear stress in this case exceeded 4.0 MPa, which is significantly higher than the maximum shear stress in the companion beam without anchors. Based on the results of these two beams, the anchors may be responsible for the ability of the Beam SA90 to resist such high shear stresses.

Figure 5-32: Shear stress along the CFRP laminate in Beam SA90

Figure 5-33: Shear stress along the CFRP laminate in Beam SA90 at failure

The shear stress variation along the FRP-concrete interface in Beam LN901 is shown in Figs.5-34 and 5-35. The variation is rather unusual because at one end it seems to be quite high while at the other end it is relatively small. It would appear that the right-end of the laminate might not have been properly bonded or the strain gauges might not have functioned properly. Due to the rather erratic pattern of these stresses, it is difficult to analyze them. We do, however, notice a maximum shear stress of almost 5.0 MPa at delamination, which is considered to be large.

Figure 5-34: Shear stress along the CFRP laminate in Beam LN901

Figure 5-35: Shear stress along the CFRP laminate in Beam LN901 at failure

The interfacial shear stress variation for Beam LA901 is shown in Figs. 5-36 and 5-37. At lower load levels, as Fig.5-36 indicates, the stress is relatively low in the centre region and high near the ends, but as the load increases and approaches the delamination load ($P=321.2$ kN), the centre regions begin to resist higher stress. In this case the maximum shear stress approaches 8.0 MPa, which is very high. As stated earlier, this may be a nominal stress since in fact the shear is resisted by both the FRP-concrete interface and the FRP- anchor interface, which means that the actual shear stress at each interface may be only half of this value. Nevertheless, we have observed so far that the beams with anchor resist significantly higher interfacial shear stress before delamination than the companion beams without anchors.

Figure 5-36: Shear stress along the CFRP laminate in Beam LA901

Figure 5-37: Shear stress along the CFRP laminate in Beam LA901 at failure

The interfacial shear stress variation for Beam SN901 is shown in Figs. 5-38 and 5-39. Here again, as in the case of Beam SN90, which was similarly reinforced with FRP sheet and no anchors, the stresses vary in a more complex manner, and do not follow elastic theory based on full bond. It is possible that due to the difference in the type of the adhesive and the presence of several plies of sheet, slippage occurs at the various interfaces between the different plies of the sheet and between the sheet and the concrete. The inter-ply slippage would lead to complex variation of the longitudinal strains on the outer ply where the strain gauges were placed. It is particularly important to notice that high shear stress occurs away from the end regions. The maximum shear stress is less than 4.0 MPa, but on the right of the sheet, where the strain gauge near the end failed, the stress might have been higher, thus causing delamination. The complexity of this stress

variation warrants more in-depth investigation about the stresses at the FRP-concrete interface.

Figure 5-38: Shear stress along the CFRP laminate in Beam SN901

Figure 5-39: Shear stress along the CFRP laminate in Beam SN901 at failure

Figs. 5-40 and 5-41 show the interfacial shear stress variation in Beam SA901. It may be recalled that this beam was retrofitted with extra anchors both in the end and centre regions of the sheet. The beam delaminated at 335.1 kN, but the strain gauges failed earlier. At 320 kN, the beam had a relatively small and constant shear in the end regions, but a significant peak in the central region. This implies that the anchors might have delayed delamination in the centre region and therefore reduced the shear demand on the end zones. The multiple peaks in Fig.5-41 occur in the vicinity of the anchors. Thus, if sufficient number of anchors installed, it is possible to distribute the shear stresses more evenly along the interface.

Figure 5-40: Shear stress along the CFRP laminate in Beam SA901

Figure 5-41: Shear stress along the CFRP laminate in Beam SA901 at failure

The preceding results show that the basic flexural behaviour of FRP strengthened beams before delamination can be predicted using basic beam theory and proper constitutive laws for concrete, steel and FRP. However, it is more difficult to assess the distribution and magnitude of the FRP- concrete interfacial shear and normal stresses. Due to the presence of the adhesive, the interface is sensitive to flaws and the associated stress concentrations. Hence, locally very high stresses can develop which would initiate delamination and increase the stress in the adjoining sections. Despite these complexities, it would appear that in general the interfacial shear stresses near the ends of the FRP are higher than in the central regions. This cannot be explained by conventional beam theory, but more advance analysis confirms it. The results also reveal that the anchors allow the FRP to resist much higher local shear stresses before initiation of delamination. In the following section, we will investigate whether two well-known methods of analysis

would be able to predict the advent of delamination in and the consequent failure of the current test beams.

5.4 Further Analysis of Failure Load

In Chapter 2 the methods of El-Mihilmy and Tedesco and of Smith and Teng for predicting the failure load in FRP strengthened beams was presented. It was stated that among the available methods, these have been demonstrated to be the most accurate. Since the methods were presented in full, the reader is referred to Chapter 2 for their details. Between the two methods, the El-Mihilmy and Tedesco method is more detailed because it relies on the analysis of combined stresses, involving both the longitudinal shear and normal stresses and the transverse normal stress (peeling stress). It also uses a biaxial stress failure criterion for concrete in the interface region. The Smith and Teng model relies on more traditional calculations of the shear and moment at the cut-off point of the FRP sheet/laminate and a linear moment-shear interactive diagram.

To further investigate the accuracy of these methods, including the proposed change in the so-called Ψ factor, a number of beams were selected from available literature and were analyzed. Table 5-12 gives the full details of these beams, including their width, b , overall depth, h , amount of tension and compression steel reinforcement, A_s and A'_s , their effective depth, d , the breadth, b_f , thickness t_s and elastic modules, E_f , of the externally bonded FRP, their concrete strength f'_c , span length, L , unsheathed length, L_f , adhesive thickness t_a and adhesive elastic modulus E_a . All the beams were simply supported and tested in either three point or four point bending.

Table 5-13 compares the actual failure load of the Beams LN90, LN901, SN90 and SN901 and their corresponding predicted values by the two methods. It is obvious

the El-Mihilmy and Tedesco (2001) model predicts the failure load of these beams relatively accurately while the Smith and Teng model (2001) gives more conservative results. The mean value of the ratio of the experimental/theoretical failure load based on the El-Mihilmy and Tedesco model is 0.995, with a relatively small standard deviation, while the corresponding value for the Smith and Teng model is 1.17.

However, in applying El-Mihilmy and Tedesco model, the quantity Ψ in Eq. (2-20) of Chapter 2 must not be less than 0.415, irrespective of the magnitude of unsheathed length to beam span (L_f/L) ratio. With this minor modification, Ψ has a bilinear variation. Note, however, that these methods can be only applied to beams without anchors because their scope is limited to such members.

Table 5-1: Full details of beams used in analysing failure load of FRP strengthened beams

Reference	Beam	b, mm	h mm	A_s mm ²	d mm	A_s' mm ²	d' mm	b_f mm	t_f mm	E_f GPa	f_c' MPa	L mm	L_f mm	t_a mm	E_a MPa
Present study	LN90	250	400	1200	365	200	30	100	1.2	165	52.2	3000	474.8	1	11700
	LN901	250	400	1200	365	200	30	50	1.2	165	52.2	3000	474.8	1	11700
	SN90	250	400	1200	365	200	30	166.7	0.495	227	52.2	3000	474.8	1	11700
	SN901	250	400	1200	365	200	30	83.33	0.495	227	52.2	3000	474.8	1	11700
Ahmed& Van Gemert (2000)	AF2	125	225	100.5	200	56.55	20	75	0.334	240	41	1500	200	2.5	7200
	AF2-1	125	225	100.5	150	56.55	20	75	0.334	240	41	1500	150	2.5	7200
	AF3	125	225	100.5	100	56.55	20	75	0.334	240	41	1500	100	2.5	7200
	DF4	125	225	150.8	50	56.55	20	75	0.668	240	40.5	1500	50	2.5	7200
El-Mihilmy & Tedesco (2001)	F31	155	225	942	190	0	0	125	6	200	45.6	2250	50	1.5	280
	A1b	100	100	85	85	56	15	80	1.2	49	59.5	900	20	1	7000
	A1C	100	100	85	85	56	15	80	1.2	49	59.5	900	20	2	7000
	A2B	100	100	85	85	56	15	80	1.2	49	35.7	900	20	1	7000
	A2C	100	100	85	85	56	15	80	1.2	49	35.7	900	20	2	7000
	A3	200	200	308	163	308	37	150	1.3	167	33	2000	150	1	11000
	A5	200	200	308	163	308	37	150	2.6	167	33	2000	150	1	11000
	SM2	320	160	226	110	226	50	300	1	235	36	1100	50	1.5	2000
	SM5	320	160	226	110	226	50	140	1	235	36	1100	50	1.5	2000
	ST4	320	160	226	110	226	50	300	1	380	36	1100	50	1.5	2000
	C11	100	150	100	125	0	0	80	1.6	200	73.9	1100	50	1.6	6700
	B7	75	150	14	130	0	0	50	1.2	150	37	1500	10	2.5	10000
	IA	127	203	142	164	0	0	127	1	34	36.5	2288	25	1	4500
	IIA	127	203	142	164	0	0	127	1	33.4	36.5	2288	25	1	2900
	B2	100	100	85	85	56	15	80	1.2	49	45.1	900	20	2	7000
	B4	100	100	85	85	56	15	60	1.6	49	45.1	900	20	2	7000

Table 5-2: Full details of beams used in analysing failure load of FRP strengthened beams (cont.)

Reference	Beam	b mm	h mm	A _s mm ²	d mm	A _s ' mm ²	d' mm	b _f mm	t _f mm	E _f GPa	f _c ' MPa	L mm	L _f mm	t _a mm	E _a MPa
El-Mihlmy & Tedesco (2001)	B6	100	100	85	85	56	15	80	1.2	119	45.1	900	20	2	7000
	204	155	255	942	220	0	0	125	3	200	56.4	2300	50	1.5	2000
	205	155	255	942	220	0	0	125	6	200	59.7	2300	50	1.5	2000
	208	155	255	942	220	0	0	125	3	200	57.8	2300	50	3	2000
	209	155	255	942	220	0	0	125	6	200	61.4	2300	50	3	2000
	217	155	255	942	220	0	0	125	3	200	58.3	2300	50	6	2100
	218	155	255	942	220	0	0	125	6	200	60.8	2300	50	6	2100
Smith & Teng (2001)	B	205	455	1013	400	253	55	152	6	37.2	35	4575	155	1.5	8500
	B3	100	100	85	84	57	16	30	1.2	49	42.4	900	20	2	11560
	SB1	200	300	402	252	402	48	120	1.4	165	51.2	3600	150	2.1	8500
	SB2	200	300	402	252	402	48	120	1.4	155	52	3600	200	2.4	8500
	SB3	200	300	402	252	402	48	120	1.4	155	52	3600	300	3	8500
	MB1	200	300	402	252	402	48	120	1.4	210	56	3600	150	2.4	8500
	HB1	200	300	402	252	402	48	100	1.4	300	56	3600	150	2.1	8500
	FB1	200	300	402	252	402	48	150	2.4	95	51.2	3600	150	0.4	8500

Table 5-13: Experimental versus the theoretically predicted failure load of FRP

strengthen beams

Reference	Beam	El-Mihilmy and Tedesco			Smith and Teng		
		P_{exp}	P_{th}	P_{exp}/P_{th}	P_{exp}	P_{th}	P_{exp}/P_{th}
Present Study	LN90	367.5	360.3	1.0	367.5	294.0	1.2
	LN901	347.7	343.5	1.0	347.7	294.0	1.2
	SN90	326.2	339.7	1.0	326.2	294.0	1.1
	SN901	334.4	336.6	1.0	334.4	294.0	1.1
Ahmed& Van Gemert (2000)	AF2	83.0	82.3	1.0	83.0	53.5	1.6
	AF2-1	85.7	85.5	1.0	85.7	53.5	1.6
	AF3	96.5	93.6	1.0	96.5	53.4	1.8
	DF4	125.0	125.9	1.0	125.0	61.0	2.1
EL-Mihilmy & Tedesco (2001)	F31	183.3	196.4	0.9	183.3	130.8	1.4
	A1B	46.3	48.3	1.0	46.3	29.1	1.6
	A1C	44.2	48.0	0.9	44.2	29.1	1.5
	A2B	37.8	36.8	1.0	37.8	24.6	1.5
	A2C	38.2	36.5	1.0	38.2	24.6	1.6
	A3	104.8	104.8	1.0	104.8	87.4	1.2
	A5	92.5	89.8	1.0	92.5	87.4	1.1
	SM2	152.8	163.7	0.9	152.8	87.3	1.7
	SM5	113.8	97.5	1.2	113.8	87.3	1.3
	ST4	158.1	164.9	1.0	158.1	87.3	1.8
	C11	58.3	53.3	1.1	58.3	42.1	1.4
	B7	24.3	21.5	1.1	24.3	14.7	1.7
	IA	138.8	123.0	1.1	138.8	51.8	2.7
	IIA	115.6	123.9	0.9	115.6	51.8	2.2
	B2	34.5	41.4	0.8	34.5	26.6	1.3
	B4	35.2	33.7	1.0	35.2	26.6	1.3
	B6	40.0	33.1	1.2	40.0	26.6	1.5
	204	268.5	241.7	1.1	268.5	153.0	1.8
	205	212.3	191.1	1.1	212.3	155.9	1.4
	208	262.7	244.0	1.1	262.7	154.2	1.7
	209	202.5	193.8	1.0	202.5	157.4	1.3
	217	258.6	240.1	1.1	258.6	154.7	1.7
	218	177.5	190.2	0.9	177.5	156.9	1.1
Smith and Teng (2001)	B	250.0	253.5	1.0	250.0	223.2	1.1
	B3	24.6	26.1	0.9	24.6	25.8	1.0
	SB1	142.8	142.6	1.0	142.8	142.9	1.0
	SB2	151.0	155.3	1.0	151.0	143.7	1.1
	SB3	147.8	147.8	1.0	147.8	143.7	1.0
	MB1	159.2	160.8	1.0	159.2	147.2	1.1
	HB1	160.2	162.9	1.0	160.2	147.2	1.1
	FB1	148.8	151.6	1.0	148.8	142.9	1.0
Mean				1.015			1.4302
S. Deviation				0.074			0.371

Results of the analysis are shown in Table 5-13, and illustrated in Fig.5-42 to Fig.5-44. From Table 5-13 and Fig.5-42 it is obvious that the ratio of the experimental/predicted failure load, using the El-Mihilmy and Tedesco model, is close to 1.0 in most cases. The mean and standard deviation of this ratio are 1.015 and 0.074, respectively. Fig.5-43 shows that all the predicted values are within the $\pm 20\%$ of the mean, with the majority being within $\pm 10\%$.

Figure 5-42: Predicted/experimental failure load ratios according to
(El-Mihilmy and Tedesco model)

Figure 5-43: Comparison of the predicted failure loads (kN) according to El-Mihilmy and Tedesco model with their experimental values

Figure 5-44: Predicted anchorage failure loads (kN) according to Smith and Teng model versus the experimental results

Considering the complexity of this problem, the El-Mihilmy and Tedesco method is remarkably accurate and it gives better results than any other method. As Fig.5-44 reveals, the Smith and Teng method results are practically always conservative but in many cases the method grossly underestimates the actual delamination load. Accordingly, it is recommended here that El-Mihilmy and Tedesco model be further investigated to see whether it can be extended to beams with anchors. One feasible approach would be to apportion the interfacial shear to the FRP-concrete interface and the FRP-anchor interface and then analyze the FRP-concrete interface similarly to the beams without anchors. However, the actual implementation of this approach is beyond the scope of the present study.

Chapter 6:

Summary, Conclusions, and Recommendations for Future Work

6.1 Summary

Nine simply supported under-reinforced concrete beams were tested; eight beams were strengthened with externally bonded CFRP sheet or laminate while one beam was not strengthened and used as a control beam. Four of the strengthened beams were retrofitted with a new FRP anchor system to delay delamination of the external FRP reinforcement and to increase the beams strength and ductility. The strengthened beams were tested to failure in three point bending over a 3.0 m span.

The main objective of the study was to examine the effectiveness of the new anchor system. The anchor is Π - shape and is extracted from the well-known NEFMAC grids. The legs of the Π are embedded into the concrete on the tension side of the beam while the cross- bar is bonded to the bottom of the beam and to the FRP sheet/laminate. Due to the deep embedment of the anchors inside the concrete, it is anticipated that they will not pullout. The other test parameters were the type of CFRP external reinforcement and its amount. Two types of CFRP were used, the SikDur 30 laminate or the Wabo

MBrace CF 130 sheet. The beams were strengthened with either 60 mm² or 120 mm² of laminate, or with 41.25 mm² or 82.5 mm² of the CFRP sheet, comprising three plies.

Due to the preliminary nature of the study and due to lack of any design guidelines for such anchors, the number and spacing of the anchors were based on practical considerations. In the case of one beam the number of anchors was increased in both the mid-span and the end regions of the CFRP sheet. To provide insight into the effect of anchor spacing on the delamination load and the ultimate strength of the beam.

The tests were successfully completed, but during the test some modifications become necessary. For example, to prevent premature shear failure of the beams, they had to be retrofitted with external shear reinforcement in the form of steel bandages. It is believed that these modifications would not affect the flexural behaviour of the beams, and the test results confirmed it.

The test beams failure loads were compared with their theoretical ultimate load capacities based on either the CSA S806-02 method or the strain compatibility and nonlinear analysis method employed by the computer programs **WIZARD** and Response. The **WIZARD** calculations were based on an ultimate concrete strain of 0.0035 and strain hardening of steel. In addition, the failure loads of the beams without anchors were compared with their theoretical values based on two well-known methods which are El-Mihilmy and Tedesco (2001) and Smith and Teng (2001)

6.2 Conclusions

From the experimental results and their analysis for the given beams, the following conclusions can be stated:

- 1- In every case, delamination started in the mid-span region and then it propagated towards the supports.
- 2- In only one of the tested beams full delamination occurred in one half of the span; in all the other cases, it was more gradual.
- 3- The advent of delamination was always heralded by a loud noise and by a noticeable drop in the applied load.
- 4- Beams with anchors had generally 5-10% higher delamination load than their companion beams without anchors, but the true significance of the observed increase can be only measured by proper statistical analysis.
- 5- Beams with anchors generally experienced higher deformation at failure compared to their companion beams without anchors.
- 6- Despite the fact that all the strengthened beams were over-reinforced, they failed in a ductile fashion because the failure was due to the crushing of the concrete and the steel had undergone large strains, well past its yield strain.
- 7- Although complete separation of the external reinforcement from the concrete was not observed in any of the beams with anchors, there was substantial slip at one end of the FRP reinforcement.
- 8- It is believed that the partial debonding and end slip contributed to the observed ductile response of the beams.
- 9- Due to the relatively small axial rigidity of the external FRP reinforcement, compared to the axial rigidity of the internal tension steel reinforcement, the external reinforcement did not contribute significantly

to the stiffness of the beam. This was particularly the case once delamination occurred, which always occurred after the steel had yielded.

- 10- The anchors did not directly contribute to the flexural stiffness of the beam. Therefore, once delamination occurred at a section, the concrete and FRP strains increased dramatically, regardless of the presence of the anchors.
- 11- The anchors were effective in limiting the extent of delamination along the interface, thus indirectly contributing to the flexural stiffness of the beam by limiting the extent of cracking along the span.
- 12- All the beams externally reinforced with FRP had failure loads that exceeded their theoretical failure loads based on the CSA S806-02 method, but the control beam failed at a lower load due to excessive shear combined with high moment.
- 13- None of the beams reinforced with the higher FRP reinforcement ratio achieved their theoretical capacities as calculated by program **WIZARD** based on a maximum concrete strain of 0.0035 and strain hardening.
- 14- All the beams reinforced with the lower FRP reinforcement ratio achieved, or surpassed, their theoretical calculated capacities from **WIZARD** or **Response**, while each beam with anchors had a higher failure load than its companion beam without anchors.
- 15- Increasing the number of anchors in one beam enabled it to carry a higher load and to exhibit remarkable ductility than a similar beam with fewer anchors.

- 16- One of the available theoretical methods for predicting the failure load of beams externally strengthened with FRP provided a remarkably close estimate of the actual failure loads of beams without anchors. However, to achieve this accuracy, a minor modification had to be made to the original method
- 17- Since the failure load could be predicted with such accuracy, it may be possible to estimate the strength of a strengthened beam with greater certainty, and to determine the need for anchors.
- 18- Overall the proposed anchors system was found to be effect in delaying delamination and in achieving a more ductile mode of failure, but further investigation is needed to study its applicability to other practical situations.
- 19- The anchors performed equally well for both the sheet and the laminate.

6.3 Recommendations for Future Work

- 1- Test beams with different anchor spacing to determine its effect on the delamination load, ultimate strength and ductility of beams externally strengthened with FRP.
- 2- Modify the current anchor system, for example, by increasing the width of its spine, or cross-bar, and reducing its thickness. This will increase its contact area with the FRP external reinforcement and will enable it to carry greater amount of interface shear.
- 3- Test beams with different anchor configurations. It may be advantages to place the anchors near the end region of the FRP reinforcement only.

- 4- In order to obtain an optimum embedment depth, test anchors with different embedment depths.
- 5- Investigate the extension of the El-Mihilmy and Tadescio method to beams with anchors.
- 6- Develop a full and complete design method for calculating the delamination load, ultimate capacity and ductility of beams externally reinforced with FRP.
- 7- Test beams strengthened with glass FRP or glass FRP anchors to explore the applicability of the proposed anchor system to other FRP materials, particularly glass FRP, which is widely used in practice.
- 8- Test beams with different development and unsheeted lengths for the FRP sheets/laminates.

References

- Ahmed, O. and Gemert, D.V. (1999), "Effect of longitudinal carbon fiber reinforced plastic laminates on shear capacity of reinforced concrete beams," In: Dolan, CW., Rizkalla, SH. and Nanni, A. editors. Proc. of the 4th International Symposium on Fiber Reinforcement Polymers for Reinforced Concrete Structures. Maryland, USA, pp. 933-943.
- Ahmed, O., Gemert, D.V. and Vandewalle, L. (2001), "Improved model for plate-end shear of CFRP strengthened RC beams," *Cement & Concrete Composite*, V.23, pp. 3-19.
- Arduini, M. and Nanni, (1997), "Behaviour of pre-cracked RC beams strengthened with carbon FRP sheets," *Journal of Composites for Construction*, V.1, No.2, May, pp. 63-70.
- Arduini, M. and Nanni, (1997), "Parametric study of beams with externally bonded FRP reinforcement," *ACI Structural Journal*, V.94, No.5, Sep.-Oct., pp. 493-501.
- Bonfiglioli, B., Pascale, G., and Ibarra, Perez J. (2003), "Flexural behaviour of reinforced concrete beams repaired with externally bonded FRP," *Composite in Construction*, pp. 295-300.
- Chajes, MJ., Thomson, TA., Januszka, TF. and Finch, WW. Jr. (1994), "Flexural strengthening of concrete beams using externally bonded composite materials," *Construction and Building Materials*, V.8, No.3, pp.191-201.
- CSA. (2002), "Design and construction of building components with fiber-reinforcement polymers," CSA Standard S806-02, CSA, Rexdale (Toronto). Ontario.
- Duthinh D. and Starnes M. (2001), "Strengthening of reinforced concrete beams with Carbon FRP," Proc. of the International Conference Composites in Constructions-CCC2001, Porto, Portugal, pp. 493-498.
- El-Mihilmy, M. and Tedesco, J. (2001), "Prediction of anchorage failure for reinforced concrete beams strengthened with fiber-reinforced polymer plates," *ACI Structural Journal*, May-June 2001, pp.301-314.
- Gardent, H.N. and Hollaway, L.C. (1998), "An experimental study of the influence of plate end anchorage of carbon fibre composite plates used to strengthen reinforced concrete beams," *Composite Structures*, V.42, pp. 175-188.
- Heffernan, PJ. and Erki, MA.(1996), "Equivalent capacity and efficiency of reinforced concrete beams strengthened with carbon fiber reinforced plastic sheets," *Canadian Journal of Civil Engineering*, V.23, pp.21-29.

- Jansze, W. (1997), "Strengthening of RC members in bending by externally bonded steel plates," Delft University of technology.
- Jones, R., Swamy, R.N. and Charif, A., (1988), "Plate separation and anchorage of reinforced concrete beams strengthened by epoxy bonded steel plates," *Structural Engineering*, V.66, No.5, Mar., pp. 85-94.
- Juvandes, L., Figueiras, J.A. and Marques A.T.(1998), "Performance of concrete beams strengthened with CFRP laminates," *Proc. of 2nd International Conference On Composite In Infrastructure, ICC 98, University Of Arizona, Tucson*, pp. 126-137.
- Lamanna, A.J, Bank, L.C, and Scott, D.W. (2001), "Flexural strengthening of RC beams using fasteners and FRP strips," *ACI Structural Journal*, V.93, No. 3, pp. 368-376.
- Malek, A.M. and Saadatmanesh, H. (1998), "Design equations and guidelines for reinforced concrete beams strengthened with FRP plates," *Second International Conference On Composite Infrastructure, ICCI 98, University of Arizona, Tucson*, pp.603-617.
- Malek, A.M., Saadatmanesh, H. and Ehsani, M.R. (1998), "Prediction of failure load of R/C beams strengthened with FRP plate due to stress concentration at the plate end," *ACI Structural Journal*, V.95, No.2, Mar.-Apr., pp. 142-152.
- Meier, H. (1998). CFK-Schubverstärkungselementer. Schweizer Ingenieur und Architekt, No.43/1998.
- Meier, U. (1995), "Strengthening of structures using carbon fiber/epoxy composites," *Construction and Building Materials*, V.9, No.6, pp. 341-351.
- Neubauer, U. and Rostasy F.S. (2001), "Debonding and model for CFRP-plates as external reinforcement for concrete members," *Proc. of the International Conference Composite in Constructions, CCC2001, Porto, Portugal*, pp.467-472
- Oehlers, DJ (1992), "Reinforced concrete beams with plates glued to their soffits," *Journal of Structural Engineering. ASCE*; V.118, No.8, pp.2023-2038.
- Oehlers, DJ. and Moran, JP. (1990), "Premature failure of externally plated reinforced concrete beams," *Journal of Structural Engineering, ASCE*; V.116, No.4, pp.978-995.
- Oller, E., Salcedo, D.Cobo and Mari, A. (2001), "Flexural strengthening of reinforced concrete beams with externally bonded CFRP laminates," *Proc. of Composite in Constructions, Porto, Portugal, Balkema Publishers*, pp. 473-478.
- Poulsen, E., Bendtsen, L., Mortensen, J. and Ottosen, N.S. (2001), "Anchorage and laps of CFRP strips for the strengthening of RC structural components" *Composite in Construction*, pp.205-209.

Quantrill, R.J., Hollaway, L.C and Thorne, A.M. (1996), " Predications of the maximum plate end stresses of FRP strengthened beams: Part II," Magazine of Concrete Research, V.48, No.177, Dec., pp. 343-351.

Rabinovitch, O. and Frostig, Y.(1999)," Edge effects in retrofitting of concrete beams using fiber reinforced polymer stripes closed form, high-order theory approach ," Proc. of 4th International Symposium Fiber Reinforced Polymer Reinforcement for Reinforced Concrete Structures, PP. 179-191

Raof, M. and Hassanen, M.(2000)," Peeling failure of reinforced concrete beams with fiber-reinforced plastic or steel plates glued to their soffits," Proc. of the Institute of Civil Engineers: Structures and Building; 140(August), pp. 291-305.

Ritchie, PA., Thomas, DA., Lu LW., and Connely, GM.(1991)," External reinforcement of concrete beams using fiber reinforced plastics," ACI Structural Journal, V.88, No.4, pp. 490-550.

Roberts, T.M. and Haji-Kazemi, H. (1989)," A theoretical study of the behaviour of reinforced concrete beams strengthened by externally bonded steel plates," Proc. of Institution of Civil Engineers, Part 2, V.87, Mar., pp. 39-55.

Roberts, T.M. (1989)," Approximate analysis of shear and normal stress concentration in the adhesive layer of plated RC beams, " Structural Engineer, V.67, No.12, pp. 228-233.

Ross, CA., Erome, DM., Tedesco, JW. and Hughes, ML.(1999)," Strengthening of reinforced concrete beams with externally bonded composite laminates," ACI Structural Journal, V.96, No.2, pp. 212-220.

Rostasy, F.S. (1998), Expert opinion n° 98/0322. Assessment if the suitability if CRP plates from the S&P CRP system for use as adhesive- bonded reinforcement to strength concrete constructional elements , and bases of assessment for their general approval by the construction supervisory authorities. Braunschweig: S&P Clever Reinforcement Company.

S&P Clever reinforcement company (2000). Design guide FRP fiber reinforcement polymer for S&P products.

Saadatmanesh, H. and Malek, AM. (1998)," Design guidelines for flexural strengthening of RC beams with FRP plates/ Journal of composite for construction," ASCE, V.2, No.4, pp. 158-164.

Saadatmansesh, H, Ehsani, MR. (1991)," RC beams strengthened with GFRP plates. I: Experimental study," Journal of Structural Engineering, V.117, No.11.pp.3417-3433.

Seim, W., Karbhari, V. and Seible, F. (1999), "Post-strengthening of concrete slabs with externally bonded fiber reinforced polymer (FRP) strips-analytical approach and design recommendations," San Diego: SSRP Report, University of California.

Sharif A., Al-Sulaimani, Basunbul, G.J., Baluch, M.H. and Ghaleb, B.N. (1994), "Strengthening of initially loaded reinforced concrete beams using FRP plates," *ACI Structural Journal*, V.91, No.2, Mar.-Apr., pp.166-168

SIKA Carbodur. (1997) Engineering guidelines for the use of Sika CarboDur (CFRP) laminate for structural strengthening of concrete structures.

Smith, S.T, Teng, J.G. (2001), "FRP strengthened RC beam. II assessment of debonding strength models," *Engineering Structures*, V.24, pp.397-417.

Smith, S.T, Teng, J.G. (2001), "FRP-strengthened RC beams I: review of debonding strength models," *Engineering Structures*, V.24, pp.385-395

Swamy, RN., Jones, R. and Bloxham, JW. (1987), "Structural behaviour of reinforced concrete beams strengthened by epoxy-bonded steel plates," *Structural Engineer*, V65A, No.2, pp. 59-68.

Svecova, D. and Razaqpur, A.G. (2000), "Flexural behaviour of concrete beams reinforced with CFRP prestressed prisms," *American Concrete Institute Structure Journal*, V.97, No.5, Sep.-Oct., pp.731-738.

Takahashi, Y., Sato, Y.(1997), "Experimental behaviour of RC beams externally bonded CFRP reinforcement," *Proc. of the 3rd International Symposium*, V.1, pp. 685-690.

Takahashi, Y., Sato, Y., Ueda, T., Maaeda, T. and Kobayashi, A. (1997), "Flexural behaviour of RC beams with externally bonded carbon fiber sheet," *Proc. of the 3rd International Symposium*, V.1, pp. 327-334.

Takeda, K., Mitsui, Y., Murakami, K., Sakai, H. and Nakamura, M. (1996), "Flexural behaviour of reinforced concrete beams strengthened with carbon fiber sheets," *Composite Part A*, V.27A, pp. 981-987.

Taljste, N.B.(1997), "Strengthening of beams by plate bonding," *Journal of Materials in Civil Engineering*, ASCE, V.9, No.4, Nov, pp.206-212.

Taljsten, B.(1999), "Concrete beams strengthened for bending using CFRP-sheets," in: forde MC, editor. *Proc. of the 8th International Conference on Advanced Composites for Concrete Repair*. London, UK.

Triantafillou, TC. and Plevris, N. (1992), "Strengthening of RC beams with epoxy-bonded fiber-composite materials," *Materials and Structures*, V.25, pp.201-211.

Tumialan, G., Belarbi, A. and Nanni, A. (1999),” Reinforced concrete beams strengthened with CFRP composites: failure due to concrete cover delamination,” Department of Civil Engineering, Centre for Infrastructure Engineering Studies, Report No.CIES-99/01, University of Missouri-Rolla, USA.

Varastehpour, H. and Hamelin, P. (1997), “Strengthening of concrete beams using Fibre-reinforced plastics,” *Material and Structures*, V.30, pp.160-166.

Vasquez, A.(1999),” the use of carbon fiber reinforced polymer strips for the external strengthening of slabs,” MS thesis. San Diego, University of California.

Wang, CY. and Ling, FS. (1998),” Prediction model for the debonding failure of cracked RC beams with externally bonded FRP sheets,” In: *Proc. of the 2nd International Conference of Composite in Infrastructure (ICCI)*, Arizona, USA. pp. 548-562.

Zaghloul A.(2002),” Behaviour and strength of CFRP reinforced flat plate interior column connections subjected to shear and unbalanced moments,” M.A.Sc. Thesis, Carleton University.

Zhang, S., Raoof, M. and Wood, L.A. (1995), “ Prediction of peeling failure of reinforced concrete beams with externally bonded steel plates,” *Proc. of the Institute of Civil Engineers: Structures and Building*, V.110(August), pp.257-268.

Ziraba, YN., Baluch, MH., Basunbul, IA., Sherif, AM., Azad, A. and Al-Sulaimani, GJ. (1994),” Guidelines towards the design of reinforced concrete beams with external plates,” *ACI Structural Journal*, V.91, No.6, pp.639-646.

Appendix A:

Design of Test Beams

A.1 Steel Reinforced Concrete Beams I

$$f'_c = 40 \text{ MPa} \quad , \quad b = 250 \text{ mm} \quad , \quad h = 400 \text{ mm}, \quad d = 365 \text{ mm}$$

$$\text{Cover assumed} = 35 \text{ mm} \quad , \quad d' = 30 \text{ mm}$$

$$E_c = 4500 \times 40^{0.5} = 28460.49 \text{ MPa}$$

$$E_s = 200 \text{ GPa} \quad , \quad n_{st} = n_{sc} = 200000 / 28460.49 = 7.03$$

$$f_y = 400, \quad \rho = 0.3 \rho_b \text{ (assumed)}$$

$$\alpha = 0.85 - 0.0015 \times 40 = 0.79 \geq 0.67$$

$$\beta = 0.97 - 0.0025 \times 40 = 0.87 \geq 0.67$$

$$\rho_b = \frac{\alpha_1 \beta_1 \phi_c f'_c}{\phi_s f_y} \left(\frac{700}{700 + f_y} \right)$$

$$\text{For Test} \quad \phi_c = \phi_s = 1$$

$$\rho_b = \frac{0.79 \times 0.87 \times 40}{400} \left(\frac{700}{1100} \right) = 0.0437$$

$$\rho = 0.3 \rho_b \Rightarrow \rho = 0.3 \times 0.0437 = 0.01312$$

$$A_s = \rho b d = 0.01312 \times 250 \times 365 = 1197.31 \text{ mm}^2$$

$$\text{Use 4 bars 20 mm diameter} = 1200 \text{ mm}^2$$

$$A_{s1} = A_s \left(\frac{f_y - \alpha_1 f'_c}{f_y} \right) = 200 \times \left(\frac{400 - 0.79 \times 40}{400} \right) = 184.2 \text{ mm}^2$$

$$A_{s2} = A_s - A_{s1} = 1200 - 184.2 = 1015.8 \text{ mm}^2$$

$$M_1 = A_{s1} x f_y x (d - d') = 184.2 \times 400 \times (365 - 30) = 24.68 \text{ kN.m}$$

For M_2

$$a = \frac{A_{s2} f_y}{\alpha_1 f_c' b} = \frac{1015.8 \times 400}{0.79 \times 40 \times 250} = 51.43 \text{ mm}$$

$$M_2 = A_{s2} f_y (d - 0.5 a) = 1015.8 \times 400 \times (365 - 0.5 \times 51.43) = 137.86 \text{ kN.m}$$

$$M = M_1 + M_2 = 24.68 + 137.86 = 162.54 \text{ kN.m}$$

$$c = a / 0.87 = 51.43 / 0.87 = 59.11 \text{ mm}$$

Check if compression steel yield:

$$\epsilon_s' = \frac{0.0035 \times (59.11 - 30)}{59.11} = 0.0017 < \epsilon_y \quad \text{Repeat calculations}$$

$$f_s' = E_s \epsilon_s' = E_s \frac{0.0035}{c} (c - d') = \frac{700}{c} (c - d')$$

$$A_s f_s = A_s' \left(\frac{700}{c} (c - 30) - 0.79 \times 40 \right) + 0.79 \times 40 \times 250 \times 0.87 c$$

$$1200 \times 400 c = 200 [700 c - 21000 - 31.6 c] + 6873 c^2$$

$$480 \times 10^3 c = 140000 c - 42 \times 10^5 - 7320 c + 6873 c^2$$

$$c^2 - 50.54 c - 611.09 = 0$$

$$c = 60.59 \text{ mm}$$

$$\epsilon_s' = \frac{0.0035}{60.59} (60.59 - 30) = 0.00177$$

$$f_s' = E_s \epsilon_s' = 200000 \times 0.00177 = 354.0 \text{ MPa}$$

$$M_n = C_s (d - d') + C_c (d - 0.5 a)$$

$$C_s = A_s' (f_s' - 31.6) = 200 \times (354 - 31.6) = 64480 \text{ N}$$

$$a = \beta c = 0.87 \times 60.59 = 51.71 \text{ mm}$$

$$\begin{aligned}
 M_n &= 64480 \times 335 + 0.79 \times 40 \times 52.71 \times 250 \left(365 - \frac{52.71}{2} \right) \\
 &= 21.61 \times 10^6 + 141.01 \times 10^6 \\
 &= 162.61 \text{ kN.m} \quad (\text{Very close to previous calculation value})
 \end{aligned}$$

Design for shear

$$\text{Let } \phi_c = \phi_s = \phi_F = 1$$

$$V_c = 0.2 \lambda \phi \sqrt{f'_c} b_w d = 0.2 \times 1 \times 1 \times \sqrt{40} \times 250 \times 365 = 115.423 \text{ kN}$$

$$\begin{aligned}
 V_{\max} &= 0.8 \lambda \phi \sqrt{f'_c} b_w d + V_c \\
 &= 0.8 \times 1 \times 1 \times \sqrt{40} \times 250 \times 365 + 115.423 = 577.1155 \text{ kN}
 \end{aligned}$$

for beam under two point loads $V_f = 162.61 \text{ kN} < V_{\max}$

$\therefore$ Section is adequate

Design for Stirrups use $A_s = 200 \text{ mm}^2$

$$S \leq \frac{A_s f_y}{0.06 b_w \sqrt{f'_c}} = 834.27 \text{ mm}$$

$$V_s = V_f - V_c = 162.61 - 115.422 = 47.188 \text{ kN}$$

$$S = \frac{200 \times 400 \times 365}{47.188 \times 1000} = 618.8 \text{ mm}$$

$$\frac{V_f}{b_w d} = \frac{162.61 \times 1000}{250 \times 365} = 1.78 \text{ kN/mm}^2 < 0.1 \lambda \phi_c f'_c = 4.0$$

spacing must not exceed 600 mm or $0.7d = 255.5$

use stirrups every 250 mm $A_s = 200 \text{ mm}^2$

A.2 Steel Reinforced Concrete Beams II

These calculations were done for the control beam under two point loads after testing the concrete cylinders and before testing any beam or applying any CFRP.

$$f'_c = 52.2 \text{ MPa} , b = 250 \text{ mm} , h = 400 \text{ mm}, d = 365 \text{ mm}$$

$$\text{Cover assumed} = 35 \text{ mm} , d' = 30 \text{ mm}$$

$$E_c = 4500 \times 52.2^{0.5} = 32512.3 \text{ MPa} , n_{st} = n_{sc} = 6.152$$

$$E_s = 200 \text{ GPa} , n_{st} = n_{sc} = 200000 / 32512.3 = 6.152$$

$$f_y = 400 , \rho = 0.3 \rho_b \text{ (assumed)}$$

$$A_{st} = 1200 \text{ mm}^2 , A_{sc} = 200 \text{ mm}^2$$

$$\alpha = 0.85 - 0.0015 \times 52.2 = 0.7717 \geq 0.67$$

$$\beta = 0.97 - 0.0025 \times 52.2 = 0.8395 \geq 0.67$$

$$y' = \frac{400 \times 250 \times 200 + 6.152 \times 1200 \times 35 + 6.152 \times 200 \times 365}{400 \times 250 + 6.152 [1200 + 200]} = 190.654 \text{ mm}$$

$$I_g = \frac{250 \times 400^3}{12} + 250 \times 400 [200 - 190.654]^2 + \\ 6.152 \times 1200 [190.654 - 35]^2 + \\ 6.152 \times 200 [190.654 - 365]^2 = 1557616846 \text{ mm}^4$$

$$M_{cr} = \frac{0.6 \sqrt{52.2} \times I_g}{190.654} = 35.41 \text{ kN.m}$$

$$\therefore P_{cr} = 70.811 \text{ KN}$$

assume both tension and compression steel yielded

$$0.7717 \times 52.2 \times 250 \times 0.8395c + 200 \times 400 = 1200 \times 400$$

$$c = 47.32 \text{ mm}, a = 39.72 \text{ mm}$$

$$\epsilon'_s = \frac{0.0035(47.32 - 30)}{47.32} = 0.00128 < \epsilon_y$$

Different than the assumption

$$\epsilon_s = \frac{0.0035(365 - 47.32)}{47.32} = 0.0235 > \epsilon_y$$

Assume only tension reinforcement yield:

$$8454.34c + 200 \times 0.0035 \times 200000 \times (c - 30)/c = 1200 \times 400$$

$$8454.34c^2 + 140000c - 4200000 = 480000c$$

$$8454.34c^2 - 340000c - 4200000 = 0$$

$$c = 50.127 \text{ mm}, a = 42.08 \text{ mm}$$

Check for tension and compression steel if yielded:

$$\epsilon'_s = \frac{0.0035(50.127 - 30)}{50.127} = 0.00141 < \epsilon_y$$

As assumed

$$\epsilon_s = \frac{0.0035(365 - 50.127)}{50.127} = 0.022 > \epsilon_y$$

$$M_n = 1200 \times 400 \times [365 - 0.5 \times 0.8395 \times 50.127] \\ + 200 \times 200000 \times 0.0035 \times [50.127 \times 0.5 - 30]/30 = 165.08 \text{ kN.m}$$

$$P_n = 330.388 \text{ kN}$$

Design for shear

$$V_c = 0.2\lambda\phi\sqrt{f'_c}b_wd = 0.2 \times 1 \times 1 \times \sqrt{52.2} \times 250 \times 365 = 131.855 \text{ kN}$$

$$V_{\max} = 0.8\lambda\phi\sqrt{f'_c}b_wd + V_c \\ = 0.8 \times 1 \times 1 \times \sqrt{52.2} \times 250 \times 365 + 131.855 = 659.2 \text{ kN}$$

for beam under two point loads $V_f = 165.194 \text{ kN} < V_{\max}$

$\therefore$ Section is adequate

Design for Stirrups use $A_s = 200 \text{ mm}^2$

$$S \leq \frac{A_s f_y}{0.06 b_w \sqrt{f'_c}} = 738.18 \text{ mm}$$

$$V_s = V_f - V_c = 165.08 - 131.855 = 33.22 \text{ kN}$$

$$S = \frac{200 \times 400 \times 365}{33.22 \times 1000} = 878.93 \text{ mm}$$

$$\frac{V_f}{b_w d} = \frac{165.08 \times 1000}{250 \times 365} = 1.81 \text{ kN/mm}^2 < 0.12 \phi_c f'_c = 5.2$$

spacing must not exceed 600mm or $0.7d = 255.5$

use stirrups every 250 mm $A_s = 200 \text{ mm}^2$

A.3 Beam with CFRP (Three Laminates)

Assume both tension and compression steel yield

$$f'_c = 52.2 \text{ MPa}, b = 250 \text{ mm}, h = 400 \text{ mm}, d = 365 \text{ mm}$$

$$\text{Cover assumed} = 35 \text{ mm}, d' = 30 \text{ mm}$$

$$E_c = 4500 \times 52.2^{0.5} = 32512.3 \text{ MPa}, n_{st} = n_{sc} = 6.152$$

$$E_s = 200 \text{ GPa}, f_y = 400, A_{st} = 1200 \text{ mm}^2, A_{sc} = 200 \text{ mm}^2$$

$$\alpha = 0.85 - 0.0015 \times 52.2 = 0.7717 \geq 0.67$$

$$\beta = 0.97 - 0.0025 \times 52.2 = 0.8395 \geq 0.67$$

$$A_f = 120 \text{ mm}^2, E_f = 165000 \text{ MPa}$$

From equilibrium (concrete force, compression steel, tension steel and FRP force):

$$0.7717 \times 250 \times 52.2 \times 0.8395c + 400 \times 200 = 1200 \times 400 + 180 \times 165000 \times 0.0035 \times (400 - c)/c$$

$$8454.34C^2 + 80000C = 480000C + 41580000 - 103950C$$

$$8454.34C^2 - 296050C - 4158000 = 0$$

$$c = 89.79 \text{ mm}, \quad a = 75.38 \text{ mm}$$

$$\epsilon'_s = \frac{0.0035(89.79 - 30)}{89.79} = 0.0023 > \epsilon_y$$

$$\epsilon_s = \frac{0.0035(365 - 89.79)}{89.79} = 0.011 > \epsilon_y$$

$$\epsilon_f = \frac{0.0035(400 - 89.79)}{89.79} = 0.0121 < \epsilon_{\max}$$

$$M_n = 1200 \times 400 \times (365 - 0.5 \times 75.38) + 200 \times 400 \times (0.5 \times 75.38 - 30) \\ + 180 \times 165000 \times 0.0139 \times (400 - 0.5 \times 75.38) = 287.93 \text{ kN.m}$$

$$\therefore P_n \text{ total} = 575.85 \text{ kN (Beam with two point load)}$$

A.4 Beam with CFRP (Two Laminates)

Assume both tension and compression steel yield

$$f'_c = 52.2 \text{ MPa}, \quad b = 250 \text{ mm}, \quad h = 400 \text{ mm}, \quad d = 365 \text{ mm}$$

$$\text{Cover assumed} = 35 \text{ mm}, \quad d' = 30 \text{ mm}$$

$$E_c = 4500 \times 52.2^{0.5} = 32512.3 \text{ MPa}, \quad n_{st} = n_{sc} = 6.152$$

$$E_s = 200 \text{ GPa}, \quad f_y = 400, \quad A_{st} = 1200 \text{ mm}^2, \quad A_{sc} = 200 \text{ mm}^2$$

$$\alpha = 0.85 - 0.0015 \times 52.2 = 0.7717 \geq 0.67$$

$$\beta = 0.97 - 0.0025 \times 52.2 = 0.8395 \geq 0.67$$

$$A_f = 120 \text{ mm}^2, \quad E_f = 165000 \text{ MPa}$$

From equilibrium (concrete force, compression steel, tension steel and FRP force):

$$0.7717 \times 250 \times 52.2 \times 0.8395c + 400 \times 200 = 1200 \times 400 + 120 \times 165000 \times 0.0035 \times (400 - c)/c$$

$$8454.34C^2 + 80000C = 480000C + 27720000 - 69300C$$

$$8454.34C^2 - 330700C - 27720000 = 0$$

$$c = 80.067 \text{ mm}, \quad a = 67.216 \text{ mm}$$

$$\epsilon'_s = \frac{0.0035(80.067 - 30)}{80.067} = 0.002189 > \epsilon_y$$

$$\epsilon_s = \frac{0.0035(365 - 80.067)}{80.067} = 0.0125 > \epsilon_y$$

$$\epsilon_f = \frac{0.0035(400 - 80.067)}{80.067} = 0.0139 < \epsilon_{\max}$$

$$M_n = 1200 \times 400 \times (365 - 0.5 \times 67.216) + 200 \times 400 \times (0.5 \times 67.216 - 30)$$

$$+ 120 \times 165000 \times 0.0139 \times (400 - 0.5 \times 67.216) = 259.935 \text{ kN.m}$$

$$\therefore P_n \text{ total} = 346.58 \text{ kN (Beam with one point load)}$$

A.5 Beam with CFRP (One Laminate)

Assume both tension and compression steel yield

$$f'_c = 52.2 \text{ MPa}, \quad b = 250 \text{ mm}, \quad h = 400 \text{ mm}, \quad d = 365 \text{ mm}$$

$$\text{Cover assumed} = 35 \text{ mm}, \quad d' = 30 \text{ mm}$$

$$E_c = 4500 \times 52.2^{0.5} = 32512.3 \text{ MPa}, \quad n_{st} = n_{sc} = 6.152$$

$$E_s = 200 \text{ GPa}, \quad f_y = 400, \quad A_{st} = 1200 \text{ mm}^2, \quad A_{sc} = 200 \text{ mm}^2$$

$$\alpha = 0.85 - 0.0015 \times 52.2 = 0.7717 \geq 0.67$$

$$\beta = 0.97 - 0.0025 \times 52.2 = 0.8395 \geq 0.67$$

$$A_f = 60 \text{ mm}^2, \quad E_f = 165000 \text{ MPa}$$

From equilibrium (concrete force, compression steel, tension steel and FRP force):

$$0.7717 \times 250 \times 52.2 \times 0.8395c + 400 \times 200 = 1200 \times 400 + 60 \times 165000 \times 0.0035 \times (400 - c)/c$$

$$8454.34C^2 + 80000C = 480000 C + 13860000 - 34650 C$$

$$8454.34 C^2 - 365350C - 13860000 = 0$$

$$c = 67.5 \text{ mm} , \quad a = 56.67 \text{ mm}$$

$$\epsilon'_s = \frac{0.0035x(67.5 - 30)}{67.5} = 0.00194 < \epsilon_y$$

$$\epsilon_s = \frac{0.0035x(365 - 67.5)}{67.5} = 0.0154 > \epsilon_y$$

$$\epsilon_f = \frac{0.0035x(400 - 67.5)}{67.5} = 0.0172 < \epsilon_{\max}$$

Repeat calculations as compression steel not yielded

$$0.7717x250x52.2x0.8395c + 200x200000x0.0035x(c-30)/c$$

$$= 1200x400 + 60x165000x0.0035x(400-c)/c$$

$$8454.34C^2 + 140000C - 4200000 = 480000 C + 13860000 - 34650 C$$

$$8454.34 C^2 - 305350C - 18060000 = 0$$

$$c = 67.68 \text{ mm} , \quad a = 56.82 \text{ mm}$$

$$\epsilon'_s = \frac{0.0035x(67.68 - 30)}{67.68} = 0.001948 > \epsilon_y$$

$$\epsilon_s = \frac{0.0035x(365 - 67.68)}{67.68} = 0.0154 > \epsilon_y \quad \text{As Assumed}$$

$$\epsilon_f = \frac{0.0035x(400 - 67.68)}{67.68} = 0.0172 < \epsilon_{\max}$$

$$M_n = 1200x400x(365 - 0.5x56.82) + 200x200000x0.001948x(0.5x56.82 - 30)$$

$$+ 60x165000x0.0172x(400 - 0.5x56.82) = 224.713 \text{ kN.m}$$

$$\therefore P_n \text{ total} = 299.62 \text{ kN (Beam with one point load)}$$

All the calculations which were done using the new value of the yield strength of the steel after testing the bars were done using WIZARD.

A.6 Beam with CFRP Sheet (SN90)

It is required to achieve the same $M_u = 259.935 \text{ kN.m}$ as beam with two laminates

$$f'_c = 52.2 \text{ MPa} , b = 250 \text{ mm} , h = 400 \text{ mm}, d = 365 \text{ mm}$$

$$\text{Cover assumed} = 35 \text{ mm} , d' = 30 \text{ mm}$$

$$E_c = 4500 \times 52.2^{0.5} = 32512.3 \text{ MPa}$$

$$E_s = 200 \text{ GPa} , f_y = 400 , A_{st} = 1200 \text{ mm}^2 , A_{sc} = 200 \text{ mm}^2$$

$$\alpha = 0.85 - 0.0015 \times 52.2 = 0.7717 \geq 0.67$$

$$\beta = 0.97 - 0.0025 \times 52.2 = 0.8395 \geq 0.67$$

$$A_f = \text{required mm}^2 , E_f = 227000 \text{ MPa}$$

Assume both tension and compression steel yielded

$$M_{frp} = 100838406.2 \text{ N.mm} , d_{frp} = 400 \text{ mm}$$

$$f_{frp} = 252096.0155 \text{ N} = A * E * \epsilon = 0.165 * b * 227000 * 0.0167$$

$$b_{frp} = 400 \text{ mm}$$

3 layers each 166.67mm width*0.165 mm thickness

$$0.7717 \times 250 \times 52.2 \times 0.8395 \times c + 400 \times 200 = 1200 \times 400 + 82.5 \times 227000 \times 0.0035 \times (400 - c) / c$$

$$8454.34C^2 + 80000C = 480000C + 26218500 - 65546.25C$$

$$8454.34C^2 - 334453C - 26218500 = 0$$

$$c = 78.877 \text{ mm} , a = 66.217 \text{ mm}$$

$$\begin{aligned}\epsilon_s' &= \frac{0.0035(78.877 - 30)}{78.877} = 0.00217 > \epsilon_y \\ \epsilon_s &= \frac{0.0035(365 - 78.877)}{78.877} = 0.0127 > \epsilon_y \\ \epsilon_f &= \frac{0.0035(400 - 78.877)}{78.877} = 0.0142 < \epsilon_{\max}\end{aligned}$$

$$\begin{aligned}M_n &= 1200 \times 400 \times (365 - 0.5 \times 66.217) + 200 \times 400 \times (0.5 \times 66.217 - 30) \\ &\quad + 82.5 \times 227000 \times 0.0142 \times (400 - 0.5 \times 66.217) = 257.124 \text{ kN.m}\end{aligned}$$

$$\therefore P_n \text{ total} = 342.83 \text{ kN (Beam with one point load)}$$

A.7 Beam with CFRP Sheet (SN901)

It is required the amount of area to achieve $M_u = 224.7 \text{ kN.m}$ as the beam with one laminate.

Assume both tension and compression steel yield

$$f_c' = 52.2 \text{ MPa}, b = 250 \text{ mm}, h = 400 \text{ mm}, d = 365 \text{ mm}$$

$$\text{Cover assumed} = 35 \text{ mm}, d' = 30 \text{ mm}$$

$$E_c = 4500 \times 52.2^{0.5} = 32512.3 \text{ MPa}, n_{st} = n_{sc} = 6.152$$

$$E_s = 200 \text{ GPa}, f_y = 400, A_{st} = 1200 \text{ mm}^2, A_{sc} = 200 \text{ mm}^2$$

$$\alpha = 0.85 - 0.0015 \times 52.2 = 0.7717 \geq 0.67$$

$$\beta = 0.97 - 0.0025 \times 52.2 = 0.8395 \geq 0.67$$

$$A_f = 41.25 \text{ mm}^2, E_f = 227000 \text{ MPa}$$

Assume both tension and compression steel yielded

$$0.7717 \times 250 \times 52.2 \times 0.8395 c + 400 \times 200 = 1200 \times 400 + 41.25 \times 227000 \times 0.0035 \times (400 - c)/c$$

$$8454.34 c^2 + 80000 c = 480000 c + 13109250 - 32773.125 c$$

$$8454.34 c^2 - 367226.875 c - 13109250 = 0$$

$$c = 66.69 \text{ mm} , \quad a = 55.985 \text{ mm}$$

$$\epsilon'_s = \frac{0.0035x(66.69 - 30)}{46.27} = 0.00193 < \epsilon_y$$

$$\epsilon_s = \frac{0.0035x(365 - 66.69)}{66.69} = 0.0156 > \epsilon_y$$

$$\epsilon_f = \frac{0.0035x(400 - 66.69)}{66.69} = 0.0175 > \epsilon_{\max}$$

Repeat calculations as compression steel not yielded

$$0.7717x250x52.2x0.8395c+200x200000x0.0035x(c-30)/c$$

$$=1200x400+41.25x227000x0.0035x(400-c)/c$$

$$8454.34 c^2 + 140000 c - 4200000 = 480000 c + 13109250 - 32773.125 c$$

$$8454.34 c^2 - 307226.875 c - 17309250 = 0$$

$$c = 66.93 \text{ mm} , \quad a = 56.187 \text{ mm}$$

$$\epsilon'_s = \frac{0.0035x(66.93 - 30)}{66.93} = 0.00193 < \epsilon_y$$

$$\epsilon_s = \frac{0.0035x(365 - 66.93)}{66.93} = 0.0155 > \epsilon_y \quad \text{As Assumed}$$

$$\epsilon_f = \frac{0.0035x(400 - 66.93)}{66.93} = 0.0174 > \epsilon_{\max}$$

$$M_n = 1200x400x(365 - 0.5x56.187) + 200x200000x0.00193x(0.5x56.187 - 30)$$

$$+41.25x227000x0.0174x(400 - 0.5x56.187) = 222.31 \text{ kN.m}$$

$$\therefore P_n \text{ total} = 296.4 \text{ kN (Beam with one point load)}$$

All the calculations which were done using the new value of the yield strength of the steel after testing the tension steel bars were done using **WIZARD**.

Note: using the actual value of f_y for the tension steel which is equal to $f_y = 466 \text{ MPa}$

in the calculations will give a value of $c = 73.55 \text{ mm}$, and $\epsilon_f = 0.0155 < 0.167$

A.8 Choosing the Unsheeted Length

It was found in the literature that equations were developed Neubauer and Rostasy (2001)

to calculate the unsheeted length as follow:

$$L_t = \sqrt{\frac{Et}{2f_{ctr}}} \quad , L_l = \sqrt{\frac{Et}{\sqrt{f'_c}}}$$

Where,

E = young's modules

t = plate thickness

f_{ctr} = tensile strength

f'_c = compressive concrete strength

For the laminate:

$$L_t = \sqrt{\frac{Et}{2f_{ctr}}} = \sqrt{\frac{165000 \times 1.2}{2 \times 4.1}} = 156.25 \text{ mm}$$

$$L_l = \sqrt{\frac{Et}{\sqrt{f'_c}}} = \sqrt{\frac{165000 \times 1.2}{\sqrt{52.2}}} = 165.56 \text{ mm}$$

For the Sheet:

$$L_t = \sqrt{\frac{Et}{2f_{ctr}}} = \sqrt{\frac{227000 \times 0.165}{2 \times 4.1}} = 68.11 \text{ mm}$$

$$L_l = \sqrt{\frac{Et}{\sqrt{f'_c}}} = \sqrt{\frac{227000 \times 0.165}{\sqrt{52.2}}} = 72.17 \text{ mm}$$

The length used for all of the beams was 90 mm from where the laminate/sheet was theoretically not needed [theoretical cut-off point].