

**FLEXURAL STRENGTHENING OF CORRODED REINFORCED
CONCRETE BEAMS USING HYBRID SYSTEM**

BY

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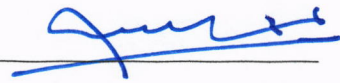
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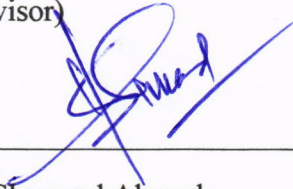
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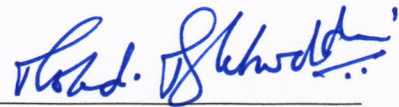
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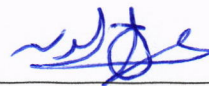
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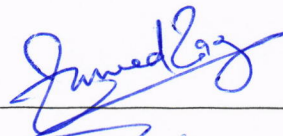
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Dedicated to

My **beloved** parents, for their love and **support**,
my brother and sister for their **support** and **encouragement.**

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TABLE OF CONTENTS

ACKNOWLEDGMENTS	V
TABLE OF CONTENTS	VI
LIST OF TABLES.....	VIII
LIST OF FIGURES.....	IX
LIST OF ABBREVIATIONS.....	XIII
ABSTRACT	XV
ملخص الرسالة	XVII
CHAPTER 1 INTRODUCTION.....	1
1.1 General.....	1
1.2 Significance of the research	3
1.3 Objectives.....	4
CHAPTER 2 LITERATURE REVIEW	5
2.1 Usage of CFRP as Strengthening Material.	5
2.2 Usage of UHPC as a Strengthening Material	11
2.3 Usage of hybrid system of UHPC and CFRP as a strengthening material	18
CHAPTER 3 EXPERIMENTAL PROGRAM	20
3.1 General.....	20
3.2 Test Specimens	20
3.3 Materials and Properties	24
3.3.1 Normal Concrete (NC).....	24
3.3.2 Steel Reinforcements	26
3.3.3 UHPC.....	27
3.3.4 Repair Material – Bonding Agent	29
3.3.5 CFRP – Epoxy resin	30
3.3.6 Bond between UHPC and Normal Concrete.....	31
3.3.7 Bond between UHPC and CFRP.....	32
3.4 Accelerated Corrosion Process.....	33
3.5 Cover Replacement Technique	36
3.6 CFRP Strengthening Technique	37

3.7	UHPC Jacket Application Technique	39
3.8	Setup Instruments and Loading.....	40
CHAPTER 4 DISCUSSION OF THE EXPERIMENTAL RESULTS.....		42
4.1	General	42
4.2	Corrosion Induced Concrete Cracking.....	42
4.3	Gravimetric Analysis and Steel Mass Loss	43
4.4	Mechanical Performance of Beam Specimens.....	46
4.4.1	Strengthening of Corroded Beams with CFRP Laminates along with Different Cover Replacements.	46
4.4.2	Strengthening beams with hybrid System	59
CHAPTER 5 ANALYTICAL SOLUTION		67
5.1	General	67
5.2	Analytical Model of Strengthened Corroded RC Beams with CFRP Along with Different Cover Replacement	67
5.3	Analytical Model of Strengthened Corroded RC Beams with Hybrid System of CFRP and UHPC....	73
CHAPTER 6.....		80
CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE WORK.....		80
REFERENCES.....		84
VITAE.....		88

LIST OF TABLES

Table 1: Summary of corroded strengthened beam results by [11].	9
Table 2: Percentage of pre-loading and curing type.	12
Table 3: Flexural test results of beam specimens.	12
Table 4: The description of the beam specimens with different cover repairing strategies and strengthening strategies.	22
Table 5: UHPC mix proportions.	27
Table 6: Gravimetric analysis results.	45
Table 7: Beams experimental results (control beams and beams strengthened with CFRP only).	49
Table 8: Results of beam specimens (hybrid).	64
Table 9: Experimental and analytical results	72
Table 10: Analytical Results of Beam Specimens	78

LIST OF FIGURES

Figure 1: Two different strengthening configurations [5].	6
Figure 2: The figure on the left is for configuration b. The figure on the right is for configuration	6
Figure 3: Five CFRP strengthening configurations [6].	7
Figure 4: C0 represents the control beam, C15 represents corroded beam by 15%, S15 represents strengthened corroded beam, RS15 represents replaced covered by a new one and strengthened corroded beam, SP15 strengthened + U-shaped corroded beam.	8
Figure 5: a-First method, 10% avg. mass loss. b- Third method 20% avg. mass loss.....	10
Figure 6: Strengthened beams have lower deflection values and higher load capacities.	11
Figure 7: UHPC jacket strengthening configurations for RC beam specimens.....	13
Figure 8: Composite beam set-up.	13
Figure 9: Force vs. midspan deflection curves for beams strengthened with UHPC.	14
Figure 10: Beam dimensions and UHPC strengthening configurations.	15
Figure 11: UHPC strengthening Configurations by [20].	15
Figure 12: UHPC bonding methods. (a) Using epoxy. (b) In situ casting after sandblasting.	16
Figure 13: Analytical solution model for slab strengthened with UHPC proposed by [22].	17
Figure 14: UHPC layer strengthened with steel bars.....	17
Figure 15: [25] configurations. a) without shear connector. b) with shear connector	18
Figure 16: Beam geometry and reinforcement dimensions.	21
Figure 17: Original beam + repair material cover + CFRP + UHPC jacket	23
Figure 18: Corroded beam with the repaired cover and CFRP strengthening	23
Figure 19: Corroded beam with repaired cover and a hybrid strengthening (CFRP + UHPC)	24
Figure 20: Casting of the beam Specimens.....	25
Figure 21: (a) NC compression + elastic modulus test. (b) NC split tensile test.....	25
Figure 22: Compressive stress-strain curve of NC.	26
Figure 23: Steel tensile test	26

Figure 24: Steel fibers used in UHPC (a) Straight. (b) Hooked.	27
Figure 25: Material testing for UHPC: (a) Elastic modulus. (b) Dogbone direct tensile test	28
Figure 26: Stress-strain curve for UHPC in compression.....	29
Figure 27: Stress-strain curve for UHPC from a direct tensile test.	29
Figure 28: Modulus of elasticity test for the repair material <i>MasterFlow 980</i>	30
Figure 29: Split tensile test between UHPC and normal concrete.....	31
Figure 30: Slant shear test between UHPC and CFRP	32
Figure 31: Bond test setup	33
Figure 32: Corrosion system components.....	34
Figure 33: Detailed description of corrosion setup components.	35
Figure 34: Water spray system in corrosion tank.	35
Figure 35: (a) Cover removal. (b) Bonding agent application. (c) Repair material placement. (d) Water curing.....	37
Figure 36: U-shaped CFRP dimensioning	38
Figure 37: CFRP work.....	39
Figure 38: Sandblasted beams (after corrosion)	40
Figure 39: UHPC casting.....	40
Figure 40: Details of the test setup	41
Figure 41: Corrosion cracks.....	43
Figure 42: Gravimetric analysis. (a) extracting the bar. (b) submerging the bars in HCl solution. (c) brushing the bars. (d) drying the bars.	44
Figure 43: An example showing the mass after each cycle of cleaning corrosion products.	45
Figure 44: Load vs. deflection curve for control corroded and un-corroded beams.....	47
Figure 45: Flexural cracks at failure for (a) 10% corroded beam with original cover (CA). (b) 20% corroded beam with original cover (CB). (c) Beams with repair material cover (CAR and CBR). (d) Beam with UHPC cover (CBH).....	48
Figure 46: Load vs. deflection curve for corroded un-strengthened beams with different cover material.	49

Figure 47: Load vs. deflection curve for corroded beams strengthened with one layer of CFRP over the original cover	50
Figure 48: Cracking behavior of one layer of CFRP strengthening on (a) original cover. (b) repair material cover. (c) UHPC cover.	52
Figure 49: Load vs. deflection curve for corroded beams strengthened with one layer of CFRP over different cover material	53
Figure 50: (a) Rupture for bBeams with one CFRP Layer. (b) de-bonding for Beams with Two Layers of CFRP by rupture of U-shaped CFRP.....	55
Figure 51: Approximate stiffness enhancement % with respect to CAR and CBR beams.	56
Figure 52: Load vs. deflection curves for corroded beams with the same cover replacement on variable CFRP layers.	56
Figure 53: Load vs. strain for beams strengthened with one and two CFRP layers.	58
Figure 54: Crack behavior for beams strengthened with (a) UHPC + one layer of CFRP (b) UHPC + two CFRP layers.	60
Figure 55: Percentage reduction in the ductility for beams corroded to 10% and strengthened with the hybrid system.	61
Figure 56: Percentage reduction in the ductility for beams corroded to 20% and strengthened with the hybrid system.	61
Figure 57: Approximate stiffness enhancement for 10% corrosion for CA beam	63
Figure 58: Approximate stiffness enhancement for 20% corrosion for CB beam.....	63
Figure 59: Percentage of maximum load enhancement for the corrosion control beam 'CA.'	64
Figure 60: Percentage of maximum load enhancement for the corrosion control beam 'CB.'	65
Figure 61: Load vs. deflection for control corroded and un-corroded beams.	65
Figure 62: Load vs. deflection for hybrid beams and only CFRP strengthened beams corroded to 10%	66
Figure 63: Load vs. deflection for hybrid beams and only CFRP strengthened beams corroded to 20%	66
Figure 64: Analytical model for beams strengthened with CFRP only.	68

Figure 65: Analytical model flow chart	71
Figure 66: Comparison between analytical and experimental results	72
Figure 67: (a) UHPC behavior in tensile. (b) Steel behavior in tensile.	74
Figure 68: Analytical Model definition	74
Figure 69: Analytical model flow chart	77
Figure 70: Comparison between the analytical and the experimental results.....	78

LIST OF ABBREVIATIONS

A_s	:	<i>Area of a non-corroded steel bar. (mm^2)</i>
A_{sc}	:	<i>Area of the corroded steel bar. (mm^2)</i>
A_f	:	<i>Area of one layer of CFRP. (mm^2)</i>
A_c	:	<i>Cross-sectional area of the replaced concrete cover. (mm^2)</i>
N	:	<i>Number of CFRP layers.</i>
a	:	<i>Depth of the effective compression block in a concrete beam. (mm)</i>
b	:	<i>Width of the beam. (mm)</i>
x	:	<i>Depth from top to the neutral axis. (mm)</i>
h_c	:	<i>Height of concrete beam.</i>
d	:	<i>Effective depth (from top to the tension steel). (mm)</i>
T	:	<i>The sum of tension forces. (kN)</i>
C	:	<i>The sum of compression forces. (kN)</i>
T_s	:	<i>Tensile force in the tension steel. (kN)</i>
T_f	:	<i>Tensile force in CFRP. (kN)</i>
T_{ct}	:	<i>Tensile force in the replaced concrete cover. (kN)</i>
C_c	:	<i>Compression force in concrete. (kN)</i>

C_u	:	Compression force in UHPC. (kN)
$f_{c'}$	:	Compressive strength of the concrete. (MPa)
$f_{cu'}$	:	Compressive strength of the UHPC. (MPa)
f_y	:	Yield stress in tension steel. (MPa)
f_{ut}	:	Tensile stress in the replaced concrete cover. (MPa)
E_f	:	Tensile modulus of elasticity of CFRP. (GPa)
E_s	:	Steel modulus of elasticity. (GPa)
ϵ_f	:	Strain in CFRP.
ϵ_s	:	Actual strain in the steel.
ϵ_y	:	Yield strain in steel.
β_1	:	Factor relating depth of equivalent rectangular compressive stress block to neutral axis depth.
b_c	:	Width of the concrete beam.
b_u	:	Thickness of the UHPC Jacket x 2.
$b_{u,b}$	:	$b_u + b_c$.
h_u	:	Height of UHPC bottom jacket.

ABSTRACT

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The load-carrying capacity of reinforced concrete (RC) structures is significantly reduced in many situations, such as overloading, dynamic loading, shrinkage, creep, and reinforcement corrosion. The loss of load-carrying capacity of the RC members due to reinforcement corrosion is one of the major causes of the reduction in the service life of RC structural members. Considering the necessity of developing a strategy for effective strengthening of the corroded RC members, the proposed study aims to investigate the flexural performance of corroded RC beams strengthened using the Carbon Fiber Reinforced Polymer (CFRP) sheets alone as well as using a hybrid system comprising of CFRP sheets and the layers of Ultra-High-Performance Concrete (UHPC). Twenty RC beam specimens were designed, prepared, and then subjected to accelerated reinforcement corrosion using the impressed current technique for causing loss of steel reinforcement to the targeted degrees. Nine beam specimens were corroded to cause a 10% mass loss, while the other eleven beam specimens were corroded for achieving a 20% mass loss. One extra RC beam specimen was prepared and used as an un-corroded control specimen. The two groups of the beam specimens having different percentages of loss in bar mass, as mentioned above, were strengthened using similar configurations. All the twenty-one RC beam specimens were tested in flexure to evaluate the performance of strengthening using

the load-deflection data recorded during the flexural testing. Finally, analytical models were developed and validated to predict the flexural capacity of the corroded RC beams strengthened using the CFRP sheets alone as well as using the hybrid system of CFRP and UHPC. The experimental and analytical results showed a good agreement. |

ملخص الرسالة

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CHAPTER 1

INTRODUCTION

1.1 General

Reinforcement corrosion is one of the common durability problems that commence in reinforced concrete (RC) members either due to chloride penetration or carbonation, or both. Corrosion of steel bars affects both the steel bar itself and the surrounding concrete area since corrosion products consume part of the cross-sectional area of the steel bar along with the deposition of a higher volume of the corrosion products on the surface of the bars that causes cracking and delamination in the concrete cover. With the increase of the loss in the steel reinforcement cross-sectional area, the RC member's flexural strength decreases, and the delamination increases until reaching a stage where the member is no longer safe. An immediate repairing/strengthening action is required. Many studies have been conducted to determine the solutions to delay/prevent the initiation of reinforcement corrosion in the past. However, it is observed and found that the reinforcement corrosion can be delayed, it cannot be fully prevented, and hence the risk of damage of a RC member due to reinforcement corrosion is always there. Therefore, it is highly desirable to keep repairing/strengthening corrosion-damaged RC members using suitable materials and methods. In the past, damaged RC structures, such as bridges, were strengthened using very heavy steel plates. However, these plates were prone to corrosion and required special instruments to install them, making the strengthening expensive and inconvenient.

CFRP strengthening has been a common repair solution for the last few years due to its easy, fast installment and high strength to weight ratio. However, this technique has two main drawbacks. First, it is relatively costly compared to the other repair and strengthening materials. Second, and most important, it has low fire resistance. CFRP laminates consist of carbon fibers embedded in hardened epoxy resin, and carbon fibers are considered thermally and mechanically stable that can maintain their strength at elevated temperature, while epoxy resin change from the hardened glassy stage into the viscous stage at low temperature (80-100 °C) that causes loss in the bond between the fibers and the host concrete, thus carbon fibers become ineffective. Moreover, at a temperature over (300-400 °C), the epoxy resin decomposes and vaporizes, releasing toxic vapors and combustible volatiles that affect the environment and limit the vision range [1–4].

Recently, ultra-high-performance concrete (UHPC) has emerged as a smart concrete material that can be used for the construction of special structures and repairing/strengthening material with numerous advantages. A mixture of UHPC consists of a high percentage of cementitious materials (Portland cement and silica fume), very fine sand as a sole aggregate, steel fibers (to offer ductility and fracture toughness), and prepared using a very low water/cementitious materials ratio maintaining the consistency with the help of a relatively higher dosage of superplasticizer. UHPC possesses over 150 MPa compressive strength after 28 days of curing. The use of UHPC as a strengthening material for corrosion damaged RC members has not been widely reported in the literature. The most focus is on undamaged beams since the material has been recently developed. The only drawback of using the layers of UHPC to strengthen a RC member is its higher weight compared with CFRP sheets.

To overcome the relative drawbacks of both UHPC and CFRP, when used alone for strengthening beams, the performance of a hybrid strengthening system, comprising of both CFRP and UHPC, was evaluated through the present research work. An accelerated corrosion set-up and a specific procedure were used to induce targeted degrees of reinforcement corrosion in the bars embedded in different RC beam specimens, discussed in detail under the experimental program. The main experimental variables that were considered in the present study included degree of reinforcement corrosion, materials for repairing the damage after removing the damaged concrete cover, number of CFRP layers, and thickness of UHPC layers. The strengthening strategies (i.e., different combinations of the variables) were evaluated using the load versus midspan deflection data recorded during testing of the beam specimens under flexure. Furthermore, the experimental data generated through the present study was used to develop the analytical models towards an attempt to predict the load-carrying capacity of the strengthened RC beams, strengthened by CFRP alone as well as by a hybrid of CFRP and UHPC.

1.2 Significance of research

Through the literature review, it is noticed that the sizable research is carried out on the utility of the CFRP and UHPC for strengthening the conventional RC members for enhancement of their load-bearing capacity. However, there is a lack of information available in the literature about the performance of using UHPC and/or UHPC-CFRP hybrid system in the strengthening of corroded RC beams to restore/upgrade their load-bearing capacity. In the present research work, an integrated mechanistic and analytical approach was adopted to study the flexural behavior of strengthened beams as an attempt towards developing a new approach to carry out the parametric studies. Furthermore, the

set-up, which was used for inducing real-life reinforcement corrosion in large-size beams, is probably the latest development in the field of accelerated corrosion of steel in concrete.

1.3 Objectives

The main objective of this study was to explore the possibility of developing a new hybrid system of UHPC and CFRP to strengthen the corrosion-damaged RC beams leading towards more effective and advantageous solutions.

The specific objectives of this research were as follow:

- 1 To investigate the performance of the CFRP system for the strengthening of the corroded RC beams.
- 2 To investigate the performance of UHPC-CFRP hybrid system for strengthening the corroded RC beams as an alternative option.
- 3 To develop and validate analytical models to predict the ultimate moment carrying capacity of RC corroded beams strengthened with the CFRP sheets alone as well as with the hybrid system comprising of UHPC layers and the CFRP sheets.

CHAPTER 2

LITERATURE REVIEW

In this chapter, a detailed literature review is presented to update the knowledge about strengthening the damaged RC beams using different options of strengthening materials and methods. Emphasis is given to the information about recent research and developments on strengthening corroded and un-corroded beams using CFRP or UHPC or a combination of both.

2.1 Use of CFRP as Strengthening Material

Kaminski and Trapko [5] conducted an experimental study to test the effect of different intensity reinforcement of CFRP laminates around external column sides, and to test the effect of reinforcement type and shape on the capacity and strain of the column. Kaminski and Trapko conducted two series; one with only longitudinal CFRP laminates with variable strengthening intensities named (a), second was exactly similar to (a) but with additional transverse CFRP reinforcement named (b) as shown in Figure 1 and Figure 2.

It was observed that reinforcing with longitudinal CFRP laminates (a) increased the load-carrying capacity and strain by 24% for 5a and 6a while reinforcing with both longitudinal and transverse CFRP increased the load-carrying capacity and strain by 43% on average for 5b and 6b.

Linghoff et al. [6, 7] analyzed the behavior of steel beams reinforced with CFRP laminates. Linghoff and Al-Emrani used five different configurations besides the reference un-

reinforced beam by varying the area, places of the CFRP laminates, and the type of epoxy, as shown in Figure 3.

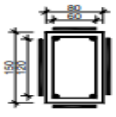
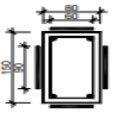
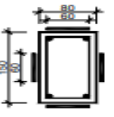
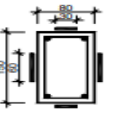
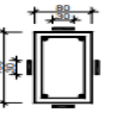
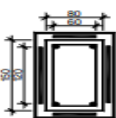
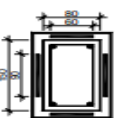
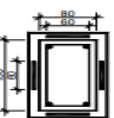
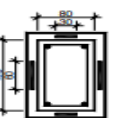
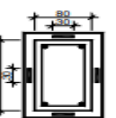
	S6a	S5a	S4a	S3a	S2a
					
Amount of elements	1	1	1	1	1
Strip cross-sectional area [mm]	120x1,4 - 2 pcs 60x1,4 - 2 pcs	90x1,4 - 2 pcs 60x1,4 - 2 pcs	60x1,4 - 4 pcs	60x1,4 - 2 pcs 30x1,4 - 2 pcs	30x1,4 - 4 pcs
Area of strips [cm ²]	5,04	4,20	3,36	2,52	1,68
Reinforcement intensity [%]	4,20	3,50	2,80	2,10	1,40
	S6b	S5b	S4b	S3b	S2b
					
Amount of elements	1	1	1	1	1
Section of band wraps [mm]	25x0,13	25x0,13	25x0,13	25x0,13	25x0,13

Figure 1: Two different strengthening configurations [5].

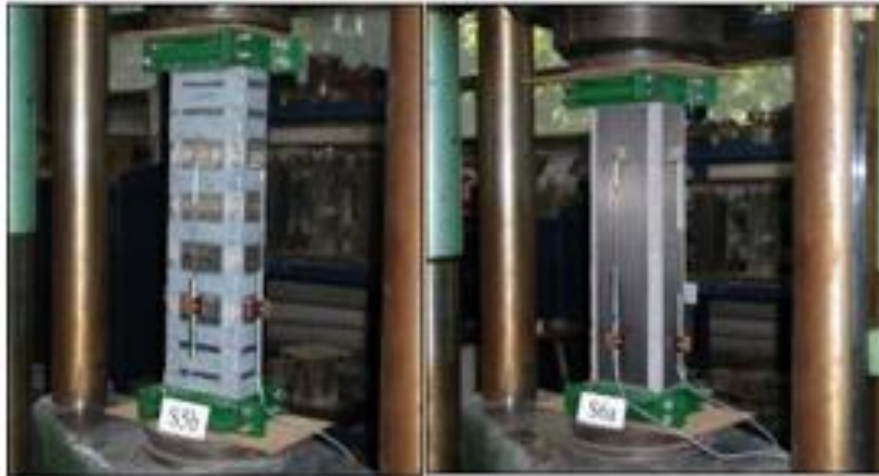
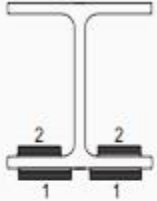


Figure 2: The figure on the left is for configuration b. The figure on the right is for configuration

In general, they concluded that the flexural capacity increases with a higher reinforcement area when using high CFRP reinforcement, but the steel beam's ductility decreases with a higher reinforcement area.

Obaidat et al. [8] examined the effect of strengthening RC beams with CFRP laminates. To accomplish their objectives, they considered three variables, the internal reinforcement ratio, retrofitting position, and the length of CFRP laminates. The ultimate tensile strength of CFRP laminates used (1.2 mm thick, 50 mm wide) is 2640 MPa. They concluded that the usage of CFRP laminates increased the flexural strength between 7% to 33% depending on variables applied to the beam, reducing the crack width and increase beam stiffness. The length of CFRP laminates should be sufficient to cover the whole moment-controlled area to exploit the full strength of CFRP, and finally, the governing failure mode was debonding.

Test specimen	CFRP laminate	Epoxy	$A \text{ (mm}^2\text{)}$	Position
Beam Ref	-	-	-	-
Beam 1-2	CFRP 3	Epoxy 2	290.9	1
Beam 1-3	CFRP 1	Epoxy 1	226.8	1
Beam 1-4	CFRP 1	Epoxy 1	340.2	1 and 2
Beam 1-5	CFRP 2	Epoxy 1	180	1



Materials	Dimensions (mm) (thickness × width)	$f_t \text{ (MPa)}$	$E \text{ (GPa)}$
Steel	HEA180	330	212
CFRP 1	1.4 × 81	3300	200
CFRP 2	1.8 × 50	1500	330
CFRP 3	2.4 × 60.6	3100	165
Epoxy 1	Thickness: ~ 2	25	7
Epoxy 2	Thickness: ~ 2	30	45

Figure 3: Five CFRP strengthening configurations [6].

Al-Saidy and Al-Jabri [9] studied the differences between replacing the cover of corrosion damaged beams strengthened with CFRP laminates and keeping the old cover. The 40 Mpa compressive strength cover increased the load resistance by 13% compared to beams strengthened on the old cover. Al-Saidy and Al-Jabri examined the effect of the U-shaped

transverse reinforcement. They concluded that using the U-shaped transverse CFRP reinforcement has increased the beam's ductility, as shown in Figure 4.

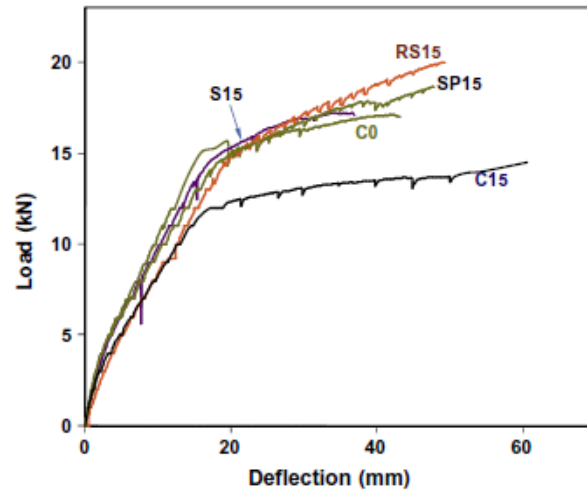


Figure 4: C0 represents the control beam, C15 represents corroded beam by 15%, S15 represents strengthened corroded beam, RS15 represents replaced cover by a new one and strengthened corroded beam, SP15 strengthened + U-shaped corroded beam.

Triantafyllou et al. [10, 11] conducted a pull-of test to a cover of 6% mass loss corroded beam and found the strength of 1.1 MPa on average while they got 3.6 MPa for the similar non-corroded beam. They tested the difference between strengthening with laminate sheets of CFRP and Near Surface Mounted reinforcement (NSM). They found the following data in Table 1.

Table 1: Summary of corroded strengthened beam results [11].

Beams	<i>At concrete cracking</i>		<i>At steel yielding</i>		<i>At maximum load</i>		<i>At ultimate load</i>		% mass loss of steel bars (average)
	P_{cr} (kN)	δ_{cr} (mm)	P_y (kN)	δ_y (mm)	P_{max} (kN)	$\delta_{P_{max}}$ (mm)	P_u (kN)	δ_u (mm)	
Uncorroded RC-N	28.8	1.2	144.4	9.6	156.9	21.6	129.9	76.3	-
Corroded RC-COR1	30.1	1.3	136.8	8.3	150.9	28.6	134.2	78.1	7.56
CFRP-Laminate RC-COR1S1	66.7	2.9	156.6	8.5	191.7	17	191.7	17	7.56
CFRP-NSM RC-COR1S2	56.9	2.4	153.3	8.6	226.6	24.1	226.6	24.1	7.56

Xie and Hu [12] corroded 32 RC beam samples with different mass loss percentages, ranging between 3% and 78%. They strengthened those beams using three different methods. First, keeping the original corroded cover and cure it with putty filler and other curing material, then placing CFRP on the cover. Second, replacing the corroded cover with polymer concrete mortar after de-rusting, then place the CFRP on it. Third, using a special cutting tool, they made a V-shaped cut around the tensile steel bar, followed by rust treatment, filling the gap with polymer concrete mortar, and finally placing the CFRP sheets. All methods have a U-shaped CFRP sheet on flexural CFRP sheets (two for each end). Referring to their experimental results, the first method can be used for a mass loss of 15% or less. Several failure modes were found, such as crushing of concrete and steel yield before CFRP fails and de-bonding. They concluded that the higher the number of CFRP laminates, the higher the de-bonding possibility. Figure 5 shows load vs. midspan deflection for beams repaired with two different methods.

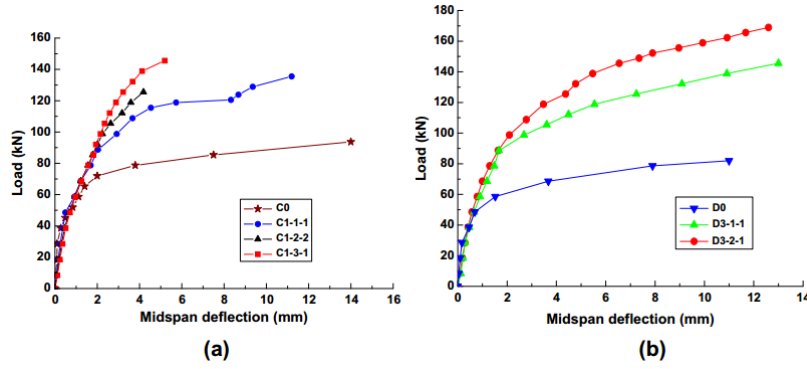


Figure 5: a-First method, 10% avg. mass loss. b- Third method 20% avg. mass loss.

El Maddawy and soudki [13] tested the viability of bonding CFRP laminates corroded beams' external cover. They corroded 3.2 m beams using accelerated corrosion by an electric DC into three different corrosion levels, low, moderate, and high of 31% mass loss of tensile steel bars. A CFRP laminate was used on the flexural side of the beam, and U-shaped laminates were wrapped transverse to the flexural laminates before starting the corrosion to prevent de-bonding of the cover while testing the beam and to provide enough pressure that prevents crack propagation. The previous method led to lower corrosion rate and provided a stronger bond between the steel and concrete cover especially for a high percentage of mass loss. Strengthening using CFRP increased the strength of the corroded beams. For example, the flexural strength of 31% mass loss beams was 71% higher than similar un-strengthened beams and 31% higher than the un-corroded and un-strengthened beam. CFRP repaired beams showed lower values of deflection than corroded un-strengthened beams, as shown in Figure 6.

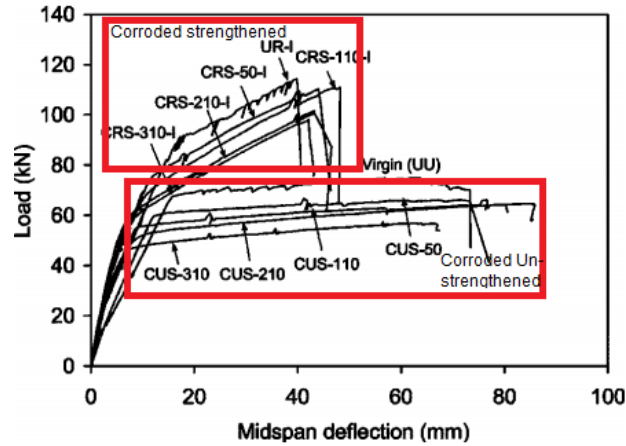


Figure 6: Strengthened beams have lower deflection values and higher load capacities.

2.2 Usage of UHPC as a Strengthening Material

Prem et al. [14] examined the effect of 20mm UHPC strengthening of 80% and 90% preloaded beams. They prepared seven beams and applied 80% of the ultimate load for four beams and 90% for the other four and leaving one to be tested to failure as a control. Then by keeping the UHPC repair layer 20mm for all of the beams, the only variable left was the curing type. Table 2 and Table 3 show curing types and test results, respectively.

Table 2: Percentage of pre-loading and curing type.

S. No		Tested (X1) (beam dimension 100 mm × 200 mm × 1,500 mm)	Curing of retrofitted UHPC overlay (Y1) 100 mm × 20 mm × 1,500 mm	Final composite beam (X1 + Y1)
1	B1	Failure (P1—ultimate load)	–	100 mm × 220 mm × 1,500 mm
2	B2	90 % of P1	Heat curing (HC)	
3	B3	80 % of P1	HC	
4	B4	90 % of P1	Steam curing (SC)	
5	B5	80 % of P1	SC	
6	B6	90 % of P1	Normal curing (NC)	
7	B7	80 % of P1	NC	

Table 3: Flexural test results of beam specimens.

S. No.	Beam type	Ultimate load (P_{ult}) (kN)	Increase in failure load (kN)
1	B1	76.27	–
2	B2	95.97	25.8
3	B3	99.82	30.8
4	B4	83.29	9.2
5	B5	84.34	10.5
6	B6	90	18
7	B7	91	19.3

$$\text{Increase in failure load} = \{ \text{Retrofitted beam } (P_{ult}) - \text{Control beam } (P_{ult}) \} / \text{Control beam } (P_{ult}) \times 100$$

Yin et al. [15] investigated the behavior of strengthening RC slabs using UHPC. They prepared nine rectangular elements. The elements were divided into two series, first is a rehabilitation series (Replacement or partial replacement) by maintaining the same thickness. The second series is done by overlaying the UHPC over the normal concrete. The experimental results showed that both series had an increment in their ultimate load. The rehabilitation series showed more deflection values before failure than the overlay series. In the rehabilitation configurations, as the thickness of the UHPC increase, the failure type changes from brittle shear to ductile flexure.

Martinola et al. [16] investigated the strengthening technique using a thin layer of (HPFRC). Full-scale beams were cast and strengthened with HPFRC layers, as shown in Figure 7. Results obtained from the experimental and numerical studies showed that using

an HPFRC jacket for strengthening or retrofitting has a significant role in increasing the load-bearing capacity (increasing in the ultimate load up to 2.15 times). Moreover, a good improvement in the RC beams' durability was noticed using the HPFRC jacket.

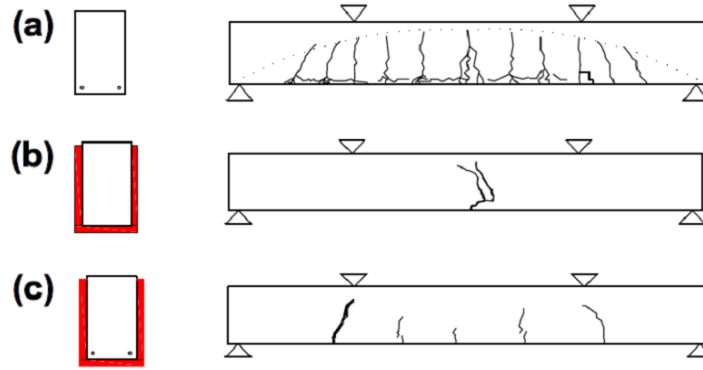


Figure 7: UHPC jacket strengthening configurations for RC beam specimens.

Habel et al. [17] conducted experimental investigations on the structural behaviour of twelve full-scale conventional concrete beams that failed mainly in flexure and having UHPC layers in the tension zone. It was noticed that the UHPC enhanced the structural capacity of the composite beams significantly. Figure 8 and Figure 9 display the test set-up of the composite beam and curves of force deflection of tested beams, respectively.

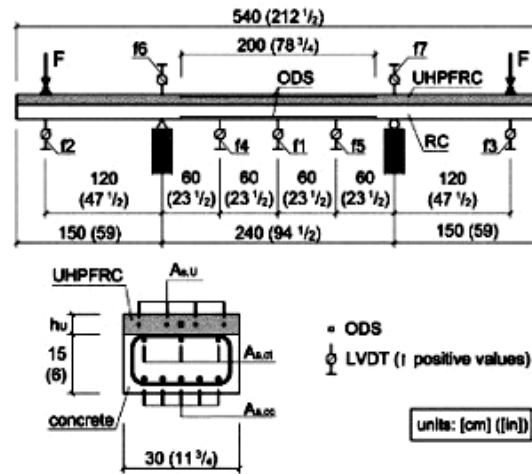


Figure 8: Composite beam set-up.

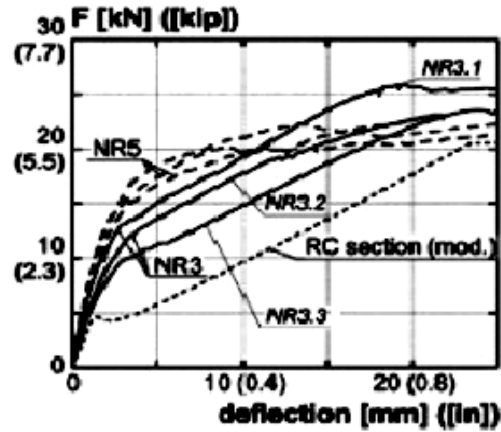


Figure 9: Force vs. midspan deflection curves for beams strengthened with UHPC.

Zhang et al. [18] conducted a very big and multi-purposes experiment on damaged bridge decks made of RC and strengthened by UHPC using different configurations to examine flexural capacity. Their research tested the decks in positive and negative moments, requiring adding the UHPC layer in tension or compression zones. They concluded that laying the UHPC in the tension zone has increased cracking and composites' ultimate capacity by about 2.5 and 2 times, respectively. On the other hand, locating the UHPC layer in the compression zone has increased the ultimate load capacity by 30%, whereas no change was noticed in the cracking pattern.

Bahraq et al. [19] studied the effect of using a UHPC jacket on RC beams' shear behavior. To strengthen the bond between RC beams and the UHPC, the beams were sandblasted before the UHPC casting. The bond between sandblasted concrete and UHPC was tested using slant shear and split tensile test. Figure 10 shows the strengthening configurations of the beams. The results of the strengthened beams showed the efficiency of retrofitting the RC beams using different configurations of UHPC layers, which improved the shear capacity of the beams and shifted the failure mode from brittle to ductile.

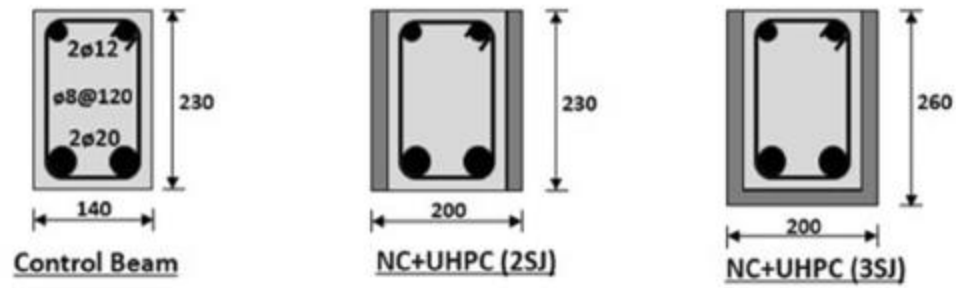


Figure 10: Beam dimensions and UHPC strengthening configurations.

Al-Osta et al. [20] studied two approaches in their research. The first approach was to strengthen RC beams with three different UHPC configurations, bottom only, two sides, and three sides (Jacketing), as shown in Figure 11. The second approach was to catch the difference between bonding the UHPC on RC beams via a very strong epoxy and casting UHPC on RC beams after sandblasting the beams, as shown in Figure 12. Their results showed that no significant difference between the two bonding methods. Casting UHPC on sandblasted beams showed a better overall performance.

Strengthening configurations (thickness of UHPFRC Jacket = 30 mm)	
Strengthening pattern	Specimen identification
	RC-Control
	RC-BOT SJ
	RC-2 SJ
	RC-3 SJ

Figure 11: UHPC strengthening Configurations by [20].

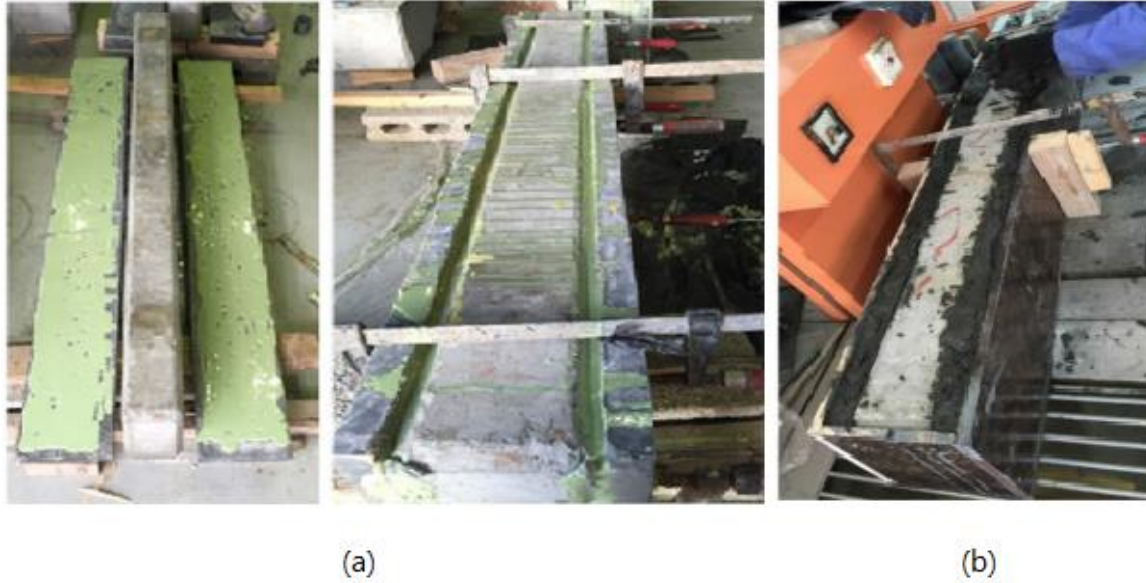


Figure 12: UHPC bonding methods. (a) Using epoxy. (b) In situ casting after sandblasting.

Husain and Amleh [21] studied the flexural and shear capacities of a composite beam of normal concrete and Ultra-High-Performance Fiber Reinforced Concrete (UHPFRC). The stirrups were excluded in order to provide a clear vision of shear capacity, a noticeable increase in the shear and flexural capacities, which proves their proposed work's efficiency.

Masse and Brühwiler [22] made an experimental study on strengthening concrete bridge slabs with UHPC overlay to determine slabs' flexural performance under high traffic loading. They developed an analytical model for the strengthened slab, as shown in Figure 13, to validate the experimental results. They concluded that the UHPC strengthening had increased the load-bearing capacity in the bridge. In addition, they mentioned that this method is easy to apply in the case of bridges.

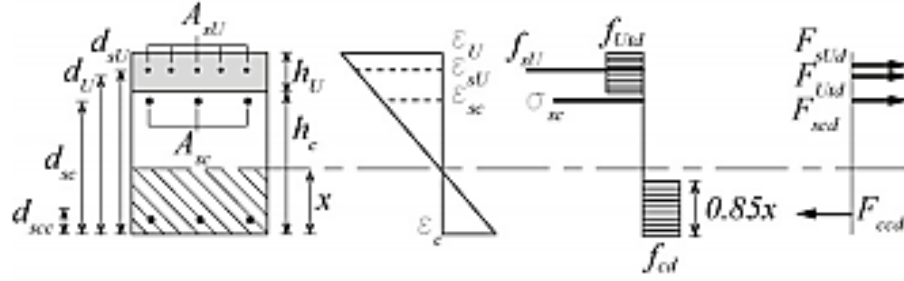


Figure 13: Analytical solution model for slab strengthened with UHPC proposed by [22].

Zhang et al. [23] investigated the flexural behavior of damaged RC beams strengthened by reinforced UHPC overlay. Their research found that the UHPC layer has improved the flexural performance of the beams and prevented the cracks from developing through the concrete beam. Figure 14 shows the strengthening configuration of the strengthened beam.

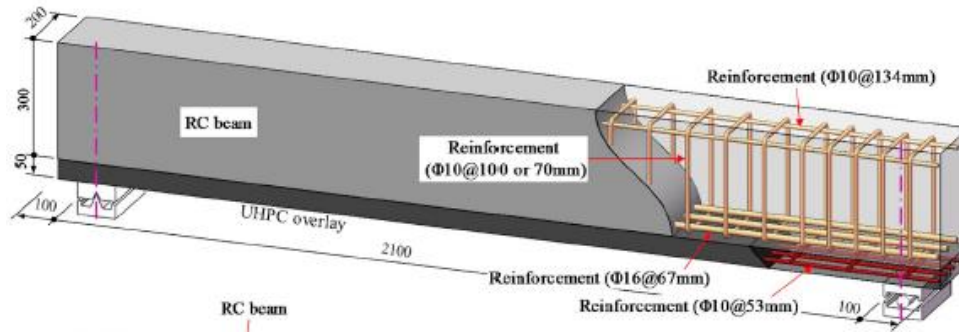


Figure 14: UHPC layer strengthened with steel bars.

Lampropoulos et al. [24] examined the effect of strengthening beams with Ultra-High-Performance Fiber Reinforced Concrete (UHPFRC). They strengthened three beams, one on the tension side, the second on the compression side, and the third was strengthened by jacketing on the tension side. They conducted extensive modeling on these beams resulting in an excellent increment in the load-carrying capacities.

2.3 Use of a hybrid system of UHPC and CFRP as a strengthening material

Moon et al. [25] examined the effect of strengthening the RC slab with a hybrid system of UHPC and CFRP. In their research, they made two slabs; one was kept as a control, CFRP and UHPC strengthen the second one in the presence of shear connectors. Their current research relied on their previous research, where they did the same thing but without shear connectors, which resulted in shear failure before developing the full strength of the hybrid system. After applying the shear connectors, the failure happened in the flexure, which yielded the following results: First, the ultimate load-carrying capacity has increased by 70%, the toughness by 128%, the stiffness by 60%. Figure 15 shows the strengthening configurations.

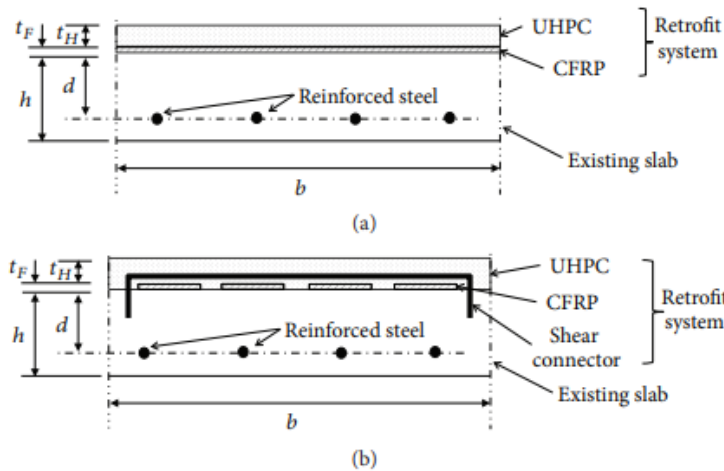


Figure 15: [25] configurations. a) without shear connector. b) with shear connector

Kim et al. [26] investigated the flexural behaviour of continuous slab strengthened with a hybrid system of UHPC and CFRP. In their study, they proposed different configurations. In all the configurations, the CFRP laminates were embedded under the UHPC layer. In

some slab samples, the UHPC was extended into the compression zone. This hybrid system has increased the load-bearing capacity by 130%.

Mosallam et al. [27] strengthened three one-way – two spans slabs (bridge decks) using a hybrid system of HPC and CFRP laminates to see the increment the flexural capacity of those slabs. The CFRP laminates were adhered to the top of the slabs using an epoxy resin. Then a 24 mm thick UHPC layer was applied on the top. The hybrid retrofit system has provided an ultimate load capacity enhancement of up to 164%, extra deflection (ductility) 122%, and an energy intake of up to 190%.

CHAPTER 3

EXPERIMENTAL PROGRAM

3.1 General

RC beam specimens were designed and cast using normal strength concrete and conventional steel. After casting and curing of the RC beam specimens they were subjected to an accelerated corrosion process for causing loss of the mass of the reinforcing bars by 10 and 20%. The corroded RC beam specimens were divided into different groups for strengthening using different strategies made by considering different combinations of cover replacement and types of strengthening configurations (CFRP alone and hybrid system of CFRP and UHPC). The experimental program presented in the following sections explains the steps followed to obtain the experimental data that were used to evaluate the effectiveness of the hybrid system strengthening technique.

3.2 Test Specimens

The experimental work included structural design, casting, curing, corroding, repairing, strengthening, and testing of 21 RC beam specimens. Out of the 21 beams, one beam was un-corroded to be as a control. The cross-section of the beam specimens was 140 mm (width) and 230 mm (depth) and the length was 1600 mm. A clear cover of 20 mm was used for all sides of the beam. There were 2 Φ 10mm longitudinal bars on the compression side and 2 Φ 12mm longitudinal bars on the tension side. Stirrups having 8mm diameter and a center/center spacing of 50 mm were used as shear reinforcement. The high number of

stirrups (by using a lower spacing of 50 mm) was used to prevent shear failure and focus the study on flexural behavior of the beams. Epoxy-coated steel bars were used for both compression and stirrups to avoid affecting them by accelerated corrosion (only the main bottom longitudinal tensile bars were allowed to corrode). Figure 16 shows the dimensions of steel reinforcement and RC beam.

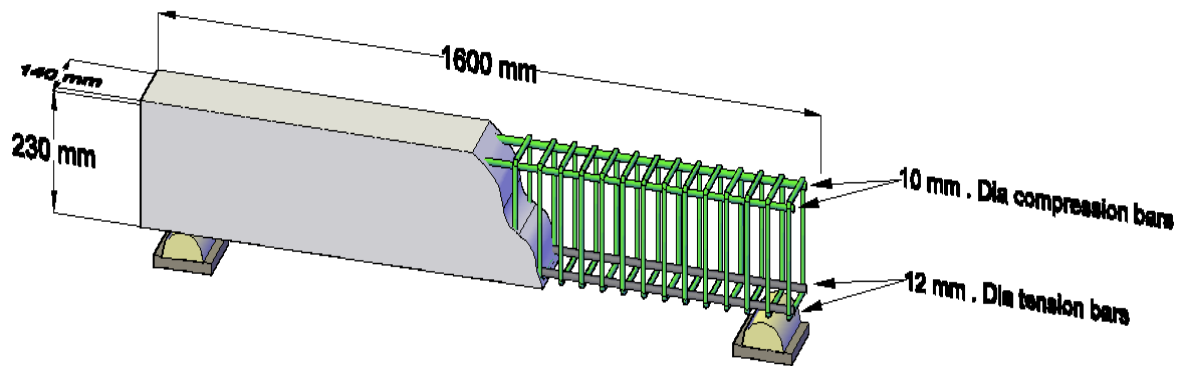


Figure 16: Beam geometry and reinforcement dimensions.

The beam specimens were grouped and labelled based on corrosion level and strengthening configuration, as detailed in Table 4. One beam was kept as a control, which was neither corroded nor strengthened and given the name of (UU). Twenty beams were corroded to cause 10% and 20% reduction in mass of reinforcing bars. A 10% reduction in the mass of bars was caused in nine beam specimens while the 20% reduction in mass of bars was caused in eleven beam specimens. These two groups of the beam specimens were labelled as CA and CB corresponding to reduction in the mass of bars by 10% and 20%, respectively. One beam from each corrosion level was tested as a control without repairing. The other eighteen beams were divided into three groups. Fourteen beams had their damaged cover replaced with normal repair material and given the letter 'R' in their naming system, two beams had their cover replaced with UHPC having the letter 'H' and two beams

were kept with their original cover. The beams strengthened with one layer of CFRP had the number ‘I’ in their naming system, while beams strengthened with two layers of CFRP had the number ‘II’. The beams that had a UHPC jacket thickness of 20 mm had the number ‘20’ in their name while the beams that had a UHPC jacket thickness of 30 mm had the number ‘30’ in their naming. Each strengthened beam has four U-shaped CFRP strengthening around the flexural CFRP. Figure 17, Figure 18, and Figure 19 show the details of a repaired beam strengthened with a hybrid system of CFRP and UHPC.

Table 4: Description of the beam specimens with different cover repairing strategies and strengthening strategies.

Beams	Percentage of bar mass loss	Cover repair material	# of CFRP Strengthening layers	Thickness of UHPC Jacket (mm)
UU	None	None	None	None
CA	10	None	None	None
CB	20	None	None	None
CAR	10	Repair Material*	None	None
CBR	20	Repair Material*	None	None
CBH	20	UHPC	None	None
CA-I	10	None	One	None
CB-I	20	None	One	None
CBH-I	20	UHPC	One	None
CAR-I	10	Repair Material*	One	None
CBR-I	20	Repair Material*	One	None
CAR-II	10	Repair Material*	Two	None
CBR-II	20	Repair Material*	Two	None
CAR-I-20	10	Repair Material*	One	20
CBR-I-20	20	Repair Material*	One	20
CAR-II-20	10	Repair Material*	Two	20
CBR-II-20	20	Repair Material*	Two	20
CAR-I-30	10	Repair Material*	One	30
CBR-I-30	20	Repair Material*	One	30
CAR-II-30	10	Repair Material*	Two	30
CBR-II-30	20	Repair Material*	Two	30

*The repair material is ‘*MasterFlow 980*’, which is a cementitious material provided by BASF company specialized for concrete repair. More details about the material are provided in the ‘Material Properties’ section.

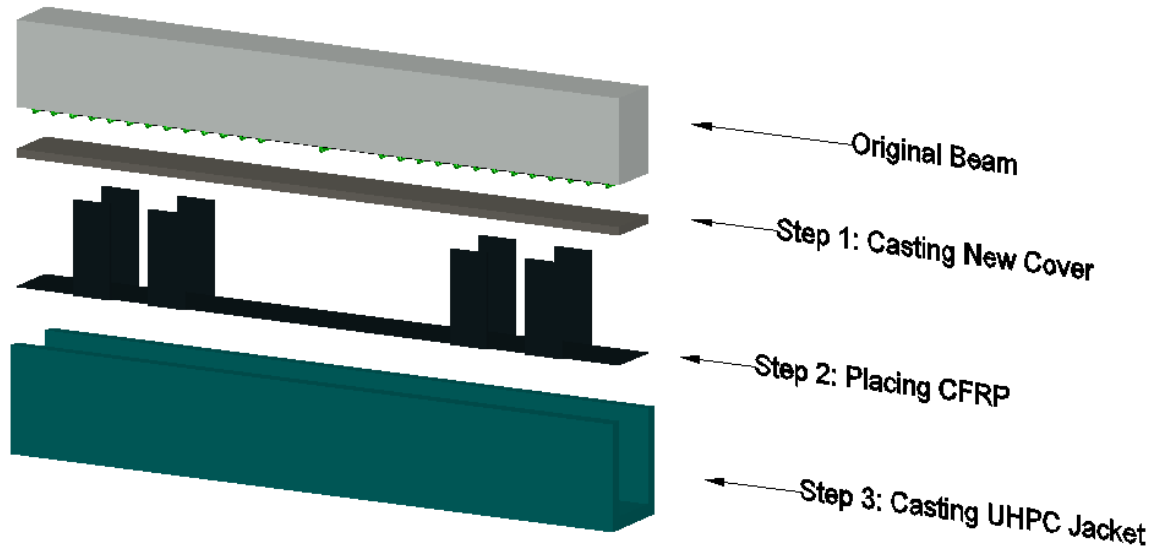


Figure 17: Original beam + repair material cover + CFRP + UHPC jacket

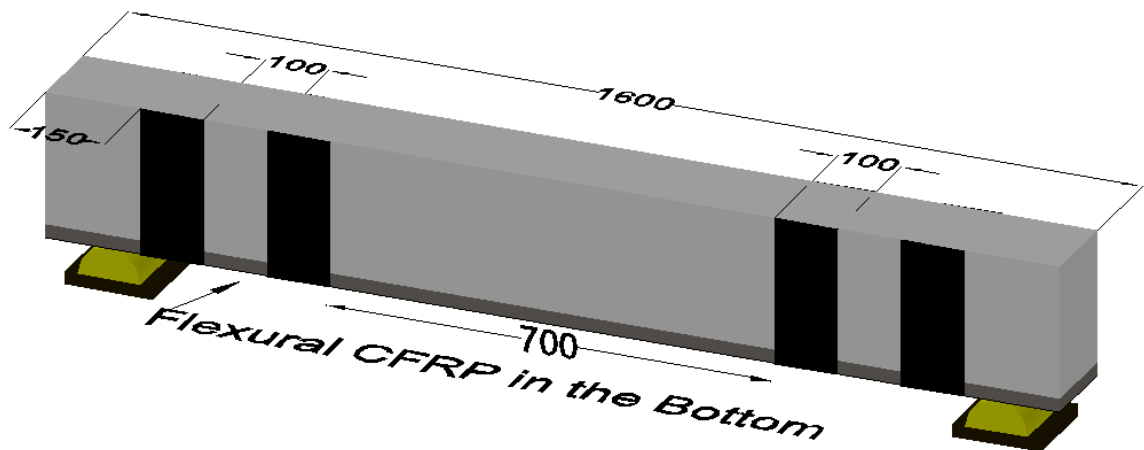


Figure 18: Corroded beam with the repaired cover and CFRP strengthening

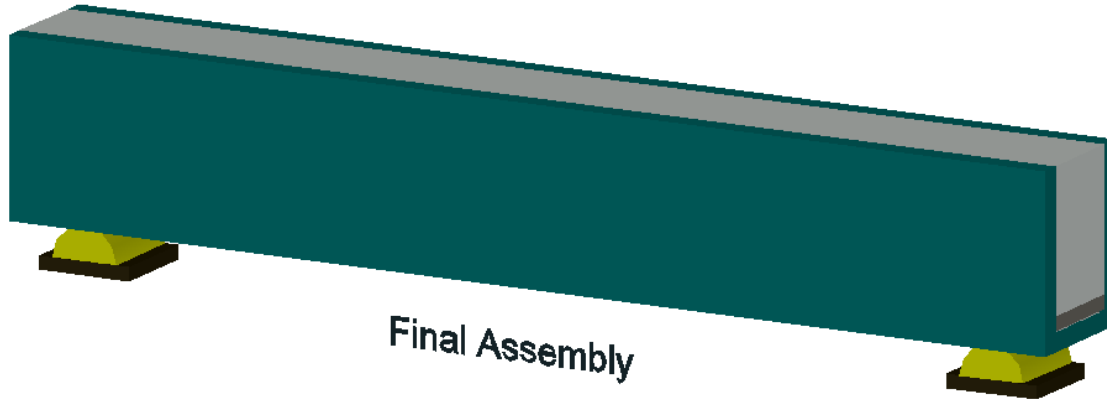


Figure 19: Corroded beam with repaired cover and a hybrid strengthening (CFRP + UHPC)

3.3 Materials and Properties

3.3.1 Normal Concrete (NC)

All beam specimens were cast at PRAINSA company using normal concrete (NC), as shown in Figure 20. After casting, the beams were exposed to steam curing for 6 hours continuously. Then they were kept in the air for 28 days to gain their full strength. Concrete cylinders with standard dimensions (75 mm diameter and 150 mm length) were cast along with the casting of beams to determine the compressive strength and modulus of elasticity of NC. The compressive strength and elastic modulus of concrete were determined by testing 14 cylinders in accordance with ASTM C39 [28]. The average values of the compressive strength and modulus of elasticity of NC were found as 50 MPa and 33 GPa, respectively, having a standard deviation of 3.6 MPa and 1.05 GPa, respectively. Figure 22 shows a typical compressive stress-strain curve for NC. A splitting tensile test was performed on concrete cylinders in accordance with ASTM C496 [29] to determine the average tensile strength by testing 11 cylinders. The splitting tensile test showed a value of

3.2 MPa as an average tensile strength of NC, having a standard deviation of 0.3 MPa.

Figure 21 shows the compression and splitting tensile tests on NC.



Figure 20: Casting of the beam Specimens.



(a)



(b)

Figure 21: (a) NC compression + elastic modulus test. (b) NC split tensile test

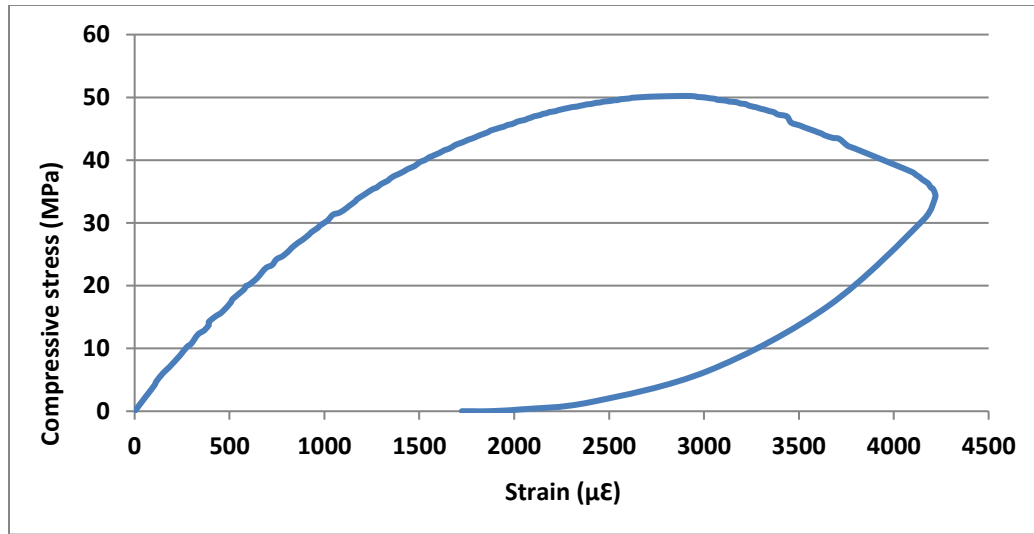


Figure 22: Compressive stress-strain curve of NC.

3.3.2 Steel Reinforcement

There were three different sizes of steel reinforcement bars that were used in reinforcing the beam specimens. Two steel samples of each bar size were tested in tensile in accordance with ASTM A370 standard [30]. The average tensile strengths for the 12 mm, 10 mm, and 8 mm bars were 610, 580, and 575 MPa, respectively, having an average standard deviation of 9 MPa. Figure 23 shows the tensile test experiment of the steel bar.



Figure 23: Steel tensile test

3.3.3 UHPC

For UHPC preparation, the materials used were Type I cement, sweet water, fine dune sand, superplasticizer (Gelenium), micro-silica, and steel fibers. The mix proportion of UHPC, developed by Ahmad et al. [31] and shown in Table 5, was adopted to prepare UHPC in the present study. Two types of steel fibers were used in the UHPC mix, as shown in Figure 24, with a percentage of 50% each. The first type of steel fibers was straight steel fibers with a diameter and length of 0.1 mm and 12.5mm, respectively. The second type was hooked steel fibers with a diameter and length of 0.2 mm and 25 mm, respectively. The two types of steel fibers were copper coated, having a tensile strength of 2500 MPa.

Table 5: UHPC mix proportions [31].

Quantity of Ingredients in Kilograms for a batch size of 1 m ³						
Cement (kg)	MS (kg)	Water (kg)	SP (kg)	Sand (kg)	Fiber (kg)	TOTAL (kg)
900	220	162.4	40.32	1005.34	157	2484.85

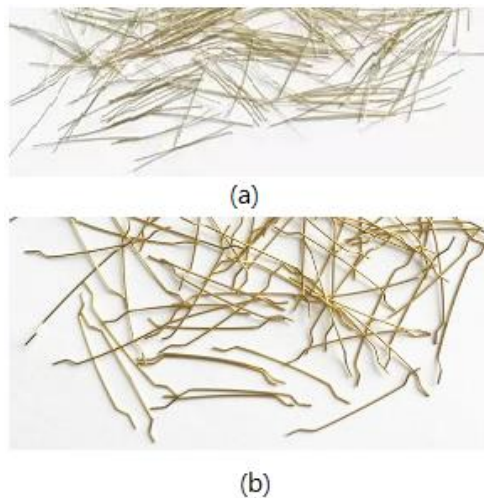


Figure 24: Steel fibers used in UHPC (a) Straight. (b) Hooked.

Compression tests were performed to determine the compressive strength and modulus of elasticity of the UHPC in accordance with ASTM C39 [28] for cylinders (75 mm diameter

and 150 mm length) and BS EN 12390-3 [32] for cubes (50 mm side and 150 mm side). The test results showed 165 MPa and 46 GPa for compressive strength and elastic modulus of the UHPC, respectively, by testing thirteen cylinders, seven 150 mm side cubes, and twenty 50 mm side cubes having standard deviations of 5 MPa, 5.5 MPa, and 7 MPa, respectively. ASTM C496 standard [29] was used to perform split tensile strength tests of UHPC. The average tensile strength obtained from the splitting tensile strength test (five samples) was 18.4 MPa with a standard deviation of 1.1 MPa. The average tensile strength obtained from the direct tension test of 4 dog bones was 7.4 MPa, with a standard deviation of 0.5 MPa. Figure 25 shows the tests performed on UHPC. Figure 26 shows the stress-strain curve for UHPC from the compression test. Figure 27 shows the stress-strain curve obtained from the direct tensile test of the dog bones.

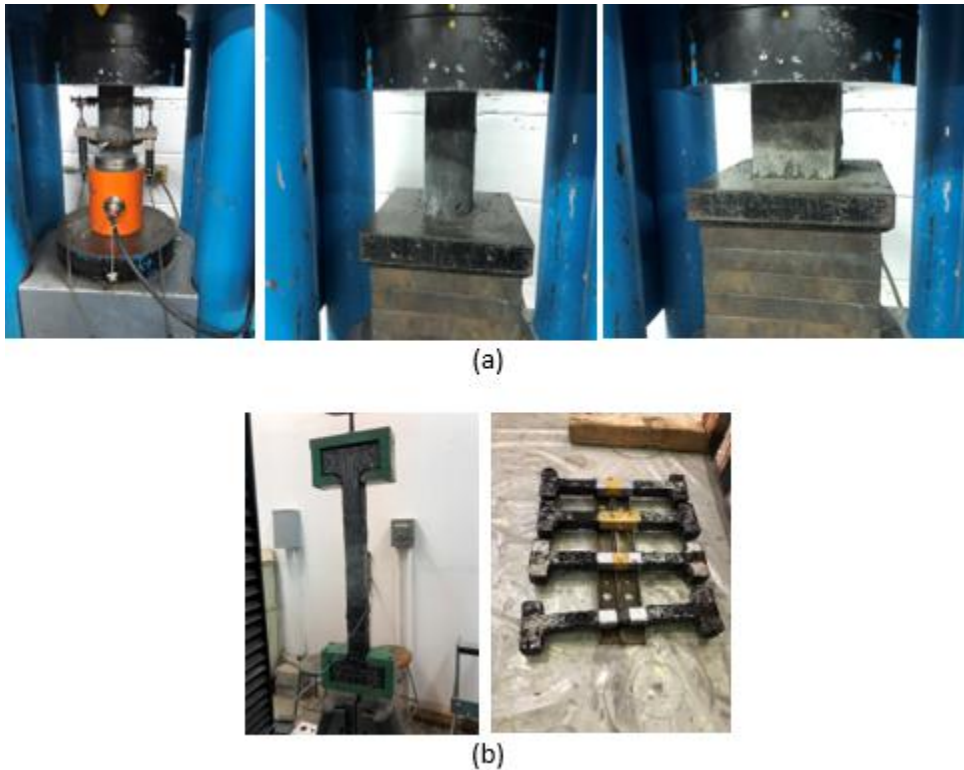


Figure 25: Material testing for UHPC: (a) Elastic modulus. (b) Dogbone direct tensile test

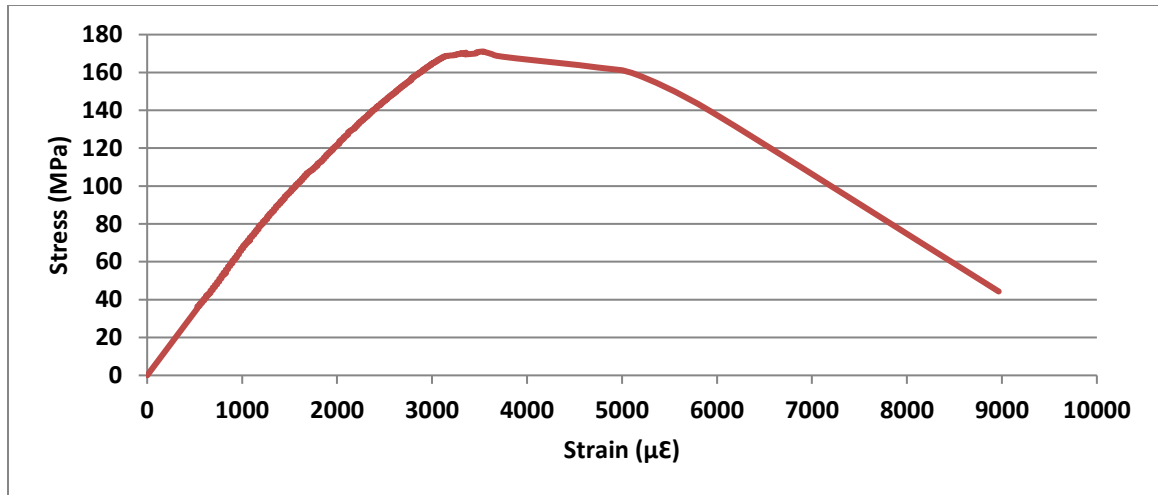


Figure 26: Stress-strain curve for UHPC in compression.

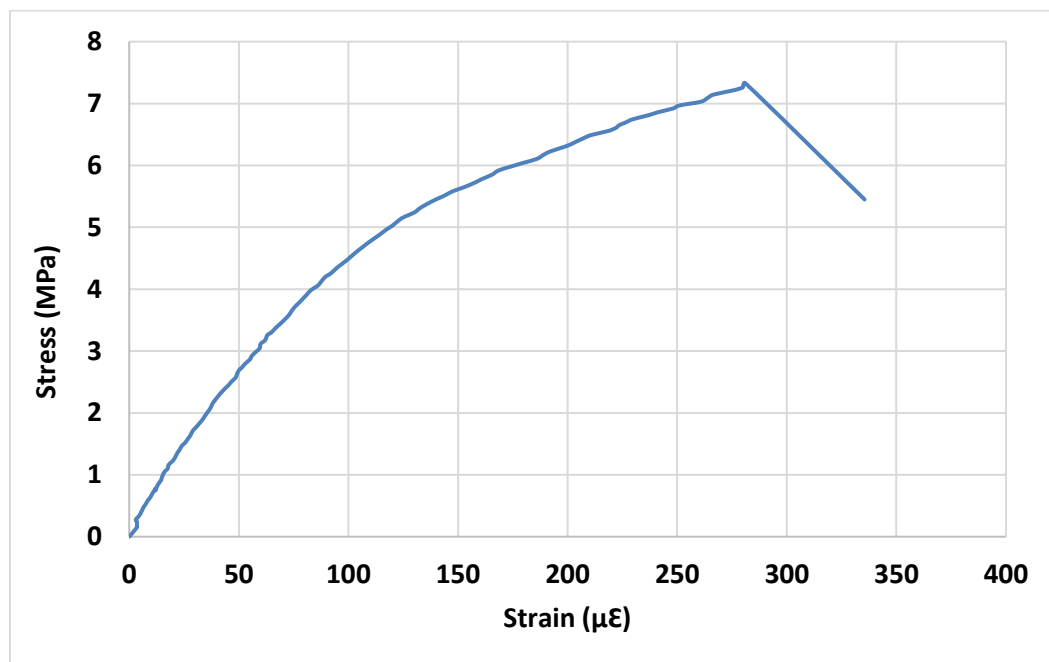


Figure 27: Stress-strain curve for UHPC from a direct tensile test.

3.3.4 Repair Material – Bonding Agent

To prepare the corroded RC beams for CFRP bonding, the damaged cover was removed from all corroded beams and replaced by a special repair material (*MasterFlow 980*).

Compression tests for the repair material showed the values of compressive strength and modulus of elasticity as 80 MPa and 27000 MPa, respectively, by testing eight standard cylinders (75 mm diameter and 150mm length) having standard deviations of 6.5MPa and 1100 MPa, respectively. A special bonding agent from the same company named (*MasterCast 121*) was used on the old concrete surface to enhance the bond between the old surface and the new repair material cover. Figure 28 shows the modulus of elasticity test on the repair material.



Figure 28: Modulus of elasticity test for the repair material *MasterFlow 980*.

3.3.5 CFRP – Epoxy resin

SikaWrap®-530 C is the type of CFRP laminates that were used to strengthen the corroded beam specimens. The CFRP's mechanical properties showed a tensile strength of 3500 MPa, an elastic modulus in the tension of 225 GPa. The nominal thickness of CFRP is 0.29 mm, and the dry fiber elongation at break is about 1.7%. For this type of CFRP laminates, the specifications recommended using the wet application process in which the CFRP laminates need to be saturated with a semi-liquid epoxy resin when applied. *Sikadure 300c* epoxy resin was used to bond the CFRP laminates to the beam specimens. Mechanical tests

for this type of epoxy resin showed an elastic modulus in flexure of 2800 MPa, an elastic modulus in the tension of 3500 MPa, and an elongation at break of 1.5% as provided by the manufacturer data sheet [33, 34].

3.3.6 Bond between UHPC and Normal Concrete

The bond strength between UHPC and CFRP is tested in two ways. First, split tensile test to determine the tensile bond. Second, slant shear test to determine the tensile and shear bond. The concrete cylinders were cut and sandblasted. Then the UHPC was cast in situ and left to cure for 28 days by the water. Five samples for each type of test were tested. The split tensile test results showed that the average tensile bond strength is 4.5 MPa with a standard deviation of 1.4 MPa. The slant shear test results showed that the average shear bond strength was 18 MPa while the average tensile bond strength was 5 MPa with a standard deviation of 1.9 and 1.35 MPa, respectively. Figure 29 and Figure 30 show the split tensile and slant shear tests, respectively, between UHPC and NC.



Figure 29: Split tensile test between UHPC and normal concrete



Figure 30: Slant shear test between UHPC and CFRP

3.3.7 Bond between UHPC and CFRP

A bond test was performed between CFRP and UHPC using a bi-surface test. Six samples were prepared in this test by casting two-third of 150 mm x 150 mm cube by repair material, then after water curing for 28 days, CFRP laminates were bonded to the surface and cured for seven days. Finally, the remaining one-third of the cube was cast in situ by UHPC. Figure 31 shows the test setup. The cube was placed such that the CFRP fiber direction is upward. The bond strength was calculated to be 0.47 MPa by the formula developed by Momayez et al. [35].



Figure 31: Bond test setup

3.4 Accelerated Corrosion Process

An accelerated corrosion environment system was designed and prepared to corrode the tensile steel bars inside the RC beams. First, the beams were flipped in which the tensile steel bars became at the top. Then wet towels were placed on the top surface, followed by stainless-steel plates. The tensile steel bars worked as an anode, whereas the stainless-steel plates worked as a cathode. A water supply system was installed to keep the towels in a wet condition all the time—the sprayed water contained 5% of NaCl. The water supply system was connected with a water pump that provides water every three hours for three seconds. The water reached the towel through openings in the steel plate. Plastic cups were opened from both sides and placed over the stainless-steel plate openings to ensure that the towel soaks with the whole quantity of the sprayed water. Figure 32 shows a brief description of the system. Tensile bars and stainless-steel plates are connected with a constant DC power supply. The current provided by the power supply was 0.233 mA, which corresponds to a current intensity of 0.2 mA/cm^2 . As stated by Elmaaddawy and Soudki, the current intensity should not exceed 0.2 mA/cm^2 in order to prevent damage resulting from high current intensity [36]. Following Faraday's law (equation 3.3.1), the accelerated corrosion periods were calculated to be 48 and 96 days for 10 and 20%,

respectively. Figure 33 and Figure 34 shows a description of the corrosion system components.

$$m = \frac{atI}{nF} \quad (3.4)$$

Where;

m : Mass loss.

a : Atomic mass of iron (55.85 g).

t : Time of the corrosion process.

I : Corrosion current.

n : Valence of the reacting electrode for the material (for steel $n=2$).

F : Faradays constant (96500 C/mol).

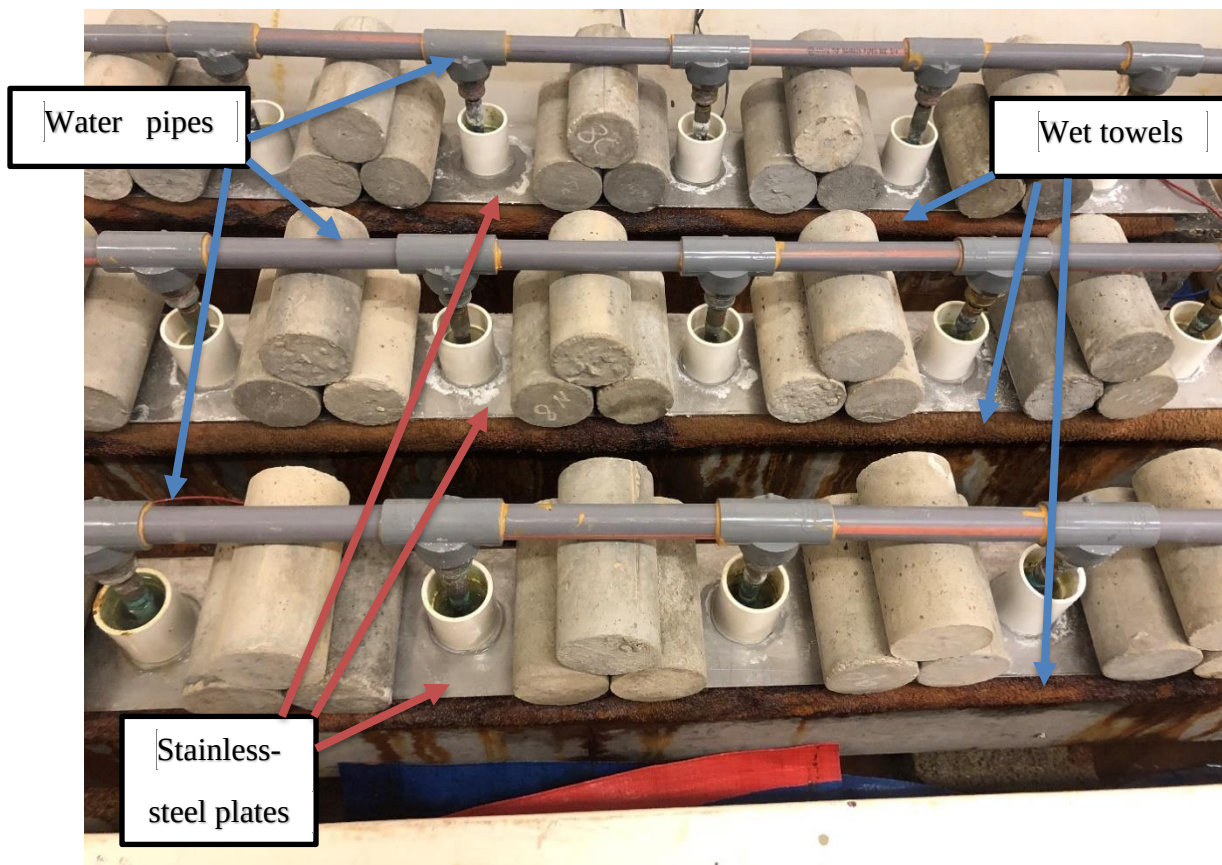


Figure 32: Corrosion system components

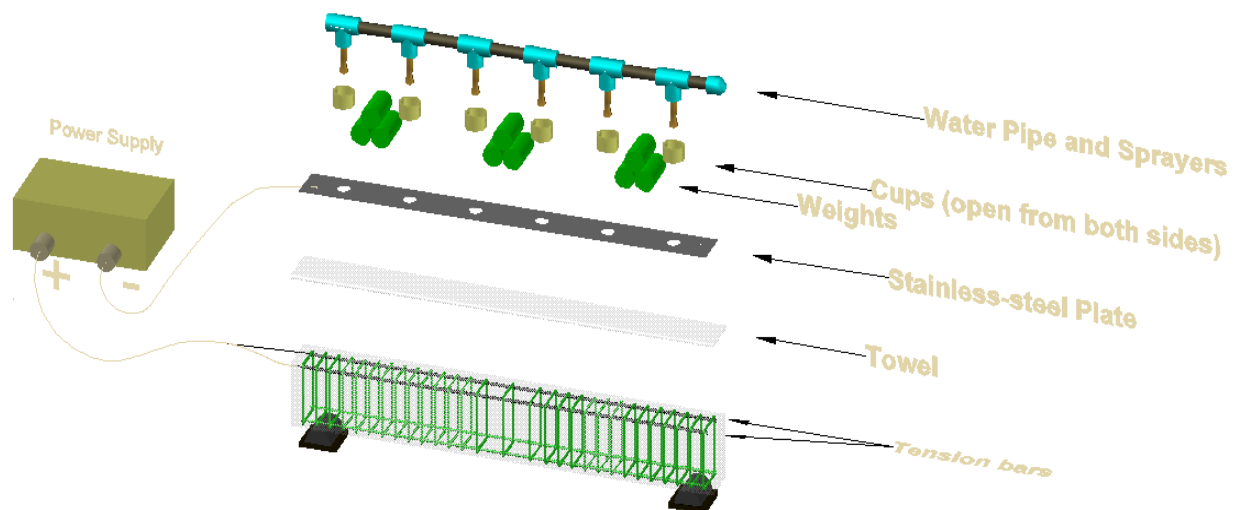


Figure 33: Detailed description of corrosion setup components.

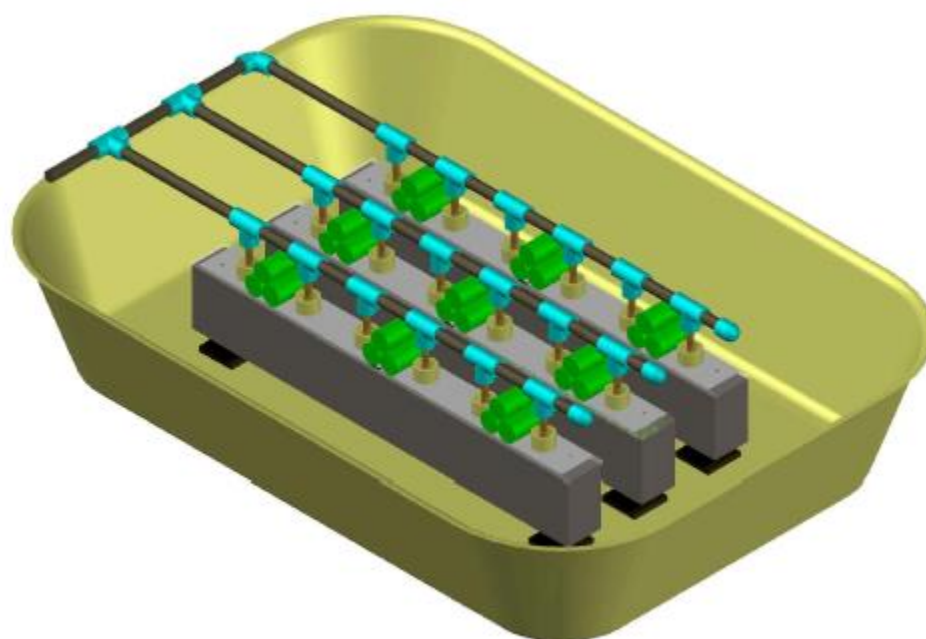


Figure 34: Water spray system in corrosion tank.

3.5 Cover Replacement Technique

Due to corrosion and since the corrosion products are larger in volume than the original steel bars, the beam specimens' concrete covers were cracked from the bottom and side surfaces. Therefore, the strengthened corroded beams were subjected to cover removal using a gentle hammer to avoid affecting the main steel bars. After that, the concrete surface and the exposed bars were cleaned using high-pressure air and a steel brush. There are two cover replacement materials in this experiment. First is the repair material (*Master Flow 980*), which was cast on the old concrete surfaces after applying a bonding agent material called (*MasterCast 121*) as prescribed by the manufacturer. The detailed methodology of applying the bonding agent and the repair material is explained in detail in the manufacturer data sheet [37, 38]. The other cover replacing the material is UHPC. This material was applied directly over the old concrete without the need for bonding agent material. Then all beams were cured by water for 28 days. Figure 35 shows the steps of cover replacement.



Figure 35: (a) Cover removal. (b) Bonding agent application. (c) Repair material placement. (d) Water curing.

3.6 CFRP Strengthening Technique

The surfaces of the beams were prepared and cleaned for CFRP placement. A high-pressure air machine was used to clean the surfaces from any dust or fine particles, affecting the CFRP laminates' bonding. The 90-degree edges at which the U-shaped CFRP laminates were wrapped were grinded in rounded shape via a grinder as advised by the CFRP manufacturer. CFRP U-shaped laminates were wrapped at specific beam locations to minimize the probability of de-bonding the CFRP laminates from concrete surfaces, as shown in Figure 36. The epoxy parts were mixed following the procedure and the

manufacturer's standard [39]. One-third of the epoxy was brushed over the beam surface, and the other two-third were brushed over the CFRP laminates, then the CFRP laminates were carefully placed over the beam. A special paint roller was rolled over the CFRP to ensure that the fibers were fully saturated with the epoxy. The beams were cured for seven days in the air at an average temperature of 36 °C. Figure 37 shows the work of the CFRP application.

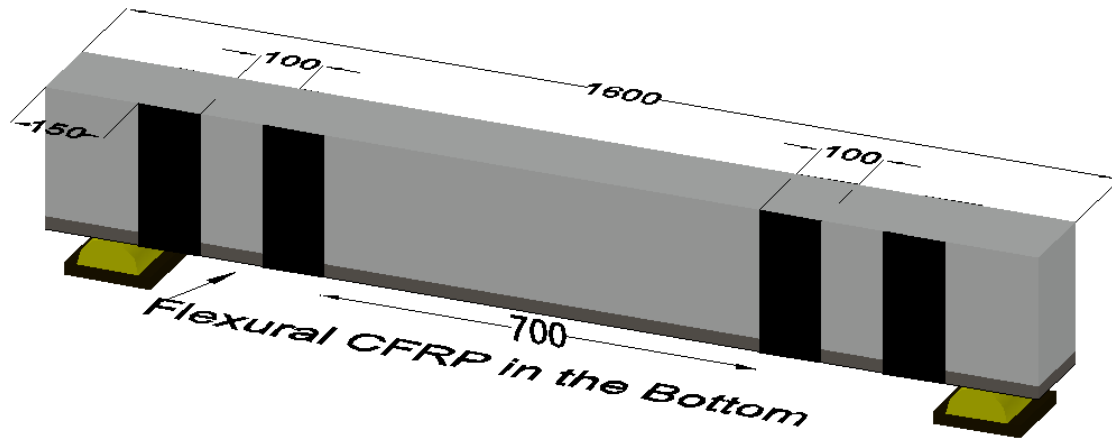


Figure 36: U-shaped CFRP dimensioning



Figure 37: CFRP work

3.7 UHPC Jacket Application Technique

To provide a good bond between the concrete of the beam and the UHPC, the concrete sides of each beam were sandblasted, as shown in Figure 38. Due to the heavyweight of UHPC, the wood molds were designed to be strong enough to carry the lateral pressure applied by the UHPC mix. Figure 39 shows the casting work of UHPC.



Figure 38: Sandblasted beams (after corrosion)



Figure 39: UHPC casting

3.8 Instruments and Loading

All beams were tested until failure using four-point monotonic loading. A hydraulic jack, which has a capacity of 300 kN was used to apply the loads, and a load cell was used to apply and measure the load, respectively. Four linear variable displacement transducers

(LVDTs) were used to measure deflections at different beam locations. Two LVDTs are placed on the top of the tested beam at support to measure the displacement of the end beams, while the other two LVDTs were placed under the center of the beam to measure the mid-span deflection. Four strain gauges were placed on the beam to measure the strains. One concrete strain gauge at the top center of the beam, whereas three steel strain gauges were distributed along the CFRP laminate centerline on the beam's bottom side. A load cell was placed on the top of the hydraulic jack to record the load increments during the experiment. All instruments were connected to a data logger that records the values for time. Figure 40 shows the details of the test setup.

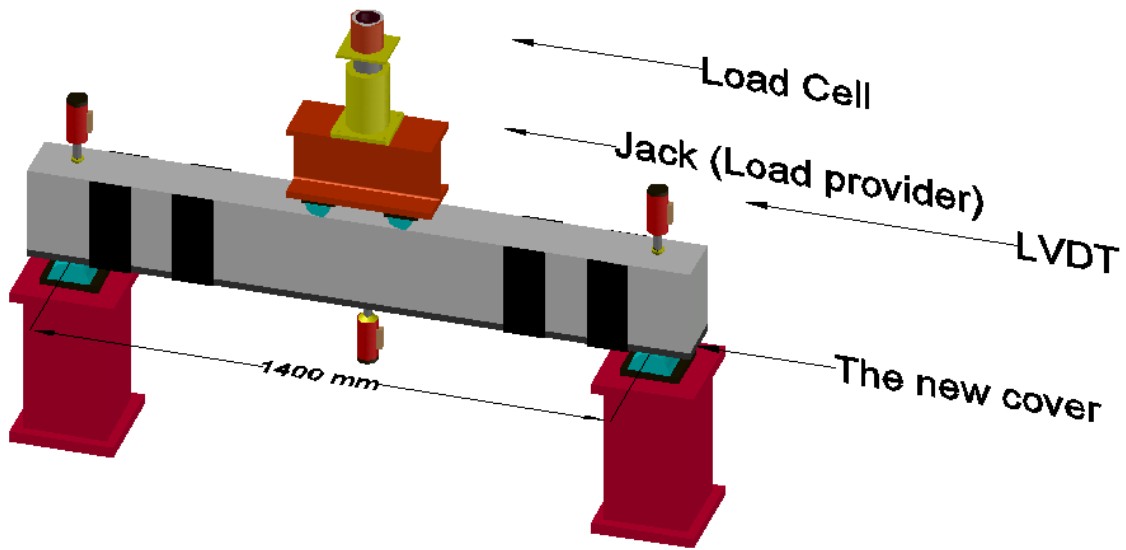


Figure 40: Details of the test setup

CHAPTER 4

Results and Discussion

4.1 General

The experimental work conducted under the present study yielded the pertaining to the degrees of damaged caused to the reinforcing bars in the beams using an accelerated corrosion set-up, flexural test, and the loss of weight of bars due to corrosion obtained using the extracted bars after flexural test on the beam specimens. In this chapter, the experimental results were presented, analyzed, and discussed to arrive at useful conclusions.

4.2 Corrosion Induced Concrete Cracking

Four days after the initiation of inducing reinforcement corrosion in the beam specimens, corrosion products started leaking out of the beams, and by the end of seven days, longitudinal and side cracks in the beam became visible. Cracks formed due to the accumulation of corrosion products. Since the corrosion products are 4-6 times larger in volume than the un-corroded steel, a tensile stress got developed in concrete surrounding the steel bars causing the cracking in concrete. In all the specimens, three to four cracks formed along each beam's length surrounding the tension bars. Fourteen beams had the same cracking behavior, one on each side of the beam and two on the tensile surface, while the other six beams showed one crack on each side of the beam and only one crack on the tensile surface. The six beams with the single crack on the tensile surface had a crack

approximately 50% wider than beams with two cracks because the corrosion products produced by the tensile corroding bars leaked out through the same crack outlet, as shown in Figure 41. No cracks appeared around the compression zone, which indicates that the epoxy-coated compression bars did not corrode.

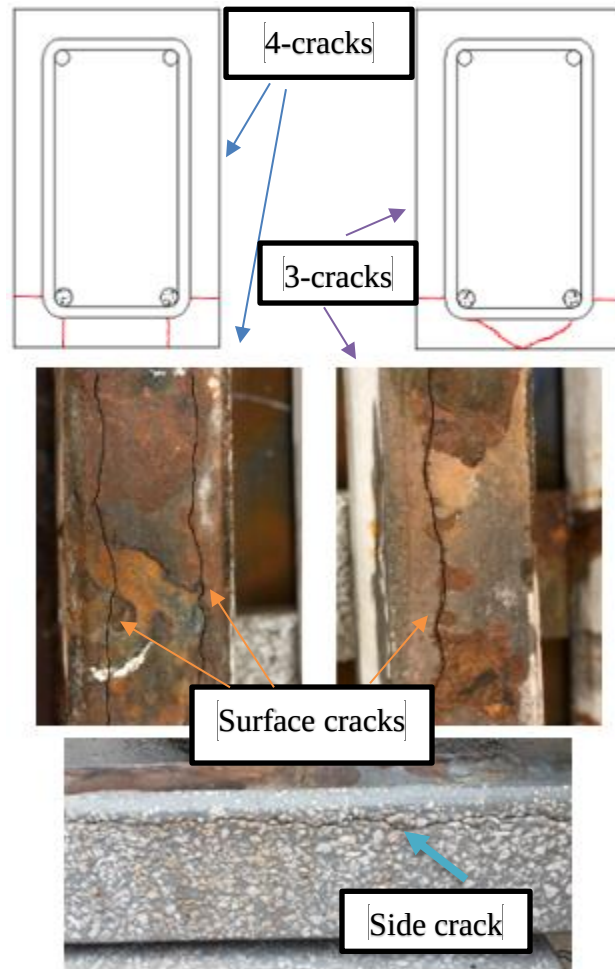


Figure 41: Corrosion cracks

4.3 Gravimetric Analysis/Steel Mass Loss

In order to ensure that the bars were corroded as planned (i.e., to secure mass losses of 10%, and 20%), the corroded bars were extracted from the beam specimens after testing them under flexure. The extracted bars were tested to determine the actual losses of mass

(gravimetric analysis) in accordance with *ASTM G1* [40]. The corroded bars were weighed and then submersed in acidic solution (HCl) in addition to small percentages of other chemical compounds according. After exposure of the bars to acid solution they were washed with tap water while brushing them with a harsh steel brush, then the bars were dried and their residual mass were recorded. Finally, the bars were inserted in the acidic solution again. This cycle was repeated until the bars' weight became constant. Equation 4.2.1 was used to calculate the percentage mass loss.

$$\% \text{ mass loss} = \frac{\text{Original weight of the bar} - \text{Weight obtained from gravimetric analysis}}{\text{Original weight of the bar}} \times 100 \quad (4.3)$$

Figure 42 shows different steps involved in carrying out the gravimetric analysis of the corroded bars. One of the steel bars extracted from beam CBR-I-30 experienced localized pitting corrosion near the mid-span of the beam, which explains the early rupture of the CFRP, as explained in the discussion later. Figure 43 shows an example of the residual masses of the same bar after each cycle of cleaning corrosion products.

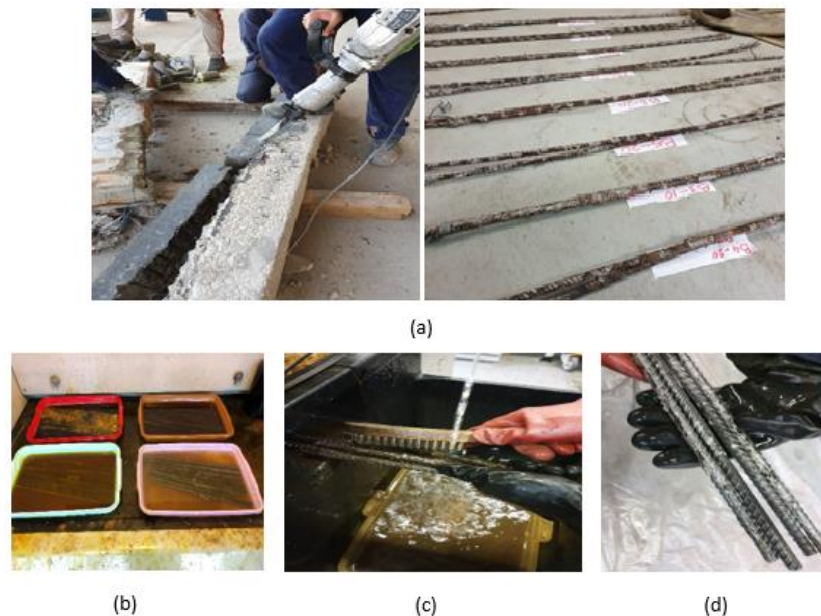


Figure 42: Gravimetric analysis. (a) extracting the bar. (b) submerging the bars in HCl solution. (c) brushing the bars. (d) drying the bars.

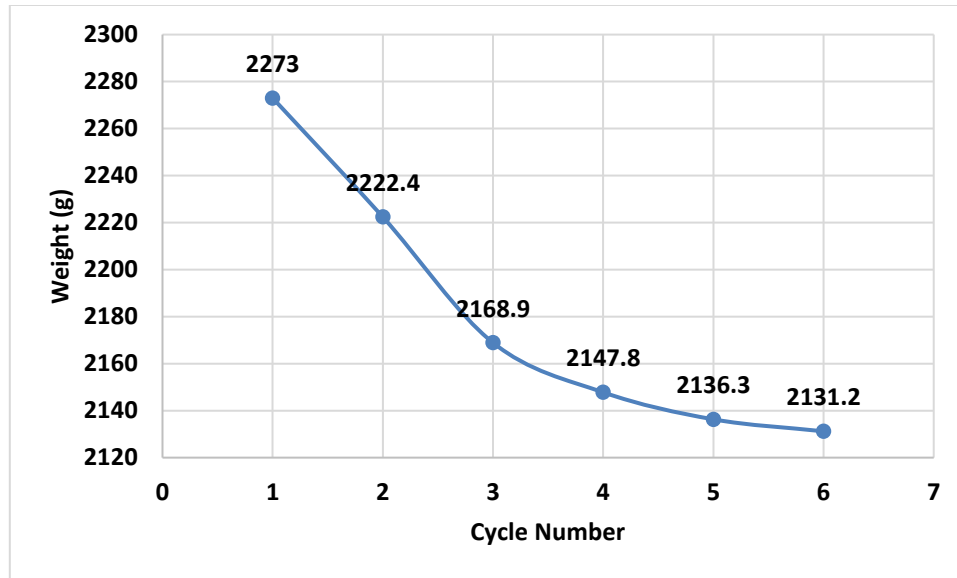


Figure 43: An example showing the mass after each cycle of cleaning corrosion products.

Table 6 shows the losses of mass of the both groups of corroded bars (i.e., one corroded to achieve a targeted 10% mass loss another targeted to achieve 20% mass loss). It can be observed from Table 6 that the actual average mass loss was 9.88% against a targeted level of 10% and average mass loss was 16.76% against a targeted level of 20%. In both cases the actual mass loss was very close to the targeted value.

Table 6: Losses of mass of the corroded bars

Percentage Mass Loss (%)			
Level (10%)		Level (20%)	
CA	9	CB	16
CAR	11.25	CBR	16.9
CA-I	9.51	CB-I	16.5
CAR-I	9.47	CBR-I	18.16
CAR-II	10	CBR-II	17.6
CAR-I-20	11	CBR-I-20	18.16
CAR-II-20	9	CBR-II-20	16
CAR-I-30	10	CBR-I-30	16.1
CAR-II-30	9.76	CBR-II-30	16.3
Average (%)	9.88	CBH	16.15
		CBH-I	16.53
		Average (%)	16.76

4.4 Mechanical Performance of Beam Specimens

4.4.1 Strengthening of Corroded Beams with CFRP Laminates along with Different Cover Replacements.

Loads versus mid-span deflections for all the tested beam specimens are illustrated in Figure 44, Figure 46, Figure 47, and Figure 49. The control un-corroded beam UU is included in all the graphs to make a comparison. All beam test results are summarized in Table 7.

4.4.1.1 Control beams

The control specimen (UU) failed at an ultimate load of 85.5 kN. For beams (UU, CA, and CB), tension-steel bars yielded before the crushing of concrete in the compression zone, which indicates a flexural failure mode. The control beams UU, CA, and CB started their first crack at the loads 18.2, 10.4, and 6.4 kN, respectively, which indicates that the 10% corrosion level had a lower effect on the concrete cover as compared with 20% corroded beams. The beam that was corroded to the level of 10% (CA) showed a reduction in the ultimate carrying load by 17%, failing at 71.1 kN. In comparison, the beam with a 20% (CB) corrosion level showed a reduction of 23.5%, failing at 65.4 kN compared to the control specimen (UU). Figure 44 shows the load vs. midspan deflection of the three control beams. No significant change in the stiffness of the corroded beams (CA and CB).

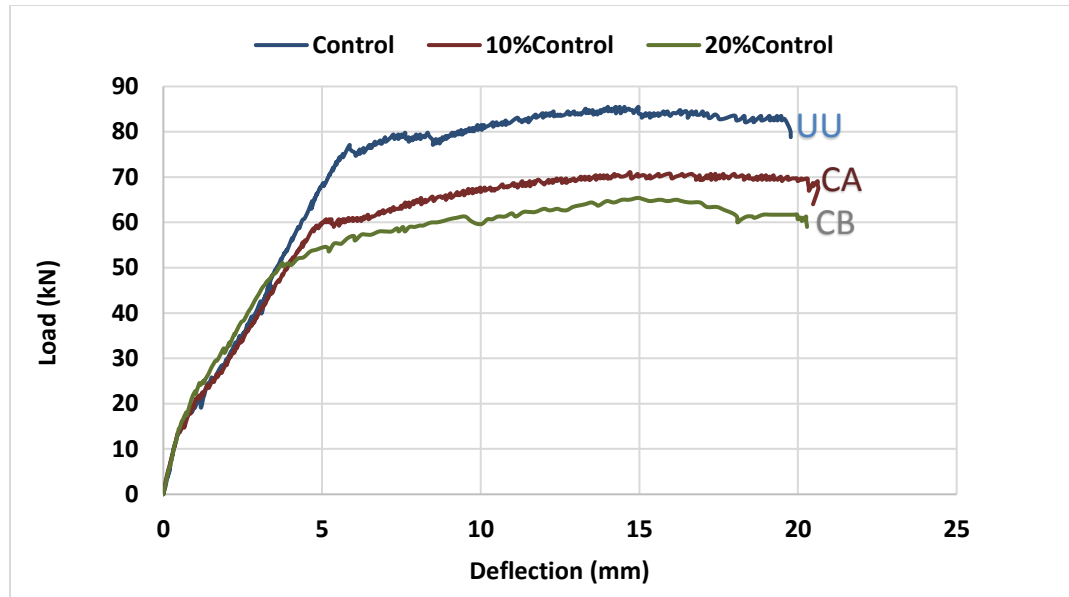


Figure 44: Load vs. deflection curve for control corroded and un-corroded beams

4.4.1.2 Beams repaired with different covers without CFRP strengthening.

The beams that were repaired with different covers showed the same failure mode. The failure mode was pure flexure, in which three main vertical cracks appeared at the beams' midspan. The repair material cover showed an extra number of cracks only on the cover, as shown in Figure 45 - c. In contrast, beams repaired with UHPC cover showed cracks that extend to the main concrete beams with lower extra cracks on the cover, as shown in Figure 45 - d. The beams started cracking at different cracking loads, as shown in Table 7. Corroded beams with repair material cover (CAR and CBR) showed an enhancement in the ultimate load by 9.5 and 19%, respectively, compared with beams having their original cover (CA and CB). Beam corroded to 20% having a UHPC cover replacement (CBH) have similar ultimate load as the beam with repair material cover (CBR) but with higher stiffness value. As Figure 46 shows, the (CBH) beam showed an enhancement in the stiffness by 7.4% compared with (CB), while beams with repair material cover showed no stiffness enhancement. The crushing of concrete commenced after tensile steel yielding.



(a)



(b)



(c)



(d)

Figure 45: Flexural cracks at failure for (a) 10% corroded beam with original cover (CA). (b) 20% corroded beam with original cover (CB). (c) Beams with repair material cover (CAR and CBR). (d) Beam with UHPC cover (CBH).

Table 7: Beams experimental results (control beams and beams strengthened with CFRP only)

Beam	At Concrete Cracking		At steel Yielding		At Maximum Load		At Ultimate Load (Rupture- Debonding-Failure)		Failure mode	
	P_{cr} (KN)	Deflection (mm)	P_y (KN)	Deflection (mm)	P_{max} (KN)	Deflection (mm)	P_u (KN)	Deflection (mm)	General	CFRP
UU	18.2	1.186	75.4	6.32	85.5	14.01	80.4	19.752	Pure flexure	None
CA	10.4	0.352	58.7	4.888	71.1	14.714	68	20.63	Pure flexure	None
CB	6.4	0.218	50.3	3.784	65.4	14.956	59.7	20.274	Pure flexure	None
CAR	12.4	0.914	61.3	4.832	77.8	16.082	76.1	25.106	Pure flexure	None
CBR	11.1	0.668	64.4	5.008	78.1	16.64	72.7	20.495	Pure flexure	None
CBH	22.8	1.12	74.7	5.28	79.1	6.86	61.3	15.88	Pure flexure	None
CA-I	27.2	1.73	114.6	10.3	144.5	19.36	144.5	19.36	Pure flexure	Rupture
CB-I	24.1	1.52	101.6	8.46	143.8	17.1	142.8	18.98	Pure flexure	Rupture
CAR-I	36.2	2.06	112.6	8.34	134.4	15.32	130.4	21.32	Pure flexure	Rupture
CBR-I	24.8	1.56	71.7	5.68	113.3	20.3	104.2	23.58	Pure flexure	Rupture
CBH-I	32.2	1.716	113.3	6.97	145.8	11.886	134.8	15.008	Pure flexure	Rupture
CAR-II	40.2	1.962	123.4	6.673	186.4	19.624	184.7	20.172	Pure flexure	Debonding
CBR-II	38.5	2.66	108.9	7.36	166.3	20.86	165.6	20.65	Pure flexure	Debonding

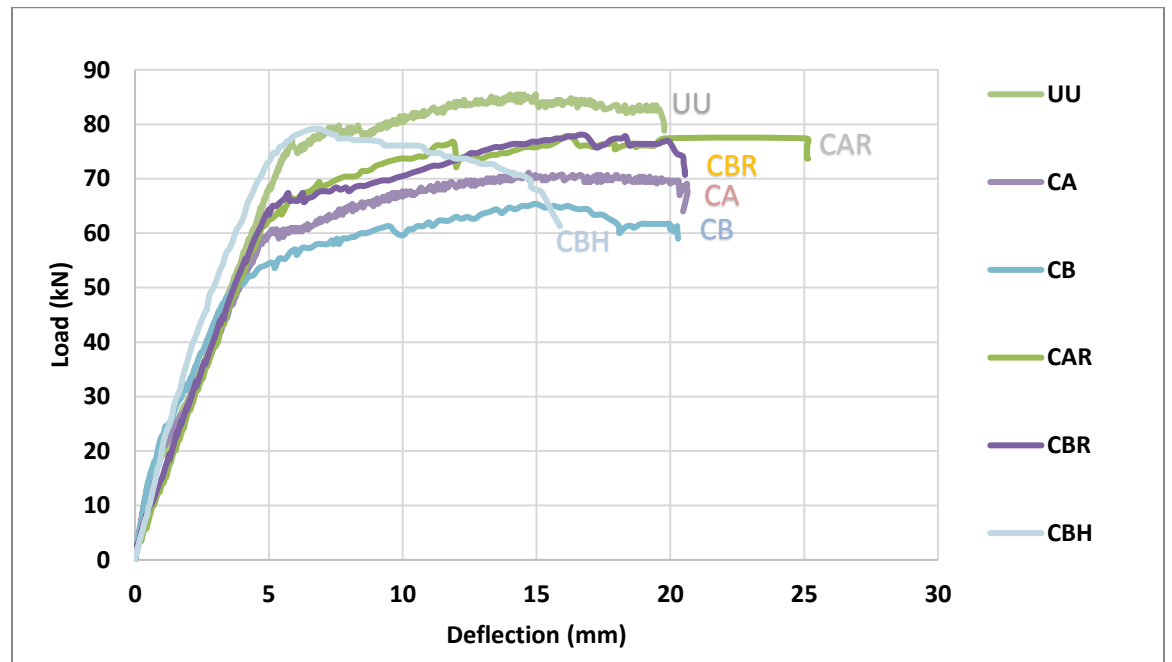


Figure 46: Load vs. deflection curve for corroded un-strengthened beams with different cover material.

4.4.1.3 Beams strengthened with one layer of CFRP over the original cover.

The corroded beams strengthened with one layer of CFRP over the original cover (CA-I and CB-I) showed an enhancement in the flexural capacity compared with un-strengthened beams (CA and CB). The beams' failure mode strengthened with one layer of CFRP over the old cover (CA-I and CB-I) is pure flexure by CFRP rupture, as shown in Figure 48(a).

For 10% corrosion level, CA-I showed an ultimate load of 144.5 kN, which is an enhancement of 103% compared with CA. For 20% corrosion level, CB-I showed an ultimate load of 143.8 kN, which is an enhancement of 120% compared with CB. The CFRP strengthened beams showed no significant enhancement in the stiffness than beams with no CFRP strengthening (CA and CB). Figure 47 shows the load vs. deflection for CA-I and CB-I with the control specimens (UU, CA, and CB).

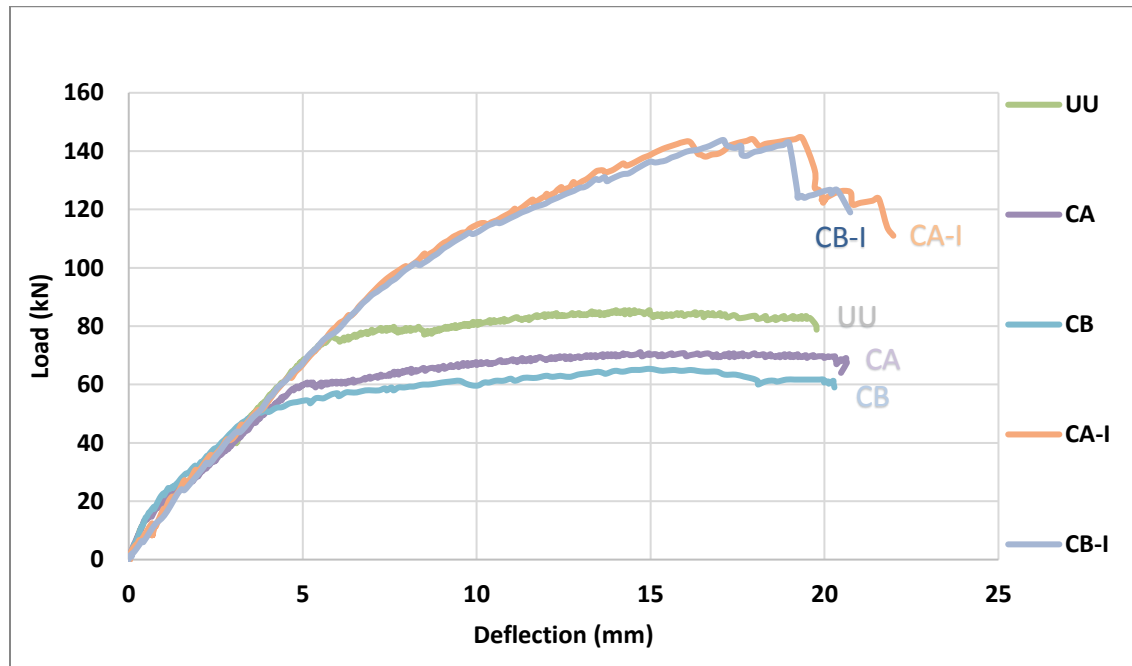


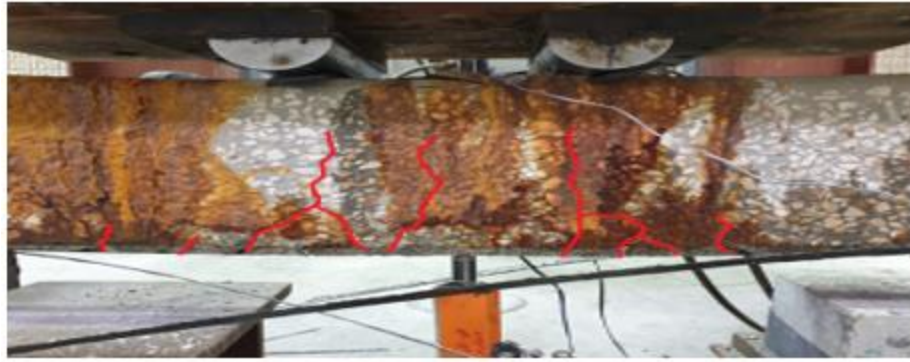
Figure 47: Load vs. deflection curve for corroded beams strengthened with one layer of CFRP over the original cover

4.4.1.4 Beams strengthened with one layer of CFRP on different covers, replacing the material.

To see the effect of different covers under CFRP strengthening, one CFRP layer is kept constant for all beams in Figure 49 (CA-I, CB-I, CAR-I, CBR-I, CBH-I). Beams strengthened with CFRP over repair material cover showed an enhancement in the load-bearing capacity by 89 and 73% compared with CA and CB, respectively, and an

enhancement in the stiffness by 30 and 5%. Beams strengthened with CFRP over the original cover showed a higher enhancement in the load-bearing capacity and maintained CA and CB's original stiffness. The beam strengthened with CFRP over UHPC cover showed an enhancement in the load-bearing capacity by 123% compared with CB and an increment in the stiffness by 35%, which is higher than beams having original and repair material cover. Beams strengthened with CFRP over repair material cover (CAR-I and CBR-I) showed higher ductility before failure by 15% compared with beams strengthened over original covers (CA-I and CB-I).

On the other hand, the beams strengthened with CFRP over the UHPC cover showed a reduction in the ductility by 35% compared with (CB-I). All the beams with one CFRP layer failed in pure flexure by CFRP rupture with different cracking behavior. Figure 48 shows the beams' failure strengthened with one layer of CFRP over the three different covers.



(a)



(b)



(c)

Figure 48: Cracking behavior of one layer of CFRP strengthening on (a) original cover. (b) repair material cover. (c) UHPC cover.

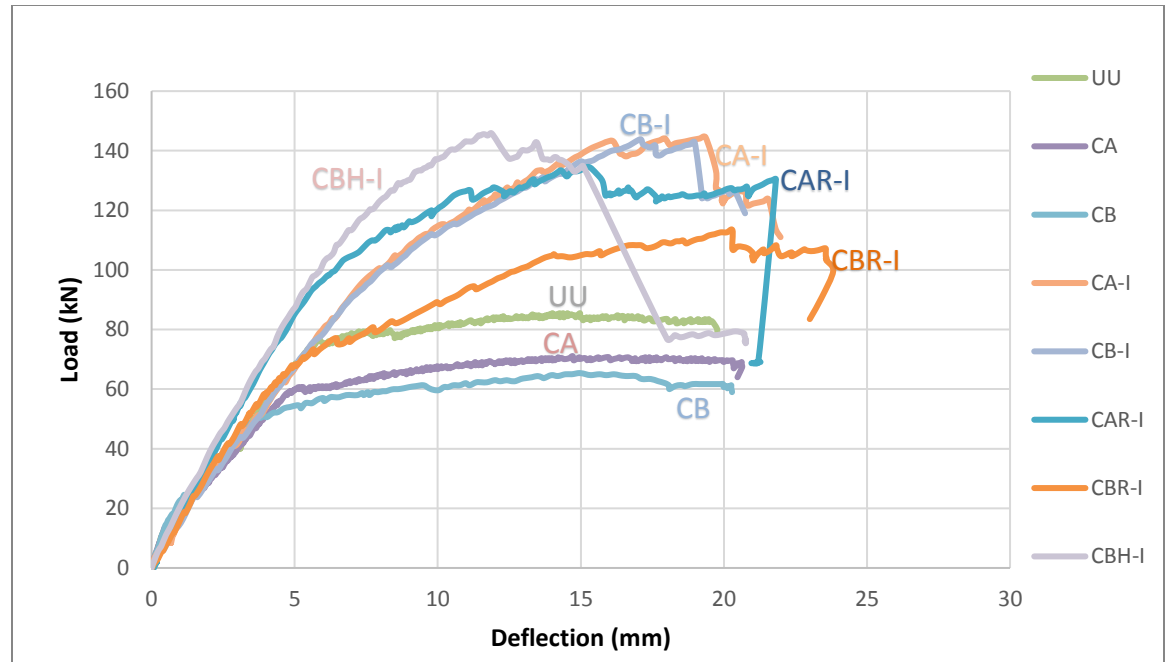


Figure 49: Load vs. deflection curve for corroded beams strengthened with one layer of CFRP over different cover material

4.4.1.5 Beams strengthened with one layer and two layers of CFRP

The repaired beam corroded to 10% and strengthened with one layer of CFRP (CAR-I) showed an ultimate load of 134.4 kN with an increment of 89% compared with (CAR). In comparison, the beam that is corroded to the same level and strengthened with two layers of CFRP (CAR-II) showed an ultimate load of 186.4 kN with an increment of 162% compared with (CAR). For the second level of corrosion (20%), the repaired beams showed an enhancement of 73% and 154% for one layer and two layers of CFRP strengthening, respectively, as Figure 52 shows. The first crack started at around 36.2 and 24.8 kN for CAR-I and CBR-I, respectively, while the first crack started at 40.2 and 38.5 kN for CAR-II and CBR-II. Beams strengthened with two CFRP layers showed lower ductility by an average value of 9% compared with beams strengthened with one CFRP layer. The failure mode for beams with one layer of CFRP strengthening (CAR-I and CBR-I) was pure

flexure by rupture of the CFRP at the middle of the beam, as Figure 50 shows. The failure mode for beams with two CFRP strengthening layers (CAR-II and CBR-II) was pure flexure by de-bonding the CFRP laminates. For beams strengthened with two layers, the main CFRP laminate in flexure deboned from the cover after the rupture and de-bonding in the U-shaped reinforcement as shown in Figure 50.

The beams strengthened with CFRP showed an enhancement in the stiffness depending on the number of CFRP layers. Figure 51 shows the average percentage of enhancement in the beams stiffnesses with respect to CAR and CBR.



(a)



(b)

Figure 50: (a) Rupture for bBeams with one CFRP Layer. (b) de-bonding for Beams with Two Layers of CFRP by rupture of U-shaped CFRP

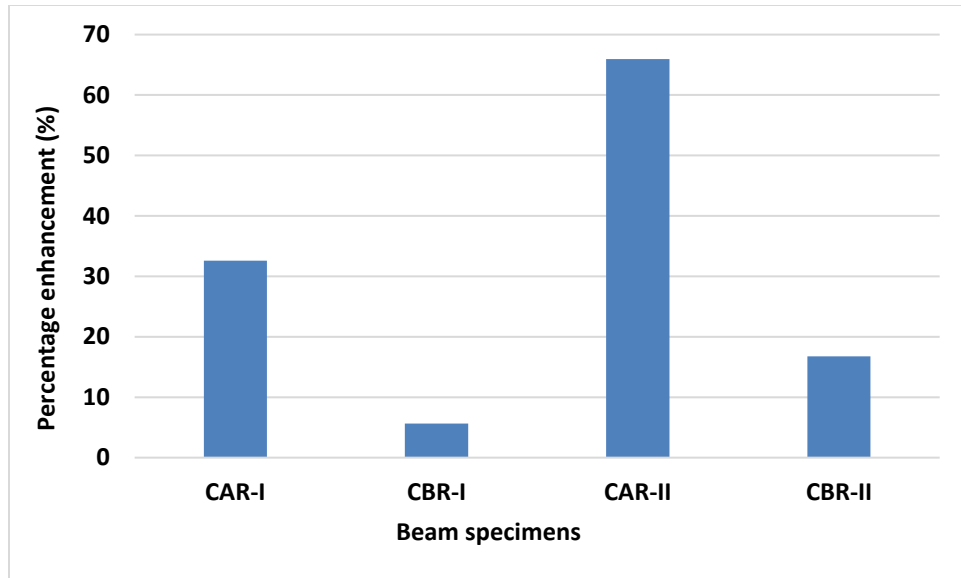


Figure 51: Approximate stiffness enhancement % with respect to CAR and CBR beams.

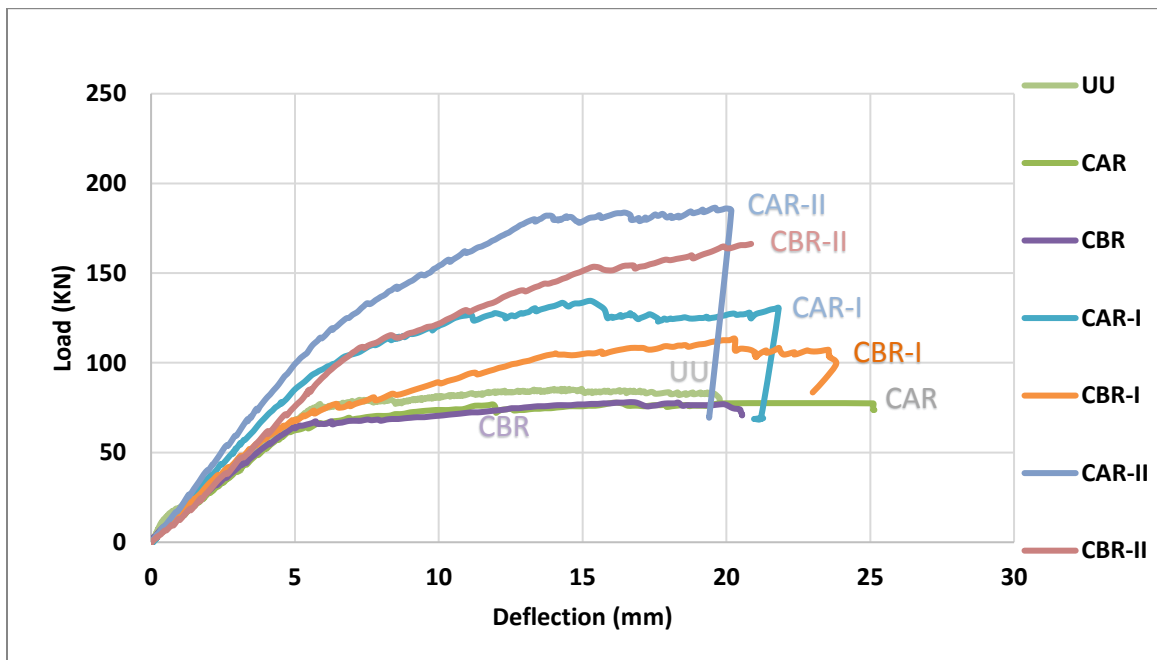


Figure 52: Load vs. deflection curves for corroded beams with the same cover replacement on variable CFRP layers.

4.4.1.6 Strain Evolvement

Strain graphs show another point of view of the beam behavior under loading since they can indicate the elastic stage, cracking stage, and yielding stage as the flexural stiffness graphs do. Figure 53 shows the load vs. concrete strain, steel strain, and mid-span CFRP strain for beams strengthened with one and two CFRP layers. The point where the graph changes its behavior from linear to non-linear is considered the first actual crack. Table 7 and Table 8 show the loads at which the first crack occurs.

CFRP beams' ultimate strain value with one layer was in the range of 9000 to 11000 $\mu\epsilon$, while the beams with two CFRP layers deboned at a strain of 6000 $\mu\epsilon$. CFRP beams' ultimate strain value with one layer is higher than those strengthened with two layers because the two CFRP layers deboned before reaching their ultimate stress capacity. On the contrary, the beams' concrete ultimate strain strengthened with two layers of CFRP are slightly higher than beams strengthened with one layer of CFRP.

The steel strain gauges installed on the tensile bars of the corroded beams did not work because the corrosion has damaged the majority of them. The ultimate steel strain value for beam CA-I was 3102 $\mu\epsilon$ which exceeds the yielding strain of the steel. All the tested beams reached concrete crushing (3000 $\mu\epsilon$). A small piece of concrete crust, where the concrete strain gauge was installed, deboned from the concrete surface before the failure of the beam (as in Figure 48), which prevented the concrete strain gauge from reading after the value of 2600 $\mu\epsilon$ as shown in all the beam cases in Figure 53.

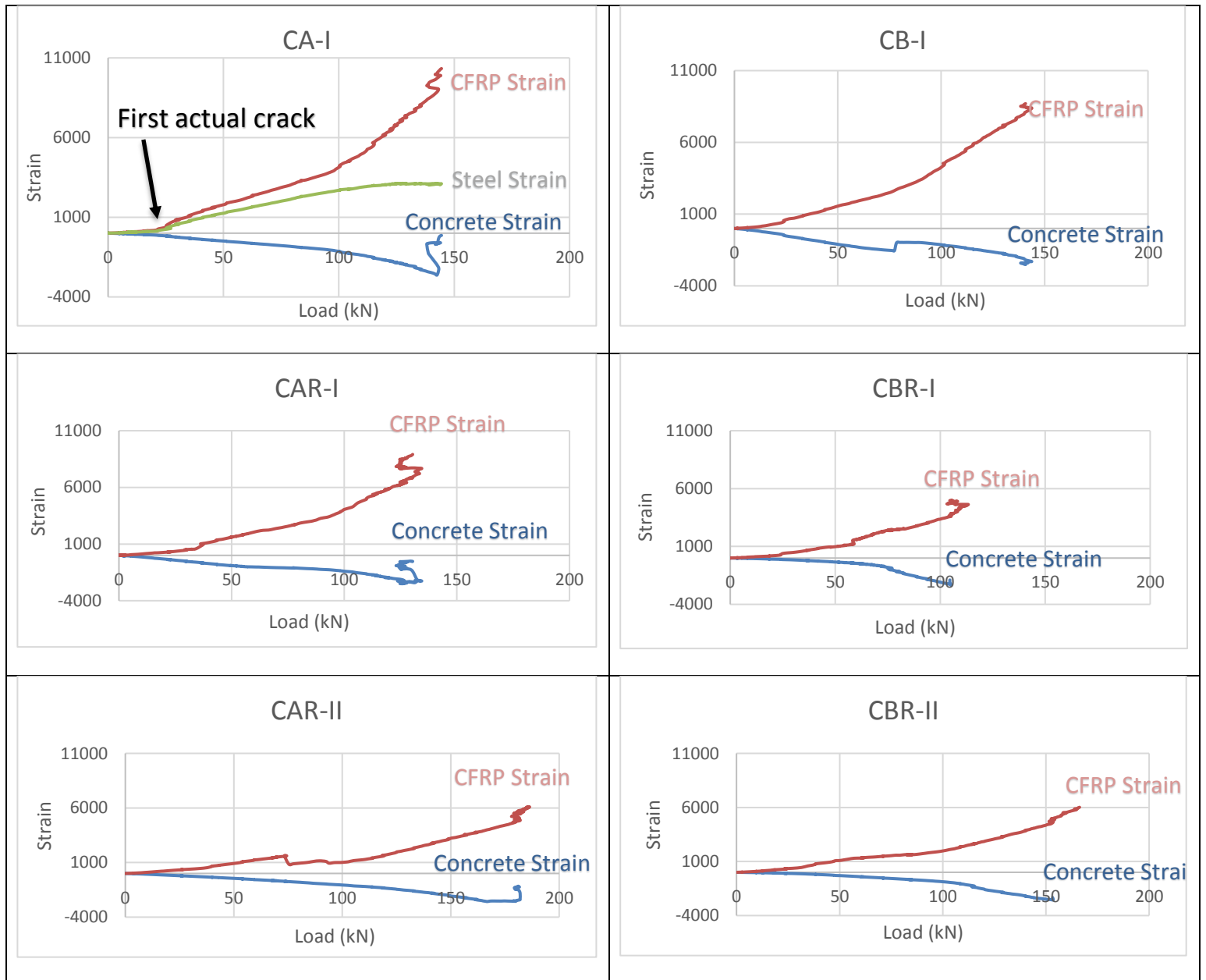


Figure 53: Load vs. strain for beams strengthened with one and two CFRP layers.

4.4.2 Strengthening beams with hybrid System

4.4.2.1 Failure mode

All the hybrid beams strengthened with one CFRP layer showed a pure flexural behavior with one main crack and other small secondary cracks. The cracks started at a load ranging 55-65 kN except for CBR-I-20, where the first crack occurred at an earlier load of 37 kN. CFRP embedded in the hybrid beams failed by rupture for both beams strengthened with one layer and to layers of CFRP, which indicates that the UHPC jacket prevented the CFRP from de-bonding forced the CFRP to attain its ultimate stress. For hybrid beams strengthened with two layers of CFRP, the failure mode was flexural-shear by CFRP rupture. At the beginning of the test, the hybrid beams (strengthened with two CFRP layers) cracked at an average load of 75 kN for beams strengthened with 20 mm UHPC and around 95 kN for beams strengthened with 30 mm UHPC forming three cracks, one in the middle and two at the sides. The two cracks on the sides grew and widened as a pure flexural crack until reaching a load of around 200 kN for 20 mm UHPC strengthening and 220 kN for 30 mm UHPC strengthening where the two cracks on the side started following a new flexure-shear path until the failure occurred. Figure 54 shows the cracking behavior of UHPC beam strengthened with one and two layers of CFRP.

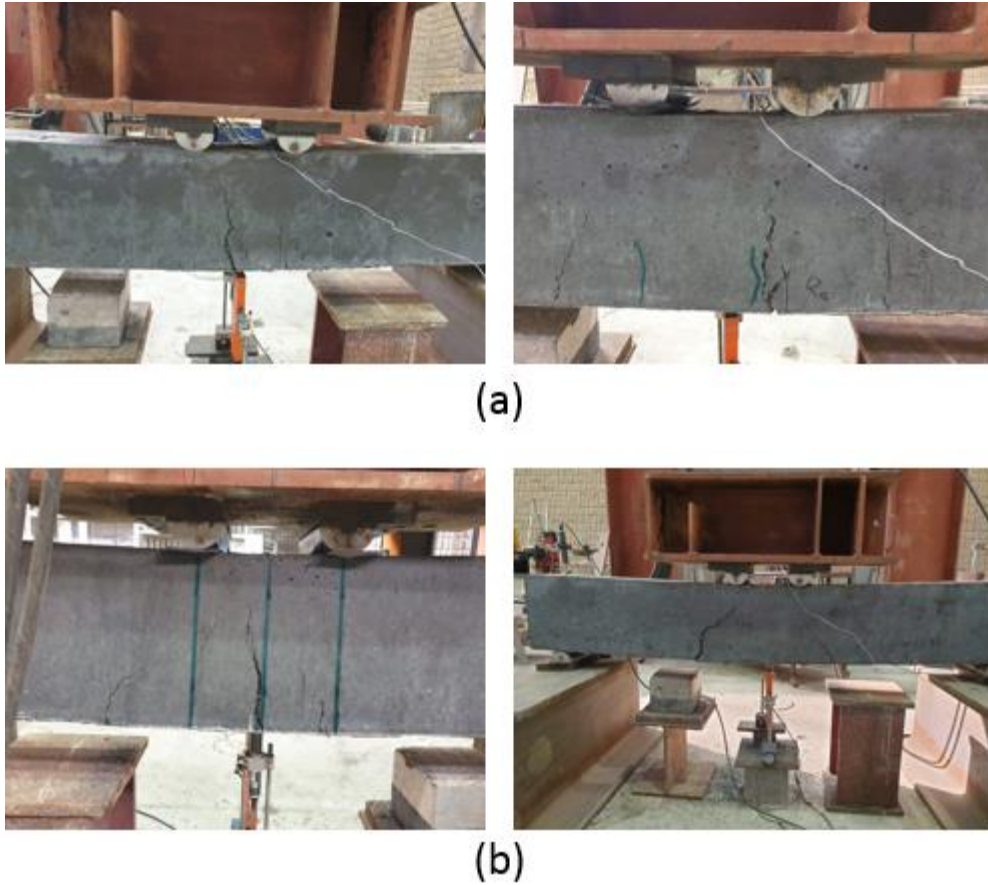


Figure 54: Crack behavior for beams strengthened with (a) UHPC + one layer of CFRP (b) UHPC + two CFRP layers.

4.4.2.2 Ductility

As shown in Figure 44, the ductility of the corroded control beams was not affected. The ductility has increased by 5% for CA and 4% for CB than the original un-corroded beam (UU). The beams strengthened with one layer, and two layers of CFRP were able to maintain a ductile behavior as the original corroded beams (CA and CB). The beams with a single CFRP layer showed higher ductility than beams strengthened with two CFRP layers by 9%. The beam strengthened with the hybrid system of CFRP and UHPC showed a reduction in the ductility before CFRP rupture. Figure 55 and Figure 56 show the reduction in the beams' ductility strengthened with the hybrid system compared with beams

strengthened with one CFRP layer. The reduction in the ductility for hybrid beams strengthened with two CFRP layers was lower than hybrid beams strengthened with one CFRP layer. After the CFRP rupture, all the beams remain ductile but at lower loads.

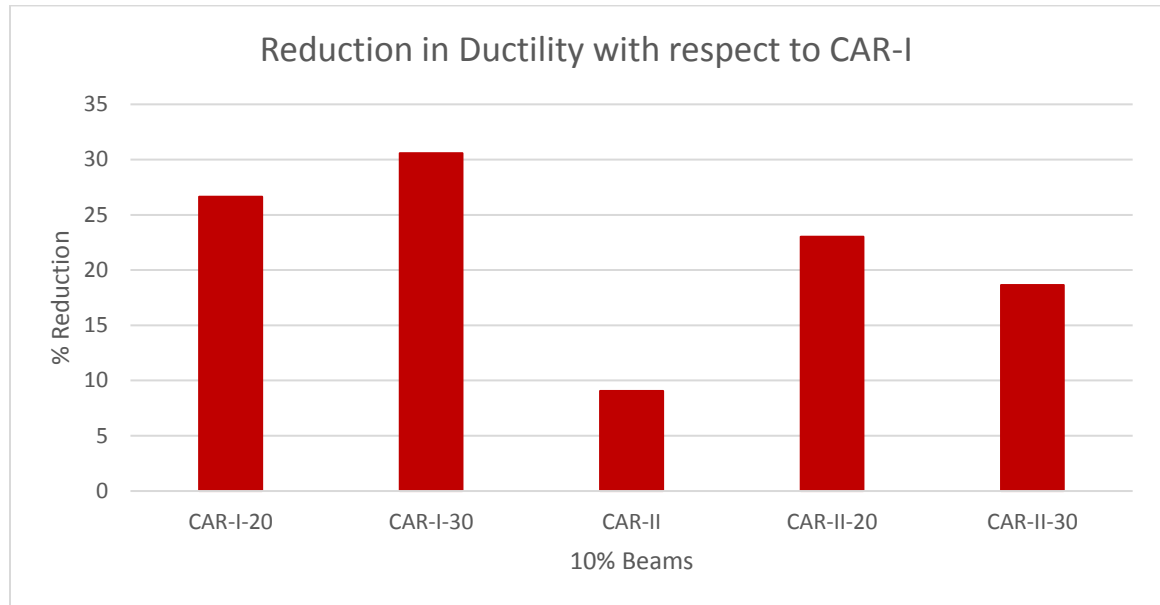


Figure 55: Percentage reduction in the ductility for beams corroded to 10% and strengthened with the hybrid system.

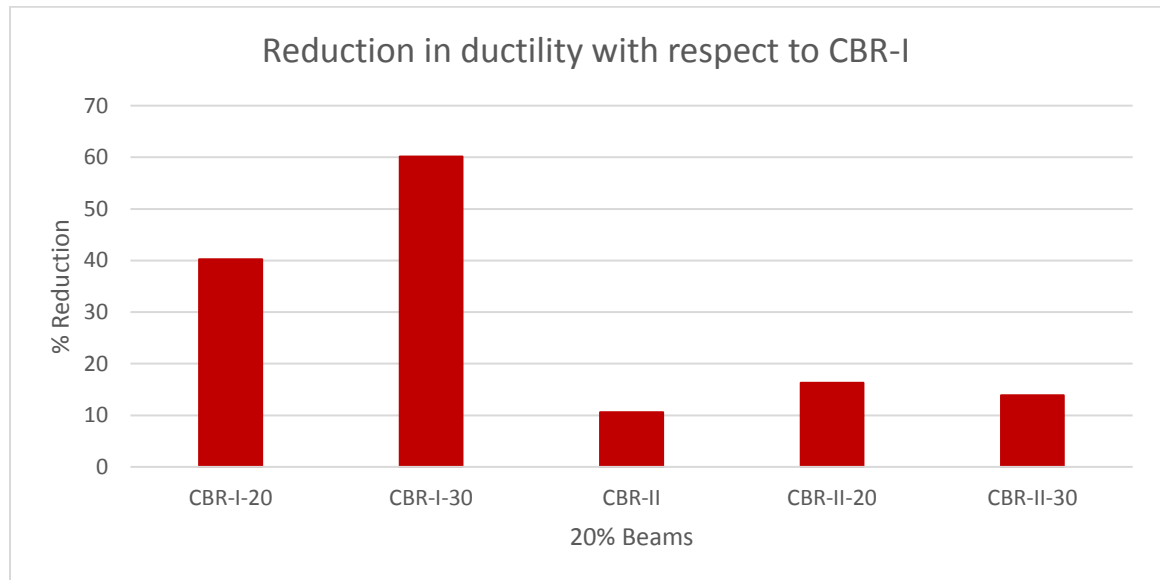


Figure 56: Percentage reduction in the ductility for beams corroded to 20% and strengthened with the hybrid system.

4.4.2.3 Flexural performance and Stiffness

Table 8 shows the beams' flexural test results, including cracking load, maximum load, and ultimate (failure) load.

For the hybrid beams, flexural capacity and stiffness increased with the increasing thickness of the UHPC jacketing. Figure 62 and Figure 63 show the load vs. midspan deflection for the beams strengthened with the hybrid system. The beam CAR-I-20 showed an enhancement in the maximum load by 137.5% compared with the beam strengthened with CFRP only (CAR-I) that showed an enhancement by 89%. By increasing the UHPC thickness to 30 mm as done with CAR-I-30, the enhancement was further improved to achieve 181% compared with CAR-I, which is almost double the enhancement provided by CAR-I (89%). Figure 59 and Figure 60 show bar charts explaining the enhancement percentage in the maximum moment provided by each corrosion configuration with respect to the control corrosion beams CA and CB. The stiffness of the hybrid beams was also enhanced with the increasing thickness of the UHPC Jacket. The beam CAR-I-20 showed an improvement in the stiffness by around 66.5% compared with CAR-I that had an enhancement of 35%. Figure 57 and Figure 58 show an approximate stiffness enhancement for the control beams CA and CB. Beam CBR-I-30 showed relatively lower ultimate load due to localized pitting corrosion commenced in one of its bars. In terms of flexural capacity and stiffness, the beams strengthened with the hybrid system of one CFRP layer and a UHPC jacket of 20 or 30 mm showed similar results as beams strengthened with only two CFRP layers which indicates that the UHPC jacket substituted the extra CFRP layer successfully. The beams strengthened with the hybrid system of two CFRP layers and

UHPC jacket showed a huge enhancement in terms of stiffness and flexural strength as shown in Figure 57Figure 58Figure 59Figure 60.

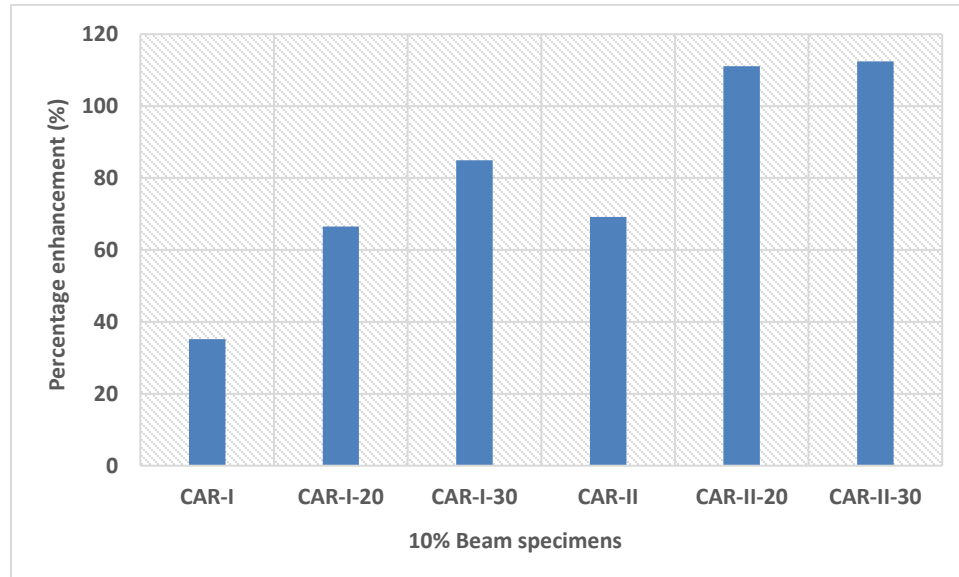


Figure 57: Approximate stiffness enhancement for 10% corrosion for CA beam

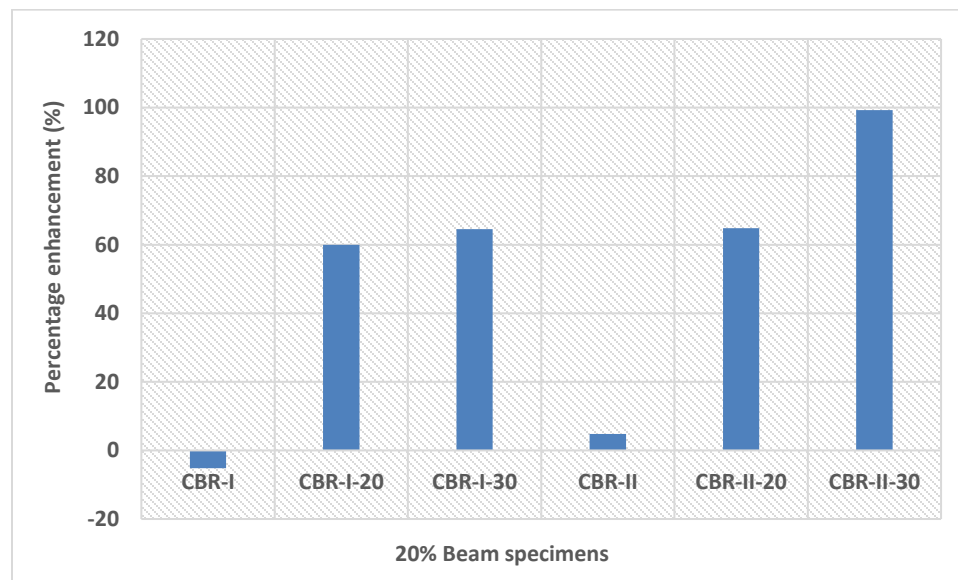


Figure 58: Approximate stiffness enhancement for 20% corrosion for CB beam

Table 8: Results of beam specimens (hybrid).

Beam	At Concrete Cracking		At steel Yielding		At Maximum Load		At Ultimate Load (Rupture-Debonding-Failure)		Failure mode	
	P_{cr} (KN)	Deflection (mm)	P_y (KN)	Deflection (mm)	P_{max} (KN)	Deflection (mm)	P_u (KN)	Deflection (mm)	General	CFRP
UU	18.2	1.186	75.4	6.32	85.5	14.01	80.4	19.752	Pure flexure	None
CA	10.4	0.352	58.7	4.888	71.1	14.714	68	20.63	Pure flexure	None
CB	6.4	0.218	50.3	3.784	65.4	14.956	59.7	20.274	Pure flexure	None
CAR-I	36.2	2.06	112.6	8.34	134.4	15.32	130.4	21.32	Pure flexure	Rupture
CBR-I	24.8	1.56	71.7	5.68	113.3	20.3	104.2	23.58	Pure flexure	Rupture
CAR-II	40.2	1.962	123.4	6.673	186.4	19.624	184.7	20.172	Pure flexure	Debonding
CBR-II	38.5	2.66	108.9	7.36	166.3	20.86	165.6	20.65	Pure flexure	Debonding
CAR-I-20	61.7	2.608	122.7	6.16	168.9	15.546	167.6	16.074	Pure flexure	Rupture
CBR-I-20	37.5	1.42	131.1	5.805	177	12.62	165.6	14.28	Pure flexure	Rupture
CAR-II-20	77.4	2.4	153.5	5.93	228.9	13.18	224.2	16.68	flexural shear	Rupture
CBR-II-20	73.7	3	128.4	5.628	221.2	19.566	217.9	20	flexural shear	Rupture
CAR-I-30	65.4	2.58	139.4	6.222	200.4	15.146	200.4	15.146	Pure flexure	Rupture
CBR-I-30	52.3	1.8	119.3	5.64	152.5	9.62	151.5	9.52	Pure flexure	Rupture
CAR-II-30	91.8	3.236	177.7	7.168	260.5	17.656	252.7	17.752	flexural shear	Rupture
CBR-II-30	100.2	3.356	159.9	5.8	264.8	15.732	238.3	20.584	flexural shear	Rupture

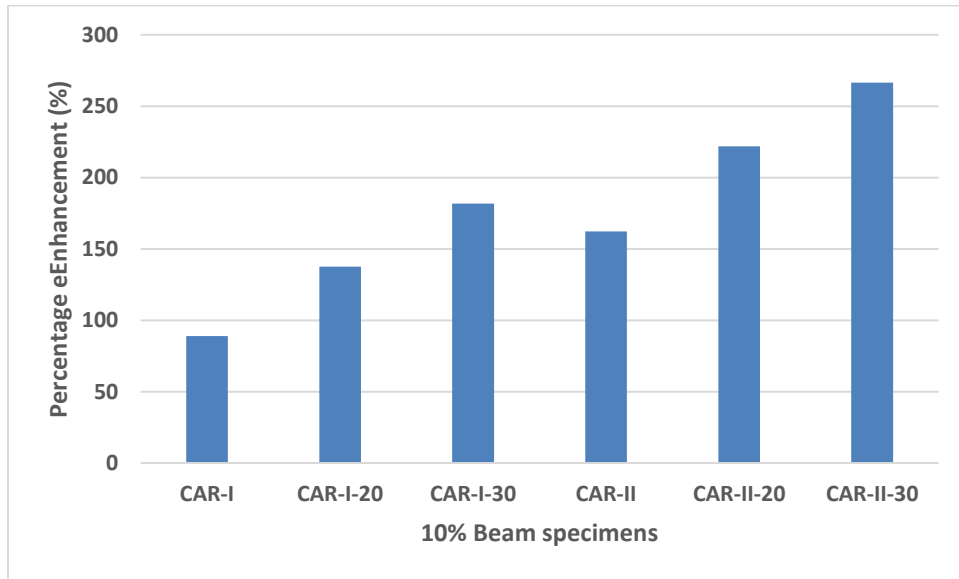


Figure 59: Percentage of maximum load enhancement for the corrosion control beam 'CA.'

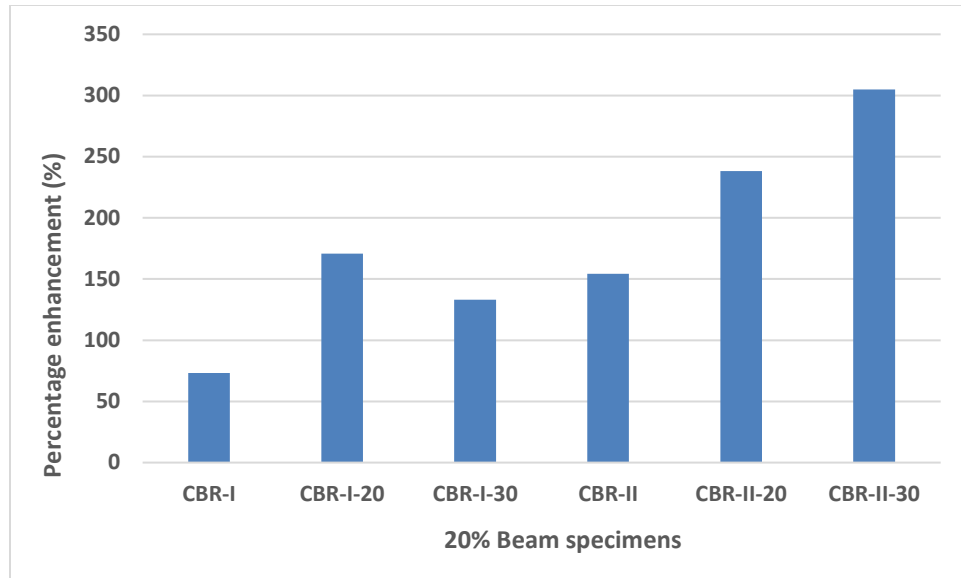


Figure 60: Percentage of maximum load enhancement for the corrosion control beam 'CB.'

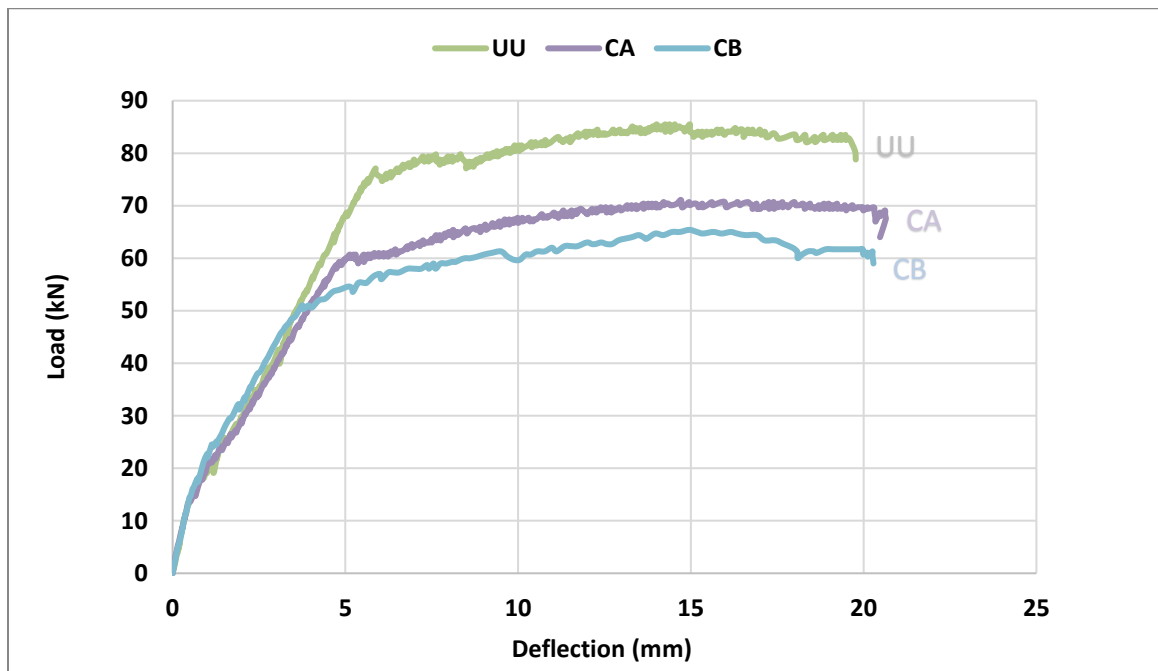


Figure 61: Load vs. deflection for control corroded and un-corroded beams.

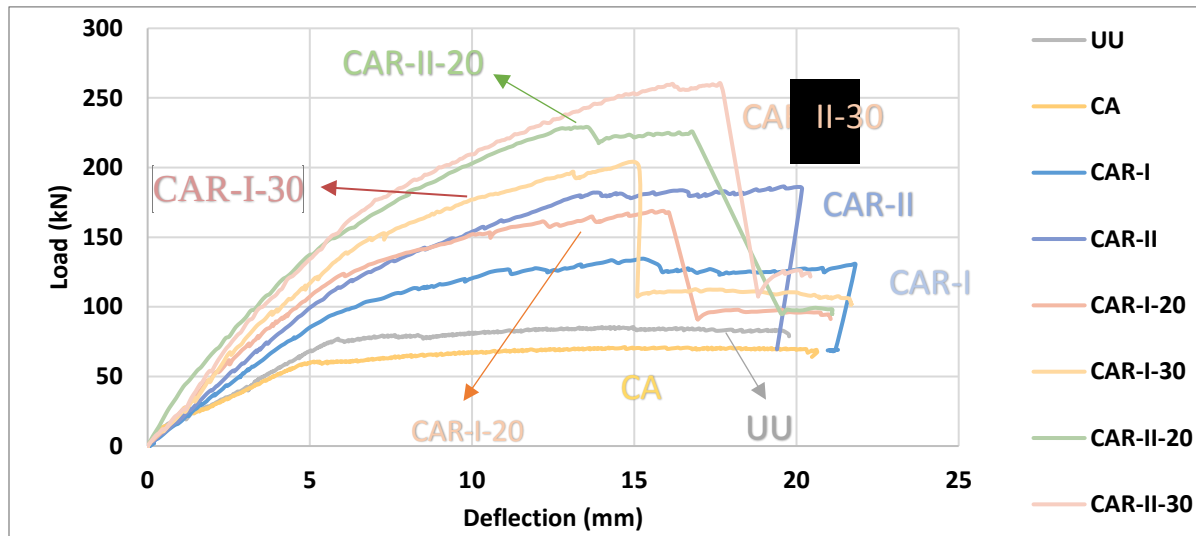


Figure 62: Load vs. deflection for hybrid beams and only CFRP strengthened beams corroded to 10%

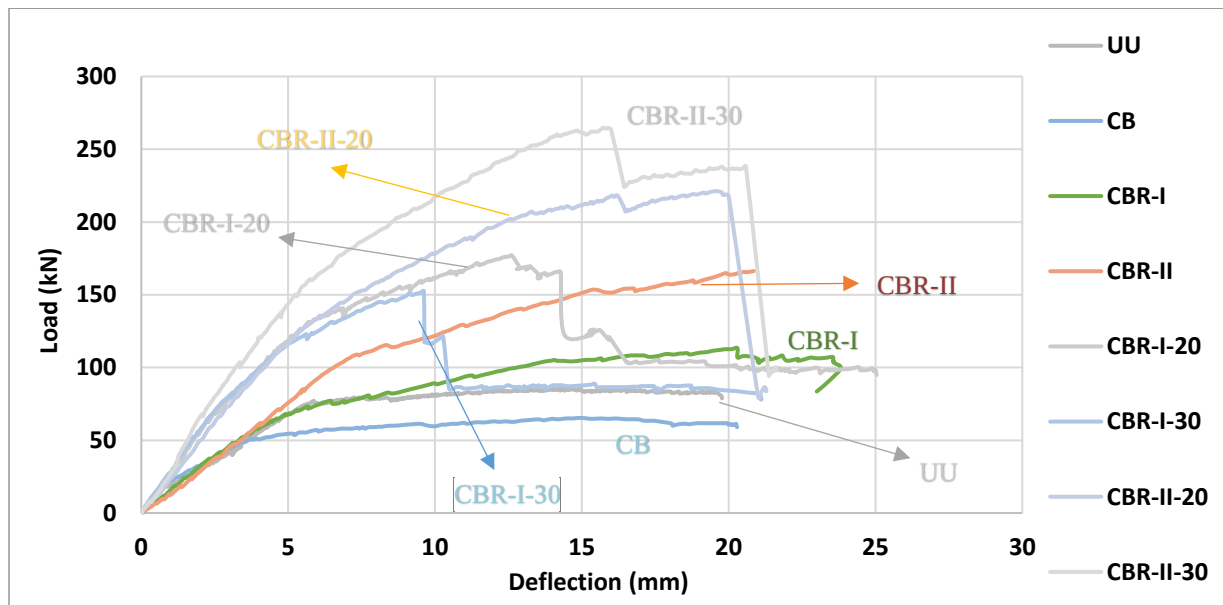


Figure 63: Load vs. deflection for hybrid beams and only CFRP strengthened beams corroded to 20%

CHAPTER 5

Analytical Solution

5.1 General

In this chapter, analytical models were developed to predict two types of strengthened beams' flexural strength. First, beams strengthened with CFRP, including the cover tensile strength. Second, beams strengthened with the hybrid system of CFRP and UHPC. The predicted maximum moments and loads are calculated according to force equilibrium (between tensile and compressive) along the beam's cross-section. The results obtained from the analytical solution are compared with the experimental results obtained earlier.

5.2 Analytical Model of Strengthened Corroded RC Beams with CFRP Along with Different Cover Replacement

To estimate the ultimate moment that a CFRP beam can achieve theoretically, the following decisions are made:

- 1- The tensile stress block of the repair material and the UHPC will be considered a rectangle instead of a trapezoid since the distance is small (the cover only) and makes the calculation simpler.
- 2- There is no relative slip between the CFRP and the concrete surface.
- 3- There is no relative slip between the tensile steel and its surrounding concrete.
- 4- The beam will be considered as singly reinforced. Thus, only tension bars will be included in the calculations.
- 5- The compression strain in the concrete is considered to be equal to 0.003.

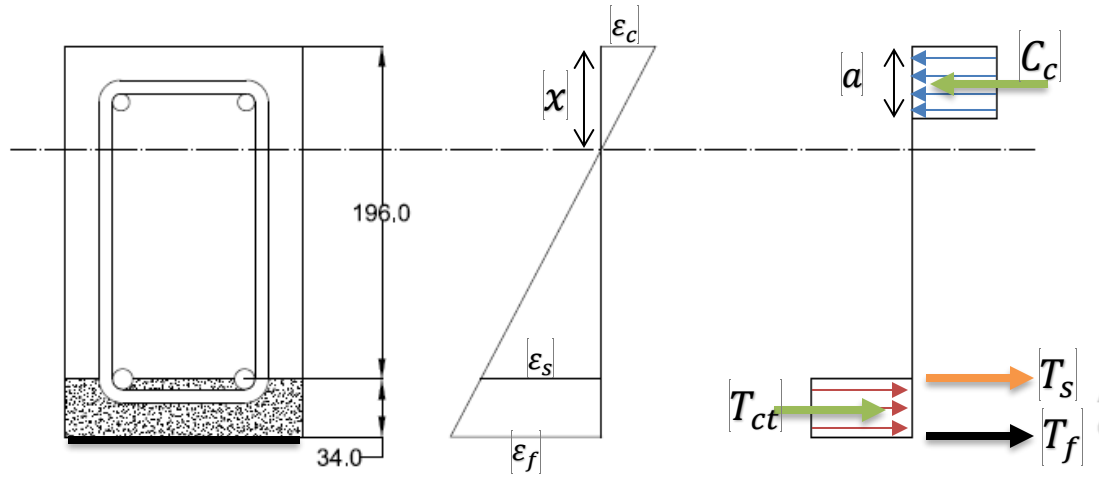


Figure 64: Analytical model for beams strengthened with CFRP only.

Steps and equations used in the Analytical Solution.

Step 1: Calculating the new cross-sectional area of the corroded bar using the percentage loss obtained from gravimetric analysis

To calculate the cross-sectional area of the corroded bars, Eq. 5.2.1 was used.

$$A_{sc} = A_s * (1 - \alpha)^2 \quad (5.2.1)$$

α = percentage reduction in the bar diameter.

$$\text{Where } \alpha = \frac{2 * P_r * T}{D} \quad (5.2.2)$$

T = corrosion duration (days)

D = original diameter of steel bar (cm)

$$P_r = \frac{W}{F * \gamma_{st}} * I_{corr} = \frac{J_r}{\gamma_{st}} \quad (5.2.3)$$

P_r = penetration rate cm/year by Ijsseling [41]

I_{corr} = corrosion current density mA/cm²

γ_{st} = density of steel = 7.85 g/cm³

J_r = corrosion rate g/cm²/year

W = equivalent weight of steel = 55.85/2 = 27.925 g

F = faraday's constant = 96487 coulombs (Amp-sec)

Step 2: Preparing the forces. The forces are calculated by multiplying the stresses with the cross-sectional area covered by each stress. The forces are set in terms of the depth of the natural access 'x'.

$$C_c = 0.85 * f'_c * \beta_1 * x * b \quad (5.2.4)$$

$$\text{Where } \beta_1 = 0.85 - \frac{0.05 * (f'_c - 28)}{7} \quad (5.2.5)$$

$$T_s = A_{sc} * f_y \quad (5.2.6)$$

$$T_f = N * A_f * E_f * \frac{h_c - x}{x} * 0.003 \quad (5.2.7)$$

Where h_c = full depth of the beam, N= number of CFRP layers

$$T_{ct} = f_{ut} * h_u * b_c \quad (5.2.8)$$

Where f_{ut} = tensile stress in cover, h_u = cover thickness, b_c = width of the beam.

Step 3: The equations from 5.2.4- 5.2.8 were simplified and expressed in terms of x (depth of the natural axis), then the value of x is iterated until the following force equilibrium equation became equal to Zero:

$$\Sigma F = 0$$

$$T_s + T_f + T_{ct} - C_c = 0 \quad (5.2.9)$$

Step 4: Calculate the value of 'a.'

$$a = \beta_1 * x \quad (5.2.10)$$

Step 5: Checking if tension-steel yielded by making sure that $\varepsilon_s > \varepsilon_y$ and check that CFRP stress < ultimate stress (3500 MPa).

For steel:

$$\varepsilon_s = \frac{d-x}{x} * 0.003 \quad (5.2.11)$$

$$\varepsilon_y = \frac{f_y}{E_s} \quad (5.2.12)$$

For CFRP:

$$E_f * \frac{h_c-x}{x} * 0.003 < 3500 \text{MPa (Ultimate tensile stress)} \quad (5.2.13)$$

Step 6: Calculating the predicted moment by multiplying each force with its lever arm around the natural depth 'x.'

$$M_{pred} = C_c * \left(x - \frac{a}{2}\right) + T_s * (d - x) + T_f * (h_c - x) + T_{ct} * \left(h_c - x - \frac{h_u}{2}\right) \quad (5.2.14)$$

Figure 65 shows a flow chart that briefly explains the analytical model.

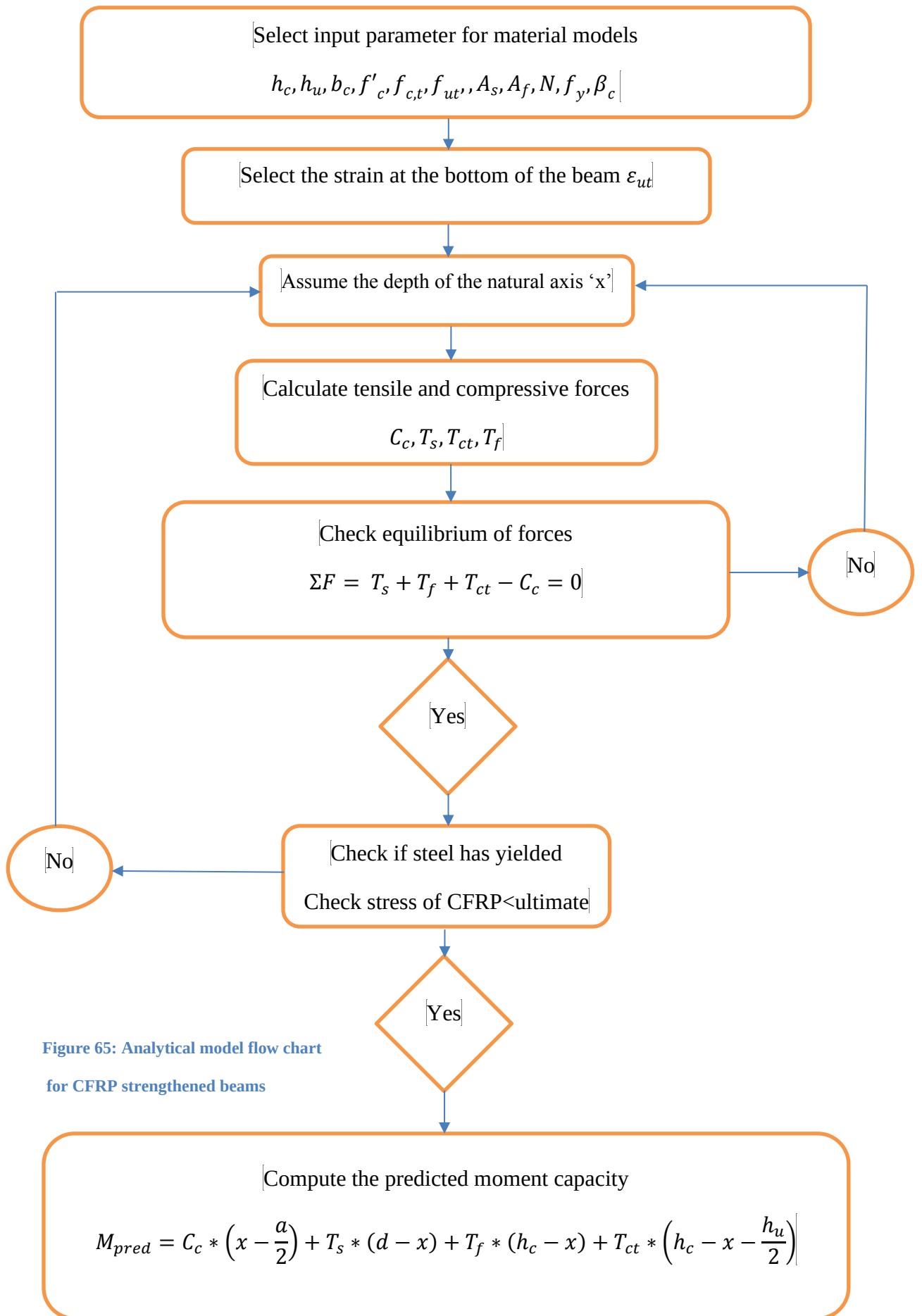


Table 9 shows the theoretical and experimental results of the ultimate moment capacity of the beams. Figure 66 shows a comparison between the analytical and the experimental results.

Table 9: Experimental and analytical results

Beam	Experimental		Theoretical		Error (%)
	$M_{exp} (kN.m)$	$P_{exp} (kN)$	$M_{th} (kN.m)$	$P_{th} (kN)$	
UU	25.65	85.5	25.05	83.51	2.33
CA	21.33	71.1	21.11	70.37	1.02
CB	19.62	65.4	19.56	65.22	0.28
CAR	23.34	77.8	22.72	75.72	2.67
CBR	23.43	78.1	21.40	71.35	8.65
CBH	23.73	79.1	25.21	84.04	-6.24
CA-I	43.35	144.5	40.49	134.97	6.59
CB-I	43.14	143.8	39.92	133.05	7.47
CAR-I	40.32	134.4	38.04	126.81	5.65
CBR-I	33.99	113.3	36.81	122.69	-8.29
CBH-I	43.74	145.8	37.11	123.70	15.16
CAR-II	55.92	186.4	46.79	155.96	16.33
CBR-II	49.89	166.3	45.97	153.23	7.86

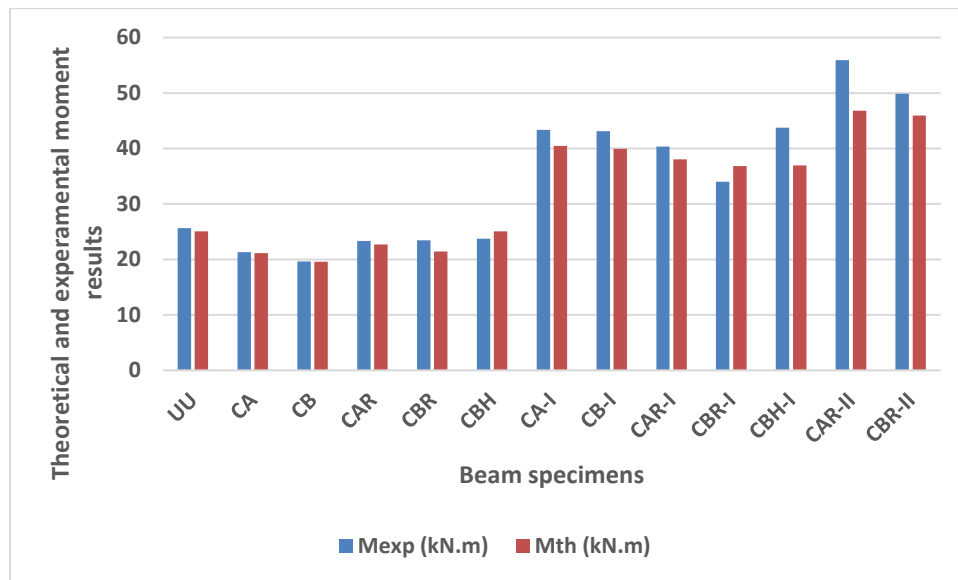


Figure 66: Comparison between analytical and experimental results

As shown in Figure 66, the analytical model could predict the beams' ultimate load capacity with a reasonable error percentage. The majority of the beams strengthened with CFRP showed slightly higher experimental values by an average of 7% due to the efficient anchoring system, especially when the beams were strengthened with two CFRP strengthening layers.

This model can predict the corroded beams' strength to higher corrosion levels and use different material properties.

5.3 Analytical Model of Strengthened Corroded RC Beams with Hybrid System of CFRP and UHPC

This section provides an analytical solution to determine the maximum bending moment at failure for beams strengthened with the hybrid system of CFRP and UHPC. The failure moment was calculated based on the internal forces for a certain cross-section at the equilibrium between the tensile forces and the compressive forces. The material properties that were used in the analytical solution were obtained from the experimental results. A bilinear stress-strain behavior of the steel reinforcement is used where it behaves as elastic perfectly plastic. The UHPC and steel behaviors in tensile are shown in Figure 67.

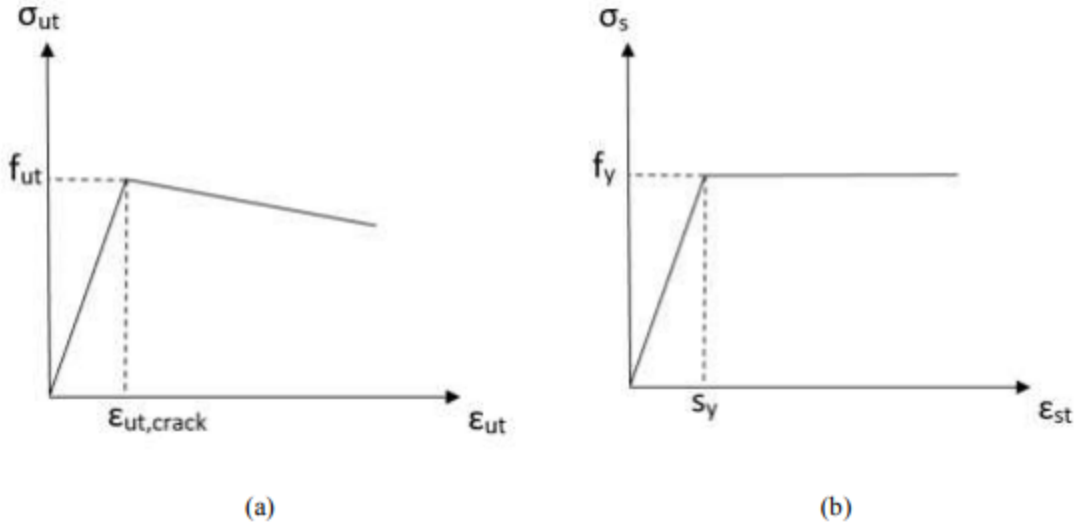


Figure 67: (a) UHPC behavior in tensile. (b) Steel behavior in tensile.

For normal concrete Whitney's stress block was used, while For UHPC, a triangular block was used. Figure 68 shows the analytical model definition.

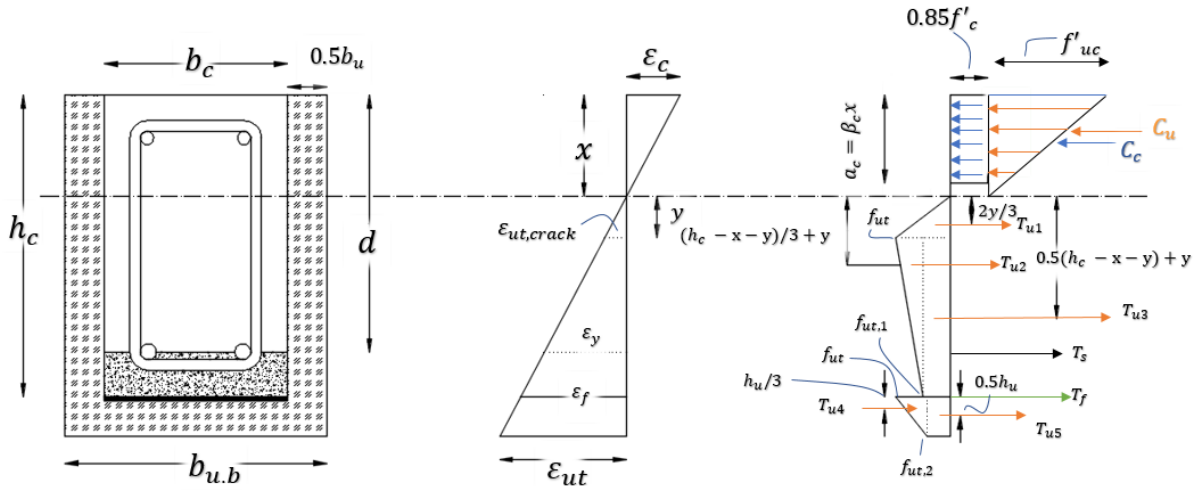


Figure 68: Analytical Model definition

The steps of the calculation are as follows:

Step 1: calculating the new cross-sectional area after the mass loss from the corrosion, where the mass loss was obtained from the gravimetric analysis.

To calculate the cross-sectional area of the corroded bars, Eq. 5.3.1 was used.

$$A_{sc} = A_s * (1 - \alpha)^2 \quad (5.3.1)$$

α = percentage reduction in the bar diameter.

$$\text{Where } \alpha = \frac{2 * P_r * T}{D} \quad (5.3.2)$$

T = corrosion duration (days)

D = original diameter of steel bar (cm)

$$P_r = \frac{W}{F * \gamma_{st}} * I_{corr} = \frac{J_r}{\gamma_{st}} \quad (5.3.3)$$

P_r = penetration rate cm/year by Ijsseling [41]

I_{corr} = corrosion current density mA/cm²

γ_{st} = density of steel = 7.85 g/cm³

J_r = corrosion rate g/cm²/year

W = equivalent weight of steel = 55.85/2 = 27.925 g

F = faraday's constant = 96487 coulombs (Amp-sec)

Step 2: Preparing the forces. The forces are calculated by multiplying the stresses with the cross-sectional area covered by each stress.

$$C_c = 0.85 f'_c \beta_c x b_c \quad (5.3.4)$$

$$C_u = 0.5 f'_{uc} x b_u \quad (5.3.5)$$

Where:

$$\beta_c = 0.85 - 0.05 \frac{f'_c - 28}{7} f'_c > 28 \text{ MPa} \quad (5.3.6)$$

$$T_s = A_{sc} f_y \quad (5.3.7)$$

$$T_{u1} = 0.5 f_{ut} b_u y \quad (5.3.8)$$

$$T_{u2} = 0.5 (f_{ut} - f_{ut,1}) b_u (h_c - x - y) \quad (5.3.9)$$

$$T_{u3} = b_u (h_u - x - y) f_{ut,1} \quad (5.3.10)$$

$$T_{u4} = 0.5 h_u (f_{ut} - f_{ut,2}) b_{u,b} \quad (5.3.11)$$

$$T_{u5} = h_u b_{u,b} f_{ut,2} \quad (5.3.12)$$

$$T_f = \varepsilon_f E_f A_f \quad \text{where} \quad \varepsilon_f = \frac{h_c - x}{x} * (0.003) \quad (5.3.13)$$

The equations from 5.3.4 - 5.3.13 were simplified and expressed in terms of x (depth of the natural axis). The value of x is iterated until the following force equilibrium equation became equal to Zero:

$$\Sigma F = 0$$

$$(T_s + T_{u1} + T_{u2} + T_{u3} + T_{u3} + T_{u4} + T_{u5} + T_f) - (C_c + C_u) = 0 \quad (5.3.14)$$

Finally, the theoretical moment is calculated by taking a moment around the neutral axis, where the x value is obtained after iteration. Each force is multiplied with its moment arm, then by taking the summation as follows:

$$M_{the} = T_s(d - x) + T_{u1} \left(\frac{2y}{3} \right) + T_{u2} \left(\frac{1}{3} (h_c - x - y) + y \right) + T_{u3} \left(\frac{1}{2} (h_c - x - y) + y \right) + T_{u4} \left(h_c + \frac{1}{3} h_u \right) + T_{u5} \left(h_c + \frac{1}{2} h_u \right) + T_f (h_c - x) + C_c \left(x - \frac{a_c}{2} \right) + C_u \left(x - \frac{x}{3} \right) \quad (5.3.15)$$

The procedure for computing the predicted moment capacities of the beams is shown in Figure 69.

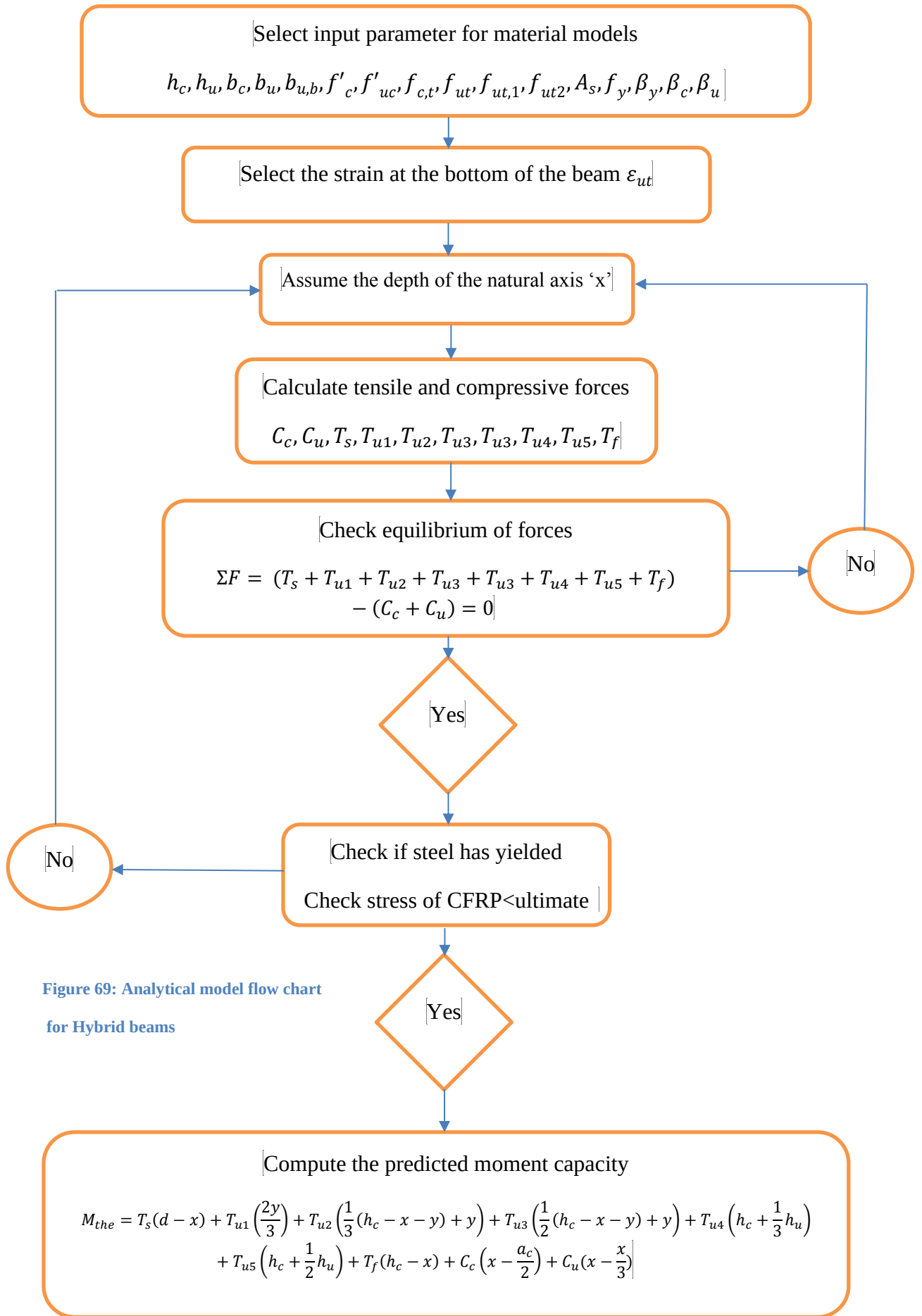


Table 10: Analytical Results of Beam Specimens

Beam	$M_{exp} (kN.m)$	$M_{the} (kN.m)$	Difference (%)
UU	25.65	25.44	-0.80
CA	21.33	21.24	-0.42
CB	19.62	18.72	-4.60
CAR-I-20	50.67	55.7	9.8
CBR-I-20	53.1	54.8	3.3
CAR-II-20	68.67	69.3	0.9
CBR-II-20	66.36	67.8	2.2
CAR-I-30	60.12	63.5	5.6
CBR-I-30	45.75	62.1	35.7
CAR-II-30	78.15	78.5	0.5
CBR-II-30	79.44	77.2	-2.8

Figure 70 shows comparison charts between the analytical and the ultimate experimental moment.

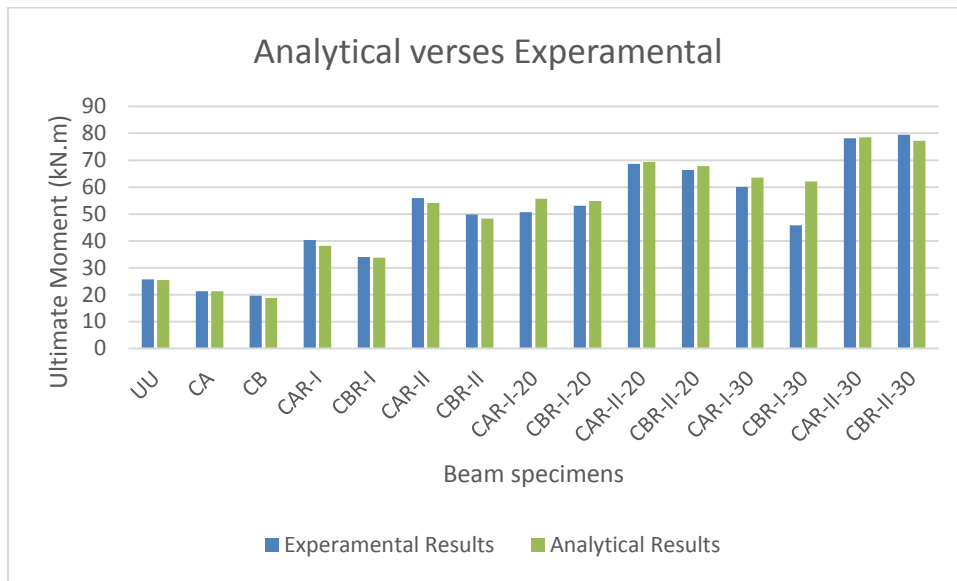


Figure 70: Comparison between the analytical and the experimental results.

As shown in Figure 70, the analytical solution showed an excellent matching with the experimental results with a small percentage of error. In most of the beams, the error does not exceed 5%. As mentioned earlier, the beam CBH-I-30 showed a low experimental

value due to pitting corrosion in one of its bars, so the analytical solution helped predict its potential experimental result.

The model can predict the load-carrying capacity of strengthened RC beams corroded to a higher corrosion level using different hybrid strengthening configurations.

CHAPTER 6

CONCLUSIONS, APPLICATIONS AND RECOMMENDATIONS FOR FUTURE WORK

6.1 Conclusions

The present research work resulted in generation of experimental data pertaining to flexural behavior on corroded RC beams strengthened with a hybrid system of UHPC and CFRP and using different types of cover repairing. Based on the analysis of the experimental results and analytical modeling, the following conclusions can be drawn:

- i. Corroded un-strengthened beams with their original cover showed a reduction in the load-bearing capacity by 17% for 10% corrosion and 23.5% for 20% compared with the control beam (Un-corroded and Unstrengthen beam).
- ii. Corroded beams with only cover repairing without strengthening, showed an increment in ultimate load capacity by 9.5% for 10% mass loss and 19% for 20% mass loss compared with corroded beams without repairing the cover. UHPC showed excellent performance as a cover repairing material. The 20% corroded beam with UHPC cover showed an enhancement in the load-carrying capacity by 21% and an increment in the stiffness by 7.5%.
- iii. Corroded beams strengthened with one layer of CFRP applied on the cover repaired using three types of repairing materials showed the following result: the beams with original cover showed a better performance than the repaired material beam cover.

In contrast, the beam with UHPC cover showed an excellent performance in both the ultimate load and modulus of elasticity with lower ductility by 35% than the beams strengthened over the other two types of covers.

- iv. The repair material showed a good overall flexural performance by maintaining the original beam stiffness, enhancing the load capacity, and high ductility before CFRP failure.
- v. Beams strengthened by one layer of CFRP showed an enhancement of 89% for 10% corrosion and 73% for 20% corrosion with rupture failure mode, while beams strengthened by two layers of CFRP showed an enhancement of 162% for 10% corrosion and 154% for 20% corrosion with debonding failure mode.
- vi. Beams strengthened with two CFRP layers showed lower ductility than beams strengthened with one CFRP layer by 9.5%.
- vii. The corroded beams strengthened with one CFRP layer's hybrid system and 20 mm UHPC jacket showed an average enhancement in the load-bearing capacity and stiffness by 138 and 66% respectively for 10% corrosion, and 170% and 60% respectively for 20% corrosion. On the other hand, the corroded beams strengthened with one CFRP layer's hybrid system and 30 mm UHPC jacket showed an average enhancement in the load-bearing capacity and stiffness by 181 and 84% respectively for 10% corrosion and 133% and 64%, respectively for 20% corrosion. The previous results indicate that using one layer of CFRP and a UHPC jacket of 20 or 30 mm gives the same enhancement provided by using two layers of CFRP.
- viii. The corroded beams strengthened with the hybrid system of two CFRP layers and 20 mm UHPC jacket showed an average enhancement in the load-bearing capacity

and stiffness by 222 and 111% respectively for 10% corrosion, and 238% and 64% for respectively for 20% corrosion. On the other hand, the corroded beams strengthened with the hybrid system of two CFRP layers and 30 mm UHPC jacket showed an average enhancement in the load-bearing capacity and stiffness by 266 and 113% respectively for 10% corrosion and 305% and 100%, respectively for 20% corrosion.

- ix. The hybrid beams strengthened with one CFRP layer (except for CBH-I-30, where pitting corrosion occurred in one of its bars) experienced an average reduction in the ductility by 28% for 10% corrosion level and 40% for 20% corrosion level. On the other hand, the hybrid beams strengthened with two CFRP layers showed an average reduction in the ductility by 20% for 10% corrosion level and 15% for 20% corrosion level.
- x. The beams strengthened with only two CFRP layers failed by CFRP debonding, while the hybrid beams strengthened with two CFRP layers failed by CFRP rupture. The previous information indicates that the UHPC jacket helped shift the failure mode from debonding to rupture, allowing the full utilization of the CFRP strength.
- xi. Before strengthening with CFRP, cover replacement is essential where the RC beam member is corroded to more than 10% loss in the steel diameter.
- xii. The analytical model showed a good prediction of the experimental results with a reasonable percentage of error. The analytical model can predict the strength of beams with higher corrosion levels, which helps decide the CFRP amount required to substitute the loss in the strength due to corrosion.

6.2 Applications

- i. Since the CFRP sheets used for strengthening the beams have debonding problems at high temperature, the hybrid strengthening (using CFRP and UHPC) technique can be used in nuclear and power plants where the elevated temperature is common.
- ii. In case of beams with 10% to 20% loss of mass of the bars due to corrosion, strengthening with a hybrid system of one CFRP layer and 20 mm thick UHPC layer is enough to restore the original strength with an extra enhancement of 90%.
- iii. To prevent CFRP debonding for beams strengthened with two CFRP layers, providing a 20 mm thick UHPC layer will benefit from CFRP's full strength.
- iv. Shear connectors can utilize to obtain the two layers of CFRP's full tensile capacity and avoid debonding to achieve higher load-carrying capacity.

6.3 Future Work

- i. This research was conducted on RC beams; future work is needed to be conducted on other RC members like columns and slabs.
- ii. The flexural behavior of hybrid beams should be investigated under fire load to catch at which degree and duration of temperature exposure the hybrid system will starts to lose its strength.
- iii. Corrosion levels higher than 20% should be investigated to see their effect on hybrid beams.
- iv. Post-corrosion of the hybrid beams should be investigated to see corrosion's effect on the strengthened RC beams.

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