NOTE TO USERS

This reproduction is the best copy available.

UMI[®]

STRENGTHENING FULL-SCALE DAMAGED PRECAST/PRESTRESSED CONCRETE DOUBLE-TEE GIRDERS USING CFRP SHEETS

BY

Antonio Cofini, B. Eng

Ryerson University, 2005

A Project Report

Presented to Ryerson University

In partial fulfillment of the

Requirements for the degree of

Master of Engineering

In the program of

Civil Engineering

Toronto, Ontario, Canada, 2005

Antonio Cofini 2005 ©

**PROPERTY OF
RYERSON UNIVERSITY LIBRARY**

UMI Number: EC53427

INFORMATION TO USERS

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 bleed-through, substandard margins, and improper alignment can adversely affect reproduction.

In the unlikely event that the author did not send 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.

UMI Microform EC53427
Copyright 2009 by ProQuest LLC
All rights reserved. This microform edition is protected against
unauthorized copying under Title 17, United States Code.

ProQuest LLC
789 East Eisenhower Parkway
P.O. Box 1346
Ann Arbor, MI 48106-1346

AUTHOR'S DECLARATIONS

I hereby declare that I am the sole author of this thesis.

I authorize Ryerson to lend this document to other institutions or individuals for the purpose of scholarly research.

Antonio Cofini

I further authorize Ryerson University to reproduce the document by photocopying or by other means, in total or part, at the request of other institutions or individuals for the purpose of scholarly research.

Antonio Cofini

BORROWERS PAGE

Ryerson University requires the signatures of all persons using or photocopying this thesis.

Please sign below, and give address and date.

This image shows a single sheet of white paper with horizontal ruling lines. The lines are evenly spaced and run across the width of the page. There are no margins, text, or other markings on the paper.

ABSTRACT

Canada is facing a major crisis with the deterioration of its infrastructure (bridges, harbors, buildings, water structures, sewers, parking garages, etc.). Many of these structures were built using precast prestressed concrete members. These members may be under-strength because of deficiencies in design, increase in applied loads, loss of prestress, or damage due to the effects of corrosion, collision or military operations. Epoxy-bonding composite materials to the tension side of prestressed concrete girders is an effective technique for shear/flexural rehabilitation and strengthening of such members. To ensure successful and cost-effective applications of such materials, engineers need to improve their knowledge with respect to the actual behavior of full-size girders so that they can gain confidence on using these materials on structural strengthening and rehabilitation. This can be achieved by providing more data on testing to-collapse and on field conditions full-scale prestressed girders strengthened using FRP materials. This study details the use of Carbon Fiber Reinforced Polymer (CFRP) Sheets to repair and strengthen precast prestressed concrete Double-Tee (DT) girders in flexure and shear. Three actual-size partially-damaged DT precast pretensioned girders were obtained from the manufacturer. All three of the girders were repaired and strengthened, and then tested to failure to determine flexural and shear capacities. The stems of two of these girders were strengthened in flexures using U-shape un-directional carbon fiber reinforced polymer sheets (CFRP), extending from the mid-span to the quarter points of the girders. Two girders were strengthened in shear the dapped ends using 0° wrapping technique around the stem, while the dapped ends of the third girder were strengthened using $0^\circ/90^\circ$ wrapping technique. Each girder was loaded incrementally up to failure using Jersey Barriers. This project summarizes the loading history and reports test results that can be further used to demonstrate the practicality of girder strengthening with CFRP sheets in field conditions.

ACKNOWLEDGEMENTS

I wish to express my deepest appreciation to my advisor Dr. K. Sennah at Ryerson University, for his constant support and valuable supervision during the development of this research. Dr. Sennah devoted his time and effort to make this study a success. His guidance is greatly appreciated. I also wish to thank G. Cofini, N. Jaalouk and D. Peneff for their significant assistance in the experimental work. This research was made possible by funding from EllisDon Corporation, Prestressed Systems Inc., Toronto District School Board, and Materials and Manufacturing Ontario. The in-kind contribution in the form of CFRP sheets and adhesives from RJ Watson Inc. of New York, USA, and Fyfe Co. LLC The Fibrewrap Company, San Diego, CA, USA, is greatly appreciated. Special thanks to Kaneff Development Inc. of Brampton for providing the off-campus site for testing.

To my colleagues and friends, thanks for the memorable times we had over the past five years. The author is also very grateful to his wife, parents, brother and sister for their great support and encouragement during the course of this study.

TABLE OF CONTENTS

Author's Declarations	ii
Borrower's Page.....	iii
Abstract.....	iv
Acknowledgements.....	v
Table of Contents.....	vi
List of Tables.....	viii
List of Figures.....	ix

CHAPTER

I- INTRODUCTION

1.1 General.....	1
1.2 The Problem.....	3
1.3 Objective.....	4
1.4 Scope.....	5
1.5 Contents and Arrangements of this Study.....	5

II- LITERATURE REVIEW

2.1 General	
2.2 Literature Review of Beam Strengthening.....	6
2.2.1 Flexure.....	6
2.2.2 Shear.....	12
2.2.3 Combined Flexure and Shear.....	14
2.3 History of Fiber Reinforced Polymer.....	17
2.4 Composition of FRP	
2.4.1 Fibres.....	19
2.4.2 Epoxy Resins.....	20
2.4.3 Protective Coatings.....	21
2.4.4 Fire Resistance of FRP.....	21
2.4.5 Application Procedure of FRP.....	23

III- EXPERIMENTAL PROGRAM

3.1 General	
3.2 DT Girder Configuration.....	26
3.2.1 DT-41 Girder.....	26
3.2.2 DT-100 Girder	27
3.2.3 DT-102A Girder	28
3.2.4 Girder Characteristics.....	29

3.3 Construction of Girder Supports	
3.3.1 Site Preparation.....	32
3.3.2 Construction of Supports.....	34
3.3.3 Delivery of Girders.....	35
3.4 Carbon Fiber Reinforced Polymer Application	
3.4.1 CFRP Material Properties.....	40
3.4.2 Substrate Penetration.....	41
3.4.3 Pre-Existing Damage Repair.....	41
3.4.4 Application of Carbon Fiber Reinforced Polymer Sheet.....	41
3.5 Carbon Fiber Reinforced Polymer Sheet Application	
3.5.1 CFRP Arrangement of DT-41 Girder.....	43
3.5.2 CFRP Arrangement of DT-100 Girder.....	47
3.5.3 CFRP Arrangement of DT-102A Girder.....	51
3.6 Instrumentation	
3.6.1 Strain Gauge Application.....	56
3.6.2 Linear Variable Displacement Transducer, LVDT's.....	61
3.6.3 Testing Equipment.....	64
3.6.3.1 Crane.....	64
3.6.3.2 Data Acquisition System.....	65
3.6.3.3 Loading Using Jersey Barriers.....	66

IV- EXPERIMENTAL RESULTS AND DISCUSSIONS

4.1 General	68
4.2 Loading History and Crack Pattern	72
4.2.1 DT41 Girder.....	72
4.2.2 DT102A Girder.....	81
4.2.3 DT100 Girder.....	91

V- CONCLUSIONS AND RECOMMENDATIONS

5.1 General	98
5.2 Conclusions and Recommendations for Future Research.....	99

References	100
Appendix.....	104

List of Tables

<u>Table</u>		<u>Page</u>
Table 3.1	Girder Configurations	30
Table 3.2	Girder Reinforcing Details	30
Table 3.3	Tyfo Epoxy Material Properties	40
Table 3.4	Tyfo SCH-41S Carbon Fiber Properties	40
Table 4.1	Compressive Strength of Concrete	69
Table A1	LVDT's Readings for DT-41 Girder	100
Table A2	LVDT's Readings for DT-102A Girder	101
Table A3	LVDT's Readings for DT-100 Girder	102
Table A4	Strain Gauge Readings at Support of DT-41 Girder	102
Table A5	Strain Gauge Readings at Support of DT-102A Girder	103
Table A6	Strain Gauge Readings at Support of DT-100 Girder	104
Table A7	Strain Gauge Readings at Top of Deck Slab of DT-41 Girder	104
Table A8	Strain Gauge Readings at Top of Deck Slab of DT-102A Girder	105
Table A9	Strain Gauge Readings at Top of Deck Slab of DT-100 Girder	106
Table A10	Strain Gauge Readings in the Stem of DT-41 Girder	107
Table A11	Strain Gauge Readings in the Stem of DT-102A Girder	108
Table A12	Strain Gauge Readings in the Stem of DT-100 Girder	109

List of Figures

<u>Figure</u>		<u>Page</u>
Figure 1.1	Parking Garage Leaster B. Pearson International Airport, Toronto, Canada	2
Figure 2.1	CFRP System	18
Figure 2.2	Strain vs Stress Distribution Comparison to Conventional Steel	20
Figure 3.1	DT41 Girder Elevation (left side)	26
Figure 3.2	DT41 Girder Elevation (right side)	27
Figure 3.3	DT41 Girder Cross Section Details	27
Figure 3.4	DT100 Girder Elevation (right side)	27
Figure 3.5	DT100 Girder Elevation (left side)	28
Figure 3.6	DT100 Girder Cross Section Details	28
Figure 3.7	DT102A Girder Elevation (right side)	28
Figure 3.8	DT102A Girder Elevation (left side)	29
Figure 3.9	DT102A Girder Cross Section Details	29
Figure 3.10	View of Existing Damage in DT102A Girder	31
Figure 3.11a	Map of Pre-Existing Damage to DT-41 Girder	31
Figure 3.11b	Map of Pre-Existing Damage to DT-100 Girder	31
Figure 3.11c	Map of Pre-Existing Damage to DT-102A Girder	31
Figure 3.12	Site Clearing and Grubbing	32
Figure 3.13	Excavation of Drive Isles for Crane	33
Figure 3.14	Fine Grading of Granular Subgrade	33
Figure 3.15	Support Formwork and Reinforcing Steel	34
Figure 3.16	Supports for Precast Double T Girders completed	34
Figure 3.17	Placement of Steel Plate Bearing Seats	35
Figure 3.18	Alignment of Double T Girder Supports Complete	36
Figure 3.19	Delivery of Girders by Tractor and Flatbed Trailer	36
Figure 3.20	Placement of Double T Girder on Supports	37
Figure 3.21	Placement of Double T Girder on Supports-Close up	37
Figure 3.22	Placement of Double T Girder by Mobile Crane	38
Figure 3.23	Double T Girder Suspended In Position	38
Figure 3.24	Specimen Layout	39
Figure 3.25	Substrate Preparation by Mechanical Grinding	41
Figure 3.26	CFRP Sheet Application	42
Figure 3.27	Layout of CFRP sheets for DT-41 Girder	43
Figure 3.28	View of DT-41 Girder after installation of CFRP sheets	44
Figure 3.29	View of the N-E side of the DT-41 Girder	44
Figure 3.30	View of the S-E side of the DT-41 Girder	45
Figure 3.31	View of the N-W side of the DT-41 Girder	45
Figure 3.32	View of the S-W side of the DT-41 Girder	46
Figure 3.33	View of the Underside of the DT-41 Girder	46
Figure 3.34	View of CFRP Sheets at Supports of the DT-41 Girder	47
Figure 3.35	Layout of CFRP Sheets for DT-100 Girder	47
Figure 3.36	View of DT-100 Girder after installation of CFRP sheets	48

Figure 3.37	View of the N-E side of the DT-100 Girder	48
Figure 3.38	View of the S-E side of the DT-100 Girder	49
Figure 3.39	View of the N-W side of the DT-100 Girder	49
Figure 3.40	View of the S-W side of the DT-100 Girder	50
Figure 3.41	View of the Underside of the DT-100 Girder	50
Figure 3.42	View of CFRP Sheets at Supports of the DT-100 Girder	51
Figure 3.43	Layout of CFRP Sheets for DT-102A Girder	52
Figure 3.44	View of DT-102A Girder after installation of CFRP sheets	52
Figure 3.45	View of the N-E side of the DT-102A Girder	53
Figure 3.46	View of the S-E side of the DT-102A Girder	53
Figure 3.47	View of the N-W side of the DT-102A Girder	54
Figure 3.48	View of the S-W side of the DT-102A Girder	54
Figure 3.49	View of the Underside of the DT-102A Girder	55
Figure 3.50	View of CFRP Sheets at Supports of the DT-102A Girder	55
Figure 3.51	Location of Longitudinal Strain Gauges at Mid-Span Section	56
Figure 3.52	Location of 45° Angle Strain Gauges at Supports	57
Figure 3.53	Side View of Strain Gauges at Supports	57
Figure 3.54	Layout of Strain Gauges at Support of DT-41 Girder	58
Figure 3.55	Layout of Strain Gauges at Mid-Span of DT-41 Girder	58
Figure 3.56	Layout of Strain Gauges at Support of DT-100 Girder	59
Figure 3.57	Layout of Strain Gauges at Mid-Span of DT-100 Girder	59
Figure 3.58	Layout of Strain Gauges at Support of DT-102A Girder	60
Figure 3.59	Layout of Strain Gauges at Mid-Span of DT-102A Girder	60
Figure 3.60	General Layout of Linear Variable Displacement Transducers at Mid-Span of DT-41 Girder	61
Figure 3.61	General Layout of Linear Variable Displacement Transducers at Support of DT-41 Girder	62
Figure 3.62	Layout of Linear Variable Displacement Transducers for DT-41 Girder	62
Figure 3.63	Layout of Linear Variable Displacement Transducers DT-100	63
Figure 3.64	Layout of Linear Variable Displacement Transducers DT-102A	63
Figure 3.65	View of the Crane Used for Load Application	64
Figure 3.66	View of Haulage Equipment-Flat Bed	65
Figure 3.67	Data Acquisition System	66
Figure 3.68	Loading Procedures	67
Figure 4.1	View of Concrete Coring Sampling	69
Figure 4.2	View of Concrete Core Sample After Failure	69
Figure 4.3	Dimensions of a Jersey Barrier	70
Figure 4.4	Experimental Setup DT-41	70
Figure 4.5	Experimental Setup DT-100	71
Figure 4.6	Experimental Setup DT-102A	71
Figure 4.7	Sequence of the Application of Jersey Barriers on the DT-41 Girder	73
Figure 4.8	Cross-Sectional View of Barrier Locations for the DT-41 Girder at Mid-span	73
Figure 4.9	Application of the 9 th Barrier Load on the DT-41 Girder	74

Figure 4.10	Application of the 11 th Barrier Load on the DT-41 Girder	74
Figure 4.11	Application of the 13 th Barrier Load on the DT-41 Girder	75
Figure 4.12	Deflected Shape of the DT-41 Girder After Loading the Last Barrier	75
Figure 4.13	Crack Pattern on the N-E Side of DT-41 Girder after Removing the Barriers	76
Figure 4.14	Crack Pattern on the N-W Side of DT-41 Girder after Removing the Barriers	76
Figure 4.15	Delamination Failure at the End of the CFRP Sheet of DT-41 Girder	77
Figure 4.16	Close-up of the Cracks and Peeling of CFRP Sheets of DT-41 Girder	77
Figure 4.17	Crack Pattern on the S-W Side of DT-41 Girder after Removing the Barriers	78
Figure 4.18	Close-up of Main Shear-Flexure Crack on the S-W Side of DT-41 Girder	78
Figure 4.19	Crack Pattern on the S-E Side of DT-41 Girder after Removing the Barriers	79
Figure 4.20	Close-up of Main Shear-Flexure Crack on the S-E Side of DT-41 Girder	79
Figure 4.21	Sketch of General Crack Pattern of the DT-41 Girder	80
Figure 4.22	Sequence of the Application of Jersey Barriers on the DT-102A Girder	82
Figure 4.23	Cross-Sectional View of Barrier Locations for the DT-102A Girder at Mid-span	82
Figure 4.24	Application of the 12 th Barrier Load on the DT-102A Girder	83
Figure 4.25	Application of the 17 th Barrier Load on the DT-102A Girder	83
Figure 4.26	Deflected Shape of the DT-102A Girder After Loading the Last Barrier	84
Figure 4.27	Crack Pattern on the N-W Side of DT-102A Girder, Closer to Support after	84
Figure 4.28	Crack Pattern on the N-W Side of DT-102A Girder, Closer to the Mid-span	85
Figure 4.29	Crack Pattern on the North Side of DT-102A Girder at Mid-span	85
Figure 4.30	Crack Pattern on the N-E Side of DT-102A Girder, Closer to the Mid-span	86
Figure 4.31	Crack Pattern on the N-E Side of DT-102A Girder, Closer to Support after	86
Figure 4.32	Close-up of Crack Pattern on the N-E Side of DT-102A Girder	87
Figure 4.33	Close-up of Crack on the N-W Side of DT-102A Girder	87
Figure 4.34	Crack Pattern on the S-W Side of DT-102A Girder, Closer to the Support	88
Figure 4.35	Main Shear-Flexure Crack on the S-W Side of DT-102A Girder	88

Figure 4.36	Main Shear-Flexure Crack on the S-W Side of DT-102A Girder, Closer to Support	89
Figure 4.37	Close-Up of Main Shear-Flexure Crack on the S-W Side of DT-102A Girder	89
Figure 4.38	Sketch of General Crack Pattern of the DT-102A Girder	90
Figure 4.39	Sequence of the Application of Jersey Barriers on the DT-100 Girder	92
Figure 4.40	Cross-Sectional View of Barrier Locations for the DT-100 Girder at Mid-span	92
Figure 4.41	Application of the 4 th Barrier Load on the DT-100 Girder	93
Figure 4.42	Application of the 9 th Barrier Load on the DT-100 Girder	93
Figure 4.43	Application of the 12 th Barrier Load on the DT-100 Girder	94
Figure 4.44	View of Deflected Shape of the DT-100 Girder After Failure	94
Figure 4.45	Close-up of Cracks at Mid span of DT-100 Girder After Failure	95
Figure 4.46	Close-up of Failure Zone of DT-100 Girder	95
Figure 4.47	View of East Side of the DT-100 Girder After Failure	96
Figure 4.48	Close-up of East Side of the DT-100 Girder After Failure	96
Figure 4.49	Sketch of General Crack Pattern of the DT-100 Girder	97

CHAPTER I

INTRODUCTION

1.1 General

Double-Tee (DT) precast pretensioned girder members are a vital part of our modern infrastructure, they are used to construct civil projects nation wide. They can be found in residential, industrial and commercial buildings as well as bridge decks, and parking garages. Predominately used in parking garages, these girders make up a key component to the overall design. The design of parking garages is generally made up in such a way as to blend serviceability issues to cost effectiveness and most importantly speed up the construction phase otherwise known as down-time of a project. Parking garages are mainly built up of precast members. As the foundation and slab on grade floor system is developed, the only members that are cast in-situ are the columns. Once the columns are set and poured in to place, precast members making up the support beams are lifted and simply supported by these columns at each end. The next step is to install the double-Tee girders whom span from support beam to support beam much like a joist in conventional residential construction. Once one floor of the parking garage is completely finished a surface course made up of three to four inches of concrete is poured in-situ over the whole area. This process not only acts as a surface wearing course but also as a cover to external detrimental elements that would deteriorate the precast sections. As one floor is completed these steps are repeated to complete the construction for the successive floors. Figure 1.1 shows a photo of flat slab parking garage located at Pearson International Airport in Toronto, Ontario.

1.2 The Problem

Figure 1.1- Parking Garage Leaster B. Pearson International Airport,
Toronto, Canada

Cities and Municipalities depend on structures such as parking garages to perform their activities and serve the public. The increasing age of these structures is becoming a major concern, requiring new and improved rehabilitation methods. The conventional means of rehabilitating a damaged double T girder would be to close that section of the parking garage, break away at the damaged concrete and pour a new concrete patch over the effect area. This method is not only very expensive, but will only act as a band-aid and solve the problem temporarily, giving way for reoccurring deterioration. A superior way to ensure the durability and serviceability of these structures must be found. In order to meet this challenge, this thesis presents an innovative rehabilitation technique that involves the use of externally bonded carbon fiber reinforced polymers (CFRP) to strengthen the deteriorated or damaged structure.

1.2 The Problem

Parking structures and bridge decks in many parts of Canada and the USA are exposed to severe environmental conditions. The main being de-icing chemicals. Chemicals such as salt cause corrosion of the steel reinforcement in concrete structures. It has been found that parking structures may start to suffer from reinforcing steel corrosion due to de-icing salt only 10 years after their construction (Walker, 1986). In Canada, the estimated cost of repairing existing parking structures is in the range of four to six billion dollars. The estimated repair and rehabilitation cost of the existing highway bridges and parking structures in the USA is over 50 billion dollars (Michaluk et al. 1998).

A number of methods and precautions have been introduced to prevent corrosion of steel reinforcement. Among those methods are decreasing the permeability of concrete by using dense concrete, adding additives to concrete, coating concrete with impermeable layers, coating steel rebar with epoxy and cathodic protection of reinforcement, and providing thicker cover to the reinforcement. However, most of these protection measures increase the cost of the structure and do not prevent corrosion completely; rather they delay the problem, and they may even worsen the problem if not done properly.

There are many conventional systems that can be approached to retrofit old parking garages. Most of which only act as a temporary fix and not a cure. An ideal solution is to reinforce the girder by using fiber reinforced polymer sheets (FRP) which are immune to corrosion. Of course, the new reinforcement must have adequate strength and stiffness and must be reasonably economical. It must also be able to withstand other environmental and short and long term loading effects. Among the FRP, carbon fiber

reinforced polymers (CFRP) are known to be practically immune to chemical attacks and they have high strength and relatively high modulus elasticity. To ensure successful and cost-effective applications of such materials, engineers need to improve their knowledge with respect to the actual behavior of full-size girders so that they can gain confidence on using these materials on structural strengthening and rehabilitation. This can be achieved by providing more data on testing to-collapse and on field conditions full-scale prestressed girders strengthened using FRP materials. This study details the use of Carbon Fiber Reinforced Polymer (CFRP) Sheets to repair and strengthen precast prestressed concrete Double-Tee (DT) girders in flexure and shear.

1.3 Objectives

The proposed research is aimed at conducting field-testing of full-scale deteriorated double-tee (DT) precast pretensioned girders strengthened with carbon fiber polymer (CFRP) sheets so that further studies can be conducted to answer the following questions:

1. How is the safety level of a double-tee (DT) precast/prestressed girder compromised if it is strengthened using carbon fiber reinforced polymers (CFRP) sheets?
How much strengthening is beneficial, and when is it detrimental?
2. How the experimental findings correlate with the available design equations for flexure and shear for the precast pretensioned concrete girder?
3. How the recently developed FRP design codes fit with the experimental findings recorded in this thesis?

1.4 Scope

The focus of this study is on three precast prestressed double-tee girders of different geometric characteristics and level of damage. All three of the girders were repaired and strengthened, and then tested to failure to determine flexural and shear capacities. The stems of two of these girders were strengthened in flexures using U-shape un-directional carbon fiber reinforced polymer sheets (CFRP), extending from the mid-span to the quarter points of the girders. While all the three girders were strengthened in shear at the dapped-ends. Two girders were strengthened in shear the dapped ends using 0° wrapping technique around the stem, while the dapped ends of the third girder were strengthened using $0^\circ/90^\circ$ wrapping technique. Each girder was loaded incrementally up to failure using Jersey Barriers. The results from testing were reported for further studies to demonstrate the practicality of strengthening a damaged structure with bonded CFRP sheets in field conditions through comparison between the experimental findings and the theoretical results from the available equation for flexural and shear capacities of such girders was made.

1.5 Contents and Arrangement of this Study

The literature review on the behavior, the strengthening and rehabilitation of concrete beams are presented in Chapter II. Chapter III presents the test program, which describes the configurations of the tested girders, the test setup and material properties. Chapter IV discusses the experimental results. Chapter V concludes this research and presents recommendations for further research.

CHAPTER II

LITERATURE REVIEW

2.1 General

In the last couple of decades, a significant amount of research has been conducted to predict the behavior of different types of fibre reinforced polymers (FRP) as used for strengthening, rehabilitation or retrofitting of structural members. However, very few studies have been undertaken to investigate the application of such material to full-scale structural members. This chapter deals with literature review pertained to concrete beams of various sizes and design configurations strengthened in flexure, shear or combined flexure and shear. Also, this chapter summarizes the characteristics of FRP material used in structural applications.

2.2 Literature Review of Beam Strengthening

2.2.1 Flexure

Few authors dealt with the behavior of concrete beams strengthened using FRP. Bonacci and Maalej (2001) studied the performance of conventionally reinforced concrete (RC) beams strengthened in flexure with externally bonded fiber-reinforced polymers (EB-FRP). A database of experimental testing results was compiled on which 127 specimens from 23 separate studies were included in the database. A profile of specimens in the database was given, followed by an analysis of trends in failure mode, strength gain, and deformability. Failure by de-bonding of FRP was prevalent among specimens in the database. One-third of the specimens with external reinforcement added

showed strength increases of 50% or more in combination with considerable deflection capacity. It was clear from the experimental studies that the followed procedures were most representative of member strengthening rather than repair. Most of the specimens in the database were not subjected to sustained loading or damage causing loss of original capacity before external reinforcement was added. To assess the real potential of using FRP for expedient and economical field repair and strengthening of RC members, it was concluded that future research on the application of FRP to RC members should focus on conditions that are similar to what is observed in the field, including the effects of sustained load during repair/strengthening as well as corrosion- and load-induced damage.

Rahimi and Hutchinson (2001) investigated the structural behavior of reinforced concrete beams strengthened with adhesively bonded fiber-reinforced plastics (FRP). The experimental work included flexural testing of 2.3-m-long concrete beams with bonded external reinforcements. The test variables included the amount of conventional (internal) reinforcement and also the type and amount of external reinforcement. For comparison, some of the beams were strengthened with bonded steel plates. Theoretical analyses included 2D nonlinear finite-element modeling incorporating a "damage" material model for concrete. In general there were reasonably good correlations between the experimental results and nonlinear finite-element models. It was suggested that the detachment of bonded external plates from the concrete at ultimate loads was governed by a limiting principal stress value at the concrete/external plate interface.

Fanning and Kelly (2001) studied the ultimate response of reinforced concrete

beams strengthened with carbon fiber reinforced polymer (FRP) plates. The beams in this study were precracked with an initial loading of 30 to 70 percent of the ultimate capacity of the member, below the yielding of the tensile steel. It was observed that at higher load levels, the crack patterns in beams with externally bonded fiber reinforced polymer strips have exhibited a shift from several widely spaced large cracks to many smaller cracks at a much closer spacing. It was calculated that thicker strips provide a larger area of FRP and can provide greater amounts of strengthening. As the thickness of a FRP strip is increased, the peeling stresses at the strip end increase as a result of moving the tensile force resultant farther from the bottom surface of the concrete beam. These peeling stresses tend to pull the end of the strip away from the beam, causing a delamination failure. It was also observed that the smaller the length of the sheet is extended into the shear span of a beam loaded in three or four point bending, the more likely the strip is to peel off.

De Lorenzis et al. (2001), studied Fiber-reinforced polymer (FRP) laminates as being successfully used worldwide for strengthening existing reinforced concrete structures. The bond of FRP reinforcement to the concrete substrate was of critical importance for the effectiveness of the technique. In this project, flexural test specimens were prepared to address some of the factors expected to affect bond, namely: bonded length, concrete strength, number of plies (stiffness), ply width, and, to a limited extent, surface preparation. A linear bond stress-slip relationship, along with a simple shear model for the evaluation of the slip modulus, was used to predict the strain distribution at moderate load levels. Finally, expressions of the peeling load and the effective bond

length were presented. A design equation was proposed for calculating the effective FRP ultimate strain to be used in design to account for bond-controlled failure.

Bonacci and Maalej (2000) studied the effect of externally bonded fiber-reinforced polymer for rehabilitation of corrosion damaged concrete beams. The authors reported the results of an experimental program designed to provide a realistic assessment of the potential of using fiber-reinforced polymer (FRP) materials in the repair and strengthening of reinforced concrete (RC) flexural members. The experimental program included seven RC flexural beams 270 x 400 mm in cross section and 4350 mm in length. Four of the seven RC beams were reinforced externally with one or two layers of carbon fiber-reinforced polymer (CFRP) composite. Different variables considered in this experimental program included state of damage (damaged versus undamaged) and loading condition during bonding (loaded versus unloaded). Damage was introduced in four of the seven RC beams using an accelerated corrosion technique developed at University of Toronto. Tests in the current study show that it is necessary to consider the effects of corrosion- and load-induced damage as well as sustained load on the load-carrying and deflection capacities of externally reinforced flexural members. Also, it is concluded that it is possible to achieve adequate corrosion repair with externally bonded CFRP with minimal intervention. In particular, the current study shows that it is important to optimize CFRP layout to balance strength recovery with control of faulting and splitting, which could lead to premature member failure.

Sebastian (2001) investigated the significance of midspan debonding failure in

fiber reinforced polymer-plated concrete. Reinforced concrete beams enhanced in flexure with adhesively-bonded fibre reinforced polymer plates are susceptible to a brittle form of failure, defined by delamination of the cover concrete attached to the adhesive that causes the plates to debond from the beam. The author demonstrates that, while previous research has focused on one debonding mode in which concrete delamination progresses from the ends of the plates inwards, there exists another critical debond mode that initiates near flexural cracks in the midspan region of the plated beam and propagates out to the ends of the plates. Data from large-scale experimental work are presented to show that midspan debonding action is triggered by high shear stresses transmitted from the plates through the adhesive to the cover concrete. These stresses arise initially from tension stiffening in the cracked concrete and corrosion of the embedded steel. It is shown that strain gauge data are required from both the bonded and exposed surfaces of the plate for accurate quantification of these shear stresses. The shear span of the external load and the stiffness of the plate are cited as parameters that may influence whether in practice end peel or midspan debond occur in plated beams. The paper concludes by describing compulsory features of any numerical modeling of the midspan debond phenomenon.

Spadea et al. (2001) studied the strength and ductility of reinforced concrete beams repaired with bonded carbon fiber reinforced polymers. Three series of tests on eleven reinforced concrete beams were carried out and their ultimate load capacities and ductilities were evaluated. The variables considered in this experimental program were longitudinal steel ratio, the volume of the internal stirrups, and the location and configuration of external anchorages. The results show that both deflection and energy

absorption are drastically reduced when beams are strengthened with bonded carbon fiber reinforced polymers plates without external anchorages. Suitably designed and positioned external anchorages allow much of this lost ductility to be regained; however, even then the ductility of the strengthened beam can not be restored to its original level. It is shown that definitions of ductility based on deflection and energy are able to give a good and rational representation of the physical aspects of ductility of reinforced concrete beams strengthened with bonded carbon fiber reinforced polymer laminates with or without external anchorages.

Brena et al. (2003) predicted the increasing flexural capacity of reinforced concrete beams using carbon fiber reinforced polymer composites. A series of reinforced concrete beams strengthened in flexure using different carbon fiber-reinforced polymers composite systems were fabricated and tested in the laboratory to examine the effects of the strengthening configuration on the specimen behavior. The main goal of these tests was to explore different strengthening configurations capable of developing the strength of the composite laminates and preclude failure by debonding of the composite systems from the concrete surface. Results indicate that relying on the contact area between the composite laminates and the concrete surface is not sufficient to eliminate debonding. Strengthening configurations involving techniques such as placement of transverse straps along the composite laminates or bonding the composites on the side surface of the specimens controlled debonding and provided a more ductile failure mode than placement on the bottom surface of the beams.

Harraq, et al. (2002) made an investigation of prestressed concrete girders strengthened with externally bonded carbon fibre sheets. Prestressed concrete beams may be under-strength because of deficiencies in design, increases in applied loads, loss of prestress, damage due to the effect of collision, corrosion or military operations and salt damage. There has been limited research on the repair of environmental exposure, inadequate design and heavy traffic. Because they are impermeable, the laminates resist moisture and the resulting corrosion. An added advantage of carbon laminates is their ease installation due to their extremely light weight compared to steel or concrete. The authors examined the strength enhancement provided to the flexural capacity of prestressed concrete girders by the external bonding of carbon fiber reinforced polymer laminates through a large scale beam study. Seven half scale standard CPCI 900 mm pretensioned bridge girders comprising two unreinforced and five reinforced by carbon fiber reinforced polymer were considered in this study. They conclude that by bonding the CFRP sheets the loss of prestressing can be compensated and the durability can be improved.

2.2.2 Shear

Antonopoulos and Triantafillou (2003) investigated experimentally the strengthening of reinforced concrete beam-column joints. The results of a comprehensive experimental program, aimed at providing a fundamental understanding of the behavior of shear-critical exterior reinforced concrete joints strengthened with fiber reinforced polymers under simulated seismic load, were presented. The role of various parameters on the effectiveness of fiber reinforced polymers was examined through 2/3-scale testing

of 18 exterior reinforced concrete joints. Conclusions were drawn on the basis of certain load versus imposed displacement response characteristics, comprising the strength (maximum lateral load), the stiffness, and the cumulative energy dissipation capacity. The results demonstrate the important role of including the distribution of fiber reinforced polymers between the beam and the column, column axial load, internal joint (steel) reinforcement, initial damage, carbon versus glass fibers, sheets versus strips, and effect of transverse beams.

Chaallal et al. (2002) investigated retrofitting techniques for reinforced concrete bridges for shear using fiber reinforced polymers. Damage sustained by reinforced members was observed to be due to problems related to marine environments, use of deicing salts and an overall increase in the number and permissible weight end bearing. A total of 14 T-girders were reinforced in shear by externally bonding CFRP sheets to the web and were tested under static loading. Deflection and strain responses were observed yielding positive results for the reinforced specimens as compared to the control ones.

Deniaud and Cheng (2001) investigated the different design methods found in the literature for reinforced concrete beams strengthened externally with FRP sheets. The FRP shear design methods presented include the effective FRP strain and the bond mechanism method with shear friction approach. A total of 16 full-scale T-beam test results were used in the evaluation. The test results showed that the mechanical model using the strip method with shear friction approach evaluates better the FRP shear contribution. The predicted capacities from this mechanical model are also found

conservative and in excellent agreement with the test results. In conclusion, with the exception of the shear friction method, all the design models yield very conservative and scattered predictions with a test-to-predicted coefficient variation exceeding 20%.

Naito et al. (2002) evaluated bridge beam-column joints under simulated seismic loading. The studies of 1989 Loma Prieta earthquake effects resulted in higher design requirements for transverse reinforcement in reinforced concrete bridge beam-column joints constructed in California. In general, the proposed reinforcement details can be congested and difficult to construct. An experimental investigation examined four large-scale interior joints with details typical to those required in California. The experimental program included tied square cross-section columns and spirally reinforced circular cross-section columns. Both conventional and headed joint reinforcement configurations were investigated. Experimental results show that existing design requirements produce joints that remain essentially elastic to relatively large drifts, whereas the columns develop inelastic rotations adjacent to the joints. The use of headed reinforcement within the joint regions was shown to be effective in reducing congestion and thereby improving constructability while maintaining comparable structural behavior.

2.2.3 Combined Flexure and Shear

Sheikh et al. (2002) investigated retrofitting techniques for concrete structures for shear and flexural using fiber reinforced polymers. Damage sustained by foundation walls and large beams in a building was simulated in full-size to near-full-scale model specimens in the laboratory. The damaged specimens were repaired with carbon and

glass fiber-reinforced polymer sheets and tested to failure. Control specimens were tested to failure without rehabilitation to provide a basis for comparison and evaluate the effectiveness of the repair techniques. The results showed that fiber-reinforced polymers were active in strengthening for flexure as well as shear. It was also shown that over reinforcing in flexure resulted in shifting the failure to shear mode, which in some cases may be undesirable. Strengthening of a member in shear, on the other hand, resulted in increasing the ultimate displacement by more than tenfold, and toughness by a factor of more than 26. Available analytical procedures and building code provisions (ACI 318-95) were found to simulate the behaviour of specimens retrofitted with fiber reinforced polymers reasonably well.

Bencardion et al. (2002) studied the strength and ductility of reinforced concrete beams externally reinforced with carbon fiber fabric. They presented results obtained from an experimental investigation of reinforced concrete beams strengthened in flexure and shear using externally bonded epoxy with bi-directional carbon fiber fabric. Two series of three large-scale concrete beam models were built and tested under four-point bending. The beams of the two series were internally reinforced with high-tensile steel for both shear and flexure. One beam for each series without external reinforcement was used as control beam. The difference between the two series was in the amount of internal stirrups and concrete strength. To study the effect of the external composite reinforcement, variations in the number of layers applied and in the external end arrangements were made. One type of external reinforcement consisted of two layers of fabric bonded to the tension face of the beam and partly extended up to the vertical sides.

The second arrangement consisted of four layers of fabric with supplementary end anchorages. The composite reinforcement led to an increase both in the load-carrying capacity at ultimate and flexural stiffness as compared to those for the control beams.

Tung and Campbell (2002) studied the ductility of pretensioned concrete beams with hybrid fiber reinforced polymer and stainless steel reinforcements. Naturally corrosion-resistant materials such as fibre reinforced polymer or stainless steel have been used to adder the corrosion problems in reinforced concrete structures. The authors investigated the behavior of pretensioned concrete beams containing a hybrid arrangement of prestressed carbon fiber reinforced polymer tendons and stainless steel bars. It was observed that conventional measures of ductility based on the yielding of reinforcement are not applicable to concrete beams with only fiber reinforced polymer because of the linearly elastic characteristic of fiber reinforced polymer up to failure. A proposed ductility index based on the consumed plastic and stored elastic energies has been adopted. The application of these energies requires knowledge of the complete loading and unloading load-deflection relationship of a beam. Five hybrid carbon fiber reinforced polymer and stainless steel concrete beams, with variable bonding pattern of carbon fiber reinforced polymer tendons, have been constructed and tested to obtain the complete load-deflection relationships and related ductility indices. Five pretensioned concrete beams with partially-bonded prestressed carbon fiber reinforced polymer tendons and non-prestressed stainless steel bars were constructed and tested to investigate the influence of the bonding pattern of the tendons on the loading and unloading behaviors, and hence on the deformability and ductility of the beams. As many

researchers have concluded, increasing unbonded tendon length improves the deformability of concrete beams. However, the current study showed that bonding pattern has an insignificant effect on the energy dissipation and hence on ductility of the beams.

2.3 History of Fiber Reinforced Polymers

The concept of FRP can be traced back to 3000 B. C., where ancient Egyptians used straw to reinforce mud bricks. The first use of actual FRP was in the early 1950's in aerospace engineering. The materials lightweight, high strength, and stiffness properties were a perfect fit for the industry. The first civil engineering application of FRP was in 1968. It was used on a dome structure built in Benghazi, Libya. Slowly other applications of the composite material followed.

Fibre reinforced polymers are a class of materials that are referred to as composite materials. Composite materials (composites), consist of two or more materials that, when combined, retain their respective chemical and physical characteristics. Composites allow the properties of each material to be utilized to their greatest effect (Mair, 1999). These FRP composites offer the designer many properties (lightweight, ease of application, and high strength increase) that were not available in traditional materials. It is possible to introduce the fibres in the polymer matrix at highly stressed regions in a specific position, direction, and volume in order to obtain the maximum efficiency from the reinforcement within the member; Because of this, the reinforcement within the member can be reduced to a minimal amount at regions of low stress value. Furthermore, FRP's high strength-to-weight ratio is of significant benefit. It has also been shown that FRP response to axial stress exhibit a linear elastic behavior (Chung, 2002). FRP has many positive qualities

some of which: it is resistant to corrosion, low construction costs, and greater efficiency in design compared with more conventional materials. However, like most structural materials, FRP has its disadvantages. These disadvantages have created uncertainty in the civil engineering/construction industry. These disadvantages include, brittle behaviour, susceptibility to deformation under long-term loading, UV degradation, photo degradation (from exposure to light), poor fire resistance, lack of design codes, and most importantly, lack of awareness (Chung, 2002).

2.4 Composition of FRP

The FRP system is comprised of several different parts. The main part of the system is fibre reinforcement, which provides the strength to the system. The layer system that makes up the product placement is shown in Figure 2.1.

Figure 2.1- CFRP System

2.4.1 Fibres

Fibres provide tensile strength and stiffness to the system and are the important constituents in FRP system. The fibre generally occupies 30-70 percent of the matrix volume in the composites and may be in the form of unidirectional strand, chopped strand mat, continuous roving or woven roving. A wide range of amorphous and crystalline materials can be used as the fibre in FRP. In the constructions industry, the most common used fibres are of glass and carbon, both increase strength and ductility in a structural member. Aramid fibres can be used instead of glass fibres to give increased stiffness to the composite.

Carbon fibres can be divided into five grades: general purpose, high strength, ultra high strength, high modulus, and ultra high modulus. Carbon fibers increase compressive strength in a member by a greater percentage than any of the other fibers (see Figure 2-2). This is because the lateral strength developed due to the carbon fiber is much higher than that of glass. Exposure to wet-dry environments has little effect on the compressive strength and ductility of carbon fiber reinforced polymer (CFRP); whereas, exposure to freeze-thaw environments shows a significant degradation in strength and ductility of CFRP (Toutanji & Balaguru, 1998). It should also be noted that carbon fibers have a higher modulus of elasticity.

Glass fibres come in four types, E-glass, AR-glass, A-glass, and Class S high strength glass. The most common used in the construction industry are Class E and a

higher strength Class S. Glass fiber reinforced polymer (GFRP) also improves compressive strength, but not to the same degree as CFRP. Exposure to wet-dry, and freeze-thaw environments show significant reduction in strength and ductility in GFRP (Toutanji & Balaguru, 1998).

Figure 2.2 gives indication of the strength of these composite systems relative to conventional steel.

Figure 2.2- Strain vs Stress Distribution Comparison to Conventional Steel

2.4.2 Epoxy Resins

There are two main types of resin that are used for the application of FRP: thermoplastics and thermosets. Thermoplastics polymers are normally produced in one step and then made into products in a subsequent process. They become soft and

formable when heated, and can be formed and shaped in this soft state. When cooled significantly below their softening point they again become rigid. This type of polymer can be readily recycled because each time it is heated it can again be reshaped or formed into a new article. Thermoplastic resins that are used in composite manufacture include: polyolefins, polyamides, vinylic polymer, polyacetals, polysulphones, polycarbonates, polyphenylenes, and polyimides. Thermosetting polymers are normally produced and formed in the same step; because when heated thermosetting polymers will become soft, but they cannot be shaped or formed to any great extent. The types of thermosetting polymers that are used in the construction industry are polyesters and epoxides.

2.4.3 Protective Coatings

Protective coatings are used to protect the FRP sheets from environmental factors. Factors that include: moisture, chlorine concentration, freezing, thawing, fire, and UV exposure. The reason for the application of protective coatings is the inadequacy of the fiber's to resist many of these factors, and the damage they may cause.

2.4.4 Fire Resistance of FRP

Currently there are serious concerns associated with the behavior of FRP materials at high temperatures and thus reluctance to use this technology. In the past, the applications of FRP have been generally focused on bridges where fire resistance is not a major concern. Before accepting the use of FRP materials in buildings the properties of the material at high temperatures must be known and able to be controlled. The many different types and combinations of FRP available make a general solution to fire

resistance very difficult to attain because different types of fibers have different properties and changes in matrix formulations can have large effects on the high temperature performance of the composite material. Most commonly used epoxies and polyesters form a matrix that is combustible and has mechanical properties, which decline due to even mild increases in temperature. The combustion of the polymer matrix may increase the flame spread and amount of smoke produced in a fire. The smoke produced by the epoxy matrix also has the potential to be toxic. Experiments have shown that a protective layer of intumescent latex added to the exterior of the FRP wrap has successfully yielded acceptable flame spread and smoke generation characteristics.

The increase in temperature due to fire causes a decrease in the mechanical properties of FRP due to the deterioration of the polymer matrix. This reduction in mechanical properties results in a loss of the composite action between the fibres and epoxy as well as a reduction in the bond strength between the FRP system and the underlying concrete member. The matrix will soften at temperatures between 65 and 150°C, which is the glass transition temperature (GTT) for the matrix. If the temperature increases beyond the GTT the strength and stiffness of the epoxy resin are greatly reduced. This leads to the conclusion that any increase in strength or ductility achieved in the member due to the addition of externally bonded FRP would be lost suddenly during a fire if not protected and a dangerous situation could arise. Carbon fibers have good thermal resistance and have properties that are able to withstand temperatures in excess of 1000°C. A reduction in the stress redistribution among fibers due to the failure of the resin at high temperatures significantly reduced the tensile properties of the overall FRP

system at high temperatures. In externally bonded FRP the resin is relied on to transfer the load from the member to the fibers. The loss of composite action between the concrete and FRP will occur at temperatures below 200°C, which are within 10 to 20°C of the GTT.

Based on the data currently available for the fire resistance of FRP general relationships have been derived for the effect of increased temperature on strength and ductility properties of externally bonded FRP. The relationships indicate that the increase in the strength obtained from FRP will be lost within 15-30 minutes of exposure to fire.

The main areas of concern with FRP and fire are as follows:

- Reduction in strength and stiffness due to the softening of the matrix.
- Loss of bond with the underlying concrete due to the softening of the matrix
- Increased heat generation increasing the temperatures locally around the member
- Potential for spalling due to the entrapment of water inside the impermeable wrap

2.4.5 Application Procedure of CFRP

The application of CFRP is one of the key aspects in its versatility. Arriving on the site as a dry flexible fabric, along with the resin, CFRP can be applied in a matter of hours. Its lightweight and ease of handling reduce labour costs significantly over traditional rehabilitation methods.

The following is the step by step application procedure:

- 1. Structure preparation** - Basic repairs must be made to the structure prior to strengthening. Spalled concrete removed, corroded or damaged steel addressed and major cracks injected.
- 2. Surface preparation** - The surface to be repaired is typically abraded to smooth out irregularities, remove contaminants and radius sharp corners. This can be performed by sand blasting or grinding.
- 3. Primer** - In order to promote adhesion and prevent the surface from drawing resin from the FRP, a low viscosity epoxy primer is applied with a roller until the substrate is locally saturated.
- 4. Putty** - An adhesive, high viscosity putty is applied when necessary to the surface to fill in 'bug holes' offsets or voids.
- 5. Cutting fabric-** In a clean area away from the resins, the fabric is carefully measured and cut in accordance with the specifications.
- 6. Saturating fabric** - On large, high volume projects, the fabric can be saturated mechanically before application. For lower volumes & shorter strips, the fabric can be either saturated on a table, or the surface can be coated with resin and the dry fabric applied.
- 7. Applying fabric-** The pre-wetted, or dry, fabric is carefully laid onto the surface and smoothed out to remove air bubbles and ensure that the fibers are straight.
- 8. Quality Control monitoring-** During the cure, 2 to 6 hrs depending on ambient conditions, the fabric is checked to ensure that all air bubbles are removed and that the fabric is not sagging
- 9. Quality assurance-** Good record keeping in accordance with manufacturers QA/QC

procedures ensures a successful finish product.

10. Top coat- Once cured and inspected, the system can be covered with any coating, for aesthetic blending and low maintenance protection.

EXPERIMENTAL PROGRAM

3.1 General

The experimental program was conducted to examine a rehabilitation and strengthening technique for selected precast prestressed double-Tee (DT) girders by externally bonding CFRP sheets. Full description of the girder configurations and material properties, instrumentation, test set-up and test procedure is presented in this chapter.

3.2 DT Girder Configurations

3.2.1 DT-41 Girder

The DT-41 girder is made up of double-tee section with total depth of 810mm. Figures 3.1 to 3.3 show arrangements of prestressing strands for flexure, welded wire mesh for shear, and special bars at the dapped ends as supplied by the precast concrete manufacturer.

Figure 3.1-DT41 Girder Elevation (left side)

Figure 3.2-DT41 Girder Elevation (right side)

Figure 3.3-DT41 Girder Cross Section Details

3.2.2 DT-100 Girder

The DT-100 girder has double-tee cross section with a total depth of 760mm. Figures 3.4 to 3.6 show arrangements of prestressing strands for flexure, welded wire mesh for shear, and special bars at the dapped ends as supplied by the precast concrete manufacturer.

Figure 3.4-DT100 Girder Elevation (right side)

Figure 3.5-DT100 Girder Elevation (left side)

Figure 3.6-DT100 Girder Cross Section Details

3.2.3 DT-102A Girder

The DT-102A girder has double-tee cross section with a total depth of 760mm. Figures 3.7 to 3.9 show arrangements of prestressing strands for flexure, welded wire mesh for shear, and special bars at the dapped ends as supplied by the precast concrete manufacturer.

Figure 3.7-DT102A Girder Elevation (right side)

Table 3.1 Girder Configurations

Figure 3.8-DT102A Girder Elevation (left side)

Figure 3.9-DT102A Girder Cross Section Details

3.2.4 Girder Characteristics

Table 3.1 summarizes the general dimensions of each girder as obtained from the structural drawings supplied by the precast concrete manufacturer, while table 3.2 summarizes the reinforcing details of such girders. Figures 3.11a, 3.11b and 3.11c show mapping of the pre-existing damage in DT-41, DT-100 and DT-102A girders as observed after installing them over the pedestals at the test site. Figure 3.10 shows view of the existing damage at the dapped end of the DT102A girder.

Table 3.1 Girder Configurations

GIRDER CHARACTERISTIC		GIRDER DESIGNATION		
		DT-41	DT-100	DT-102A
Over all Dimension (mm)	Length	16900	19150	19150
	Width	3200	2350	2430
	Depth	810	760	760
	Depth of slab	100	100	100
Dapped End Details (mm)	Depth	445	500	500
	Length	165	200	200
Web Details (mm)	Bottom Width	100	100	100
	Top Width	170	170	170
Distance Between Webs (mm)	Length	1500	1500	1500

Table 3.2 Girder Reinforcing Details

GIRDER CHARACTERISTIC	GIRDER DESIGNATION		
	DT-41	DT-100	DT-102A
Longitudinal Reinforcement	7- 12.7mm, 270K-LL 7 Wire strands pulled to 70%	5-15.8mm Starnd stressed to 72%	5-15.8mm Starnd stressed to 72%
Shear Reinforcement	100x100Wx4 WWF	150x150 w75x75	150x150 w75x75
Distance from ends (mm)	530	500	500
Flange Reinforcement	300x100 W75x100 WWF (1 Layer)	300x100 W50x100 WWF	300x100 W50x100 WWF
Flange Connectors	10 @ 125 o/c both sides	varied over length	varied over length

Figure 3.10- View of Existing Damage in DT102A Girder

Figure 3.11a- Map of Pre-Existing Damage to DT-41 Girder

Figure 3.11b- Map of Pre-Existing Damage to DT-100 Girder

Figure 3.11c- Map of Pre-Existing Damage to DT-102A Girder

3.3 Construction of Girder Supports

3.3.1 Site Preparation

Given the magnitude of the project an off-campus facility had to be used. Once a large enough facility was found, extensive work efforts were undertaken to adapt the bearing capacity of site to be able to adequately receive the added loads from the various equipment and materials that were required to complete the experiment. Please refer to figures 3.12 to 3.14 for a general rendering of such work.

Figure 3.12- Site Clearing and Grubbing

Figure 3.13- Excavation of Drive Isles for Crane

Figure 3.14- Fine Grading of Granular Subgrade

Figure 3.16- Supports for Precast Double-T Girders completed

3.3.3 Delivery of Girders

3.3.2 Construction of Supports

Before delivery of the precast double-T girders could be expected supports big enough to bare the weight of the beams had to be designed and constructed. A total of 12 supports were constructed, 4 per girder, as shown in Figures 3.15 and 3.16. Form work was made up of 2"x12" planks nailed together to form a 1000 mm width x 1000 mm length x 600 mm height block. Details of the materials used are explained in the next section.

Figure 3.15- Support Formwork and Reinforcing Steel

Figure 3.16- Supports for Precast Double-T Girders completed

3.3.3 Delivery of Girders

Once the site was prepared and the supports were positioned, the three girder were delivered to the site. Figure 3.17 to 3.24 demonstrate the procedures used to carry out this work.

Figure 3.17- Placement of Steel Plate Bearing Seats

Figure 3.19- Delivery of Girders by Tractor and Flatbed Trailer

Figure 3.18- Alignment of Double T Girder Supports Complete

Figure 3.19- Delivery of Girders by Tractor and Flatbed Trailer

Figure 3.20- Placement of Double T Girder on Supports

Figure 3.21- Placement of Double T Girder on Supports-Close up

Figure 3.22- Placement of Double T Girder by Mobile Crane

Figure 3.24- Specimen Layout

Figure 3.23- Double T Girder Suspended In Position

3.4 Carbon Fiber Reinforced Polymer Application

3.4.1 Carbon Fiber Material Properties

The reinforcing material, which was applied to the girders, was Tyfo SCH-418 Carbon Fiber Composite Fiber System. This system consists of unidirectional carbon fibers with a 3000 gsm weight-average basis weight. The fibers are woven in a 0° pattern.

Figure 3.24- Specimen Layout

Property	Typical Test Value
Temperature Gradient	82° C
Tensile Strength	50.7 MPa
Tensile Modulus	2.23 GPa
Elongation %	3.5%
Flexural Strength	86.4 MPa
Flexural Modulus	2.18 GPa

Table 3.4- Tyfo SCH-418 Carbon Fiber Properties

Primary carbon fiber Properties (typical ASTM test value)	
Property	Design Value
Ultimate Tensile Strength in primary fiber direction	876MPa
Elongation at Break	1.21%
Tensile Modulus	72.4 GPa
Ultimate Tensile Strength @ 90° to primary fiber Direction	34.5 MPa
Laminate design thickness	1 mm per layer

3.4 Carbon Fiber Reinforced Polymer Application

3.4.1 CFRP Material Properties

The reinforcement material, which was applied to the girders, is Tyfo SCH-41S Fabric Composite Fiber System. This system consists of un-directional carbon composite with Kevlar (white) cross fibers. It has been stitched, with the carbon material oriented in 0° direction, and aramid fibers at 90°. The system properties are summarized in the Tables 3.3 and 3.4 below.

Table 3.3- Tyfo Epoxy Material Properties

Epoxy Material Properties	
Property	Typical Test Value
Temperature Gradient	82° C
Tensile Strength	50.7 MPa
Tensile Modulus	2.23 GPa
Elongation %	3.5%
Flexural Strength	86.4 MPa
Flexural Modulus	2.18 GPa

Table 3.4- Tyfo SCH-41S Carbon Fiber Properties

Primary carbon fiber Properties (typical ASTM test value)	
Property	Design Value
Ultimate Tensile Strength in primary fiber direction	876MPa
Elongation at Break	1.21%
Tensile Modulus	72.4 GPa
Ultimate Tensile Strength @ 90° to primary fiber Direction	34.5 MPa
Laminate design thickness	1 mm per layer

3.4.2 Substrate Preparation

Before the CFRP application can start the concrete substrate must be grinded and or sandblasted to ensure that the interface between the epoxy resin and the concrete is tacky by using a roller, and (iv) apply the carbon fiber reinforced polymers so that it is smooth and uniform, and working out of bubbles. This procedure can be seen in Figure 3.25 below.

Figure 3.25- Substrate Preparation by Mechanical Grinding

3.4.3 Pre-existing Damage Repair

After grinding operations are complete, any voids and or damages are corrected with a high strength concrete patching compound so that a complete and full bonding surface area is available for the CFRP application.

3.4.4 Application of Carbon Fiber Reinforced Polymer Sheet

The installation procedure of the CFRP sheets is as follows; (i) making sure that the surface was free from sharp edges and voids in concrete had to be filled, the corners

must be rounded, brooming the surface to make sure it is clean, placing a protective cover to keep the surrounding areas clean, (ii) mix 100 parts component A and 42 parts component B by volume, (iii) allowing the epoxy to soak into the concrete and become tacky by using a roller, and (iv) applying the carbon fiber reinforced polymers so that it is smooth and uniform, and working out of bubbles. This procedure can be seen in Figure 3.26 below.

Figure 3.26- CFRP Sheet Application

3.5 Carbon Fiber Reinforced Polymer Sheet Application

The CFRP sheets were applied in accordance with the methods recommended by the manufacturer of Tyfo SCH-41S Composite Fiber Systems. Figure 3.27, 3.35 and 3.43 depict the layout and orientation of the CFRP sheets for the DT-41, DT-100 and DT-102A specimens, respectively.

3.5.1 CFRP Arrangement of DT-41 Girder

DT-41 girder was strengthened in flexure and shear. The stems of two of the girder were strengthened in flexures using U-shape un-directional carbon fiber reinforced polymer sheets (CFRP), extending from the mid-span to the quarter points of the girders. Also, the dapped ends of girder stems were strengthened in shear using CFRP wrapping with fibers being oriented horizontally. While the part of the stem over the support is also strengthened with CFRP sheet with fibers being oriented vertically as shown in Figure 3.27. One layer of CFRP sheets was used in each direction of installation. Figures 3.28 to 3.34 show different views of the girder with CFRP sheets.

Figure 3.28- View of DT-41 Girder After Installation of CFRP Sheets

Figure 3.29- View of the N-E Side of the DT-41 Girder

Figure 3.31- View of the N-W Side of the DT-41 Girder

Figure 3.30- View of the S-E Side of the DT-41 Girder

Figure 3.31- View of the N-W Side of the DT-41 Girder

Figure 3.32- View of the S-W Side of the DT-41 Girder

Figure 3.33- View of Underside of DT-41 Girder

Figure 3.34- View of CFRP Sheets at Supports of DT-41 Girder

Figure 3.36- View of DT-100 Girder After Installation of CFRP Sheets

3.5.2 CFRP Arrangement of DT-100 Girder

DT-100 girder was strengthened only in shear. The dapped ends of girder stems were strengthened in shear using CFRP wrapping with fibers being oriented horizontally. Then, other layer of CFRP sheets was added with fibers being oriented vertically as shown in Figure 3.35. Figures 3.26 to 3.42 show different views of the girder with CFRP sheets.

Figure 3.35- Layout of CFRP Sheets for DT-100 Girder

Figure 3.36- View of DT-100 Girder After Installation of CFRP Sheets

Figure 3.37- View of the N-E Side of the DT-100 Girder

Figure 3.38- View of the S-E Side of the DT-100 Girder

Figure 3.39- View of the N-W Side of the DT-100 Girder

Figure 3.40- View of the S-W Side of the DT-100 Girder

Figure 3.41- View of Underside of DT-100 Girder

Figure 3.42- View of CFRP Sheets at Supports of DT-100 Girder

Figure 3.43- Layout of CFRP Sheets for DT-102A Girder

3.5.3 CFRP Arrangement of DT-102A Girder

DT-102A girder has similar arrangement to DT-41 girder for flexure and shear strengthening except that the former has additional layer of CFRP sheets in the vertical direction at the dapped ends as shown in Figure 3.43. Figures 3.44 to 3.50 show different views of the girder with CFRP sheets.

Figure 3.44- View of DT-102A Girder After Installation of CFRP Sheets

Figure 3.43- Layout of CFRP Sheets for DT-102A Girder

Figure 3.44- View of DT-102A Girder After Installation of CFRP Sheets

Figure 3.45- View of the N-E Side of the DT-102A Girder

Figure 3.46- View of the S-E Side of the DT-102A Girder

Figure 3.47- View of the N-W Side of the DT-102A Girder

Figure 3.48- View of the S-W Side of the DT-102A Girder

Figure 3.49- View of Underside of DT-102A Girder

Figure 3.50- View of CFRP Sheets at Supports of DT-102A Girder

3.6 Instrumentation

The instrumentation used for these tests consisted of concrete strain gauges to measure concrete strains and normal strain in CFRP sheets and Linear Variable Differential Transformers (LVDT's) to measure slab deflection.

3.6.1 Strain Gauge Application

Concrete normal strains as well as strains in CFRP sheats at the mid-span section were monitored using concrete strain gauges of 50 mm length. The strain gauges were glued at the predetermined locations on the specimens after the CFRP was applied. The contact surface was first cleaned with CSM-1A degreaser and rubbed with M-Prep conditioner A and left to dry, then the surface was rubbed again with M-Prep Neutralizer 5A. The adhesive used to attach the gauge was M-Bond 200. A total of concrete strain gauges per girder were installed at the mid-span section as shown in Figure 3.51. Other 6 strain gauges were installed over the CFRP sheets at 45° from the girder supports as shown in Figures 3.52 and 3.53. Figures 3.54 to 3.59 show the location of the strain gauges for each girder.

Figure 3.51- Location of Longitudinal Strain Gauges at Mid-Span Section

Figure 3.52- Location of 45° Angle Strain Gauges at Supports

Figure 3.53- Side View of Strain Gauges at Supports

Figure 3.54- Layout of Strain Gauges at the Support of DT-41 Girder

Figure 3.55- Layout of Strain Gauges at Mid-Span of DT-41 Girder

Figure 3.56- Layout of Strain Gauges at the Support of DT-100 Girder

Figure 3.57- Layout of Strain Gauges at Mid-Span of DT-100 Girder

3.6.2 Linear Variable Displacement Transducers, LVDT's

DT-102A were used to measure the slab deflection at the support and at the general arrangement

Figure 3.58- Layout of Strain Gauges at the Support of DT-102A Girder

Figure 3.59- Layout of Strain Gauges at the Mid-Span of DT-102A Girder

3.6.2 Linear Variable Displacement Transducers, LVDT's

Linear Variable Differential Transducers (LVDT's) were used to measure the slab deflection. All of the LVDT had a 250 mm stroke. LVDT's were also positioned at the supports to detect any possible settlement. Figures 3.60 and 3.64 show the general arrangement of the LVDT's at the support and mid-span locations.

Figure 3.60-General Layout of Linear Variable Displacement Transducers at Mid-Span of DT-41 Girder

Figure 3.62- Layout of Linear Variable Displacement Transducers for DT-41 Girder

Figure 3.61- General Layout of Linear Variable Displacement Transducers at Support of DT-41 Girder

Figure 3.63- Layout of Linear Variable Displacement Transducers DT-100

Figure 3.62- Layout of Linear Variable Displacement Transducers for DT-41 Girder

3.6.3 Testing Equipment

3.6.3.1 DT-100

Figure 3.63- Layout of Linear Variable Displacement Transducers DT-100

Figure 3.64- Layout of Linear Variable Displacement Transducers DT-102A

3.6.3 Testing Equipment

3.6.3.1 Crane

A mobile crane having a capacity of 30 metric tons was used for the application of the load. Load increments of 2.2 tons were lifted one at a time by the mobile crane from a tractor with a flat bed trailer and deposited onto the test specimen. Figures 3.65 and 3.66 show the loading crane and flat-bed tractor trailer.

Figure 3.65- View of the Crane Used for Load Application

Figure 3.66- View of Haulage Equipment- Flat Bed

Figure 3.67- Data Acquisition System

3.6.3.2 Data Acquisition System

Data from sensors were recorded using a test control software (TCS) with a SYSTEM 6000 data acquisition unit. The used Data acquisition system is shown in Figure 3.67.

that the precast members were all of the same cross section and length, we were able to accurately measure the dimensions of one barrier and use them incrementally. A check was also calculated by the crane's capacity gauge as it picked up each specimen. It was found that the barrier weight is 2200 kg. Figure 3.68 shows shows view of the loading procedure using the crane.

Figure 3.67- Data Acquisition System

Figure 3.68- Loading Procedure

3.6.3.3 Loading using Jersey Barriers

Loading of girders was carried out by the use concrete Barriers. These precast members are generally used to temporarily divert traffic during road construction works. Given that the precast members were all of the same cross section and length, we were able to accurately measure the dimensions of one barrier and use them incrementally. A check was also calculated by the crane's capacity gauge as it picked up each specimen. It was found that the barrier weight is 2200 kg. Figure 3.68 shows shows view of the loading procedure using the crane.

Figure 3.68- Loading Procedures

After testing the girders, two cores were taken for the averages of each girder to determine concrete compressive strength experimentally. Table 4.1 summarizes the results from core tests against the design compressive strength of concrete as stated in the structural drawings. It should be noted that for a core diameter-to-height ratio of 1, the correction factor for compressive strength of concrete as obtained from core tests is 0.87 (ASTM, 1997). Figures 4.1 and 4.2 show views of core sampling and a core sample after failure, respectively.

Experimental Results and Discussion

4.1 General

As part of a large project to investigate the use of Carbon Fiber Reinforced Polymer (CFRP) Sheets to repair and strengthen damaged or deteriorated precast prestressed concrete Double-Tee (DT) girders in flexure and shear, this project details the strengthening and testing to-collapse three actual-size partially-damaged DT precast pretensioned girders as obtained from the precast manufacturer. All three of the girders were repaired and strengthened, and then tested to failure to determine flexural and shear capacities. The stems of two of these girders were strengthened in flexures using U-shape un-directional carbon fiber reinforced polymer sheets (CFRP), extending from the mid-span to the quarter points of the girders. Two girders were strengthened in shear the dapped ends using 0° wrapping technique around the stem, while the dapped ends of the third girder were strengthened using $0^\circ/90^\circ$ wrapping technique.

After testing the girders, two cores were taken for slab overhangs of each girder to determine concrete compressive strength experimentally. Table 4.1 summarizes the results from core tests against the design compressive strength of concrete as stated in the structural drawings. It should be noted that for a core diameter-to-height ratio of 1, the correction factor for compressive strength of concrete as obtained from core tests is 0.87 (ASTM, 1997). Figures 4.1 and 4.2 show views of core sampling and a core sample after failure, respectively.

Table 4.1 Compressive Strength of Concrete

	Core -1			Core -2			Compressive strength from core tests (MPa)	Design compressive strength (MPa)
	D (mm)	H (mm)	Ult. Load (kN)	D (mm)	H (mm)	Ult. Load (kN)		
T-41 girder	100.5	107.0	340	100.5	107.0	336	37	35
DT-100 girder	100.5	104.5	482	100.5	104.5	447	51	45
DT-102A girder	100.5	110.0	428	100.5	110.0	429	47	45

Note: D: core diameter; H: core height

Figure 4.1- View of Concrete Core Sampling

Figure 4.2- View of a Concrete Core Sample After Failure

Each girder was loaded incrementally up to failure using Jersey Barriers. Each barrier weighs approximately 2200 kg and dimensions shown in Figure 4.3. Figures 4.4, 4.5 and 4.6 show views of the girders before applying barrier weights. This Chapter summarizes crack propagation for each girder as well as the results from sensors as recorded by the data acquisition system. These results can be used in a further study to investigate the performance of CFRP sheet is shear and flexural strengthening.

Figure 4.3- Dimensions of a Jersey Barrier

Figure 4.4- Experimental Setup for DT-41 Girder

Figure 4.5- Experimental Setup for DT-100 Girder

Figure 4.6- Experimental Setup for DT-102A Girder

4.2 Loading History and Crack Pattern

4.2.1 DT-41 Girder

The stems of this girder was strengthened in flexure using U-shape un-directional carbon fiber reinforced polymer sheets (CFRP), extending from the mid-span to the quarter points of the girders. Also, the girder was strengthened in shear the dapped ends using 0° wrapping technique around the stem as detailed in Chapter III. It was intended to strengthen this girder in flexure up to the quarter points and load it with barriers at its middle part between the quarter points to examine whether stopping CFRP sheets in flexure at a point within the girder span, rather than extending them to the supports, would affect the failure mode as well as the load carrying capacity. As such, the girder was loaded by concrete barriers in increments as shown in Figures 4.7 and 4.8. The total number of barriers used to load this girder is 13 as shown in Figure 4.7, in addition to barrier No. 14 after which the test stopped due to observed major wide shear-flexure cracks at the quarter points of each girder.

It should be noted that after applying the 10th barrier, major shear-flexure cracks appeared at the ends of the CFRP sheets between the mid height of the girder and stem bottom face, approximately at the girder quarter points. This appears in views of the girder after failure in Figures 4.13 to 4.19. Delamination of CFRP and concrete was also observed at the ends of the CFRP sheets as shown in Figures 4.15, 4.16 and 4.20. The delamination mode of failure in CFRP strengthened girders is among the least understood and yet, its effect is not desirable from the design point of view because it is a brittle failure mode that can have catastrophic consequences.

Figure 4.7- Sequence of the Application of Jersey Barriers on the DT-41 Girder

Figure 4.8- Cross-sectional View of Barrier Locations for the DT-41 Girder at Mid-span

Figure 4.9- Application of the 9th Barrier Load on the DT-41 Girder

Figure 4.10- Application of the 11th Barrier Load on the DT-41 Girder

Figure 4.11- Application of the 13th Barrier Load on the DT-41 Girder

Figure 4.12- Deflected Shape of DT-41 Girder after Loading the Last Barrier

Figure 4.13- Crack Pattern on the N-E Side of DT-41 Girder after Removing the Barriers

Figure 4.15- Delamination Failure at the End of the CFRP Sheet of DT-41 Girder

Figure 4.14- Crack Pattern on the N-W Side of DT-41 Girder after Removing the Barriers

Figure 4.16- Close-up of the Cracks and Peeling Off of CFRP Sheet of DT-41 Girder.

Figure 4.15- Delamination Failure at the End of the CFRP Sheet of DT-41 Girder

Figure 4.16- Close-up of the Cracks and Peeling Off of CFRP Sheets of DT-41 Girder

Figure 4.17- Crack Pattern on the S-W Side of DT-41 Girder after Removing the Barriers

Figure 4.18- Close-Up of the Main Shear-Flexure Crack on the S-W Side of DT-41 Girder

Figure 4.19- Crack Pattern on the S-E Side of DT-41 Girder after Removing the Barriers

Figure 4.20- Close-Up of the Main Shear-Flexure Crack on the S-E Side of DT-41 Girder

4.2.2 DT-102A Girder

Figure 4.21- Sketch of General Crack pattern of the DT-41 Girder

In addition to the major flexural-shear crack occurred at the ends of CFRP sheets glued to the girder stem, diagonal cracks were observed between this major crack and the mid-span section, with irregular spacing ranging between 150 to 450 mm. Figures 4.13, 4.14, 4.17 and 4.19 show this crack pattern on each side of the double-tee girders. While Figure 4.21 shows a sketch of the general crack pattern after failure.

As it was mentioned in Chapter III, reading from sensors were recorded after applying each barrier weight. Table A.1 shows deflection values at the mid-span section and at the support as obtained from the LVDTs. It can be observed that the maximum girder deflection at the mid-span section at failure was about 158 mm. While, insignificant deflection of the supports was observed, maximum of 2.14 mm after failure. This proves that settlement of the concrete foundation built over ground had insignificant effect on the LVDT readings at the mid-span section. Figure 4.12 shows view of the girder deflected shape after applying the 13th barrier. Tables A.4, A.7 and A.10 show the strain gauge reading at the locations shown in Figure 3.51 and 3.53. These values will be used in further studies to compare the experimental findings with the analytical solution of such girders when strengthened using CFRP sheets.

4.2.2 DT-102A Girder

The stems of this girder was strengthened in flexure using U-shape un-directional carbon fiber reinforced polymer sheets (CFRP), extending from the mid-span to the quarter points of the girders. Also, the girder was strengthened in shear the dapped ends using 0° / 90° wrapping technique around the stem in two layers as detailed in Chapter III. It was intended to strengthen this girder in flexure up to the quarter points and load it with barriers forming uniformly distributed load allover its length to examine whether stopping CFRP sheets in flexure at a point within the girder span, rather than extending them to the supports, would affect the failure mode as well as the load carrying capacity. As such, the girder was loaded by concrete barriers in increments as shown in Figures 4.22 and 4.23. The total number of barriers used to load this girder is 24 as shown in Figure 4.22. Figures 4.24, 4.25 and 4.26 show views of the girder deflected shape during loaded it with barriers.

It was observed that crack were distributed almost uniformly between the mid-span section and the supports, with cracks slopes decreasing towards the supports as a result of the effect of shear combined with flexure. In contrast to the failure mode in DT-41 girder, no major flexural-shear was observed at the ends of the flexural CFRP sheets. This may be attributed to the fact that the applied barrier loading was uniformly distributed all over span. Figure 4.38 shows a sketch of the general crack pattern of DT-102A girder.

Figure 4.22- Sequence of the Application of Jersey Barriers on DT-102A Girder

Figure-4.23 Cross Sectional View of Barrier Locations for DT-102A Girder at Mid-span

Figure 4.24- Application of the 12th Barrier Load on the DT-102A Girder

Figure 4.25- Application of the 17th Barrier Load on the DT-102A Girder

Figure 4.26- Deflected Shape of DT-102A Girder Before Loading the Last Barrier

Figure 4.27- Crack Pattern of the N-W Side of DT-102A Girder, Closer to the Support

Figure 4.28- Crack Pattern of the N-W Side of DT-102A Girder, Closer to the Mid-span

Figure 4.29- Crack Pattern on the North Side of DT-102A Girder at Mid-Span

Figure 4.30- Crack Pattern of the N-E Side of DT-102A Girder, Closer to the Mid-Span

Figure 4.31- Crack Pattern of the N-E Side of DT-102A Girder, Closer to the Support

Figure 4.32- Close-up of Crack Pattern on the N-E Side of the DT-102A Girder

Figure 4.33- Close-Up of the Crack on the N-W Side of the DT-102A Girder

Figure 4.34- Crack Pattern on the S-W Side of DT-102A Girder, Closer to the Support

Figure 4.35- Main Shear-Flexure Crack on the S-W Side of the DT-102A Girder

Figure 4.36- Main Shear-Flexure Crack on the S-W Side of the DT-102A Girder Closer to Support

Figure 4.37- Close-Up of the Main Shear-Flexure Crack on the S-W Side of DT-102A Girder

4.2.3 DT-100 Girder

As a result of the level of deterioration shown in Chapter II, this girder was only strengthened in shear the dapped ends using 0°/90° wrapping technique around the stem in

Figure 4.38- Sketch of General Crack Pattern of the DT-102A Girder

As it was mentioned in Chapter III, reading from sensors were recorded after applying each barrier weight. Table A.2 shows deflection values at the mid-span section and at the support as obtained from the LVDTs. It can be observed that the maximum girder deflection at the mid-span section at failure was about 136 mm. While, insignificant deflection of the supports was observed, maximum of 4.48 mm after failure. This proves that settlement of the concrete foundation built over ground had insignificant effect on the LVDT readings at the mid-span section. Tables A.5, A.8 and A.11 show the strain gauge reading at the locations shown in Figure 3.51 and 3.53. These values will be used in further studies to compare the experimental findings with the analytical solution of such girders when strengthened using CFRP sheets.

4.2.3 DT-100 Girder

As a result of the level of deterioration shown in Chapter III, this girder was only strengthened in shear the dapped ends using $0^\circ/90^\circ$ wrapping technique around the stem in two layers as detailed in Chapter III. The girder was loaded by concrete barriers in increments as shown in Figures 4.39 and 4.40, simulating a failure load with almost uniformly distributed loading. The total number of barriers used to load this girder is 12 as shown in Figure 4.39, however, more barriers were added to examine whether fracture of the prestressing strands would occur.

Figures 4.41 to 4.44 show views of the girder while applying the barrier weight. It should be noted that after applying the 8th barrier, major flexure cracks appeared at the mid-span section. These cracks are repeated towards the supports with increase of barrier loads. The failure of the girder was due to pure flexural crack. No sign of shear cracks were observed near the supports. Figures 4.45 and 4.46 show close-up of the flexural crack at the mid-span section after loading the girder with many barriers so that it rest over the ground. While Figure 4.47 shows view of crack pattern in the girder between the mid-span and the supports after removing the barriers. Figure 4.49 shows a sketch of the general crack pattern after failure with crack regularly spaced at 300 mm.

Figure 4.39- Sequence of the Application of Jersey Barriers on the DT-100 Girder

Figure 4.40- Cross Sectional View of Barrier Locations for the DT-100 girder at Mid-span

Figure 4.41- Application of the 4th Barrier Load on the DT-100 Girder

Figure 4.42- Application of the 9th Barrier Load on the DT-100 Girder

Figure 4.43- Application of the 12th Barrier Load on DT-100 Girder

Figure 4.44- View of the Deflected Shape of DT-100 Girder After Failure

Figure 4.45- Close up of Cracks at Mid-span of DT-100 Girder After Failure

Figure 4.46- Close up of Failure Zone of the DT-100 Girder

Figure 4.47- View of the East side of the DT-100 Girder After Failure

Figure 4.48- Close up of the East Side of the DT-100 Girder After Failure

Figure 4.49- Sketch of General Crack Pattern of the DT-100 Girder

Readings from sensors were recorded after applying each barrier weight. Table A.3 shows deflection values at the mid-span section and at the support as obtained from the LVDTs. It can be observed that the maximum girder deflection at the mid-span section after applying the 12th barrier was about 110 mm. While, insignificant deflection of the supports was observed, maximum of 0.34 mm after failure. This proves that settlement of the concrete foundation built over ground had insignificant effect on the LVDT readings at the mid-span section. Tables A.7, A.9 and A.12 show the strain gauge reading at the locations shown in Figure 3.51 and 3.53. These values will be used in further studies to compare the experimental findings with the analytical solution of such girders when strengthened using CFRP sheets.

CHAPTER V

CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE RESEARCH

5.1 General

Research has been recently directed to the use of epoxy-bonding composite materials in strengthening deteriorated prestressed concrete girders. To ensure successful and cost-effective applications of such materials, engineers need to improve their knowledge with respect to the actual behavior of full-size girders so that they can gain confidence on using these materials on structural strengthening and rehabilitation. This can be achieved by providing more data on testing to-collapse and on field conditions full-scale prestressed girders strengthened using FRP materials. This study details the use of Carbon Fiber Reinforced Polymer (CFRP) Sheets to repair and strengthen precast prestressed concrete Double-Tee (DT) girders in flexure and shear. Three actual-size partially-damaged DT precast pretensioned girders were obtained from the manufacturer. All three of the girders were repaired and strengthened, and then tested to failure to determine flexural and shear capacities. The stems of two of these girders were strengthened in flexures using U-shape un-directional carbon fiber reinforced polymer sheets (CFRP), extending from the mid-span to the quarter points of the girders. Two girders were strengthened in shear the dapped ends using 0° wrapping technique around the stem, while the dapped ends of the third girder were strengthened using $0^\circ/90^\circ$ wrapping technique. Each girder was loaded incrementally up to failure using Jersey Barriers. This project summarizes the loading history and reports test results that can be further used to demonstrate the practicality of girder strengthening with CFRP sheets in field conditions.

5.2 Conclusions and Recommendations for Future Research

The following are the conclusions and recommendations for future research resulting from this research.

1- Experimental results show that strengthening CFRP sheets in flexure for part of the girder span within the mid-span may triggers major flexural-shear crack at the ends of the CFRP sheets. Thus, it is recommended either to extend the flexural CFRP sheets to the supports to provide enough anchorage or to use local anchors at the ends of the CFRP sheets to prevent peeling.

2- Delamination mode of failure in CFRP strengthened girder was observed. Since it is a brittle failure mode that can cause catastrophic consequences, it is recommended that future work focus on developing a good understanding of the mechanisms governing this mode of failure. Such an understanding is crucial for the development of comprehensive design provisions for CFRP strengthened girder.

3- It should be noted that this project report is part of a large project to evaluate the use of CFRP sheets in rehabilitation of precast girders. As such, the recorded data from sensors, reported in Appendix A shall be used to compare the experimental findings with the analytical solution for the load history of such girders, using either strain compatibility method or the materially non-linear finite-element modeling.

REFERENCES

- Antonopoulos, C. P., and Triantafillou, T. C. 2003. "Experimental Investigation of FRP-Strengthened RC Beam-Column Joints," *Journal of Composites for Construction*, Vol. 7, No. 4, pp. 39-49, February.
- Arduini, M., Nanni, A., and Romagnolo, M. 2002. "Performance of Decommissioned RC Girders Strengthened with FRP Laminates," *ACI Structures Journal*, Vol. 99, No. 5, Sept.-Oct., pp. 652-659.
- Arduini, M., Nanni, A. 1997. "Behavior of Precracked RC Beams Strengthened with Carbon FRP Sheets", *Journal of Composites for Construction*, Vol. 1, No. 2, pp. 63-70.
- Bencardino, F., Spadea, G., and Swamy, R. N. 2002. "Strength and Ductility of Reinforced Concrete Beams Externally Reinforced with Carbon Fiber Fabric," *ACI Structural Journal*, Vol. 99, No. 2, pp. 163-171, March-April.
- Bonacci, J. F., and Maalej, M. 2000. "Externally Bonded Fiber-Reinforced Polymer for Rehabilitation of Corrosion Damaged Concrete Beams," *ACI Structural Journal*, Vol. 97, No. 5, pp. 703-711, September-October.
- Bonacci, J. F., and Maalej, M. 2001. "Behavior Trends of RC Beams Strengthened with Externally Bonded FRP," *ASCE Journal of Composites for Construction*, Vol. 5, No. 2, pp. 102-113, May.
- Breña, S. F., Bramblett, R. M., Wood, S. L., and Kreger, M. E. 2003. "Increased Flexural Capacity of Reinforced Concrete Beams Using Carbon Fiber-Reinforced Polymer Composites," *ACI Structural Journal*, Vol. 100, No. 1, pp. 36-46, January-February.
- Chaallal, O. Nollet, M. and Perraton, D. 2002. "Strengthening of reinforced concrete beams with externally bonded fiber-reinforced-plastic plates: design guidelines for shear and flexure" *Canadian Journal of Civil Engineering*, Vol. 25, No. 4, Aug, pp. 692-704.
- Chung, Natalie Y. L. 2002. "Fibre Reinforced Polymer (FRP Composites)", *Personal Research found on www.gnatching.tripod.com/FRP/*
- De Lorenzis, L., Miller, B., and Nanni, A. 2001. "Bond of Fiber-Reinforced Polymer Laminates to Concrete," *ACI Materials Journal*, Vol. 98, No. 3, pp. 256-264, May-June.
- Deniaud, B., and Cheng, O. 2001. "Review of Shear Design Methods for Reinforced Concrete Beams Strengthened with Reinforced Polymer Sheets," *Canadian Journal of Civil Engineering*, 28:271-281.

El-Mihilmy, M., and Tedesco, W. 2000. "Analysis of Reinforced Concrete Beams Strengthened with FRP Laminates", *Journal of Structural Engineering*, Vol. 126, No. 6, pp. 684-691.

El-Mihilmy, M., and Tedesco, Joseph W. 2000. "Deflection of Reinforced Concrete Beams Strengthened with Fiber-Reinforced Polymer (FRP) Plates", *ACI Structural Journal*, Vol. 97, No. 5, pp. 679-688.

Fanning, P. J., and Kelly, O. 2001. "Ultimate Response of RC Beams Strengthened with CFRP Plates," *ASCE Journal of Composites for Construction*, Vol. 4, No. 2, pp. 122-127, May.

Foley, C., Buckhouse, E. R. 1999. "Method to Increase Capacity and Stiffness of Reinforced Concrete Beams" *Practice Periodical on Structural Design and Construction*, Vol. 4, No. 1, pp. 36-42.

Harraq, A., Neale, K. W., and Paultre, P. 2002. "Investigation of Prestressed Concrete Girders Strengthened with Externally Bonded Carbon Fiber Sheets," *4th Structural Specialty Conference of the Canadian Society for Civil Engineering*, Montréal, Québec, Canada Society for Civil Engineering, PP 1-10.

Honorio, U., Wight, R. G., and Erki, M. A. 2002. CFRP Sheets for Strengthening Full-Scale Severely Damaged Concrete Structures, *4th Structural Specialty Conference of the Canadian Society of Civil Engineering*, p. 1-10.

Hunag, P.C., Myers, J. J., and Nanni, A. 2000. Dapped-End Strengthening in Precast Prestressed Concrete Double-Tee Beams with FRP Composites, *Proceedings of the 3rd International Conference on Advanced Composite Materials in Bridges and Structures*, Ottawa, Canada, p. 545-552.

Khalifa, A., Gold, W., Nanni, A., and Abdel Aziz, M. 1998. Contribution of Externally Bonded FRP to Shear Capacity of RC Flexural Members. *ASCE Journal of Composites for Construction*, Vol. 2, No. 4, pp. 195-202.

Khalifa, A., and Nanni, A. 1999. Rehabilitation of Rectangular Simply Supported RC Beams with Shear Deficiencies using CFRP Composites, *Construction and Building Materials*, April, pp. 1-23.

Mair, Robert Ian. 1999. "Fibre Reinforcement Polymers-From Aerospace to Infrastructure", *ATSE Focus Journal*, No. 107, May/June.

Michaluk, C. J., Moehie, J. P., and Mosalem, K. 2002. "Evaluation of Bridge Beam-Column Joints under Simulated Seismic Loading", *ACI Structural Journal*, Vol. 95, No. 3, May-June, p. 353-365.

Naito, C. J., Moehie, J. P., and Mosalem, K. 2002. "Evaluation of Bridge Beam-Column Joints under Simulated Seismic Loading," *ACI, Structural Journal*, Vol. 99, No. 1, pp. 62-71, January-February.

Rahimi, H., and Hutchinson, A. 2001. "Concrete Beams Strengthened with Externally Bonded FRP Plates," *ASCE, Journal of Composites for Construction*, Vol. 5, No. 1, pp. 44-56, February.

Raoof, M., El-Rimawi, J. A., Hassanen, M. A. H. 2000. "Theoretical and Experimental Study on Externally Plated R.C. Beams". *ASCE, Journal of Engineering Structures*, Vol. 22, No.1, pp. 85-101.

Reed, C. E., and Peterman, R. J. 2004. Evaluation of Prestressed Concrete Girders Strengthened with Carbon Fiber Reinforced Polymer Sheets, *ASCE Journal of Bridge Engineering*, Vol. 9, No. 2, pp 185-192.

Saadatmanesh, H., Malek, A. 1998. "Design Guidelines for Flexural Strengthening of RC Beams with FRP Plates" *ASCE, Journal of Composites for Construction*, Vol. 2, No. 4, Nov, pp. 158-164.

Sayed-Ahmed, E., Riad, A. H., and Shrive, N. G. 2004. Flexural Strengthening of Precast Reinforced Concrete Bridge Girders using Bonded Carbon Fiber Reinforced Polymer Strips or External Post-Tensioning, *Canadian Journal of Civil Engineering*, 31: 499-512.

Sebastian, W. M. 2001. "Significance of Midspan Debonding Failure in FRP-Plated Concrete Beams," *ASCE, Journal of Structural Engineering*, Vol. 127, No. 7, pp. 792-798, July.

Shahawy, M., Beitelman, T. E. 1999. "Static and fatigue performance of RC beams strengthened with CFRP laminates" *ASCE, Journal of Structural Engineering*, Vol. 125, No. 6, pp. 613-621.

Spadea, G., Swamy, R. N., and Bencardino, F. 2001. "Strength and Ductility of RC Beams Repaired with Bonded CFRP Laminates," *Journal of Bridge Engineering*, Vol. 6, No. 5, pp. 349-355, September/October.

Sheikh S. A., DeRose, D., and Mardukhi, J. 2002. "Retrofitting of Concrete Structures for Shear and Flexure with Fiber-Reinforced Polymers," *ACI Structural Journal*, Vol. 99, No. 4, pp. 451-459, July-August.

Toutanji, H., and Balaguru, P. 1998. "Durability and Characteristics of Concrete columns wrapped with FRP tow sheets", *Journal of Materials in Civil Engineering*, Vol. 10, No. 1, Feb., pp. 52-57.

Tung, D. P., and Campbell, T. I. 2002. "Ductility of Pretensioned Concrete Beams with Hybrid FRP/Stainless Steel Reinforcements," *4th Structural Specially Conference of the*

Canadian Society for Civil Engineering, Montréal, Québec, Canada Society for Civil Engineering, P. 1-10.

Walker, H. C. 1986. "Design Fundamentals for Durable Parking Structures Floor Slabs", Reprinted in ACI, pp. 98-31 to 98-40.

APPENDIX – A

ROW DATA FROM SENSORS

Table A.1- LVDTs Readings for DT-41 Girder

Barrier load increment	Deflection (mm)				
	LVDT # 1	LVDT # 2	LVDT # 3	LVDT # 4	LVDT # 5
1	3.28	3.11	3.34	0.04	0.04
2	7.27	7.17	7.16	0.11	0.12
3	11.14	10.99	10.87	0.12	0.14
4	14.70	14.61	14.31	0.16	0.18
5	19.22	18.38	18.43	0.19	0.22
6	25.24	23.16	23.74	0.25	0.29
7	32.94	29.21	30.38	0.31	0.32
8	49.96	48.42	40.93	0.49	0.49
9	53.56	50.94	45.38	0.69	0.56
10	72.77	63.72	55.07	0.75	0.62
11	102.89	101.93	103.3	1.01	0.67
12	121.69	119.35	124.23	1.18	0.70
13	142.21	137.86	142.98	1.57	0.72
14	154.32	155.85	157.23	2.14	0.81

Note: LVDTs # 1, 2, and 3 are at mid-span and LVDTs # 4 and 5 are at the support.

Table A.2- LVDTs Readings for DT-102A Girder

Barrier load increment	Deflection (mm)				
	LVDT # 1	LVDT # 2	LVDT # 3	LVDT # 4	LVDT # 5
1	1.63	1.36	1.22	0.00	0.00
2	5.72	5.13	5.15	0.05	0.00
3	7.63	7.04	7.07	0.08	0.01
4	10.23	9.21	9.36	0.14	0.01
5	12.51	11.30	11.59	0.19	0.05
6	14.55	13.45	13.71	0.22	0.07
7	19.2	16.73	17.71	0.33	0.12
8	25.54	20.38	19.54	0.43	0.17
9	51.57	45.32	48.3	0.82	0.37
10	54.66	58.65	52.4	1.11	0.46
11	67.77	77.02	72.3	1.33	0.53
12	70.08	82.75	78.6	1.21	0.49
13	74.30	84.50	80.3	1.55	0.63
14	78.79	87.35	84.6	1.55	0.63
15	88.73	96.71	92.3	1.76	0.77
16	98.80	106.48	101.3	2.02	0.86
17	108.72	112.38	110.45	2.22	0.97
18	111.35	115.00	112.00	2.3	1.11
19	115.69	121.00	118.00	2.31	1.20
20	119.85	125.00	123.00	2.51	1.40
21	124.36	129.00	126.00	3.03	1.63
22	128.21	130.00	129.00	3.57	1.81
23	131.87	134.00	132.00	4.06	1.99
24	134.55	136.00	135.00	4.48	2.28

Note: LVDTs # 1, 2, and 3 are at mid-span and LVDTs # 4 and 5 are at the support

Table A.3- LVDTs Readings for DT-100 Girder

Barrier load increment	Deflection (mm)				
	LVDT # 1	LVDT # 2	LVDT # 3	LVDT # 4	LVDT # 5
1	2.15	3.61	3.19	0.05	0.04
2	4.87	7.28	6.71	0.07	0.04
3	7.84	10.81	10.22	0.07	0.04
4	11.01	14.97	14.26	0.06	0.04
5	14.26	18.56	17.78	0.06	0.01
6	29.26	32.73	32.4	0.06	-0.01
7	52.55	47.67	51.37	0.06	-0.02
8	78.51	74.00	78.79	0.06	-0.04
9	100.61	95.84	101.84	0.02	-0.06
10	105.32	103.54	104.63	-0.08	-0.15
11	108.69	108.21	109.36	-0.15	-0.18
12	-	-	-	-0.22	-0.34

Note: LVDTs # 1, 2, and 3 are at mid-span and LVDTs # 4 and 5 are at the support.

Table A.4- Strain Gauge Readings at Support of DT-41 Girder

Barrier load increment	Strain ($\mu\text{mm/mm}$)					
	Gauge 1	Gauge 2	Gauge 3	Gauge 4	Gauge 26	Gauge 27
1	4.00	-14.00	2.00	0.00	0.00	1.00
2	12.00	-11.00	4.00	4.00	6.00	-4.00
3	13.00	-16.00	9.00	12.00	5.00	-2.00
4	27.00	-2.00	10.00	15.00	10.00	-7.00
5	28.00	-6.00	13.00	30.00	9.00	-6.00
6	28.00	-8.00	18.00	60.00	6.00	-1.00
7	27.00	-11.00	26.00	91.00	4.00	0.00
8	30.00	-7.00	34.00	132.00	1.00	-23.00
9	32.00	2.00	40.00	161.00	3.00	-18.00
10	13.00	-32.00	43.00	243.00	-11.00	-18.00
11	13.00	-22.00	62.00	467.00	-19.00	-10.00
12	-5.00	-54.00	72.00	565.00	-25.00	-24.00
13	-50.00	-102.00	102.00	866.00	-52.00	-19.00
14	-44.00	-86.00	116.00	995.00	-45.00	-51.00

Note: See Figure 3.51 for location of gauges.

Table A.5- Strain Gauge Reading at Supports of Girder DT-102A

Barrier load increment	Strain ($\mu\text{mm/mm}$)					
	Gauge 1	Gauge 2	Gauge 3	Gauge 4	Gauge 26	Gauge 27
1	9	3	0	-7	0	0
2	15	6	-2	-10	3	1
3	14	10	-2	3	3	1
4	17	16	-1	-15	6	11
5	13	19	-2	-13	5	8
6	6	24	-3	-15	8	14
7	-2	32	-5	-16	7	15
8	30	25	-3	-29	10	20
9	61	37	-3	-23	10	21
10	55	40	0	-27	8	21
11	17	42	-1	-35	8	21
12	54	40	0	-27	6	19
13	-18	39	-2	-44	5	28
14	-18	39	-2	-44	5	28
15	35	-122	1	-35	6	31
16	43	-107	0	-48	3	30
17	30	-96	6	-47	5	37
18	-46	22	5	-49	0	38
19	12	20	5	-44	0	40
20	5	45	-2	-40	9	32
21	19	19	12	-3	-3	51
22	-33	34	36	55	-4	75
23	-31	30	47	85	-7	91
24	21	99	41	91	12	83

Note: See Figure 3.51 for location of gauges.

Table A.6- Strain Gauge Reading at Supports of Girder DT-100

Barrier load increment	Strain ($\mu\text{mm/mm}$)					
	Gauge 1	Gauge 2	Gauge 3	Gauge 4	Gauge 26	Gauge 27
1	-6	-13	6	27	-3	2
2	-4	-17	17	46	0	4
3	-6	-21	21	66	-1	7
4	-10	-30	37	103	-2	18
5	-9	-35	48	129	-2	21
6	-6	-35	58	134	0	24
7	-7	-40	70	61	-2	35
8	-11	-45	72	89	-6	31
9	-10	-40	93	479	-5	44
10	-11	-37	112	442	-7	55
11	-13	-28	121	402	-9	66
12	0	39	142	358	-3	77

Note: See Figure 3.51 for location of gauges.

Table A.7- Strain Gauge Readings at Top of Deck Slab of DT-41 Girder

Barrier load increment	Strain ($\mu\text{mm/mm}$)					
	Gauge 5	Gauge 6	Gauge 7	Gauge 8	Gauge 9	Gauge 10
1	-22	-22	-17	-22	-23	-19
2	-54	-52	-48	-51	-57	-49
3	-78	-75	-71	-73	-83	-77
4	-105	-102	-99	-98	-110	-103
5	-123	-122	-119	-119	-133	-129
6	-146	-158	-164	-154	-174	-173
7	-199	-202	-206	-191	-221	-220
8	-300	-294	-278	-256	-289	-276
9	-373	-365	-350	-326	-354	-342
10	-460	-446	-410	-384	-404	-400
11	-559	-539	-492	-457	-450	-470
12	-634	-616	-565	-522	-501	-525
13	-762	-734	-661	-618	-596	-623
14	-889	-953	-733	-731	-704	-729

Note: See Figure 3.51 for location of gauges.

Table A.8- Strain Gauge Reading at Top of Deck Slab of Girder DT-102A

Barrier load increment	Strain ($\mu\text{mm/mm}$)					
	Gauge 5	Gauge 6	Gauge 7	Gauge 8	Gauge 9	Gauge 10
1	-10	-26	7	-17	-11	-6
2	-39	-49	-23	-40	-38	-21
3	-42	-64	-34	-51	-45	-21
4	-71	-81	-54	-68	-63	-26
5	-83	-97	-70	-81	-76	-29
6	-99	-112	-84	-94	-86	-34
7	-114	-134	-106	-110	-106	-30
8	-147	-170	-146	-157	-145	-17
9	-246	-249	-242	-238	-220	37
10	-263	-280	-277	-269	-255	3
11	-287	-330	-306	-294	-278	-7
12	-252	-316	-283	-273	-259	-3
13	-309	-356	-332	-321	-298	-11
14	-309	-356	-332	-321	-298	-11
15	-326	-372	-358	-345	-318	-17
16	-343	-402	-382	-368	-340	-20
17	-358	-420	-404	-385	-356	-19
18	-378	-441	-422	-408	-371	-18
19	-416	-496	-475	-451	-432	-15
20	-483	-559	-542	-508	-509	-7
21	-540	-620	-607	-563	-574	-4
22	-574	-654	-648	-594	-594	-2
23	-604	-689	-688	-637	-623	-5
24	-654	-744	-732	-676	-643	-5

Note: See Figure 3.51 for location of gauges.

Table A.9- Strain Gauge Reading at Top of Deck Slab of Girder DT-100

Barrier load increment	Strain ($\mu\text{mm/mm}$)					
	Gauge 5	Gauge 6	Gauge 7	Gauge 8	Gauge 9	Gauge 10
1	-30	-27	-26	-22	-21	-18
2	-59	-53	-52	-48	-46	-42
3	-88	-81	-79	-74	-73	-70
4	-119	-109	-108	-99	-97	-91
5	-149	-135	-135	-121	-121	-116
6	-221	-162	-212	-192	-153	-188
7	-277	-188	-276	-249	-201	-251
8	-284	-196	-261	-258	-210	-268
9	-357	-243	-356	-329	-310	-343
10	-428	-293	-431	-394	-419	-409
11	-545	-385	-548	-499	-543	-522
12	-712	-538	-716	-655	-698	-727

Note: See Figure 3.51 for location of gauges.

Table A.10- Strain Gauge Readings in the Stem of DT-41 Girder

Barrier load increment	Strain ($\mu\text{mm/mm}$)																
	Gauge 11	Gauge 12	Gauge 13	Gauge 14	Gauge 15	Gauge 16	Gauge 17	Gauge 18	Gauge 19	Gauge 20	Gauge 21	Gauge 22	Gauge 23	Gauge 24	Gauge 25	Gauge 28	Gauge 29
1	-10	7	22	-17	74	51	39	20	6	-12	-11	5	13	22	71	56	62
2	-25	15	58	47	184	126	92	52	17	-24	-27	10	36	60	157	132	125
3	-35	23	81	90	251	181	133	75	20	-34	-39	18	52	89	253	187	170
4	-49	28	106	144	322	240	178	101	31	-41	-49	26	72	122	325	251	243
5	-55	37	132	191	382	285	212	120	38	-51	-55	32	91	152	385	299	286
6	-72	50	177	284	496	371	279	158	51	-65	-63	65	132	341	1458	386	380
7	-86	75	239	388	542	398	334	227	83	-76	-43	99	182	1075	2223	408	401
8	-25	8	419	2402	3008	2761	1676	1365	875	-39	13	34	352	1350	2178	385	380
9	55	-16	906	3003	3633	3277	2300	2190	836	-14	45	21	607	1593	2384	421	415
10	196	-27	1251	3017	3810	3433	2546	2491	767	11	69	-6	801	1859	2717	1582	1578
11	414	139	1659	3514	4503	3833	3068	2883	645	37	96	-6	1133	2365	3437	1926	1920
12	552	321	2092	4077	5283	4329	3516	3193	642	52	115	-13	1362	2574	4015	2441	2435
13	727	330	2635	4883	5907	5103	4129	3420	578	75	120	-18	1843	2895	4760	3318	3315
14	1038	114	3277	6143	6645	6348	5020	3691	545	83	117	-21	2447	3276	5681	4539	4535

Note: See Figure 3.51 for location of gauges.

Table A.11- Strain Gauge Reading in the Stem of Girder DT-102A

Barrier load increment	Strain ($\mu\text{mm/mm}$)																
	Gauge 11	Gauge 12	Gauge 13	Gauge 14	Gauge 15	Gauge 16	Gauge 17	Gauge 18	Gauge 19	Gauge 20	Gauge 21	Gauge 22	Gauge 23	Gauge 24	Gauge 25	Gauge 28	Gauge 29
1	10	18	36	40	45	27	21	9	-3	-3	10	25	38	53	0	0	0
2	26	54	104	130	135	79	62	19	-11	-8	28	70	108	147	3	1	1
3	35	72	130	193	171	99	77	27	-15	-10	36	89	136	186	3	1	1
4	46	100	179	331	243	136	106	39	-20	-12	50	121	187	253	6	11	10
5	54	120	258	550	350	163	125	47	-22	-14	59	145	224	302	5	8	7
6	69	151	480	912	529	219	151	58	-23	-13	69	168	258	349	8	14	12
7	95	214	957	1458	874	348	194	76	-24	-10	82	189	282	348	7	15	13
8	126	842	1845	1879	1416	700	887	1182	-3	58	58	527	638	1179	10	20	15
9	-8	2177	2667	2164	1982	1371	2456	-	43	7	64	1212	914	1869	10	21	20
10	-7	2414	2679	2383	2244	1619	2822	-	43	-3	68	1792	1166	2186	8	21	20
11	-7	2533	2799	2591	2441	1768	3088	-	35	3	50	2028	1408	2466	8	21	20
12	-7	2367	2658	2436	2277	1638	2843	-	47	-4	67	1898	1297	2273	6	19	18
13	-10	2669	2890	2800	2666	1929	3243	-	6	24	34	2158	1571	2738	5	28	27
14	-10	2669	2890	2800	2666	1929	3243	-	6	24	34	2158	1571	2738	5	28	27
15	-13	2751	2974	2907	2909	1814	3384	-	-4	28	31	2269	1739	2995	6	31	30
16	-13	2833	3088	3089	3118	1939	3522	-	-13	30	29	2393	1888	3249	3	30	29
17	-13	2787	3193	3266	3312	2002	3588	-	-21	31	25	2466	2030	3461	5	37	35
18	-12	2717	3335	3447	3289	2098	3689	-	-34	30	24	2591	2295	3414	0	38	35
19	-12	2954	3734	4010	3824	2427	3972	-	-51	33	25	2886	2710	3949	0	40	38
20	-18	3268	4260	4625	4563	2928	4210	-	-54	36	31	3231	3450	4660	9	32	30
21	-23	3653	4809	3569	5265	3386	4406	-	2	37	37	3633	4456	5463	-3	51	50
22	-28	3850	5225	2857	5110	3504	4619	-	-1	39	35	3864	5021	5919	-4	75	70
23	-31	4104	5640	2825	5434	3604	4859	-	-13	42	35	4079	5514	6353	-7	91	90
24	-29	4540	5877	3042	5925	3834	5237	-	-21	40	34	4374	6024	6961	12	83	80

Note: See Figure 3.51 for location of gauges.

Table A.12- Strain Gauge Reading in the Stem of Girder DT-100

Barrier load increment	Strain ($\mu\text{mm/mm}$)																
	Gauge 11	Gauge 12	Gauge 13	Gauge 14	Gauge 15	Gauge 16	Gauge 17	Gauge 18	Gauge 19	Gauge 20	Gauge 21	Gauge 22	Gauge 23	Gauge 24	Gauge 25	Gauge 28	Gauge 29
1	-10	11	28	51	70	67	48	30	9	-13	-5	0	12	20	44	59	58
2	-21	20	59	105	140	132	97	62	19	-26	-10	5	37	68	106	129	125
3	-30	33	86	157	208	197	142	94	29	-35	-19	14	65	114	166	206	201
4	-38	44	117	210	282	269	191	130	41	-45	-21	21	92	153	258	280	275
5	-47	57	148	265	352	322	240	164	54	-54	-28	27	122	203	322	401	389
6	-32	94	171	288	332	304	309	209	107	-35	-17	61	148	202	268	409	398
7	-22	101	176	290	316	295	321	220	116	-22	2	67	147	212	285	398	388
8	-26	102	179	295	323	300	319	221	114	-23	2	70	150	214	282	439	429
9	-9	102	170	271	299	280	292	210	121	-3	29	62	139	201	269	450	445
10	8	103	166	264	295	280	288	202	125	20	24	58	139	202	260	508	504
11	54	100	152	248	283	267	268	183	122	76	21	58	134	196	247	475	472
12	104	94	143	236	274	260	256	171	116	145	33	-	132	182	239	402	400

Note: See Figure 3.51 for location of gauges.