

**STRENGTHENING OF CORRODED REINFORCED
CONCRETE BEAMS USING ULTRA HIGH
PERFORMANCE CONCRETE (UHPC)**

BY

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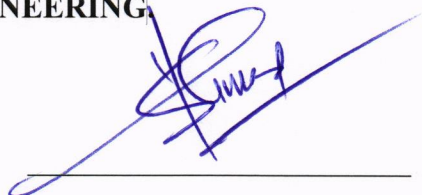


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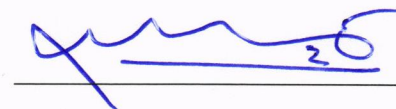


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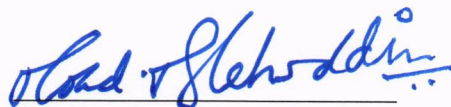
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بِسْمِ اللَّهِ الرَّحْمَنِ الرَّحِيمِ

Dedicated to

*My lovely parents, wife and son for
all their love, support and encouragement. |*

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TABLE OF CONTENTS

ACKNOWLEDGMENTS.....	V
TABLE OF CONTENTS.....	VII
LIST OF TABLES	XI
LIST OF FIGURES	XII
LIST OF ABBREVIATIONS.....	XIX
ABSTRACT	XXI
ملخص الرسالة.....	XXIII
CHAPTER 1 INTRODUCTION	1
1.1 General	1
1.2 Research significance	3
1.3 Objectives.....	4
CHAPTER 2 LITERATURE REVIEW	6
2.1 General	6
2.2 Corrosion of steel reinforcing in RC members	6
2.3 Impressed current technique (ICT)	9
2.4 Strengthening of corroded RC beams	21
2.5 Usage of UHPC as a strengthening material	28
CHAPTER 3 EXPERIMENTAL PROGRAM.....	40
3.1 General	40
3.2 Specimens Details	40
3.3 Materials Properties.....	47

3.3.1 Normal Concrete (NC)	47
3.3.2 Steel Reinforcement.....	51
3.3.3 UHPC	52
3.3.4 Bond between UHPC and Normal Concrete (NC)	55
3.4 Accelerated Corrosion Setup	57
3.5 Preparing the specimens for UHPC casting.....	62
3.6 UHPC strengthening and repairing Techniques	63
3.7 Testing setup and instrumentation.....	65
3.8 Extracting the corroded bars for gravimetric analysis	67
CHAPTER 4 RESULTS AND DISCUSSION	69
4.1 General	69
4.2 Corrosion cracking.....	69
4.3 Gravimetric analysis	72
4.4 Structural behavior of RC Beam Specimens.....	77
4.4.1 Control beams (corroded and uncorroded).....	77
4.4.1.1 Flexural behavior	77
4.4.1.2 Failure mechanisms	81
4.4.1.3 Ductility	82
4.4.1.4 Stiffness.....	83
4.4.2 Strengthened corroded beams with UHPC	84
4.4.2.1 Flexural behavior	84
4.4.2.2 Failure modes.....	88

4.4.2.3 Enhancement percentage of the load-bearing capacity	91
4.4.2.4 stiffness	94
4.4.2.5 Ductility and energy absorption	96
4.4.3 Repaired corroded beams with UHPC	97
4.4.3.1 Flexural behavior	97
4.4.3.2 Failure mechanism	100
4.4.3.3 Enhancement percentage of load-carrying capacities	103
4.4.3.4 stiffness	107
4.4.3.5 Ductility and energy absorption	109
4.4.4 Post-strengthened corroded beams with UHPC	110
4.4.4.1 Flexural behavior	110
4.4.4.2 Failure mechanism	113
4.4.4.3 Enhancement percentage of load-carrying capacities	115
4.4.4.4 stiffness	118
4.4.4.5 Ductility and energy absorption	119
CHAPTER 5 ANALYTICAL SOLUTION	121
5.1 General	121
5.2 Analytical model of the control RC beam specimens	121
5.3 Analytical model of strengthened corrosion-damaged RC beams	126
5.4 Analytical model of repaired corrosion-damaged RC beams	130
CHAPTER 6 FINITE ELEMENT MODELLING	134
6.1 General	134

6.2	Material models	135
6.2.1	Concrete damage plasticity model (CDPM)	135
6.3	Model constraints	139
6.3.1	Tie constraint	139
6.3.2	Embedded Region Constraint	140
6.4	Analysis of FEM	140
6.5	Meshing elements.....	141
6.5.1	3 – D Stress elements.....	141
6.5.2	Truss elements	142
6.6	Results of the finite element modelling.....	143
6.7	Parametric study	157
CHAPTER 7 CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE WORK.....		159
REFERENCES		164
VITAE		173

LIST OF TABLES

Table 2.1: Summary of accelerated corrosion research of RC specimens using ICT.....	18
Table 2.2: Experimental testing parameters for repaired corroded beams with CFRP	25
Table 2.3: Ductal® mix proportion (Lubbers 2003).....	29
Table 2.4: Ingredients quantities to produce one m ³ of the optimum mix of UHPC	30
Table 2.5: Properties of UHPC optimum mixture developed by [92]	30
Table 3.1: Detailed description of test specimens.	43
Table 3.2: Mix design analysis of ordinary concrete.....	48
Table 3.3: Composition of cement and MS (mineral filler).....	53
Table 3.4: UHPC mixture proportion [113].....	54
Table 4.1: Mass losses results of the corroded beams.	75
Table 4.2: Summary of the flexural testing results for all control beams.....	80
Table 4.3: Ductility of all control beam specimens.	83
Table 4.4: Flexural testing results of the strengthened beams.....	86
Table 4.5: Ductility and energy absorption of strengthened beams.....	97
Table 4.6: Flexural testing results of all repaired beams.	99
Table 4.7: Ductility and energy absorption for repaired specimens.	110
Table 4.8: Summary of flexural results for strengthened and post-strengthened beams..	111
Table 4.9: Ductility and energy absorption of post-strengthened beam specimens.	120
Table 5.1: Analytical results of the corroded beam specimens.	124
Table 5.2: Analytical results of the strengthened corroded beam specimens	129
Table 5.3: Analytical results of the repaired corroded beam specimens	133
Table 6.1: Input parameters values for concrete and UHPC to define the CDPM	139
Table 6.2: Parametric study I results (varying UHPC strengths).....	157
Table 6.3: Ultimate loads of parametric study II (increasing corrosion degree)	158

LIST OF FIGURES

Figure 2.1: Delamination and concrete spalling caused by corrosion in the main steel bars and stirrups [3].	7
Figure 2.2: Typical schematic setup of ICT in RC beams [34].	10
Figure 2.3: Typical setup of the ICT for concrete prisms [35].	10
Figure 2.4: Typical setup of ICT for a number of RC prisms connected in series [12].	12
Figure 2.5: Setup for inducing corrosion in large-size RC beams by Azad et al. [54].	15
Figure 2.6: Setup for inducing corrosion in large-size RC beams by AlSaidy et al. [55].	15
Figure 2.7: Setup for inducing corrosion in large-size RC beams by Xie and Hu. [56].	16
Figure 2.8: Details of corroded beams by Elghazy et al. [49]. (All dimensions in mm).	17
Figure 2.9: Setup for inducing corrosion in large-size RC beams by Yuan et al. [29].	17
Figure 2.10: Diagram of corroded beams with several strengthening techniques [82]	22
Figure 2.11: Strengthening procedure [83]	23
Figure 2.12: Details of RC beam and configuration of repaired UHTCC material [85].	24
Figure 2.13: a-1 st method, 10% avg. mass loss. b- 3 rd method 20% avg. mass loss [56].	27
Figure 2.14: C0 represents virgin beam, C15 represents the corroded beam to 15 %, S15 represents strengthened corroded beam, RS15 represents replaced coved by a new one and strengthened corroded beam SP15 strengthened + U-shaped corroded beam [64].	27
Figure 2.15: Strengthened beams have lower deflection and higher load capacities [89].	28
Figure 2.16: UHPC layer strengthened with steel bars [94].	31
Figure 2.17: Beam dimensions and UHPC strengthening configurations [96].	32
Figure 2.18: Strengthening scheme of the bridge deck [97].	33
Figure 2.19: UHPC strengthening Configurations [99].	34
Figure 2.20: Configurations of strengthening RC beam specimens by UHPC in (a) tension side, (b) compression side, and (c) three sides. [100]	35

Figure 2.21: Crack pattern in FEM for the three configurations of strengthening beam specimens with UHPC in (a) tension side, (b) compression side, and (c) three sides. [100].	35
Figure 2.22: Reinforcement pattern and test configuration [102].	36
Figure 2.23: Cracking pattern for composite beams [102].	36
Figure 2.24: Analytical solution model for strengthened RC slab with UHPC [104].	37
Figure 2.25: Specimen and test set-up [105].	38
Figure 2.26: (a) Composite beam setup (b) Force vs. mid-span deflection Curves [106].	39
Figure 3.1: Details of beam specimens (dimensions and reinforcement) unit: mm.	44
Figure 3.2: Schematic procedure of the testing beams (a) Strengthening beams (CSB & CSC); (b) Post-strengthened beams (CPB & CPC).	45
Figure 3.3: Fixing of the electrical wires to the main bars.	46
Figure 3.4: Applying an epoxy coating to the connection portion.	47
Figure 3.5: Concrete casting (a) Casting of beam Specimens (b) Casting of cylinders.	48
Figure 3.6: Steam curing of concrete beam specimens at PRAINSA factory	49
Figure 3.7: RC beam specimens laid inside the KFUPM structural lab	49
Figure 3.8: (a) Compressive strength and elastic modulus test (b) Splitting tensile test.	50
Figure 3.9: Compressive stress-strain curve of NC.	51
Figure 3.10: Steel tensile test	52
Figure 3.11: UHPC ingredients used for strengthening.	52
Figure 3.12: Material test of UHPC: (a) Elastic modulus. (b) Direct tensile test using dog bone. (c) Split tensile test.	54
Figure 3.13: Stress-strain curve for UHPC in compression.	55
Figure 3.14: Stress-strain curve for UHPC from a direct tensile test.	55
Figure 3.15: Split tensile test between UHPC and normal concrete	56
Figure 3.16: Slant shear test between UHPC and CFRP	56
Figure 3.17: DC power source used to induce current to RC beams.	57

Figure 3.18: Positions of RC beams (a) Normal position, (b) Corrosion setup position..	58
Figure 3.19: Schematic drawing of the reinforcement corrosion setup.	59
Figure 3.20: Schematic diagram of sequential electrical connections of the specimens..	60
Figure 3.21: 3D schematic drawing of the reinforcement corrosion setup.....	61
Figure 3.22: Sandblasted beams (after reinforcement corrosion process).....	62
Figure 3.23: Remove concrete cover and cleaning the corroded bars	63
Figure 3.24: Configuration and thicknesses of UHPC layers for strengthened and post- strengthened RC beam specimens	63
Figure 3.25: Configuration and thicknesses of UHPC layers for repaired RC beams.....	64
Figure 3.26: (a) UHPC casting; (b) UHPC curing.....	65
Figure 3.27: Testing setup of RC beam specimens.....	66
Figure 3.28: Testing setup and instrumentation arrangement.....	66
Figure 3.29: Extracting of corroded bars (a) Remove the concrete cover (b) Remove the surrounding concrete from the bars.	67
Figure 3.30: Reinforcement bars after demolishing the specimen (a) Top bars and stirrups (b) Main corroded bars	68
Figure 3.31: Set of corroded bars after being extracted from the beam specimens.....	68
Figure 4.1: Longitudinal cracks of corroded beams.	70
Figure 4.2: Post-strengthened RC beams after corrosion (phase II) (a) Bottom surfaces of beams CPA-20, 30, and 40 (b) Side surface of beam CPB-40	71
Figure 4.3: Corroded steel bars: a) Group A corroded bars, b) Group B corroded bars, and c) Group C corroded bars.	72
Figure 4.4: Preparation of the solution to conduct gravimetric analysis in accordance with ASTM-G1 [53].....	73
Figure 4.5: Change of the gravimetric analysis weight for the beam, i.e., CSB-30	74
Figure 4.6: Theoretical and actual mass losses versus corrosion levels.	75

Figure 4.7: Disparity of mass losses percentage between phase I and phase II for both corrosion groups B and C.....	77
Figure 4.8: Load vs. midspan deflection curves of corroded control specimens.....	78
Figure 4.9: Load vs. midspan deflection plots (a) Present study (b) Azad et al. [54]	79
Figure 4.10: Reduction of ultimate load capacity with a degree of corrosion.....	81
Figure 4.11: Failure mechanisms of beams (a) Control beam UU (b) Beam CUA (c) Beam CUB (d) Beam CUC.	82
Figure 4.12: Stiffness degradation of the corroded beams.	84
Figure 4.13: Load vs. deflection of strengthened group A beams.	86
Figure 4.14: Load vs. deflection of strengthened group B beams.	87
Figure 4.15: Load vs. deflection of strengthened group C beams.	87
Figure 4.16: Failure modes of group A (a) CSA-20, (b) CSA-30, and (c) CSA-40.....	88
Figure 4.17: Failure modes of group B (a) CSB-20, (b) CSB-30, and (c) CSB-40.....	89
Figure 4.18: Failure modes of group C (a) CSC-20, (b) CSC-30, and (c) CSC-40.....	90
Figure 4.19: Ultimate load-bearing capacity of strengthened RC specimens (Group A).	91
Figure 4.20: Ultimate load-bearing capacity of strengthened RC specimens (Group B).	92
Figure 4.21: Ultimate load-bearing capacity of strengthened RC specimens (Group C).	92
Figure 4.22: Variation of the ultimate capacity of strengthened RC beams.	93
Figure 4.23: Stiffness enhancement for the strengthened beams of group A.	94
Figure 4.24: Stiffness enhancement for the strengthened beams of group B.	95
Figure 4.25: Stiffness enhancement for the strengthened beams of group C.	95
Figure 4.26: Stiffness enhancement of strengthened RC beams.....	96
Figure 4.27: Load vs. deflection plots for repaired beams of group A.	99
Figure 4.28: Load vs. deflection plots for repaired beams of group B.	100
Figure 4.29: Load vs. deflection plots for repaired beams of group C.	100
Figure 4.30: Failure modes of group A (a) CRA-20, (b) CRA-40, and (c) CRA-60.....	101
Figure 4.31: Failure modes of group B (a) CRB-20, (b) CRB-40, and (c) CRB-60.....	102

Figure 4.32: Failure modes of group C (a) CRC-20, (b) CRC-40, and (c) CRC-60.....	103
Figure 4.33: Ultimate load-bearing capacities of repaired specimens (Group A).	104
Figure 4.34: Ultimate load-bearing capacities of repaired specimens (Group B).	105
Figure 4.35: Ultimate load-bearing capacities of repaired specimens (Group C).	105
Figure 4.36: Variation of the ultimate capacity of repaired RC beams.	106
Figure 4.37: Stiffness enhancement for the repaired beams of group A.	107
Figure 4.38: Stiffness enhancement for the repaired beams of group B.....	108
Figure 4.39: Stiffness enhancement for the repaired beams of group C.....	108
Figure 4.40: Stiffness enhancement of repaired RC beams.....	109
Figure 4.41: Load-deflection plots for group B specimens (Phase I & II).	112
Figure 4.42: Load-deflection plots for group C specimens (Phase I & II).	113
Figure 4.43: Failure modes of post-strengthened group B beams (a) CPB-20, (b) CPB-30, and (c) CPB-40.....	114
Figure 4.44: Failure modes of post-strengthened group C beams (a) CPC-20, (b) CPC-30, and (c) CPC-40.....	115
Figure 4.45: Enhancement percentage of the strengthened and post-strengthened RC beam specimens (Group B).	116
Figure 4.46: Enhancement percentage of the strengthened and post-strengthened RC beam specimens (Group C).	117
Figure 4.47: Stiffness enhancement for the strengthened and post-strengthened RC beams of group B.....	118
Figure 4.48: Stiffness enhancement for the strengthened and post-strengthened RC beams of group C.....	119
Figure 5.1: Behavior of materials under uniaxial tension: (a) Steel reinforcement, (b) Ordinary concrete and UHPC	122
Figure 5.2: Analytical model definition of control beams.	123

Figure 5.3: Comparison of experimental and analytical ultimate loads for control beams.	125
Figure 5.4: Comparison of experimental and analytical ultimate loads for control beams.	125
Figure 5.5: Analytical model definition of strengthened beams.....	126
Figure 5.6: Analytical model flow chart of strengthened beams.....	128
Figure 5.7: Comparison between Experimental and analytical results for strengthened beams..	129
Figure 5.8: Analytical model definition of repaired beams.	130
Figure 5.9: Analytical model flow chart of repaired beams	132
Figure 5.10: Comparison between experimental and analytical results for repaired beams.	133
Figure 6.1: Stress strain curve of the uniaxial compressive behavior in tension (Abaqus Manual).....	136
Figure 6.2: Stress strain curve of the uniaxial tensile behavior in tension (Abaqus Manual)	136
Figure 6.3: Define of eccentricity and dilation angle in meridian plane	138
Figure 6.4: Yield surfaces in the deviatoric plane	138
Figure 6.5: A 3-D stress 8-noded element type	141
Figure 6.6: Discretization of the concrete beam specimen (a) concrete elements (b) steel bars and stirrups elements	142
Figure 6.7: A 2-D noded linear 3-D truss element	143
Figure 6.8: FEM vs. Experimental results for beam UU.	144
Figure 6.9: FEM vs. Experimental results for beam CUA.	144
Figure 6.10: FEM vs. Experimental results for beam CUB.....	145
Figure 6.11: FEM vs. Experimental results for beam CUC.....	145
Figure 6.12: FEM vs. Experimental results for beam CSA-20.....	146

Figure 6.13: FEM vs. Experimental results for beam CSA-30.....	147
Figure 6.14: FEM vs. Experimental results for beam CSA-40.....	147
Figure 6.15: FEM vs. Experimental results for beam CSB-20.....	148
Figure 6.16: FEM vs. Experimental results for beam CSB-30.....	148
Figure 6.17: FEM vs. Experimental results for beam CSB-40.....	149
Figure 6.18: FEM vs. Experimental results for beam CSC-20.....	149
Figure 6.19: FEM vs. Experimental results for beam CSC-30.....	150
Figure 6.20: FEM vs. Experimental results for beam CSC-40.....	150
Figure 6.21: FEM vs. Experimental results for beam CRA-20.....	151
Figure 6.22: FEM vs. Experimental results for beam CRA-40.....	152
Figure 6.23: FEM vs. Experimental results for beam CRA-60.....	152
Figure 6.24: FEM vs. Experimental results for beam CRB-20.....	153
Figure 6.25: FEM vs. Experimental results for beam CRB-40.....	153
Figure 6.26: FEM vs. Experimental results for beam CRB-60.....	154
Figure 6.27: FEM vs. Experimental results for beam CRC-20.....	154
Figure 6.28: FEM vs. Experimental results for beam CRC-40.....	155
Figure 6.29: FEM vs. Experimental results for beam CRC-60.....	155
Figure 6.30: Experimental and FEM crack pattern for RC-Control beam UU.....	156
Figure 6.31: Experimental and FEM crack pattern for strengthened beam CSB-20.....	156

LIST OF ABBREVIATIONS

- A_{st} : Area of the original cross section of the tensile reinforcing bars. (mm^2)
- A_{sc} : Area of the corroded cross section of the tensile reinforcing bars. (mm^2)
- a_c : Depth of the effective compression block in the concrete beam. (mm)
- b_c : Original width of the concrete beam. (mm)
- b_u : Total width of both sides of UHPC layers. (mm)
- $b_{u,t}$: Total width of the strengthened beams = $b_c + b_u$ (mm)
- y : Depth of the cracking strain measured from the neutral axis. (mm)
- x : Depth measured from the top edge of a beam to neutral axis. (mm)
- h_c : Height of the concrete beam. (mm)
- h_u : Height of the UHPC bottom layers. (mm)
- d : Effective depth from top edge of a beam to center of the tensile bars. (mm)
- T : The sum of tension forces. (kN)
- C : The sum of compression forces. (kN)
- T_s : Tensile force in the tensile bars. (kN)
- C_c : Compression force in normal concrete. (kN)
- C_u : Compression force in UHPC. (kN)

$f_{c'}$: Compressive strength of normal concrete. (MPa)

f'_{uc} : Compressive strength of the UHPC. (MPa)

f_y : Yield stress in tension steel bars. (MPa)

f_{yc} : Yield stress of the corroded tension steel bars. (MPa)

f_{ut} : Cracking stress of the UHPC. (MPa)

$f_{ut,1}$: Tensile strength of UHPC at a distance d . (MPa)

$f_{ut,2}$: Ultimate tensile strength of UHPC at full depth of the beams. (MPa)

E_s : Elastic modulus of reinforcing steel bars. (GPa)

ϵ_s : Actual strain in the reinforcing steel bars.

ϵ_y : Yield strain in steel bars.

β_c : Factor that relates the depth of an equivalent rectangular compressive stress block to the depth of the neutral axis.

M_p : The actual degree of corrosion in the tensile bars. (%).

ABSTRACT

Full Name : [Mohammed Abdulkareem Al-Huri]
Thesis Title : [Strengthening of Corroded Reinforced Concrete Beams Using Ultra High Performance Concrete (UHPC)]
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Reinforcement corrosion in concrete structures is one of the main reasons behind the loss of load-bearing capacities and degradation of the structural performance of reinforced concrete (RC) elements resulting in a significant reduction in the service life of concrete members. Therefore, it is critical to develop new approaches to enhance the service life of damaged RC members by applying long-lasting, efficient, safe, and cost-effective strengthening materials. Strengthening the undamaged RC members using ultra-high-performance concrete (UHPC) has become very popular in recent years. However, the use of UHPC to strengthen the damaged concrete elements due to reinforcement corrosion is hardly reported in the literature. Therefore, the present study aims to investigate the influence of different thicknesses and/or configurations of UHPC layers on structural behavior of corroded RC beam specimens after strengthening/repairing to improve load-bearing capacities. Twenty-eight RC beam specimens were designed, prepared, and cast to study their behavior before and after strengthening with UHPC layers. A new setup for inducing reinforcement corrosion (based on the concept of impressed current technique) was developed ensuring natural pattern of the occurrence of reinforcement corrosion. One uncorroded RC beam specimen was tested as a control specimen. The remaining twenty-seven RC beam specimens were subjected to reinforcement corrosion using the developed

setup to cause targeted degrees of reinforcement corrosion. Three different reinforcement corrosion levels: 10, 20, and 30% (mass loss due to induced reinforcement corrosion) were considered as medium, severe, and very severe corrosion levels, respectively. Each group contained seven RC beam specimens to cause the same degree of reinforcement corrosion. In addition, six RC beam specimens were used to study the post-strengthened performance against reinforcement corrosion. For this, the specimens were corroded to a certain degree and then strengthened using different thicknesses of UHPC layers, after that the specimens were further subjected to corrosion. Two different strengthening configurations were used, including three sides and one bottom side using UHPC layers. For each strengthening configuration, three different thicknesses of UHPC layers were used. Layer thicknesses of 20, 40, and 60 mm were used in the case of one-sided repair and layer thicknesses of 20, 30, and 40 mm were used in the case of three-sided strengthening. All the strengthened and un-strengthened beam specimens were tested in flexure to evaluate the effect of UHPC strengthening layers on the flexural performance of the corroded RC beam specimens using the load-deflection data recorded during flexural testing. Moreover, mechanistic/analytical modelling was carried out to predict the ultimate load-carrying capacities of the corroded RC beams strengthened with UHPC using different sets of configurations and thicknesses. The models were validated using the experimental data generated under the present work. In addition, finite element modelling (FEM) was performed using ABAQUS software to capture the structural performance of corroded beam specimens strengthened/repared using the UHPC layers. The FEMs were validated with the results obtained from the experiments and were used to carry out various parametric studies. The experimental, analytical, and numerical results showed high degree of agreement.

في الحياة الواقعية. تم اختبار جسر واحد من الجسور المصبوبة بدون أحداث أي اضرار بها وبدون تقوية وتم تصنيفها كعينات تحكم أساسية. تم تعريض الجسور السبعة والعشرين المتبقية للتآكل المتسارع باستخدام تقنية التيار القسري المسلط لإحداث درجات معينة من فقدان الكتلة في قضبان حديد التسليح. تم تضمين ثلاثة مستويات تآكل مختلفة (10، 20، 30٪) في الدراسة والتي تمثل تآكل متوسط وشديد جداً. تحتوي كل مجموعة على سبع عينات من الجسور ليتم تأكلها بنفس الدرجة من مستوى التآكل. تم استخدام ست عينات من الجسور المصبوبة لدراسة الأداء المعزز بعد التآكل بسبب التآكل التالف من خلال تأكلها أولاً إلى درجة تآكل معينة وبعد ذلك تقويتها باستخدام UHPC ومن ثم إعادتها إلى نظام التآكل المتسارع مرة أخرى. تم استخدام تكوينات مختلفة في نظام التقوية بما في ذلك تقوية الجوانب الثلاثة أو تقوية جانب واحد فقط من الأسفل. كما تم استخدام سماكات مختلفة لكل تكوين تقوية بما في ذلك 20 و 40 و 60 ملم في حالة التقوية من جانب واحد و 20 و 30 و 40 ملم في حالة التقوية ثلاثية الجوانب. تم اختبار جميع الجسور التي تمت أو لم تتم تقويتها باستخدام اختبار الانحناء لتقدير سلوك تقوية UHPC على الجسور المتأكلة باستخدام بيانات انحراف الحمل المسجلة أثناء اختبار الانحناء. علاوة على ذلك، تم تطوير النماذج الميكانيكية / التحليلية والتحقق من صحتها من خلال التجارب للتنبؤ بقدرة الحمل على الجسور المتأكلة والتي تمت تقويتها باستخدام UHPC. بالإضافة إلى ذلك، تم إجراء نمذجة العناصر المحدودة (FEM) باستخدام برنامج ABAQUS لتقييم ودراسة سلوك الجسور الخرسانية المتأكلة والتي تمت تقويتها باستخدام UHPC. تم التحقق من صحة النماذج FEMs من خلال النتائج التجريبية وكذلك استخدمت هذه النماذج لإجراء دراسات بارامترية مختلفة. أظهرت النتائج التجريبية والتحليلية والعديدية وجود توافق كبير فيما بينها مع نسبة خطأ صغيرة.

CHAPTER 1

INTRODUCTION

1.1 General

Concrete is a vital material that is used to construct many types of structures around the world. The reinforcement steel was used to take care of the tensile stress in the concrete, which is characterized as a brittle material. Reinforced Concrete (RC) structural elements display a perfect response in terms of mechanical properties, durability, as well as economy. However, during the service life, the RC structures, if exposed to the aggressive exposure condition, suffer from many problems such as corrosion of steel reinforcement that causes significant loss in the load-bearing capacities leading to failure of reinforced concrete structures before completion of its targeted service life. Therefore, the strengthening of the partially damaged RC members is highly desirable for restoring the design load-bearing capacity to avoid the failure of concrete elements. Besides load-bearing capacity enhancement, the layers of strengthening material applied on the exposed surfaces of the RC members provide resistance against penetration by the aggressive species preventing the deterioration in the post-strengthened period.

Strengthening the structural members using CFRP sheets has been widely reported as an approach to strengthen the partially damaged members. However, in recent years, Ultra-High-Performance Concrete (UHPC) emerged as a new concrete material that many researchers consider as an alternative material for strengthening the partially damaged RC structures. UHPC possesses high strength (tensile strength over 8 MPa, compressive

strength above 150 MPa, flexural tensile strength above 30 MPa), very high modulus of elasticity (above 40 GPa), high ductility due to the presence of fibers and excellent durability characteristics (negligible permeability) due to its very dense homogenous microstructure. The densification of the microstructure of UHPC is achieved by using a very high dosage of cementitious materials, very fine sand particles, very low water/cementitious materials ratio, and a high dosage of superplasticizer and steel fibers. The UHPC, besides its excellent strength and durability, possesses a high bond strength that makes the use of UHPC advantageous for enhancing the load-carrying capacities, durability, and overall serviceability of the present structures. Due to self-compactability and the high tensile and compressive strength of UHPC mixtures, the strengthening can be carried out by applying the UHPC layers on the faces (tensile or compressive) of normal concrete members in different thicknesses and configurations.

In the current study, an experimental investigation was conducted to investigate the flexural performance of the corrosion-damaged RC beams before and after strengthening them using UHPC to investigate the effects of strengthening the beam in terms of improvements in the load-carrying capacities and other associated parameters. The key factors influencing degrees of reinforcement corrosion and the effectiveness of UHPC strengthening were considered as the experimental variables. These variables will include the density and period of the induced current to cause accelerated reinforcement corrosion, strengthening configuration, and the thickness of the UHPC layers applied on the faces of the RC beams as strengthening material. The experimental data were utilized to develop mechanistic and empirical models to determine the ultimate load-carrying capacities of the corroded RC beams strengthened with UHPC in different configurations and thicknesses. Furthermore,

finite element modeling using ABAQUS software was performed to capture the flexural performance of strengthened RC beam specimens. These models were utilized in performing parametric studies of the strengthened corroded beams by varying UHPC strength and degrees of the corrosion level.

1.2 Research significance

Until now the focus of researchers is to explore the possibility of using UHPC material to strengthen/repair the undamaged RC beams. There is a lack of sufficient information reported in the literature on the use of UHPC to strengthen partially damaged RC beam specimens. Considering the fact that most of the existing RC structures found in Saudi Arabia are severely corroded due to aging and exposure to aggressive conditions, it is critically essential to investigate the possibility of strengthening the corrosion-damaged concrete beams using UHPC. The strengthening effectiveness of corroded RC beams will differ from that of undamaged RC beams due to the difference in the flexural performance of the corroded and un-corroded beams. Therefore, a study to evaluate the behavior and response of corroded RC beam specimens strengthened using UHPC should be conducted considering the key factors.

In the current research study, a new corrosion setup was developed to induce accelerated corrosion in the steel reinforcement bars for large-size RC beams to simulate the realistic corrosion process. To date, there are no data available in the literature to implement different degrees of corrosion, different thicknesses, and different configurations of UHPC layers to strengthen the corroded beam specimens to examine the flexural behavior of beam specimens. In addition, the post strengthened mechanism of using UHPC to prevent and/or reduce the effect of continuation of corrosion process was investigated as corrosion-

damaged beams could be subjected to the same deterioration mechanism and/or corrosive environments after strengthening. Also, to date, there are no data in the literature regarding the post-strengthened behavior of UHPC strengthening corroded beams. Moreover, integrated mechanistic and analytical models were adopted to verify the performance of strengthened corroded beam specimens. Furthermore, finite element modelling was conducted in ABAQUS to examine and validate the experimental results of the corroded beams as an attempt toward developing a new approach to carry out various parametric studies.

1.3 Objectives

The main objective of conducting this work is to explore the behavior of strengthening/repairing the corrosion-damaged RC beam specimens using layers of UHPC at different configurations and thicknesses to enhance the flexural behavior and load-carrying capacities of the corroded beams.

The previous main objective can be achieved by addressing the following sub-objectives:

- 1 To develop a setup for inducing different degrees of reinforcement corrosion in large-size RC beam specimens that will simulate the real-life natural reinforcement corrosion.
- 2 To investigate the effect of strengthening, carried out by applying UHPC layers on the concrete faces in different configurations, on the flexural behavior of RC beams subjected to different degrees of reinforcement corrosion.
- 3 To investigate the post-strengthened effect of induced current on the corroded RC beams after strengthening with different thicknesses of UHPC layers.

- 4 To develop and validate analytical models to predict the ultimate load-bearing capacities of the corroded beams strengthened/repaired with different thicknesses of UHPC material.
- 5 To develop and validate finite element models to capture the flexural performance of strengthened/repaired corroded RC beams using ABAQUS software.

CHAPTER 2

LITERATURE REVIEW

2.1 General

In this chapter, an extensive detailed literature review is presented to highlight information pertaining to the techniques used for inducing accelerated corrosion in RC members and the various strengthening strategies used to repair and enhance the load-bearing capacity of the corroded beam specimens. In addition, the influence of using UHPC mixtures to strengthen the RC beams and their structural performance after UHPC strengthening is also presented. Emphasis is given to the information about recent research and developments on the strengthening of corroded and uncorroded RC beams using UHPC material.

2.2 Corrosion of steel reinforcing in RC members

The reinforcement corrosion of steel bars is one of the greatest significant issues affecting the performance of reinforced concrete (RC) structural members. When corrosion initiates, the steel bars dissolve into the surrounding area as the passive film is degraded locally. The depassivated regions expand as a consequence of raising the amount of rust. When the corrosion products become solid, they have a larger volume than the initial metal, which causes a high pressure around the embedded steel bars. As a result, the concrete cover expands, potentially causing cracks and delamination of the corroded member, as illustrated in [Figure 2.1](#) [1–4]. In addition, the corrosion process has multiple interconnected effects: reduction in the area of steel bars cross-section due to the formation

of rust and degradation of steel–concrete bonding. The severity of reinforcement corrosion impacts the RC structures, including flexural strength, ductility, bond behaviour, and failure mode. These phenomena compromise the structural integrity of concrete members and could cause them to fail prematurely [5–8]. Because of the importance of this problem, an enormous amount of research was carried out to study the influence of corrosion on the structural behaviour of RC elements.

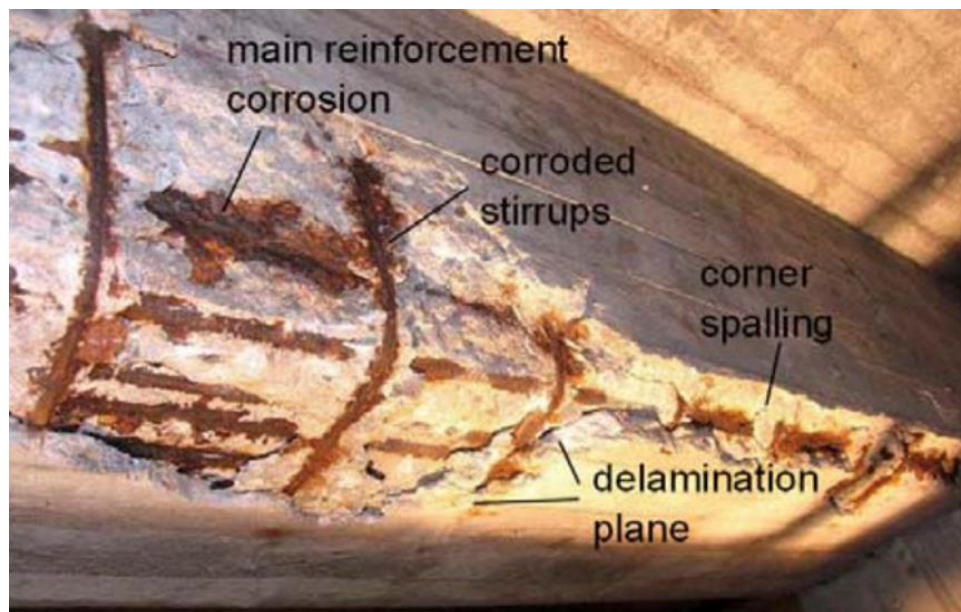


Figure 2.1: Delamination and concrete spalling caused by corrosion in the main steel bars and stirrups [3].

Reinforcement corrosion is a very gradual process in natural conditions that depend on the availability of several factors such as moisture, humidity, oxygen, aggressive conditions, and concrete quality. It was found that the rate of corrosion in low-quality concrete is higher than that in high-quality concrete [9]. Normally, the first corrosion crack on the concrete surface could take several years to be observed. Thus, the corrosion process must be accelerated in laboratory investigations to accomplish experimental tests in a short

duration. Therefore, a constant potential can be used to achieve this aim [10,11]. The impressed current approach is commonly used to accelerate the corrosion process in reinforcing bars [12–14]. This is done in order to cause severe corrosion in steel bars inside concrete elements in an acceptable amount of time. The electrochemical potential is applied between reinforcing bars acting as an anode and a cathode to cause corrosion. Previous research investigations have used current densities typically varied from 100 to 2000 $\mu\text{A}/\text{cm}^2$ [15–20], which is substantially higher than the corrosion rates measured in the field for normal environmental conditions, which is around 10 $\mu\text{A}/\text{cm}^2$ [21,22]. Andrade et al. [31] measured the maximum corrosion rate to be between 100 and 200 $\mu\text{A}/\text{cm}^2$ in very severe environmental conditions. Almusallam et al. [23] reported the maximum corrosion current density used in research work to be 10,400 $\mu\text{A}/\text{cm}^2$, while the minimum corrosion current density was reported by Pedrosa and Andrade [24] to be 5 $\mu\text{A}/\text{cm}^2$.

There is a variation between actual and theoretical corrosion mass loss for several reasons, such as the demand for electrical energy to commence corrosion, bar composition, concrete resistivity, and the electrical characteristics of minerals in concrete [25]. Liu et al. [26] referred to the variation of the mass loss to the decrease in the rust production rate as the corrosion process progresses. Maaddaawy and Soudki [12] evaluated the impact of various current densities on responses of concrete strain produced by corrosion products. They reported that raising the density of induced current above 200 $\mu\text{A}/\text{cm}^2$ could cause a significant increase in the strain reaction and width of cover cracking. Also, they found that changing the current density to achieve various degrees of reinforcement corrosion over the same amount of time can cause unintended consequences that might lead to misinterpretation of the test results [12].

There are other alternative strategies reported in the literature for speeding up steel corrosion in concrete members. Chloride induced corrosion method has been considered one of the most popular prevalent degradation techniques in concrete structures subjected to marine environmental conditions or aggressive weathering [27,28]. Yuan et al. [29] presented an innovative approach to causing rapid reinforcement corrosion using a controlled temperature condition. Ha, et al. [30] employed a microcell corrosion approach with an impressed voltage technique. Hussain et al. [31] employed AC impedance methods to detect the failure of passive layers and evaluate the reinforcing bars' effectiveness inside concrete in a considerably short amount of time. Chunlei et al. [32] have investigated the performance of corrosion inhibitors very rapidly by accelerating the chloride ion diffusing electrically over the surfaces of embedded reinforcing bars inside concrete elements to induce reinforcement corrosion. Another method for accelerating reinforcement corrosion was developed by Yang et al. [33] that is built on electrochemical technique to accelerate chloride ions migration in the cement-based materials to evaluate their penetrability using accelerated chloride migration test (ACMT).

2.3 Impressed current technique (ICT)

The impressed current technique (ICT) has been employed to induce reinforcement corrosion in RC structures for the purpose of completing research experiments in an acceptable amount of time. The main principle of the ICT contains applying a continuous current from a DC power supply to reinforcing bars embedded inside concrete members making it the anode to cause a considerable degree of corrosion damage. A counter electrode (cathode) is connected to a negative terminal to represent an inverted cathodic protection (CP) system. The impressed current causes corrosion in the reinforcement steel

bars while the counter electrode (cathode) is protected. The typical schematic setup of the impressed current method in RC beams is illustrated in Figure 2.2. The sodium-chloride solution (NaCl) acts as an electrolyte to allow current to pass from the cathode to the anode. Figure 2.3 shows the typical setup used to induce accelerated corrosion in prisms adopting the ICT method.

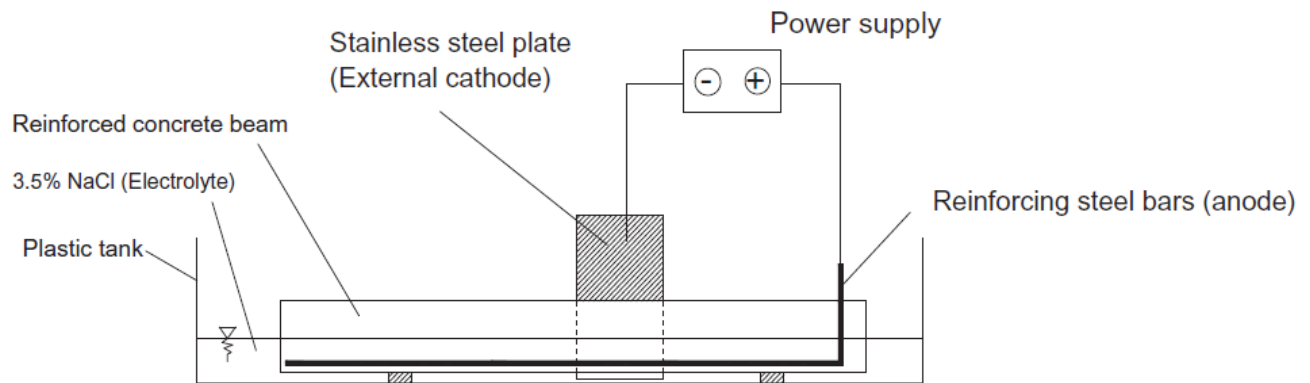


Figure 2.2: Typical schematic setup of ICT in RC beams [34].

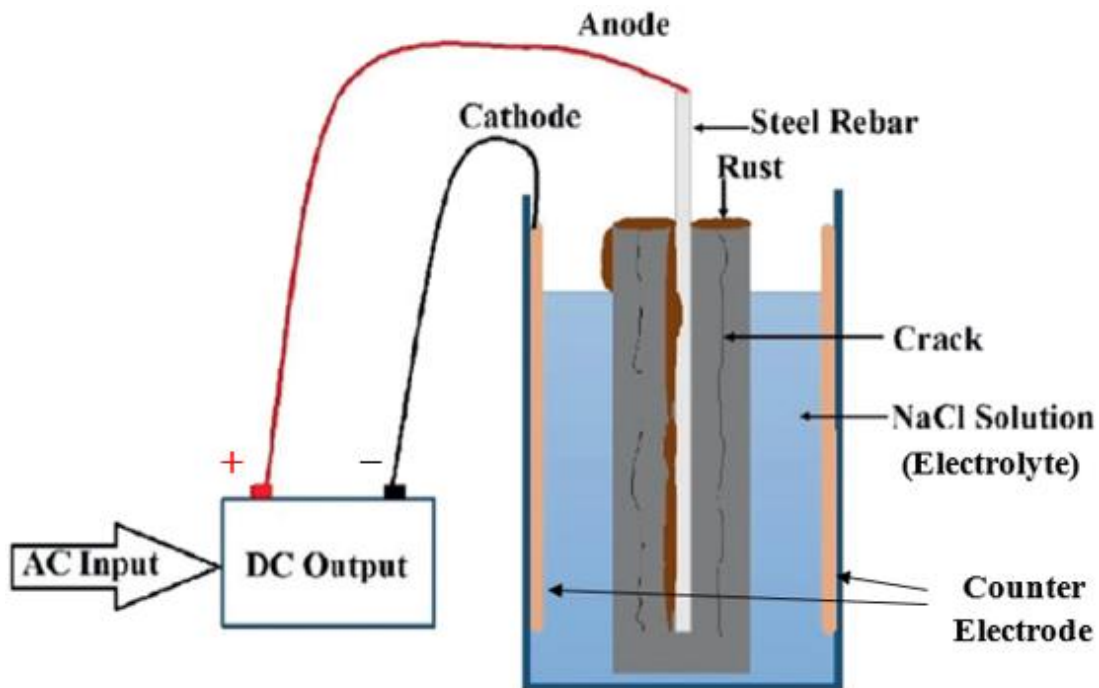


Figure 2.3: Typical setup of the ICT for concrete prisms [35].

The impressed current method was widely utilized to investigate the impact of corrosion on the cracking of concrete covers, bonding of steel bars, Residual flexural strength, and load-bearing capacity of RC structural elements [14,23,25,36–44]. The impressed current technique for accelerated corrosion provides several features besides saving money and time. The capability to control the corrosion rate, which generally differs because of changes in the concrete resistivity, contents of oxygen, and temperature, is an advantage over other corrosion accelerated approaches [45]. The experiment of accelerated corrosion using the impressed current method has been proven as a suitable method for studying the process of reinforcement corrosion in RC members and its impact on concrete cover damage [14]. There is a strong scientific explanation for utilizing the impressed current approach to accelerate reinforcement corrosion in RC members. It decreases the required initiation time for steel depassivation from several months to a few days and fixes the desired corrosion rate without compromising the certainty of generated rust [46]. Concrete coating, cladding, sealants, and coatings on reinforcing steel could be investigated in a fairly brief amount of time because of the flexibility of impressed current method [47]. Ahmed et al. [48] exploited the impressed corrosion method to assess the corrosion resistance of RC elements by implementing extra cementitious materials such as furnace slag, silica fume, and fly ash. In addition, The corrosion behavior of reinforcing bars in fly ash cement concrete was assessed using impressed current approach [30].

Several specimens can be connected in series to the DC power source to be corroded simultaneously. The number of specimens connected in series to the DC power source is based on DC equipment's voltage capacity. Researchers connected several sequential specimens in series to induce corrosion in their reinforcing bars for the same degree of

corrosion [49–52]. Figure 2.4 shows the corrosion setup developed by Maaddaawy and Soudki [12] to accelerate corrosion in numerous RC prisms linked in series.

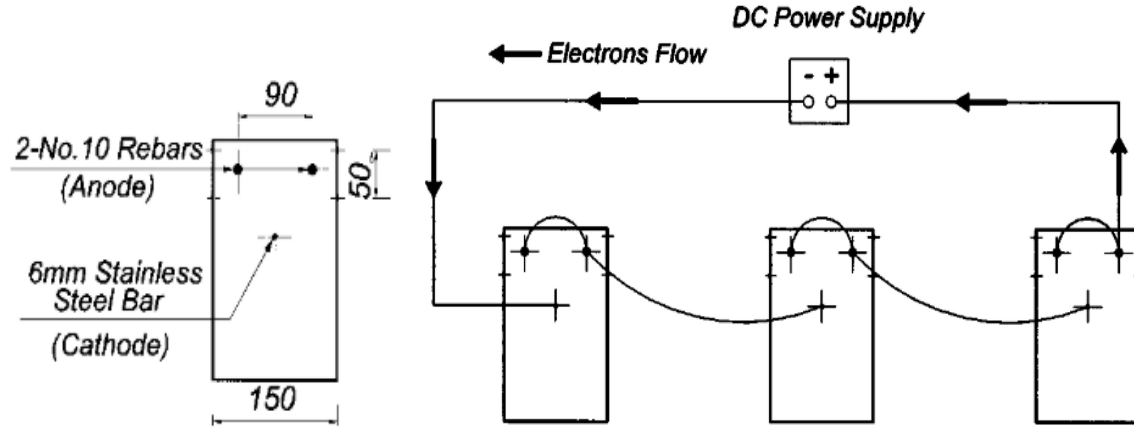


Figure 2.4: Typical setup of ICT for a number of RC prisms connected in series [12].

The theoretical degree of reinforcement corrosion can be estimated by implementing Faraday's law (Eq. 2.1) for a given duration and current intensity.

$$m = \frac{I t a}{n F} \quad (\text{Eq. 2.1})$$

where: m is the mass loss in (gm), I is impressed corrosion current in (A), t is duration of the corrosion process in (s), a is the atomic mass of steel which is equal to 55.847 gr for steel bars, n is number of electrons transported during corrosion process which is equal to 2 for steel bars, and F is Faraday's constant equal to 96487 C / mol. (Eq. 2.1) is simplified for steel reinforcing calculations (Eq. 2.2) to calculate the theoretical mass loss m_{th} as follows:

$$m_{th} = 25 I t \quad (\text{Eq. 2.2})$$

where: m_{th} in (gm), I in (A), and t in (days).

The actual mass loss m_{act} because of reinforcement corrosion can be calculated using a gravimetric analysis conducted on the extracted corroded bars in accordance with ASTM - G1 [53], as explained in (Eq. 2.3).

$$m_{act} = m_i - m_f \quad (\text{Eq. 2.3})$$

where: m_i is initial mass of the steel bars in (gm) and m_f is final mass of the steel bars obtained from gravimetric analysis in (gm)

The impressed current can be calculated by equating (Eq. 2.2) and (Eq. 2.3) assuming that actual and theoretical mass loss are equivalent (for instance, $I_{app} = I_{corr}$).

$$I = \frac{m_i - m_f}{25 t} \quad (\text{Eq. 2.4})$$

The corrosion current density I_{corr} can be calculated using (Eq. 2.5).

$$I_{corr} = \frac{I}{A_{st}} \quad (\text{Eq. 2.5})$$

where: I_{corr} is corrosion current density in (A/cm²), I is induced current in (A), and A_{st} is surface area of the steel bars in (cm²) = $\pi D L$. D and L are the original diameter and length of the reinforcement bars before corrosion, respectively. The corresponding corrosion current density can be calculated by implementing the actual values of reinforcing mass lost due to accelerated corrosion by implementing (Eq. 2.4) into (Eq. 2.5).

$$I_{app} = \frac{m_i - m_f}{25 \pi D L t} \times 10^6 \quad (\text{Eq. 2.6})$$

where: I_{app} is equivalent corrosion current density in (μA/cm²).

A DC power supply, counter electrodes, and electrolyte substance are utilized in setups to introduce reinforcement corrosion through the impressed current technique (ICT). The counter electrode is mainly used to close the electrical circuits in the cell. It is typically made of inactive material such as copper or stainless steel and is usually not involved in the electrochemical reaction. The reinforcing bars (anode) are linked to the positive terminal of a DC power source, while the counter electrode is connected to the negative terminal (cathode). The electrolyte is a substance that undergoes ionization when dissolved in water. It plays an important role for the corrosion process to complete because it allows the current to pass from the counter electrode to the reinforcing bars through concrete. Usually, sodium chloride (NaCl) solution is used as an electrolyte substance to induce corrosion setups and to allow current to pass through the electrical circuit.

Same setup components can be employed for the corrosion of large-size beam specimens. [Figure 2.5](#) illustrates the setup used by Azad et al. [54] to corrode steel reinforcing bars in large size specimens. Stainless-steel plates were used as counter electrodes, and 5% NaCl solution was used as the electrolyte substance. The same approach was adopted by Al-Saidy et al. [55] to induce corrosion into the beam specimens but replace the stainless steel plates with several stainless steel bars, as shown in [Figure 2.6](#). Xie and Hu. (2013) [56] coated the bottom surfaces of the beams with a copper meshing, then connected the copper mesh to the negative terminal of the DC source to act as cathode while steel bars connected to the positive terminal, as illustrated in [Figure 2.7](#).

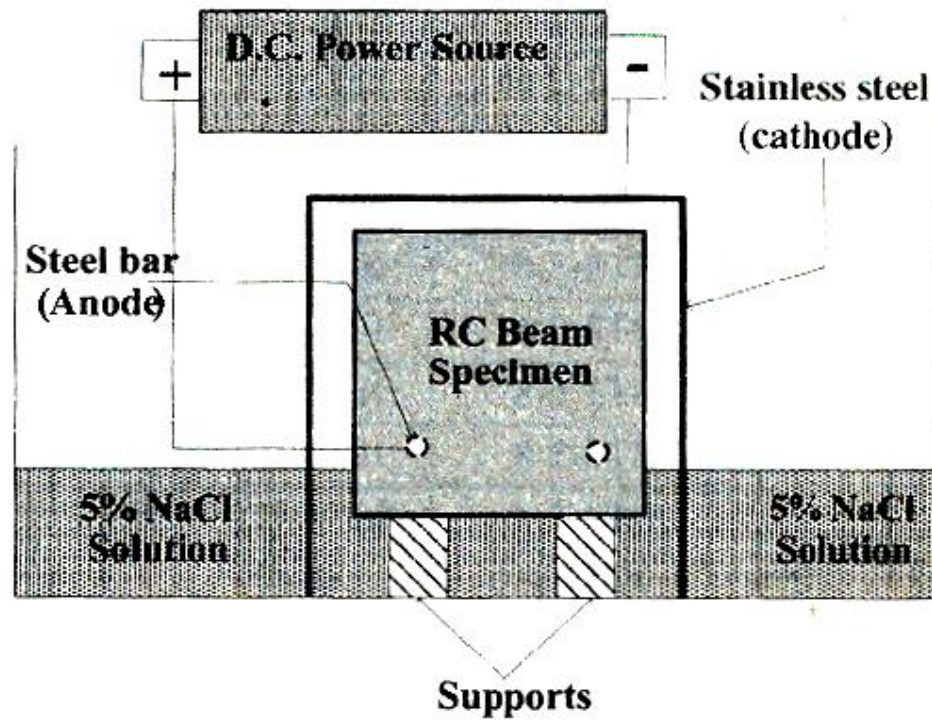


Figure 2.5: Setup for inducing corrosion in large-size RC beams by Azad et al. [54].

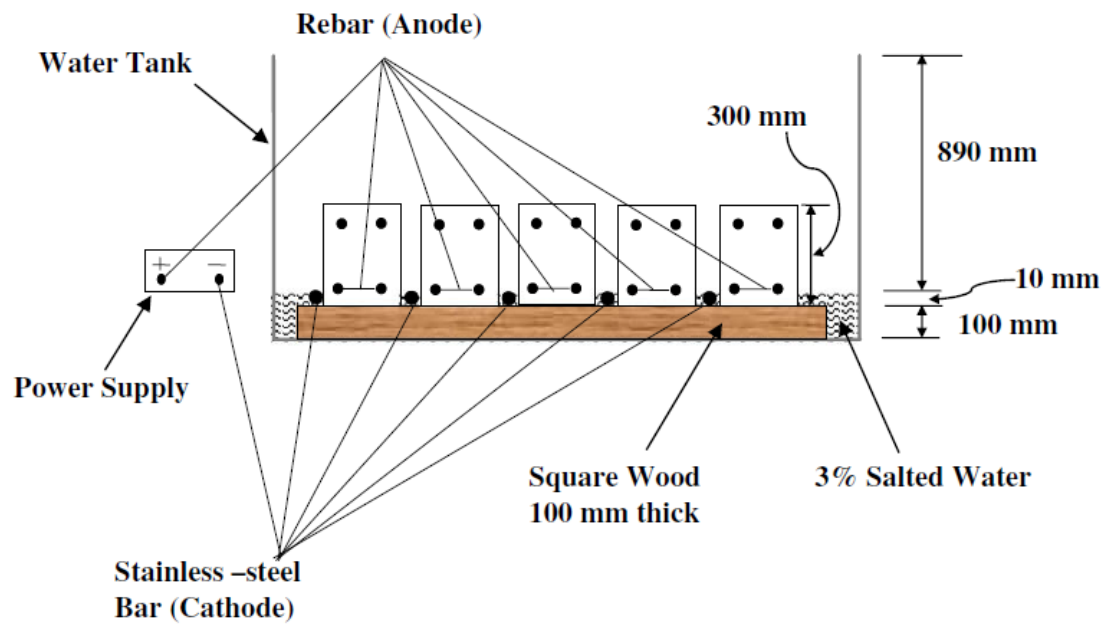
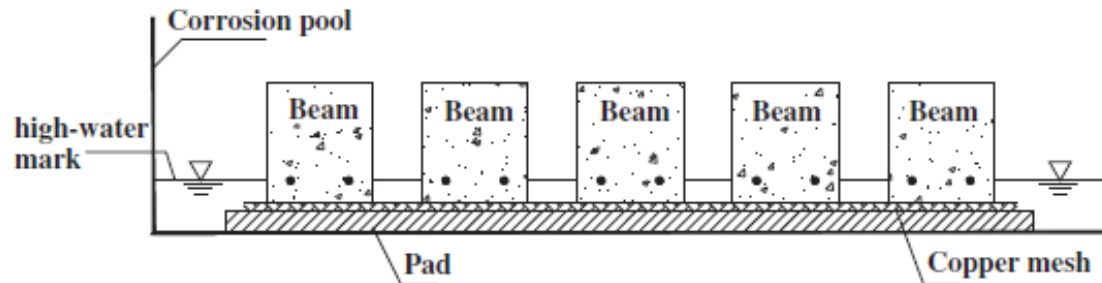
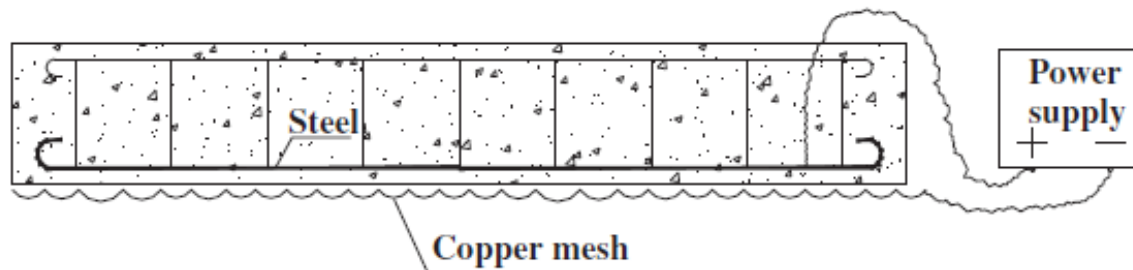


Figure 2.6: Setup for inducing corrosion in large-size RC beams by Al-Saidy et al. [55].



Schematic diagram of corrosion pool



Circuit of accelerated corrosion

Figure 2.7: Setup for inducing corrosion in large-size RC beams by Xie and Hu. [56].

In the previous corrosion setups, it was noticed that the counter electrodes and the electrolyte substance were placed outside the RC specimens. However, there is a corrosion setup where both counter electrodes and the electrolyte substance are placed inside the specimens during the concrete casting of the specimens. Elghazy et al. [49] cast and corrode several beam specimens using the impressed current technique by placing stainless steel tubes inside the specimens and adding salted concrete at the portion of desired corrosion to act as an electrolyte, as shown in Figure 2.8. Another alternative setup developed by Yuan et al. [29] in which they applied the same concept of placing the stainless-steel bar inside the beam during casting but without adding salt to the concrete. Then, the beams were placed in a tank containing NaCl solution to act as electrolyte through the process of reinforcement corrosion, as illustrated in Figure 2.9.

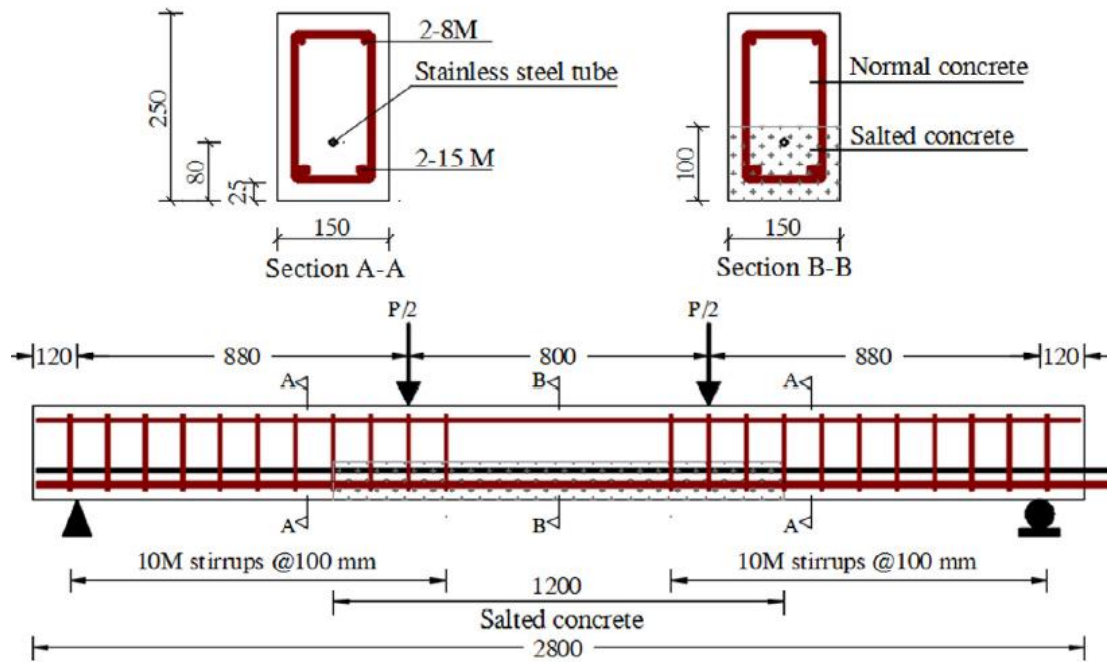


Figure 2.8: Details of corroded beams by Elghazy et al. [49]. (All dimensions in mm).

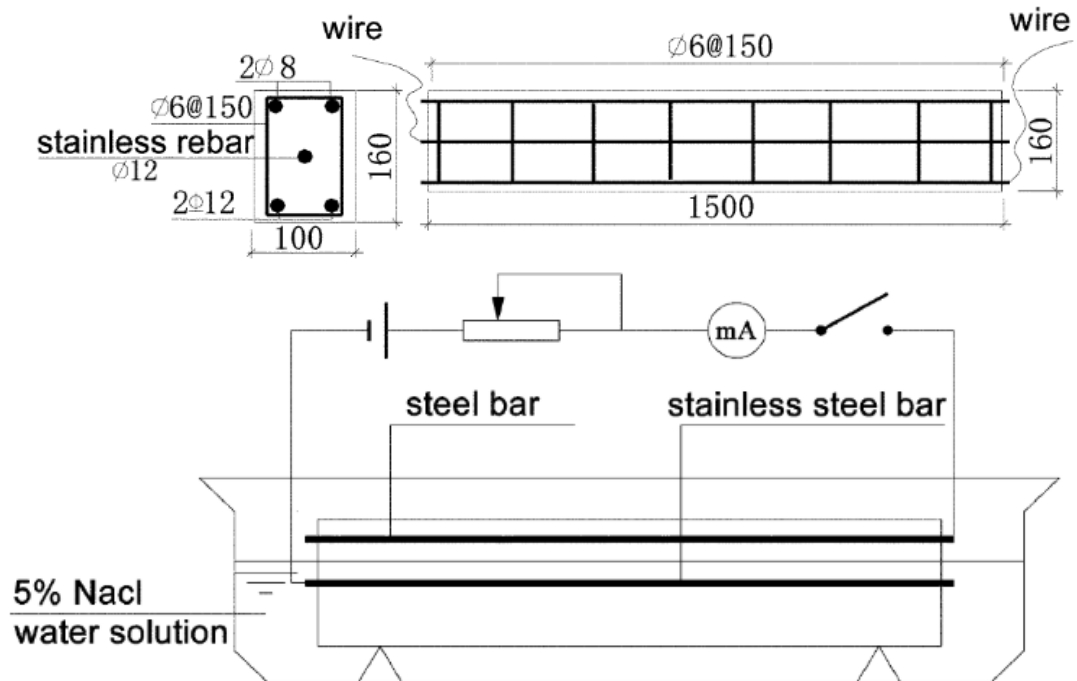


Figure 2.9: Setup for inducing corrosion in large-size RC beams by Yuan et al. [29].

Various studies were found in the literature on the induced corrosion of the RC members using the ICT method. Different RC members were corroded and summarized in this study,

including small- and large-sized specimens such as beams, columns, and prisms. The current density values used for corrosion varied from 5 to 4000 $\mu\text{A}/\text{cm}^2$. Different cathode types, Corrosion environments, and wet-dry cycles were used to induce corrosion to the reinforcing bars. The location of the counter electrode within the corrosion setup and the concentration and technique of the electrolyte substance were also varied. [Table 2.1](#) summarizes earlier accelerated corrosion research studies for RC specimens using the ICT method.

Table 2.1: Summary of accelerated corrosion research of RC specimens using ICT.

<i>Study Reference</i>	<i>Type of specimen</i>	<i>Density of Current ($\mu\text{A}/\text{cm}^2$)</i>	<i>Type of Cathode</i>	<i>Corrosion environmental conditions</i>	<i>wet-dry cycles</i>
El Maaddawy & Soudki (2003) [12]	Prism	100, 200, 350, & 500	embedded stainless steel bar	5 % NaCl of the cement weight added during concreting	Continuous wetting of wrapped burlap sheets
Badawi & Soudki (2005) [57]	Beam	150	embedded stainless steel bar	2.25 % NaCl of the cement weight added during concreting	Continuous RH close to 100%
El Maaddawy & Soudki (2005) [52]	Beam	150	embedded stainless steel tube	2.25 % NaCl of the cement weight added during concreting	Water mist was continuously sprayed over the specimens
El Maaddawy et al. (2005) [58]	Beam	215 mA*	embedded stainless steel tube	2.25 % NaCl of the cement weight added during concreting	Water mist was continuously sprayed over the specimens
Masoud & Soudki (2006) [59]	Beam	140	embedded stainless steel bar	2.15 % NaCl of the cement weight	2.5 days wet followed by one day dry

				added during concreting	
Azad et al. (2007) [54]	Beam	2000 and 3000	External stainless steel plate	2 % NaCl of the cement weight added during concreting	Partial immersed in the 5% NaCl solution
Yuan et al. (2007) [29]	Beam	From 1000 to 2000	embedded stainless steel plate	immersed in 5 % NaCl solution for 168 hours before corrosion	T = 40 C°, RH = 80% and 5% NaCl solution spraying one hour every eight hours
El Maaddawy et al. (2007) [60]	Beam	150	embedded stainless steel tube	2.25 % NaCl of the cement weight added during concreting	Water mist was continuously sprayed over the specimens
Caré & Raharinaivo (2007) [14]	Prism	100 & 500	External carbon fiber mesh	Immersed in two types of solution 1. alkaline solution 2. alkaline solution with NaCl	Totally immersed in the solutions
Al-Saidy et al. (2010) [55]	Beam	207	External stainless steel bar	3 % NaCl of the cement weight added during concreting	Partial immersed in 3% NaCl solution
Gadve et al. (2010) [61]	Beam	300 mA*	External stainless steel mesh rolled	immersed in 3.5% NaCl solution for one day	Partial immersed in the solution
Maalej et al. (2010) [62]	Beam	the constant potential of 12 V**	External wire mesh	immersed in 3% NaCl solution	84 hours wet followed by 84 hours dry
Ray et al. (2011) [63]	Beam	178	embedded stainless steel tube	5 % NaCl of the cement weight added during concreting	partially sheltered with wetted burlap and plastic sheet only for five days per week

Al-Saidy & Al-Jabri (2011) [64]	Beam	487 mA*	External stainless steel bar	1 % NaCl of cement weight spread at the level of main reinforcing bars	Partial immersed in 3% NaCl solution
Xie & Hu (2012) [56]	Beam	-	External copper mesh	Partial immersed in 2% NaCl solution	One day wet followed by two days dry
Hariche et al. (2012) [65]	Beam	150	External steel rod located under the beams	immersed in 3 % NaCl solution	Partial immersed in 3 % NaCl solution
Azam & Soudki (2012) [66]	Beam	150	embedded stainless steel tube	2.3 % NaCl of the cement weight added during concreting	Water mist was continuously sprayed over the specimens
El Maaddawy et al. (2013) [67]	Beam	300 mA*	embedded stainless steel tube	3 % NaCl of the cement weight added during concreting	Water mist continuously sprayed over specimens
Azad & Al-Osta (2014) [68]	Column	2500	External stainless steel plate	Sack act as an electrolyte wetted continuously	Water was continuously sprayed over the specimens
El-Maaddawy & El Refai (2015) [69]	T- beam	200	embedded stainless steel tube	3 % NaCl of the cement weight added during concreting	48 hours wet followed by 48 hours dry
Elghazy et al. (2017) [49]	Beam	180	embedded stainless-steel tube	5 % NaCl of the cement weight added during concreting	72 hours wet followed by 72 hours dry
Pedrosa and Andrade (2017) [24]	Prism	From 5 to 500	External stainless steel mesh	CaCl ₂ .2H ₂ O was added to the concrete matrix	Continuous wetting of wrapped sponge cloth

Fu et al. (2018) [70]	Prism	250	embedded stainless-wire	3.5 % NaCl of the cement weight added during concreting	immersed in 3.5 % NaCl solution for 48 hours before corrosion
Bicer et al. (2018) [71]	Beam	Range of (0-10) A*	External copper plate	immersed in 3.5 % NaCl solution	Totally immersed in the solution
Zhang et al. (2019) [72]	Prism	50, 100, 200, & 300	External copper plate	immersed in 5% NaCl solution	Partial immersed in the solution for 14 days
Hong et al. (2020) [13]	Prism	305 & 500	External silver wrapped sheet	immersed in 3.5 % NaCl solution for 6 hours before corrosion	Continuous RH close to 100%
Hong et al. (2020) [73]	Prism	20, 75, 150, 300, & 600	External silver wrapped sheet	Cotton act as an electrolyte wetted regularly using 3.5% NaCl solution	Continuous RH close to 95%
Radhi et al. (2020) [74]	Column	50, 200, & 500	External stainless-steel mesh	immersed in 3.5 % NaCl solution for 3 days before corrosion	1 day wet of 3.5 NaCl solution followed by 3/6 days dry

T is temperature, RH is relative humidity *The current was reported in the study.

** The voltage was reported in the study.

2.4 Strengthening of corroded RC beams

A comprehensive assessment of the existing experimental and analytical methods in the relevant literature relating to corrosion of RC beams and various strengthening materials and techniques was carried out in Ref. [75,76]. Several traditional methods were used to repair and strengthen the corroded RC structural elements, such as carbon fiber reinforced polymers (CFRPs) [52,57,77,78], fiber-reinforced polymers (FRPs) laminates [79,80], and fabric-reinforced cementitious matrix (FRCM) composites [49,81].

A new calculation technique to estimate the ultimate load-carrying capacities of corroded RC beam specimens repaired/strengthened using different techniques was established by Yuan et al. [82]. Figure 2.10 illustrates the schematic plot of the corrosion-damaged RC beam specimen and the corresponding strengthening techniques used in the study. To strengthen the beam specimens, the authors used steel plates at the bottom of corroded specimens, while U-shape plates with steel bars were used to strengthen the shear zone region. The result of the experiments shows the significant impact of reinforcement corrosion on decreasing the mechanical properties of reinforcement bars, weakening the bonding strength with concrete, and increasing the development of cracking, leading to the collapse of the beams. In addition, the impact of corrosion damage on the area of steel bars has a significant effect on the ultimate load-bearing capacities, deformations, and collapse mechanism of corroded beam specimens.

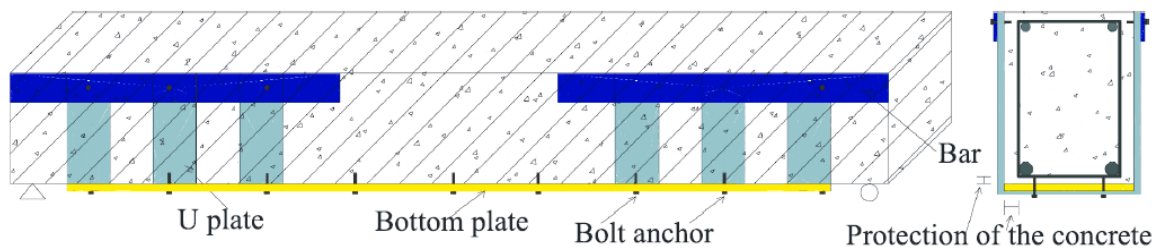


Figure 2.10: Diagram of corroded beams with several strengthening techniques [82]

Chen et al. [83] examined the flexural behavior of using a hybrid system of UHPC and FRP composites for shear strengthening of corrosion-damaged RC beam specimens. Figure 2.11 illustrates the schematic and complete process of the strengthening technique. Results of experiments show that shear strengthening of corroded beams using a hybrid system of UHPC–FRP greatly enhanced the shear capacity of beam specimens and inhibited the formation of shear fractures. Using narrow FRP meshes in UHPC significantly reduced

crack width in concrete elements. In addition, there was no debonding occurred between UHPC and FRP sheets.

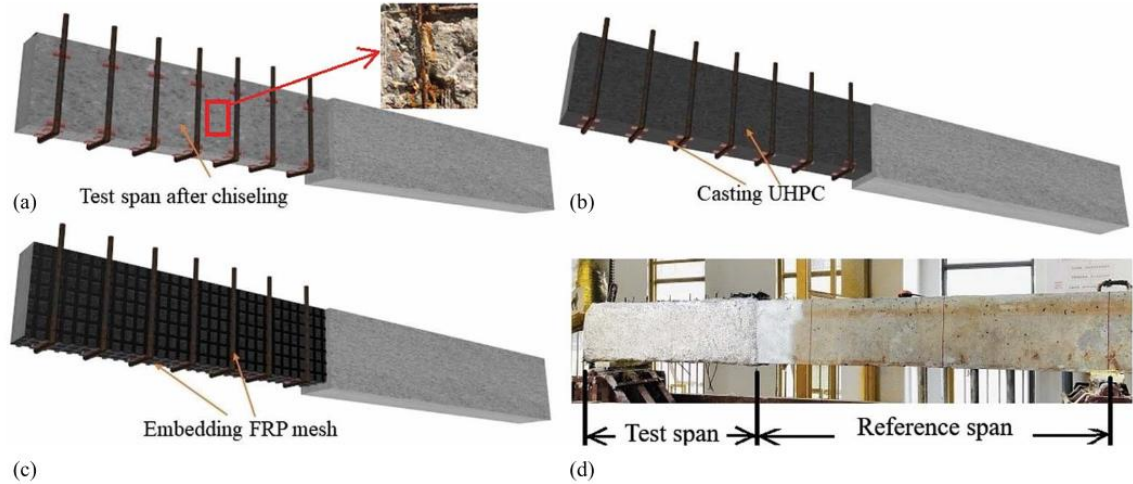


Figure 2.11: Strengthening procedure [83]

The shear performance of concrete beams with corroded stirrups that were repaired using laminates of FRP sheets considering the influence of span to depth ratios was examined by M.ASCE at el. (2020) [84]. Results show the significant impact of shear depth to span ratio on the performance of beam specimens strengthened by externally bonded reinforcement fiber reinforced polymer (EBR FRP) composites. It can even cause a shear collapse mechanism of the concrete beams. It was also found that the high corrosion level of the stirrups could severely reduce the shear capacity of both strengthened and un-strengthened RC beams.

Hou et al. [85] studied the analytical and experimental examination of the flexural behavior of corroded RC beams repaired by layers of ultra-high toughness cementitious composite (UHTCC), as illustrated in Figure 2.12. Reinforcement corrosion was conducted to three different degrees of mass loss, including 5, 10, and 15 %. The UHTCC material has been

implemented to replace damaged concrete of corroded beam specimens leading to an 11.2% recovery of the load-bearing capacities of the beams. Furthermore, an analytical model was introduced to calculate the flexural capacities of repaired corroded beams, considering the effect of pitting corrosion.

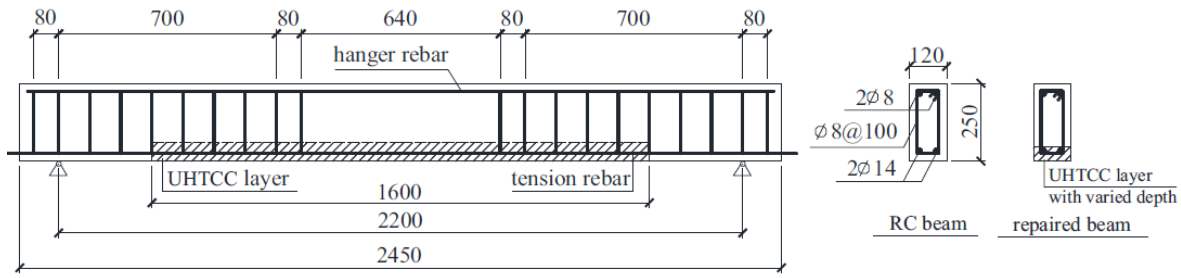


Figure 2.12: Details of RC beam and configuration of repaired UHTCC material [85].

Elghazy et al. [81] studied the numerical and the experimental results of corroded beam specimens repaired using externally bounded composites in flexural. The study included various degrees of corrosion mass loss and various layers of composite FRP and FRCM. Results of the numerical modelling and the experimental testing showed that both FRCM and FRP composites could enhance/restore the load-carrying capacities of beams.

Triantafyllou et al. [86] conducted experimental testing to examine the efficiency of patch repairing using FRP bonded laminates for retrofitting corroded RC beam specimens. Variation of the mass losses of main bars was 7.5 and 24 %. The experimental results and load vs. deflection plots revealed that corrosion damage on the load-bearing capacities and bonding strength between steel bars and concrete was harmful to a high corrosion damage level.

Triantafyllou et al. [76] reviewed the available analytical and experimental methods in the literature related to corrosion of RC beams and various strengthening materials and

regimes. Table 2.2 illustrates a brief summary of parameters and experimental tests of the strengthened corrosion-damaged beams in the international literature. In addition, he introduced a model related to bar diameter, depth of concrete cover, crack width, and mechanical characteristics of the concrete, for evaluating reinforcement bar weight loss.

Table 2.2: Experimental testing parameters for repaired corroded beams with CFRP [76]

Reference	Corrosion process		Method	Material	Type
	Type	Time			
Badawi & Soudki (2005) [30]	accelerated	— Further corroded after repair	EBR	CFRP	Laminate Sheets
El-Maaddawy & Soudki (2005) [27]	accelerated	—	EBR	CFRP	laminate
Masoud & Soudki (2006) [32]	accelerated	Further corroded after repair	EBR	CFRP	Sheets
Soudki et al. (2007) [33]	accelerated	Corroded after repair		GFRP CFRP	Laminate Sheets
El Maaddawy et al. (2007) [37]	accelerated	Further corroded after repair	EBR	CFRP	Sheets
Deng et al. (2009) [42]	accelerated	—	EBR	AFRP	Sheets
Hu et al. (2010) [28]	accelerated	—	EBR	CFRP	Sheets
Al-Saidy et al. (2010) [29]	accelerated	—	EBR	CFRP	Sheets
Gadve et al. (2010) [31]	accelerated	Further corroded after repair	EBR	CFRP GFRP	Sheets
Malumnela et al. (2011) [38]	natural	—	EBR	CFRP	Laminate Sheets
Ray et al. (2011) [36]	accelerated	Further corroded after repair	EBR	CFRP	Sheets
Kreit et al. (2011) [40]	natural	—	NSM	CFRP	rod
Al-Saidy & Al-Jabri (2011) [34]	accelerated	—	EBR	CFRP	Sheets
Xie & Hu (2012) [35]	accelerated	—	EBR	CFRP	Sheets
El Maaddawy et al. (2013) [43]	accelerated	—	Powder-actuated fasteners	Hybrid carbon & glass fiber composite with a vinylester matrix	
Almassri et al. (2014) [41]	natural	—	NSM	CFRP	rod
El Maaddawy & El Refai (2015) [44]	accelerated	—	EBR Internally embedded	Carbon/Basalt Fabric-Reinforced Cementitious Matrix	

Triantafyllou et al. [87] examined numerically and experimentally the behavior of four corroded beam specimens with a mass loss of 7.5 % and retrofitted with outwardly bonded EBR and/or NSM-FRP laminates. Strengthened beams with EBR-FRP showed an ultimate load of 22.1 % greater than the virgin un-strengthened specimen.

Almassri et al. [88] carried out a numerical and experimental study to study the structural behavior of using the near-surface mounted reinforcement method (NSM) to strengthen RC beam specimens subjected to reinforcement corrosion. In this method, rods of CFRP were located in the pre-cut grooves and attached to concrete using an appropriate epoxy adhesive. Besides studying the corrosion of reinforcement steel bars in the RC beams, failure modes, ultimate, and yielding capacities have also been discussed. Results illustrated the significant improvement in the ultimate performance of the uncorroded and corroded beam specimens strengthened using the NSM techniques.

Xie and Hu [56] corroded thirty-two RC beam specimens to various degrees of mass loss percentages, ranging between 3 and 78 %. Then, the beams were strengthened using three different techniques. First, keep the original corroded cover and cure it with putty filler and other curing material, then place CFRP on the cover. Second, replace the damaged cover with polymer concrete mortar after de-rusting, then place the CFRP on the new cover. 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). The result of experiments shows the ability to use the first method for RC beams having a mass loss less than or equal to 15 %. Results concluded that the more the quantity of CFRP laminates, the more the de-bonding possibility. [Figure 2.13](#) shows the load-deflection curves for specimens strengthened with two various methods.

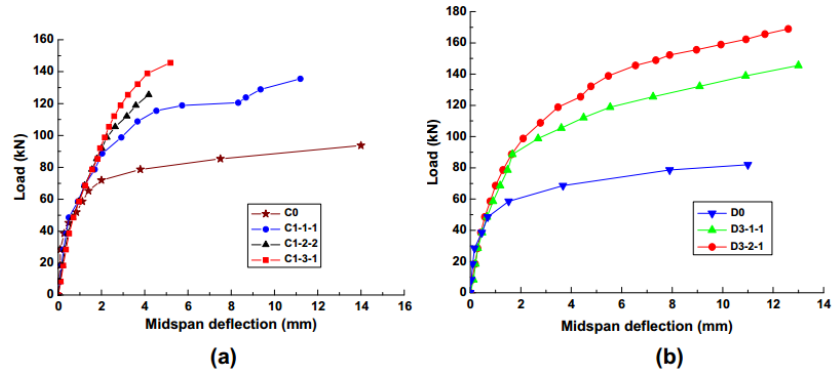


Figure 2.13: a-1st method, 10% avg. mass loss. b- 3rd method 20% avg. mass loss [56].

Al-Saiidy and Al-Jabrii [64] investigate the performance of replacing the original cover of corrosion-damaged beam specimens strengthened using CFRP laminates. Moreover, the behavior of beam specimens without replacing the original concrete cover was also examined. The replacement of the concrete cover with a material having a compressive strength of 40 MPa increased the load resistance by 13% compared to beams strengthened directly on the original cover. The effect of the U-shaped transverse reinforcement was also examined. Experimental results show that using the U-shaped transverse CFRP reinforcement has increased the beam's ductility as well, as shown in Figure 2.14.

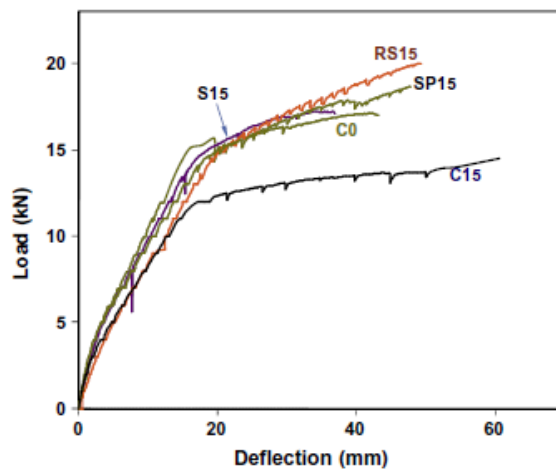


Figure 2.14: C0 represents the virgin beam, C15 represents the corroded beam to 15 %, S15 represents the strengthened corroded beam, RS15 represents replaced covered by a new one and strengthened corroded beam SP15 strengthened + U-shaped corroded beam [64].

El Maaddaawy and Sooudki [89] examined the possibility of bonding CFRP laminates to strengthen corroded beams' external cover. Accelerated corrosion using an electric DC power source and three different corrosion levels were implemented in the study. 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 induced corrosion. The result shows the improvement in the maximum strength of corroded beams after strengthening using CFRP. CFRP strengthened beams showed lower deflection values and higher loads values than the corroded un-strengthened beam specimens, as illustrated in Figure 2.15.

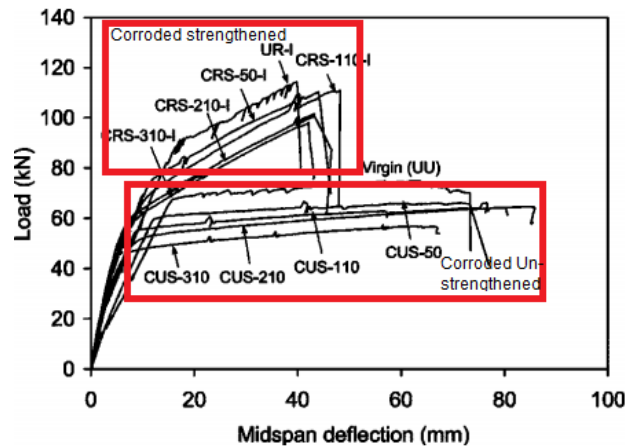


Figure 2.15: Strengthened beams have lower deflection and higher load capacities [89].

2.5 Usage of UHPC as a strengthening material

UHPC emerged in the recent decade as a new concrete material that many researchers consider as an alternative material for strengthening partially damaged reinforced concrete structures. UHPC possesses very high strength (flexural tensile strength above 30 MPa, tensile strength over 8 MPa, and compressive strength above 150 MPa), very high modulus of elasticity (above 40 GPa), high ductility due to the presence of fibers and excellent durability characteristics (negligible permeability) due to its very dense homogenous

microstructure. The densification of the microstructure of UHPC is achieved by the use of a very high dosage of cementitious materials, very fine sand particles, a very small amount of water/cementitious materials ratio, and a high dosage of superplasticizer and steel fibers. The UHPC, besides its excellent strength and durability, possesses a high bond strength that makes the use of UHPC advantageous for enhancing the load-bearing capacity, durability, and overall serviceability of the existing structures. Due to self-compactability and the high tensile and compressive strength of UHPC mixtures, the strengthening can be carried out by applying the UHPC layers on the faces (tensile or compressive) of normal concrete members in different configurations. Lubbers et al. [90] reported that the compressive strength of UHPC might extend to 2 to 3 times and flexural strength two to six times higher than that of the HPC. UHPC was first launched as a patented concrete material with a brand name of Ductal® consisting of the four ingredients that included Premix, water, superplasticizer, and steel fibers. Mix proportions of the Ductal® concrete are illustrated in Table 2.3. Hakeem et al. [91] carried out an examination on the characterization of Ductal® concrete mixture. He found that the Ductal® concrete has a compressive strength reaching up to 162 MPa and a modulus of elasticity of 56 GPa. The result has shown the values of flexural tensile strength could increase from 15.3 MPa with 0% fibers to 24.5 MPa with only 3 % fibers and 31.5 MPa with 6 % fibers.

Table 2.3: Ductal® mix proportion (Lubbers 2003)

Ductal Mix Component	Weight (kg/m³)	Percent by weight
Premix	2202	87.2
Water	136	5.4
Superplasticizer	30	1.2
Steel Fibers	157	6.2

Ahmed et al. [92] developed an optimum mix of the UHPC using the materials locally available in Saudi Arabia. Experimental tests were conducted on the samples which the selected mix of the UHPC has prepared to determine the durability characteristics and mechanical properties. The quantities of the ingredients to produce one cubic meter of UHPC mix are illustrated in Table 2.4. The performance of the UHPC mix which was developed by Ahmad et al. [92], is presented in Table 2.5.

Table 2.4: Ingredients quantities to produce one m3 of the optimum mix of UHPC [92]

Ingredient	Quantity (kg)
Water	163
Cement	900
Micro-silica	220
Fine dune sand	1005
Plasticizer-Glenium® 51	40
Steel fibers	157
All together	2485

Table 2.5: Properties of UHPC optimum mixture developed by [92]

Property	Unit	Value	Remarks
Flow	mm	230	More than the minimum requirement
28-day compressive strength	MPa	161	More than 3 times as compared to high strength concrete
28-day flexural strength	MPa	31	More than 4 times as compared to high strength concrete
28-day stress intensity factor	MPa $\sqrt{m}$	8	More than 7 times as compared to high strength concrete
Drying shrinkage (after 150 days)	Micro-strain	450	Less than for high strength concrete
Water absorption	%	.43	Less than 10 times as compared to high strength concrete
Water permeability	mm	0	Negligible water permeability
Rapid chloride permeability	Coulomb	9	Negligible chloride permeability
Electrical resistivity	k Ω -cm	180	Low to negligible likelihood of active reinforcement corrosion
Loss of compressive strength after 6 months of exposure to sulphate solution	%	7	Negligible loss due to sulphate attack
Average corrosion current density, I_{corr}	$\mu A/cm^2$	.0383	Around 8 times less than the threshold value of I_{corr} for active reinforcement corrosion
Maximum current passing during accelerated corrosion test	mA	.21	Around 8 times less than that for high strength concrete under same applied constant voltage

Al-Osta et al. [93] conducted numerical and experimental examinations to investigate the structural performance of RC columns strengthened using UHPC. Loading eccentricity and UHPC thickness were considered the main variables in the study. In addition, 3D finite element modelling was established using ABAQUS software. The obtained results from the numerical models matched the experiments. Eventually, analytical analysis was performed to estimate the ultimate loads and moments for the UHPC-strengthened columns. Results showed the efficiency of using UHPC for strengthening the RC columns under eccentric loads.

Zhang et al. [94] examined the flexural behavior of damaged beam specimens strengthened using a reinforced UHPC overlay. Research results illustrated that the UHPC layer has enhanced the flexural behavior of beams and prevented cracks from developing through the specimen. The reinforced UHPC layers can significantly reduce the RC beam cracking, causing fracture width to propagate slowly with a load. Figure 2.16 illustrates the strengthening configuration of strengthened beams.

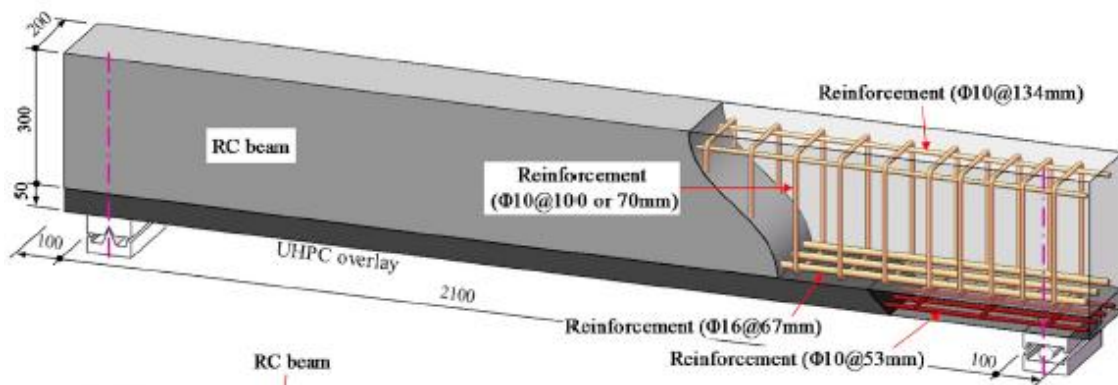


Figure 2.16: UHPC layer strengthened with steel bars [94].

Al-Osta et al. [95] examined the shear behavior of beam specimens retrofitted using precast panels of UHPC using an appropriate epoxy adhesive material. Two different configurations were used for strengthening: (i) retrofitting three sides, including the bottom faces, and (ii) retrofitting only two sides. The strengthened beams showed a significant enhancement in the load-bearing capacities, stiffnesses, and failure modes. Moreover, 3-D (FEM) using ABAQUS software was established to evaluate the ultimate loads and overall behavior of strengthened beam specimens.

Bahraq et al. [96] studied the influence of using jacketing made of UHPC on the shear behavior of beam specimens. The UHPC was cast in situ over rough surfaces prepared by sand blasting of the beams. The bond between the two concrete was studied by conducting slant shear and splitting tensile strength tests. The strengthening configurations of the beam specimens are illustrated in Figure 2.17. Results of the strengthened beams illustrated the efficiency of retrofitting the beam specimens by layers of UHPC, which enhanced the performance of the shear capacity of the beams.

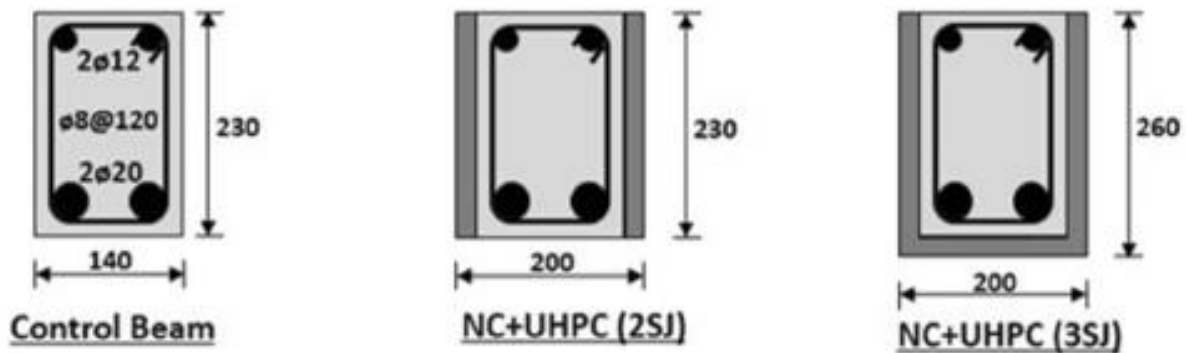


Figure 2.17: Beam dimensions and UHPC strengthening configurations [96].

Zhang et al. [97] carried out a very large experimental investigation on damaged bridge decks made of concrete and strengthened using layers of UHPC at various configurations to study their behavior under flexural failure, as illustrated in Figure 2.18. The experiments were carried out on the bridge decks in positive and negative moments, including placing the UHPC layers in compression or tension zones. By placing the UHPC layers in the tension zone, the cracking and composites' ultimate capacities have enhanced by 250 and 200 %, respectively. Nevertheless, placing the UHPC layer in the compression zone has enhanced the maximum load-carrying capacities of concrete members by 30%, with negligible changes in the cracking pattern.

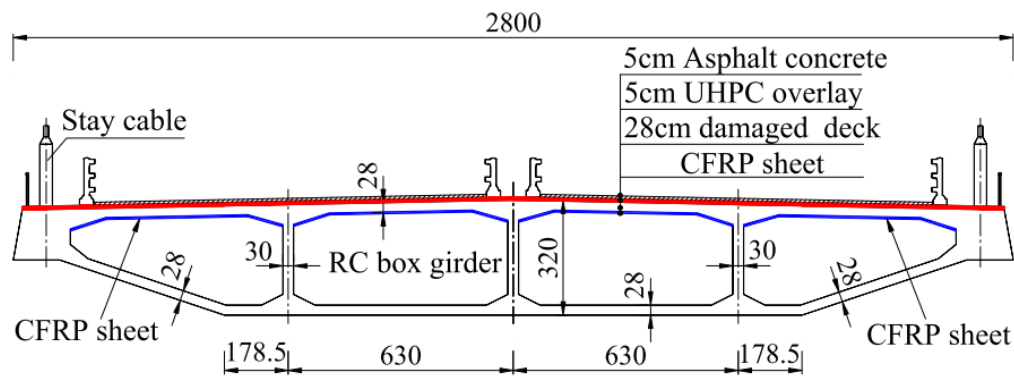


Figure 2.18: Strengthening scheme of the bridge deck [97].

Yin et al. [98] investigated the structural performance of UHPC-strengthening material for RC slabs. This study used two different configurations, including a rehabilitation series with the same thickness and overlaying series employing the UHPC material over the ordinary concrete. Results of the experimental program showed that both series had a significant enhancement of the ultimate load-bearing capacities of concrete slabs. 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.

Al-Osta et al. [99] examined the efficiency of two various strengthening approaches for beam specimens using UHPC, for instance; (a) bonding prefabricated UHPC layers to beam specimens using a very adhesive epoxy material and (b) sand blasting reinforced concrete beams surfaces then casting UHPC in-situ inside suitable moulds, as shown in [Figure 2.19](#). In addition, he strengthened the beam specimens in three dissimilar approaches: (1) one side strengthening from the bottom only (2) two side longitudinal strengthening (3) three sides longitudinal strengthening. Experimental results have shown significant positive development in flexural performance and load-bearing capacities of strengthened beam specimens especially the three sides strengthened specimens. Analytical and numerical modelling was carried out to capture the performance of the beam samples. The models showed a good match with the results from the experiments.

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

Figure 2.19: UHPC strengthening Configurations [99].

Lamporopoulos et al. [100] examined the effectiveness of using UHPC to strengthen conventional RC beam specimens. The experimental investigation and numerical modelling were conducted. Different types of configurations of UHPC layers were used,

on the tension side, compression side, and the three sides, as illustrated in Figure 2.20. The results illustrated a significant moment increment when three sides jacket was used, as illustrated in Figure 2.21.

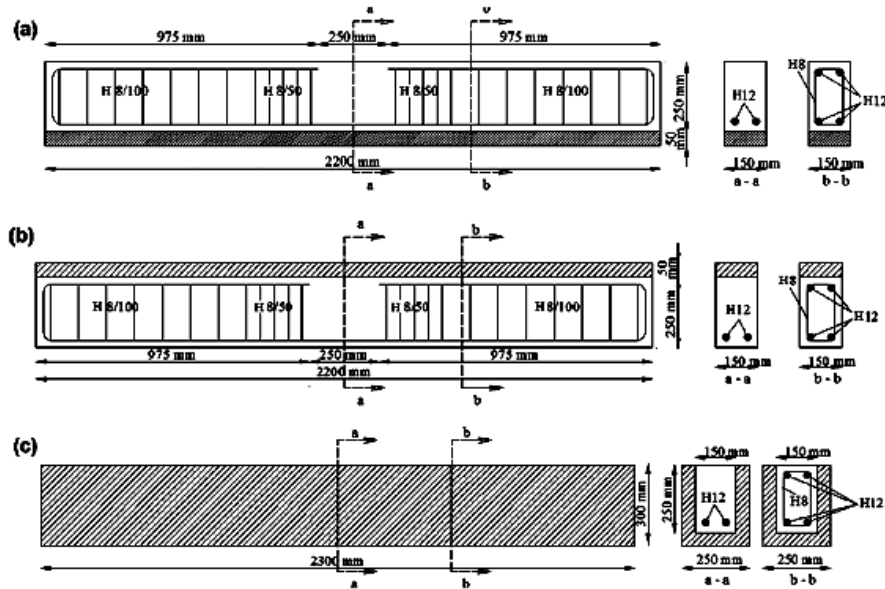


Figure 2.20: Configurations of strengthening RC beam specimens by UHPC in (a) tension side, (b) compression side, and (c) three sides. [100]

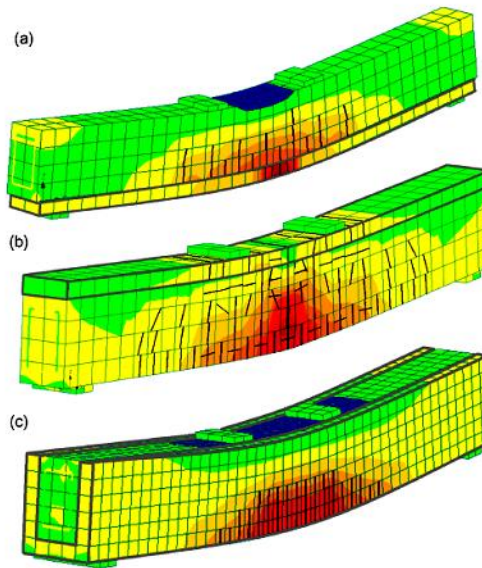


Figure 2.21: Crack pattern in FEM for the three configurations of strengthening beam specimens with UHPC in (a) tension side, (b) compression side, and (c) three sides. [100]

Iqbal et al. [101] investigated the usage of SHLSCC to strengthen the concrete beam specimens. The authors developed analytical modelling to predict the bending moment capacity using the proposed technique of strengthening. They claimed that the SHLSCC technique is a perfect technique for strengthening of concrete elements in flexure. Results of experiments illustrated that the improvement of the ultimate load of the strengthened beams is a function of the thickness of SHLSCC layers in the tension side. Hussein and Amleh [102] studied the flexural capacity of UHPC – a normal concrete composite beam without any shear stirrups. The beam has the normal concrete in the compression part while the UHPC was placed in the tension zone. The results illustrated that the behavior of the composite beam system had been improved in shear and flexural capacities. Figure 2.22 and Figure 2.23 illustrate the reinforcement patterns, test configurations, and cracking patterns for UHPC -NSC flexural prisms, respectively.

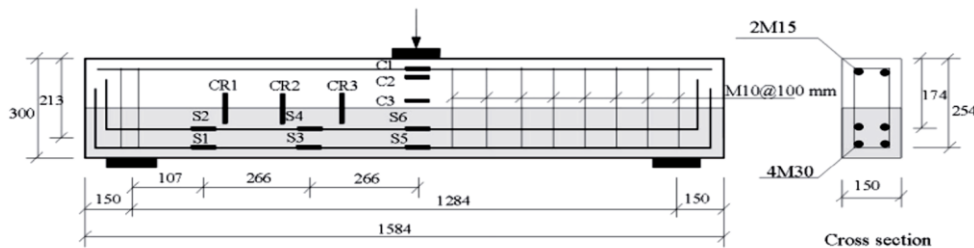


Figure 2.22: Reinforcement pattern and test configuration [102].

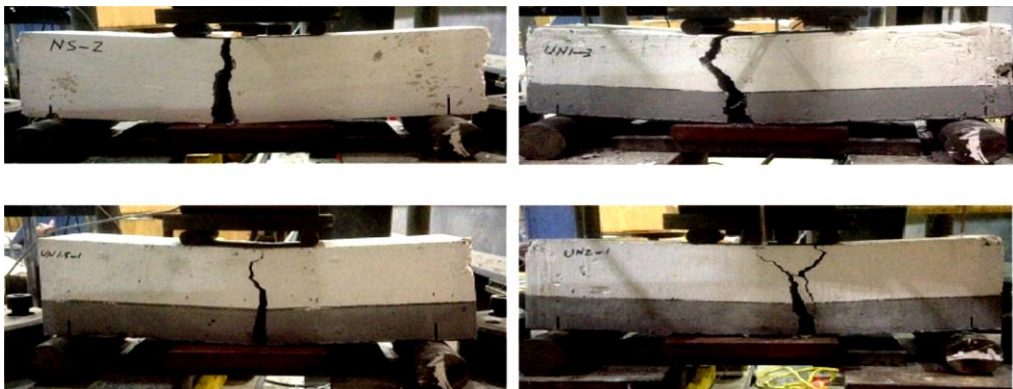


Figure 2.23: Cracking pattern for composite beams [102].

Prem et al. [103] examined the impact of using a layer of 20 mm UHPC thickness for strengthening of 80 and 90% of pre-loaded beam specimens. Seven beams were prepared and subjected to applied loads till 80 % of the maximum load of the control specimen. In contrast, four RC specimens were exposed to 90 % of the maximum load of the control specimen. Layers of UHPC were added to the tension sides of the specimens using appropriate adhesive epoxy and tested under flexure. Results showed a significant enhancement in load-carrying capacities and ductility of pre-loaded beams strengthened with UHPC overlay.

Maasse and Brühwiiler [104] examined the usage of UHPC material as a retrofitting method for bridge deck slabs. Results illustrated a significant enhancement of the load-bearing capacity of the bridge deck by implementing the UHPC layer on the reinforced concrete section of the deck. Analytical modeling for the UHPC- strengthened RC slab was developed, as shown in Figure 2.24, to validate the results obtained from the experiments. It was found that the strengthening using UHPC material had enhanced the load-carrying capacity of the bridge.

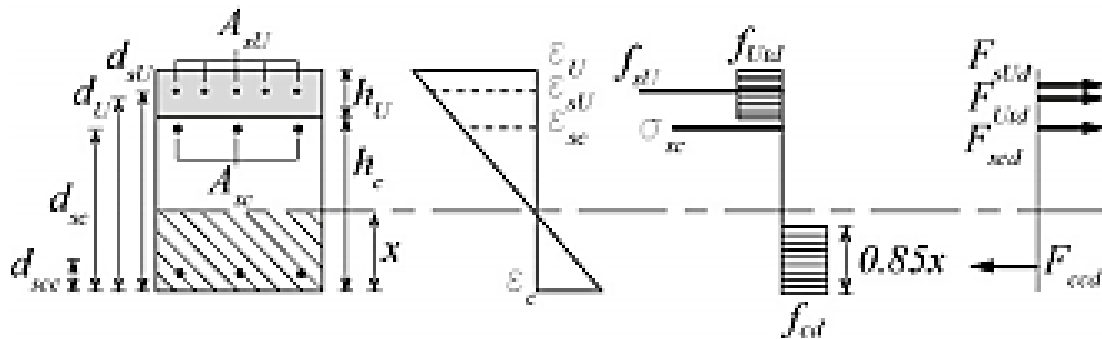


Figure 2.24: Analytical solution model for strengthened RC slab with UHPC [104].

Martiinola et al. [105] examined the influence of using RC fibers in repairing and strengthening large-size RC beam specimens. Both numerical and experimental approaches were adopted in the investigation. Results demonstrated that the suggested technique enhanced the performance of strengthened beams at both ultimate and serviceability limit states. Figure 2.25 illustrates the setup used for testing the specimen and strengthened beams.

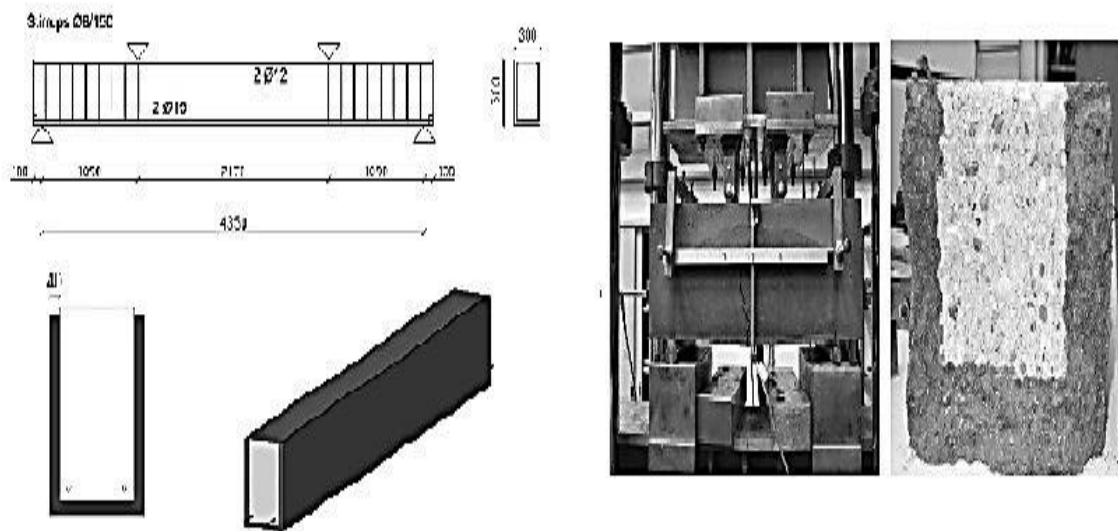


Figure 2.25: Specimen and test set-up [105].

Habel et al. [106] conducted experimental investigations to examine the behavior of twelve large-size conventional beam specimens 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 2.26 (a) and (b) display the test set-up of composite beam and curves of force vs. deflection of the tested beams, respectively.

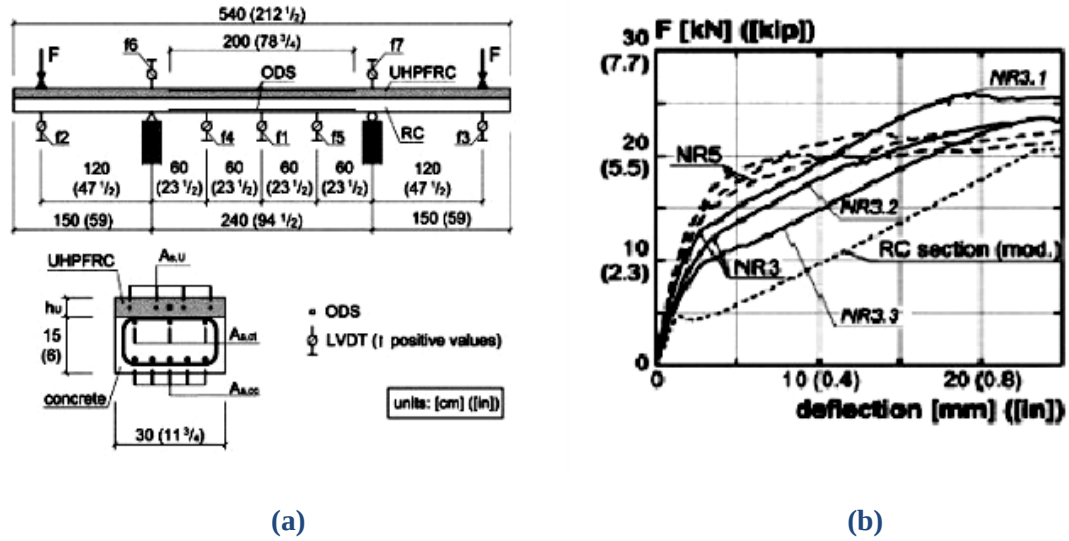


Figure 2.26: (a) Composite beam setup (b) Force vs. mid-span deflection Curves [106].

Farhat et al. [107] studied the response of high-performance fiber reinforced cementitious composite for strengthening and retrofitting of damaged beams. The experimental work consists of testing twenty-four under-reinforced RC beams. Sixteen specimens were prepared and strengthened using the repairing material strips and the remaining left as control. The experimental results have shown that if the strengthened configuration is used on three-sided jacketing, the ultimate loads will enhance up to 86 %. Moreover, the thermal cycling load was conducted on the retrofitted beams to evaluate the bonding strength between the parent concrete and the repaired materials. Results of experimental testing exhibit a very good bond after the thermal loading. Therefore, this type of strength is recommended to use in a hot climate.

CHAPTER 3

EXPERIMENTAL PROGRAM

3.1 General

The RC beams were designed, prepared, and cast using normal strength concrete and conventional steel bars. After casting and curing the RC beams, they were separated into different groups and exposed to accelerated corrosion procedures to cause various percentages of mass loss in their reinforcing bars. Three different corrosion levels were implemented in this work (10, 20, and 30%) mass loss represents moderate, severe, and very severe corrosion damage. A new accelerated corrosion setup was developed and used to introduce corrosion to the reinforcement bars in the RC beams. Then, the corroded beams were strengthened/repared using UHPC material, considering varying the thickness of applied layers and configuration of the UHPC layers. The experimental program presented in the following sections explains all the steps followed to obtain the experimental data used to evaluate the effectiveness of using the UHPC strengthening technique on damaged RC beams due to corrosion.

3.2 Specimens Details

Twenty-eight specimens were prepared and tested to investigate the flexural behavior of corroded RC beams with and without strengthened/repared using UHPC layers. All specimens were prepared and cast at PRAINSA precast Saudi Arabia company. The specimens were designed according to the ACI 318-11 building code [47]. One beam

specimen was chosen as a control beam without neither corrosion nor strengthening labelled by the notation UU, which means uncorroded un-strengthened. In contrast, twenty-seven specimens were exposed to accelerated induced corrosion using impressed current technique. Three RC beams were tested as a control for the targeted corrosion levels of 10, 20, and 30 % represented by the notations CUA, CUB, and CUC, respectively. The first letter (C) indicates that the specimens were corroded, while the second letter (U) refers to un-strengthened beams. The third letter indicates the theoretical corrosion level of the beam (10, 20, or 30) % represented by the notations A, B, and C, respectively. Flexural test results of the beams CUA, CUB, and CUC were compared to uncorroded beam UU to observe the influence of corrosion degree on the RC beams and to examine the influence of UHPC strengthening strategies by comparing their results with those of the rest of other strengthened/repaired specimens. The remaining corroded beams were strengthened either from one side called (repair) and denoted by the letter R or from three sides called (strengthening) and denoted by the letter S. In addition, three different thicknesses of UHPC layers were used for both repair and strengthening configuration including (20, 30, and 40) mm for strengthening and (20, 40, and 60) mm for repair.

A detailed description of all RC beams specimens is illustrated in [Table 3.1](#). The strengthened beams were labelled as CSA, CSB, and CSC to represent the corroded strengthened beams to the targeted corrosion degrees of 10, 20, and 30 %, respectively. The labels were followed by the thickness of the UHPC layers used at the bottom and side surfaces of the beams (20, 30, and 40) mm. In contrast, the repaired beams were labelled as CRA, CRB, and CRC to represent the corroded repaired beams to the targeted corrosion degrees of 10, 20, and 30 %, respectively. The labels were followed by the thickness of the

UHPC layers used only at the bottom surface of the beams (20, 40, and 60) mm. Moreover, the efficiency of using UHPC layers to strengthen beam specimens subjected to further corrosion was investigated. Six specimens were used to investigate the post-strengthened impact of corrosion-damage beams after strengthening using UHPC layers. The letter P is used to indicate the term post-strengthened beams. The post-strengthened beams were labelled as CPB and CPC to represent the corroded strengthened beams to the targeted corrosion degrees of 20 and 30 %, respectively. Same as strengthening, the labels were followed by the thickness of the UHPC layers used at the bottom and side surfaces of the beams (20, 30, and 40) mm.

The specimens have a total length of 1600 mm, with a rectangular cross-section having a depth of 230 mm and a width of 140 mm. The beams were reinforced by 2 Φ 12 mm deformed bars as main bars and 2 Φ 10 mm at the top of the beam to support stirrups. For the purpose of avoiding shear failure of the beam specimens, Φ 8 mm stirrups were placed along the length of the beams at a space of 50 mm except under the loads since the shear force at the middle of the beam is very small. The high number of stirrups (by using a lower spacing of 50 mm) was used to avoid failure in shear and focus the study on the flexural performance of the beams. Epoxy-coated deformed bars were used for both stirrups and top bars to eliminate induced current effects on them. Typical dimensions and reinforcing details of the tested RC beams are illustrated in [Figure 3.1](#)

Table 3.1: Detailed description of test specimens.

<i>Specimen's label</i>	<i>Targeted corrosion level %</i>	<i>Actual corrosion level %</i>	<i>Strengthened, Repaired, or post-strengthened</i>	<i>Thickness of UHPC (mm)</i>
<i>UU</i>	<i>0</i>	<i>0</i>	<i>No</i>	<i>0</i>
<i>CUA</i>	<i>10</i>	<i>9.8</i>	<i>No</i>	<i>0</i>
<i>CUB</i>	<i>20</i>	<i>17.4</i>	<i>No</i>	<i>0</i>
<i>CUC</i>	<i>30</i>	<i>20.8</i>	<i>No</i>	<i>0</i>
<i>CSA-20</i>	<i>10</i>	<i>10.2</i>	<i>S</i>	<i>20</i>
<i>CSA-30</i>	<i>10</i>	<i>9.5</i>	<i>S</i>	<i>30</i>
<i>CSA-40</i>	<i>10</i>	<i>9.1</i>	<i>S</i>	<i>40</i>
<i>CSB-20</i>	<i>20</i>	<i>17.5</i>	<i>S</i>	<i>20</i>
<i>CSB-30</i>	<i>20</i>	<i>16.7</i>	<i>S</i>	<i>30</i>
<i>CSB-40</i>	<i>20</i>	<i>16.8</i>	<i>S</i>	<i>40</i>
<i>CSC-20</i>	<i>30</i>	<i>21.2</i>	<i>S</i>	<i>20</i>
<i>CSC-30</i>	<i>30</i>	<i>22.3</i>	<i>S</i>	<i>30</i>
<i>CSC-40</i>	<i>30</i>	<i>20.9</i>	<i>S</i>	<i>40</i>
<i>CRA-20</i>	<i>10</i>	<i>10</i>	<i>R</i>	<i>20</i>
<i>CRA-40</i>	<i>10</i>	<i>10.8</i>	<i>R</i>	<i>40</i>
<i>CRA-60</i>	<i>10</i>	<i>11.8</i>	<i>R</i>	<i>60</i>
<i>CRB-20</i>	<i>20</i>	<i>18.1</i>	<i>R</i>	<i>20</i>
<i>CRB-40</i>	<i>20</i>	<i>17.8</i>	<i>R</i>	<i>40</i>
<i>CRB-60</i>	<i>20</i>	<i>18.3</i>	<i>R</i>	<i>60</i>
<i>CRC-20</i>	<i>30</i>	<i>22.1</i>	<i>R</i>	<i>20</i>
<i>CRC-40</i>	<i>30</i>	<i>20.8</i>	<i>R</i>	<i>40</i>
<i>CRC-60</i>	<i>30</i>	<i>22.6</i>	<i>R</i>	<i>60</i>
<i>CPB-20</i>	<i>20</i>	<i>14.4</i>	<i>P</i>	<i>20</i>
<i>CPB-30</i>	<i>20</i>	<i>13.2</i>	<i>P</i>	<i>30</i>
<i>CPB-40</i>	<i>20</i>	<i>12.6</i>	<i>P</i>	<i>40</i>
<i>CPC-20</i>	<i>30</i>	<i>20.1</i>	<i>P</i>	<i>20</i>
<i>CPC-30</i>	<i>30</i>	<i>19.7</i>	<i>P</i>	<i>30</i>
<i>CPC-40</i>	<i>30</i>	<i>19.1</i>	<i>P</i>	<i>40</i>

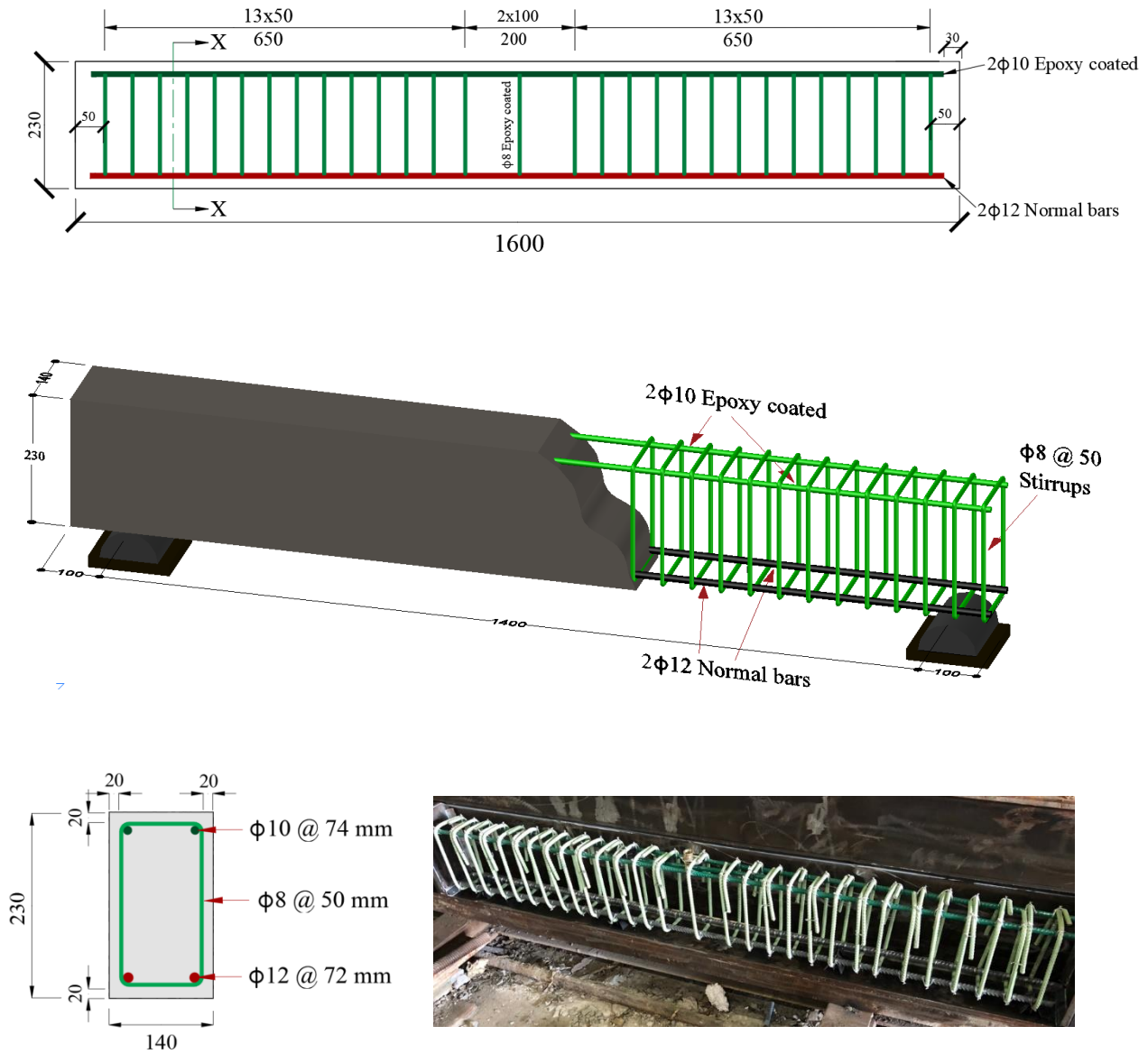


Figure 3.1: Details of beam specimens (dimensions and reinforcement) unit: mm.

For the post-strengthened study of the corroded beams, twelve beams were examined and separated into two phases. Phase I contains six beams separated into two groups labeled B and C exposed to accelerated reinforcement corrosion for 96 and 144 days to represent a targeted mass loss of 20 and 30 %, respectively. After reinforcement corrosion of phase I,

the beams were strengthened using UHPC jacketing layers and tested in flexure. Phase II also contains six beams separated into two groups and exposed to accelerated reinforcement corrosion for 48 and 96 days to represent a targeted mass loss of 10 and 20 %, respectively. After reinforcement corrosion of phase II, the beams were strengthened using UHPC jacketing layers and subjected to a more induced reinforcement corrosion process for another 48 days to represent an extra targeted mass loss in their reinforcing bars of 10 % before flexural testing. The two phases of the experimental investigation are described in the diagram in Figure 3.2. The beams condition in this study are defined by the following letters UU, CU, CR, CS, and CP, which stand for Uncorroded-Unstrengthened, Corroded-Unstrengthened, Corroded-repaired, Corroded-strengthened, and Corroded-Post strengthened, respectively. The following number represents the thickness of UHPC material applied to the concrete surfaces of the beam specimens as repairing or strengthening layers.

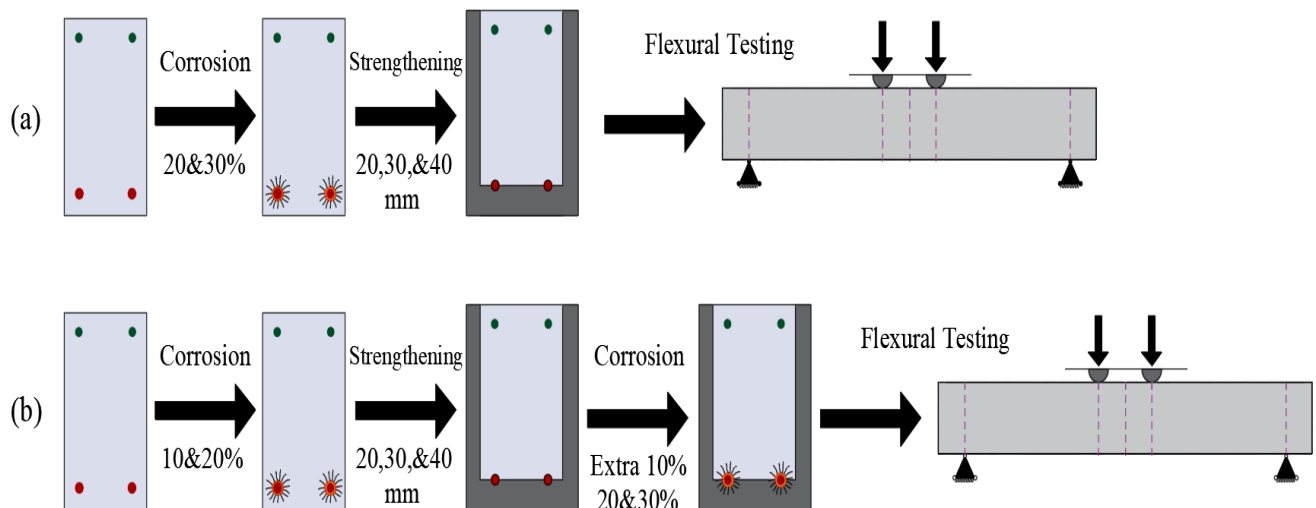


Figure 3.2: Schematic procedure of the testing beams (a) Strengthening beams (CSB & CSC); (b) Post-strengthened beams (CPB & CPC).

Before concreting, copper wires with a thickness of 2.5 mm were fixed to the ends of the main bars from both sides. The wires were tied strongly to the bars using pliers to reduce any chance of movement or disconnecting during the inducing current, as shown in Figure 3.3. The wires were then covered with epoxy to prevent generalized reinforcement corrosion, which could occur when two dissimilar metals are connected, such as steel and copper, as shown in Figure 3.4.

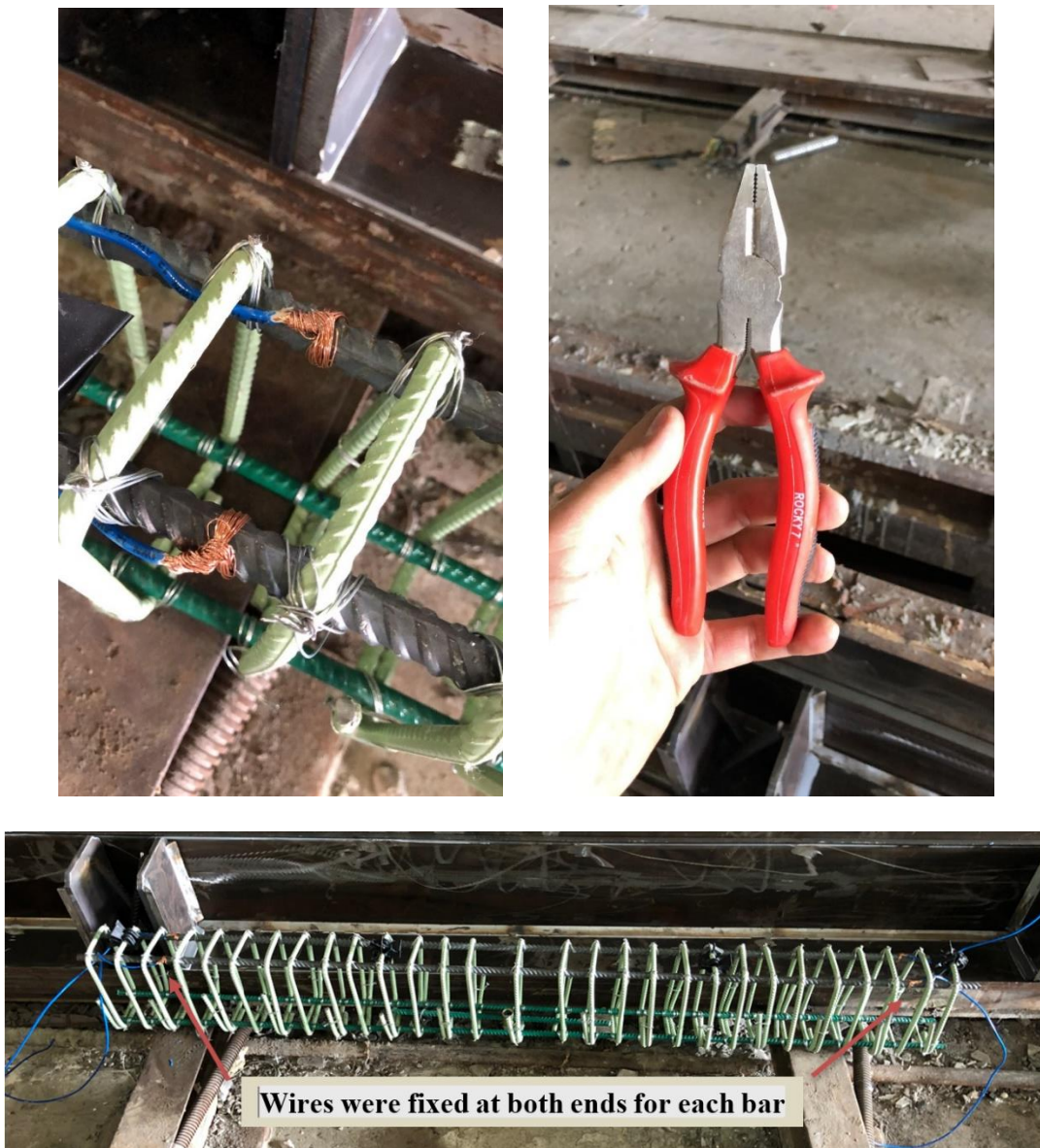


Figure 3.3: Fixing of the electrical wires to the main bars.



Figure 3.4: Applying an epoxy coating to the connection portion.

3.3 Materials Properties

3.3.1 Normal Concrete (NC)

All beam specimens were cast at PRAINSA company using normal concrete (NC), as shown in [Figure 3.5](#). Ordinary concrete was designed and produced at PRAINSA company to cast all the beam specimens. Steel molds were fabricated with the dimension of the beams to cast all the specimens simultaneously in order to maintain the quality and strength of concreting for all beams. A mechanical vibrator was used during the casting of the beams to ensure concrete compaction. The ordinary concrete was composed of ASTM C150 [108] OPC cement Type - 1 grey, natural dune sand, crushed limestone (white), sweet water, and a high amount of water reducer as per (ASTM C494 Type A&F) [109]. The water-cement

ratio of the mixture is 0.4, which gives a slump of 60 cm in diameter. The mix design analysis of ordinary concrete as given from PRAINSA is shown in [Table 3.2](#).



Figure 3.5: Concrete casting (a) Casting of beam Specimens (b) Casting of cylinders.

Table 3.2: Mix design analysis of ordinary concrete.

<i>Material</i>	<i>Weight / m³</i>	<i>SP. GRAVITY (gm/cc)</i>	<i>VOLUME (m³)</i>
<i>Cement (grey)</i>	<i>400 Kg</i>	<i>3.15</i>	<i>0.127</i>
<i>Natural sand</i>	<i>660 Kg</i>	<i>2.66</i>	<i>0.248</i>
<i>Aggregate (3/8")</i>	<i>545 Kg</i>	<i>2.69</i>	<i>0.203</i>
<i>Aggregate (3/4")</i>	<i>689 Kg</i>	<i>2.69</i>	<i>0.256</i>
<i>Water</i>	<i>160 Kg</i>	<i>1.00</i>	<i>0.16</i>
<i>Admixture</i>	<i>2.5 Liter</i>	<i>1.08</i>	<i>0.003</i>
<i>Total</i>	<i>2456.5 Kg</i>	<i>Total</i>	<i>1.00</i>

The RC beam specimens were cured at PRAINSA company using steam curing method. The concrete surfaces of the beam specimens were finished. The beams were covered using plastic sheets to maintain humidity and subjected to steam curing for six hours continuously, as shown in [Figure 3.6](#). After that, the specimens were kept in the air for 28 days to maintain the full strength of normal concrete. Then, the specimens were brought to the KFUPM lab to start the reinforcement corrosion process. [Figure 3.7](#) shows the concrete beam specimens laid inside the KFUPM structural lab.



Figure 3.6: Steam curing of concrete beam specimens at PRAINSA factory



Figure 3.7: RC beam specimens laid inside the KFUPM structural lab

Concrete cylinders with standard dimensions (150 mm length and 75 mm diameter) were cast along with the casting of the beams to calculate the elastic modulus and compressive strength of NC. A compressive test was conducted following ASTM C39/C39M standards [110] to estimate the mechanical properties of normal concrete used for casting the RC beam specimens. Fourteen cylinders were used to estimate the compressive strength and elastic modulus of NC, as illustrated in [Figure 3.8](#). The average values of the elastic modulus and compressive strength of NC were determined as 33 GPa and 50 MPa, respectively, having a standard deviation of 4.5 MPa and 1.1 GPa, respectively. The typical compressive curve of stress-strain for the tested NC is illustrated in [Figure 3.9](#). The tensile strength of NC is measured using the splitting tensile test in accordance with ASTM C496/C496M [111]. The splitting tensile tests were performed on eleven cylinders to estimate the average tensile strength of NC. The splitting tensile test showed a value of 3.3 MPa as an average tensile strength of NC, having a standard deviation of 0.35 MPa. [Figure 3.8](#) shows the compression and splitting tensile tests on NC.

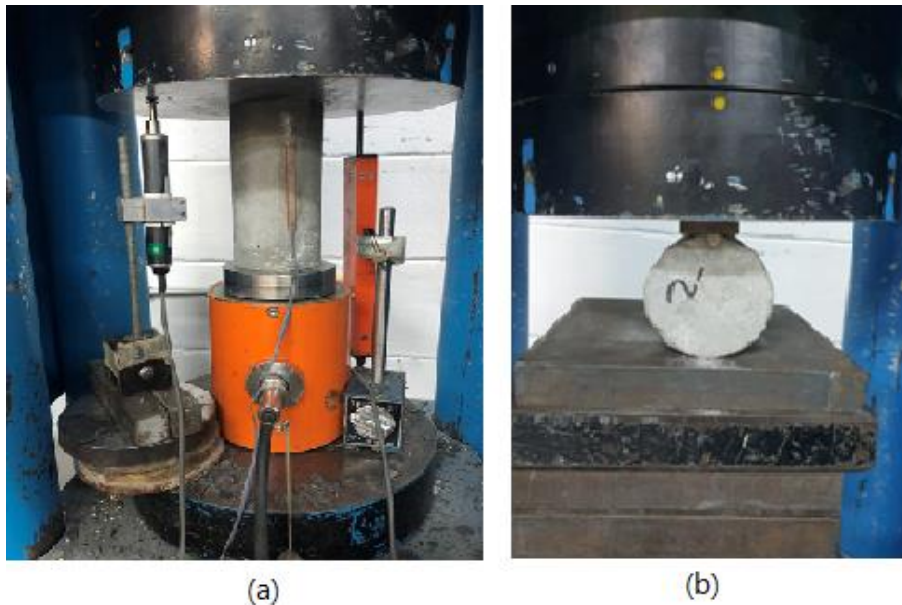


Figure 3.8: (a) Compressive strength and elastic modulus test (b) Splitting tensile test.

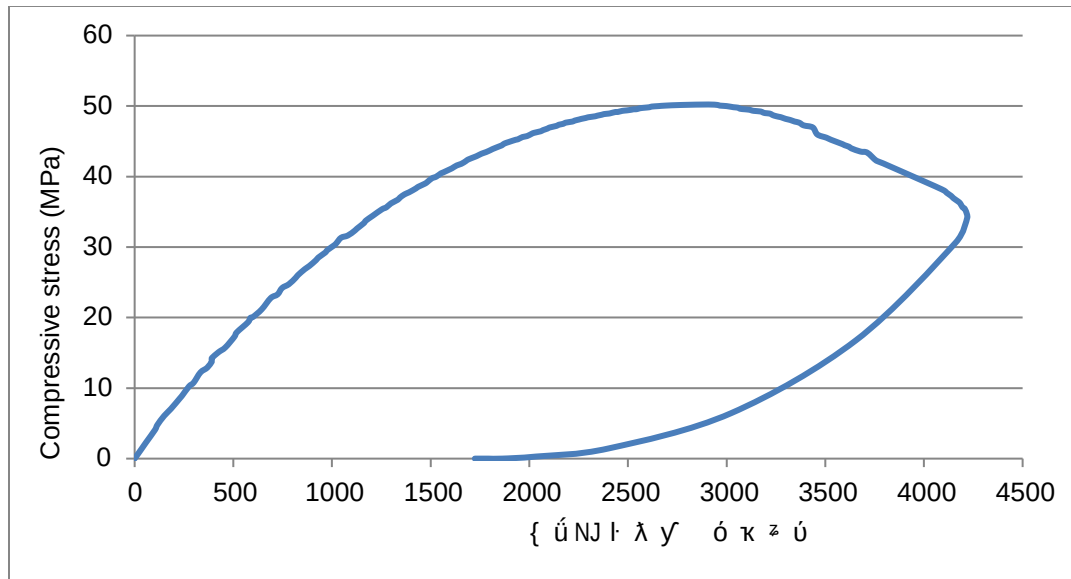


Figure 3.9: Compressive stress-strain curve of NC.

3.3.2 Steel Reinforcement

Three different sizes of bars were used to reinforce the RC beam specimens. Tensile testing was carried out three steel samples of each bar size in accordance with ASTM E8/E8M-13a [112] to estimate the mechanical characteristics of steel reinforcement bars. The bottom reinforcing bars of 12 mm size have nominal yield strength of 600 MPa, while the top reinforcing bars of 10 mm size have nominal yield strength of 580 MPa with an average standard deviation of 27 and 24 MPa, respectively. Stirrups of 8 mm diameter bars have yield strengths of 550 MPa with an average standard deviation of 26 MPa. All reinforcing bars had an elastic modulus of 200 GPa. Figure 3.10 shows an experimental sample of the tensile test carried on 12 mm steel bar.

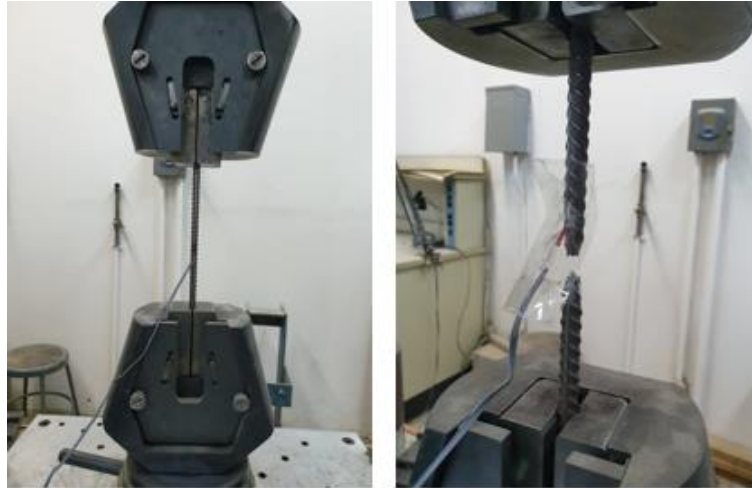


Figure 3.10: Steel tensile test

3.3.3 UHPC

UHPC was made of ASTM C150 [108] Type I cement, Elkem® micro-silica (MS), MasterGlenium® 51, a polycarboxylic ether-based hyperplasticizer, sweet water, and mixed steel fibers, as shown in Figure 3.11. The chemical compositions and specific gravities of Type I cement and micro-silica (MS) used in the UHPC mixture are shown in Table 3.3.



Figure 3.11: UHPC ingredients used for strengthening.

Table 3.3: Composition of cement and MS (mineral filler).

Component	% by mass	
	Cement	MS
$CaCO_3$	-	-
CaO	64.4	0.49
SiO_2	21.9	92.4
Al_2O_3	5.63	0.71
Fe_2O_3	3.79	0.95
K_2O	0.35	0.85
MgO	2.10	1.79
Na_2O	0.2	0.51
<i>Sp. gravity</i>	3.15	2.25

For enhancing the tensile behavior of UHPC, two different shapes of steel fibers were used in the UHPC mix with a percentage of 50% each. The total volume fraction of the steel fibers (6%) including 3% straight fibers with a diameter of 0.1 mm and a length of 12.5 mm and 3% of copper-coated hooked-end fiber having a diameter of 0.2 mm and a length of 25 mm. The tensile strength and modulus of elasticity for the steel fibers are 2500 MPa and 200 GPa, respectively. Fine dune sand was used as the aggregate phase for the UHPC mixture. The mixed proportion of UHPC used in this study was developed by Ahmad et al. [113]. The weights of constituent constituents to produce 1 m³ of UHPC are illustrated in [Table 3.4](#). The various material tests used, including multiple cylinders, cubes, prisms, and dog bone samples, to estimate the mechanical properties of UHPC are illustrated in [Figure 3.12](#).

Table 3.4: UHPC mixture proportion [113].

Ingredient's quantity in (kg) for a batch size of 1 m ³						
Cement (kg)	SP (kg)	Sand (kg)	MS (kg)	Water (kg)	Fiber (kg)	TOTAL (kg)
900	40.32	1005.13	220	162.4	157	2484.9

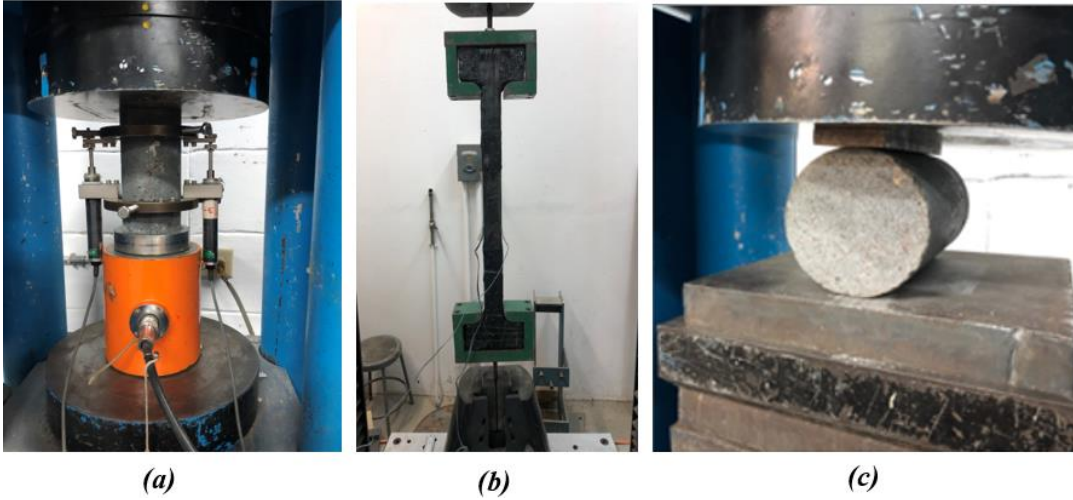


Figure 3.12: Material test of UHPC: (a) Elastic modulus. (b) Direct tensile test using dog bone. (c) Split tensile test.

Compression tests were conducted to estimate the compressive strength and elastic modulus of the UHPC in accordance with ASTM C39 [110] for cylinders (150 mm length and 75 mm diameter) and BS EN 12390-3 [114] for cubes (50 mm side and 150 mm side). The test results showed 169 MPa and 46 GPa with a standard deviation of 7.5 MPa and 1.6 GPa for compressive strength and modulus elasticity for UHPC, respectively. ASTM C496 standard [111] was used to perform split tensile strength tests of UHPC. The average tensile strength found from the splitting tensile test was 18.4 MPa with an average standard deviation of 1.3 MPa. In contrast, results of direct tension tests of 4 dog bones showed a value of 7.3 MPa, with an average standard deviation of 0.5 MPa. Figure 3.13 illustrates the stress vs strain curve for UHPC from the compression test. Figure 3.14 illustrates the stress vs. strain curve from the direct tensile test of dog bones.

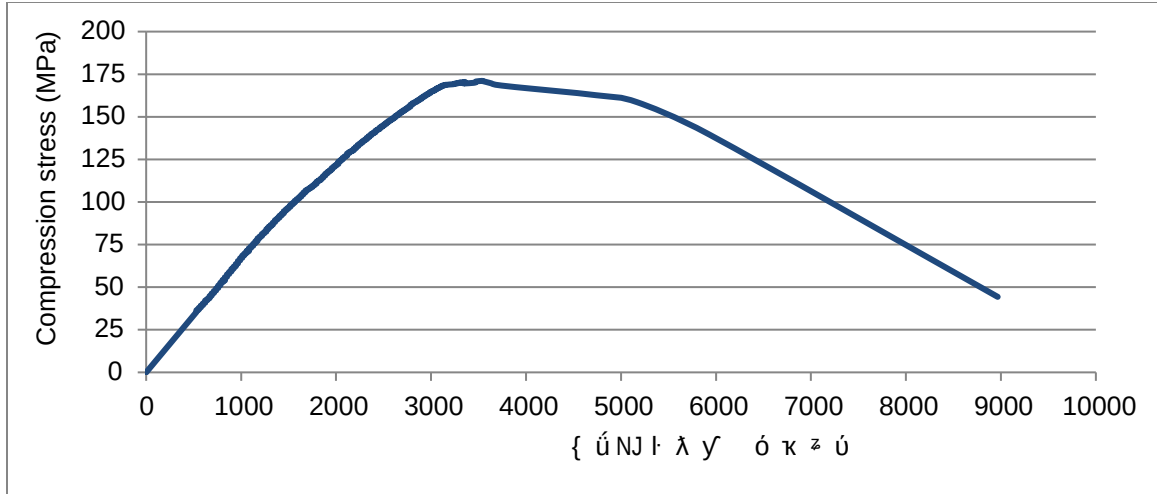


Figure 3.13: Stress-strain curve for UHPC in compression.

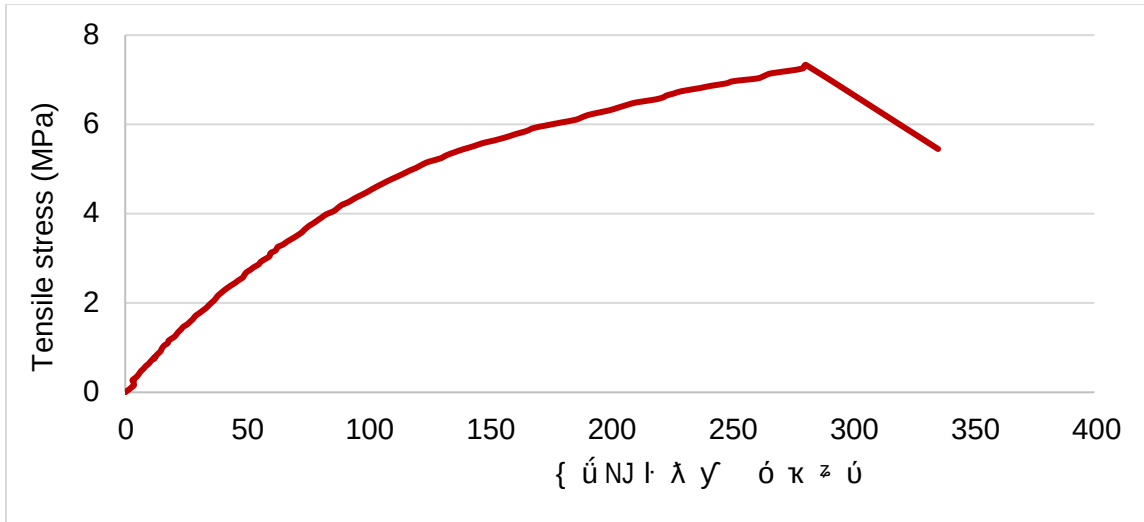


Figure 3.14: Stress-strain curve for UHPC from a direct tensile test.

3.3.4 Bond between UHPC and Normal Concrete (NC)

The bond strength between UHPC and NC is tested in two ways. The split tensile test was used to calculate the tensile bonding strength, while slant shear tests were used to estimate the tensile and shear bond strength. Seven concrete cylinders were cut in half and sandblasting was carried out on the inner surfaces of the samples. Then, the UHPC material was cast in situ and water cured in a water tank at a room temperature of $23 \pm 2 \text{ }^{\circ}\text{C}$ for 28

days. Seven samples for each type of experimental test were used. The split tensile test results illustrated that the average tensile bond strength is 4.5 MPa with an average standard deviation of 0.7 MPa. On the other hand, the slant shear testing result illustrated that the average shear bonding strength was 18 MPa while the average tensile bond strength was 5 MPa with an average standard deviation of 1.7 and 1.1 MPa, respectively. [Figure 3.15](#) and [Figure 3.16](#) illustrate the split tensile and slant shear tests between UHPC and NC, respectively.



Figure 3.15: Split tensile test between UHPC and normal concrete



Figure 3.16: Slant shear test between UHPC and CFRP

3.4 Accelerated Reinforcement Corrosion Setup

The developed setup of induced reinforcement corrosion consists of a sophisticated DC power supply, which has the ability and sufficient voltage to corrode the steel bars inside the beam specimens. The positive terminal was connected directly to the reinforcing bars of the first specimen acting as the anode, while negative terminal was connected to the stainless-steel plate of last specimen acting as a cathode. [Figure 3.17](#) illustrates the DC power source used in reinforcement corrosion setup developed in this study. It has a maximum voltage of 700 volts which was sufficient to introduce current to groups of RC beams simultaneously. When turning on the DC power supply after connecting to the beams, the voltage jumped to high values and kept unstable for about 12 to 24 hours. Then, it started to decrease after the first crack initiation in the specimens' concrete surface and became stable for the remaining duration of the reinforcement corrosion process. The most advantage of using this particular DC power supply to induce reinforcement corrosion in RC members is to stabilize the current applied to the beams while the voltage can fluctuate up and down according to the resistance of the RC specimens.



Figure 3.17: DC power source used to induce current to RC beams.

The whole RC beam specimens were twirled up-down, so the main steel bars which needed to be corroded became at the top surfaces of the tested beams, as illustrated in Figure 3.18. The aim of twirling the beams is to develop the electrical circuits between the steel bars acting as anode in the electrical circuit while the stainless-steel sheets acting as a cathode. To complete the circuit between the steel bars and stainless-steel sheets, saturated towels were placed at the upper face of the beams to act, with the beam cover, as an electrolyte portion. The towels remain wet during the reinforcement corrosion process to allow the current to pass continuously between anode and cathode. Specific holes were fabricated on the stainless-steel sheet. Nozzles were prepared along the sheet to spray 5% salt water every 3 hours to the towels. This technique helps to keep the towels saturated during reinforcement corrosion without dissolving of the oxidation products. The schematic drawing of the reinforcement corrosion setup is presented in Figure 3.19.

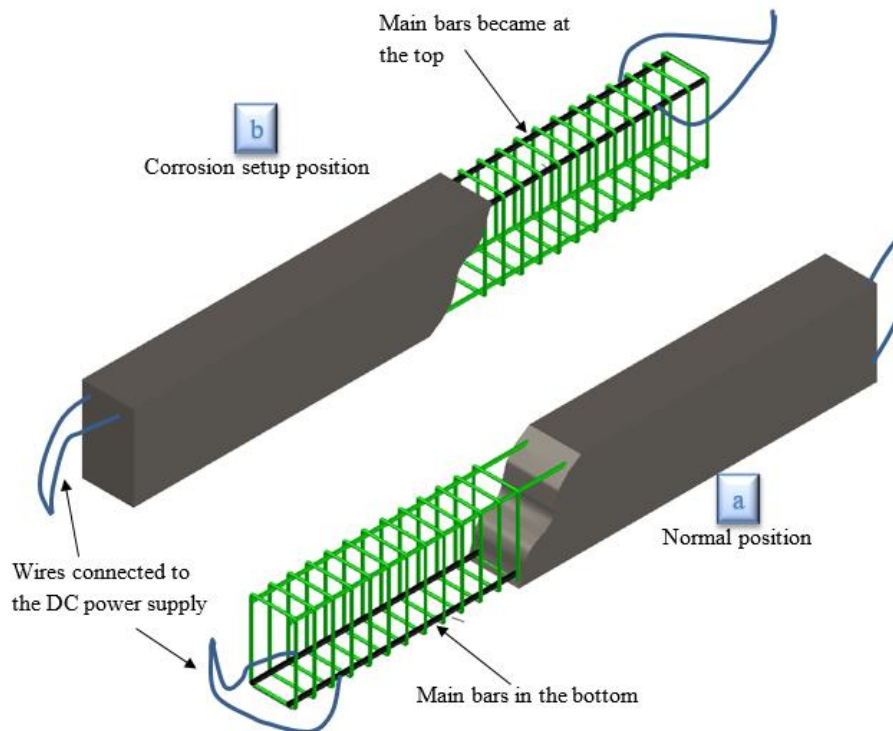


Figure 3.18: Positions of RC beams (a) Normal position, (b) corrosion setup position.

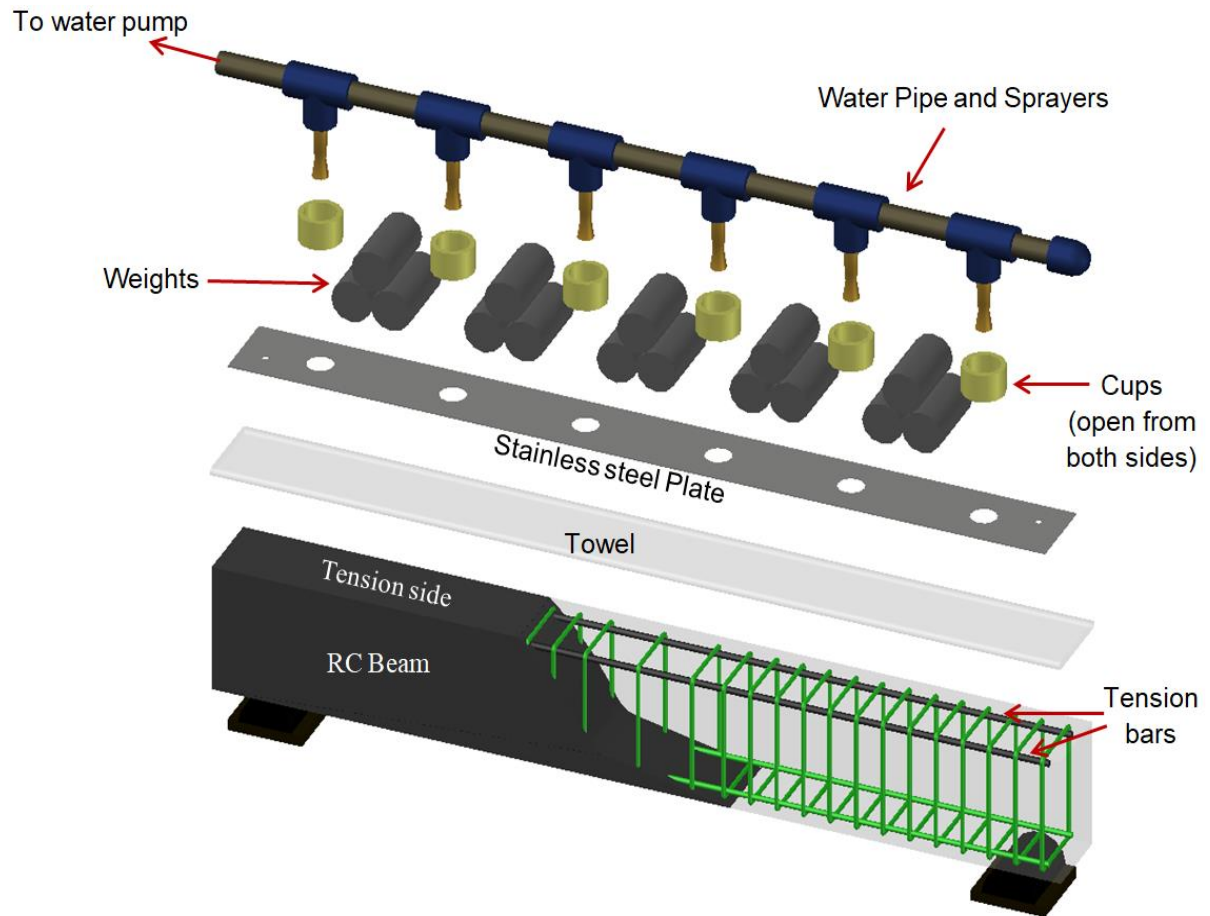


Figure 3.19: Schematic drawing of the reinforcement corrosion setup.

A density of $200 \mu\text{A}/\text{cm}^2$ was applied to beam specimens for the total time of reinforcement corrosion procedure. The current density level was chosen to be $200 \mu\text{A}/\text{cm}^2$ to represent the actual natural reinforcement corrosion that could be faced in real structures. In addition, this density level is highly recommended by [12] to avoid the expected bond loss, which can be occurred at the concrete/steel interfaces due to high-density current level. All specimens were connected in series, as illustrated in Figure 3.20. A constant current was applied through the electrical circuit for 48, 96, and 144 days for the beams A, B, and C, respectively. Figure 3.21 illustrates a 3D schematic drawing of reinforcement corrosion set-up developed in the current work.

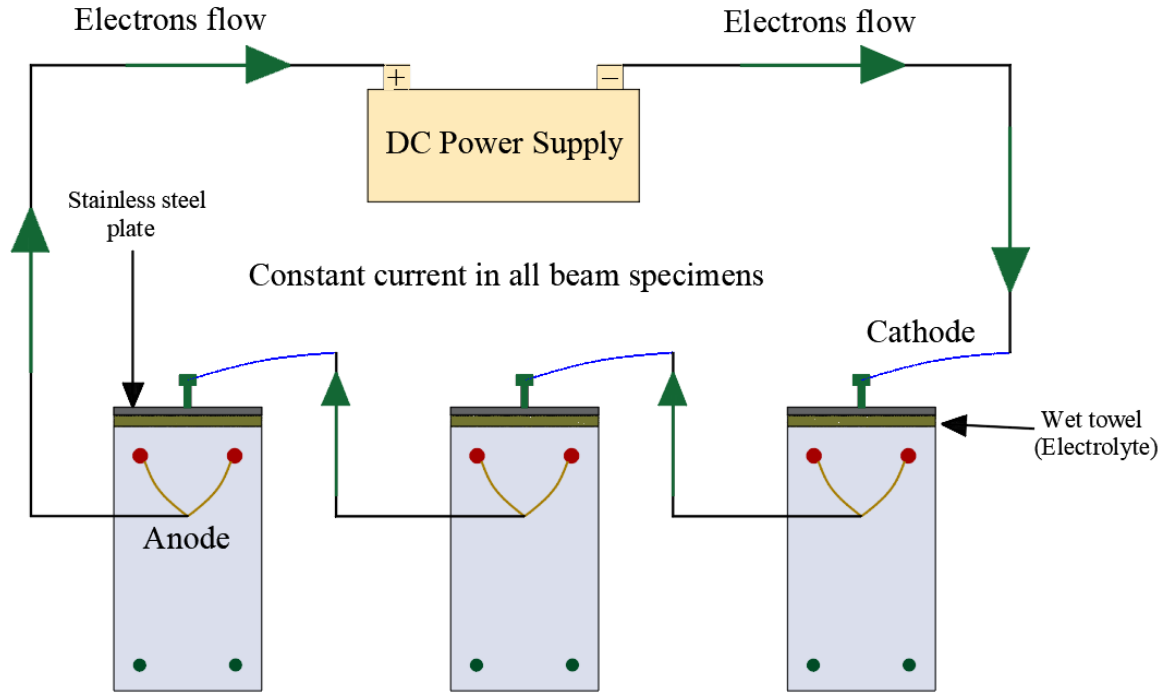


Figure 3.20: Schematic diagram of sequential electrical connections of the specimens.

Law of Faraday (Eq. 3.1) was used to estimate the targeted mass losses of the steel bars and the period required for each reinforcement corrosion level.

$$m = \frac{I t a}{n F} \quad (\text{Eq. 3.1})$$

where: m is mass loss in grams, n is number of electrons transferred during the reinforcement corrosion process which = 2 for steel bars, F is the Faraday's constant equal to 96500 C/mol, I is impressed current in Ampere, t is the duration of reinforcement corrosion process in seconds, and a is atomic mass of steel which is equal to 55.847 gr for steel bars. As previously stated, this study considered three levels of reinforcement corrosion mass loss: 10, 20, and 30 % for group names A, B, and C, respectively. The targeted reinforcement corrosion levels (10, 20, and 30) % were chosen to represent the modest, critical, and very critical levels of reinforcement corrosion loss, respectively.

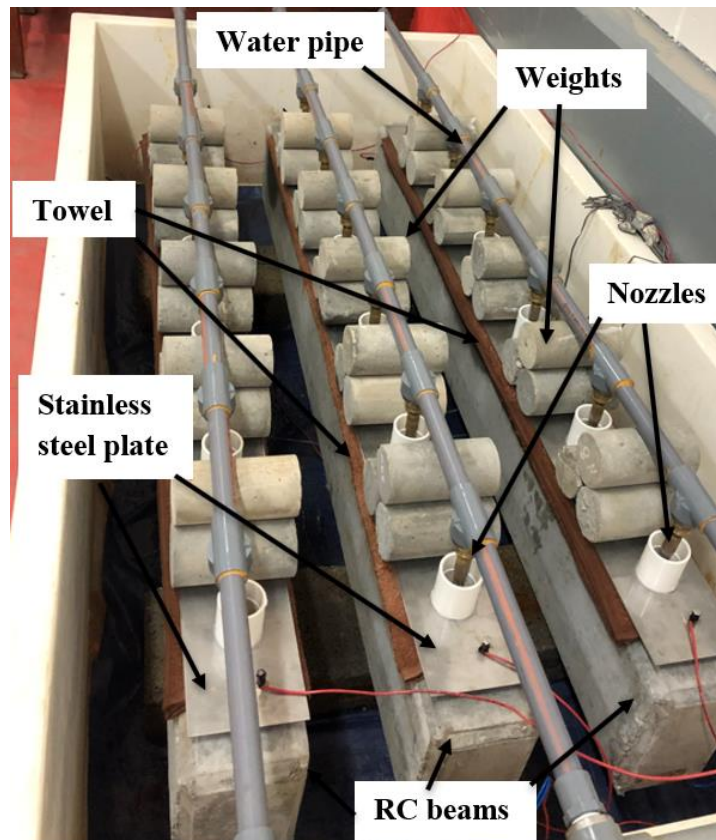
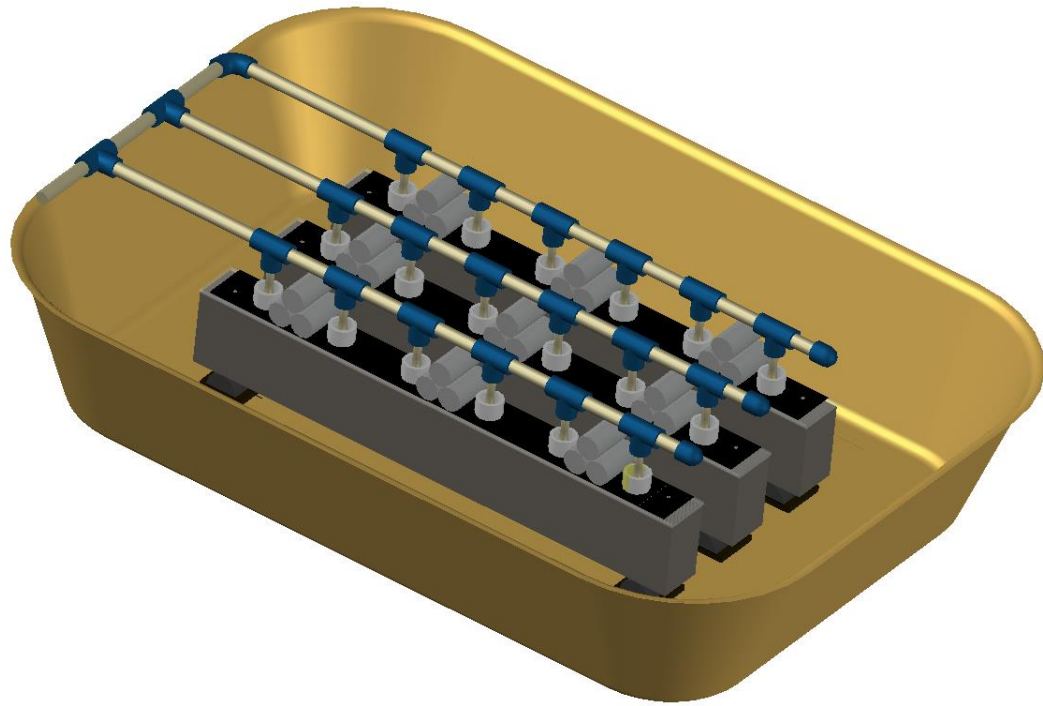


Figure 3.21: 3D schematic drawing of the reinforcement corrosion setup.

3.5 Preparing the specimens for UHPC casting

After completing the reinforcement corrosion process for the RC beams, sandblasting was applied to the side surfaces of the corroded beams to an average depth of 2.5 mm, as illustrated in [Figure 3.22](#). The sandblasting was conducted to improve the bonding strength between old concrete material and UHPC layers. The concrete surfaces of the beam specimens were cracked from the bottom and side surfaces due to induced reinforcement corrosion in their main reinforcing bars and because the reinforcement corrosion rust is greater in volume than actual reinforcing bars. The cracked cover was detached using a steel hammer, and corroded bars were cleaned by steel brushes and high-pressure air to remove the reinforcement corrosion rust, as illustrated in [Figure 3.23](#). All breaking and cleaning processes were performed gently so the repairing and cleaning procedures did not affect the steel bars.



Figure 3.22: Sandblasted beams (after reinforcement corrosion process)



Figure 3.23: Remove concrete cover and cleaning the corroded bars

3.6 UHPC strengthening and repairing Techniques

UHPC was cast to strengthen the corroded RC beams from the three sides for strengthened and post-strengthened RC beam specimens, including the bottom side. A configuration of three sides jacketing was used with three different thicknesses of UHPC layers (20, 30, and 40) mm, as shown in Figure 3.24. In contrast, three different UHPC thicknesses (20, 40, and 60) mm were used for repairing corroded beam specimens from only the bottom side for the repaired RC beams, as illustrated in Figure 3.25.

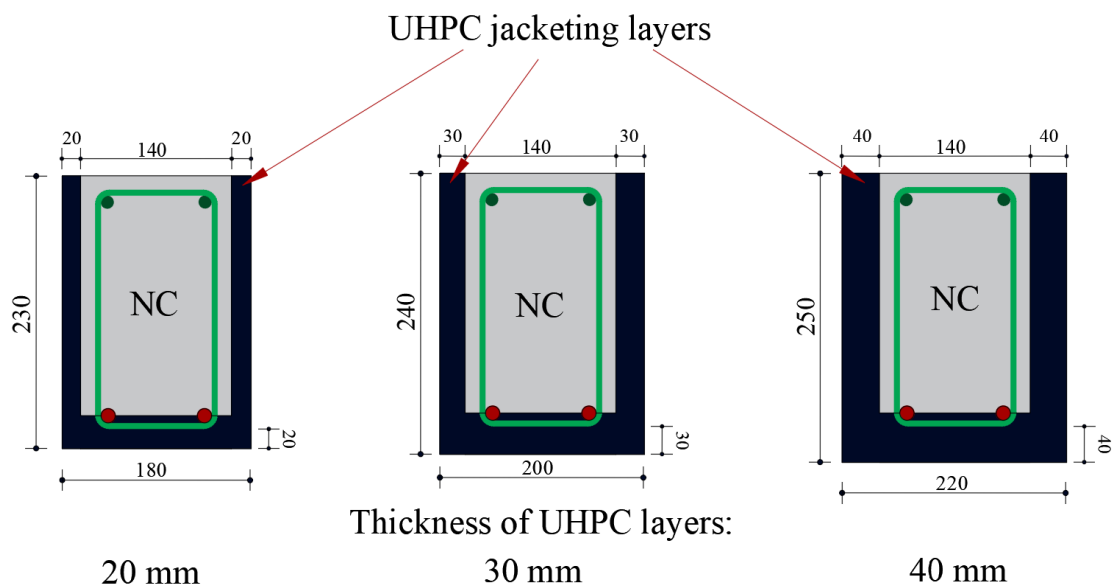


Figure 3.24: Configuration and thicknesses of UHPC layers for strengthened and post-strengthened RC beam specimens

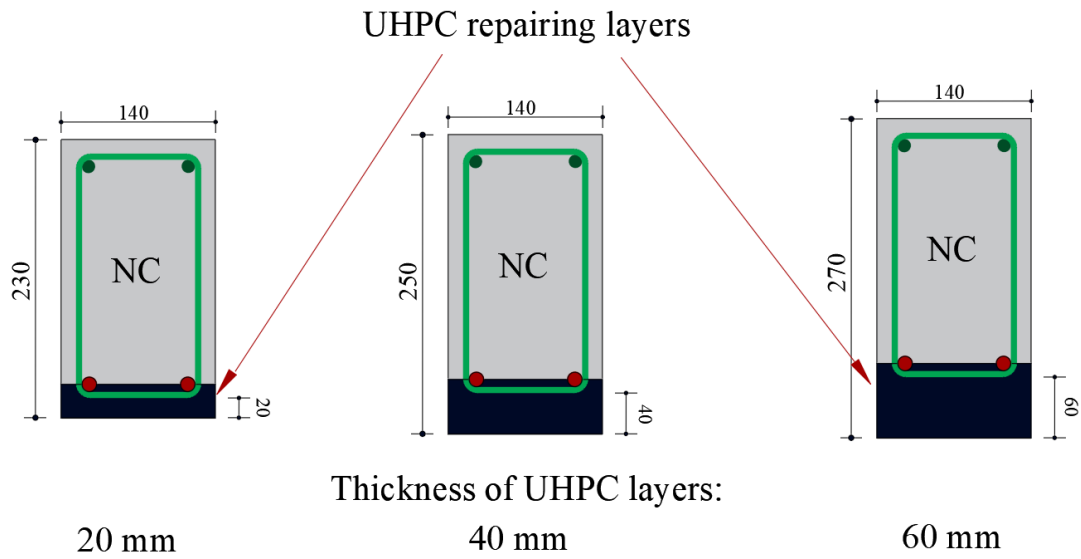


Figure 3.25: Configuration and thicknesses of UHPC layers for repaired RC beams

Due to the heavy weight of UHPC material, wood molds, which prepared for UHPC casting, were designed to be very strong to sustain the applied lateral pressure which could applied while casting of UHPC mixture. Figure 3.26(a) shows the casting work of UHPC. Then, all the UHPC casted beams were placed together, wrapped with burlap sheets, covered with plastic sheets to maintain humidity and moisture for UHPC to gain its full compressive strength, as illustrated in Figure 3.26(b). Water was sprayed regularly on the burlap twice daily for continuous 28 days.



(a)



(b)

Figure 3.26: (a) UHPC casting; (b) UHPC curing

3.7 Testing setup and instrumentation

The flexural setup was prepared to test strengthened and un-strengthened RC beam specimens under flexural failure considering the required specifications of ASTM, as shown in Figure 3.27. All beams were examined and loaded till collapse in a four-point bending configuration setup using a hydraulic jack having a capacity of 300 kN. A steel beam was placed at the mid-span above the tested beams to distribute the loads from the hydraulic jack for achieving a pure bending region. The distance between the two supports of the tested beams was fixed to be 1400 mm. All experiments were conducted under displacement control at an approximate rate of 0.5 mm per minutes. During testing, a load cell of 3000 kN capacity was utilized for measuring the applied loads. To measure the strain in the concrete, 60 mm concrete strain gauges were attached at the upper surface of the tested beams at the mid-span, while 6 mm steel strain gauges were placed at the critical locations of the tensile bars and stirrups. As illustrated in Figure 3.28, 4 LVDTs were utilized at various positions on the beam to measure the deflection at the midspan and

rotations at the beam's support. All LVDTs, gauges, and load cell linked to a 40-channel acquisition data logger which recorded readings every two seconds.

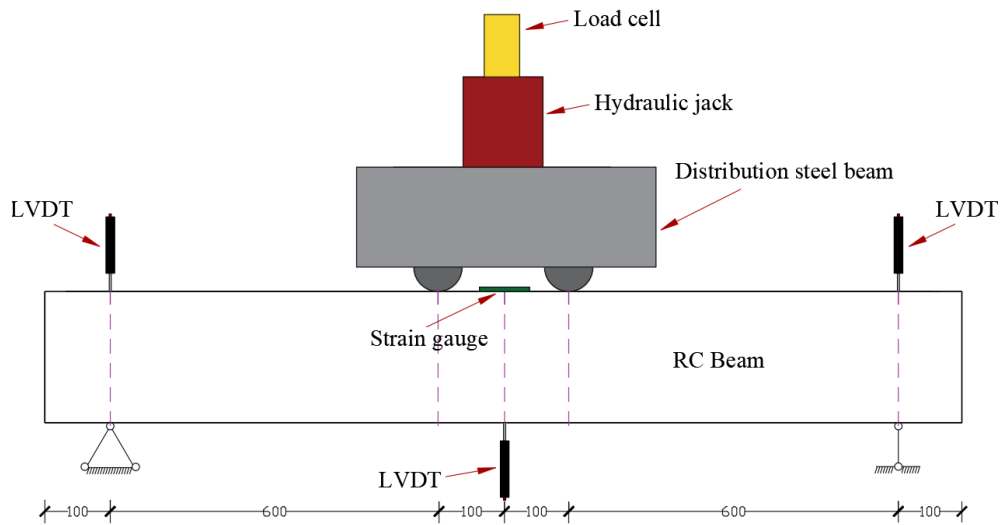


Figure 3.27: Testing setup of RC beam specimens

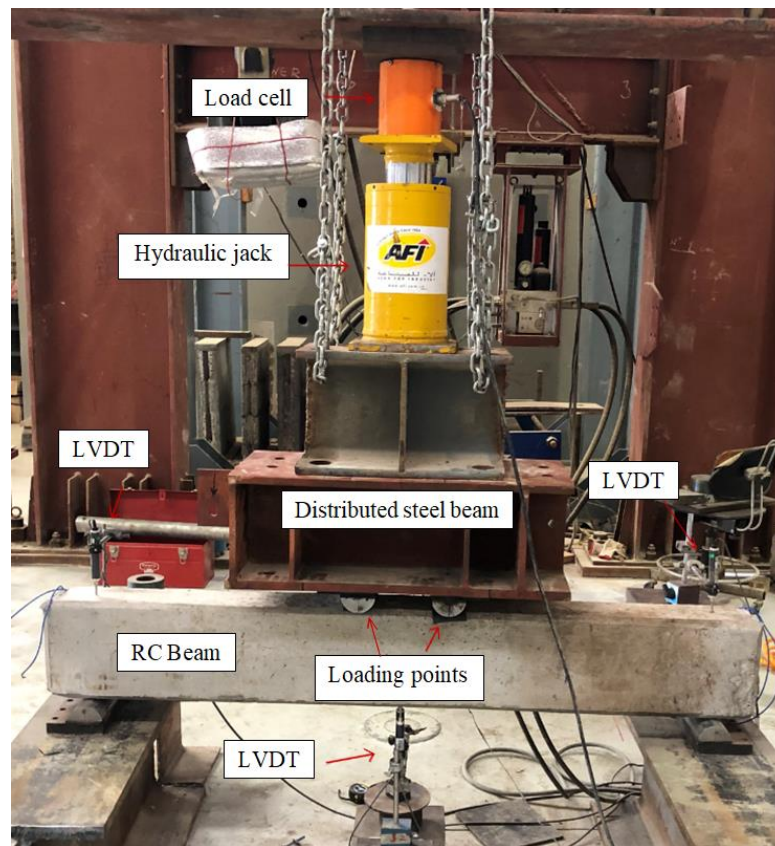


Figure 3.28: Testing setup and instrumentation arrangement.

3.8 Extracting the corroded bars for gravimetric analysis

All the corroded beam specimens were demolished to extract the corroded bars for gravimetric analysis. The extraction of the corroded bars was held carefully to avoid any extra damage or loss of the material of the steel bars. A jackhammer was used first to remove the cover of the beam. Then an iron hammer was used gently to take off the surrounding concrete from the corroded steel bars, as illustrated in [Figure 3.29](#).



Figure 3.29: Extracting of corroded bars (a) Remove the concrete cover (b) Remove the surrounding concrete from the bars.

As planned during the design of the beam specimens, the top bars and stirrups were not affected by the induced current during the reinforcement corrosion process. [Figure 3.30](#)

shows the steel reinforcement bars of one beam sample after demolishing the specimen.

Figure 3.31 shows a set of corroded bars after being extracted from the beam specimens.



(a)



(b)

Figure 3.30: Reinforcement bars after demolishing the specimen (a) Top bars and stirrups (b) Main corroded bars .



Figure 3.31: Set of corroded bars after being extracted from the beam specimens.

CHAPTER 4

RESULTS AND DISCUSSION

4.1 General

The experimental results of the current study were pertaining to the damage corrosion levels of the reinforcing steel bars in RC beams under accelerated corrosion, flexural testing, and the actual mass loss of the corroded bars due to reinforcement corrosion. This chapter presents, discusses, and analyzes the experimental results of tests conducted during the current study and draws the most useful conclusions based on the results of the findings. Corrosion observations and gravimetric analysis results were presented and discussed in this chapter. In addition, the flexural testing results, including failure modes, ductility, energy absorption, and stiffness, are also presented and discussed.

4.2 Corrosion cracking

The reinforcement corrosion products appeared at the concrete surface of the beams after five days from the initiation of the induced current. Cracks at the upper surface of the inverted beams cannot be seen due to the existence of a saltwater spraying system, while side cracks are visible. At the end of each reinforcement corrosion process for each group, beam specimens were disconnected to the reinforcement corrosion set up and removed from the tanks. Specimens were kept for three days in the air then visual observation was conducted for each corroded beam. Rust stains and longitudinal cracks were observed in the bottom and side surfaces of the corroded beams parallel to tension bars, as illustrated

in Figure 4.1. In contrast, no cracks were detected in the compression side and around stirrups due to the usage of epoxy-coated bars. Tensile stress was developed in the concrete surrounding the corroded bars as a result of the reinforcement corrosion products being 4 to 6 times as large in volume as the uncorroded steel bars. Also, it was noticed that cracks became wider for higher reinforcement corrosion levels due to increasing the volume of the corroded bars compared to the original bars, i.e., four to six times higher. Cracks were measured for each beam specimen, and the average crack width, C_{avg} Was calculated and listed in Table. Moreover, all measured cracks for corroded beams of group names A, B, and C exceeded the limit of maximum crack width proposed by ACI 318-11 [115], which should not be more than 0.4 mm.

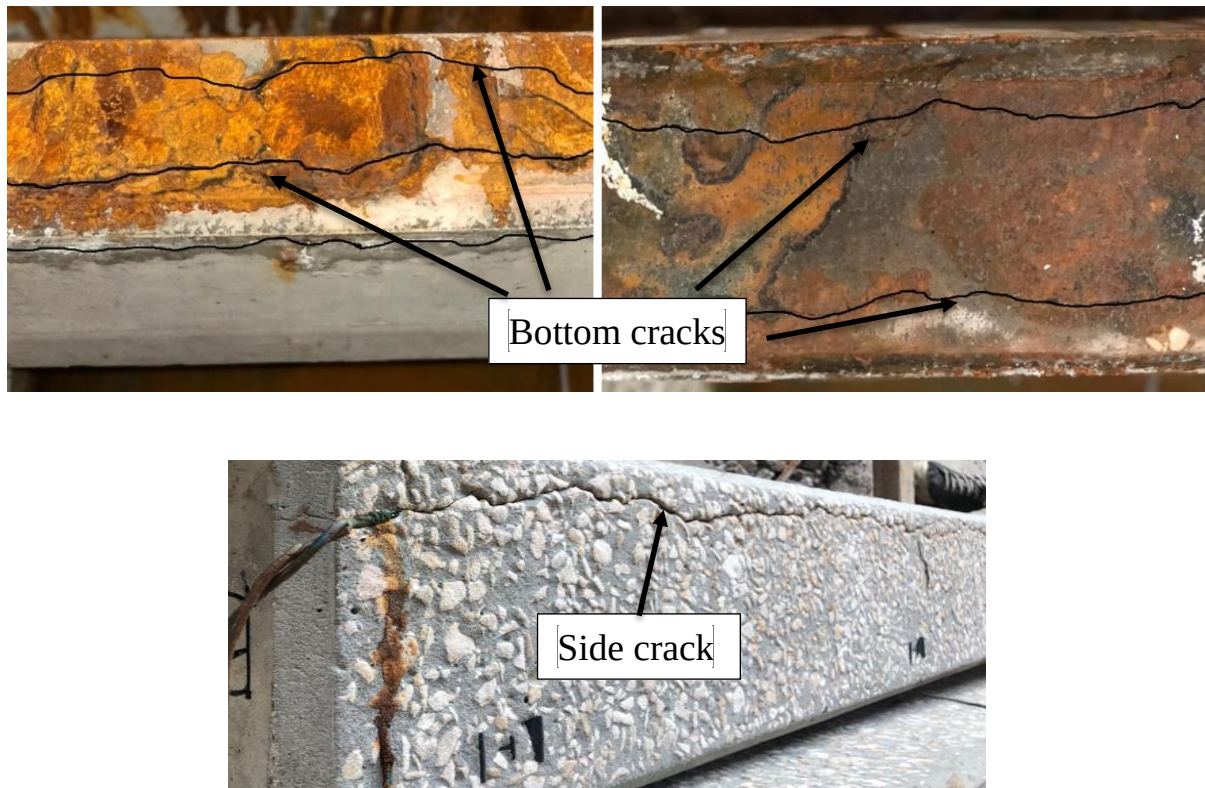


Figure 4.1: Longitudinal cracks of corroded beams.

Rust stains were noticed on the bottom and lateral surfaces of beam specimens subjected to the post-strengthened corrosive environment in Phase II, as shown in Figure 4.2. Cracks of these specimens were not detected clearly after induced reinforcement corrosion because of the high compressive and tensile strengths of UHPC strengthening layers. These findings indicate that the usage of the UHPC material as a strengthening layer can significantly improve the serviceability of the strengthened RC beams in terms of concrete cracking.



(a)



(b)

Figure 4.2: Post-strengthened RC beams after corrosion (phase II) (a) Bottom surfaces of beams CPA-20, 30, and 40 (b) Side surface of beam CPB-40

4.3 Gravimetric analysis

After conducting flexural testing on the corroded beam specimens, the beams were broken using suitable jackhammer equipment, and corroded steel bars were extracted carefully from each specimen. Visual inspection of removed steel bars showed the presence of some pitting corrosion located randomly along bar surfaces, as illustrated in [Figure 4.3](#).

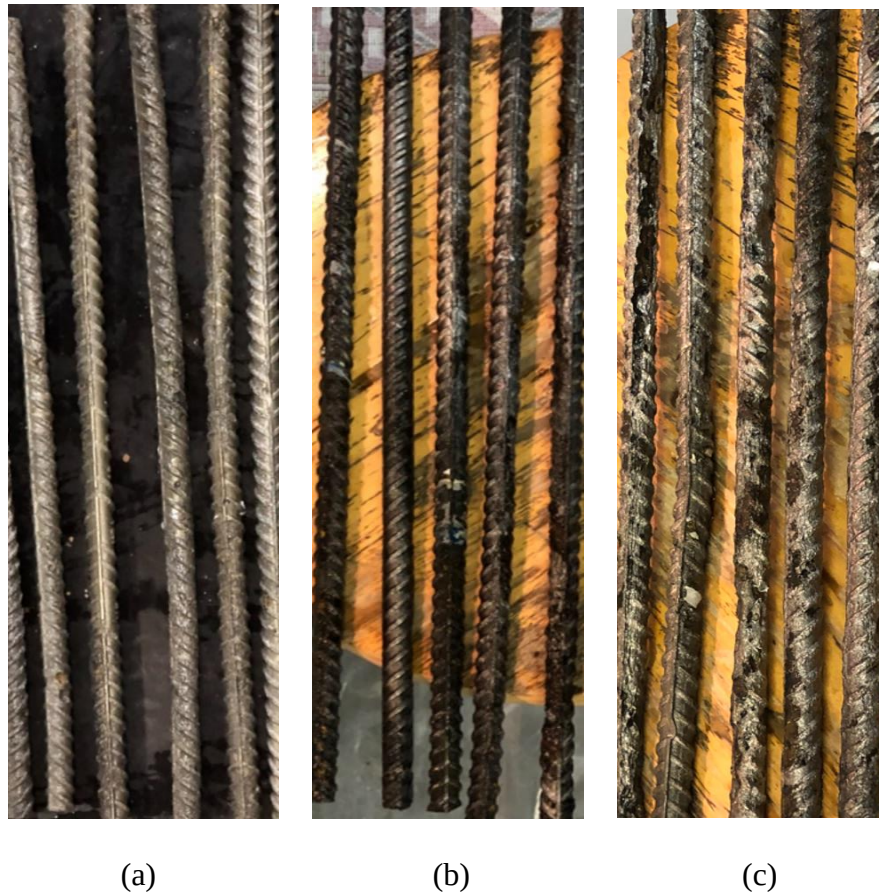


Figure 4.3: Corroded steel bars: a) Group A corroded bars, b) Group B corroded bars, and c) Group C corroded bars.

Each corroded bar was cut into four equal pieces with 385 mm length for each piece and gravimetric analysis was carried out according to ASTM G1 [53] to find the actual mass loss values of the steel bars. 20 g of Sb_2O_3 and 50 g of SnCl_2 were added to 1000 mL of hydrochloric acid with a specific gravity of 1.19 to prepare the solution in accordance with

ASTM G1 [53], as illustrated in Figure 4.4. The solution was stirred using stir equipment until dissolving of all ingredients.

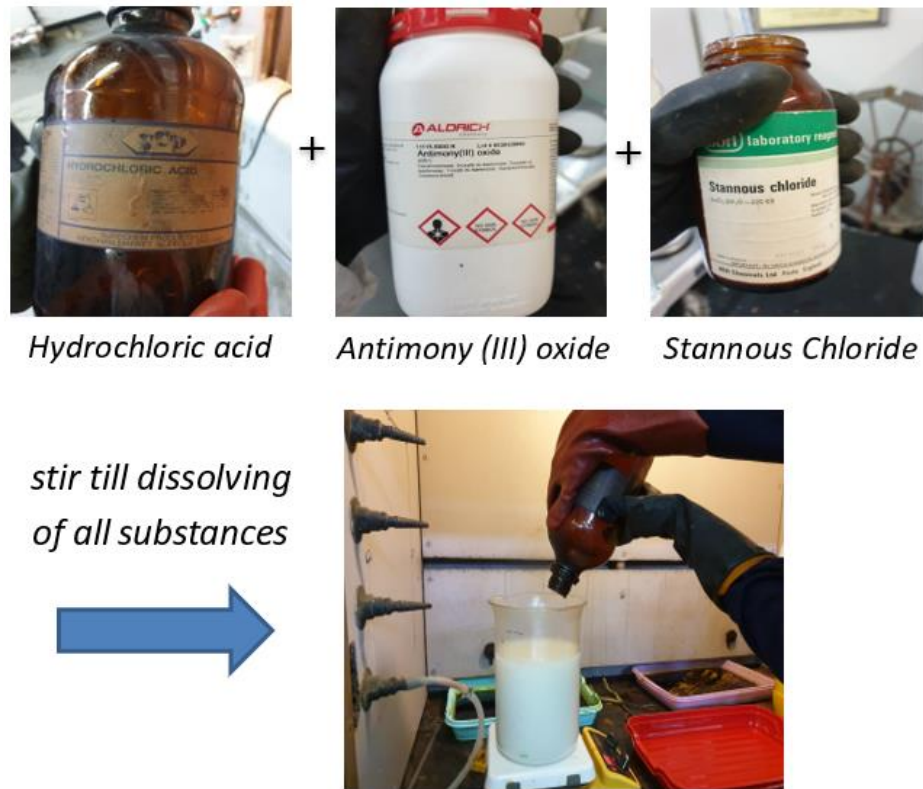


Figure 4.4: Preparation of the solution to conduct gravimetric analysis in accordance with ASTM-G1 [53].

The weight of the corroded bars was recorded first. Then the bars were submerged in the solution for 20 minutes at a room temperature of 23 ± 0.5 C°. After that, the bars were removed carefully from the solution using specific gloves and brushed gently using a harsh steel brush and cold water. The bars were dried to remove water and moisture from the surfaces of the bar then the residual weight was recorded. The bars were then submerged again in the solution for another 20 minutes, and all procedures were repeated till the residual weight of the bars became constant. Multiple cycles were carried out to clean the corroded bars then the mass loss percentages were calculated using Eq. 4.1.

$$\% \text{ mass loss} = \frac{\text{Original weight} - \text{Weight obtained from gravimetric analysis}}{\text{Original weight}} \times 100 \quad (\text{Eq. 4.1})$$

Figure 4.5 illustrates an example showing the change of the gravimetric analysis weight for the beam CSB-30.

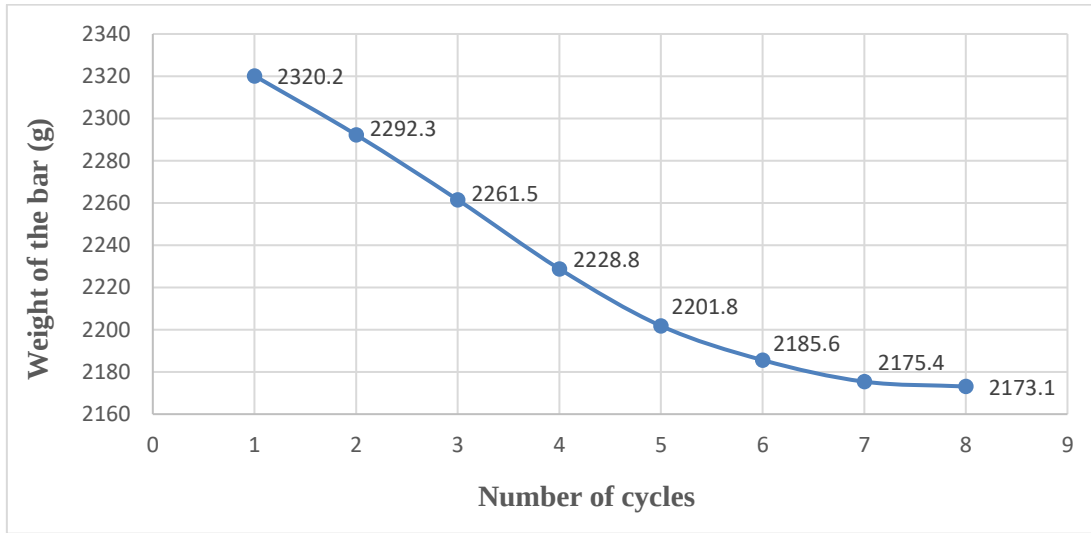


Figure 4.5: Change of the gravimetric analysis weight for the beam, i.e., CSB-30

The actual masses losses for every beam are illustrated in Table 4.1. The average mass losses of 10.2, 17.5, and 21.5% were measured for the corrosion groups A, B, and C, respectively, representing targeted mass losses of 10, 20, and 30% as expected from calculations of Faraday's equation Eq. (1). The result shows a discrepancy between actual and theoretical mass loss in corroded reinforcing bars, as illustrated in Figure 4.6. The variation between actual and targeted mass loss of reinforcing bars increases with increasing corrosion degrees. This discrepancy of mass losses can be referred to decrease in rust production rate as the corrosion process continues, which was reported by [26]. However, the exact mass loss values for each beam specimen were employed for the analytical and numerical modeling.

Table 4.1: Mass losses results of the corroded beams.

<i>Specimen's label</i>	<i>Targeted corrosion level %</i>	<i>Actual corrosion level %</i>	<i>Specimen's label</i>	<i>Targeted corrosion level %</i>	<i>Actual corrosion level %</i>
<i>UU</i>	<i>0</i>	<i>0</i>	<i>CRA-40</i>	<i>10</i>	<i>10.8</i>
<i>CUA</i>	<i>10</i>	<i>9.8</i>	<i>CRA-60</i>	<i>10</i>	<i>11.8</i>
<i>CUB</i>	<i>20</i>	<i>17.4</i>	<i>CRB-20</i>	<i>20</i>	<i>18.1</i>
<i>CUC</i>	<i>30</i>	<i>20.8</i>	<i>CRB-40</i>	<i>20</i>	<i>17.8</i>
<i>CSA-20</i>	<i>10</i>	<i>10.2</i>	<i>CRB-60</i>	<i>20</i>	<i>18.3</i>
<i>CSA-30</i>	<i>10</i>	<i>9.5</i>	<i>CRC-20</i>	<i>30</i>	<i>22.1</i>
<i>CSA-40</i>	<i>10</i>	<i>9.1</i>	<i>CRC-40</i>	<i>30</i>	<i>20.8</i>
<i>CSB-20</i>	<i>20</i>	<i>17.5</i>	<i>CRC-60</i>	<i>30</i>	<i>22.6</i>
<i>CSB-30</i>	<i>20</i>	<i>16.7</i>	<i>CPB-20</i>	<i>20</i>	<i>14.4</i>
<i>CSB-40</i>	<i>20</i>	<i>16.8</i>	<i>CPB-30</i>	<i>20</i>	<i>13.2</i>
<i>CSC-20</i>	<i>30</i>	<i>21.2</i>	<i>CPB-40</i>	<i>20</i>	<i>12.6</i>
<i>CSC-30</i>	<i>30</i>	<i>22.3</i>	<i>CPC-20</i>	<i>30</i>	<i>20.1</i>
<i>CSC-40</i>	<i>30</i>	<i>20.9</i>	<i>CPC-30</i>	<i>30</i>	<i>19.7</i>
<i>CRA-20</i>	<i>10</i>	<i>10</i>	<i>CPC-40</i>	<i>30</i>	<i>19.1</i>

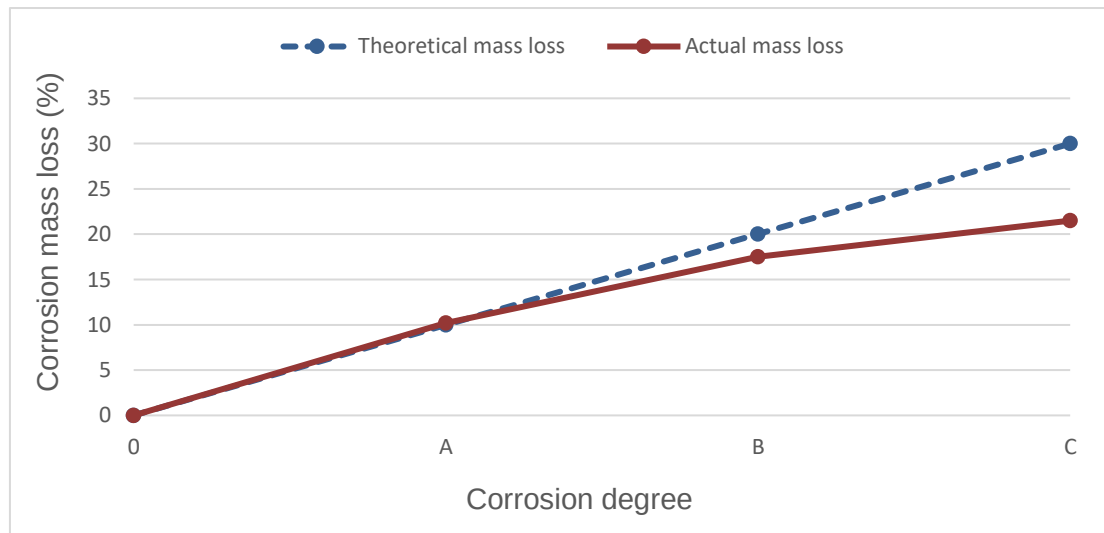


Figure 4.6: Theoretical and actual mass losses versus corrosion levels.

The values of actual mass loss for both phases I and II, representing strengthened and post-strengthened corroded beams, are also shown in [Table 4.1](#). The steel bars of phase II showed lower mass loss than phase I specimens despite the beams being exposed to the same time of induced corrosion. This was attributed to the UHPC jacketing layers, which have a very low permeability compared to the ordinary concrete, which reduced the quantity of air and water distributed to the steel reinforcement bars. This could explain the lower corrosion rate in phase II specimens. The average mass loss in the beams CPB-20, CPB-30, and CPB-40 were 14.4, 13.2, and 12.6 %, respectively, while the beams CSB-20, CSB-30, and CSB-40 showed an average mass loss of 17.6, 16.7, and 16.8 %, respectively, representing a theoretical mass loss of 20 %. In contrast, the mass loss in the beams CPC-20, CPC-30, and CPC-40 were found to be 20.1, 19.7, and 19.1 %, respectively, while the beams CSC-20, CSC-30, and CSC-40 showed an average mass loss of 21.2, 22.3, and 20.9 %, respectively, representing a theoretical mass loss of 30 %. The average mass losses of 13.4 and 19.6 % were measured for the phase II beams and corrosion groups A and B, respectively, while the average mass losses of 17 and 21.5 % were calculated for the phase I beams and corrosion groups A and B, respectively. It can be noticed for both corrosion degrees A and B that the actual mass loss of phase II is significantly less than that for phase I even though the beams were exposed to the same duration of induced corrosion. The beams of phase II and group A corrosion showed a higher reduction in the mass loss than the beams of group B. The application of UHPC strengthening material for phase II beam specimens could lower the influence of corrosion on mass losses of reinforcing bars area by 21 and 9 % for groups A and B, respectively, compared to phase I beams. It is also noticed that with increasing the UHPC thickness, the corrosion mass loss decrease

indicating the slight effect of UHPC thickness on lowering the effect of corrosion degree. The plot illustrated in Figure 4.7 shows the disparity of the mass losses percentage between phase I and phase II for both corrosion groups A and B.

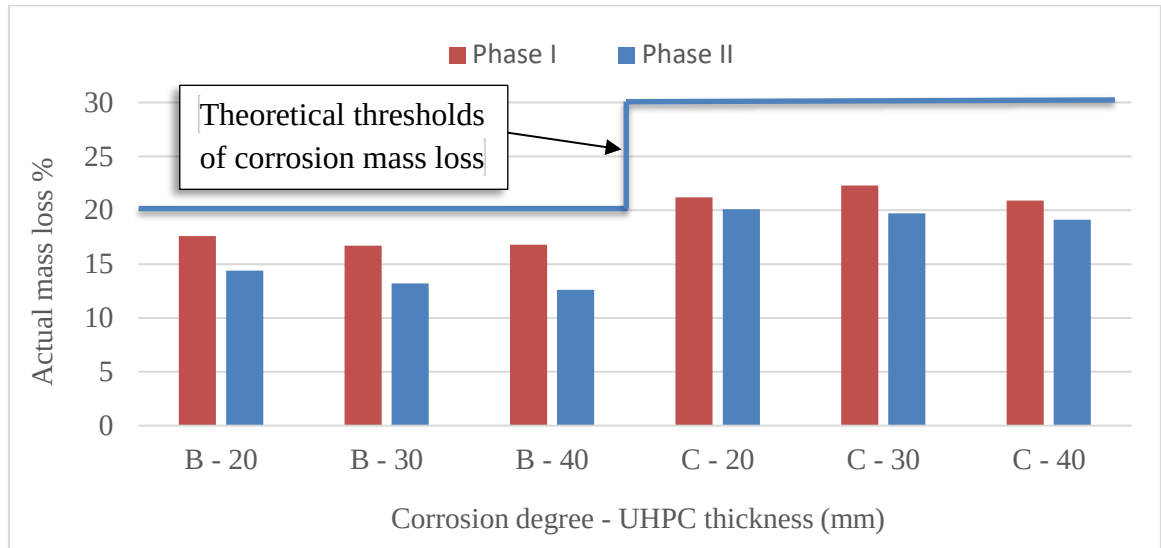


Figure 4.7: Disparity of mass losses percentage between phase I and phase II for both corrosion groups B and C.

4.4 Structural behavior of RC Beam Specimens

Loads vs. midspan deflections curves for all strengthened and repaired RC beams were plotted and compared to uncorroded control beam UU. This section contains the results of all tested beams divided into four sections, including the control, strengthened, repaired, and post strengthened beam specimens.

4.4.1 Control beams (corroded and uncorroded)

4.4.1.1 Flexural behavior

The plot of load vs. midspan deflection curves for control beams are illustrated in Figure 4.8. As noticed, the behavior of corroded specimens was affected significantly by the

corrosion of reinforcing bars, with a slightly notable impact on the stiffness of the beams. Table 4.2 illustrates the flexural experimental results of control beam specimens. The tensile reinforcing bars yielded before concrete crushing at the compression zone indicate a pure flexural failure behavior. The first cracks in control beams UU, CUA, CUB, and CUC, started at the loads 19.5, 17.4, 16.4, and 16.1 kN, respectively, which indicates that the 10% corrosion degree had a lower impact on the concrete cover as compared with 20% corroded beam and the 20% corrosion degree had a lower impact on the concrete cover as compared with 30% corroded beam. The corroded beam CUA illustrates a high drop in the ultimate strength compared to the UU beam. The corrosion of the bars in the RC members has two main influences on the structural behavior of corroded beams. Besides the decrease in the cross-sectional area of corroded bars, bonding between reinforcing bars and surrounding concrete is significantly affected. The corroded beams CUB and CUC show very comparable behavior, including ultimate load and stiffness. This is because of the small difference between the actual mass loss of the two beams.

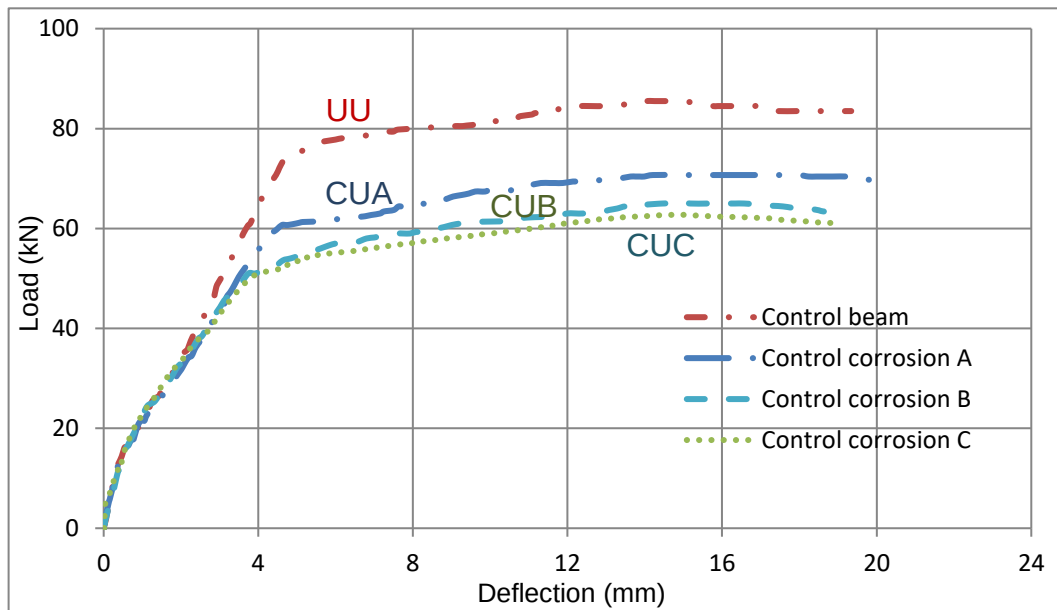


Figure 4.8: Load vs. midspan deflection curves of corroded control specimens.

The load vs. midspan deflection curves illustrated in Figure 4.8 show similar behavior between the corroded beams and the uncorroded control beam UU. This demonstrates that uniform corrosion has occurred through all the bars without severe pitting corrosion at one location, which could occur during induced corrosion. The efficiency of using the developed corrosion setup inducing reinforcement corrosion into steel bars in the current study is obviously clear from the behavior of corroded beam specimens compared to the UU beam. Results of the load-deflection curve obtained in this study were compared with another study from the literature to show the variation in the overall performance of corroded specimens. A research study conducted by Azad et al. [54] was used for comparison with the current study. Figure 4.9 (a) and (b) illustrate load vs. midspan deflection curves of the current study and Azad et al. [54] study, respectively. The results of Azad et al. [54] showed a large discrepancy between the flexural behavior of corroded beams compared to the uncorroded virgin beam, as illustrated in Figure 4.9(b).

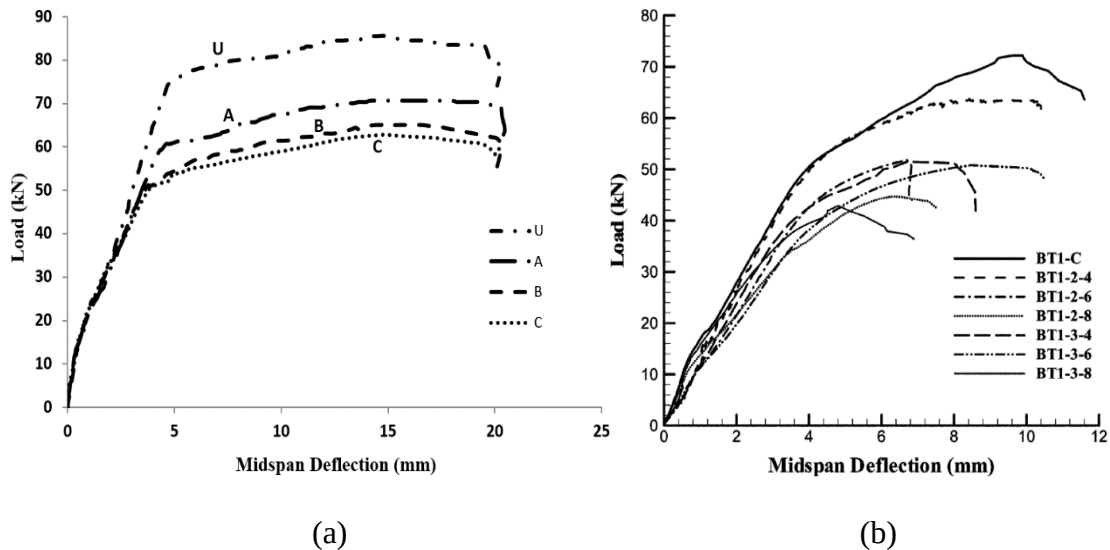


Figure 4.9: Load vs. midspan deflection plots (a) Present study (b) Azad et al. [54]

Table 4.2: Summary of the flexural testing results for all control beams.

<i>Beam ID</i>	<i>Actual mass losses</i>	<i>Average Crack width</i>	<i>At cracking</i>		<i>At yielding</i>		<i>At ultimate strength</i>	
	%	C_{avg} (mm)	P_{cr} (kN)	δ_{cr} (mm)	P_y (kN)	δ_y (mm)	P_{ult} (kN)	δ_{ult} (mm)
<i>UU</i>	0	0	19.5	0.83	75.1	5.5	85.5	14
<i>CUA</i>	9.8	0.7	17.4	0.72	59.3	4.7	70.7	14.3
<i>CUB</i>	17.4	1.2	16.4	0.61	48.2	4.1	65.4	14.9
<i>CUC</i>	20.8	1.5	16.1	0.56	46.5	3.8	62.8	15

Due to accelerated corrosion, the corroded beams CUA, CUB, and CUC yield and ultimate loads were decreased by 21 and 17%, 36 and 24%, and 38 and 27%, respectively, of the corresponding values of the main control beam (UU). The yield and ultimate strengths of corroded RC beam specimens were reduced at an average rate of 2 and 1.5%, respectively, per 1% of reinforcing mass losses. [Figure 4.10](#) shows a significant decrease in ultimate load-bearing capacities of the corroded beam specimens as a function of reinforcing corrosion degree, indicating that mass loss substantially impacts the load-bearing ability of corroded beam specimens. The combined effect of loss of bond (which occurs at a relatively lower degree of corrosion and does not increase with the degree of reinforcement corrosion) and loss of cross-section of reinforcing steel bars is confirmed by a relatively lower reduction in load-carrying capacity at a higher degree of reinforcement corrosion (which is proportional to the mass loss).

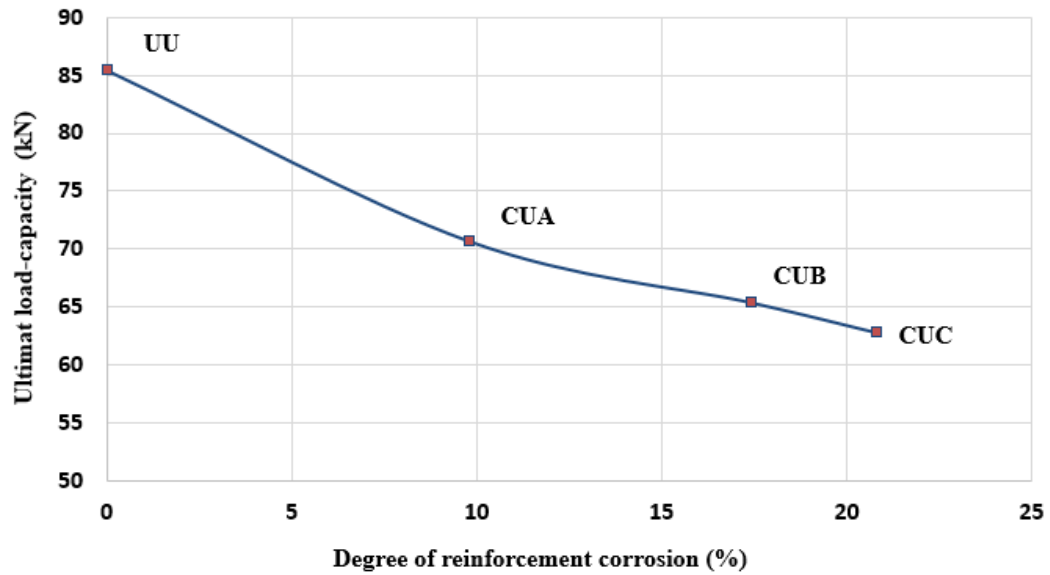


Figure 4.10: Reduction of ultimate load capacity with a degree of corrosion.

4.4.1.2 Failure mechanisms

The failure mechanism of corroded specimens CUA, CUB, and CUC are comparable to that of virgin beam (UU), as illustrated in Figure 4.11. The beams show large flexural cracks developed in the constant moment zone after the reinforcing steel yielding, indicating a typical failure mode of an under-reinforced concrete beams. The yielding of corroded bars has occurred first, followed by crushing concrete at the beams' compression side. It was observed that concrete spalling occurred at the end of the flexural testing due to concrete crushing at the compression zone.



(a)



(b)



(c)



(d)

Figure 4.11: Failure mechanisms of beams (a) Control beam UU (b) Beam CUA (c) Beam CUB (d) Beam CUC.

4.4.1.3 Ductility

The ductility of the tested corroded beams was assessed using the ductility index (μ), which is estimated by dividing deflections at maximum loads by deflections at yielding loads. In addition, the energy absorption index (Ω) could be estimated by determining the area under load vs. midspan deflection curves up to the maximum point. Results of ductility and energy absorption indices are illustrated in [Table 4.3](#).

Table 4.3: Ductility of all control beam specimens.

<i>Beam ID</i>	<i>Values of deflection (mm)</i>		μ	<i>Nor. μ</i>	Ω (kN.mm)	<i>Nor. Ω</i>
	<i>Yield</i>	<i>Maximum</i>				
<i>UU</i>	5.5	14	2.55	1.00	938.4	1.00
<i>CUA</i>	4.7	14.3	3.04	1.20	852.1	0.91
<i>CUB</i>	4.1	14.9	3.63	1.43	788.4	0.84
<i>CUC</i>	3.8	15	3.95	1.55	766.5	0.82

It was noticed that corroded beams had higher ductility indices than the control beam (UU). The ductility index showed an increase with increasing of mass loss of the corroded specimens. The samples of corroded specimens named CUA, CUB, and CUC (average mass loss 9.8, 17.4, and 20.8%, respectively) illustrated ductility indices of 3.04, 3.63, and 3.95, representing 119, 142, and 155%, respectively, of control beam UU. In contrast, the results showed that the energy absorption index decreases with the increase in mass loss of the corroded beams. It proves that increasing the degree of reinforcement corrosion significantly lowers the ultimate capacities and energy absorption of corroded beams. The same control beams' energy absorption indices showed 91, 84, and 82% of that of the UU beam, respectively. The results of the ductility and energy absorption indices of corroded control beams match with the Elghazy et al. [49] findings.

4.4.1.4 Stiffness

Stiffness was roughly defined as the slope of the elastic component of the load vs. mid-span deflection curve obtained from the flexural testing of corroded beam specimens. The stiffness graph in [Figure 4.12](#) shows the impact of steel corrosion damage on the stiffness of beam specimens. It is noticed that the stiffness of the corroded beams A, B, and C was

decreased by 10.3, 13.0, and 14.4%, respectively. This demonstrates the slight influence of corrosion damage on the stiffnesses of corroded beams.

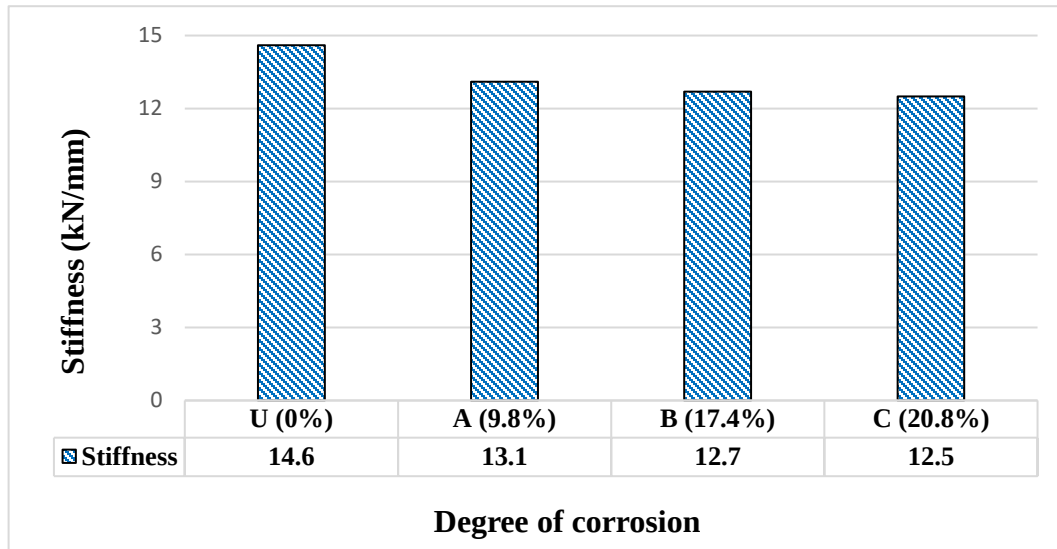


Figure 4.12: Stiffness degradation of the corroded beams.

4.4.2 Strengthened corroded beams with UHPC

4.4.2.1 Flexural behavior

The load vs midspan deflection plots for the UHPC-strengthened beam specimens of groups A, B, and C are illustrated in [Figure 4.13](#), [Figure 4.14](#), and [Figure 4.15](#), respectively. All the UHPC-strengthened beams displayed similar behavior under flexure. Each load-deflection curve contained three phases with two turning points that indicate the cracking of concrete in the tension zone and the yielding of tensile bars. The load-deflection curves also showed a good enhancement of the stiffness, yield, and ultimate load-bearing capacities of the UHPC- strengthened RC beams before steel yielding, which indicated their marginal contribution in the strengthened beam's response. [Table 4.4](#) shows the experimental results of the load-deflection data for the beam specimens corresponding to various corrosion degrees and UHPC thicknesses such as cracking, steel yielding, and

ultimate loads. It can be shown that all UHPC-strengthened beams fully restored their yield and ultimate strengths of the main control specimen. The crack load was significantly higher than that of the control beams, and cracks were almost focused in the middle span of the beams, as shown in [Table 4.4](#). The cracking load decrease with the increase in the degree of corrosion damage due to the loss of cross-section area and bonding of the steel bars. In addition, the cracking loads increase with the increase in the thickness of the UHPC layer, which proves the influence of the UHPC on the first crack initiation. This is because of the high tensile strength of the UHPC jacketing, which delayed the crack propagation and offered additional strength to the RC beams by resisting some of the tensile stresses. Results of [Table 4.4](#) were utilized to assess the effectiveness of using various UHPC strengthening thicknesses and the analytical modeling.

The gain in strengths was highly affected by the increase in the thickness of UHPC strengthening jacketing layers. Using only 20 mm UHPC jacketing layers increased the ultimate strength of corrosion-damaged RC beams by 18, 9, and 6 % for corrosion groups A, B, and C, respectively, of that of the uncorroded beam. While using jacketing layers with a thickness of 30 mm enhanced the ultimate strength of corroded RC beams significantly by 36, 32, and 28 % for corrosion groups A, B, and C, respectively, of that of the main virgin beam. The maximum enhancement of strengthened beams was for using UHPC jacketing layers with a thickness = 40 mm, which increased ultimate strength of corroded RC beams by 64, 60, and 58 % for corrosion groups A, B, and C, respectively, of that for the virgin control specimen.

Table 4.4: Flexural testing results of the strengthened beams.

Beam ID	Actual mass loss	Average Crack width	At cracking		At steel yielding		At ultimate load	
	%	C_{avg} (mm)	P_{cr} (kN)	δ_{cr} (mm)	P_y (kN)	δ_y (mm)	P_{ult} (kN)	δ_{ult} (mm)
UU	0	0	19.5	0.83	75.1	5.5	85.5	14
CUA	9.8	0.7	17.4	0.72	59.3	4.7	70.7	14.3
CUB	17.4	1.2	16.4	0.61	48.2	4.1	65.4	14.9
CUC	20.8	1.5	16.1	0.56	46.5	3.8	62.8	15
CSA-20	10.2	0.8	70.1	3.15	93.2	4.78	101.8	5.84
CSA-30	9.5	0.75	78.8	3.08	110.2	4.94	116	5.86
CSA-40	9.1	0.6	90.2	2.77	124.4	5.07	139.5	5.96
CSB-20	17.5	1.25	63.4	3.08	79.1	4.95	92	6.14
CSB-30	16.7	1.1	75.8	2.96	101.2	5.08	113.2	6.65
CSB-40	16.8	1.2	88.3	2.72	123.1	5.16	137.1	6.38
CSC-20	21.2	1.6	61.4	2.96	77.8	5.21	90.2	7.42
CSC-30	22.3	1.65	72.6	2.91	98.6	5.34	110	6.35
CSC-40	20.9	1.55	86.6	2.54	118.3	5.55	135.1	5.95

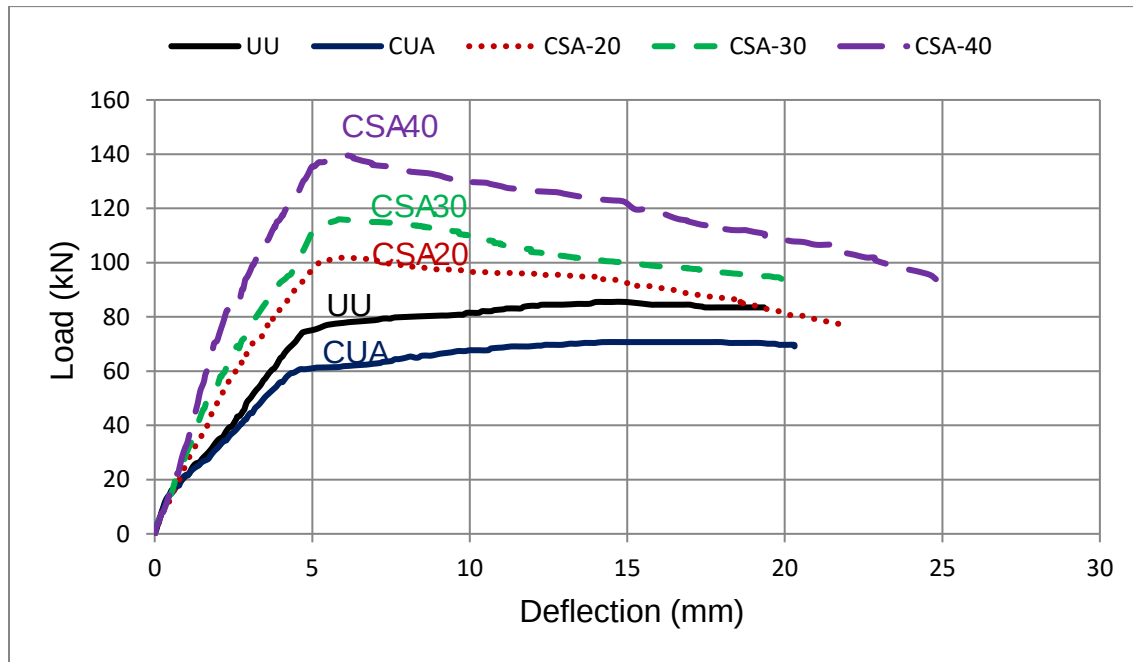


Figure 4.13: Load vs. deflection of strengthened group A beams.

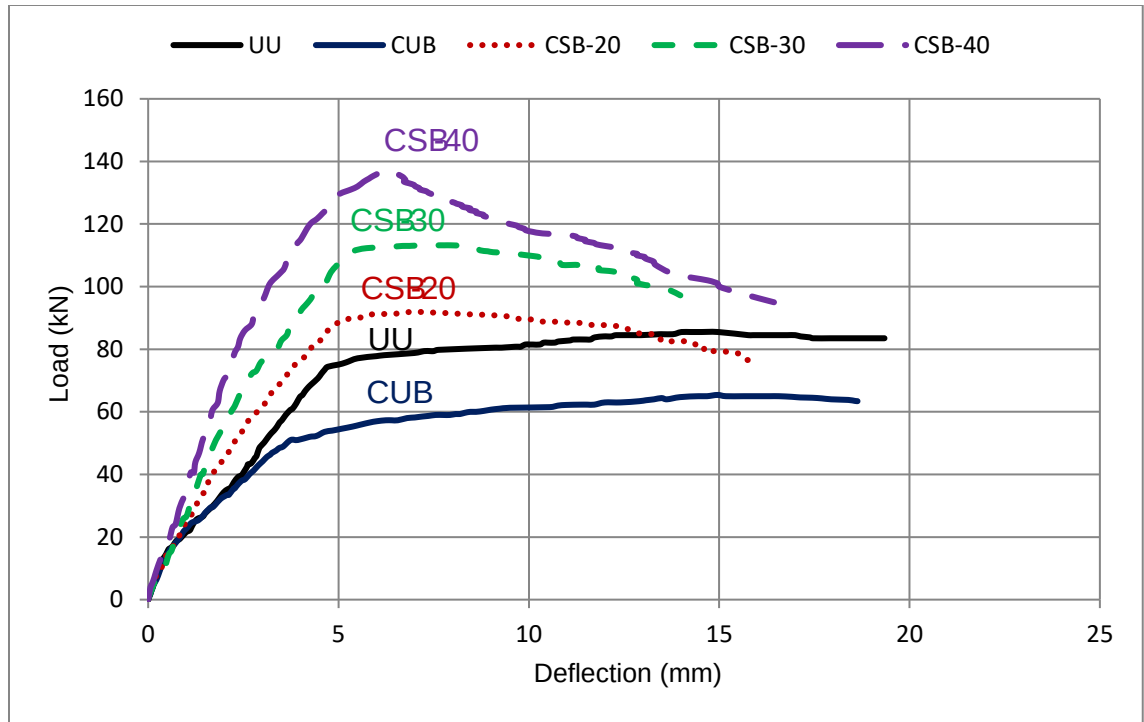


Figure 4.14: Load vs. deflection of strengthened group B beams.

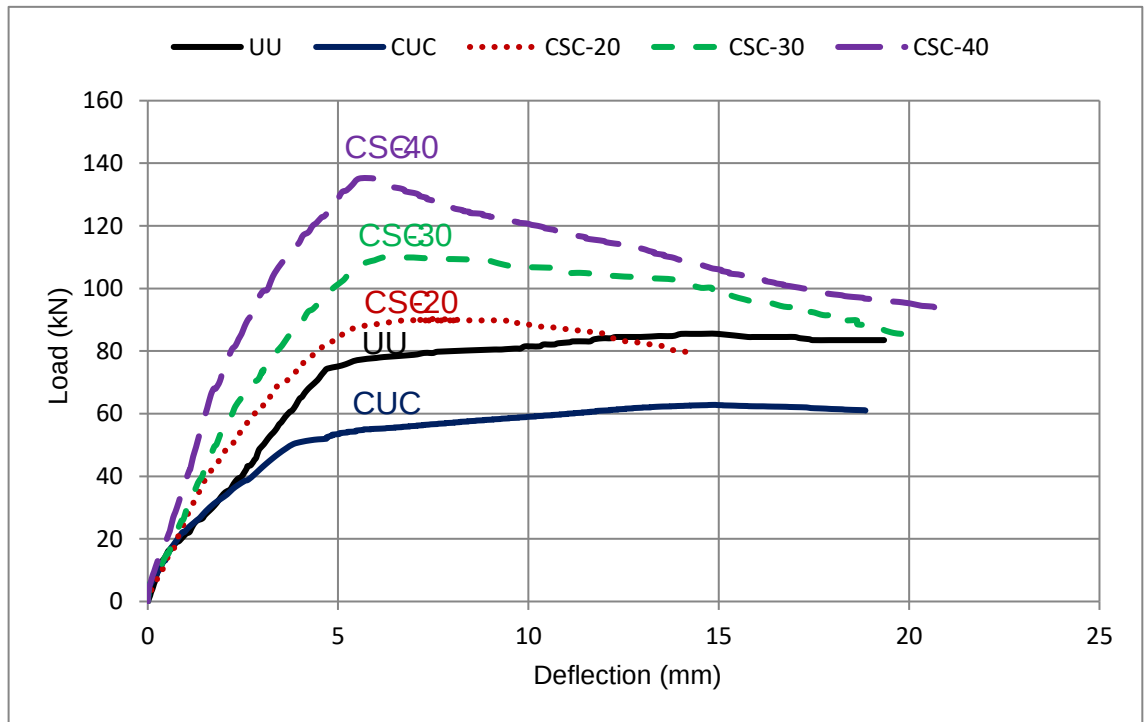


Figure 4.15: Load vs. deflection of strengthened group C beams.

4.4.2.2 Failure modes

The strengthened beams were failed under pure flexural behavior, with one major crack located within the constant moment zone. With the increase in the applied loads, sounds of steel fibers were observed, followed by the initiation of the major cracks. The cracks initiated at a load range between 60 kN (i.e., CSC-20) and 90 kN (i.e., CSA-40). This major crack became wider with increasing in the applied loads until it reached the strengthened beam's ultimate capacity. For Group A beams, the cracks initiated at a load of 60, 65, and 70 for the beams CSA-20, CSA-30, and CSA-40, respectively. The cracks widened with the increase of the applied load till failure, as presented in [Figure 4.16](#).



(a)



(b)



(c)

Figure 4.16: Failure modes of group A (a) CSA-20, (b) CSA-30, and (c) CSA-40.

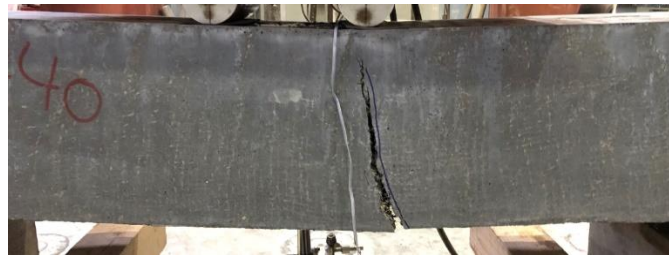
For Group B beams, the cracks initiated at a load of 60,65, and 70, for the beams CSB-20, CSB-30, and CSB-40, respectively. The major cracks occurred between the two concentric loads applied at the midspan of the beams CSB-20 and CSB-30, while it occurred beside the concentric load for the beam CSB-40. The cracks became wider with the load increasing until the failure of the beams, as illustrated in [Figure 4.17](#).



(a)



(b)



(c)

Figure 4.17: Failure modes of group B (a) CSB-20, (b) CSB-30, and (c) CSB-40.

For Group C beams, the cracks initiated at a load of 60,65, and 70, for the beams CSC-20, CSC-30, and CSC-40, respectively. The major cracks occurred between the two concentric

loads applied at the midspan of the beams CSC-20 and CSC-30. The cracks became wider with the load increasing until the failure of the beams. However, the major crack occurred at an approximate distance of 100 mm beside the concentric load for the specimen CSC-40. The crack grew and widened diagonally towards the concentric load till the failure of the beam, as illustrated in [Figure 4.18](#).



(a)



(b)



(c)

Figure 4.18: Failure modes of group C (a) CSC-20, (b) CSC-30, and (c) CSC-40.

4.4.2.3 Enhancement percentage of the load-bearing capacity

The positive effect of applying the UHPC jacketing layers on the load-carrying capacities of the corroded RC beam specimens can be seen clearly in the plots given in [Figure 4.19](#), [Figure 4.20](#), and [Figure 4.21](#). The strengthened corrosion-damage RC beams of Group A showed a 144, 164, and 197% enhancement in load-bearing capacities compared to unstrengthened corroded beam (CUA) for UHPC thicknesses of 20, 30, and 40 mm, respectively. The results showed an average load-bearing capacity enhancement rate of 24% per 10 mm of UHPC strengthening layer for group A corrosion-damage beams, as illustrated in [Figure 4.19](#).

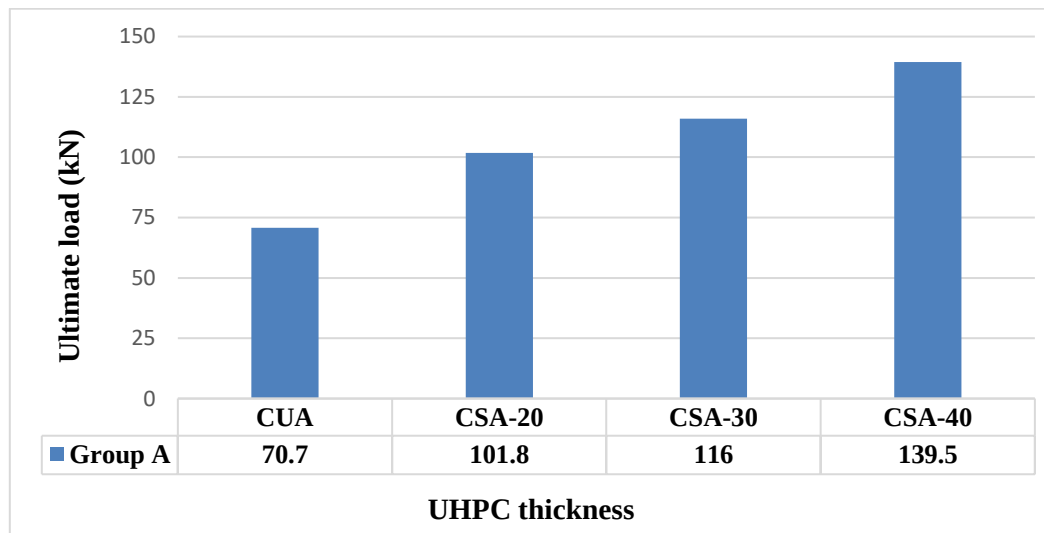


Figure 4.19: Ultimate load-bearing capacity of strengthened RC specimens (Group A).

In contrast, the strengthened RC beams of Group B corrosion degree enhanced 141, 173, and 210% in the load-bearing capacity compared to the unstrengthened corroded specimen (CUB) for UHPC thicknesses of 20, 30, and 40 mm, respectively. The results showed an average load-bearing capacity enhancement rate of 27% per 10 mm of UHPC strengthening layer for group B corrosion-damage beams, as shown in [Figure 4.20](#).

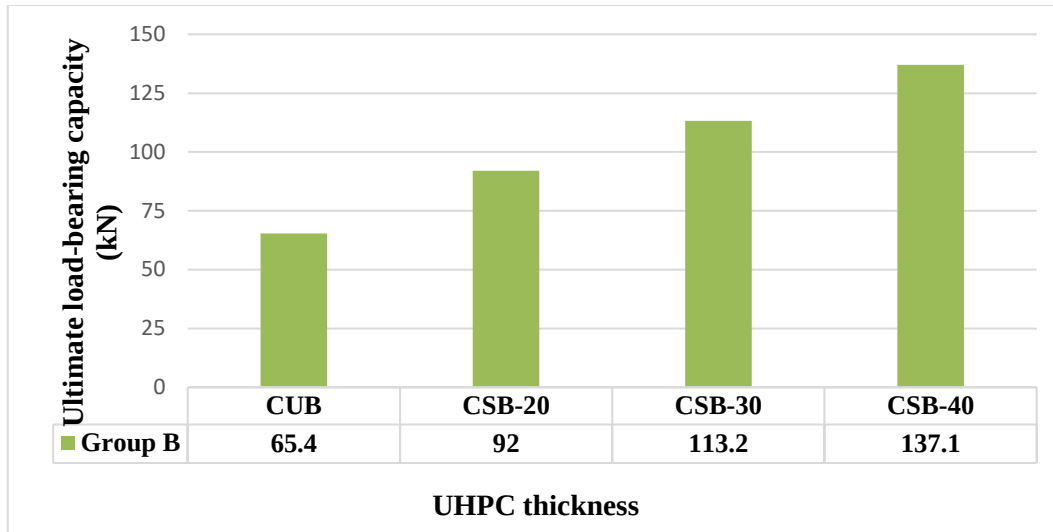


Figure 4.20: Ultimate load-bearing capacity of strengthened RC specimens (Group B).

The strengthened RC beams of Group C corrosion degree showed an enhancement of 144, 175, and 215% in the load-bearing capacities compared to the unstrengthened corroded specimen (CUC) for UHPC thicknesses of 20, 30, and 40 mm, respectively. The results showed an average load-bearing capacity enhancement rate of 29% per 10 mm of UHPC strengthening layer for group C corrosion-damage beams, as shown in Figure 4.21.

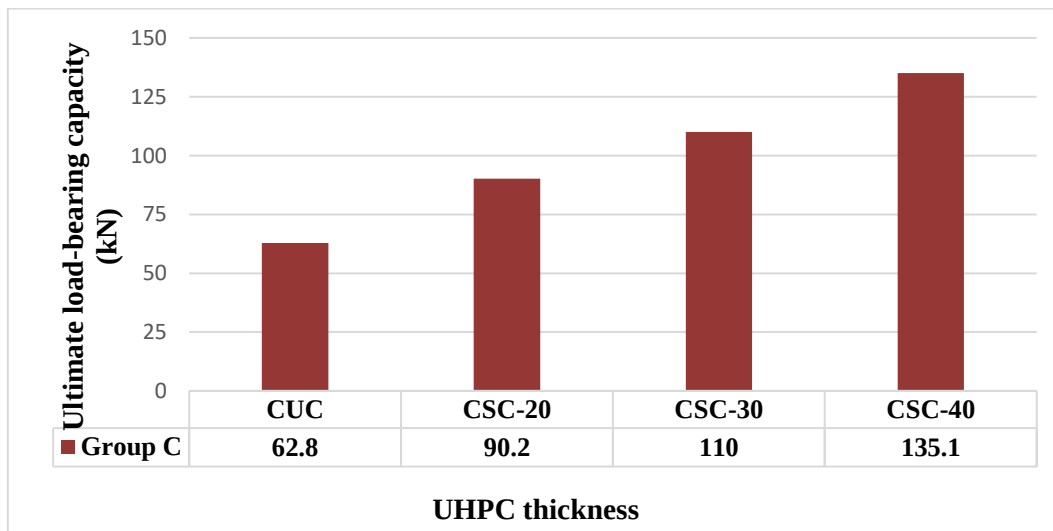


Figure 4.21: Ultimate load-bearing capacity of strengthened RC specimens (Group C).

It can be noticed that the enhancement ratio in load-bearing capacity of the strengthened specimens is increasing with the increase in the thickness of UHPC jacketing layers. The average improvement of the strengthened beams for all corrosion-damage groups is 142, 170, and 207 % for 20, 30, and 40 mm UHPC thickness, respectively, indicating a proportional enhancement in the load-carrying capacity with the increase in the thickness of UHPC jacketing layers. Moreover, it is observed that the enhancement percentage of the strengthened specimens increases with the increase in corrosion damage, indicating the efficiency of employing UHPC material to restore/strengthen a high level of corrosion-damaged RC beam specimens. It can be noticed that the enhancement ratio in the load-bearing capacity of the strengthened beams is increasing with the increase in the thickness of UHPC jacketing layers, as shown in Figure 4.22.

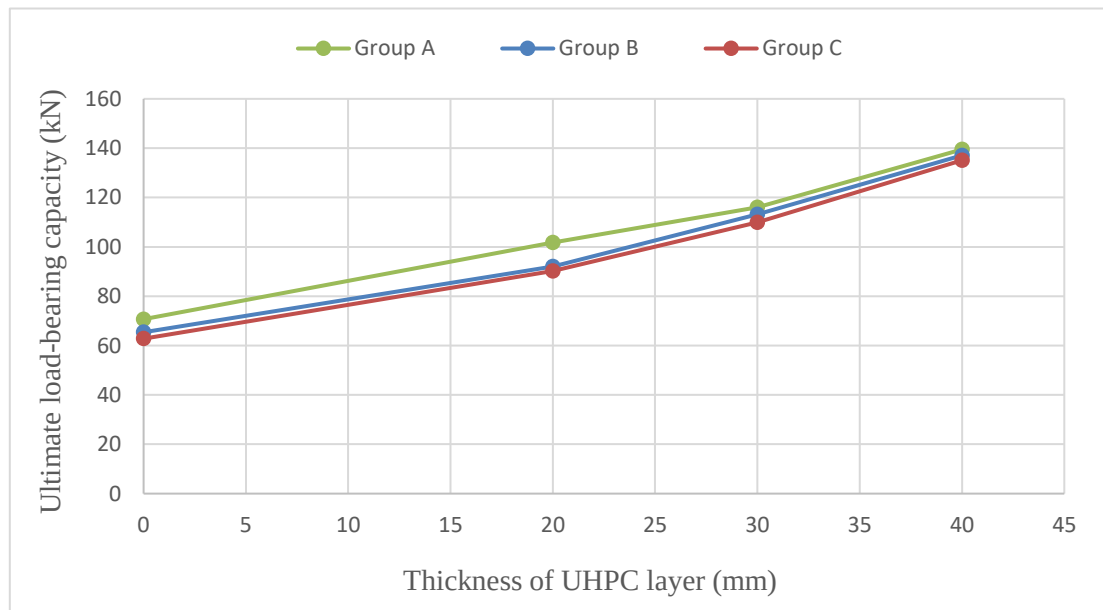


Figure 4.22: Variation of the ultimate capacity of strengthened RC beams.

4.4.2.4 stiffness

The slope of the elastic portion of the load vs. mid-span deflection curve obtained from the results of tested beams tested in flexure was roughly defined as stiffness. The stiffness graphs in Figure 4.23, Figure 4.24, and Figure 4.25 show the influence of corrosion damage and strengthening regimes on the stiffness of the RC beams. The beneficial effect of UHPC-strengthening techniques on the stiffness of corroded RC beams followed the same positive trend as the influence on the ultimate load-bearing capacities of the beams, as illustrated in Figure 4.26. For instance, applying only a 20 mm UHPC thickness to the beam CSA-20 can enhance the stiffness by 48 percent (from 13.1 to 19.4), while applying 40mm improves the stiffness of the beam by 112 percent (from 13.1 to 27.7) as the beam CSA-40.

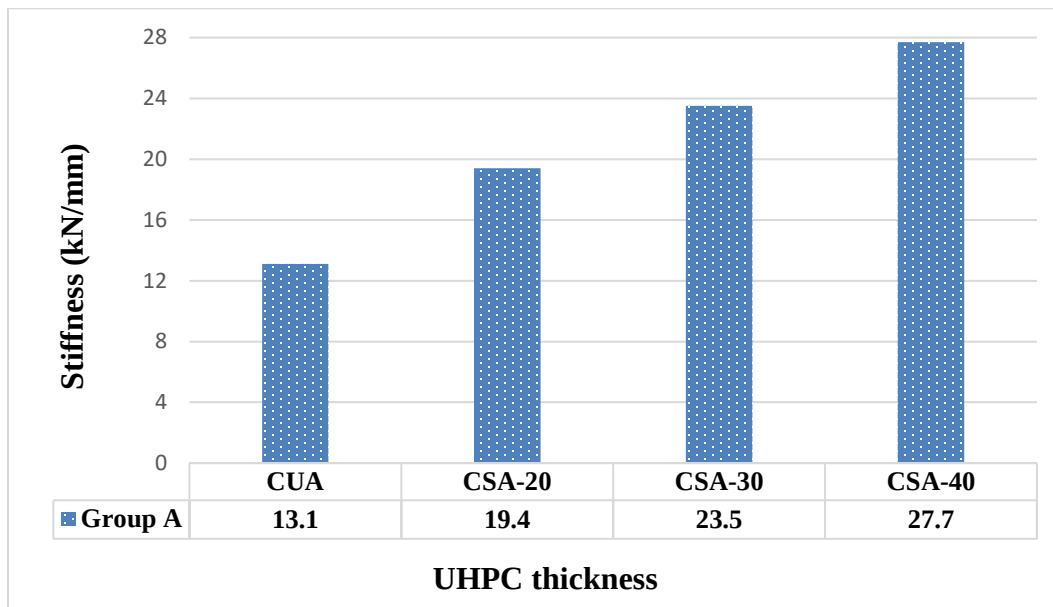


Figure 4.23: Stiffness enhancement for the strengthened beams of group A.

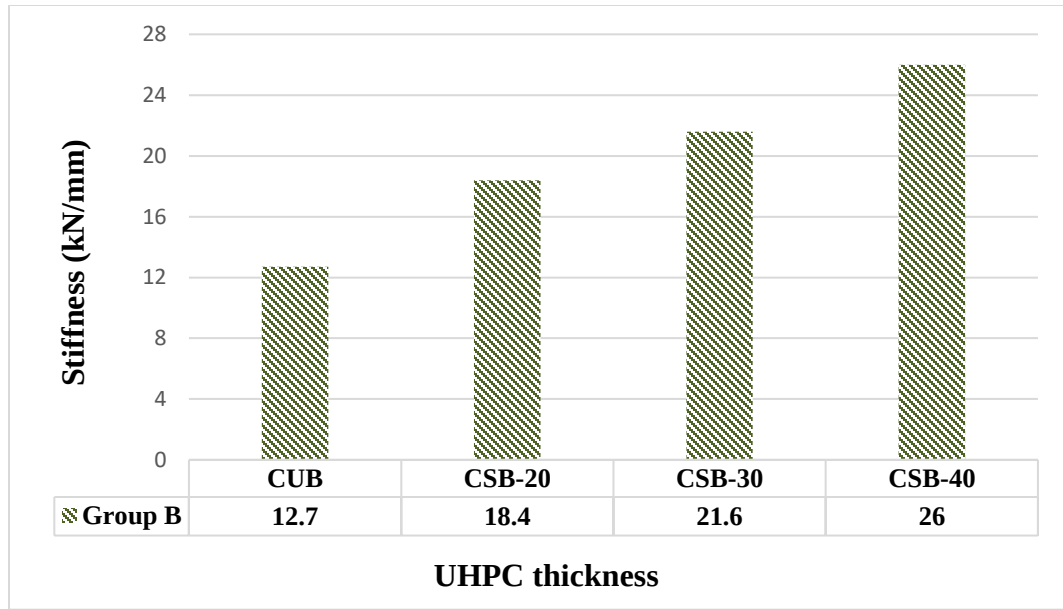


Figure 4.24: Stiffness enhancement for the strengthened beams of group B.

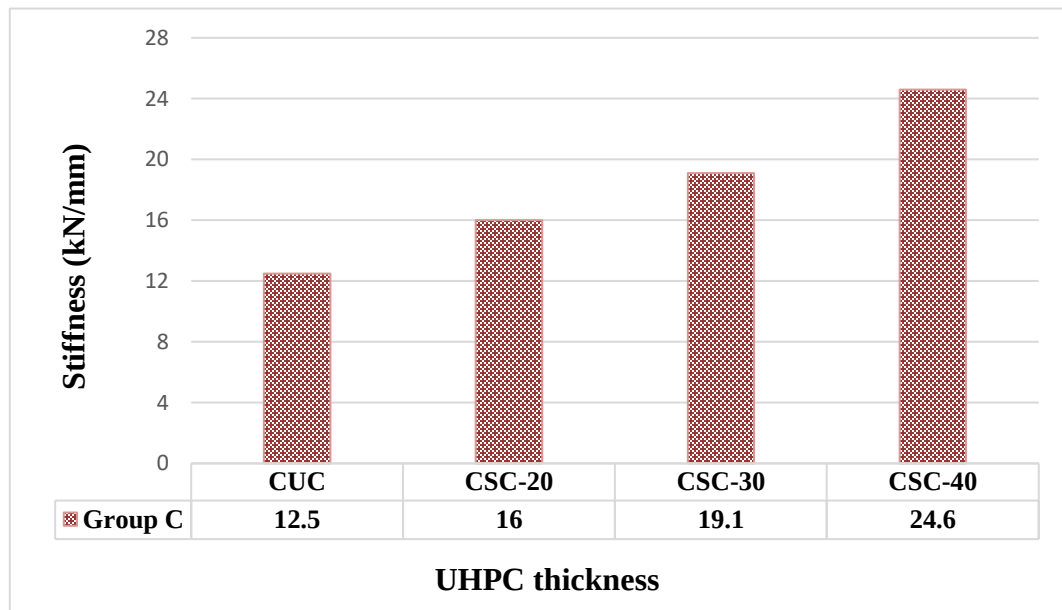


Figure 4.25: Stiffness enhancement for the strengthened beams of group C.

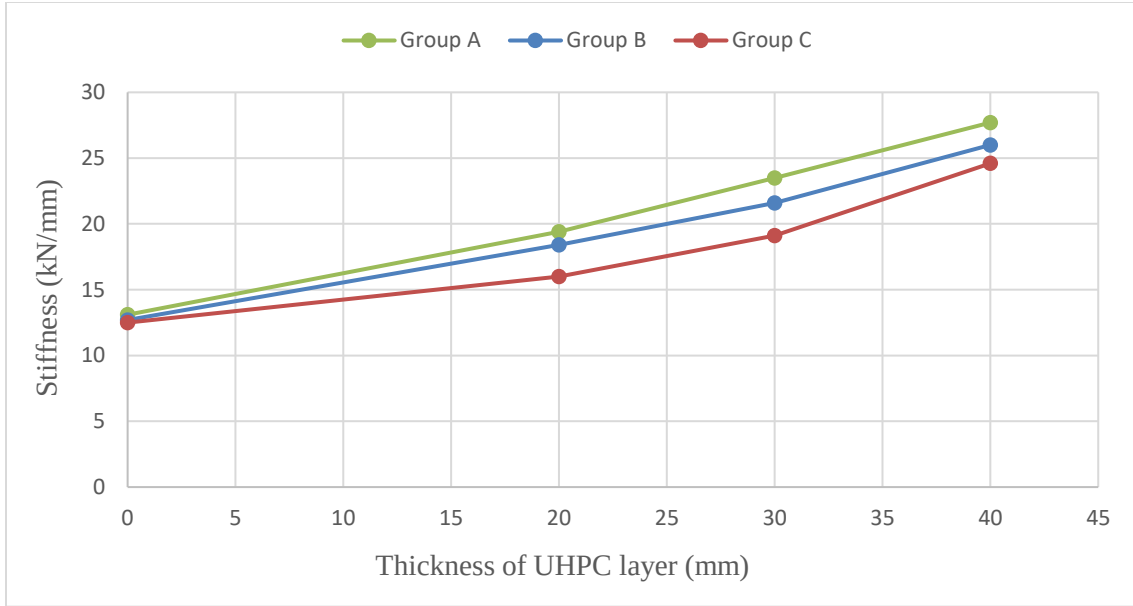


Figure 4.26: Stiffness enhancement of strengthened RC beams.

4.4.2.5 Ductility and energy absorption

The ductility of the RC beams was estimated using the ductility index (μ), which is determined as deflection at maximum loads divided by deflection at yielding loads. In addition, the energy absorption index (Ω) can be assessed by calculating the area under the load vs. deflection curves up to ultimate loads. The ductility of the tested beams was evaluated using both indices. Results of ductility and energy absorption indices are shown in Table 4.5.

It was observed that the corroded control beams had greater ductility indices than the uncorroded control beam (UU). The ductility index increased with increasing in corrosion-damage degree. The beams strengthened using UHPC layers showed a significant decrease in the ductility indices, which vary between 42 and 56% of that of the uncorroded beam UU while the energy absorption indices decreased by 38– 60% of that for the UU beam.

Table 4.5: Ductility and energy absorption of strengthened beams.

<i>Beam ID</i>	<i>Values of deflection (mm)</i>		μ	<i>Nor. μ</i>	Ω (kN.mm)	<i>Nor. Ω</i>
	<i>Yield</i>	<i>Maximum</i>				
<i>UU</i>	5.5	14	2.55	1.00	938.4	1.00
<i>CUA</i>	4.7	14.3	3.04	1.20	852.1	0.91
<i>CUB</i>	4.1	14.9	3.63	1.43	788.4	0.84
<i>CUC</i>	3.8	15	3.95	1.55	766.5	0.82
<i>CSA-20</i>	4.78	5.84	1.22	0.48	356.6	0.38
<i>CSA-30</i>	4.94	5.86	1.19	0.47	407	0.43
<i>CSA-40</i>	5.07	5.96	1.18	0.46	524.8	0.56
<i>CSB-20</i>	4.85	6.14	1.27	0.49	408.4	0.44
<i>CSB-30</i>	5.08	6.65	1.31	0.51	530.3	0.57
<i>CSB-40</i>	5.16	6.38	1.24	0.49	564.7	0.60
<i>CSC-20</i>	5.21	7.42	1.42	0.56	467.2	0.50
<i>CSC-30</i>	5.34	6.35	1.19	0.47	491.7	0.52
<i>CSC-40</i>	5.55	5.95	1.07	0.42	523.8	0.56

4.4.3 Repaired corroded beams with UHPC

4.4.3.1 Flexural behavior

The load vs. midspan deflection plots for the UHPC-repaired specimens of groups A, B, and C are illustrated in [Figure 4.27](#), [Figure 4.28](#), and [Figure 4.29](#), respectively. All the UHPC-repaired specimens of groups A, B, and C displayed similar behavior. Each load-deflection curve contained three phases with two turning points that indicate the cracking of concrete in the tension zone and the ultimate loads of the tested beams. The load-deflection curves also showed a good enhancement of the stiffness, yield, and ultimate load-carrying capacity of the UHPC-repaired specimens before steel yielding, indicating their marginal contribution in the repaired beam's response. [Table 4.6](#) shows the

experimental results of the load-deflection data for the beam specimens corresponding to various corrosion degrees and UHPC thicknesses such as cracking, steel yielding, and ultimate loads. It can be shown that all the UHPC-repaired beams were fully restored their ultimate strengths of the main control beam except for the beams CRA-20, CRB-20, and CRC-20 since the UHPC thickness is very small (only 20 mm). The crack load was significantly higher than that of the control beams, and cracks were almost focused in the middle span of the beams. The cracking load decrease with the increase in the degree of corrosion damage due to the loss of cross-section area and bonding of the steel bars. In addition, the cracking loads increase with the increase in the thickness of the UHPC layer, which proves the influence of the UHPC on the first crack initiation. This is because of the high tensile strength of the UHPC layer, which delayed the crack propagation and offered additional strength to the RC beams by resisting some of the tensile stresses. The gain in strengths was highly affected by the increase in the thickness of UHPC strengthening jacketing layers. Using only a 40 mm UHPC layer at the bottom increased the ultimate strength of corroded RC beams by 9.1, 7.6, and 6.6 % for corrosion groups A, B, and C, respectively, of that of the uncorroded beam. While using a UHPC layer with a thickness of 60 mm enhanced the ultimate strength of corroded RC beams significantly by 32, 15, and 14 % for corrosion groups A, B, and C, respectively, of that of the main control, beam UU. The minimum enhancement of the repaired beams was for using UHPC layer with a thickness of 20 mm, which increased the ultimate strength of corroded RC beams by 17, 21, and 13.4 % for corrosion groups A, B, and C, respectively, of that of the corresponding control corroded beams.

Table 4.6: Flexural testing results of all repaired beams.

Beam ID	Actual mass loss	Average Crack width	At cracking		At steel yielding		At ultimate load	
	%	C_{avg} (mm)	P_{cr} (kN)	δ_{cr} (mm)	P_y (kN)	δ_y (mm)	P_{ult} (kN)	δ_{ult} (mm)
UU	0	0	19.5	0.83	75.1	5.5	85.5	14
CUA	9.8	0.7	17.4	0.72	59.3	4.7	70.7	14.3
CUB	17.4	1.2	16.4	0.61	48.2	4.1	65.4	14.9
CUC	20.8	1.5	16.1	0.56	46.5	3.8	62.8	15
CRA-20	10	0.75	53.6	3.16	72.7	4.72	82.8	5.92
CRA-40	10.8	0.85	58	2.82	80.8	4.79	93.3	6.52
CRA-60	11.8	0.9	65.4	2.64	88.8	4.2	113	6.9
CRB-20	18.1	1.25	45.9	2.68	63.7	4.1	79.1	6.56
CRB-40	17.8	1.1	50.3	2.5	70.7	3.91	92	6.77
CRB-60	18.3	1.3	58	2.76	81	4.3	98	6.58
CRC-20	22.1	1.6	35.2	1.72	58.3	3.65	71.2	7.62
CRC-40	20.8	1.5	44.6	2.06	68.4	3.78	91.2	6.3
CRC-60	22.6	1.55	51.3	2.08	72.1	3.56	97.5	6.9

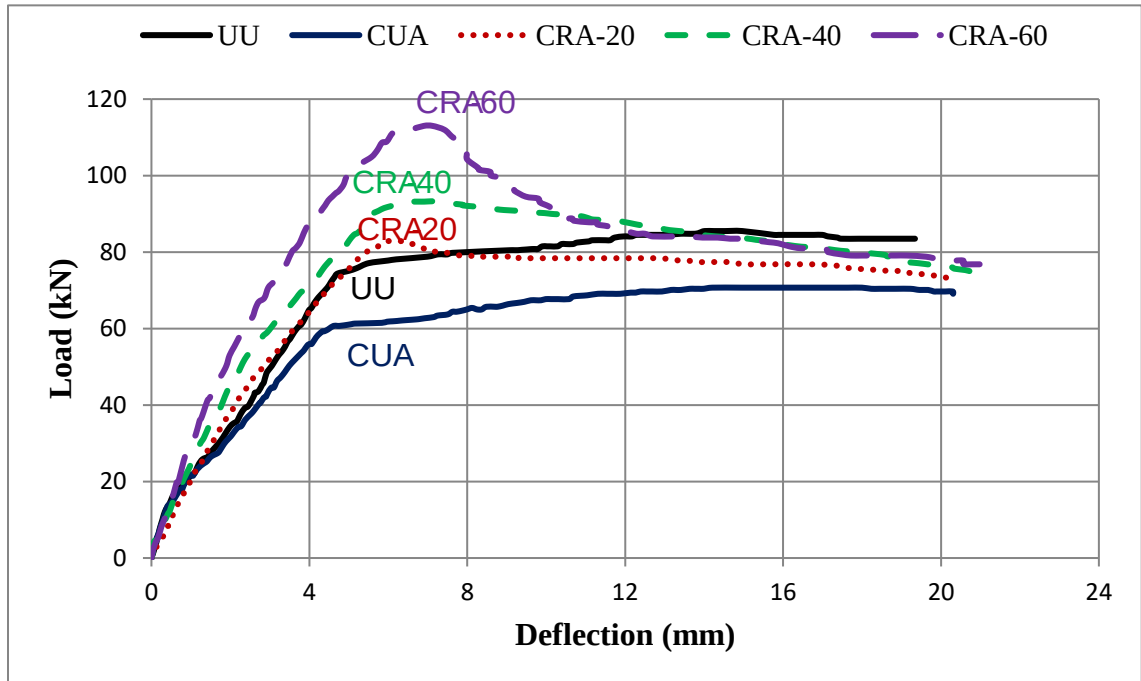


Figure 4.27: Load vs. deflection plots for repaired beams of group A.

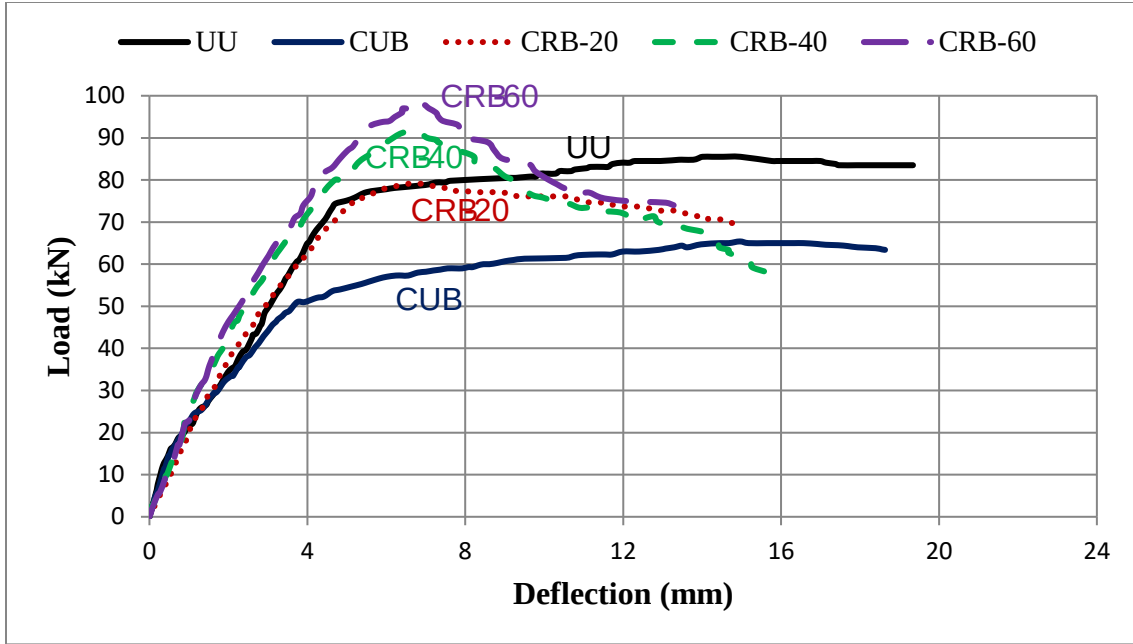


Figure 4.28: Load vs. deflection plots for repaired beams of group B.

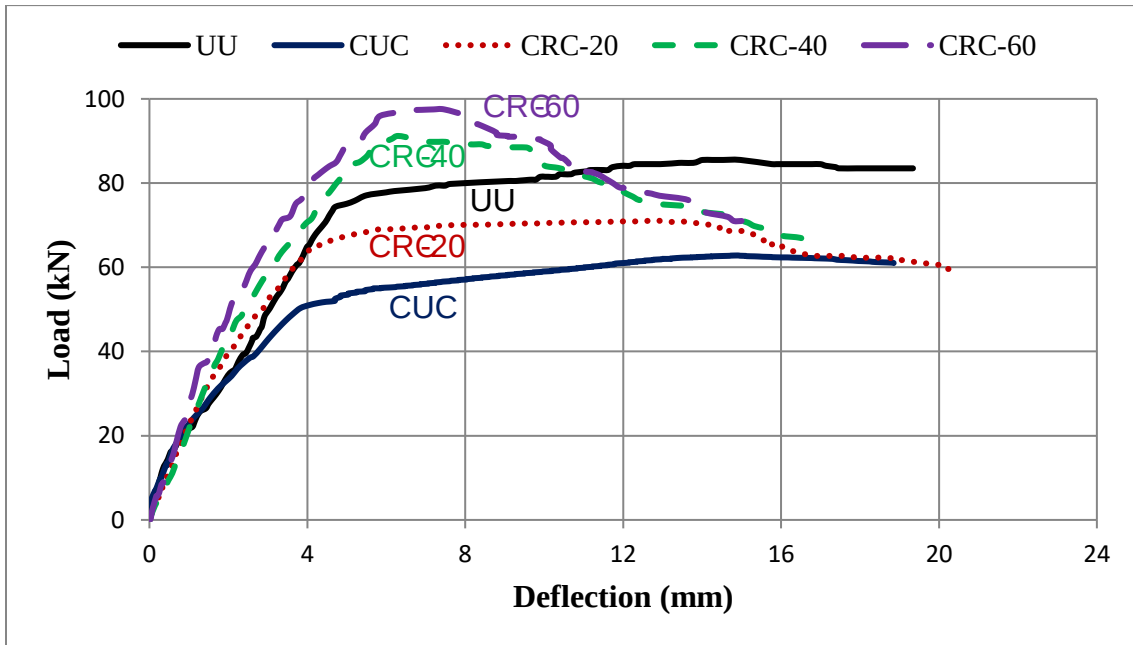


Figure 4.29: Load vs. deflection plots for repaired beams of group C.

4.4.3.2 Failure mechanism

The repaired specimens failed under pure flexural behavior, with one major crack located within the constant moment zone. With the increase in the applied loads, sounds of steel

fibers were observed, followed by the initiation of the major cracks. The cracks initiated at a load range between 35.2 kN (i.e., CRC-20) and 88.8 kN (i.e., CRA-60). This major crack became wider with the increase in the applied load until it reached the repaired beam's ultimate capacity. For Group A beams, the cracks initiated at a load of 53.6, 58, and 65.4 for the beams CRA-20, CRA-40, and CRA-60, respectively. The cracks widened with the increase of the applied load till failure, as illustrated in [Figure 4.30](#).



(a)



(b)



(c)

Figure 4.30: Failure modes of group A (a) CRA-20, (b) CRA-40, and (c) CRA-60.

For Group B beams, the cracks initiated at a load of 45.9, 50.3, and 58, for the beams CRB-20, CRB-40, and CRB-60, respectively. The major cracks occurred between the two concentric loads applied at the midspan of the beams CRB-20 and CRB-40, while it occurred beside the concentric load for the beam CRB-60. The cracks became wider with the load increasing until the failure of the beams, as illustrated in [Figure 4.31](#).



(a)



(b)



(c)

Figure 4.31: Failure modes of group B (a) CRB-20, (b) CRB-40, and (c) CRB-60.

For Group C beams, the cracks initiated at a load of 60,65, and 70, for the beams CRC-20, CRC-40, and CRC-60, respectively. The major cracks occurred between the two concentric

loads applied at the midspan of the beams CRC-20 and CRC-60. The cracks became wider with the load increasing until the failure of the beams. However, the major crack occurred at an approximate distance of 120 mm beside the concentric load for the specimen CRC-40. The crack grew and widened diagonally towards the concentric load till the failure of the beam, as illustrated in [Figure 4.32](#).

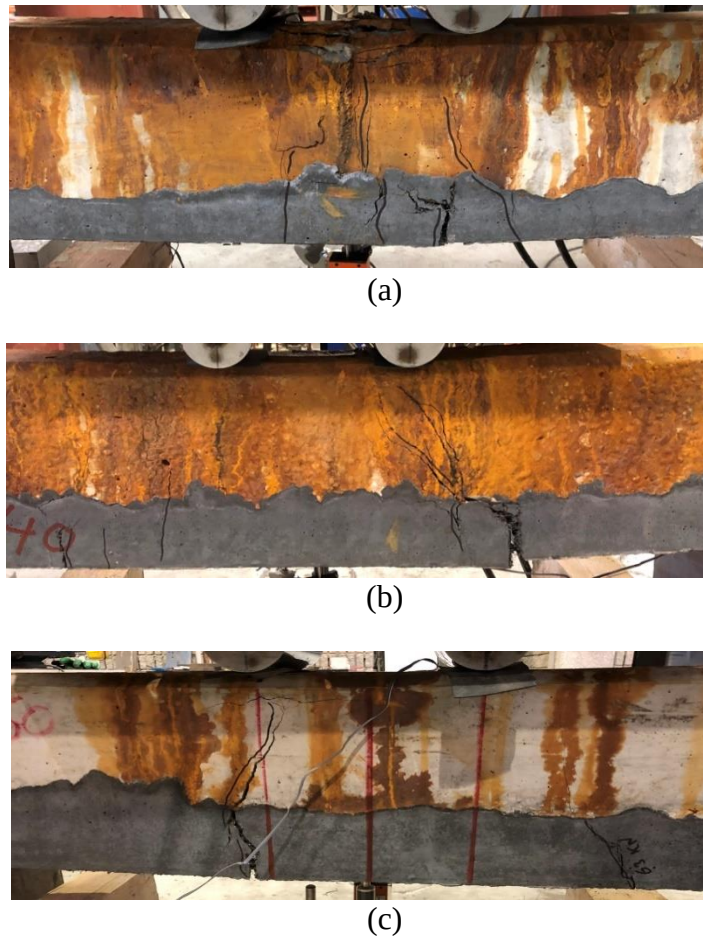


Figure 4.32: Failure modes of group C (a) CRC-20, (b) CRC-40, and (c) CRC-60.

4.4.3.3 Enhancement percentage of load-carrying capacities

The positive effect of applying the UHPC layer at bottom surfaces of the corroded beam specimens on the load-carrying capacities is obviously clear in the plots given in [Figure](#)

4.33, Figure 4.34, and Figure 4.35. The repaired corrosion-damage RC beams of Group A showed a 17, 32, and 60% enhancement in the load-bearing capacities compared to unrepaired corroded beam (CUA) for UHPC thicknesses of 20, 40, and 60 mm, respectively. The results showed an average load-bearing capacity enhancement rate of 20% per 20 mm of UHPC repairing layer for group A corrosion-damage beams, as shown in Figure 4.33.

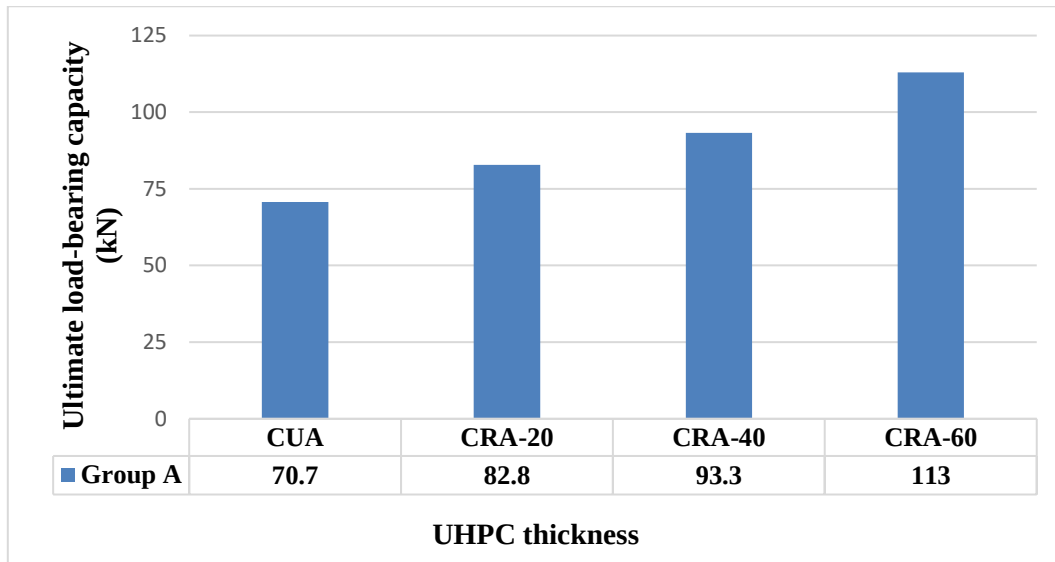


Figure 4.33: Ultimate load-bearing capacities of repaired specimens (Group A).

In contrast, the strengthened RC beams of Group B corrosion degree enhanced 21, 41, and 50% in the load-bearing capacities compared to the unrepaired corroded specimen (CUB) for UHPC thicknesses of 20, 40, and 60 mm, respectively. The results showed an average load-bearing capacity enhancement rate of 17% per 20 mm of UHPC repairing layer for group B corrosion-damage beams, as shown in Figure 4.34.

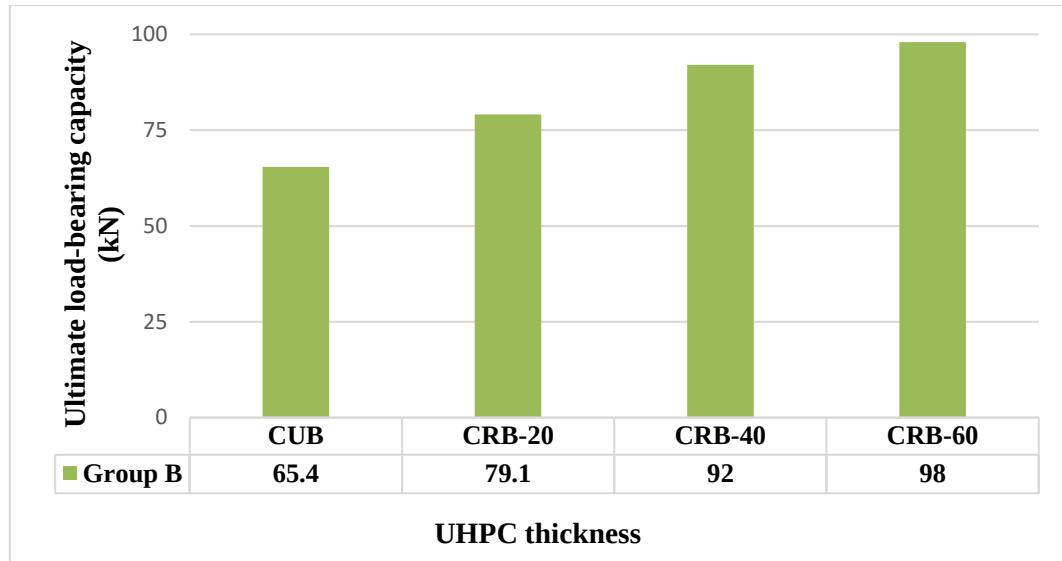


Figure 4.34: Ultimate load-bearing capacities of repaired specimens (Group B).

The strengthened RC beams of Group C corrosion degree showed an enhancement of 13, 45, and 55% in the load-bearing capacities compared to unrepaired corroded beam (CUC) for UHPC thicknesses of 20, 40, and 60 mm, respectively. The results showed an average load-bearing capacity enhancement rate of 18% per 20 mm of UHPC repairing layer for group C corrosion-damage beams, as illustrated in Figure 4.35.

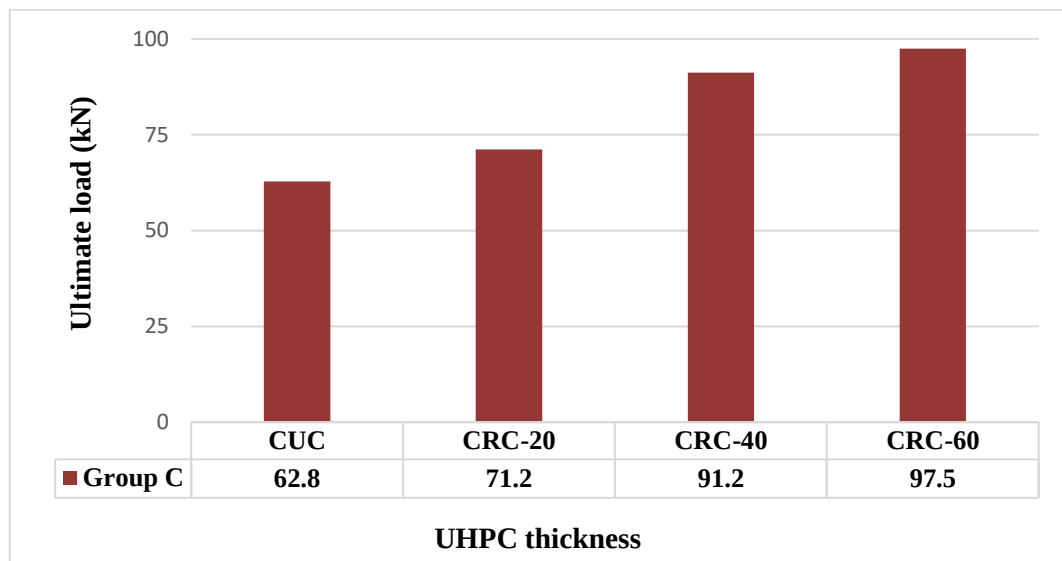


Figure 4.35: Ultimate load-bearing capacities of repaired specimens (Group C).

It can be noticed that the enhancement ratio in the load-bearing capacity of the repaired beams is increasing with the increase in the thickness of the UHPC layer. The average improvement of the repaired beams for all corrosion-damage groups is 17, 22, and 16 % for 20, 40, and 60 mm UHPC thickness, respectively, indicating a proportional enhancement in the load-bearing capacity with the increase in the thickness of the UHPC layer up to 40 mm than the average enhancement of the ultimate load decrease with the increase in the UHPC thickness above 40 mm. It can be noticed that the enhancement ratio in the load-bearing capacity of the repaired beams is increasing with the increase in the thickness of the UHPC layer, as shown in Figure 4.36. In addition, it is observed that the maximum enhancement percentage of the repaired beams is at the corrosion level of degree A then it decreases with the increase in the corrosion level, indicating the efficiency of employing a bottom layer of UHPC material to restore/strengthen corroded RC beams at a low to medium degree of corrosion damage and use the UHPC jacketing layers for the high degree of corrosion levels.

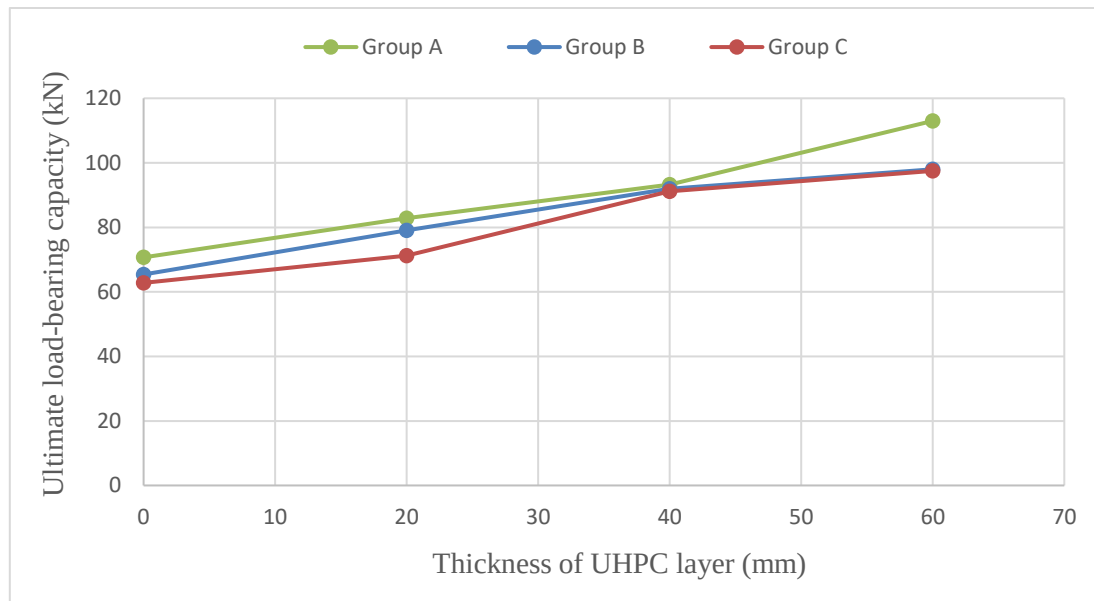


Figure 4.36: Variation of the ultimate capacity of repaired RC beams.

4.4.3.4 stiffness

The slope of the elastic portion of the load vs. mid-span deflection curve obtained from the results of tested beams tested in flexure was roughly defined as stiffness. The stiffness graphs in Figure 4.37, Figure 4.38, and Figure 4.39 show the influence of corrosion damage and repairing regimes on the stiffness of the RC beam specimens. The beneficial effect of UHPC-repairing techniques on the stiffness of the corroded RC beams followed the same positive trend as the effect on the ultimate load-bearing capacities of the specimens, as illustrated in Figure 4.40. For instance, applying only a 20 mm UHPC thickness to the beam CRA-20 can enhance the stiffness by 32 percent (from 13.1 to 17.3), while applying 60mm improves the stiffness of the beam by 81 percent (from 13.1 to 23.8) as the beam CRA-60.

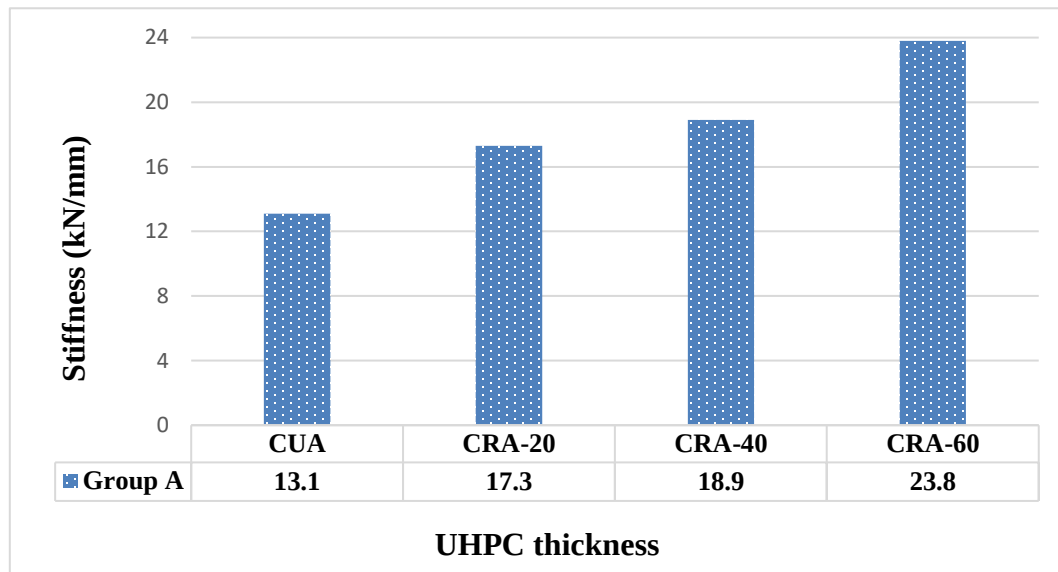


Figure 4.37: Stiffness enhancement for the repaired beams of group A.

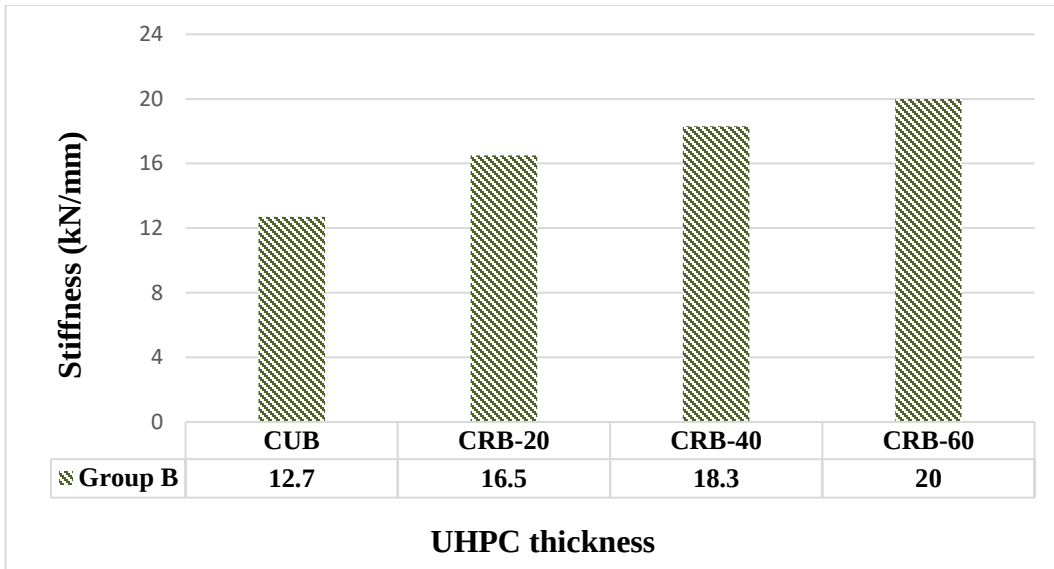


Figure 4.38: Stiffness enhancement for the repaired beams of group B.

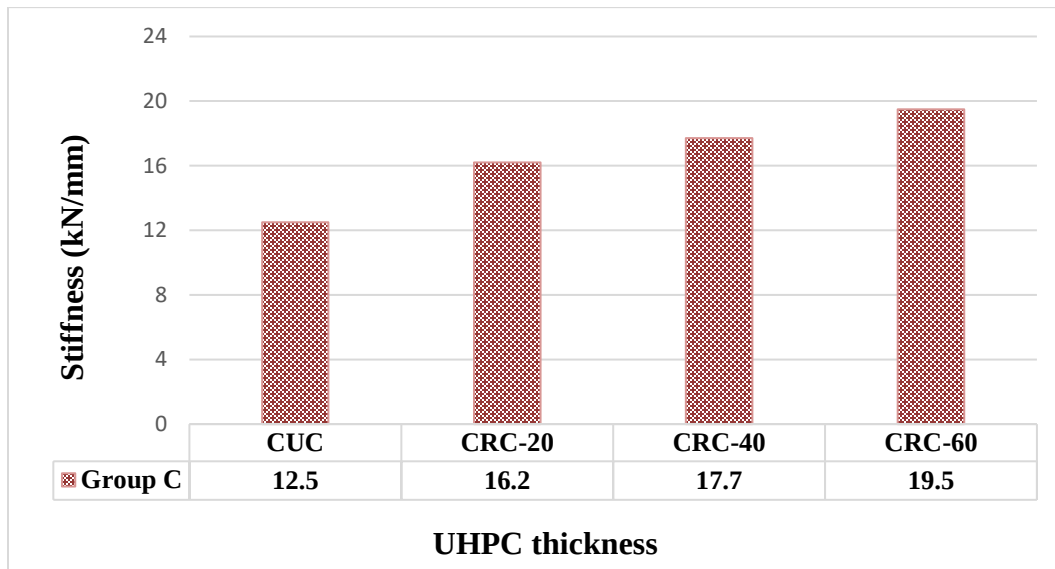


Figure 4.39: Stiffness enhancement for the repaired beams of group C.

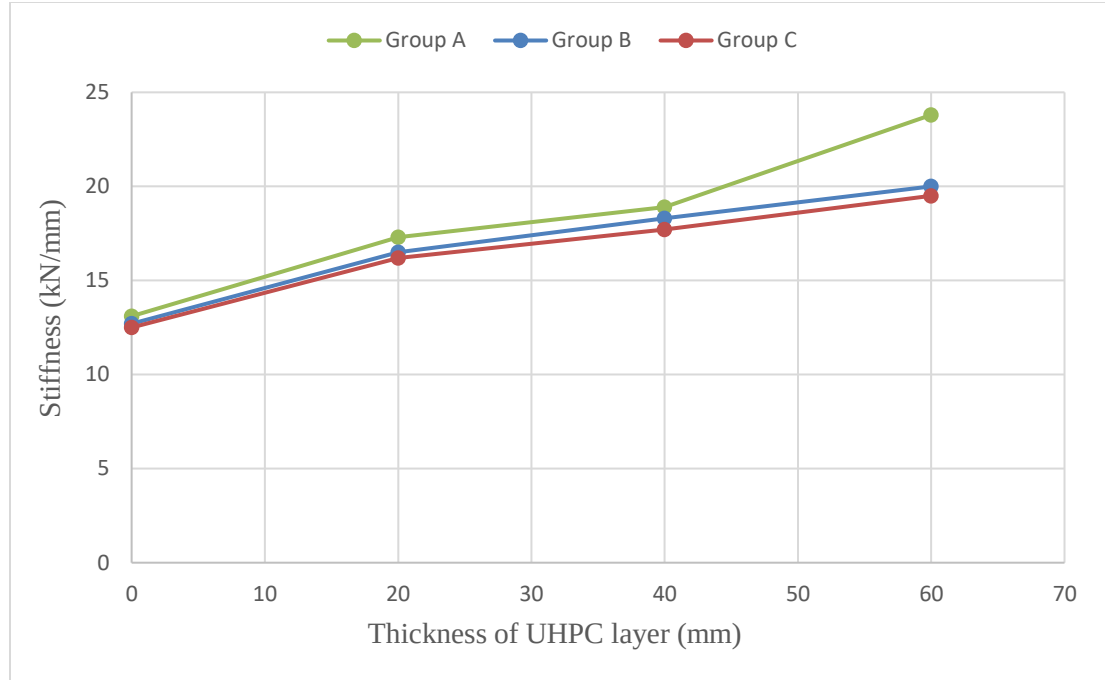


Figure 4.40: Stiffness enhancement of repaired RC beams.

4.4.3.5 Ductility and energy absorption

The ductility of the RC beams was estimated using the ductility index (μ) while the energy absorption index (Ω) were assessed by calculating the area under the load-deflection curve up to the ultimate load. The ductility of the tested beams was evaluated using both indices. Results of ductility and energy absorption indices of the repaired beams are illustrated in [Table 4.7](#). The specimens repaired with UHPC layer showed a significant reduce in ductility indices which vary between 39 and 65% of that for uncorroded control specimen UU whereas the energy absorption indices decreased by 31– 53% of that for the control beam UU.

Table 4.7: Ductility and energy absorption for repaired specimens.

<i>Beam ID</i>	<i>Values of deflection (mm)</i>		μ	<i>Nor. μ</i>	Ω (kN.mm)	<i>Nor. Ω</i>
	<i>Yield</i>	<i>Maximum</i>				
<i>UU</i>	5.5	14	2.55	1.00	938.4	1.00
<i>CUA</i>	4.7	14.3	3.04	1.20	852.1	0.91
<i>CUB</i>	4.1	14.9	3.63	1.43	788.4	0.84
<i>CUC</i>	3.8	15	3.95	1.55	766.5	0.82
<i>CRA-20</i>	4.72	5.92	1.25	0.49	288.7	0.31
<i>CRA-40</i>	4.79	6.52	1.36	0.53	379.5	0.40
<i>CRA-60</i>	4.2	6.9	1.64	0.65	496.4	0.53
<i>CRB-20</i>	4.1	6.56	1.60	0.63	317.6	0.34
<i>CRB-40</i>	3.91	6.77	1.73	0.68	396.4	0.42
<i>CRB-60</i>	4.3	6.58	1.53	0.60	429.2	0.46
<i>CRC-20</i>	3.65	7.62	2.09	0.82	337.8	0.36
<i>CRC-40</i>	3.78	6.3	1.67	0.65	349.2	0.37
<i>CRC-60</i>	3.56	6.9	1.94	0.76	445.5	0.47

4.4.4 Post-strengthened corroded beams with UHPC

4.4.4.1 Flexural behavior

The load vs. mid-span deflection plots for all beam specimens were drawn based on the data obtained during the flexural testing of the strengthened and post-strengthened RC beam specimens. Figure 4.41 and Figure 4.42 illustrate the load-deflection plots of the beams that belong to the corrosion groups B and C, respectively. The flexural performance of the uncorroded control beam specimen (UU) and the corroded-un-strengthened beams (CUA and CUB) are also compared. All the tested beams showed similar structural

behavior of under-reinforced concrete elements that failed under flexure. The load vs. mid-span deflection curves revealed a significant improvement in the stiffness, yield, and ultimate load-carrying capacities of the post-strengthened RC beam specimens. Table 4.8 illustrates the load vs. mid-span deflection data for the beam specimens of phases I and II, including cracking, steel yielding, and ultimate load. It can be seen that the yield and ultimate strengths of the post-strengthened beam specimens were totally recovered compared to control beams. The gain in strengths was highly affected by the increase in the thickness of UHPC strengthening jacketing layers for the post-strengthened corroded beams. The use of only 20 mm UHPC jacketing layers increased the ultimate strength of corroded beams by 10 and 8 % corrosion groups B and C, respectively, of the strengthened beams with the same thickness. While using jacketing layers with a thickness of 30 mm enhanced the ultimate strength of post-strengthened corroded beams by 4 and 6 % for corrosion groups B and C, respectively, of the strengthened beams with the same thickness. In contrast, using UHPC jacketing layers with a thickness of 40 mm enhanced the maximum strength of post-strengthened corroded specimens by 4 and 3 % for corrosion groups B and C, respectively, of that of the strengthened beams with the same thickness.

Table 4.8: Summary of flexural results for strengthened and post-strengthened beams.

<i>Beam ID</i>	<i>Actual mass loss</i>	<i>Average Crack width</i>	<i>At cracking</i>		<i>At steel yielding</i>		<i>At ultimate load</i>	
	%	C_{avg} (mm)	P_{cr} (kN)	δ_{cr} (mm)	P_y (kN)	δ_y (mm)	P_{ult} (kN)	δ_{ult} (mm)
<i>UU</i>	0	0	19.5	0.83	75.1	5.5	85.5	14
<i>CUB</i>	17.4	1.2	16.4	0.61	48.2	4.1	65.4	14.9
<i>CUC</i>	20.8	1.5	16.1	0.56	46.5	3.8	62.8	15

CSB-20	17.5	1.25	63.4	3.08	80.1	4.95	92	6.14
CSB-30	16.7	1.1	75.8	2.96	101.2	5.08	113.2	6.65
CSB-40	16.8	1.2	88.3	2.72	123.1	5.16	137.1	6.38
CSC-20	21.2	1.6	61.4	2.96	77.8	5.21	90.2	7.42
CSC-30	22.3	1.65	72.6	2.91	98.6	5.34	110	6.35
CSC-40	20.4	1.55	86.6	2.54	118.3	5.55	135.1	5.95
CPB-20	14.4	-	69.7	3.00	91.5	4.6	101.2	5.90
CPB-30	13.2	-	77.1	3.10	104.6	4.8	117.3	5.82
CPB-40	12.6	-	92.2	2.88	126.7	4.84	142.5	6.16
CPC-20	20.1	-	64.4	2.34	81	3.52	97	5.86
CPC-30	19.7	-	75.4	2.04	103	3.76	116.2	6.04
CPC-40	19.1	-	87.8	2.44	120	4.33	139.8	5.78

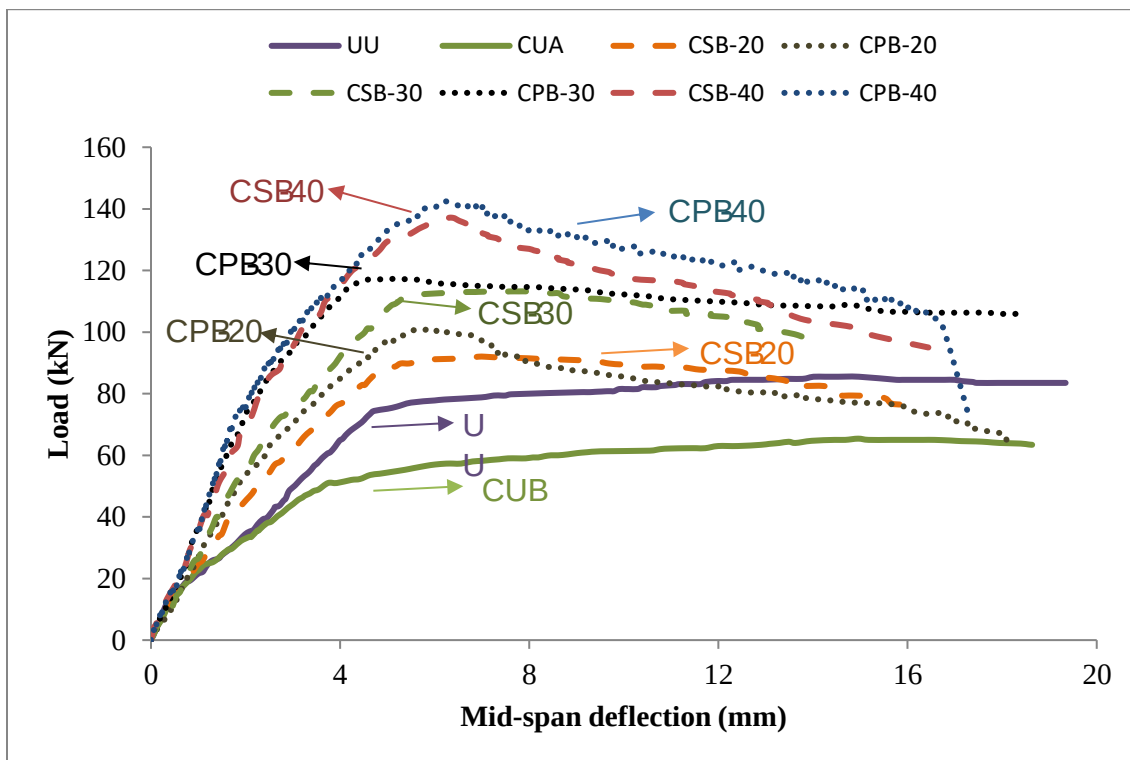


Figure 4.41: Load-deflection plots for group B specimens (Phase I & II).

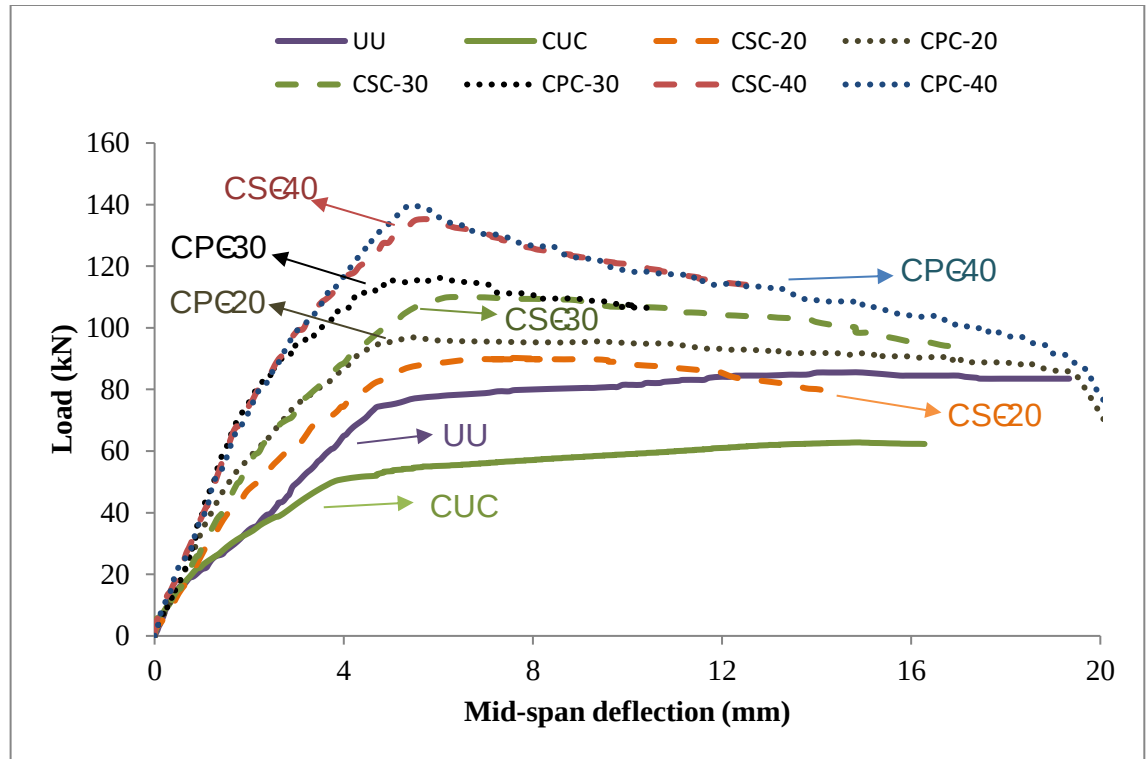


Figure 4.42: Load-deflection plots for group C specimens (Phase I & II).

4.4.4.2 Failure mechanism

All the post-strengthened beams were failed under pure flexural behavior with only one main crack. With the increase in the applied loads, sounds of steel fibers were observed, followed by the initiation of the major cracks. The cracks initiated at a load range between 64.4 kN (i.e., CPC-20) and 92.2 kN (i.e., CPB-40). This major crack became wider with the increase in the applied load until it reached the maximum capacity of the beam specimens. For Group B beams, the cracks initiated at a load of 69.7, 77.1, and 92.2 for the beams CPB-20, CPB-30, and CPB-40, respectively. The cracks widened with the increasing of the applied load till failure, as illustrated in Figure 4.43.



(a)



(b)



(c)

Figure 4.43: Failure modes of post-strengthened group B beams (a) CPB-20, (b) CPB-30, and (c) CPB-40.

For Group C beams, the cracks initiated at a load of 64.4, 75.4, and 87.8, for the beams CPC-20, CPC-30, and CPC-40, respectively. The major cracks occurred between the two concentric loads applied at the midspan of the beams CPC-20 and CPC-40. The cracks became wider with the load increasing until the failure of the beams. However, the major crack occurred under the concentric load for the beam CPC-30 and propagated diagonally towards the midspan of the beam, as illustrated in [Figure 4.44](#).



(a)



(b)



(c)

Figure 4.44: Failure modes of post-strengthened group C beams (a) CPC-20, (b) CPC-30, and (c) CPC-40.

4.4.4.3 Enhancement percentage of load-carrying capacities

Strengthening of corroded RC beams with UHPC layers had restored and exceeded the ultimate strengths of the uncorroded beam UU based on the thickness of UHPC material used. The strengthened beams of Group B showed a 41, 73, and 110% enhancement in the load-bearing capacities compared to the unstrengthened corroded specimen (CUB) for UHPC thicknesses of 20, 30 and 40 mm, respectively. The results showed an average load-

carrying capacity enhancement rate of 27% per 10 mm of UHPC strengthening layer for group B corrosion-damage beams, as presented in Figure 4.45. In contrast, the post-strengthened beams of Group B showed a 55, 79, and 118 % enhancement in the load-bearing capacities compared to unstrengthened corroded beam (CUB) for UHPC thicknesses of 20, 30 and 40 mm, respectively. The results showed an average load-carrying capacity enhancement rate of 30 % per 10 mm of UHPC strengthening layer for group B corrosion-damage beams, as shown in Figure 4.45.

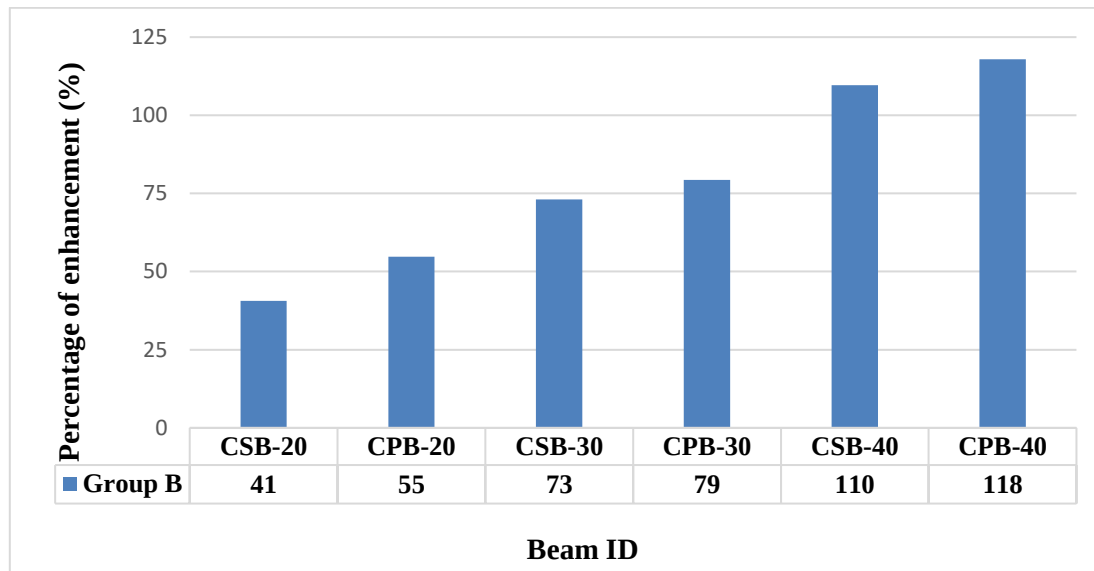


Figure 4.45: Enhancement percentage of the strengthened and post-strengthened RC beam specimens (Group B).

The strengthened beams of Group C showed a 44, 75, and 115% enhancement in the load-bearing capacities compared to the unstrengthened corroded specimen (CUC) for UHPC thicknesses of 20, 30 and 40 mm, respectively. The results showed an average load-carrying capacity enhancement rate of 29% per 10 mm of UHPC strengthening layer for group C corrosion-damage beams, as presented in Figure 4.46. In contrast, the post-strengthened beams of Group C showed a 54, 85, and 123 % enhancement in the load-

bearing capacities compared to the unstrengthened corroded specimen (CUC) for UHPC thicknesses of 20, 30 and 40 mm, respectively. The results showed an average load-carrying capacity enhancement rate of 31 % per 10 mm of UHPC strengthening layer for group C corrosion-damage beams, as shown in Figure 4.46.

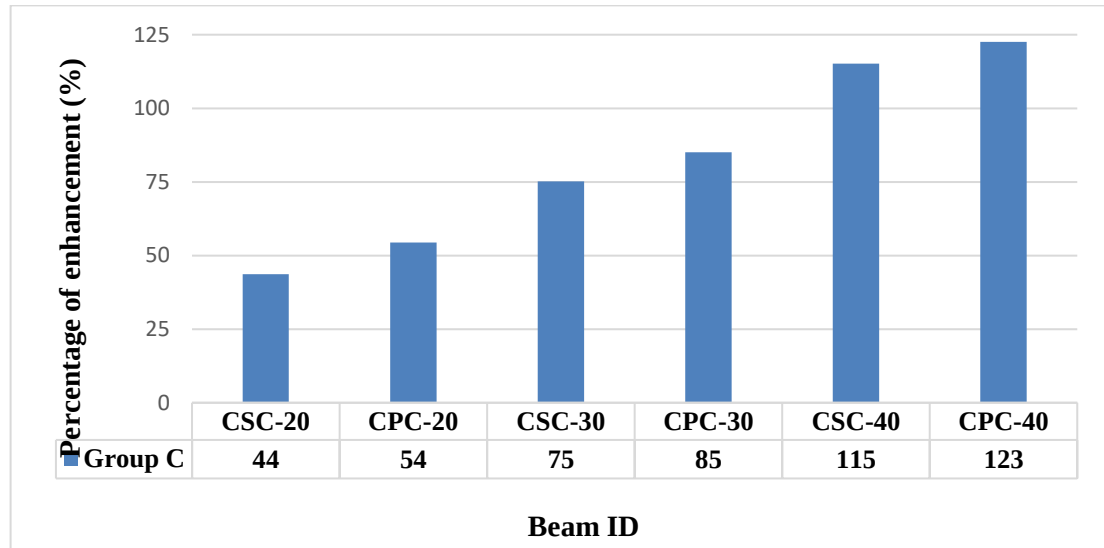


Figure 4.46: Enhancement percentage of the strengthened and post-strengthened RC beam specimens (Group C).

The enhancement ratio of the strengthened and post-strengthened RC beams in terms of the load-carrying capacities increases with the increase in the thickness of UHPC jacketing layers. The average enhancement of the strengthened beams for the corrosion-damage groups B and C is 42.5, 74, and 112.5 % for 20, 30, and 40 mm UHPC thickness, respectively, indicating a proportional enhancement in the load-bearing capacities with the increase in the thickness of UHPC jacketing layers. Furthermore, the average enhancement of post-strengthened specimens for the corrosion-damage groups B and C is 54.5, 82, and 120.5 % for 20, 30, and 40 mm UHPC thickness, respectively.

4.4.4.4 stiffness

The slope of the elastic portion of the load vs. mid-span deflection curve obtained from the results of tested beams tested in flexure was roughly defined as stiffness. The stiffness graphs in Figure 4.47 and Figure 4.48 show the impact of UHPC post-strengthened corrosion-damaged regimes on the stiffness of the RC beam specimens. It is followed the same positive trend as the effect on the ultimate load-bearing capacities of the beams, as illustrated in Figure 4.47 and Figure 4.48. Applying only a 20 mm UHPC thickness to the post-strengthened beam CPB-20 can enhance the stiffness by 47 percent instead of 28 % for the beam CSB-20. In contrast, applying a 40 mm UHPC thickness to the post-strengthened beam CPB-40 enhanced the stiffness by 101 percent instead of 81 % for the beam CSB-40.

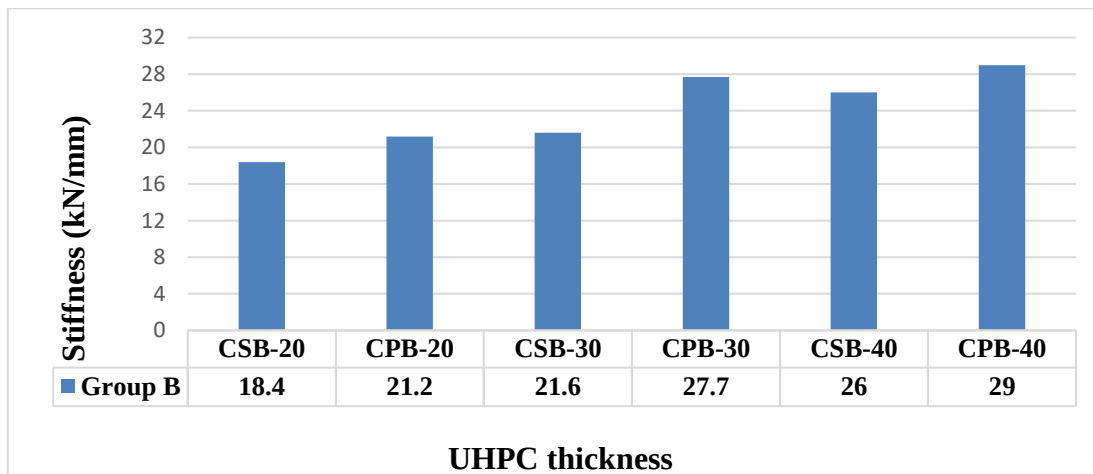


Figure 4.47: Stiffness enhancement for the strengthened and post-strengthened RC beams of group B.

Applying only a 20 mm UHPC thickness to the post-strengthened beam CPC-20 can enhance the stiffness by 45 percent instead of 11 % for the beam CSC-20. In contrast, applying a 40 mm UHPC thickness to the post-strengthened beam CPC-40 enhanced the stiffness by 92 percent instead of 71 % for the beam CSC-40.

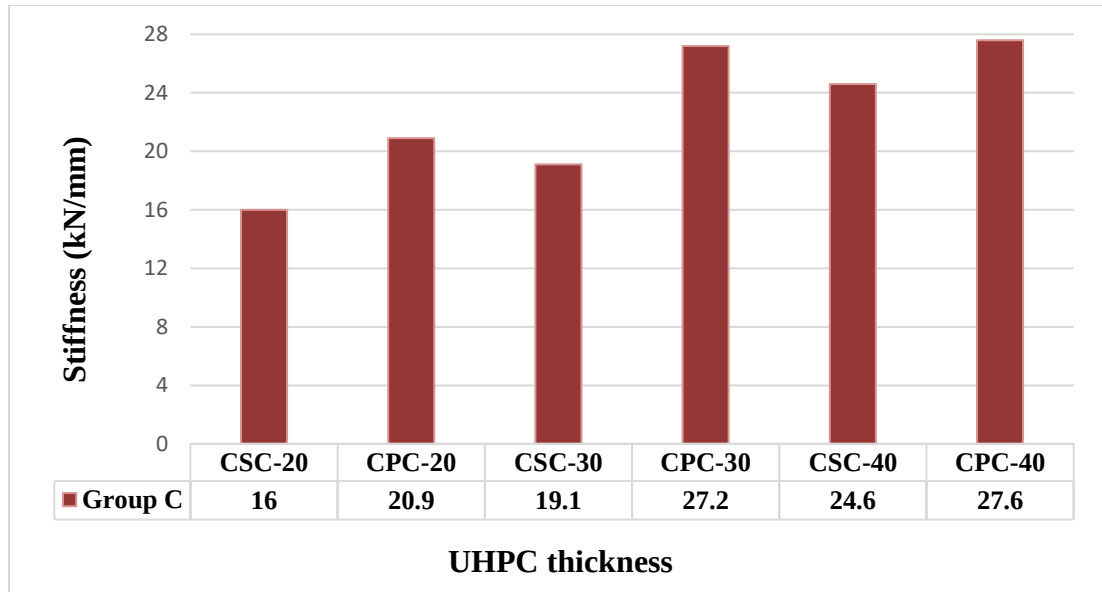


Figure 4.48: Stiffness enhancement for the strengthened and post-strengthened RC beams of group C.

4.4.4.5 Ductility and energy absorption

Results of ductility and energy absorption indices of the strengthened and post-strengthened beams are illustrated in Table 4.9. The use of UHPC layers for strengthening RC beams of phase I reduced the ductility index by an average value of 51% because the strengthened beams reached the ultimate loads at a very small deflection compared to the control beams. On the other hand, the post-strengthened beams of phase II show an increase in their ductility indices by 12%, respectively, in comparison to that of phase I strengthened beams. The corroded control beams CUA and CUB show lower energy absorption indices compared to the uncorroded control beam UU due to the lowering of the ultimate loads for the corroded control beams. The strengthened and post-strengthened beam specimens had nearly similar energy absorption indices and both were lower than those of the control beams. This was attributed to the ultra-high-strength of UHPC layers that cause the

specimens to reach their ultimate loads at a small mid-span deflection. These results are consistent with the results of the reported ductility indices as previously discussed.

Table 4.9: Ductility and energy absorption of post-strengthened beam specimens.

<i>Beam ID</i>	<i>Values of deflection (mm)</i>		μ	<i>Nor. μ</i>	Ω (kN.mm)	<i>Nor. Ω</i>
	<i>Yield</i>	<i>Maximum</i>				
<i>UU</i>	5.5	14	2.55	1.00	938.4	1.00
<i>CUB</i>	4.1	14.9	3.63	1.43	788.4	0.84
<i>CUC</i>	3.8	15	3.95	1.55	766.5	0.82
<i>CSB-20</i>	4.95	6.14	1.27	0.49	408.4	0.44
<i>CSB-30</i>	5.08	6.65	1.31	0.51	530.3	0.57
<i>CSB-40</i>	5.16	6.38	1.24	0.49	564.7	0.60
<i>CSC-20</i>	5.21	7.42	1.42	0.56	467.2	0.50
<i>CSC-30</i>	5.34	6.35	1.19	0.47	491.7	0.52
<i>CSC-40</i>	5.55	5.95	1.07	0.42	523.8	0.56
<i>CPB-20</i>	4.6	5.90	1.28	0.50	373.9	0.40
<i>CPB-30</i>	4.8	5.82	1.21	0.48	453.1	0.48
<i>CPB-40</i>	4.84	6.16	1.27	0.50	532.1	0.57
<i>CPC-20</i>	3.52	5.86	1.66	0.65	351.8	0.37
<i>CPC-30</i>	3.76	6.04	1.61	0.63	488.0	0.52
<i>CPC-40</i>	4.33	5.78	1.33	0.52	421.5	0.45

CHAPTER 5

ANALYTICAL SOLUTION

5.1 General

In this chapter, analytical models were developed to predict the load-carrying capacities of the tested beams, including the control, strengthened, and repaired beam specimens using the fundamental knowledge of structural mechanics and properties of the RC beams. The prediction of internal forces using the developed analytical models was based on the stress-strain distribution of the beam's cross-section. The moment capacity was estimated from the equilibrium internal forces of the strengthened cross-sectional member at failure. A singly reinforced behavior was considered for the analytical modelling of the strengthened beams. Thus, compression steel bars were not included in the calculations. In addition, it was assumed that there is no relative slip between the tensile steel bars and surrounding concrete. The results obtained from the analytical solution are validated with the experimental results obtained earlier with good agreement. The analytical solutions were divided into three parts, as illustrated in the following sections.

5.2 Analytical model of the control RC beam specimens

The analytical model of the corroded control beams was developed based on the material test results obtained for steel reinforcement bars and normal concrete as follows: (1) the steel reinforcement bars were modelled using a bilinear stress-strain curve since it behaves as elastic perfectly plastic as illustrated in [Figure 5.1\(a\)](#), (2) Normal concrete was modelled using Whitney's compressive stress distribution block in accordance with ACI Code

Section 10.2.7 as shown in Figure 5.1(b), and the maximum concrete compressive strain in the upper face of the beam was assumed to be equal to 0.003.

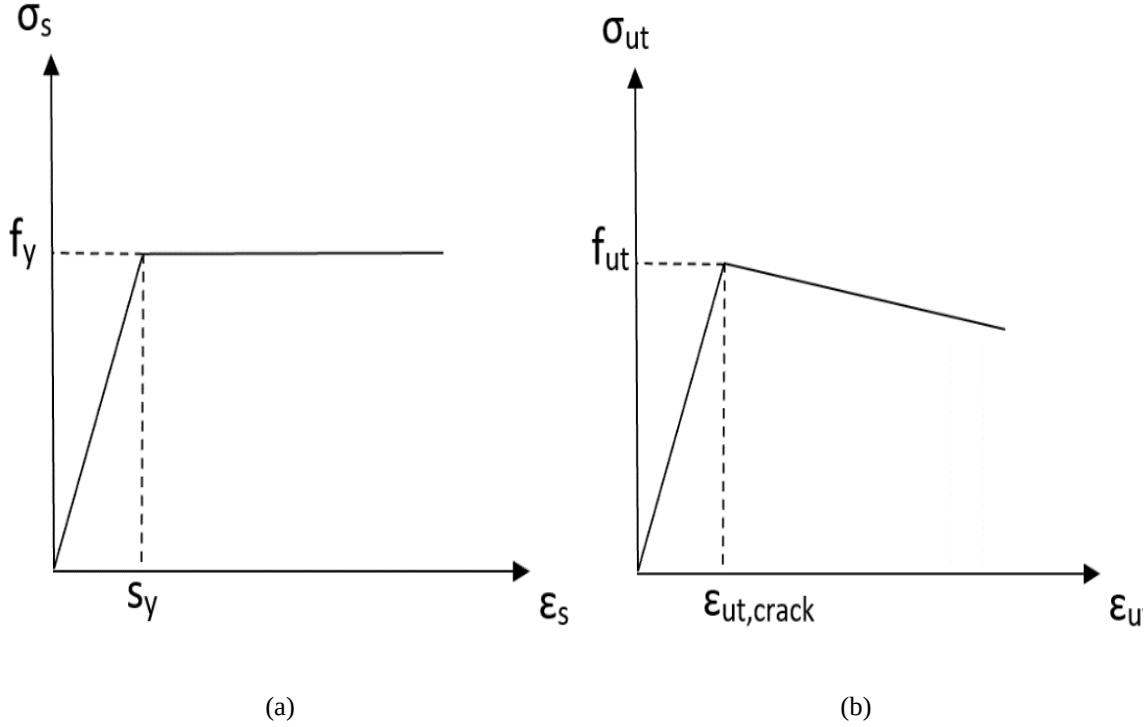


Figure 5.1: Behavior of materials under uniaxial tension: (a) Steel reinforcement, (b) Ordinary concrete and UHPC

Figure 5.2 shows the analytical model definition of the beam specimens. Compressive and tensile forces were calculated using Eqs. (5.1-5.3).

$$C_c = 0.85 f'_c \beta_c x b_c \quad (\text{Eq. 5.1})$$

$$T_s = A_{sc} f_{yc} \quad (\text{Eq. 5.2})$$

$$\text{Where } \beta_c = 0.85 - 0.05 \frac{f'_c - 28}{7} \quad f'_c > 28 \text{ MPa} \quad (\text{Eq. 5.3})$$

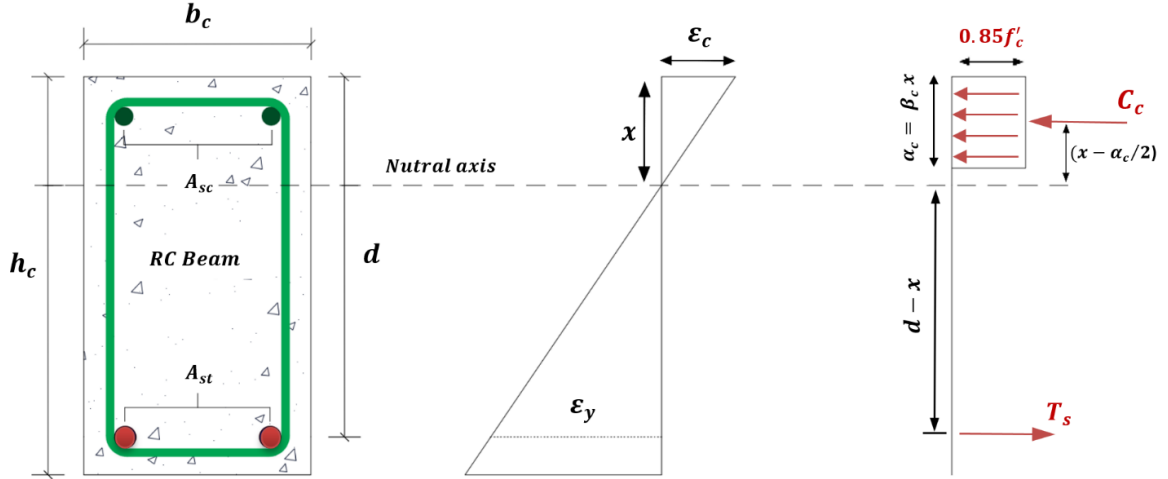


Figure 5.2: Analytical model definition of control beams.

Ignoring the loss of bond between the steel reinforcing bars and surrounding concrete, the decrease in the ultimate strength of the damaged RC beams due to reinforcement corrosion is mainly related to the reduction in the cross-sectional area of the steel bars [37,64]. The degradation of the cross-sectional area and yield stress of the corroded steel bars due to reinforcement corrosion was reported by Du et al. [116]. A reduction in the cross-sectional area and yield strength of the corroded bars can be estimated based on the actual amount of corrosion mass loss in the reinforcing bars by using the following equations proposed by Du et al. [116].

$$A_{sc} = (1 - 0.01 M_p) A_{st} \quad (\text{Eq. 5.4})$$

$$f_{yc} = (1 - 0.005 M_p) f_y \quad (\text{Eq. 5.5})$$

Therefore, the tensile force of the corroded steel bars can be calculated using Eq. 5.6.

$$T_s = (1 - 0.01 M_p) (1 - 0.005 M_p) A_{st} f_y \quad (\text{Eq. 5.6})$$

Following the force equilibrium equation, the summation of compression and tensile forces in the cross-section area is equal to zero. Therefore, the value of x can be calculated from the following equation Eq. 5.7.

$$x = \frac{A_{sc} f_{yc}}{0.85 f'_c \beta_c b_c} \quad (\text{Eq. 5.7})$$

The predicted bending moment capacity can be calculated by taking the moment of the compression and tensile forces about the neutral axis as expressed in Eq. 5.8. The difference percentage between experimental and analytical loads is expressed by Eq. 5.9.

$$M_{predicted} = A_{sc} f_{yc} (d - x) + 0.85 f'_c \beta_c x b_c \left(x - \frac{a_c}{2} \right) \quad (\text{Eq. 5.8})$$

$$Difference (\%) = \frac{P_{predicted} - P_{experimental}}{P_{experimental}} \times 100 \% \quad (\text{Eq. 5.9})$$

Table 5.1 shows the results of the analytical solution and a comparison with the experimental results. It can be seen that the predicted ultimate load matches with experiment results with good agreement. The maximum difference between the experimental and analytical results is less than 4%.

Table 5.1: Analytical results of the corroded beam specimens.

<i>Beam ID</i>	<i>P_{experimental}</i> (kN)	<i>P_{analytical}</i> (kN)	<i>Difference</i> (%)
<i>UU</i>	85.5	84.81	-0.80
<i>CUA</i>	70.7	73.40	3.82
<i>CLB</i>	65.4	64.95	-0.69
<i>CLC</i>	62.8	61.29	-2.41

The results of the analytical model are in agreement matching with the experimental results with a minor percentage of error. The maximum differences between experimental and analytical ultimate loads were not exceeding 4 % for all control beam specimens, as shown in Figure 5.3. This matches with the results obtained using the present study's data, leading

to the confidence of the developed model. Figure 5.4 illustrates a strong fit between the experimental and predicted values of the ultimate load capacity of the corroded beam specimens, as indicated by a high R^2 value of 0.97.

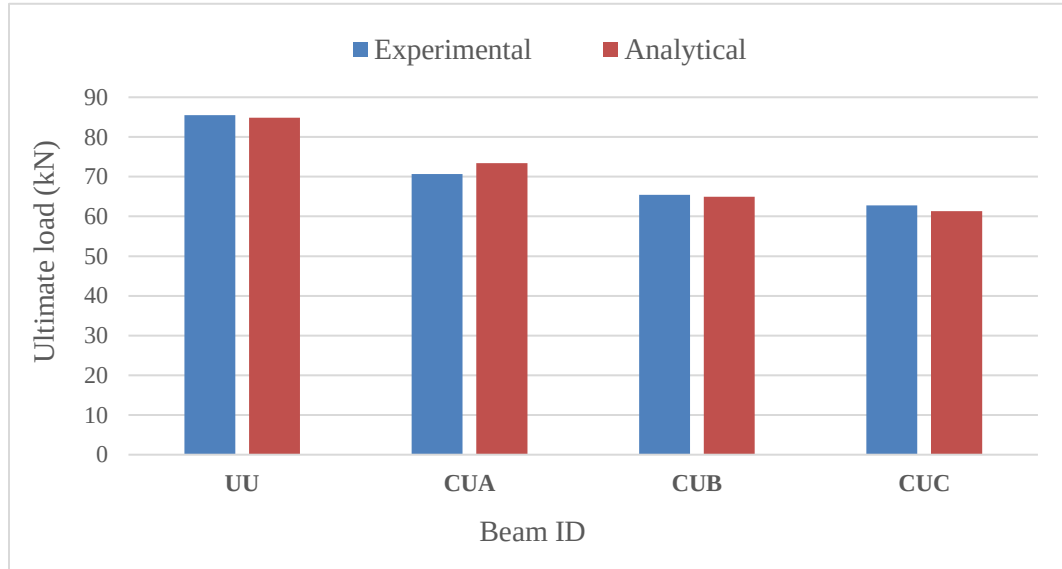


Figure 5.3: Comparison of experimental and analytical ultimate loads for control beams.

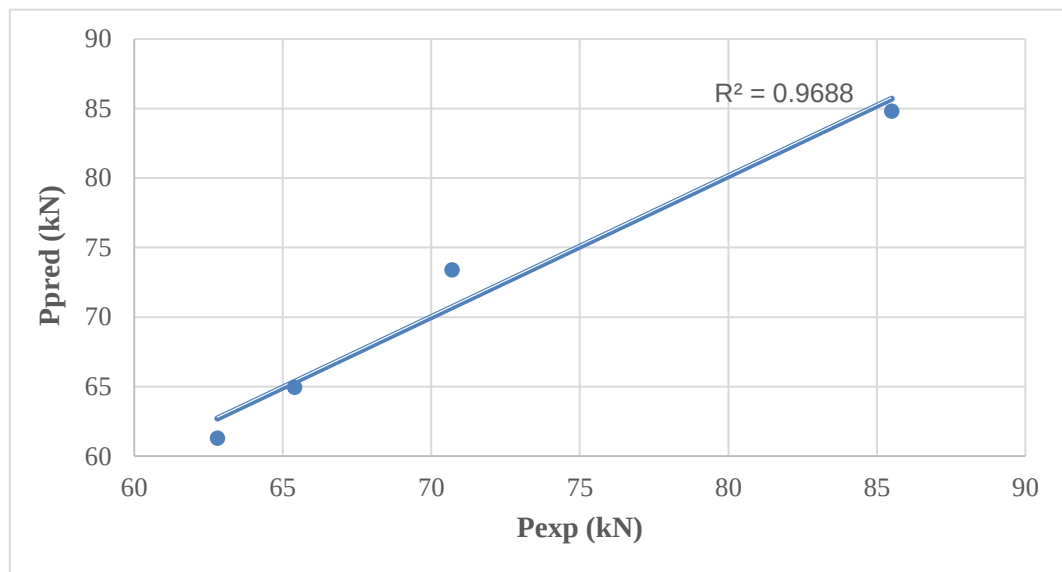


Figure 5.4: Comparison of experimental and analytical ultimate loads for control beams.

5.3 Analytical model of strengthened corrosion-damaged RC beams

This section provides an analytical solution to determine the ultimate load-bearing capacities at failure for beams strengthened with UHPC layers from three sides. The failure loads were 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. The structural behavior of the UHPC material in tension is shown in Figure 5.1(b). A triangular distribution block was used to model the behavior of UHPC for both compression and tension zones. The analytical model of the ultimate flexural capacity of the strengthened corroded beam is illustrated in Figure 5.5.

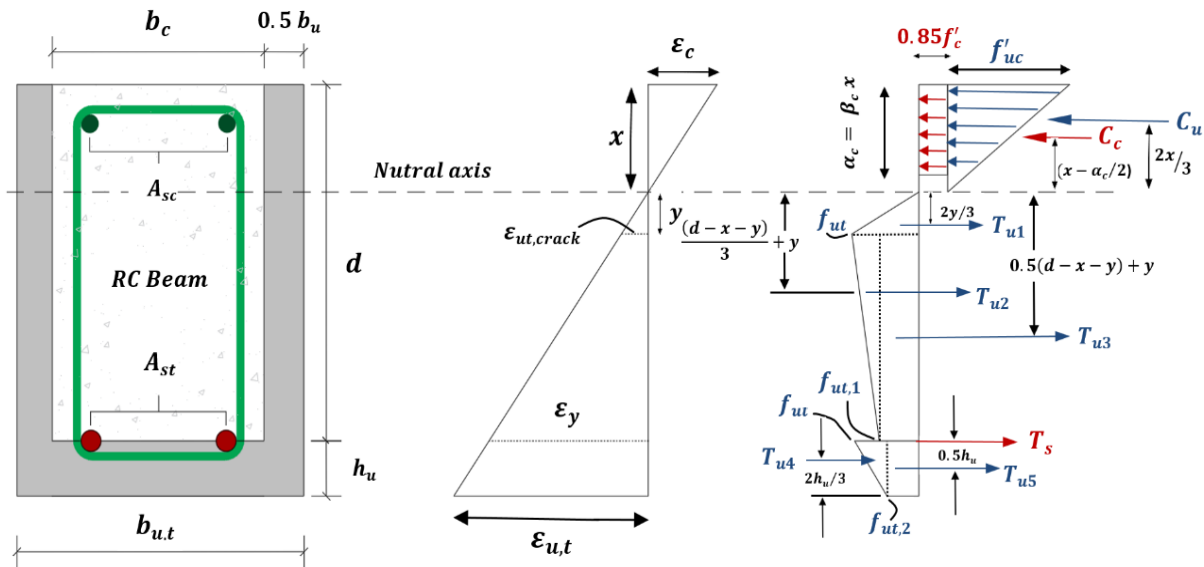


Figure 5.5: Analytical model definition of strengthened beams.

The forces of UHPC can be calculated by multiplying the stresses with the cross-sectional area covered by each stress as follows and as shown in Figure 5.5.

$$C_u = 0.5 f'_{uc} x b_u \quad (\text{Eq. 5.10})$$

$$T_{u1} = 0.5 b_u y f_{ut} \quad (\text{Eq. 5.11})$$

$$T_{u2} = 0.5 (f_{ut} - f_{ut,1}) b_u (d - x - y) \quad (\text{Eq. 5.12})$$

$$T_{u3} = b_u (d - x - y) f_{ut,1} \quad (\text{Eq. 5.13})$$

$$T_{u4} = 0.5 h_u (f_{ut} - f_{ut,2}) b_{u,t} \quad (\text{Eq. 5.14})$$

$$T_{u5} = h_u b_{u,t} f_{ut,2} \quad (\text{Eq. 5.15})$$

The equations from Eq. 5.10 to Eq. 5.15 were expressed in terms of x (depth of the natural axis). The value of x is iterated till the following force equilibrium equation becomes equal to zero:

$$\Sigma F = 0$$

$$(T_s + T_{u1} + T_{u2} + T_{u3} + T_{u4} + T_{u5}) - (C_c + C_u) = 0 \quad (\text{Eq. 5.16})$$

The predicted bending moment capacity was calculated by taking the moment of all forces about the neutral axis as expressed in Eq 5.17.

$$M_{predicted} = T_s(d - x) + T_{u1} \left(\frac{2y}{3} \right) + T_{u2} \left(\frac{1}{3}(d - x - y) + y \right) + T_{u3} \left(\frac{1}{2}(d - x - y) + y \right) + T_{u4} \left(d - x + \frac{1}{3}h_u \right) + T_{u5} \left(d - x + \frac{1}{2}h_u \right) + C_c \left(x - \frac{a_c}{2} \right) + C_u \left(\frac{2x}{3} \right) \quad (\text{Eq. 5.17})$$

Figure 5.6 shows the procedure for computing the predicted bending moment capacities of the strengthened corrosion-damaged RC beams.

The results of the predicted moment capacities using the analytical model were presented and compared with the experimental test result, as shown in Table 5.2. The results showed good agreement for the strengthened corroded beams.

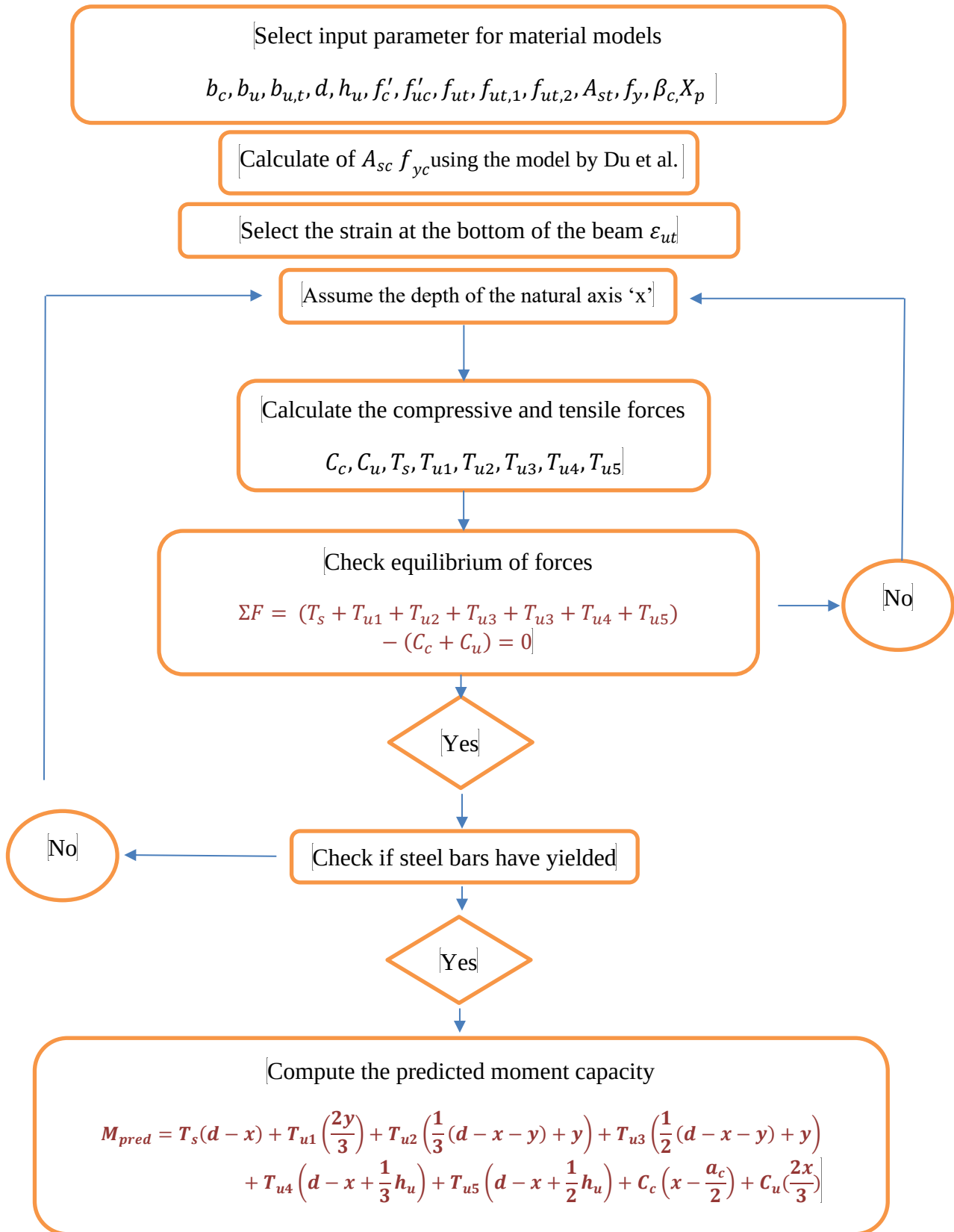


Figure 5.6: Analytical model flow chart of strengthened beams

Table 5.2: Analytical results of the strengthened corroded RC beam specimens

<i>Beam ID</i>	$P_{experimental}$ (kN.m)	$P_{analytical}$ (kN.m)	<i>Difference</i> (%)
CSA20	101.8	105.17	3.31
CSA30	116	123.69	6.63
CSA40	139.5	143.69	3.00
CSB20	92	94.39	2.60
CSB30	113.2	112.98	-0.19
CSB40	137.1	132.23	-3.56
CSC20	90.2	89.14	-1.18
CSC30	110	105.05	-4.50
CSC40	135.1	127.09	-5.93

The differences between experimental and analytical ultimate loads were not exceeding 7 % for all strengthened beam specimens, as illustrated in Figure 5.7. The developed analytical model can capture the performance of the strengthened corroded beams for different degrees of corrosion, i.e., between 9.1 and 22.3 %, with high accuracy.

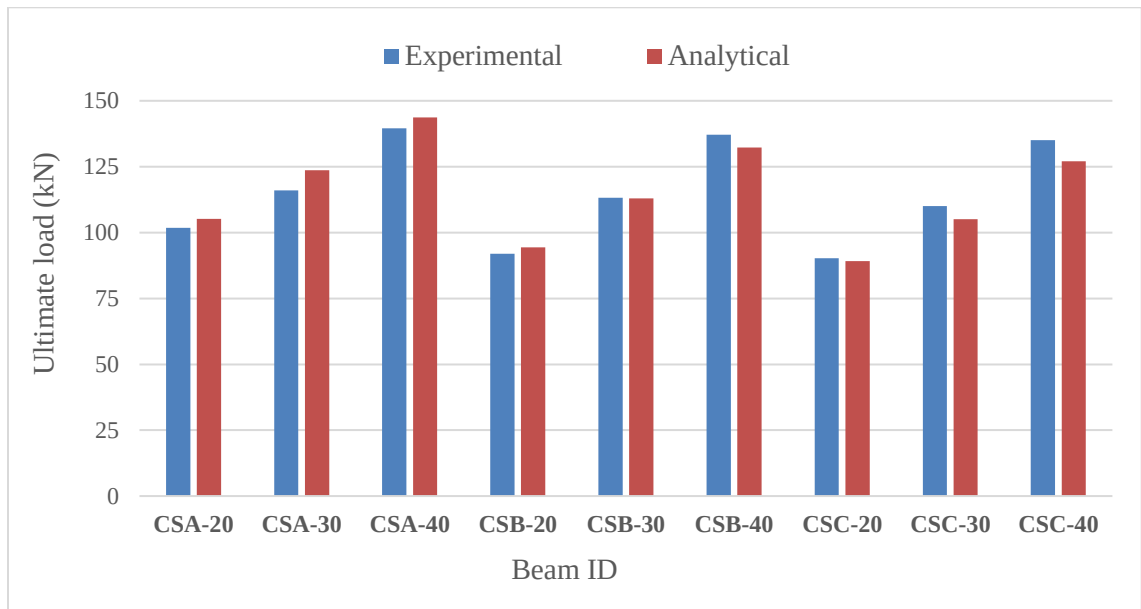


Figure 5.7: Comparison between Experimental and analytical results for strengthened beams.

5.4 Analytical model of repaired corrosion-damaged RC beams

This section provides an analytical solution to determine the ultimate load-bearing capacities at failure for beams repaired with the UHPC layer from the only bottom side. The failure loads were 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. The structural behavior of the UHPC material in tension is shown in Figure 5.1(b). A triangular distribution block was used to model the behavior of UHPC for both compression and tension zones. The analytical model of the ultimate flexural capacity of the repaired corroded beam is illustrated in Figure 5.8.

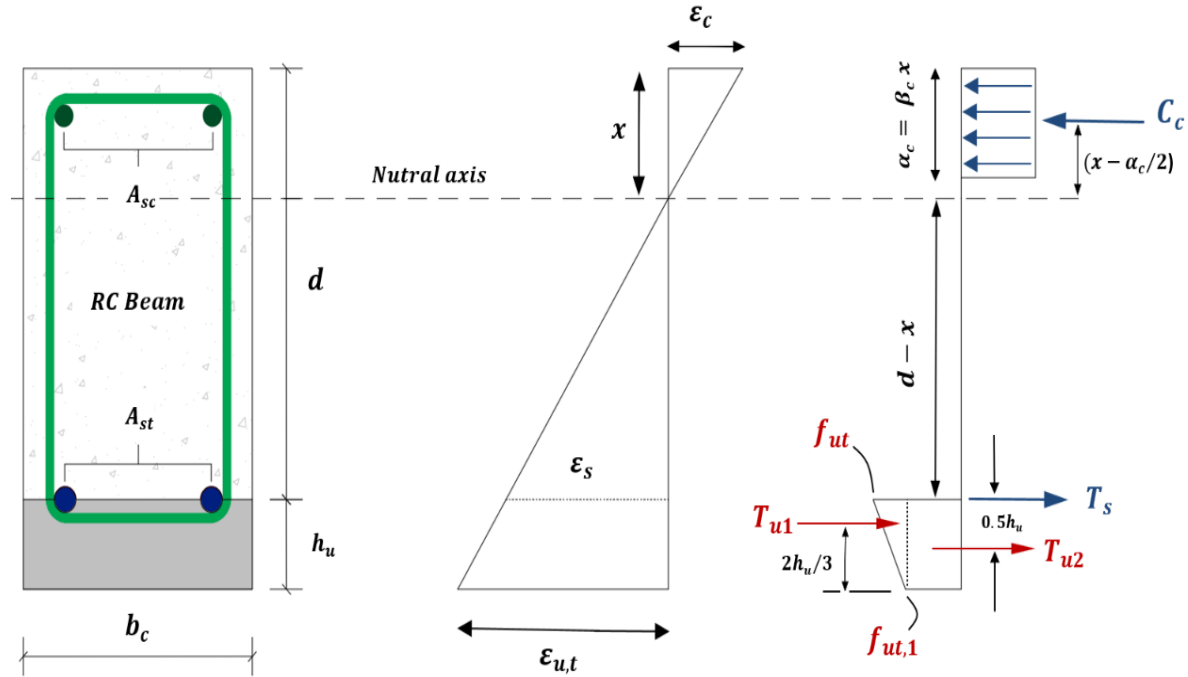


Figure 5.8: Analytical model definition of repaired beams.

The forces of UHPC can be estimated by multiplying the stresses with the cross-sectional area covered by each stress as follows and as shown in [Figure 5.8](#).

$$T_{u1} = 0.5 h_u (f_{ut} - f_{ut,1}) b_c \quad (\text{Eq. 5.18})$$

$$T_{u2} = h_u b_c f_{ut,1} \quad (\text{Eq. 5.19})$$

The equations [Eq. 5.18](#) and [Eq. 5.19](#) were expressed in terms of x (depth of the natural axis). The value of x is iterated till the following force equilibrium equation becomes equal to zero:

$$\Sigma F = 0$$

$$(T_s + T_{u1} + T_{u2}) - (C_c) = 0 \quad (\text{Eq. 5.20})$$

The predicted bending moment capacity was calculated by taking the moment of all forces about the neutral axis as expressed in [Eq 5.21](#).

$$M_{predicted} = T_s(d - x) + T_{u1} \left(d - x + \frac{1}{3} h_u \right) + T_{u2} \left(d - x + \frac{1}{2} h_u \right) + C_c \left(x - \frac{a_c}{2} \right) \quad (\text{Eq. 5.21})$$

[Figure 5.9](#) shows the procedure for computing the predicted bending moment capacities of the strengthened corrosion-damaged RC beams.

The results of the predicted moment capacities using the analytical model were presented and compared with the experimental test result, as shown in [Table 5.3](#). The results showed good agreement for the strengthened corroded beams.

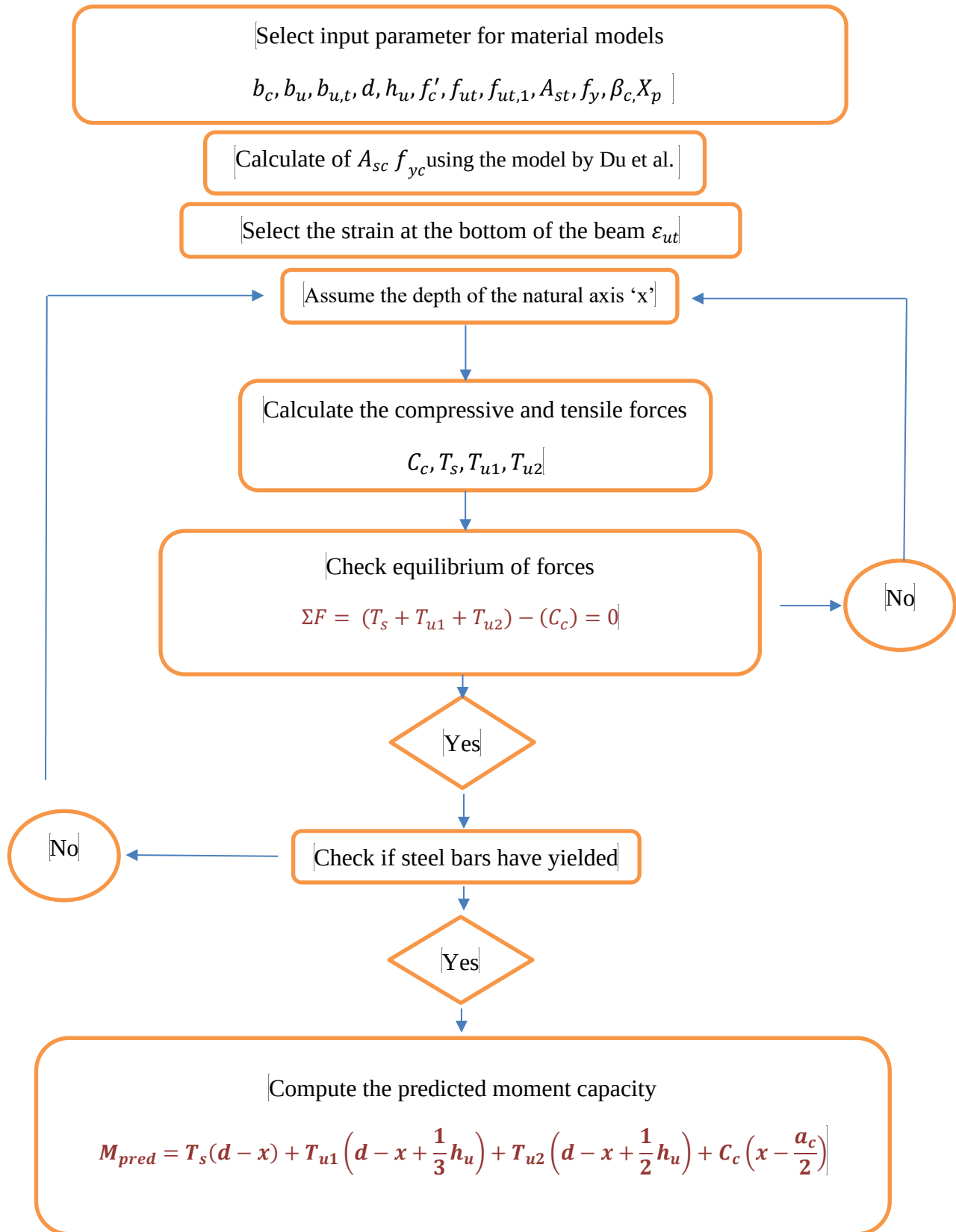


Figure 5.9: Analytical model flow chart of repaired beams

Table 5.3: Analytical results of the repaired corroded beam specimens

<i>Beam ID</i>	<i>P_{experimental}</i> (kN.m)	<i>P_{analytical}</i> (kN.m)	<i>Difference</i> (%)
<i>CRA20</i>	82.8	88.03	6.32
<i>CRA40</i>	93.3	99.19	6.32
<i>CRA60</i>	113	111.08	-1.70
<i>CRB20</i>	79.1	76.35	-3.48
<i>CRB40</i>	92	89.23	-3.01
<i>CRB60</i>	98	101.98	4.07
<i>CRC20</i>	71.2	70.79	-0.58
<i>CRC40</i>	91.2	85.09	-6.70
<i>CRC60</i>	97.5	96.17	-1.36

The differences between experimental and analytical ultimate loads were not exceeding 7 % for all repaired beam specimens, as illustrated in Figure 5.10. The developed analytical model can capture the behavior of the repaired corroded beams for different degrees of corrosion, i.e., between 10 and 22.6 %, with high accuracy.

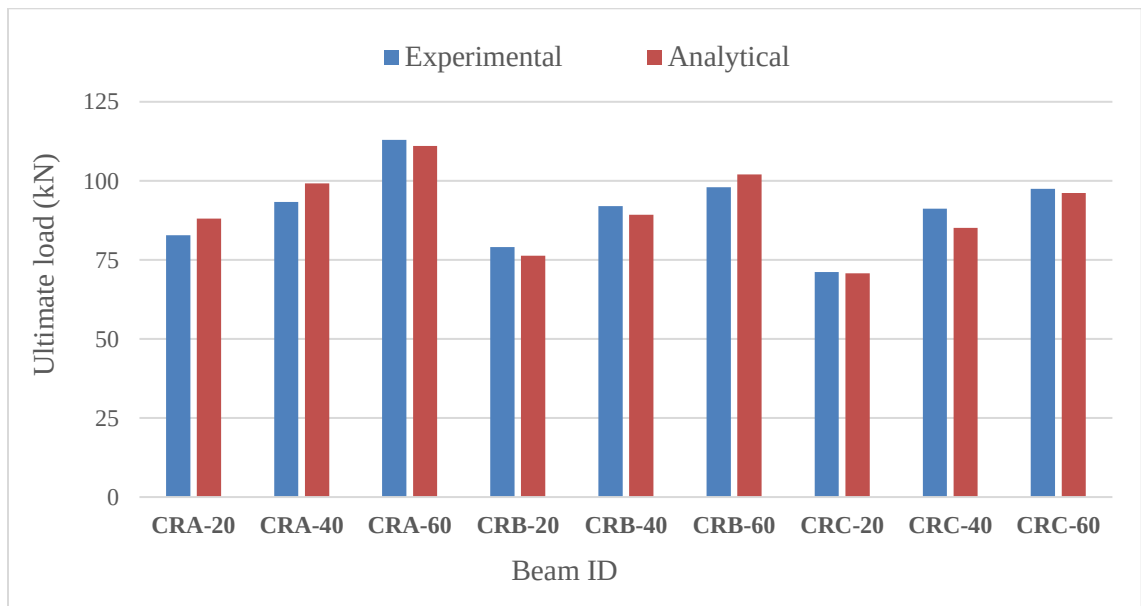


Figure 5.10: Comparison between experimental and analytical results for repaired beams.

CHAPTER 6

FINITE ELEMENT MODELLING

6.1 General

Finite element modelling and its verification with the results obtained from the experimental testing are very popular in recent years to capture the exact structural behavior of real samples. Furthermore, the numerical analysis makes it possible to anticipate the real response of the same concrete members with alternative UHPC configurations and thicknesses without conducting additional experimental testing. The commercial finite element analysis (FEA) software ABAQUS was used to model RC beam specimens. It is well known for modeling concrete structures due to its sophisticated numerical capabilities and extensive choice of concrete material models. In this chapter, finite element modelling using ABAQUS software was carried out to capture the structural behavior of corrosion-damaged RC beams strengthened/repared with different layers of UHPC material. The load vs. mid-span deflection curves were obtained from the results of numerical modelling and compared with the experimental results. The finite element modeling of the tested beams included the control, strengthened, and repaired beam specimens using the fundamental knowledge of structural behavior and properties of the RC beams and UHPC. The results of the numerical modelling are validated with the experimental results obtained earlier with good agreement. The numerical solutions of the RC beam specimens were divided into the following sections.

6.2 Material models

The elastic performance of the steel bars was modeled using the elastic material model implemented in ABAQUS up to the yielding point, and the inelastic performance was modeled using the plastic material model with isotropic hardening. Material behavior required for FEM simulation was implemented directly in the models using the stress-strain results obtained from experimental testing. The concrete damage plasticity model (CDPM) was used for modelling both normal concrete and UHPC.

6.2.1 Concrete damage plasticity model (CDPM)

The concrete damage plasticity model (CDPM) is a powerful tool for predicting concrete behavior under both static and dynamic loading conditions. It works in two different mechanisms; compressive crushing and tensile cracking concrete. In the tension mechanism, the concrete employs a multi-axial damage elasticity model, while in the compression mechanism, it employs a multi-axial plasticity model with the non-associated flow and isotropic scalar hardening.

Equivalent plastic strains in tension and compression are two hardening variables that characterize damage states in compression and tension separately. The increment values of the variables of the hardening mechanism produce compression crushing and tension cracking; these variables control the development of the yield surfaces and degradation of the elastic stiffnesses. The performance of the developed model in uniaxial compression and tension zones is illustrated in [Figure 6.1](#) and [Figure 6.2](#), respectively.

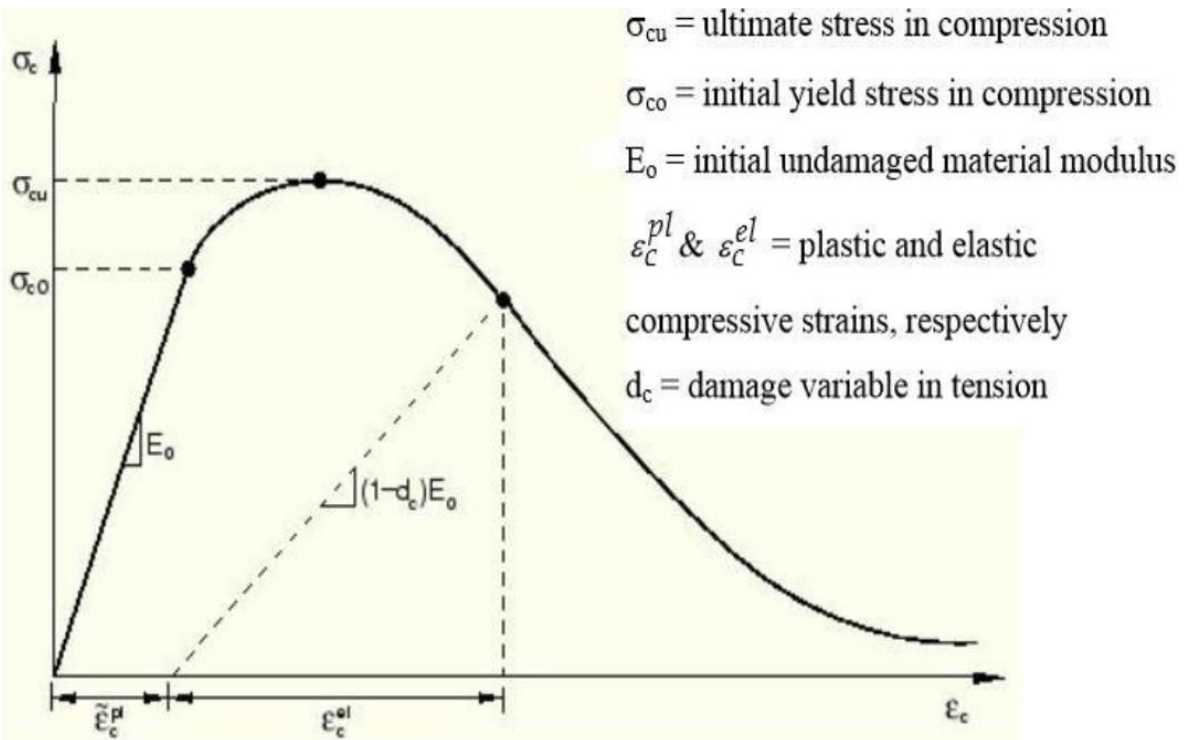


Figure 6.1: Stress strain curve of the uniaxial compressive behavior in tension (Abaqus Manual)

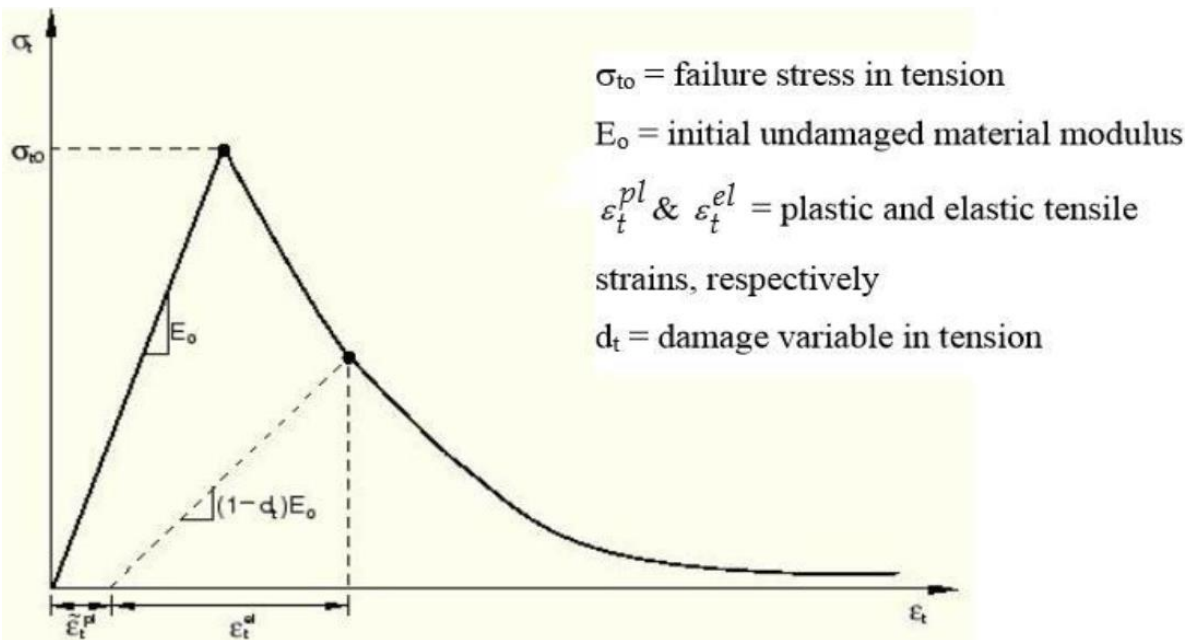


Figure 6.2: Stress strain curve of the uniaxial tensile behavior in tension (Abaqus Manual)

Lubliner et al. [117] proposed the yield function of CDPM, then it was modified by Lee and Fenves [118]. It is defined by Eq. 6.1.

$$F = \frac{1}{1 - \alpha} (\bar{q} - 3\alpha\bar{p} + \beta(\tilde{\epsilon}^{pl})\langle\bar{\sigma}_{max}\rangle - \gamma\langle-\bar{\sigma}_{max}\rangle) - \bar{\sigma}_c(\tilde{\epsilon}_c^{pl}) \quad (\text{Eq. 6.1})$$

Where: q and p are the Mises equivalent effective stress and hydrostatic pressure stress, respectively. The parameters α , $\beta(\epsilon^{-pl})$, and γ can be calculated using Eq. 6.2 to Eq. 6.4, where σ_{bo} and σ_{co} are the biaxial and uniaxial compressive strength, respectively. The default value of the ratio $\sigma_{bo} / \sigma_{co}$ is 1.16. K_c is the ratio of the tensile to the compressive meridian. γ is the parameter defines the shape of the yield surface.

$$\alpha = \frac{(\sigma_{bo}/\sigma_{co}) - 1}{2(\sigma_{bo}/\sigma_{co}) - 1} \quad (\text{Eq. 6.2})$$

$$\beta(\tilde{\epsilon}^{pl}) = \frac{\bar{\sigma}_c(\tilde{\epsilon}_c^{pl})}{\bar{\sigma}_t(\tilde{\epsilon}_t^{pl})} (1 - \alpha) - (1 + \alpha) \quad (\text{Eq. 6.3})$$

$$\gamma = \frac{3(1 - K_c)}{2K_c - 1} \quad (\text{Eq. 6.4})$$

To model the concrete behavior using the CDPM, the following input parameters are required to be defined in Abaqus.

- ¶ The dilation angle measured in the p-q plane at high confining pressure (Ψ).
- ¶ The parameter of eccentricity (ϵ) Figure 6.3.
- ¶ The initial ratio of equi-biaxial compressive yielding stress to the initial uniaxial compressive yielding stress ($\frac{\sigma_b}{\sigma_c}$).

- ¶ Ratio of the tensile to the compressive meridian and defines the shape of the yield surface in the deviatoric plane (K), as shown in Figure 6.4.
- ¶ Parameter of viscosity.

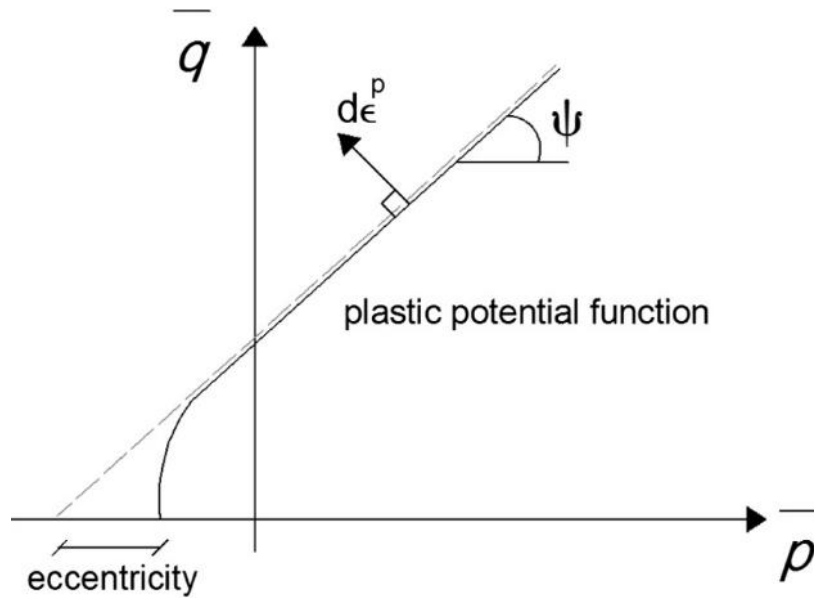


Figure 6.3: Define of eccentricity and dilation angle in meridian plane

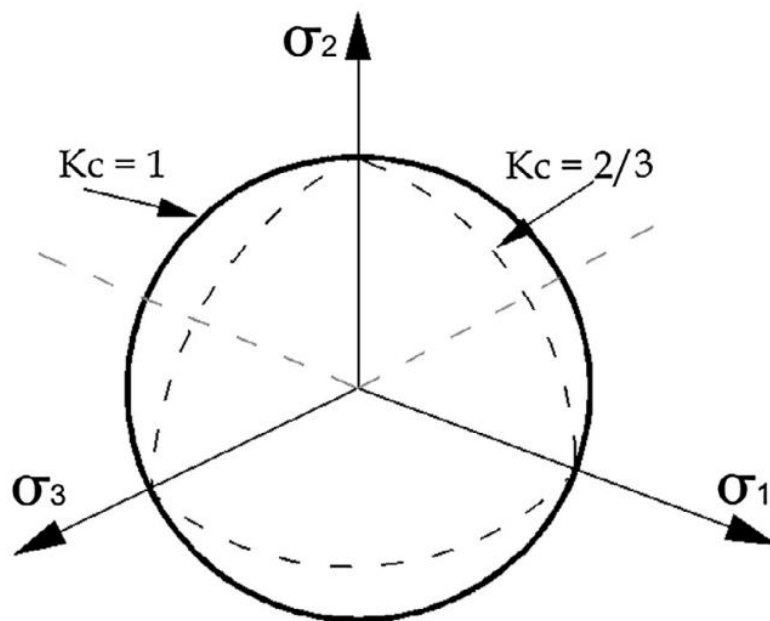


Figure 6.4: Yield surfaces in the deviatoric plane

Table 6.1 shows the input parameter values for concrete and UHPC that were used in the finite element modelling to define the CDPM.

Table 6.1: Input parameters values for concrete and UHPC to define the CDPM

Input parameter	Normal concrete	UHPC
Mass density (kg/m^3)	2.4E-06	2.5E-06
Young modulus (MPa)	3.3E+3	46E+3
Poisson ratio	0.15	0.15
ψ ($^\circ$)	36	36
ϵ	0.1	0.1
$\frac{\sigma_{bo}}{\sigma_{co}}$	1.16	1.16
K	0.667	0.667
Parameter of viscosity	0	0

6.3 Model constraints

In order to model the beam specimens in ABAQUS, a number of constraints were applied. These constraints determined the interactions between various parts of the FEM and the analysis of DOF between different regions of the model. The following sections illustrate the constraints used in the current model.

6.3.1 Tie constraint

The usage of tie constraint enabled us to combine the UHPC jacket and the normal concrete beam, although the meshes created on the normal concrete regions and the UHPC layers were different. As the UHPC surfaces are harder, they are chosen as the master surfaces,

whereas the concrete beam surfaces are chosen as the slave surfaces. Because of the tie constraint, the translation, rotation, and other degrees of freedom for a concrete-UHPC joint are all equal.

6.3.2 Embedded Region Constraint

In order to model the interaction between the reinforcement steel bars and stirrups with the concrete beams, the constraint of embedded regions was used. The embedded approach was used to simulate the bond behavior between normal concrete and UHPC with the steel reinforcement bars and stirrups, assuming a perfect bond. With the embedded region constraint, the model embeds the stirrups and longitudinal bars referred to as embedded regions in the concrete called host regions. Constraints are imposed based on the geometric relations between embedded and host elements. The translational DOF of embedded nodes are controlled by the interposed values of their host elements corresponding to the DOF. In order to model the embedded elements in host elements (i.e., top bars and stirrups embedded in normal concrete), a truss-in-solid model was used.

6.4 Analysis of FEM

For the analysis of the numerical model, an explicit dynamic analysis was implemented. This is because that explicit dynamic analysis allows the solver to define the amount of load to be applied to the model, thereby eliminating the convergence problems encountered by conducting static analysis. By using the explicitly central-difference time integration, the explicit dynamic procedures can efficiently perform a large number of small-time increments, eliminating the requirement for iterations and tangent stiffness matrices. The treatment of the contact problem is also simplified by conducting the proposed analysis.

6.5 Meshing elements

6.5.1 3-D Stress elements

In order to model ordinary concrete, UHPC, and main steel bars, Three-Dimensional stress eight-noded linear brick elements (C3D8R) were used (solid continuum). This element type is suitable for linear as well as composite nonlinear analyses involving contact, deformations, and plasticity. Furthermore, this element was recommended by ABAQUS Analysis user's manual to avoid the shear locking effect. Figure 6.5 shows the 3-D truss element used for numerical modelling.

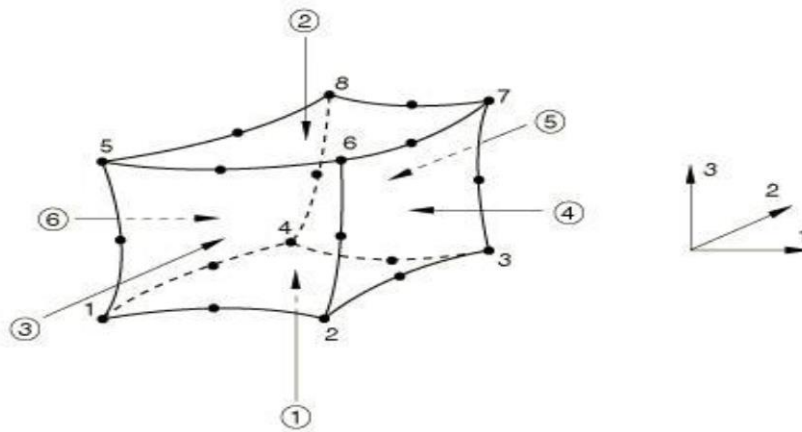
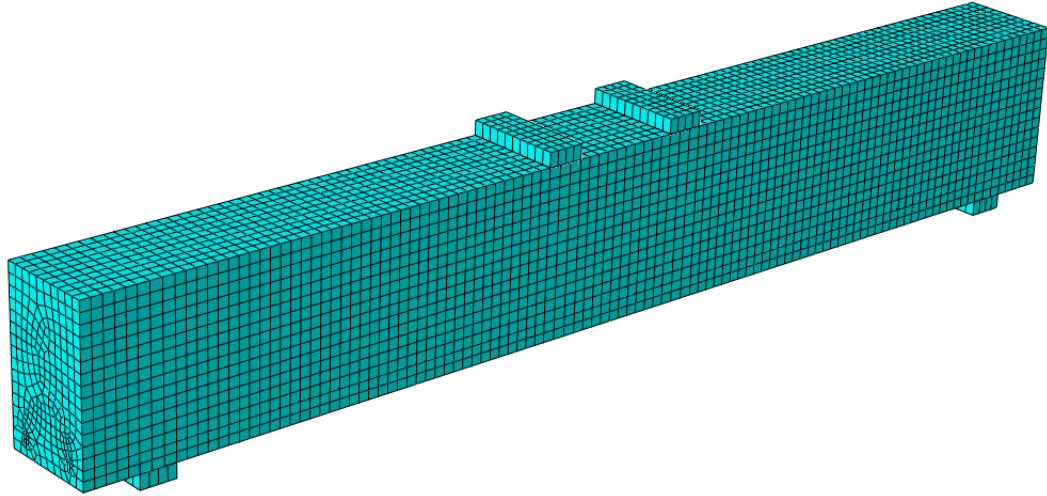
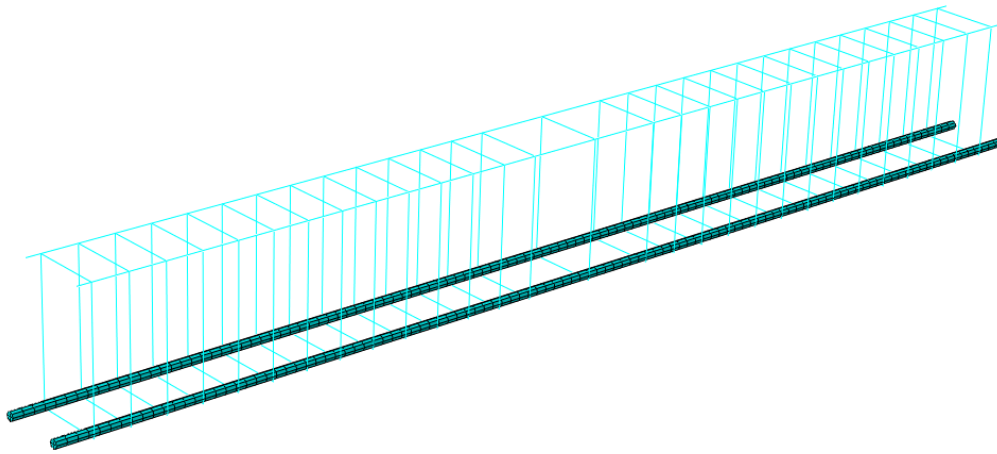


Figure 6.5: A 3-D stress 8-noded element type

In addition, steel plates were modeled at the loading points and support to avoid the stress concentration of the eccentric loads. The same FEM elements (C3D8R) were used to model the steel plates, and a perfect bonding was assumed between the plated and concrete and/or UHPC. The support plates were restrained from movement in the transverse and vertical directions. Figure 6.6 (a) and (b) show the discretized concrete beam elements and the discretized elements for the tensile steel bars and stirrups that were used in the finite element model developed in the study.



(a)



(b)

Figure 6.6: Discretization of the concrete beam specimen (a) concrete elements (b) steel bars and stirrups elements

6.5.2 Truss elements

Two-noded linear Three-Dimensional truss elements (T3D2), as illustrated in [Figure 6.7](#), were used to model the top steel reinforcement bars and stirrups. Typically, this structural

element is long and slender, and it only transmits axial loads. It allows the cross-section area of the truss elements to be defined, and the steel reinforcement can be shaped accordingly.



Figure 6.7: A 2-D noded linear 3-D truss element

6.6 Results of the finite element modelling

The load vs. midspan deflection of all tested beams were plotted based on the results of the finite element modelling and compared with the results obtained from the experimental testing of the corroded beams. It can be noticed that the FEM can capture the behavior of all control, strengthened, and repaired corroded beams with high accuracy, including the ultimate load-carrying capacity and stiffness. The load-deflection curves of the control corroded beams are shown in [Figure 6.8](#), [Figure 6.9](#), [Figure 6.10](#), and [Figure 6.11](#). The FEM captured the exact behavior of the uncorroded beam UU including stiffness, yielding point, and ultimate strength. However, the load-deflection curves of the control corroded beams showed a small variation in the yielding points and slope of the curve after yielding. This is likely due to the fact that steel bars modelled as elastic perfectly plastic, while in real the behavior of steel bars could be elasto-plastic with small hardening.

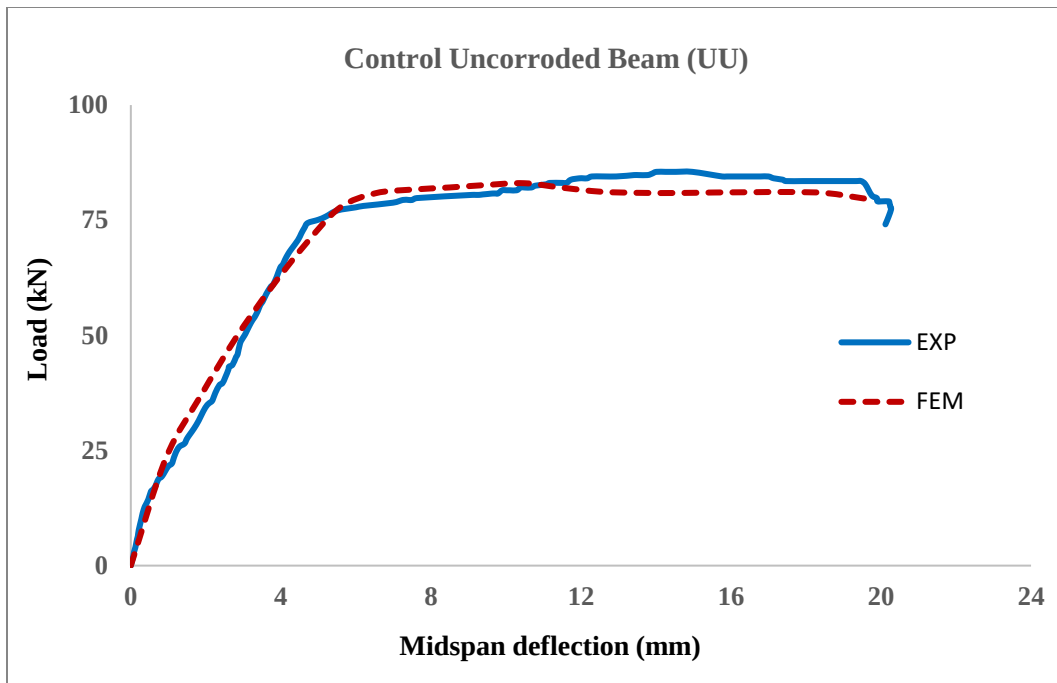


Figure 6.8: FEM vs. Experimental results for beam UU.

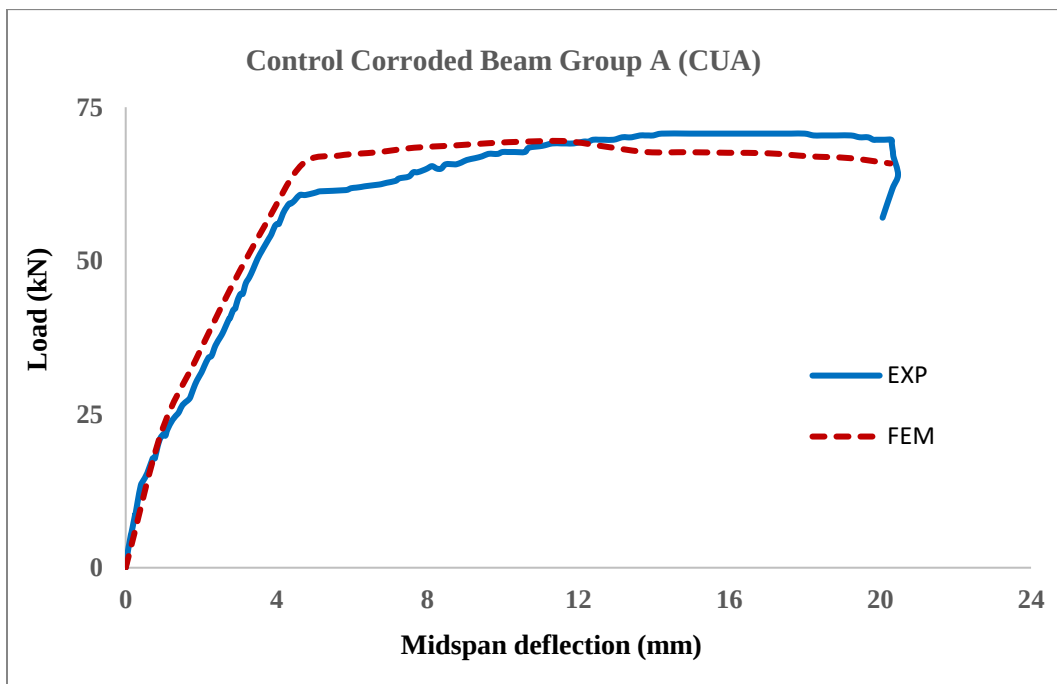


Figure 6.9: FEM vs. Experimental results for beam CUA.

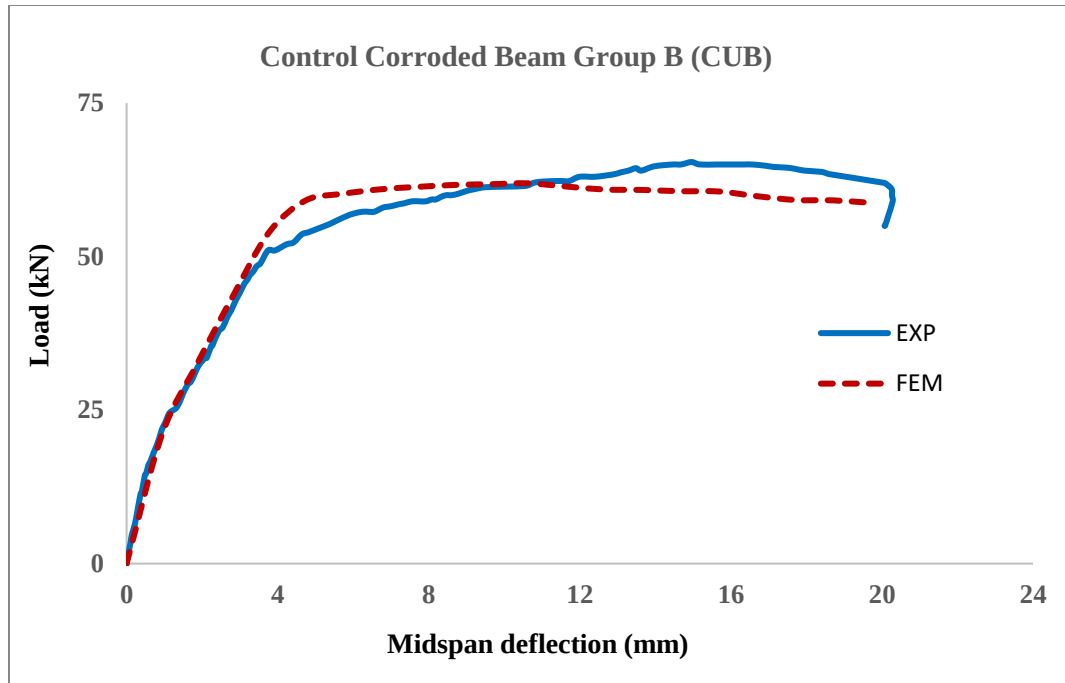


Figure 6.10: FEM vs. Experimental results for beam CUB.

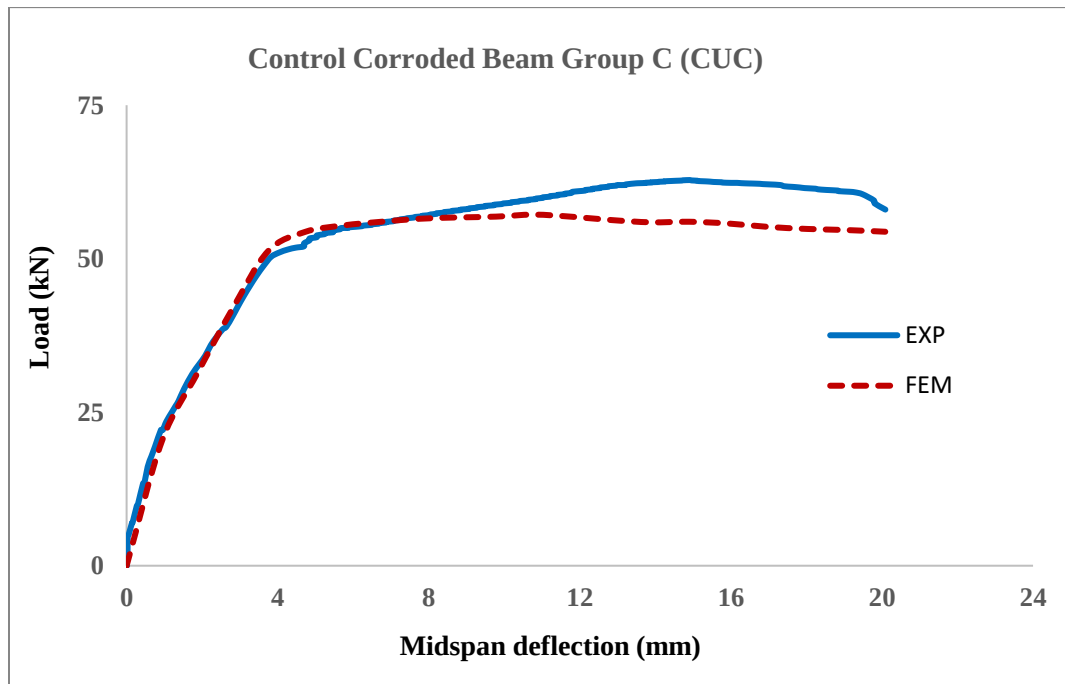


Figure 6.11: FEM vs. Experimental results for beam CUC.

Similar behavior of good matching between numerical and experimental results was observed for the UHPC-strengthened corrosion-damaged RC beams. The load vs. midspan deflection of the strengthened corroded beams was plotted and compared with the experimental testing. Results showed an excellent matching for the strengthened beams with 20 and 30 mm UHPC thickness, including stiffness, ultimate load, and overall behavior. However, the results of 40 mm showed a small variation between experimental and FEM results. The discrepancy is most likely attributable to the high thickness of UHPC that varies the concentration and orientation of steel fibers within the layers of UHPC. This variation has a significant effect on the stiffness and behavior of the strengthened beam. However, the influence of concentration and steel fibers orientation is ignored in the FEM simulation. The same results were also reported by Al-Osta et al. [99]. The figures from Figure 6.12 to Figure 6.20 show the load vs. midspan deflection curves of numerical and experimental results for all strengthened corrosion-damaged RC beams.

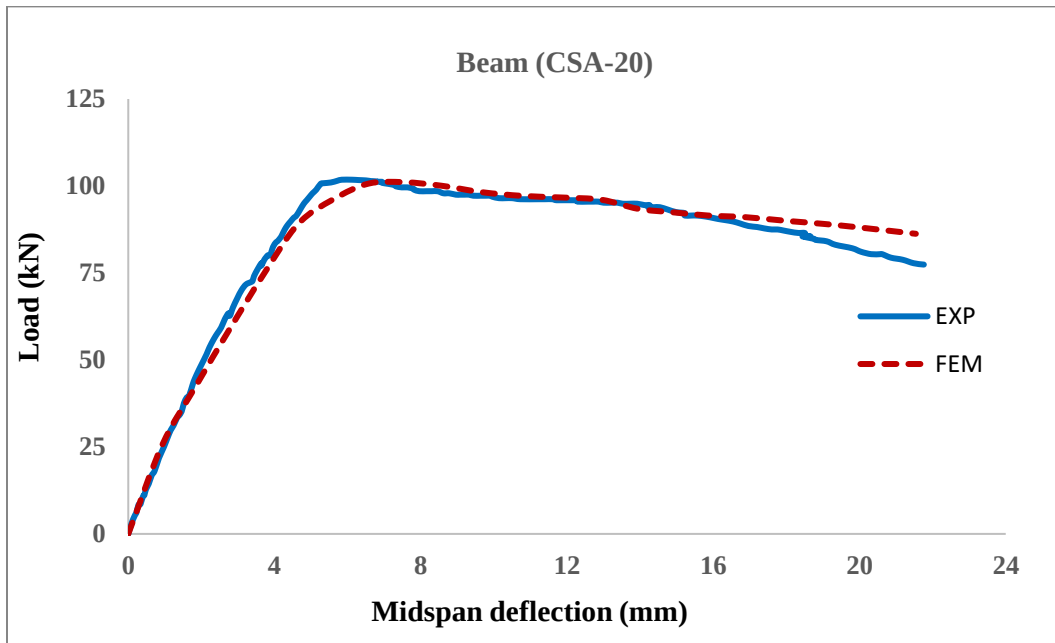


Figure 6.12: FEM vs. Experimental results for beam CSA-20.

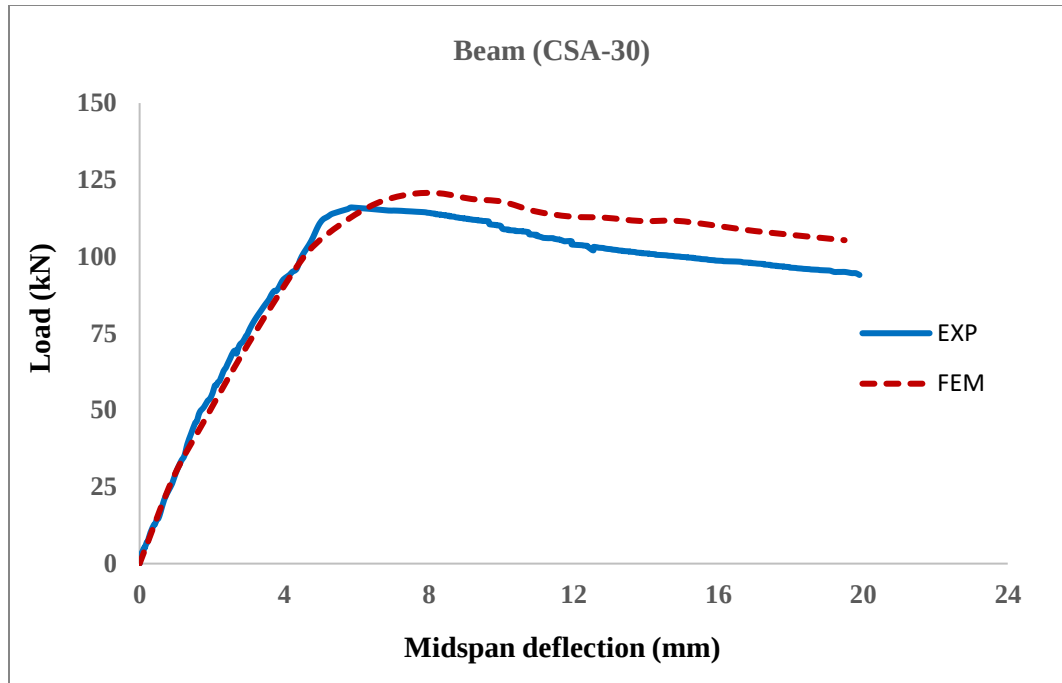


Figure 6.13: FEM vs. Experimental results for beam CSA-30.

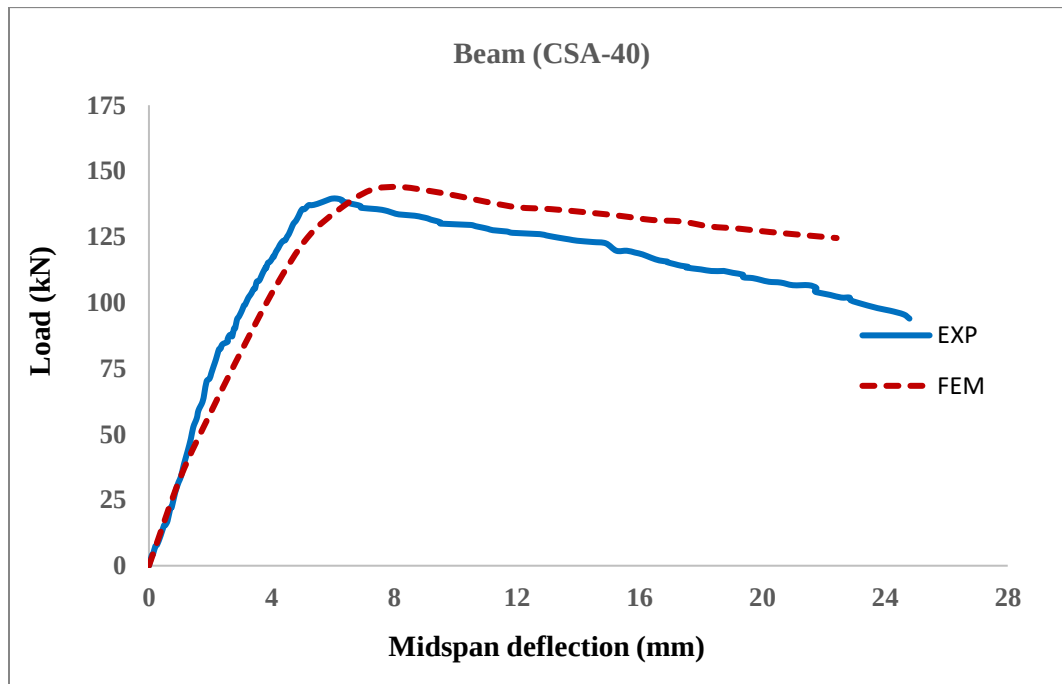


Figure 6.14: FEM vs. Experimental results for beam CSA-40.

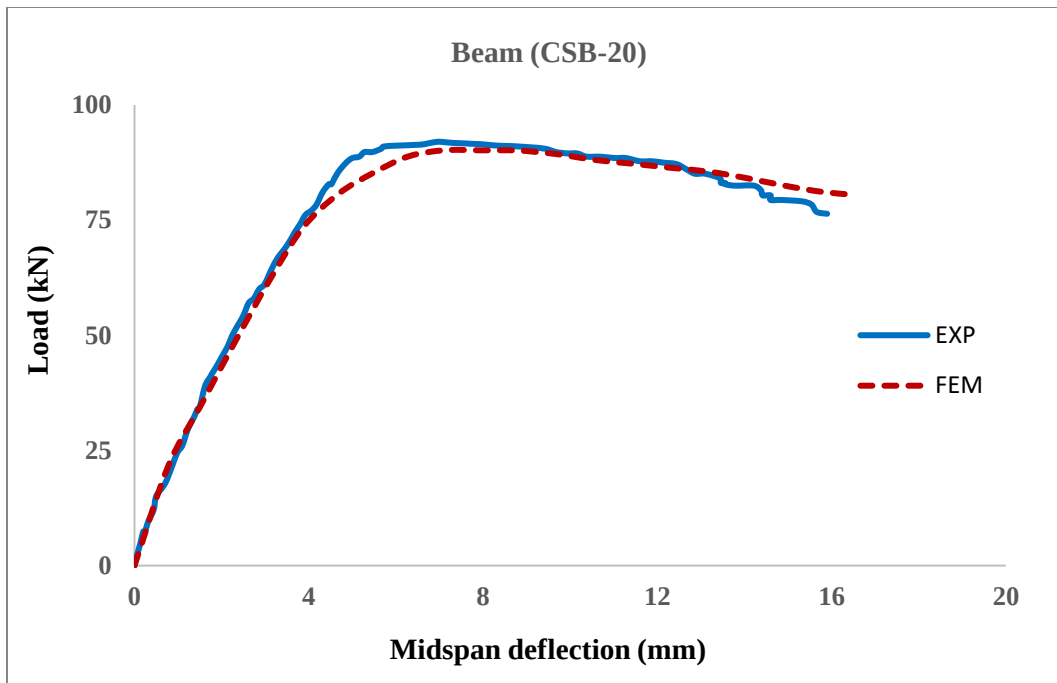


Figure 6.15: FEM vs. Experimental results for beam CSB-20.

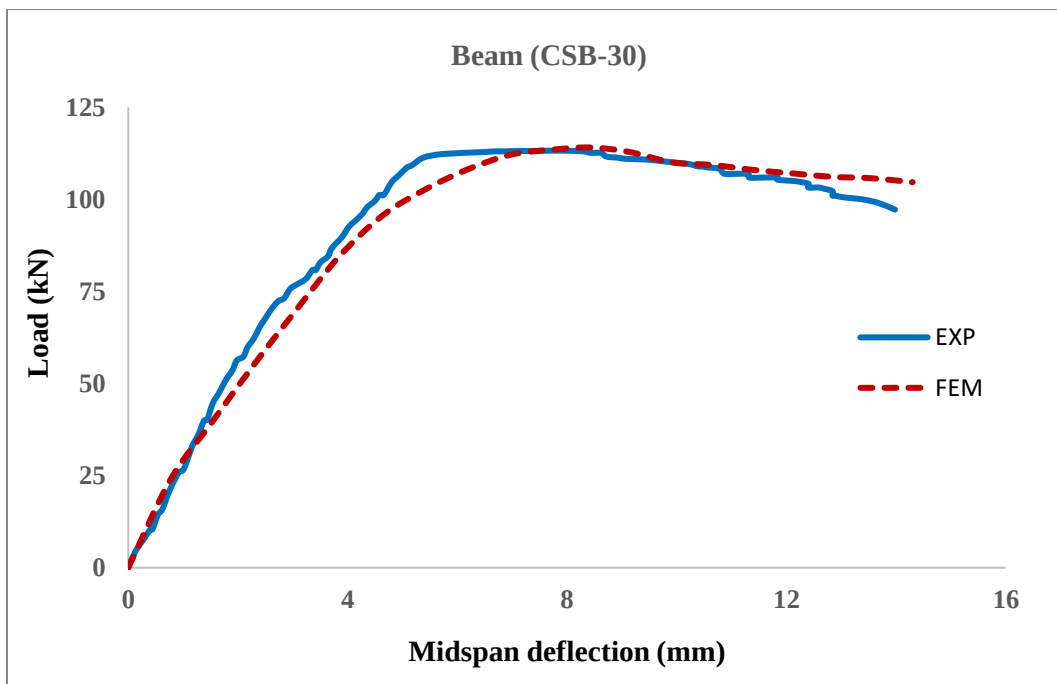


Figure 6.16: FEM vs. Experimental results for beam CSB-30.

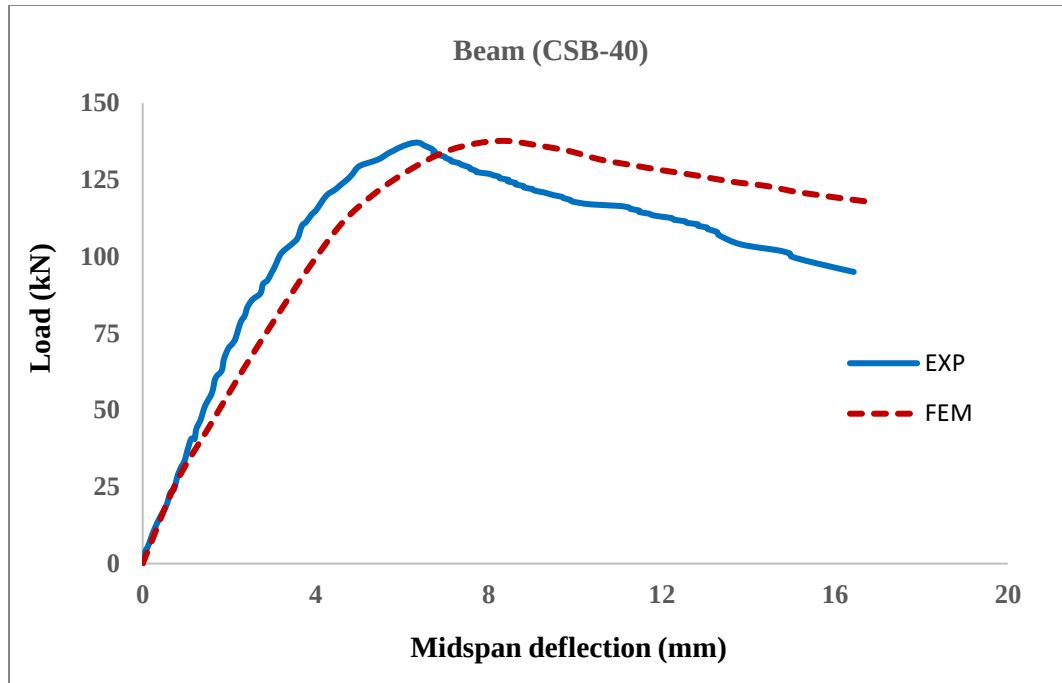


Figure 6.17: FEM vs. Experimental results for beam CSB-40.

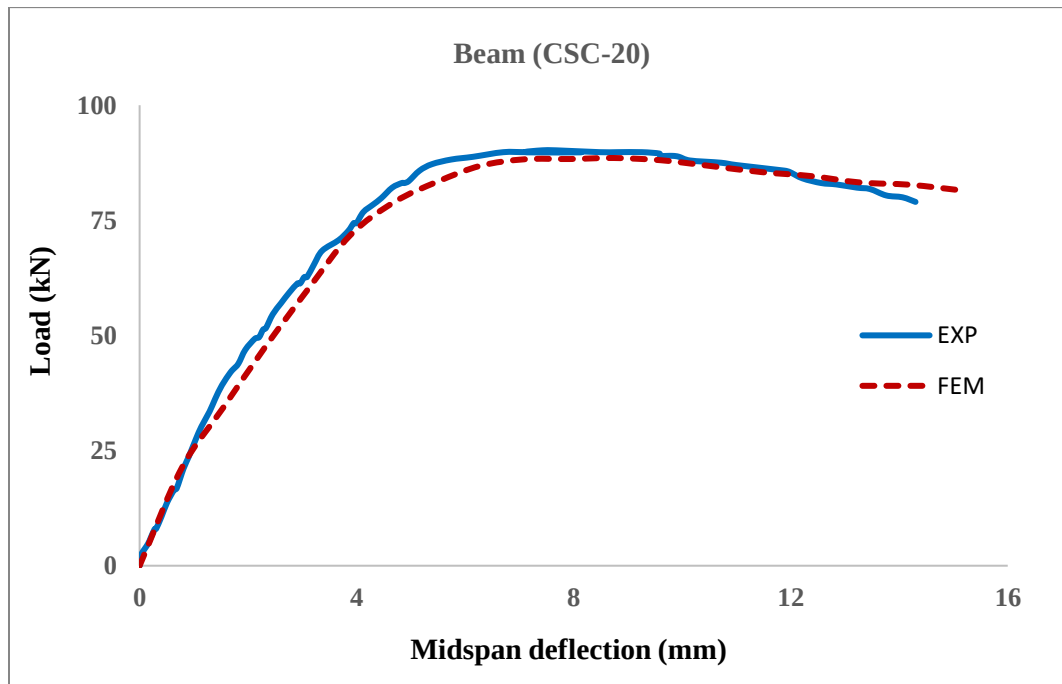


Figure 6.18: FEM vs. Experimental results for beam CSC-20.

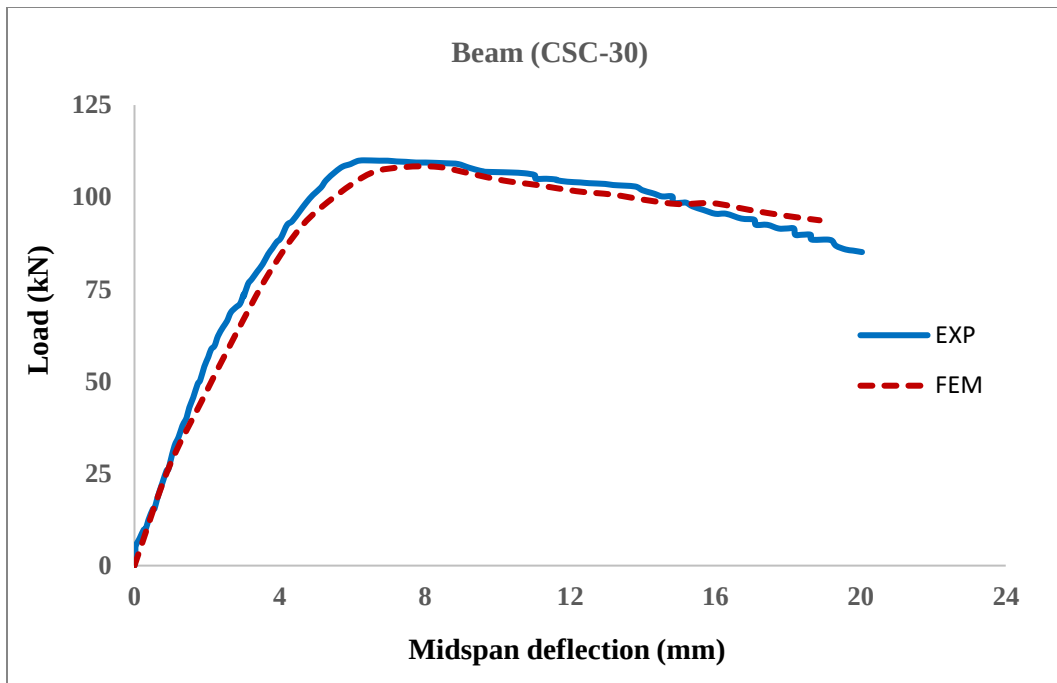


Figure 6.19: FEM vs. Experimental results for beam CSC-30.

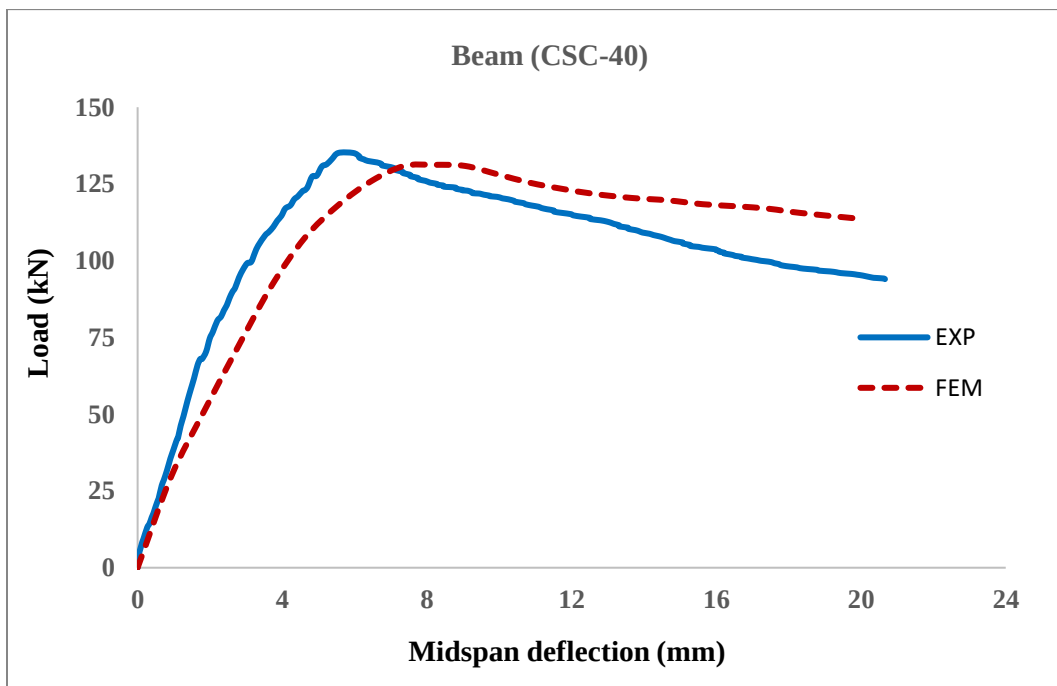


Figure 6.20: FEM vs. Experimental results for beam CSC-40.

Similar behavior of good matching between numerical and experimental results was observed for the UHPC-repaired corrosion-damaged RC beams. The load vs. midspan deflection of the repaired corroded beams was plotted and compared with the experimental testing. Results showed an excellent matching for the repaired beams with 20, 40, and 60 mm UHPC thickness, including stiffness, ultimate load, and overall behavior. However, the results of 60 mm showed a very small variation between experimental and FEM results in the behavior of the beam after the ultimate load. The discrepancy is most likely attributable to the very high thickness of UHPC used as a repair material. The heavyweight of the UHPC layer due to high thickness (60 mm) could be the reason for this discrepancy since the FEM simulated the bond between normal concrete and UHPC as a full bond, which could not always be correct for high UHPC thickness. The figures from Figure 6.21 to Figure 6.29 show the load vs. midspan deflection curves of numerical and experimental results for all repaired corrosion-damaged RC beams.

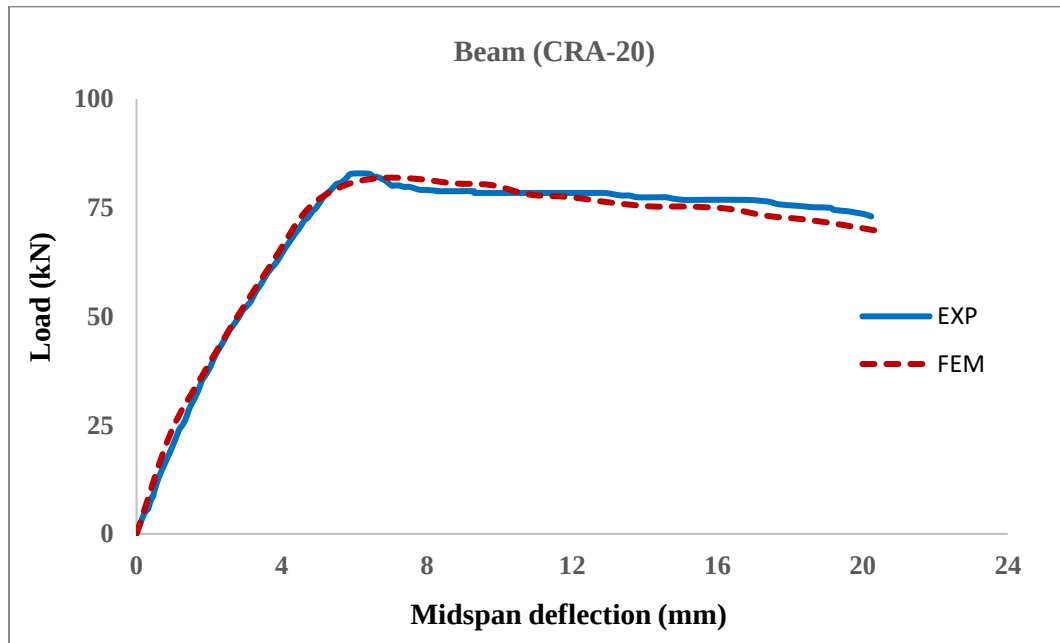


Figure 6.21: FEM vs. Experimental results for beam CRA-20.

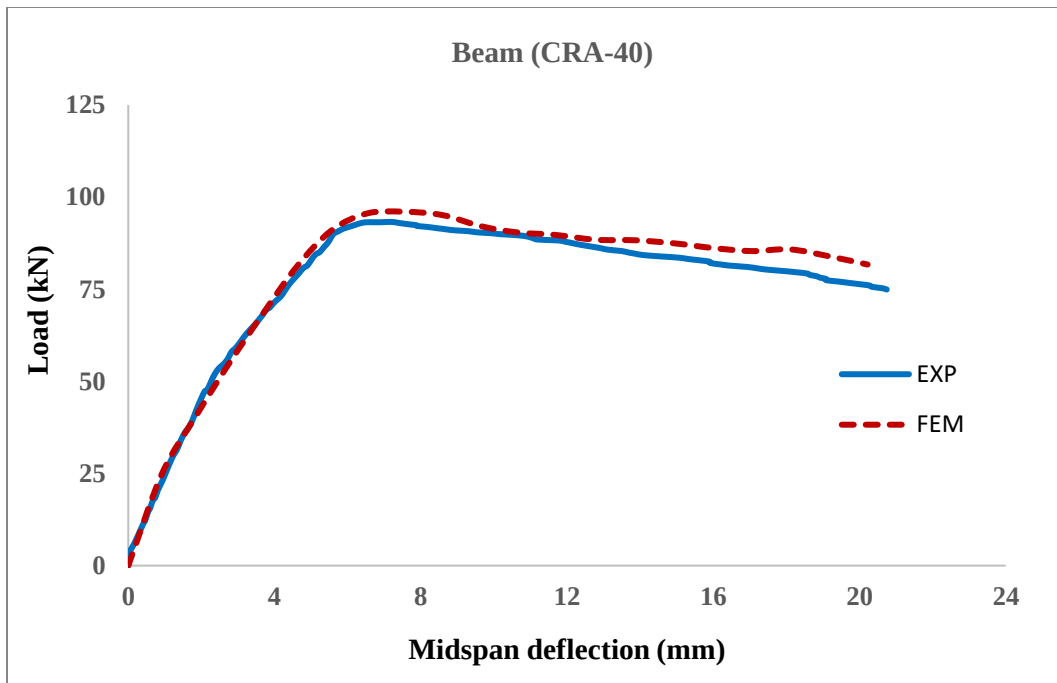


Figure 6.22: FEM vs. Experimental results for beam CRA-40.

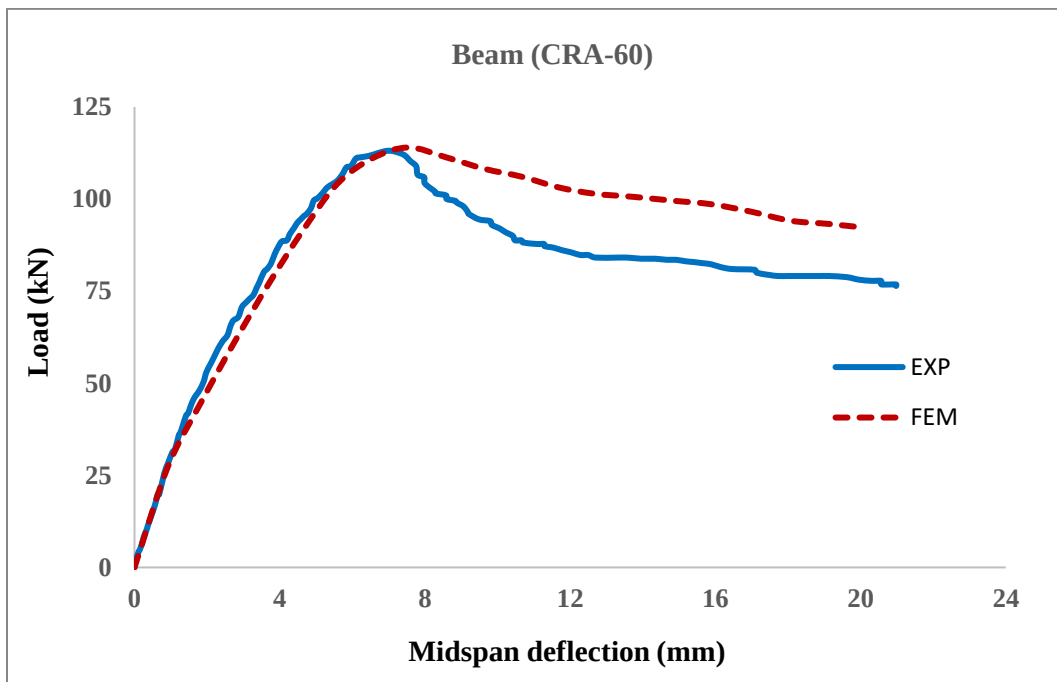


Figure 6.23: FEM vs. Experimental results for beam CRA-60.

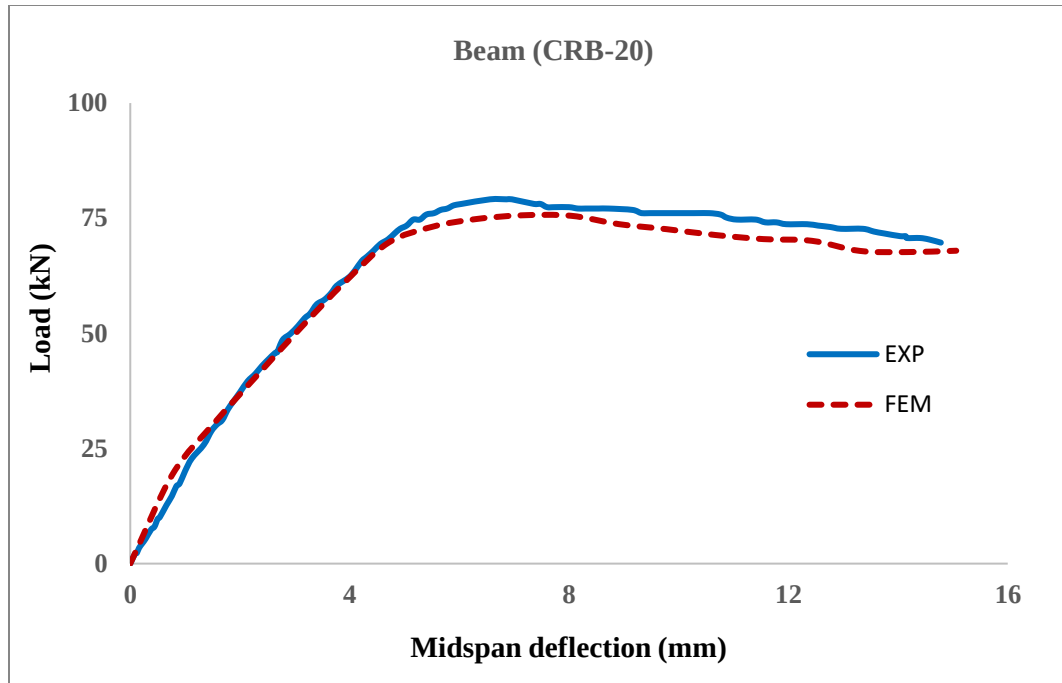


Figure 6.24: FEM vs. Experimental results for beam CRB-20.

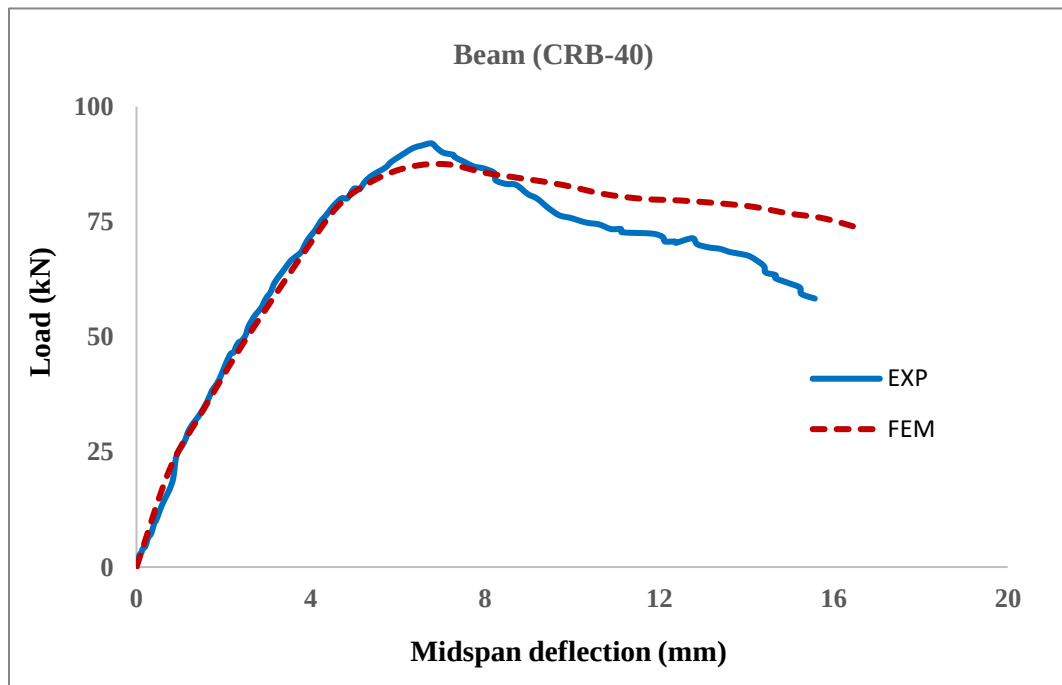


Figure 6.25: FEM vs. Experimental results for beam CRB-40.

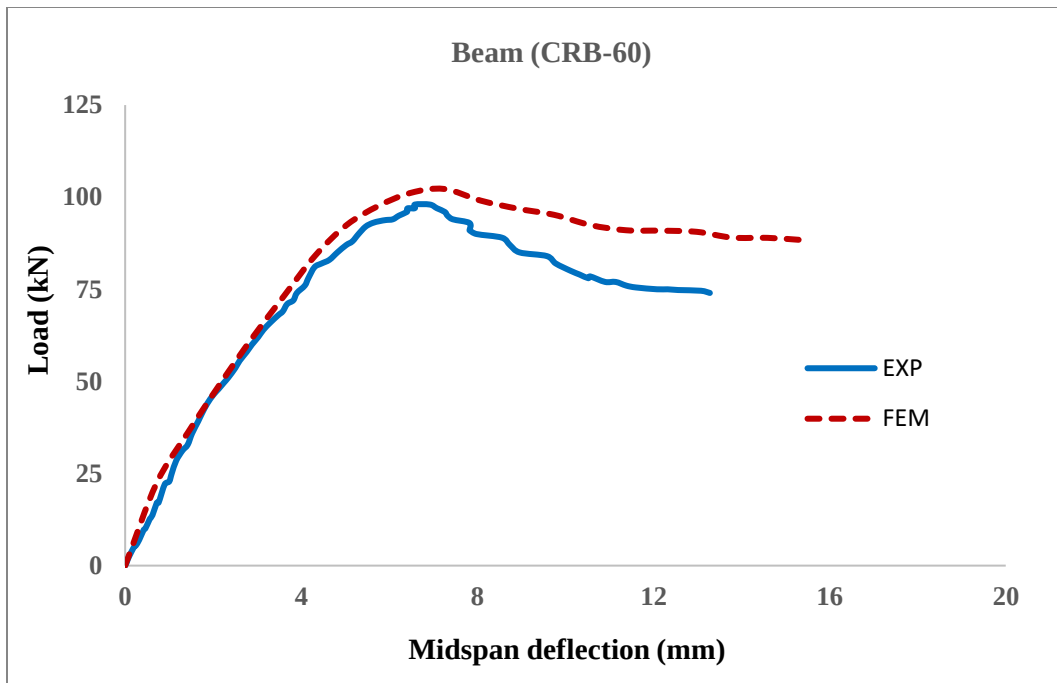


Figure 6.26: FEM vs. Experimental results for beam CRB-60.

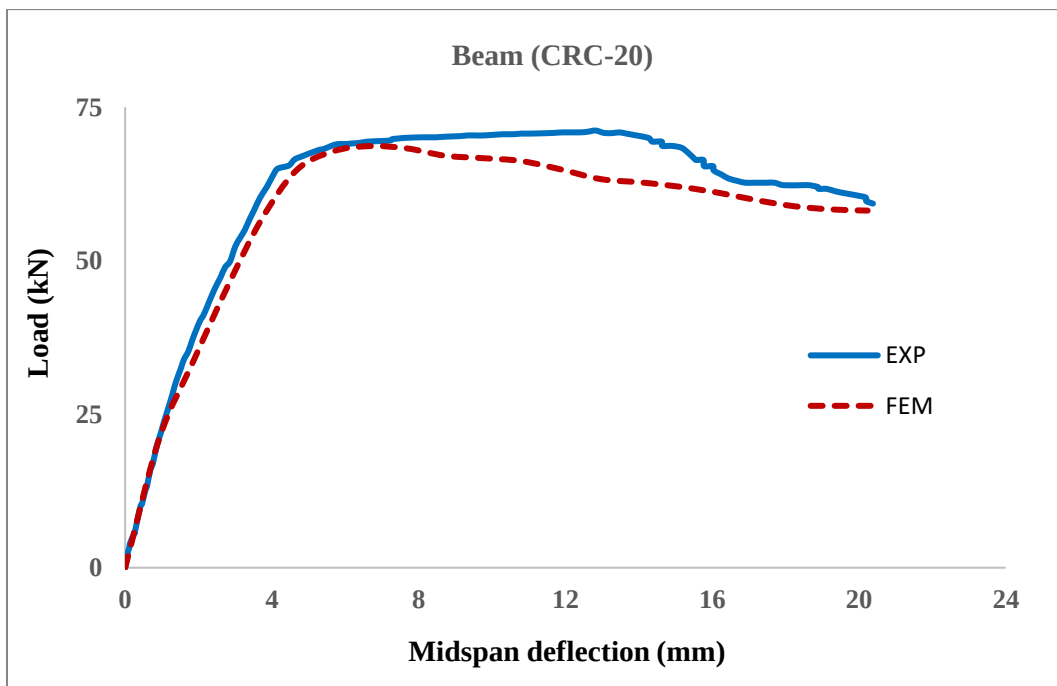


Figure 6.27: FEM vs. Experimental results for beam CRC-20.

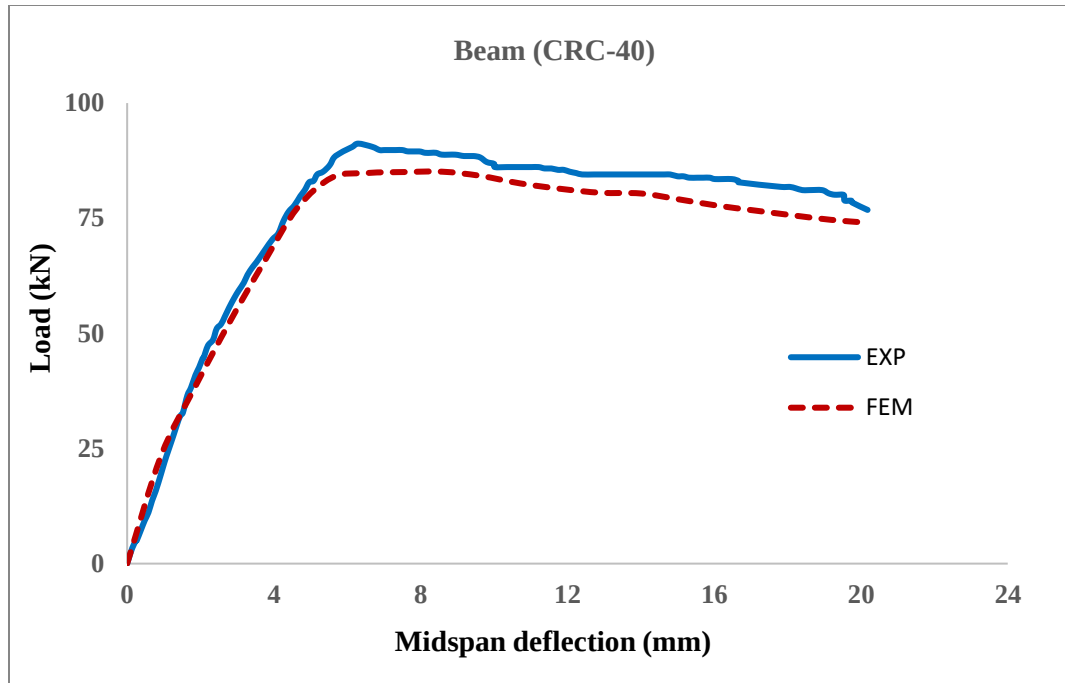


Figure 6.28: FEM vs. Experimental results for beam CRC-40.

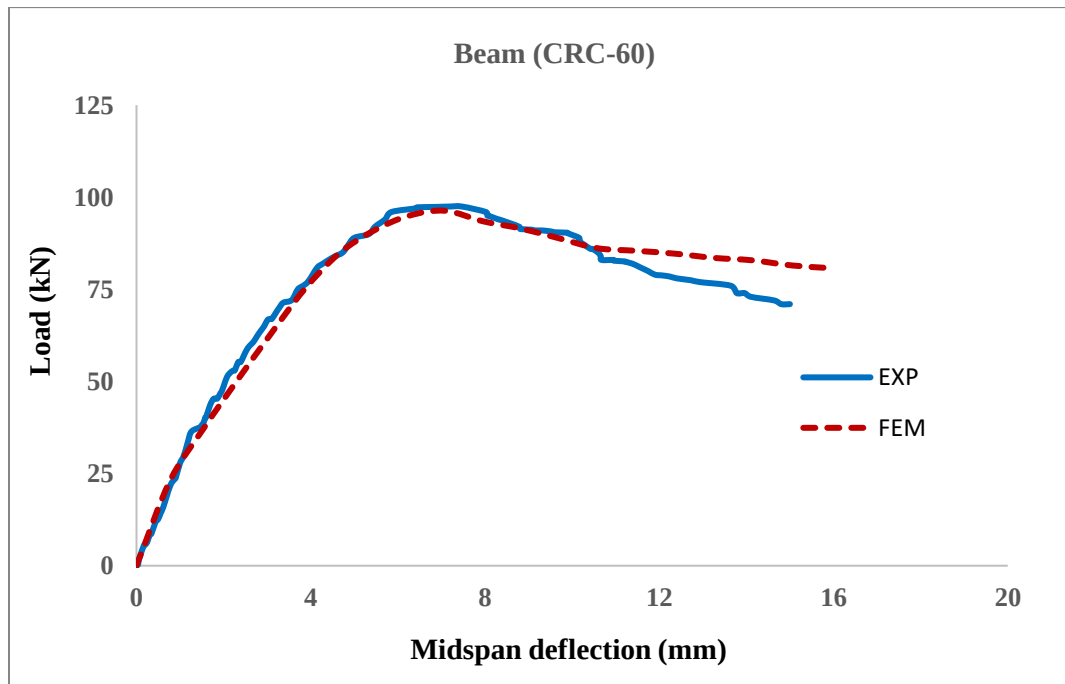


Figure 6.29: FEM vs. Experimental results for beam CRC-60.

The comparison of the crack pattern between experimental and FEM was carried out. Figure 6.30 and Figure 6.31 show the experimental and numerical results of the crack patterns for main control beam UU and CSB-20, respectively. It was observed that there is high resemblance between the experimental and numerical crack patterns. This further shows that how effective the FEMs were in predicting the behavior of the corroded RC beams specimens.

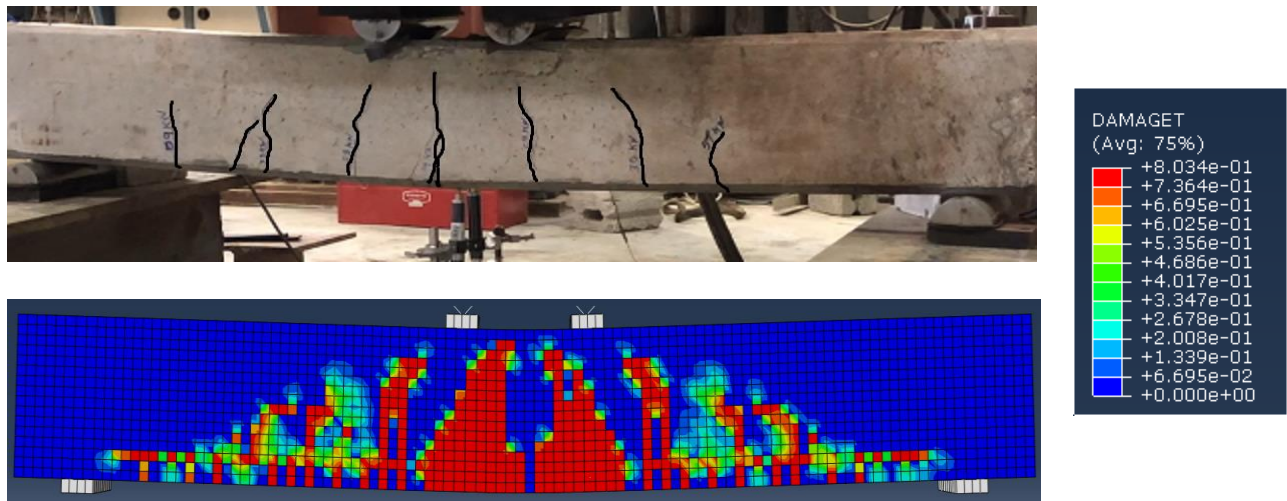


Figure 6.30: Experimental and FEM crack pattern for RC-Control beam UU.

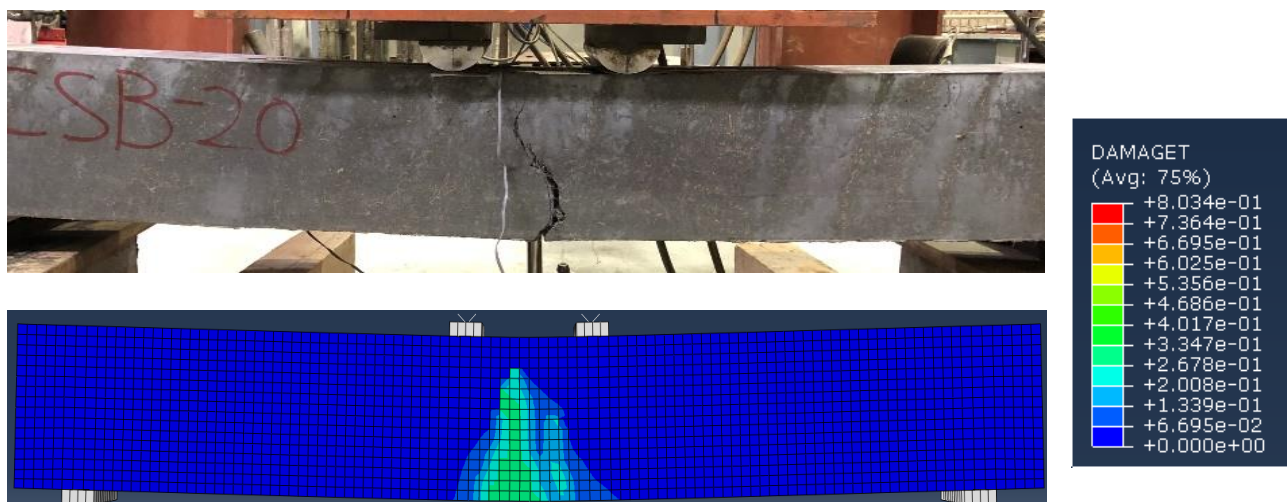


Figure 6.31: Experimental and FEM crack pattern for strengthened beam CSB-20.

6.7 Parametric study

The developed finite element models were verified using the experimental results and used to conduct parametric studies. The purpose of carrying out the parametric study is to investigate the influence of varying UHPC ultimate strength and the degree of corrosion damage on the flexural behavior of repaired/strengthened corroded beams. For parametric study I, the same verified models were used to increase the strength of UHPC layers. Three more strengths of UHPC material were used in this study for both strengthened and repaired beams, and the result is listed in Table 6.2.

Table 6.2: Parametric study I results (varying UHPC strengths)

<i>Ultimate Loads (kN)</i>					
<i>Beam ID</i>	<i>Actual mass loss (%)</i>	<i>UHPC strength (MPa)</i>			
		<i>170</i>	<i>180</i>	<i>200</i>	<i>220</i>
<i>CRA-20</i>	<i>10</i>	<i>81.8</i>	<i>82.5</i>	<i>84.1</i>	<i>85.6</i>
<i>CRA-40</i>	<i>18.1</i>	<i>75.6</i>	<i>76.3</i>	<i>78.0</i>	<i>79.0</i>
<i>CRA-60</i>	<i>22.1</i>	<i>68.7</i>	<i>69.5</i>	<i>70.8</i>	<i>72.4</i>
<i>CRB-20</i>	<i>10.8</i>	<i>96.0</i>	<i>97.5</i>	<i>100.9</i>	<i>103.4</i>
<i>CRB-40</i>	<i>17.8</i>	<i>87.5</i>	<i>88.6</i>	<i>91.6</i>	<i>94.6</i>
<i>CRB-60</i>	<i>20.8</i>	<i>85.1</i>	<i>86.5</i>	<i>90.7</i>	<i>93.8</i>
<i>CRC-20</i>	<i>11.8</i>	<i>114.0</i>	<i>115.8</i>	<i>120.6</i>	<i>123.3</i>
<i>CRC-40</i>	<i>18.3</i>	<i>102.2</i>	<i>104.5</i>	<i>108.2</i>	<i>112.4</i>
<i>CRC-60</i>	<i>22.6</i>	<i>96.3</i>	<i>98.5</i>	<i>102.9</i>	<i>106.1</i>
<i>CSA-20</i>	<i>10.2</i>	<i>101.1</i>	<i>102.4</i>	<i>105.2</i>	<i>108.4</i>
<i>CSB-20</i>	<i>17.5</i>	<i>90.2</i>	<i>91.8</i>	<i>94.8</i>	<i>97.3</i>
<i>CSC-20</i>	<i>21.2</i>	<i>88.5</i>	<i>90.1</i>	<i>92.9</i>	<i>96.0</i>

<i>CSA-30</i>	<i>9.5</i>	<i>120.6</i>	<i>122.6</i>	<i>126.9</i>	<i>131.0</i>
<i>CSB-30</i>	<i>16.7</i>	<i>114.1</i>	<i>115.4</i>	<i>120.1</i>	<i>124.4</i>
<i>CSC-30</i>	<i>22.3</i>	<i>108.2</i>	<i>110.2</i>	<i>115.0</i>	<i>119.6</i>
<i>CSA-40</i>	<i>9.1</i>	<i>143.9</i>	<i>145.6</i>	<i>152.5</i>	<i>158.2</i>
<i>CSB-40</i>	<i>16.8</i>	<i>137.7</i>	<i>138.9</i>	<i>145.4</i>	<i>150.8</i>
<i>CSC-40</i>	<i>20.4</i>	<i>131.2</i>	<i>133.6</i>	<i>139.4</i>	<i>145.6</i>

For the parametric study II, the same verified models were used with only increasing the level of corrosion damage. Four more corrosion levels were implemented in this study, up to 45 % corrosion damage for both strengthened and repaired beams, and the result are listed in [Table 6.3](#).

Table 6.3: Ultimate loads of parametric study II (increasing corrosion degree)

<i>Ultimate Loads (kN)</i>				
<i>Beam ID</i>	<i>Actual corrosion degree (%)</i>			
	<i>30</i>	<i>35</i>	<i>40</i>	<i>45</i>
<i>Repair 20 mm</i>	<i>49.3</i>	<i>41.1</i>	<i>32.6</i>	<i>23.7</i>
<i>Repair 40 mm</i>	<i>66.5</i>	<i>58.4</i>	<i>50.2</i>	<i>41.6</i>
<i>Repair 60 mm</i>	<i>83.9</i>	<i>75.5</i>	<i>68.2</i>	<i>55.0</i>
<i>Strengthening 20 mm</i>	<i>71.7</i>	<i>64.0</i>	<i>56.5</i>	<i>48.5</i>
<i>Strengthening 30 mm</i>	<i>94.9</i>	<i>87.0</i>	<i>78.9</i>	<i>71.6</i>
<i>Strengthening 40 mm</i>	<i>119. 4</i>	<i>110.4</i>	<i>102.8</i>	<i>96.5</i>

CHAPTER 7

CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE WORK

7.1 Conclusions

The present study produced experimental data pertaining to the structural behavior of corrosion-damaged RC beams strengthened with the UHPC layers considering different configurations and UHPC thicknesses. The experimental results were used to develop and validate analytical models that can be used to predict the ultimate load-carrying capacities of the UHPC-strengthened corroded RC beams. In addition, finite element modelling was used to capture the behavior of UHPC-strengthened corroded RC beams using the non-linear software program ABAQUS. Based on the analysis of the experimental results, analytical modeling, and finite element analysis, the following conclusions can be drawn:

- i. A new corrosion setup (based on induced current technique) was developed in the present study, which was found very effective in inducing reinforcement corrosion in large-size RC beam specimens simulating the real-life corrosion in a reasonably short period of time, as confirmed from the visual and cracking inspection of the corroded RC beams.
- ii. The load vs. midspan deflection curves of the control corroded beams show similar flexural behavior compared to the uncorroded control beam UU, which indicates the

effectiveness of using the developed corrosion setup to cause uniform corrosion to the large-size RC beams.

- iii. Longitudinal cracks were observed on the bottom and side surfaces of the concrete beam due to reinforcement corrosion. All measured cracks for corroded beams of groups A, B, and C exceeded the limit of maximum crack width of 0.4 mm (ACI code), indicating the necessity of strengthening/repairing.
- iv. In contrast, no cracks were detected in the strengthening layer of UHPC of the post-strengthened beams after induced corrosion due to the high compressive and tensile strength of the UHPC strengthening layers. These findings indicate that the usage of the UHPC material as a strengthening layer can significantly improve the serviceability of the strengthened RC beams.
- v. The flexural testing of the corroded beams shows the degradation effects of the degree of induced corrosion on the RC beams, including ultimate load-bearing capacity, stiffness, and ductility.
- vi. The yield and ultimate load-carrying capacities of the control corroded beams were reduced by 21 and 17%, 36 and 24%, and 38 and 27%, for the corrosion degrees A, B, and C, respectively, compared to the corresponding values of the main control uncorroded beam (UU) because of the loss in cross-sectional area and loss of the bond, which is significant up to 20% mass loss.
- vii. All the strengthened and repaired corroded beams showed good enhancement in terms of load-bearing capacity, stiffness, and overall structural performance, even with the minimum UHPC thickness of 20 mm.

- viii. The strengthened corroded RC beams with 40 mm UHPC thickness showed the highest enhancement in the ultimate capacity by 97, 109, and 115 % corresponding to the un-strengthened beams for 10, 20, and 30 % mass loss, respectively.
- ix. The stiffness of the RC beams was improved by applying the UHPC strengthening layers by a percentage ranging between 28 and 112 %, depending on the thickness of the UHPC layer used. In contrast, using the UHPC only at the bottom sides enhanced the stiffness by a percentage ranging between 26 and 82 %.
- x. The improvement of the load-carrying capacity and stiffness of the strengthened beams is not significantly affected by the degree of reinforcement corrosion, i.e., up to 22.3%. However, the UHPC thickness and configuration have the most significant influence on the enhancement results.
- xi. The strengthened beams using UHPC showed ductility and energy absorption indices ranging between 42 and 56% and between 38 and 60%, respectively, compared to the uncorroded control beam (UU).
- xii. The effect of corrosion on post-strengthened RC beams is subsequently lower than that of the strengthened beams even though both beams were subjected to the same duration of accelerated corrosion, indicating the effectiveness of using UHPC layers to reduce the influence of corrosion in RC beams. UHPC has shown excellent performance as protecting material for steel bars against the aggressive environment and further corrosion.
- xiii. The application of UHPC strengthening material for post-strengthened beams could lower the effect of corrosion on the mass loss of steel bars area by 21 and 9 % for groups B and C, respectively, compared to the strengthened beam specimens.

- xiv. The analytical models developed in this study for control, strengthened, and repaired corroded RC beams showed a good prediction of the experimental results with a minimum percentage of error, not exceeding 7%.
- xv. The developed analytical models can predict the ultimate strength of the strengthened RC beams with high degree of accuracy, which would help in selecting the optimum thickness and configuration of UHPC layers required for strengthening the corroded RC beams.
- xvi. The finite element models developed in the current study can capture the flexural behavior of the corroded RC beams strengthened/repared using layers of UHPC with high degree of accuracy, including ultimate load-bearing capacity and stiffness, indicating the efficiency of implementing the numerical modelling to predict the performance of corroded RC beams.
- xvii. The load vs. midspan deflection curves of the numerical models showed good matching with the curves obtained from the results of the experimental tests, including control, strengthened, and repaired corroded beams.
- xviii. Parametric studies carried out using the developed FE models using various corrosion degrees and UHPC compressive strengths showed the variation of the corrosion levels has a significant influence on the ultimate strength of the corroded beams other than the variation in the compressive strength of the UHPC layers.

7.2 Applications

- i. The developed novel setup for inducing reinforcement corrosion could effectively cause uniform corrosion of the steel reinforcing bars in RC members similar to natural corrosion.

- ii. The UHPC strengthening technique can be used in various RC structures such as bridges, girders, towers, water tanks, etc. utilizing UHPC which could easily be produced using readily available ingredients.
- iii. In the case of RC beams corroded to cause a mass loss of 22.5 %, the repair/strengthening strategies using layers of UHPC material can restore the original strength of the virgin beam with an extra enhancement of more than 100 %, depending on the thickness of the UHPC layers.
- iv. The use of UHPC can provide a strong passive layer on the steel bars that prevent aggressive species in harsh environments from penetrating and protect the reinforcing bars from further corrosion.

7.3 Future Work

- i. This study focused on RC beams; more research on other RC elements, such as columns and slabs, is needed for future research.
- ii. The influence of using UHPC for strengthening corroded RC beams should be investigated under sustained loadings.
- iii. Lower and higher corrosion levels than those used in this study should be examined for their effect on UHPC strengthened/repared beams.
- iv. The effect of using composite strengthening techniques on the corrosion-damaged RC elements such as UHPC layers with high strength reinforcing bars or GFRP bars needs to be investigated.

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