EXPERIMENTAL AND NUMERICAL STUDY OF RC BEAMS STRENGTHENED IN FLEXURE WITH BOLTED/BONDED AA PLATES

by
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A Thesis Presented to the Faculty of the
American University of Sharjah
College of Engineering
in Partial Fulfillment
of the Requirements
for the Degree of
Master of Science in
Civil Engineering
Sharjah, United Arab Emirates
April 2019

Approval Signatures

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Thesis Title: Experimental and Numerical Study of RC Beams Strengthened in
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Acknowledgement

I would like to express my sincere gratitude to my supervisors, Dr. Jamal A. Abdalla and Dr. Rami Hawileh, for their continuous support during my M.Sc. degree in the American University of Sharjah. Many thanks as well to the College of Engineering, Department of Civil Engineering at AUS for providing me with the teaching assistantship throughout my graduate studies. I would also like to thank my family and friends who encouraged me and supported me during my graduate career. Finally, I would like to thank Engineer Ansari and Engineer Arshi Faridi for their assistance and guidance during the experiment.

Abstract

Reinforced Concrete (RC) members are susceptible to deterioration due to many factors. Externally bonded reinforcement (EBR) such as fiber-reinforced polymers (FRP), had emerged as one of the proven techniques for flexural strengthening and retrofitting of RC members. This is due to its practicality and structural effectiveness; however there are shortcomings that include premature de-bonding/de-lamination failure or brittle FRP rupture failures. The use of mechanically anchored Aluminum Alloy (AA) plates instead has the potential of overcoming these drawbacks by providing both strength and ductility while influencing the failure modes. In this project, 16 RC beams were prepared, one beam was left unstrengthened (CB), one was strengthened with AA plate using epoxy only (CBE), and 14 beams were strengthened with AA plates with different bolt sizes, spacing, bolt layout and epoxy. The specimens were tested to failure and all specimens with bolted AA plates exhibited approximately 30% increase in strength accompanied with drastic increase in ultimate ductility (56.5%) and failure ductility (84.1%) compared to the control beam with epoxy (CBE). It is concluded that the implementation of a hybrid anchorage system (i.e., bolts with epoxy) in retrofitting applications serves as a viable option for fixing AA plates to RC beams. Furthermore, nonlinear finite element (FE) models for all specimens were developed using validated constitutive laws for capturing the nonlinear properties of the materials. Contour plots and concrete cracking patterns were generated to monitor the stress and cracking propagation for each model. The FE predictions closely resemble that of the experimental results in terms of load-deflection, cracks patterns and failure modes. This validated the use of FE as a simulation tool for further investigating the behavior of RC beams strengthened with externally bonded and bolted AA plates.

Keywords: Aluminum Alloy plates; Mechanical fasteners; Finite element modelling.

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List of Abbreviations

AA Aluminum alloy
CC Concrete crushing
CS Cover separation
EBR External bonded retrofit
ED End-plate debonding
FE Finite Element
FRP Fiber-reinforced polymers
IC Intermediate crack-induced debonding
PR Plate Rupture
RC Reinforced Concrete
SY Steel Yielding

Chapter 1. Introduction

In this chapter, a brief explanation of the strengthening techniques and materials, used to rehabilitate structural members, is provided. Past studies and articles taken from the literature and websites are used to articulate this section. Afterwards, the problem statement and research significance are presented to help construct the research hypothesis. Finally, the structure of the thesis is briefly summarized.

1.1. Overview

A significant number of high-rise buildings were constructed during the first half of the 20th century using reinforced concrete, precast and steel materials. Now, many of these buildings have reached the end of their planned service life, where deterioration in the form of steel corrosion, concrete cracking and spalling would have a detrimental effect on their structural integrity [1]. Moreover, the advancement of several building standards (i.e. ASCE7-16 [2]) have modified their load factors in which the structures that were previously designed to withstand a certain factored loading combinations are now considered structurally over-loaded [3–5]. Therefore, construction industries have implemented strengthening strategies to upgrade the structural systems of existing buildings and improve their physical performance under existing or modified loads [6, 7]. This advocated researchers to explore different strengthening techniques [8–13].

The first common method of retrofitting was implemented by bonding steel plates to the soffits of Reinforced Concrete (RC) beams using epoxy as a binding agent. Researchers observed a significant increase in capacity and ductility when loading steel plated RC beams as opposed to regularly RC beams [14–16]. However, its high-corrosive properties and large density made steel plates a poor externally bonded reinforcement (EBR) material [10]. As a result, novel techniques for rehabilitation of RC structures have been incorporated in which Fiber Reinforced Polymers (FRP), in the form of paper-thin sheets, have been bonded and wrapped around damaged structural members to increase or maintain their loading capacity. Owing to their high-strength, lightweight, non-corrosive properties, speed and ease of installation, strengthening using FRP has been a popular technique and research topic over the past two decades [17–22]. These systems have been used extensively in the aerospace, automotive, and ship-building industries, and are becoming a mainstream technology in the structural retrofit field [1]. But an underlying issue that prevents the member from utilizing the full capacity (mechanical properties) of the FRP sheets is its anchorage system. This phenomenon is known as debonding and occurs when the FRP sheet loses its bond with the adjacent concrete surface, due to a crack initiation from the large strain in the FRP sheet, and detaches from the concrete substrate. By exhibiting this premature failure mode, the strengthened section does not have enough ductility to utilize the composite’s mechanical properties. Other failure modes that occur in retrofit applications are shown in Figure 1-1, where the most popular failure mode is the plate end interfacial debonding, which was discussed earlier. In general, this failure mode occurs when anchorage techniques like wraps and anchors are not used. Another failure mode is cover separation when a large volume of the concrete is still bonded to the debonded composite possibly making the beam’s internal reinforcement visible. Intermediate crack-induced interfacial debonding is when the composite buckles from the beam’s midspan, and shear failure mode is when the beam’s section shear deficient whereby it fails predominantly in shear. Therefore, researchers have expanded this topic by implementing other anchorage techniques such as wrapping the FRP in specific orientations [23–27] or mechanically fastening the composite material to the RC member using bolts or FRP splays [26, 28–32].

If anchored correctly, FRP-strengthened sections primarily fail by rupture after the strain in the FRP sheet reaches its ultimate strain. Therefore, ductile strengthening material should be used with an effective anchorage system such that the section exhibits strain hardening followed by strain softening during its loading life. Aluminum Alloy (AA) plates are one of the ductile materials that could be implemented as an alternative to FRP composites, since it is capable of yielding considerably before failing by rupture. The comparisons associated with the previously mentioned composite materials are reflected by observing the stress versus strain curves shown in Figure 1-2. The stress versus strain values of CFRP were obtained from [20] and the steel bar and AA plate values were obtained from the coupon tests conducted in this study. Other drawbacks associated with bonding FRPs to structural members are summarized in [33]. The

Figure 1-1: Debonding failure modes [23].

4000
3500
3000
2500
CFRP Sheet
2000
AA Plate
Steel Bar
1500
1000
500
0
0 3 6 9 12 15
Strain (%)

Figure 1-2: Stress versus strain curve for strengthening materials [20].

implementation of AA plates in retrofit applications has been conducted for both shear and flexural deficient RC beams in several studies [10, 11, 34–36]. Mainly, Abdalla et al. and Rasheed et al. [10, 11] have reported that the incorporation of AA plates in retrofit applications have allowed the section to increase in strength, stiffness and ductility when compared to other types of EBR. However, the plated specimens did not reach

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Stress (MPa)

the ultimate strain of the AA plate due to premature debonding. Therefore, this study focused on: (a) preventing premature debonding; (b) increasing stiffness, ductility and failure modes of AA plated RC beams.

1.2. Problem Statement

The method of using EBR materials to strengthen aging and/or deteriorating RC structures has proven its feasibility and effectiveness in the past studies [8, 14, 37]. The implementations of steel, FRP and other strengthening materials in retrofitting applications has shown promising results in terms of increasing the capacity and stiffness in RC members. The usage of AA plates in retrofit application has proven its viability in the area of strengthening; however, these studies [10, 11, 34–36] have reported premature debonding due to the inefficient anchorage systems. Therefore, the incorporation of bolting and bonding AA plates, which have the capabilities of overcoming some of the old materials’ shortcomings, could physically supplement the RC member with both strength, stiffness and ductility. In addition, the utilization of bolts in anchoring the AA plates may overcome the typical failure modes summarized in Figure 1-1 and allow the loaded section to use the full potential of the AA plate. The introduction of bolting AA plates as a strengthening application will be advantageous to engineers and researchers locally, regionally and internationally.

1.3. Research Significance

The susceptibility, of aging and/or deterioration of RC structures has been a growing issue in the beginning of the 20th century. This resulted in weakening of RC structures and reduction in their flexural capacity. Moreover, several buildings are being considered as overloaded due to the changes in load factors given by structural building standards like the ASCE7-16 [2]. Therefore, engineers remedied these issues by utilizing FRP or steel material as EBR to effectively maintain or increase the loading capacity of the structural member. However, due to their limitations such as: brittle failure and sensitivity to fatigue discussed earlier, its deficiencies were highlighted, and triggered engineers and researchers to propose different solutions. As a result, structural design standards like the American Concrete Institute (ACI-440.2R-08) offer systematic design procedures when evaluating the FRP retrofitted sections [38]. These techniques underestimate the FRP's capacity to avoid premature failure modes, like debonding

or delamination. In general, failure modes govern whether the section is utilizing the EBR completely or not. Therefore, both new strengthening material and anchorage techniques need to be identified to mitigate these shortcomings and make a sound contribution to the field of strengthening and retrofitting using EBR.

In this research, both materials and anchorage systems were the test parameters that were investigated, where AA plates were used as the strengthening material and bolts with/without epoxy were used as the anchorage technique. These materials were obtained from external manufactuerers [39, 40]. The significance of this research is to investigate the feasibility of bolting and/or bonding AA plates, as an alternative EBR, to retrofit flexurally deficient RC beams.

1.4. Research Objectives

This research is conducted to study the flexural behavior of externally strengthened RC beams using AA plates anchored with bolts with/without epoxy. The objectives of this study are:

1. Conduct an experimental investigation to study the strength and ductility of AA plates as EBR material.
2. To study the strength, stiffness and ductility of flexurally deficient RC beams that are externally strengthened using AA plates anchored with bolts and/or bonded with epoxy.
3. Study the effect of bolt size, spacing and embedment depth on the failure mode of the externally strengthened RC section.
4. Perform a parametric study by varying the spacing and embedment depth of the

bolts.

5. Predict the load versus deflection curves of selected specimens with a nonlinear finite element analysis using a commercial software, Ansys Mechanical APDL [41].

1.5. Thesis Organization

The work described herein consisted of:

1. Chapter 1: Introduction. Provides an overview on the effects of anchoring composite materials to RC members using different techniques, addresses the signif-

icance of this project in the research community, and presents the objectives of this study.
2. Chapter 2: Literature Review. Presents a comprehensive literature review that covers different external retrofit strategies that have been investigated in past experimental and numerical studies.
3. Chapter 3: Experimental Program. Covers the geometric and mechanical properties of the RC beams, AA plates and anchorage techniques used in this study. It describes the procedure for testing the mechanical properties of the EBR system, the preparation steps taken for strengthening the RC beams, and the test setup followed during this project for the RC beam specimens.
4. Chapter 4: Results and Discussions. Provides a detailed discussion of the recorded results taken during testing, where both load versus deflection and load versus strain curves were generated for each beam. The load and deflection values at different states which include: (a) yielding of steel; (b) yielding of AA plate; (c) crushing of concrete; (d) ultimate load; (e) failure of section were identified. Furthermore, the effects of the various parameters were explored by comparing the stiffness response, load and deflection values for each specimen. Captures of all specimens, at failure, were presented and related to the strain measurements for each material. Finally, the ductility index of each specimen was evaluated to quantify the amount of ductility experienced by each specimen.
5. Chapter 5: Finite Element Modeling. Presents a comprehensive approach on modelling retrofitted sections using a commercial FE software. Previously de-

rived constitutive models were employed within the FE software to simulate the nonlinear properties of specimens. All specimens were successfully modelled in which the results obtained from the FE software correlated very well with the results obtained from the experiment. These validated models were then used to generate contour plots and concrete cracking patterns where the stress and crack propagation were observed and discussed.

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6. Chapter 6: Conclusion. Summarizes the key findings deduced from the experimental and numerical investigations carried out in this study.

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Chapter 2. Literature Review

A general increase in high-rise buildings and skyscrapers exist in countries where heavy real estate companies are based, like: United Arab Emirates, Singapore, China, Mexico City, Canada, USA and many more [42]. These immense buildings necessitate government sectors and clients to start developing strategies that would help prolong these structures for a longer period of time. Strengthening and rehabilitation of structures has been an effective solution in repairing deteriorated or damaged members since the beginning of the 20th century [1, 6, 7, 11, 43]. Namely, many investigations were conducted to study the flexural behavior of strengthened RC beams and their failure modes using FRP, steel, and AA plates [4, 10, 14, 16, 37]. This chapter presents an extensive literature review on the implementation of strengthening using various composites that vary based on their mechanical properties and anchorage techniques.

2.1. Flexural Strengthening

Externally bonding materials to the surfaces of structural members is a common retrofitting practice that several researchers have performed using different types of composite materials. Mainly, these applications include fixing materials like steel plates, Glass FRP (GFRP) sheets/plates and CFRP sheets/plates onto the soffits of RC beams to enhance their flexural capacity. Furthermore, the implementation of different FRP material with different textures (plates or sheets) were reviewed and summarized in the following subsections.

2.1.1. Steel plates. The use of bonded steel plates, as an EBR system for RC members, was first reported in 1964 when malleable steel plates with an adhesive compound were applied to load bearing beams in the basement of an apartment building, in Durbin, South Africa [44]. This motivated several researchers to conduct experimental investigations on using steel plates for external strengthening applications. Oh et al. [45] investigated the static and fatigue behavior of RC beams strengthened with steel plates. The experimental program involved 27 RC beams that were divided into two categories: static and fatigue tests. Fourteen specimens were used for the static test, where one of the specimens was reserved as a control specimen and the remaining thirteen specimens were externally strengthened with different steel plates and adhesive

thicknesses. Four RC beams were tested against fatigue, where one of the specimens was reserved as a control specimen and the remaining three were strengthened similar to the strengthened specimens in the static test. Furthermore, load versus deflection curves were generated in which all steel plated specimens were not able to fully utilize the steel plates’ capacity and ductility due to end-debonding and cover separation failure modes as shown in Figure 2-3. As a result, plate separation was the main contribution to the specimens failure in both tests, where some RC beams failed by shear; due to the enhanced strengthening of their flexural capacities.

Figure 2-3: Load versus deflection curves of steel plated specimens [45].

Moreover, Swamy et al. [46] performed an experimental study on the effect of plate thickness in the failure mode of the strengthened RC beams. It was observed that a 3 mm thick steel plate bonded to the soffit of an RC beam exhibited failures that consisted of a combination of flexure and flexure-shear failure modes, with flexure being the most common. However, at ultimate load, the section failed in concrete cover separation due to the initiation of a shear crack at the end of the plate which propagated along the beam. For 6 mm plates, the failure mode consisted of shearing of the concrete along the internal bottom reinforcement, causing a concrete cover separation failure.

As a result, the failure was sudden and brittle. Similarly, Gao et al
[18] performed a
comparative experimental study where different plating materials, loading
cases, and
end anchorage techniques were the varying parameters. However, only
plating of steel
as an EBR material will be discussed here. Furthermore, Gao et al.
prepared three
23
out of 17 RC beams where one beam was left un-strengthened (regarded as
the control
beam) and the other two specimens were bonded with steel plates. After
monotonically
loading the specimens, it was observed that a drastic increase in
capacity was achieved
in the plated specimens as compared to that of the control specimen. As a
result, the
control beam failed in flexure whereas the strengthened specimens failed
due to cover
separation.
2.1.2. FRP plates. Recently, FRP were integrated with retrofit
applications due
to their extremely light weight characteristics combined with their
superior mechanical
properties. This was accomplished by bonding FRP plates to the soffits of
RC beams,
as shown in Figure 2-4. Rahimi and Hutchinson [47] conducted an
experimental study
on bonding two types of FRP plates, CFRP and GFRP, to the soffits of RC
beams. The
test matrix consisted of three beam groups with similar cross-sectional
details, where
the first two groups were reinforced with the same longitudinal
reinforcement and var-
ied with shear reinforcement, and the last group was reinforced with more
longitudinal
reinforcement than the first two beam groups. Moreover, the strengthening
techniques
used for all specimens were consistent, in which all RC beams' soffits
were externally
bonded with FRP plates along the beams' spans. As a result, the
strengthened speci-
mens in the first two groups, whether using CFRP or GFRP laminates, have
failed pre-
maturely; mainly, in end-debonding/cover separation, shear or a
combination of both.
However, the FRP strengthened specimens in the last group failed by
concrete crushing
followed by debonding/cover separation. Therefore, Rahimi and Hutchinson
have con-
cluded that despite varying the strengthening material type, longitudinal
and vertical
reinforcements; the failure modes exhibited by flexurally strengthened RC
beams are

governed by end-debonding and cover separation, respectively.

Gao et al. [18] prepared five out of 17 RC beams that were plated with CFRP platess in which three preloading conditions were simulated: pre-unloading, sustained load at 5 kN and sustained load at 10 kN. The test setup consisted of a four point bending that followed a displacement protocal whereby all specimens were crushed until the sections' maximum capacity was reached. As a result, the CFRP plated specimens exhibited two failure modes: failure due to debonding of CFRP plates at ends and shear failure of concrete (for pre-unloaded specimens). Table 2-1 summarizes the other in-

vestigations of different plating material, mechanical behavior and failure modes. The failure modes described herein were categorized into two types; sudden and typical. The sudden failure modes were reported as end-debonding (ED), intermediate-crack debonding (IC), or cover separation (CS). The typical failure modes were reported as concrete crushing (CC) and steel yielding (SY), or plate rupture (PR) with the combination of CC and SY.

Figure 2-4: Externally strengthened continuous RC beam [48].

Table 2-1: Past investigations of different plating material.

Strengthening Material Material Behavior Failure Mode
Reference CFRP GFRP Steel Ductile Brittle Sudden1 Typical2
[47] X X X
[18] X X X
[48] X X X
[49] X X X
[50] X X X
[51] X X X
[45] X X X
[46] X X X
[18] X X X

1 ED; IC; CS
2 CC; SY; PR

2.1.3. FRP sheets. After realizing the drawbacks of using FRP platess in retrofit applications, engineers and researchers have investigated the implementation of using FRP sheets as an alternative. The flexibility of FRP sheets allows engineers to incorporate them in several applications including retrofit. Arduini et al. [52] performed an experimental investigation in which 18 RC beams were prepared and cast; mainly, two groups of nine beams were divided according to their geometric and reinforcement details. One beam of each group was left un-strengthened while the rest of the beams of each group were strengthened using CFRP sheets by varying the number of sheets used and their orientation. As a result, all the FRP-strengthened specimens exhibited an increase in flexural capacity compared to that of the control beams; however, their failure modes were sudden and involved debonding or cover separation from the concrete surface whereas the control beam typically failed in flexure. Therefore, it was concluded that the implementation of FRP sheets promoted stiffness and impeded ductility.

Other researchers arrived at similar conclusions in which the implementations of different strengthening material like CFRP, GFRP, and a hybrid of both were experimentally tested [13, 17, 53–55].

2.1.4. Aluminum alloy plates. Recently, researchers have begun studying high tensile strength AA plate as a strengthening technique [3, 10, 11, 20] due to its high strength and ductility comparable to that of steel, light weight comparable to that of FRP, and high resistance to both corrosion and temperature degradation. An experimental study was conducted by Rasheed et al. [10] where AA plates were used as an EBR retrofit with/without single-layer and double-layer U-wrapped CFRP sheets for RC beams. The program included a group of beams, strengthened and unstrengthened, and loaded monotonically to test the flexural behavior of the beams. It was observed that the strengthened beams, without end anchorage, had an increase in strength from 13% to 40% and an increase in ductility when compared to the un-strengthened specimen. In contrast, the strengthened beams anchored with variable layers of CFRP sheets also exhibited higher ductility but lower strength capacity than the un-strengthened specimen. The failure modes for the strengthened beams without end anchorage was failure by full debonding whereas the strengthened beam with end anchorage failed by localized debonding and flexure. Ultimately, a strain-hardening model was incorporated to

capture the stress-strain curves of the AA plates and to predict the strengthened RC

beams' response as an analytical approach, which agreed with the experiment's results.
Furthermore, it was shown that the debonding strain formula, adopted by ACI 440.2R-08 [38], was capable of predicting debonding strain of AA strengthened beams without end anchorage.

2.2. Anchorage Techniques

Several comparative studies were performed to prevent or postpone the overall failure mode, debonding or delamination, and force the strengthened RC beams to fail due to FRP rupture or shear. Different anchorage techniques were studied such as: FRP sheet U-wraps, FRP splay anchors and steel bolts.

2.2.1. U-wrap anchorage. Plate-end anchorage was studied to verify whether a cover separation failure mode is avoidable as Smith and Tag [56] demonstrated when adding plate-end U-jackets, which was able to shift the failure mode from cover separation or end-interfacial debonding to concrete crushing. As a result, researchers began distributing the U-jackets across the span of the RC beam, with consistent spacing. This method resulted in shifting most of the failure modes to FRP rupture [25, 57–59]. Although Pham and Al-Mahaidi [57] found that the addition of U-jacket, across the span, contributed in limiting end debonding failure mode, the failure mode was shifted to intermediate-span debonding at a higher load. This was often coupled with the rupture of the end U-jacket. Ali et al. [16] concluded that RC beams externally strengthened with CFRP sheets develop higher load capacities and shift debonding when CFRP mechanical anchors are used. In this study, an extensive review on U-wrap anchorage was conducted in which Table 2-2 was developed to summarize the anchorage configuration used to assist the RC sections in utilizing the flexural strengthening material in several studies. It can be observed that a pattern is apparent in which the implementation of end and end-intermediate U-wrap configurations has a higher probability of failing immediately as opposed to using full spanned U-wrap configurations. The benefits of fully wrapping RC beams are: (a) eliminating ED and IC failure modes [60]; (b) reinforcing against shear due to the individual fiber orientations that are lined normal to most of the shear cracks [25]; (c) contributing to the flexural behavior [54].

Table 2-2: Past investigations of using different anchorage configurations in retrofit applications.

Reference U-wrap Configuration Strengthening Material Failure ModesEnd End and Full CFRP CFRP AA Sudden1 Typical2plate Intermediate wrapped Plates Sheets Plate								
[56]	X	X	X					
[25]	X	X	X	X	X			
[60]	X	X	X					
[10]	X	X	X	X	X			
[61]	X	X	X					
[62]	X	X	X					
[54]	X	X	X					

1 ED; IC; CS
2 CC; SY; PR

2.2.2. Bolts and mechanical fasteners. Several research projects were conducted to study techniques in manipulating the physical structure of FRP sheets and form bolt-like anchors called Splay Anchors [63–65]. Kalfat et al. [64] conducted an extensive review on the different anchorage techniques reported to date, where it was reported that the incorporation of FRP anchors combined with U-wrap configurations enhanced the performance of the U-wrap. However, the failure modes were shifted from End-debonding/Intermediate-crack-debonding to FRP pullout. Moreover, Eshwar et al. [66] studied the effect of a beam’s soffit curvature on the performance of FRP anchors in flexural strengthening applications. The experimental program consisted of ten RC beams in which three of the beams consisted of flat soffits; one was left un-strengthened, one was strengthened using a three-ply CFRP wet layup laminate and one was strengthened with one CFRP precured laminate. Six beams were constructed with varying soffit-curvatures that were strengthened by bonding CFRP laminates using wet layup and precured methods, respectively, and the last specimen was strengthened with CFRP laminates and anchored with GFRP splay anchors. The results indicated that the incorporation of FRP anchors enhanced the flexural capacity and ductility of the strengthened beam as opposed to the specimens that were strengthened using the wet layup and precured method, respectively. However, it was observed that the failure mode was still sudden since it consisted of the pealing of the CFRP laminates couple with the pull-out of the FRP anchors as shown in Figure 2-5. Other researchers have studied the effects of implementing FRP anchors in structural engineering applications [21, 67, 68].

Figure 2-5: Failure mode of splay anchor in [66].

Furthermore, researchers also investigated the implementation of bolts as a form of anchorage in flexural strengthening applications. For example, Ebead and Marzouk [69] conducted an experimental investigation to study the behavior of a two-way slab strengthened on the soffit of the slab around the column. The program was directed at testing the effectiveness of two configurations of steel plates and four different arrangements of steel bolts. As a result, the slabs showed an increase in stiffness and energy absorption. Also, the load-carrying capacity of the strengthened slabs was increased by 56.55, 57.76, and 64.56% over that of the control specimen. Consequently, another study was conducted by Oehlers [70] to compare the performance of two anchorage systems - adhesive bonded and bolted plates for strengthening of RC beams. The adhesive bond anchorage has a higher stiffness increase than that of the bolted anchorage, but the failure mode for adhesively bonded plates is brittle as opposed to bolted plates, which have a more ductile failure. El-Maaddawy [71] investigated the effectiveness of different mechanically fastened composite systems for retrofitting relatively large-scale corrosion-damaged reinforced concrete beams. The experimental program comprised of beams that were retrofitted with composite plates secured with powder-actuated fasteners (PAF), expansion anchor bolts (EAB), and threaded anchor bolts (TAB). The results indicated that there was a small increase in strength for plated RC beams using PAF, but a larger increase in strength for plated RC beams with both EAB and TAB. In addition, Gao et al. [18] also tested the flexural enhancement of bolt-

ing steel plate to RC beams, where it was concluded that the bolted steel plate specimens were capable of preventing ED/IC failure modes.

2.3. Finite Element Simulations

Other numerical studies were conducted to simulate retrofit applications using finite element (FE) software packages [72–74]. These FE simulations are dependent on the constitutive laws to simulate the nonlinear properties of concrete, FRP and steel. As a result, several of these researchers have reported accurate predictions of the load versus deflection curves when compared to that of the experiment [3,74]. Hawileh et al. [74] conducted a numerical investigation to simulate an RC T-beam shear-strengthened using side-bonded CFRP sheets. This model was subjected to cyclic loading where the load versus displacement hysteresis loops were generated and compared against the experiment.

Chapter 3. Experimental Program

This project aims at investigating the effect of bolting and/or bonding AA plates on the flexural strength of flexurally deficient RC beams. The anchorage techniques and strengthening materials described herein are required to assist the strengthened specimen's in exhibiting ductile and controlled failure modes throughout its loading life.

This chapter presents an in-depth review on the details of the RC specimens' geometry, mechanical properties of the materials used, the techniques followed prior and during strengthening applications, and the loading scheme followed during the tests.

3.1. RC Beam Details

A total of 16 RC beams were designed with high shear capacity and low flexural capacity—forcing the beams to fail in flexure. This design approach followed the ACI 318-14 [75] design provisions. The dimensions of each specimen is 1840 mm × 240 mm × 125 mm, the longitudinal reinforcing bar diameters used are 10 mm and 8 mm, and the transverse reinforcement are 8 mm in diameter and they are spaced at 100 mm center-to-center as shown in Figure 3-6.

3.1.1. Geometry and design.

The experimental program was divided into three groups: C-Specimens, M10-Specimens, and M12-Specimens. The C-Specimens, shown in Figure 3-7, consist of two RC beams where one beam was un-strengthened and used as a control beam(CB), and the other specimen was strengthened by means of externally bonding an AA plate to its soffit and used as a control beam strengthened an epoxy-bonded AA plate (CBE). Since all beams are flexurally deficient, the retrofit applications that are conducted during this study are located at the soffits of the beams.

Moreover, the M10-Specimens, shown in Figure 3-8, consist of seven RC beams in which two beams are strengthened by mechanically fastening AA plates using a large number of M10 bolts [39] (BM10H), two beams with AA plates that are fastened using a low number of M10 bolts, two beams with AA plates that are bonded and bolted using epoxy and a high number of M10 bolts simultaneously (BEM10H). Finally, the M12-Specimens, shown in Figure 3-9, consist of the same number of beams and retrofit schemes; however, the mechanical fasteners used are M12 bolts (BM12H, BM12L, and

BEM12H). A summary of the test matrix is shown in Table 3-3. Further description related to the material’s mechanical properties will be discussed in the following sections.

✁ ✂ ✂ ✄ ɸ☎ 8✆ ✝m✝ ✞ m✟ @✠ ✡10☛ 0m☞ m☞ ✂ ✂ ✌ ✍ ✌
✎ ✁ ✂ ✂ ✏ 2✑ɸ✟8m✒ m✓ ✆ ✔ ✕ ✑ ✝ ✖ ✓ ✂ ✓ ✔ ☎
a
240 mm 240 mm
a 1840 mm 125 mm
2ɸ10mm Section a-a
✎ ☛ ☞ ✂ ✂ ✗ ✑ ☎ ☎ ✑ ✂ ✒ ✓ ✆ ✔ ✕ ✑ ✝ ✖ ✓ ✂ ✓ ✔ ☎

Figure 3-6: Beam geometry

a
240 mm 240 mm
a 1840 mm 125 mm
Section a-a

(a) CB

b 1840 mm 125 mm
240 mm 240 mm
b 3 mm 50 mm
1350 mm
AA Plate Section b-bEpoxy

(b) CBE

Figure 3-7: Geometry and details of CB-Specimens.

Table 3-3: Test matrix of study

Designation	AA Plates	Anchorage	Number of Bolts		Position		Number of Beams
	Plate Thickness: 3 mm	Epoxy	High	Low	Series	Edge	
CB	-	-	-	-	-	-	1
CBE	X	X	-	-	-	-	1
BEM10H	X	X	X	-	X	-	2
BEM10L	X	X	-	X	X	-	2
BEM12H	X	X	X	-	X	-	2
BEM12L	X	X	-	X	X	-	2
BM10H	X	-	X	-	X	-	2

BM12H X - X - X - 2
BEM10E X X X - - X 1
BEM12E X X X - - X 1
32
1840 mm
a 1350 mm 125 mm
240 mm 240 mm
a 3 mm 50 mm
Section a-a
AA Plate HST3 M10 100 mm Epoxy
(a) BEM10H
1840 mm
b 1350 mm 125 mm
240 mm 240 mm
b 200 mm 3 mm 50 mm
Section b-b
AA Plate HST3 M10 Epoxy
(b) BEM10L
1840 mm
c 1350 mm 125 mm
240 mm 240 mm
c 3 mm 50 mm
100 mm Section c-c
AA Plate HST3 M10
(c) BM10H
1840 mm d
1350 mm 125 mm
240 mm 240 mm
100 mm d 3 mm 50 mm
Section d-d
AA Plate HST3 M10 Epoxy
(d) BEM10E
Figure 3-8: Geometry and details of M10-Specimens.

3.1.2. Specimen instrumentation. The Strain Gages were installed such that the moment-induced compression/tension strains would be measured and recorded during the tests. In this study, all strain gages were placed at the mid-span of each specimen; whereby one strain gage was bonded to the top concrete fiber, one strain gage was bonded to the AA plate, and two strain gages were bonded to the bottom steel bars.
Figure 3-10 shows the Strain Gage locations in all the specimens tested in this project.
33
3.1.3. RC specimen preparation. The steel cages were prepared, as shown in Figure 3-11a, whereby the strain gages were bonded to the steel bars as shown in Figure 3-11b. The formwork was prepared for each specimen such that the poured concrete would occupy the designed dimensions as shown in Figure 3-11c and Figure 3-11d.

1840 mm
a 1350 mm 125 mm
240 mm 240 mm
a 3 mm 50 mm
AA Plate HST3 M12 100 mm
Section a-a
Epoxy
(a) BEM12H
1840 mm
b 1350 mm 125 mm
240 mm 240 mm
b 200 mm 3 mm 50 mm
Section b-b
AA Plate HST3 M12 Epoxy
(b) BEM12L
1840 mm
c 1350 mm 125 mm
240 mm 240 mm
c 3 mm 50 mm
100 mm
Section c-c
AA Plate HST3 M12
(c) BM12H
1840 mm d
1350 mm 125 mm
240 mm 240 mm
100 mm d 3 mm 50 mm
Section d-d
AA Plate HST3 M12
Epoxy
(d) BEM12E

Figure 3-9: Geometry and details of M12-Specimens.

Concrete Strain Gage
240 mm
3 mm
1350 mm
1840 mm
2 Steel Strain Gages AA Plate Strain Gage

Figure 3-10: Strain gage locations.

(a) Steel cage (b) Location of strain gages
(c) Steel cage within formwork (d) Finalized Specimens

Figure 3-11: RC beam specimen preparation.

3.2. Material Specification

In this section, the mechanical properties of the material were obtained by conducting both compressive and tensile tests; depending on the type of material being investigated. Concrete cubes, steel coupons, and AA dog-bone shaped plates were pre-

pared in the labs and tested according to different ASTM standards [76–78]. Further details regarding the tests and application will be discussed in the following sections.

3.2.1. Concrete compression test. The 16 specimens were cast using normal weight concrete and were designed to achieve a compressive strength equal to 40 MPa at 28 days. Since the beams were prepared in four batches, four concrete cubes were obtained from the manufacturer to conduct compressive tests as per ASTM C109/C109m standard [76]. The dimensions of each cube were 150 mm × 150 mm × 150 mm. Moreover, the cubes were crushed on the same days as the beams were tested, which provides insight on the exact mechanical properties for each RC beam during the test. The compressive strength values were used in several classical models that were, then, employed in nonlinear FE software—to capture the nonlinear behavior of concrete. Table 3-4 presents the compressive strength of each batch of concrete, and Figure 3-18 shows the concrete cubes during preparation and after testing.

Table 3-4: Compressive strength of concrete batches.

Batch Number	1	2	3	4
Cube Compressive Strength (MPa)	45	45	48	47
Cylinder Compressive Strength (MPa)	36	36	38.4	37.6

3.2.2. Tensile testing of normal strength reinforceming bars. The steel bars used to reinforce the beams are 8 mm and 10 mm in diameter. Their mechanical properties were obtained by conducting tensile tests, for both bar diameters, according to the ASTM 370-18a [77]. Instron Universal Testing was used to perform the tensile test, where the loading rate for all coupon tests was 1 mm/min [77]. Afterwards, the offset method was used, as shown in Figure 3-13, where line OM represents the user specified offset (default value of 0.2%) and line mn should be parallel to line OA. The intersection at point r was taken as the yield stress of the specimen.

3.2.2.1. Mechanical properties of φ8 mm bar. Three φ8 mm bars were prepared for tensile testing where the total length, grip, and diameter of each bar is 300 mm, 50 mm, and 8 mm, respectively, as shown in Figure 3-14. The test setup consisted of 50 mm grip length from both edges of the steel bar and a displacement controlled

(a) Un-crushed concrete cubes

(b) Crushed concrete cubes

Figure 3-12: Concrete cube tests and preparations.

Figure 3-13: Offset Method [77].

axial load was applied, with a rate of 1 mm/min [77], as shown in Figure 3-15a. The steel bars were tested to rupture as shown in Figure 3-15b, and the stress and strain curves were plotted, as shown in Figure 3-16. The offset method was implemented on

the stress-strain curve, in Figure 3-16, to calculate the yield strength (Fy) and Young's modulus of elasticity (E). Finally, these parameters were combined with the elongation and maximum tensile strength (Fu) values and tabulated in Table 3-5.

Grip Length 50 mm Grip Length 50 mm
8 mm
300 mm

Figure 3-14: Dimensions of a steel reinforcing φ8 mm bar in a tensile test.

(a) Test setup (b) Ruptured specimens

Figure 3-15: Steel tensile test φ8 mm.

700
600
500
400
300 φ8 mm(1)
φ8 mm(2)
φ8 mm(3)
200
100
0
0 0.02 0.04 0.06 0.08 0.1 0.12 0.14
Strain (mm/mm)

Figure 3-16: Stress versus strain curve for each φ8 mm bar.

Tensile Stress (MPa)

Table 3-5: Mechanical properties of each φ8 mm bar.

Specimen ID	E (GPa)	Fy (MPa)	Fu (MPa)	Elongation (%)
φ8(1)	199.9	554.7	641.4	12.5
φ8(2)	200	540	627.4	11.4
φ8(3)	199.8	574.9	651.6	11.4
Average	199.9	556.5	640.1	11.8

3.2.2.2. Mechanical properties of φ10 mm bar. Similarly, three φ10 mm bars were prepared for tensile testing where the total length, grip, and diameter of each bar is 300 mm, 50 mm, and 10 mm, respectively, as shown in Figure 3-17. The test setup consisted of 50 mm grip length from both edges of the steel bar and a displacement controlled axial load was applied, with a rate of 1 mm/min [77], as shown in Figure 3-18a. The steel bars were tested to rupture as shown in Figure 3-18b, and the stress and strain curves were plotted, as shown in Figure 3-19. The offset method was implemented on the stress-strain curve, in , to calculate the yield strength (Fy) and Young's modulus of elasticity (E). Finally, these parameters were combined with the elongation and maximum tensile strength (Fu) values and tabulated in Table 3-6.

Grip Length 50 mm Grip Length 50 mm
10 mm
300 mm

Figure 3-17: Dimensions of a steel reinforcing φ10 mm bar in a tensile test.

(a) Test setup (b) Ruptured specimens

Figure 3-18: Steel tensile test for φ10 mm.

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700
600
500
400
300
φ10 mm(1)
φ 10mm(2)
200 φ 10mm(3)
100
0
0 0.02 0.04 0.06 0.08 0.1 0.12 0.14
Strain (mm/mm)

Figure 3-19: Stress versus strain curve for each φ10 mm bar.

Table 3-6: Mechanical properties of each φ10 mm bar.

Specimen ID	E (GPa)	Fy (MPa)	Fu (MPa)	Elongation (%)
φ10(1)	200.1	542.0	634.5	13.4
φ10(2)	199.8	559.0	658.2	13.0
φ10(3)	200.2	545.7	642.7	12.3
Average	200.0	548.9	645.1	12.9

3.2.3. Tensile testing for AA plates. The AA plates used during this study are from the AA5083-H111 family. Tensile tests using the Instron universal testing machine (UTM) were conducted. Furthermore, the dog-bone shaped specimens were prepared, as per ASTM E8 [78]. The total length, gage length and grip length are 385 mm, 225 mm and 75 mm, respectively, as shown in Figure 3-20. The test setup was carried out by gripping 50 mm from both edges of each AA coupon as shown in Figure 3-21a. The UTM was programmed to subject each coupon to a displacement rate of 0.5 mm/min [78]. As a result, the AA specimens ruptured in the gage length as shown in Figure 3-21b. The stress and strain values were plotted against each other and are shown in Figure 3-22. Afterwards, the yield strength, ultimate tensile strength, modulus of elasticity, and elongation were extracted as shown in Table 3-7.

3.2.4. Mechanical fasteners. The mechanical fasteners used to anchor the AA

plates were provided by the manufacturer, Hilti [39]. The anchor models used herein
40
Tensile Stress (MPa)
50 mm
225 mm 75 mm
Figure 3-20: Dimensions of a dog-bone shaped AA plate [78].
(a) Test setup (b) Ruptured samples
Figure 3-21: AA plate tensile test.
350
300
250
200
AA1
AA2
150 AA3
AA4
AA5
100
50
0
0 0.02 0.04 0.06 0.08 0.1 0.12 0.14 0.16 0.18
Strain (mm/mm)
Figure 3-22: Stress versus strain curve for each AA coupon.
were: HST3 M10×90 and HST3 M12×105. Their mechanical properties and geometric details were obtained from Hilti’s HST3 technical datasheet [79] and are presented in Table 3-8, where futa is the ultimate tensile strength, Ase,V is the gross area of the sleeves, do is the diameter of the bolts, heff is the depth of the bolts, Smin is the min-
41
Tensile Stress (MPa)
Table 3-7: Mechanical properties of each AA coupon.

Specimen ID	E (GPa)	Fy (MPa)	Fu (MPa)	Elongation (%)
AA1	65936	161.4	329.2	16.6
AA2	64103	152.2	319.1	17.6
AA3	65396	151.7	312.1	16.5
AA4	63690	148	313.5	17.0
AA5	71036	145	309.1	18.0
Average	66032.2	151.6	316.6	17.1
Manufacturer	-	163.9	301.5	21.05

imum center-to-center spacing between the bolts based on bolt shear strength, pry-out and pull-out action, Cmin is the minimum distance from the bolt to the edge of the concrete, and T is the torque required to fix the bolts. For brevity, a simple diagram is presented to clarify the details including the geometric limitations imposed by [79], as shown in Figure 3-23. The expansion anchor models, used in this project, consisted of sleeves in the bottom such that once they are settled within the concrete element, any normal force that is aimed to jack the bolt out is resisted due to the expansion of its sleeves. Figure 3-24 shows both models used in this project, where each bolt consisted of a knut, a washer, and a sleeve. The knut was responsible for subjecting the bolt into a pretension loading configuration such that with every turn of the knut, load is transferred to the washer which then bears on the object required to be fixed.

Table 3-8: Expansion anchor properties [79].

Anchor Type	HST3 M10×90	HST3 M12×105
f_{uta} (MPa)	800	800
$A_{se,V}$ (mm^2)	58	84.3
d_o (mm)	10	12
h_{eff} (mm)	90	105
S_{min} (mm)	70	80
C_{min} (mm)	70	80
T (N-m)	45	60

Smin
Cmin
Cmin

Figure 3-23: Diagram depicting HST3 expansion anchors' geometric limitations in concrete.

(a) HST3 M10×90 (b) HST3 M12×105

Figure 3-24: Hilti expansion anchor models.

3.2.5. Epoxy. Epoxy resin is a popular adhesive that is used to anchor different retrofit systems and provide them with structural homogeneity throughout their loading life [40]. In this project, a two-component thixotropic epoxy adhesive was provided by the manufacturer, MAPEI [40], in which the adhesive consisted of two parts; mainly, the Adesilex PG2 SP (part A) and the hardener (part B), shown in Figure 3-25. Generally, these two parts are mixed together according to a specific ratio imposed by the technical datasheet [80]. In this project, the ratio of part A-to-part B was 3:1 where the compound's mechanical properties, provided by the manufacturer [40], are 80 MPa, 40 MPa, 8000 MPa, 4000 MPa and 30 MPa for Compressive strength (ASTM D-695), flexural strength (ISO-178), Modulus of elasticity under compression (ASTM D-695), Modulus of elasticity in flexural (ISO- 178), and tensile strength (ASTM D-638), respectively [80]. Mainly, the flexural modulus of elasticity will be employed within the

numerical software explained in the following chapters.
(a) Part A (b) Part B
Figure 3-25: Two-component epoxy.

3.3. External Strengthening
The content described herein describes the steps that were taken during the strengthening application. In this project, two anchorage tools were used; Adesilex PG2 SP epoxy and HST3 M10/M12 expansion anchors. Moreover, the strengthening material anchored were AA5083-H111 plates.

3.3.1. Surface preparation. Prior to external strengthening, the surfaces of the concrete specimens and the AA plates were roughened using an electrical grinder. This enhanced the bond behavior between the epoxy and the retrofit system; yielding an ideal surface profile that helps ensure the design load transfer [38]. The following sections will briefly explain the different types of surface preparations conducted during this project.

3.3.1.1. Surface preparation for AA plates. Achieving a rough surface, when conducting retrofit applications, is important for maintaining a homogeneous anchorage system and a strong bond between the strengthening material and the RC host. In this project, the plates were abraded using an electrical grinder similar to past studies [10, 11]. This was performed on one side of the AA plate; the side facing the concrete specimen as shown Figure 3-26. According to the test matrix shown in Table 3-3, a total of 15 plates were grinded for strengthening applications.
Default surface Grinded surface
Figure 3-26: Default and smoothened surface of AA plates.

3.3.1.2. Surface preparation for concrete specimens. Similarly, the concrete specimens were grinded in order to ensure ideal design load transfer as specified in the ACI 440.2R-08 [38]. Since the RC members will be subjected to flexural loading, the concrete surface preparation was classified as a bond-critical application. The bond-critical application should follow the abrasion steps found in ACI-546 [81], where thin layers of concrete were abraded, using electrical grinders, from the soffit of each RC specimen. Figure 3-27a and Figure 3-32b show the default and grinded surfaces of the beams, respectively, and the electrical grinder is shown in Figure 3-32e.

(a) Default surface (b) Grinded surface (c) Electrical grinder
Figure 3-27: Surface preparation of RC beams.
3.3.2. Mechanical fastening strategy. Before fixing the AA plates to the RC
specimens, the spacing and diameter of each hole was marked on the concrete specimen
and AA plate according to the dimensions shown in both Figure 3-8 and Figure 3-9.
Each AA plate was drilled with holes spaced at different lengths as shown in Figure 3-
28 where the diameter of each hole was dependent on the diameter of the anchors;
mainly, to avoid uneven fixtures and achieve ideal load transfer within the anchorage
system [79]. Therefore, an HST3 M10 anchor required a 12 mm diameter hole and an
HST3 M12 required a 14 mm hole as shown in Figure 3-28. Similarly, the concrete
specimens were marked with the same spacing and diameters as shown in Figure 3-29
in which the embedment depths used were 65 mm and 80 mm for the HST3 M10 and
HST3 M12 bolts, respectively, as shown in Figure 3-30.

75 mm 100 mm D12/D14 mm
50 mm
1350 mm
(a) BEM10H/BEM12H and BM10H/BM12H specimens
200 mm D12/D14 mm75 mm
50 mm
1350 mm
(b) BEM10L/BEM12L specimens
100 mm
75 mm D12/D14 mm
50 mm
1350 mm
(c) BEM10E/BEM12E specimens
Figure 3-28: Bolt location markings on AA plates.
RC Beam Soffit
1840 mm
320 mm 100 mm
125 mm
1350 mm
D12/D14 mm
AA Plate Location
(a) BEM10H/BEM12H and BM10H/BM12H specimens
RC Beam Soffit
1840 mm
320 mm 200 mm
125 mm
1350 mm
D12/D14 mm
AA Plate Location
(b) BEM10L/BEM12L specimens

RC Beam Soffit
1840 mm
320 mm 100 mm
125 mm
1350 mm
AA Plate Location D12/D14 mm
(c) BEM10E/BEM12E specimens
Figure 3-29: Bolt location markings on concrete specimens.
3.3.3. Setting instructions for HST3 bolts. Hilti’s technical datasheet [79]
offers instructions on how to mechanically fasten HST3 expansion anchors. Figure 3-
31 shows these steps during the strengthening application.

Side View of RC Specimen
240 mm
65 mm
AA Plate HST3 M10 Bolt
(a) HST3 M10
Side View of RC Specimen
240 mm
80 mm
AA Plate HST3 M12 Bolt
(b) HST3 M12
Figure 3-30: Bolt embedment depth for each model.
Setting instructions
*For detailed information on installation see instruction for use given with the package of the product
Setting instruction for HST3, HST3-BW, HST3-R, HST3-R-BW
Hammer drilling (M8, M10, M12, M16, M20, M24)
1. Drill the hole 2. Clean the hole
3a. Insert the anchor with hammer 3a. Insert the anchor with setting tool HS-SC
4. Check 5a. Torque with calibrated torque wrench (M8-M24)
5b. Torque with impact wrench with Adaptative torque module (M8-M12)
Figure 3-31: Setting instruction for HST3 expansion anchors [79].
3.3.4. Final specimens. The aforementioned preparation techniques were in-
corporated on the RC beams whereby 15 specimens were strengthened using differ-
Hollow Drill Bit (M16, M20, M24), no cleaning required
ent ancho1.r agDreill tshye shtoelem wisth. Tthhe Heoslelows dpreillc biitmens wer2ea. cInasetert gthoe rainzcehodr wiintht oham3mgerr oups; namely, CB-
Specimens, M10-Specimens, and M12-Specimens.The tested parameters were depth

2b. Insert the anchor with setting tool HS-SC 3. Check
9 Updated: Apr-18
and spacing of bolts with the presence or absence of adhesive resin (epoxy) as shown in
Figure 3-32. For brevity, a picture of each specimen, depending on its anchorage sys-

tem, is shown. For example, Figure 3-32a shows BEM10E specimen, which consists
of the same bolt arrangement as the BEM12E, but with HST3 M12 anchors. Ultimately, these specimens were subjected to a displacement controlled test in which a displacement rate of 2mm/min was employed using a four-point bending configuration, as shown in Figure 3-33, using the Universal Test Machine (UTM).

(a) BEM10E/BEM12E (b) BEM10H/BEM12H (c) BEM10L/BEM12L
(d) BM10H/BM12H (e) CBE
Figure 3-32: Final strengthened specimens.

Load
Loading plates
a = 600 mm a = 600 mm
240 mm
1740 mm
1840 mm
Support Plates
Figure 3-33: Loading scheme of specimens.

Chapter 4. Results and Discussions

The work presented in this chapter aims at studying the structural and mechanical behavior of all the specimens tested in this project. Several curves like the load versus deflection and load versus strain curves were generated to graphically simulate each specimen's stiffness, ductility, and capacity during testing. Furthermore, the failure mode of each specimen was captured and presented by means of a photograph in which each photograph was supported by monitoring strain measurements during each experiment.

4.1. Response and Behaviour of Each Specimen

The load versus deflection and load versus strain curves were highlighted during this section, whereby an in-depth exploration of the specimens' structural and mechanical responses was graphically captured. For brevity, the strain gauge of each element was abbreviated as SG followed by the element of interest. For instance, the strain gauge of concrete was abbreviated as SG Concrete. Moreover, the parameters reported herein were: load at yielding of bottom reinforcement ($P_{steel,y}$), load at yielding of AA plate ($P_{AA,y}$), load at crushing of concrete ($P_{conc,cr}$), ultimate load of each specimen (P_{ult}), load at failure of section (P_{fail}), deflection at yielding of bottom reinforcement

($\delta_{steel,y}$), deflection at yielding of AA plate ($\delta_{AA,y}$), deflection at crushing of concrete ($\delta_{conc,cr}$), deflection corresponding to ultimate load (δ_{ult}), and deflection at failure of section (δ_{fail}).

Owing to the vast amount of points obtained from both the strain gauge and UTM loading machine, the strain measurements of each element corresponding to the ultimate load were drawn adjacent to the section of interest. This granted insight into the mechanical contribution of each element to the flexural capacity of the specimen. In general, structural engineers would try to achieve crushing of concrete with the yielding of steel to fully utilize the mechanical properties of the reinforcement. In this project, the yield strains of both the AA plates and the steel reinforcement were taken from the average experimental results shown in Table 3-5, Table 3-6, and Table 3-7. Therefore, the yield strains of the AA plates (□ya) and steel reinforcement (□ys) were taken as 0.0034 and 0.0027, respectively, while the strain at which concrete crushes (□cc) was

taken as -0.003. The signs in the strain values correspond to the type of stress whereby a positive strain indicates tension and negative strain indicates compression.

4.1.1. CB-Specimens. As previously mentioned, this project consisted of two reference RC specimens, mainly: CB and CBE. CB is the unstrengthened RC beam and CBE is the RC beam externally bonded with an AA plate. Both of these beams were tested under a four-point bending configuration in which a displacement rate of 2mm/min was subjected until both concrete sections reached failure. It is worth mentioning that failure, in the scope of this project, is defined as a 10-15% drop in the ultimate load demonstrated in the load versus deflection curves.

4.1.1.1. CB. The unstrengthened RC specimen (CB) failed in flexure in which both steel yielding and concrete crushing occurred during the test. Figure 4-34 shows the load versus deflection and load versus strain curves whereby the UTM machine and strain gauges were able to capture an extensive amount of points during the testing phase. As a result, the ultimate loading capacity of CB achieved a value of 64.2 kN at 17.4 mm whereby the test was stopped when a 10% drop in the ultimate load was noticed, 57.7 kN at 30.51 mm. Using both curves in Figure 4-34a and Figure 4-34b, the loads at both yielding of steel and crushing of concrete were recorded as 53.2 kN at 6.77 mm and 63.6 kN at 17.8 mm, respectively. Figure 4-34c shows the strain measurements corresponding to the ultimate load where the strain in steel reached a value of 0.0136 and the strain in concrete reach -0.0023. It is worth mentioning that the strain in concrete should, theoretically, reach a value -0.003 to visibly exhibit crushing. However, the strain gauge was damaged due to spalling of concrete.

4.1.1.2. CBE. The RC specimen epoxy-bonded with an AA plate (CBE) failed by cover separation in which the epoxy-bonded AA plate ripped some of the concrete cover, exposing the steel reinforcement. It was observed, from the strain measurements, that yielding occurred in both steel reinforcement and AA plate, and crushing occurred in the top concrete fibers of the beam during the test. Figure 4-35 shows the load versus deflection and load versus strain curves whereby the UTM machine and strain gauges were able to capture an extensive amount of points during the testing phase. As a result,

80 80
60 60
40 40
SG Concrete
SG Steel
20 20
0 0
0 5 10 15 20 25 30 35 -4 -2 0 2 4 6 8 10 12 14 16
Deflection (mm) Strain (mm/mm) -310
(a) Load versus deflection curve (b) Load versus strain curve
ε_{cc} = -0.0023
205 mm
ε_{ys}= +0.0136
125 mm
(c) Strain in the section

Figure 4-34: Load, deflection and strain for CB at mid-span.

CBE achieved a peak load value of 84.4 kN at 14.3 mm followed by a large drop in loading capacity at 19.94 mm, due to cover separation. Using both curves in Figure 4-35a and Figure 4-35b, the loads at which yielding occurred in both the steel reinforcement and the AA Plate were 65.1 kN at 6.62 mm and 72.2 kN at 7.56 mm, respectively, whereas the load at the crushing of concrete was 83.9 kN at 14.1 mm.Figure 4-35c shows the strain measurements corresponding to the ultimate load where the strain values in both the steel reinforcement and the AA plate reached 0.00431 and 0.00495, respectively, and the strain in concrete reach -0.00301. The close proximity of the strain values in both the steel and the plate indicate that the test immediately failed, approximately, around the ultimate loading capacity of the section.

4.1.2. M10-Specimens. Seven RC specimens externally strengthened with AA plates by means of bolting, using HST3 M10 expansion anchors, with/without adhesive bonding were loaded until failure. Each specimen included a replica of itself except

52

Load (kN)
Load (kN)
100 100
80 80
60 60
40 40
SG Concrete
20 20 SG AA Plate
SG Steel
0 0
0 5 10 15 20 -4 -2 0 2 4 6
Deflection (mm) Strain (mm/mm) -310
(a) Load versus deflection curve (b) Load versus strain curve
125 mm εcc = -0.00301
205 mm
εys= +0.00431
50 mm εya= +0.00495
(c) Strain distribution in section

Figure 4-35: Load, deflection and strain for CBE at mid-span.

for specimen BEM10E, which granted the author insight into the consistency of the results corresponding to each strengthened specimen’s unique anchorage system. Furthermore, these beams were tested under a four-point bending configuration in which a displacement rate of 2mm/min was subjected until failure by means of plate rupture or IC debonding occurred.

4.1.2.1. BEM10L. The first two RC specimens that were bolted at 200 mm, center-to-center spacing, and bonded with AA plates (BEM10L-1 and BEM10L-2) failed by rupture of the plates. Their sections’ ultimate loading capacities were achieved as a result of the yielding in both steel reinforcement and AA plates, and crushing in the top concrete fibers. Figure 4-36 shows the load versus deflection and load versus strain curves, whereby the UTM machine and strain gauges were able to capture an extensive amount of points during the testing phase. It is worth noting that the load versus strain curve for BEM10L-1 was unable to record the strain measurements within the steel

53

Load (kN)

Load (kN)

reinforcement due to the damage of the strain gauge during testing. As a result, the ultimate loading capacities of BEM10L-1 and BEM10L-2 were 80 kN at 16 mm and 85.8 kN and 17.1 mm, respectively. The loading, then, experienced a slight but negligible drop where it plateaued until both specimens, BEM10L-1 and BEM10L-2, reached failure loads of 76.2 kN at 25 mm and 78 kN at 24.4 mm, respectively. Afterwards, both specimens experienced a large drop in their ultimate loading capacities due to rupture of their AA plates. Using the curves in Figure 4-36a and Figure 4-36b, the mechanical behavior of BEM10L-1 was studied in which the loads at yielding of AA plate and crushing of concrete were 69.6 kN at 7.6 mm and 79.8 kN at 14.8 mm, respectively. However, BEM10L-2 exhibited yielding in both its steel reinforcement and the its AA plate at 67.5 kN at 7.06 mm and 80.1 kN at 10.4 mm, respectively, and experienced concrete crushing at 85 kN at 17.5 mm as shown in Figure 4-36a and Figure 4-36c.

Figure 4-36d shows the strain measurements corresponding to the ultimate load where the strain values in both the steel reinforcement and the AA plate reached 0.00413 and 0.00803, respectively, and the strain in concrete reach -0.00301. A diagram pertaining the strain distribution of BEM10L-1's section was not drawn due to the loss of strain gauge data its steel reinforcement.

4.1.2.2. BEM10H. Similarly, the two plated RC specimens that were anchored with the same bolt model, but spaced at 100 mm and bonded using the same adhesive (BEM10H-1 and BEM10H-2) also failed by rupture of the plates. Their sections' ultimate loading capacities were achieved as a result of the yielding in both steel reinforcement and AA plates, and crushing occurred in the top concrete fibers of both beams during the test. Figure 4-37 shows the load versus deflection and load versus strain curves, whereby the UTM machine and strain gauges were able to capture an extensive amount of points during the testing phase. As a result, the ultimate loading capacities of BEM10H-1 and BEM10H-2 were 80.5 kN at 15.6 mm and 86.5 kN and 18.5 mm, respectively. The loading, then, experienced a slight but negligible drop where it plateaued until both strengthened specimens, BEM10H-1 and BEM10H-2, reached failure loads of 76.5 kN at 29.4 mm and 80 kN at 26.9 mm, respectively. Afterwards, both specimens experienced a large drop in their ultimate loading capacities

54

100 100
80 80
60 60
BEM10L-1
40 BEM10L-2 40 SG Concrete
SG AA Plate
20 20
0 0
0 10 20 30 -4 -2 0 2 4 6 8
Deflection (mm) Strain (mm/mm) -3×10

(a) Load versus deflection curve (b) Load versus strain curve for BEM10L-1

100
80
60
SG Concrete
40 SG AA Plate
SG Steel 125 mm $\varepsilon cc = -0.00298$
20
205 mm
0 $\varepsilon ys= +0.00413$
-4 -2 0 2 4 6 8
Strain (mm/mm) -3 $\varepsilon\times10$ 50 mm $ya= +0.00803$

(c) Load versus strain curve for BEM10L-2 (d) Strain in Section BEM10L-2

Figure 4-36: Load, deflection and strain for BEM10L specimens at mid-span.

due to rupture of their AA plates. Using the curves in Figure 4-37a and Figure 4-37b, the mechanical behavior of BEM10H-1 was studied in which the loads at yielding of both the steel reinforcement and the AA plate were 72.8 kN at 8.23 mm and 80.3 kN at 16.5 mm, respectively, and the load at crushing of concrete was 80.32 kN at 16.2 mm. However, BEM10H-2 exhibited yielding in both its steel reinforcement and the its AA plate at 67.5 kN at 7.1 mm and 80.1 kN at 10.4 mm, respectively, and experienced concrete crushing at 84.7 kN at 19 mm as shown in Figure 4-37a and Figure 4-37c.

Figure 4-37d shows the strain distribution in BEM10H-1 where the strain values corre-
sponding to the ultimate load in both the steel reinforcement and the AA plate reached
0.00598 and 0.00623, respectively, and the strain in concrete reach -0.00403. This indi-
cated that yielding occurred in both the steel reinforcement and the plate, and crushing
occurred in concrete. Similarly, yielding in both internal and external reinforcements

Load (kN) Load (kN)
Load (kN)

occurred whereby their strain values corresponding to the ultimate load were 0.00501
and 0.00695, respectively, and concrete crushing occurred at a strain of -0.00358.

100 100
80 80
60 BEM10H-1 60
BEM10H-2
40 40 SG Concrete
SG AA Plate
SG Steel
20 20
0 0
0 10 20 30 -4 -2 0 2 4 6 8
Deflection (mm) Strain (mm/mm) -3×10
(a) Load versus deflection curve (b) Load versus strain curves for BEM10H-1
100
80
60
SG Concrete
40 SG AA Plate
SG Steel 125 mm εcc = -0.00403
20
205 mm
0
-4 -2 0 2 4 6 8 εys= +0.00598
Strain (mm/mm) -3×10 50 mm εya= +0.00623
(c) Load versus strain curves for BEM10H-2 (d) Strain in section BEM10H-1
125 mm εcc = -0.00358
205 mm
εys= +0.00501
50 mm εya= +0.00695
(e) Strain in section BEM10H-2

Figure 4-37: Load, deflection and strain for BEM10H specimens at mid-span.

4.1.2.3. BM10H. The two RC specimens that were only bolted at a spacing of 100 mm center-to-center (BM10H-1 and BM10H-2) also failed by rupture of the

plates; however, only BM10H-1's ultimate loading capacities was achieved as a result
56
Load (kN) Load (kN)
Load (kN)
of the yielding in both the steel reinforcement and AA plates, and crushing in concrete, whereas BM10H-2 only exhibited yielding in steel and crushing in concrete. Figure 4-38 shows the load versus deflection and load versus strain curves, whereby the UTM machine and strain gauges were able to capture an extensive amount of points during the testing phase. As a result, the ultimate loading capacities of BM10H-1 and BM10H-2 were 74.4 kN at 16.4 mm and 79.5 kN and 22 mm, respectively. The load versus deflection curve for BM10H-1 experienced a slight drop and exhibited a linear increase until it reached a failure load of 73.5 kN at 32.7 mm followed by a substantial drop in the load; indicating plate rupture as shown in Figure 4-38a. Similarly, the load versus deflection curve of BM10H-2 experienced a slight drop until a load value of 74.1 kN at 25.9 mm as shown in Figure 4-38a. However, the curve suddenly dropped due to rupture of the plate. Using the curves in Figure 4-38a and Figure 4-38b, the mechanical behavior of BM10H-1 was studied in which the loads at yielding of both the steel reinforcement and the AA plate were 56.5 kN at 6.9 mm and 72.7 kN at 14 mm, respectively, and the load at crushing of concrete was 74.1 kN at 22.2 mm. On the other hand, BM10H-2 exhibited yielding only in the steel reinforcement at a load of 59.9 kN at 7.45 mm and crushing of concrete at 79 kN at 22.3 mm, whereas the AA plate's did not undergo yielding where the strain at ultimate load was 0.00224 as shown in Figure 4-38a and Figure 4-38c.Figure 4-38d shows the strain distribution in BM10H-1 where the strain values corresponding to the ultimate load in both the steel reinforcement and the AA plate reached 0.00596 and 0.00651, respectively, and the strain in concrete reach -0.00322. This indicated that yielding occurred in both the steel reinforcement and the plate, and crushing occurred in concrete. However, as mentioned previously, BM10H-2 did not experience AA plate yielding at its ultimate load; mainly, only the steel yielding

occurred, +0.00318, accompanied with slight crushing in concrete crushing, -0.00278, as shown in Figure 4-38e.

4.1.2.4. BEM10E. The last RC specimen that was bolted with a spacing of 100 mm, at its ends, and bonded with epoxy (BEM10E) failed by intermediate debonding, whereby its ultimate loading capacity was achieved as a result of the yielding in both its steel reinforcement and its AA plate, and crushing occurred in the top concrete fibers

57

80 80
60 60
BM10H-1
40 BM10H-2 40 SG Concrete
SG AA Plate
SG Steel
20 20
0 0
0 10 20 30 40 -4 -2 0 2 4 6 8
Deflection (mm) Strain (mm/mm) -3×10
(a) Load versus deflection curve (b) Load versus strain curve for BM10H-1
80
60
40
SG Concrete
SG AA Plate 125 mm εcc = -0.00322
20 SG Steel
205 mm
0 εys= +0.00596
-4 -2 0 2 4
Strain (mm/mm) -310 50 mm εya= +0.00651×
(c) Load versus strain curve for BM10H-2 (d) Strain in section BM10H-1
125 mm εcc = -0.00278
205 mm
εys= +0.00318
50 mm εya= +0.00224
(e) Strain in section BM10H-2

Figure 4-38: Load, deflection and strain of BM10H specimens at mid-span.

during the test. Figure 4-39 shows the load versus deflection and load versus strain curves, whereby the UTM machine and strain gauges were able to capture an extensive amount of points during the testing phase. As a result, the ultimate loading capacity of BEM10E was 82.6 kN at 14.9 mm, as shown in Figure 4-39a. Moreover, the load versus deflection curve experienced a slight drop where it plateaued until it reached a failure load of 74.3 kN at 24.8 mm, as shown in Figure 4-39a. Using the curves in Figure 4-39a and Figure 4-39b, the mechanical behavior of BEM10H-1 was studied in which the

Load (kN) Load (kN)
Load (kN)
loads at yielding of both the steel reinforcement and the AA plate were 68.3 kN at 7.09 mm and 80.2 kN at 10.81 mm, respectively, and the load at crushing of concrete was 80.1 kN at 17.1 mm. Figure 4-39c shows the strain distribution in BEM10E where the strain values corresponding to the ultimate load in both the steel reinforcement and the AA plate reached 0.00398 and 0.00676, respectively, and the strain in concrete reach -0.00308. This indicated that yielding occurred in both the steel reinforcement and the plate, and crushing occurred in concrete.
100 100
80 80
60 60
40 40 SG Concrete
SG AA Plate
20 20 SG Steel
0 0
0 5 10 15 20 25 -4 -2 0 2 4 6 8
Deflection (mm) Strain (mm/mm) -310
(a) Load versus deflection curve (b) Load versus strain curve
125 mm $\varepsilon_{cc} = -0.00308$
205 mm
$\varepsilon_{ys} = +0.00398$
50 mm $\varepsilon_{ya} = +0.00676$
(c) Strain in the section
Figure 4-39: Load, deflection and strain for BEM10E at mid-span.

4.1.3. M12-Specimens. The last seven RC specimens externally strengthened with AA plates by means of bolting, using HST3 M12 expansion anchors, with/without adhesive bonding were loaded until failure. Each specimen included a replica of it-self except for BEM12E, which granted the author insight on the consistency of the results corresponding to each strengthened specimen's unique anchorage system. Fur-thermore, these beams were tested under a four-point bending configuration in which a

Load (kN)
Load (kN)
displacement rate of 2mm/min was subjected until failure by means of plate rupture or IC debonding occurred.

4.1.3.1. BEM12L. The first two RC specimens that were bolted at a spacing of 200 mm and bonded with AA plates (BEM12L-1 and BEM12L-2) failed by rupture of plates, whereby their sections' ultimate loading capacities were achieved as a result of the yielding in both the steel reinforcement and AA plates, and crushing occurred in the top concrete fibers of both beams during the test. Figure 4-40 shows the load versus deflection and load versus strain curves, whereby the UTM machine and strain gauges were able to capture an extensive amount of points during the testing phase. As a result, the ultimate loading capacities of BEM12L-1 and BEM12L-2 were 78.1 kN at 16.1 mm and 86.4 kN and 16.5 mm, respectively. The loading, then, experienced a slight but negligible drop where it plateaued until both specimens, BEM12L-1 and BEM12L-2, reached failure loads of 72.7 kN at 24.6 mm and 79.6 kN at 25. mm, respectively. Afterwards, both specimens experienced a large drop in their ultimate loading capacities due to the rupture of the AA plates. Using the curves in Figure 4-40a and Figure 4-40b, the mechanical behavior of BEM12L-1 was studied in which the loads at yielding of both the steel reinforcement and the AA plate were 61.3 kN at 6.81 mm and 75.7 kN at 12.65 mm, respectively, and experienced concrete at a load of 77.4 kN at 16.3 mm. Similarly, BEM12L-2 exhibited yielding in both its steel reinforcement and the its AA plate at 68.8 kN at 6.69 mm and 81.8 kN at 10.9 mm, respectively, and demonstrated concrete crushing at 85.4 kN at 16.8 mm as shown in Figure 4-40a and Figure 4-40c. Figure 4-40d shows the strain measurements corresponding to the ultimate load where the strain values in both the steel reinforcement and the AA plate indicated yielding by reaching values of 0.00427 and 0.00484, respectively, and the strain in concrete showed crushing with a value of -0.00298. Similarly, Figure 4-40e shows the strain measurements corresponding to the ultimate load where the strain values in both the steel reinforcement and the AA plate indicated yielding by reaching values of 0.00594 and 0.00649, respectively, and the strain in concrete showed crushing with a value of -0.00367.

60

100 100

80 80

60 60

BEM12L-1 SG Concrete

BEM12L-2
40 40 SG AA Plate
SG Steel
20 20
0 0
0 10 20 30 -4 -2 0 2 4 6 8
Deflection (mm) Strain (mm/mm) -3×10
(a) Load versus deflection curve (b) Load versus strain curve for BEM12L-1
100
80
60
SG Concrete
40 SG AA Plate
SG Steel 125 mm cc = -0.00298
20
205 mm
0 ys= +0.00427
-4 -2 0 2 4 6 8
Strain (mm/mm) -310 50 mm ya= +0.00484×
(c) Load versus strain curve for BEM12L-2 (d) Strain in section BEM12L-1
125 mm cc = -0.00367
205 mm
ys= +0.00594
50 mm ya= +0.00649
(e) Strain in section BEM12L-2
Figure 4-40: Load, deflection and strain of BEM12L specimens at mid-span.
4.1.3.2. BEM12H. Similarly, the two plated RC specimens that were bolted at a spacing of 100 mm and bonded using the same adhesive (BEM12H-1 and BEM12H-2) also failed by rupture of plates, whereby their sections' ultimate loading capacities were achieved as a result of the yielding in both the steel reinforcement and AA plates, and crushing occurred in the top concrete fibers of both beams during the test. Figure 4-41 shows the load versus deflection and load versus strain curves, whereby the UTM machine and strain gauges were able to capture an extensive amount of points

Load (kN) Load (kN)
Load (kN)
during the testing phase. As a result, the ultimate loading capacities of BEM12H-1 and BEM12H-2 were 82.2 kN at 18.3 mm and 86.2 kN and 19.1 mm, respectively. The loading, then, experienced a slight but negligible drop where it plateaued until both specimens, BEM12H-1 and BEM12H-2, reached failure loads of 78.3 kN at 23.6 mm and 80.3 kN at 31.1 mm, respectively. Afterwards, both specimens experienced a large

drop in their ultimate loading capacities due to the rupture of the AA plates. Using the curves in Figure 4-41a and Figure 4-41b, the mechanical behavior of BEM12H-1 was studied in which the loads at yielding of both the steel reinforcement and the AA plate were 74.9 kN at 8.96 mm and 81.2 kN at 16.5 mm, respectively, and the load at crushing of concrete was 82.1 kN at 16.4 mm. Similarly, BEM12H-2 exhibited yielding in both its steel reinforcement and the its AA plate at 65.9 kN at 6.9 mm and 80.6 kN at 11.7 mm, respectively, and experienced concrete crushing at 86.1 kN at 19.3 mm as shown in Figure 4-41a and Figure 4-41c. Figure 4-41d shows the strain distribution in BEM12H-1 where the strain values corresponding to the ultimate load in both the steel reinforcement and the AA plate reached 0.00368 and 0.00584, respectively, and the strain in concrete reach -0.00326. This indicated that yielding occurred in both the steel reinforcement and the plate, and crushing occurred in concrete. Similarly, yielding in both internal and external reinforcements occurred whereby their strain values corresponding to the ultimate load were 0.00407 and 0.00586, respectively, and concrete crushing occurred at a strain of -0.00299.

4.1.3.3. BM12H. The two RC specimens that were only bolted at a spacing of 100 mm (BM12H-1 and BM12H-2) also failed by rupture of plates, whereby their sections' ultimate loading capacities were achieved as a result of the yielding in both the steel reinforcement and AA plates, and crushing in the concrete of both beams during the test. Figure 4-42 shows the load versus deflection and load versus strain curves, in which the UTM machine and strain gauges were able to capture an extensive amount of points during the testing phase. As a result, the ultimate loading capacities of BM12H-1 and BM12H-2 were 83.5 kN at 19.5 mm and 83.8 kN and 20.1 mm, respectively, followed by slight drops in the loading until their failure loads reached 79.5 kN at 26.1 mm and 78.9 kN at 28.7 mm, respectively, indicating plate rupture

100 100
80 80
60 60
SG Concrete
40 40 SG AA Plate

BEM12H-1 SG Steel
20 BEM12H-2 20
0 0
0 10 20 30 -4 -2 0 2 4 6 8
Deflection (mm) Strain (mm/mm) -3×10
(a) Load versus deflection curve (b) Load versus strain curve for BEM12H-1
100
80
60
40
SG Concrete 125 mm cc = -0.00326
SG AA Plate
20
SG Steel 205 mm
0 ys= +0.00368
-4 -2 0 2 4 6
Strain (mm/mm) -3×10 50 mm ya= +0.00584
(c) Load versus strain curve for BEM12H-2 (d) Strain in section BEM12H-1
125 mm cc = -0.00299
205 mm
ys = +0.00407

50 mm ya = +0.00586
(e) Strain in section BEM12H-2
Figure 4-41: Load, deflection and strain of BEM12H specimens at mid-span.
in both specimens, as shown in Figure 4-42a. Using the curves in Figure 4-42a and Figure 4-42b, the mechanical behavior of BM12H-1 was studied in which the loads at yielding of both the steel reinforcement and the AA plate were 64.9 kN at 7.17 mm and 81.7 kN at 16.7 mm, respectively, and the load at crushing of concrete was 83.2 kN at 20.1 mm. Similarly, BM12H-2 exhibited yielding in both the steel reinforcement and the AA plate at loads of 57.8 kN at 5.95 mm and 83 kN at 18 mm, respectively, and crushing of concrete occurred at a load of 83.4 kN at 20.3 mm as shown in Figure 4-

Load (kN) Load (kN)
Load (kN)
42a and Figure 4-42c.Figure 4-42d shows the strain distribution in BM12H-1 where the strain values corresponding to the ultimate load in both the steel reinforcement and the AA plate reached 0.00587 and 0.00545, respectively, and the strain in concrete reach -0.00298. This indicated that yielding occurred in both the steel reinforcement and the plate; however, it was observed that the strain in the plate was less than that of the re-

inforcement. An observation can be made in which the bolt holes were subjected to a concentrated flexural stress; thereby increasing the size of the hole and causing the bolt to demonstrate a rigid body rotation. This rigid body rotation was responsible for slightly damping the strain-effect in the AA plate. Moreover, BM12H-2 also experienced yielding in both the steel reinforcement and the AA plate at values of +0.00473 and +0.00424, respectively, and crushing in concrete at a strain of -0.00365. But owing to the large stress concentration in the holes, the bolt in BM12H-2 also experienced rigid body rotation in which the AA plate’s strain-effect was slight damped as shown in Figure 4-42e.

4.1.3.4. BEM12E. The last RC specimen that was bolted with a spacing of 100 mm center-to-center, at its ends, and bonded with epoxy (BEM12E) failed by intermediate debonding, whereby its ultimate loading capacity was achieved as a result of the yielding in both its steel reinforcement and its AA plate, and crushing occurred in the top concrete fibers during the test. Figure 4-43 shows the load versus deflection and load versus strain curves, whereby the UTM machine and strain gauges were able to capture an extensive amount of points during the testing phase. As a result, the ultimate loading capacity of BEM12E was 86.1 kN at 21 mm, as shown in Figure 4-43a, followed by a slight drop in the load where it plateaued until it reached a failure load of 72.1 kN at 41 mm, as shown in Figure 4-43a. Using the curves in Figure 4-43a and Figure 4-39b, the mechanical behavior of BEM12E was studied in which the loads at yielding of both the steel reinforcement and the AA plate were 67.4 kN at 7.4 mm and 78.9 kN at 9.62 mm, respectively, and the load at crushing of concrete was 85.7 kN at 21.3 mm. Figure 4-43c shows the strain distribution in BEM12E where the strain values corresponding to the ultimate load in both the steel reinforcement and the AA plate reached 0.00559 and 0.00593, respectively, and the strain in concrete reach -0.00270.

100 100
80 80
60 60
40 40

BM12H-1 SG Concrete
BM12H-2
20 SG AA Plate20
SG Steel
0 0
0 10 20 30 -4 -2 0 2 4 6
Deflection (mm) Strain (mm/mm) -3×10
(a) Load versus deflection curve (b) Load versus strain curve for BM12H-1
100
80
60
40
125 mm
SG Concrete cc = -0.00298
20 SG AA Plate
SG Steel 205 mm
0
ys = +0.00587
-4 -2 0 2 4 6
Strain (mm/mm) -3
ya = +0.00545
×10 50 mm
(c) Load versus strain curve for BM12H-2 (d) Strain in section BM12H-1
125 mm cc = -0.00365
205 mm
ys = +0.00473

50 mm ya = +0.00424
(e) Strain in section BM12H-2
Figure 4-42: Load, deflection and strain of BM12H specimens at mid-span.
This indicated that yielding occurred in both the steel reinforcement and the plate, and slight crushing occurred in concrete.

4.2. Summary of Results and Remarks

The measured loads and deflections at different limit states were tabulated in two tables as shown in Table 4-9 and Table 4-10. It can be concluded that all the strengthened specimens, despite the type of anchorage system, sustained a larger load capacity

Load (kN) Load (kN)
Load (kN)
100 100
80 80
60 60
40 40
SG Concrete
20 20 SG AA Plate
SG Steel
0 0
0 10 20 30 40 -4 -2 0 2 4 6
Deflection (mm) Strain (mm/mm) -310

(a) Load versus deflection curve (b) Load versus strain curve
125 mm
✄c c= -0.00270
205 mm
✄ y s = +0.00559
50 mm ✄ y a = +0.00593
(c) Strain in the section
Figure 4-43: Load, deflection and strain of BEM12E specimen at mid-span.

than that of the reference specimens (CB). In addition, the degree of deformation, ductility, was also an underlying parameter that distinguished the bonded specimen (CBE) with the specimens that are bolted with/without bonding (M10-Specimens and M12-Specimens). This gave room to calculate the ductility index to monitor the effectiveness of each anchorage system.

Table 4-9: Summary of ultimate and failure limits in all specimens.

Beam ID	Pult(kN)	δult(mm)	Pfail(kN)	δfail(mm)
CB	64.2	17.4	57.7	30.5
CBE	84.4	14.3	84.4	19.94
BEM10L-1	80	16	76.2	25
BEM10L-2	85.8	17.1	78	24.4
BEM10H-1	80.5	15.6	76.5	29.4
BEM10H-2	86.5	18.5	80	26.5
BM10H-1	74.4	16.4	73.5	32.7
BM10H-2	79.5	22	74.1	25.9
BEM10E	82.6	14.9	74.3	24.8
BEM12L-1	78.1	16.1	72.7	24.6
BEM12L-2	86.4	16.5	79.6	25
BEM12H-1	82.2	18.3	78.3	23.6
BEM12H-2	86.2	19.1	80.3	31.3
BM12H-1	83.5	19.5	79.5	26.1
BM12H-2	83.8	20.1	78.9	28.7
BEM12E	86.1	21	72.1	41

Load (kN)
Load (kN)

Table 4-10: Summary of mechanical response of each element.

Beam ID	P 1 1 2 2steel,y(kN)	δsteel,y(mm)	PAA,y(kN)	δAA,y(mm)	Pconc,cr(kN)	δconc,cr(mm)
CB	53.2	6.77	-	-	63.6	17.8
CBE	65.1	6.62	72.2	7.56	83.9	14.1
BEM10L-1	N.A	N.A	69.6	7.6	79.8	14.8
BEM10L-2	67.5	7.06	80.1	10.4	85	17.5
BEM10H-1	72.8	8.23	80.3	16.5	80.32	16.2
BEM10H-2	67.5	7.1	80.1	10.4	84.7	19
BM10H-1	56.5	6.9	72.7	14	74.1	22.2
BM10H-2	59.9	7.45	N.Y	N.Y	79	22.3
BEM10E	68.3	7.09	80.2	10.8	80.1	17.1
BEM12L-1	61.3	6.81	75.7	12.6	77.4	16.3
BEM12L-2	68.8	6.69	81.8	10.9	85.4	16.8
BEM12H-1	74.9	8.96	81.2	16.5	82.1	16.4

BEM12H-2	65.9	6.9	80.6	11.7	86.1	19.3
BM12H-1	64.9	7.17	81.7	16.7	83.2	20.1
BM12H-2	57.8	5.95	83	18	83.4	20.3
BEM12E	67.4	7.4	78.9	9.62	85.7	21.3

1 N.A: Strain gauge was damaged
2 N.Y: No yielding occurred

4.2.1. Effect of M10 bolts with/without bonding. The M10-Specimens were composed of RC beams that were strengthened, using AA plates, by implementing four unique anchorage configurations; namely, HST3 M10 bolts at 100 mm center-to-center plus bonding (BEM10H), HST3 M10 bolts at 200 mm center-to-center plus bonding (BEM10H), only HST3 M10 bolts at 100 mm center-to-center (BM10H), and HST3 M10 bolts at 100 mm center-to-center on the edges plus bonding (BEM10E). Their results were compared to the unstrengthened specimen (CB-Specimens) based on the load versus deflection curves shown in Figure 4-44. As a result, it was observed that the implementation of bolting HST3 M10 bolts and/or bonding AA plates to the soffits of RC beams enhanced their flexural capacities when compared to an unstrengthened beam.

Moreover, the M10-Specimens group’s load versus deflection curves demonstrated similar profiles to the CB specimens, in terms of ductility, indicating a flexural response rather than an immediate failure (i.e.,debonding/delamination of the plate). The varying parameters that impacted the flexural behavior of the specimens in the M10-Specimens group were: the process of alternating between incoporating bolts with epoxy and incorporating epoxy alone. In this section, BM10H specimens showed a delay in strength gain, roughly at 8 mm deflection, due to the plate-slip that occurred between the bolts and their surrounding holes. Afterwards, the BM10H specimens’ plates were bearing on the bolts such that the beams’ flexural stresses were transferred to the plates; resulting in a lag in strain that is graphically emphasized in the load versus strain curves

shown in the previous sections. The BEM10E specimen continued to resist loads until the deflection surpassed that of the CB specimens by, approximately, 3 mm followed by intermediate debonding. This indicated that the un-bolted and bonded length, in the maximum moment zone, demonstrated better stress transfer when compared to a uniformly bolted plate; whereby the stresses were acting on the segmented plate length between each bolt. Ultimately, the M10-Specimens group's curves demonstrated load drops as a result of plate rupture or intermediate debonding, which will be discussed later in this report.

Furthermore, Figure 4-44 shows a bi-linear profile just before the sections begin to exhibit inelastic nature. This helped conclude the simultaneous contribution between the AA plate and steel reinforcement, despite the presence of the epoxy, in the load capacity of the RC beams. Afterwards, the curves reach the plastic stage in which both hardening and softening occur at the ultimate and failure loads, respectively. To quantify these observations, a table was developed in which the percentage increase in the load, at different stages, were calculated based on the results obtained from the CB-Specimens. These stages include the load at: (a) the ultimate state; (b) the failure state; (c) concrete crushing state; (d) steel yielding state; (e) AA plate yielding state. Table 4-11 shows the load increase, in the ultimate and failure states, of the M10-Specimens.

It is worth noting that the loads at failure were extracted based on the final point prior to immediate failure (rupture of plate). Furthermore, the maximum percentage increase in the ultimate and failure state took place with BEM10H-2 at 35% and 25% respectively, as shown in Table 4-11. This indicated that the incorporation of the AA plate contributed to a large increase in the specimen's flexural capacity. Moreover, the coefficient of variation (CV) was calculated to understand the degree of variation in the experimental results as a means to observe the pattern the strengthening configuration. Equation 1 was used; where CV, σ and μ are the coefficient of variation, standard deviation and mean of the dataset. As a result, the computed CV for the load increase in the ultimate and failure states were 22.3% and 18.3%; hence, degree of variation of the flexural capacity of each section are almost similar.

$$CV = \frac{\sigma}{\mu} \quad (1)$$

Table 4-12 shows the percentage load increase when crushing and yielding of both steel reinforcement and AA plate occurred. Unlike the percentage increase shown in Table 4-11, the maximum percentage increase shown in Table 4-12 occurred for different specimens. For example, the maximum load increase at which crushing occurred was for BEM10L-2 at 34%, while the maximum load increase at which yielding in the steel reinforcement and the AA plate occurred was for BEM10H-1 at 37% and 11%, respectively. However, for M10-Specimens, the maximum load increase at which the AA plate yielded occurred for most of the specimens, which indicated ideal and consistent load transfer throughout the anchorage system imposed onto the external reinforcement. Figure 4-45 graphically summarizes the load percentage increase for each limit state; whereby ULS, FLS, CC, YS and YA stand for the load percentage increase during the ultimate load state, failure load state, concrete crushing, yielding of steel and yielding of AA plates, respectively. The bar chart demonstrated major improvements in the specimens' strength enhancements when HST3 M12 expansion anchors were used with epoxy, rather than using mechanical anchors alone. This helped conclude that the torque magnitudes, imposed by the manufacturer, combined with the shear stress distribution, between the surface area of the bolts and the concrete holes, formed a pre-tension loading-layout in the bolts; whereby the degree at which the AA plates settled into the epoxy was greater than that of the BM10H specimens without any adhesives. This controlled settlement subjected the epoxy with a normal stress and provided confinement in the cohesive layer (between the epoxy particles) such that the epoxy required larger shear stress to fail. This type of loading was maintained throughout the AA plates' span, due to the uniform bolting scheme, and helped enhance the epoxy's bond; thereby suppressing any internal cracks within the epoxy. Figure 4-44 shows this phenomenon when observing the gain in stiffness for the epoxy-filled beams in the M10-Specimens demonstrated a larger shift in the loading capacity before reaching the plastic stage, whereas the specimens that were bolted without using epoxy (BM10H) reached the plastic stage at a lower load. Another observation can be made; whereby the spacing of bolts did not contribute to the strength enhancements when viewing BEM10L and BEM10H specimens in both Figure 4-44 and Figure 4-45.

100
80
60
CB
BEM10L-1
40 BEM10L-2
BEM10H-1
BEM10H-2
20 BM10H-1
BM10H-2
BEM10E
0
0 10 20 30 40
Deflection (mm)

Figure 4-44: Load versus deflection curves of CB compared to M10-Specimens.

Table 4-11: Strength increase in the ultimate and failure states for M10-Specimens.

Beam ID	Ultimate State		Failure State	
	Pult(kN)	Increase (%)	Pfail(kN)	Increase (%)
CB	64.2	-	57.7	-
BEM10L-1	80	25	76.2	19
BEM10L-2	85.8	34	78	21
BEM10H-1	80.5	25	76.5	19
BEM10H-2	86.5	35	80	25
BM10H-1	74.4	16	73.5	14
BM10H-2	79.5	24	74.1	15
BEM10E	82.6	29	74.3	16

Table 4-12: Strength increase when concrete crushing occurs and yielding in both steel and AA plate for M12-Specimens.

Beam ID	$P_{conc,cr}$ (kN)	Increase (%)	$P_{steel,y}$ (kN)[1]	Increase (%)[1]	$P_{AA,y}$ (kN)[2]	Increase (%)[2]
CB	63.6	-	53.2	-	72.2	-
BEM10L-1	79.8	25	N.A	N.A	69.6	-4
BEM10L-2	85	34	67.5	27	80.1	11
BEM10H-1	80.32	26	72.8	37	80.3	11
BEM10H-2	84.7	33	67.5	27	80.1	11
BM10H-1	74.1	17	56.5	6	72.7	1
BM10H-2	79	24	59.9	13	N.Y	N.Y
BEM10E	80.1	26	68.3	28	80.2	11

1 N.A: Strain gauge was damaged
2 N.Y: No yielding occurred

Load (kN)
140
120 ULS
FLS
CC

100 YS
YAA
80
60
40
20
0
L-
1
L-
2 -1 -2H H H-
1 -2 0E
10 10 10 10 0 0
H 1
EM
M
EM EM M M
1 M1 BEB B B BE B B
Beam ID

Figure 4-45: Bar plot of load comparisons for the M10-Specimens group.

4.2.2. Effect of M12 bolts with/without bonding. The M12-Specimens were composed of RC beams that were strengthened using four anchorage configurations; namely, HST3 M12 bolts at 100 mm center-to-center plus bonding (BEM12H), HST3 M12 bolts at 200 mm center-to-center plus bonding (BEM12H), only HST3 M12 bolts at 100 mm center-to-center (BM12H), and HST3 M12 bolts at 100 mm center-to-center on the edges plus bonding (BEM12E). Their results were compared to the unstrengthened specimens (CB-Specimens) based on the load versus deflection curves shown in Figure 4-46. As a result, it was observed that the implementation of bolting HST3 M12 bolts and/or bonding AA plates to the soffits of RC beams also enhanced their flexural capacities when compared to an unstrengthened beam. Moreover, the M12-Specimens group’s load versus deflection followed a flexural loading profile, similar to the M10-Specimens group. Unlike the BM10H specimens, the BM12H specimens exhibited better interaction between the RC section and the plate; whereby the curves approximately resembled the flexural behaviour of the BEM12L and BEM12H specimens. Afterwards, the BM10H specimens’ plates were bearing on the bolts such that the beams’ flexural stresses were transferred to the plates; resulting in a lag in strain that is graphically emphasized in the load versus strain curves shown in the previous sections.

The BEM12E specimen continued to resist loads until the deflection surpassed that of

71

Percentage Increase in Load (%)

the CB specimens by, approximately, 10 mm followed by intermediate debonding. This indicated that the un-bolted and bonded length, in the maximum moment zone, demonstrated better stress transfer when compared to a uniformly bolted plate; whereby the stresses were acting on the segmented plate length between each bolt. Ultimately, the M12-Specimens group's curves demonstrated load drops as a result of plate rupture or intermediate debonding, which will be discussed later in this report. Figure 4-46 also shows a bi-linear profile, before the beginning of the inelastic stage, representing the contribution of both the internal and external reinforcements during loading. Afterwards, the curves reached the plastic stage in which both hardening and softening occurred at the ultimate and failure loads, respectively. To quantify these observations, a table was developed in which the percentage increase in the load, at different stages, were calculated based on the results obtained from the CB-Specimens. Table 4-13 shows the load increase, in the ultimate and failure states, of the M12-Specimens. The maximum percentage increase in the ultimate state took place for BEM12L-2 at 35%, whereas the maximum percentage increase in the failure state took place for BEM12H-2 at 39%, as shown in Table 4-13. Therefore, the CV was evaluated for the load increase in the ultimate and failure states, and were reported as 14.1% and 16.2%; hence, degree of variation of the flexural capacity of each section are much closer than that of the M10-Specimens.

Table 4-14 shows the percentage load increase when crushing and yielding of both steel reinforcement and AA plate took place in the M12-Specimens. For example, the maximum load increase at which crushing occurred was for both BEM12H-2 and BEM12E at 35%, while the maximum load increase at which yielding in the steel reinforcement and the AA plate occurred was for BEM12H-1 and BM12H-2 at 41% and 15%, respectively. Similar to the M12-Specimens, the variation of percentage load increase when AA plate yielding took place is negligible. Figure 4-47 graphically summarizes the load percentage increase for each limit state. Similar to Figure 4-45, the bar chart shows that most of the specimens, consisting of epoxy and bolts, have helped the RC section utilize its material's mechanical properties more than the specimens that consisted of anchors alone. However, the variation in strength enhancement is negligible since the change from HST3 M10 bolts to HST3 M12 bolts was accompa-

nied by an increase in the torque used to fix the plate. This increased the pre-tension force throughout the bolts, which caused the BM12H specimens' anchorage system to achieve the same shear strength capacity as the hybrid anchorage system in BEM12H, BEM12L and BEM12E specimens. It was, then, concluded that the shear stress required to overcome the friction between the plate and concrete, in the BM12H specimens, almost reached the shear stress required to overcome the epoxy's shear strength within the BEM12 specimens. Similar to the previous section, the spacing of bolts did not have an effect on the strength enhancements when viewing the BEM12L and BEM12H specimens.

100
80
60
CB
BEM12L-1
BEM12L-2
40
BEM12H-1
BEM12H-2
BM12H-1
20
BM12H-2
BEM12E
0
0 10 20 30 40 50
Deflection (mm)

Figure 4-46: Load versus deflection curves of CB compared to M12-Specimens.

Table 4-13: Strength increase in the ultimate and failure states for M12-Specimens.

Beam ID	Ultimate State		Failure State	
	Pult(kN)	Increase (%)	Pfail(kN)	Increase (%)
CB	64.2	-	57.7	-
BEM12L-1	78.1	22	72.7	26
BEM12L-2	86.4	35	79.6	38
BEM12H-1	82.2	28	78.3	36

BEM12H-2	86.2	34	80.3	39
BM12H-1	83.5	30	79.5	38
BM12H-2	83.8	31	78.9	37
BEM12E	86.1	34	72.1	25

4.2.3. Effect of different bolt models at low-uniform spacing with bonding. This section aims at describing the effect of varying the torque magnitudes, at low

73

Load (kN)

Table 4-14: Strength increase when concrete crushing occurs and yielding in both steel and AA plate for M12-Specimens.

Beam ID	Pconc,cr(kN)	Increase (%)	Psteel,y(kN)	Increase (%)	PAA,y(kN)	Increase (%)
CB	63.6	-	53.2	-	72.2	-
BEM12L-1	77.4	22	61.3	15	75.7	5
BEM12L-2	85.4	34	68.8	29	81.8	13
BEM12H-1	82.1	29	74.9	41	81.2	12
BEM12H-2	86.1	35	65.9	24	80.6	12
BM12H-1	83.2	31	64.9	22	81.7	13
BM12H-2	83.4	31	57.8	9	83	15
BEM12E	85.7	35	67.4	27	78.9	9

150
ULS
FLS
CC
YS
YAA
100
50
0
-1 -2 -1 -2 -1 -2L L 2E
12 12 H H H H 1M M M1
2
M1
2 12 12 MM M E
BE BE BBE BE B B
Beam ID

Figure 4-47: Bar plot of load comparisons for the M12-Specimens group.

spacing, in the structural response of strengthened RC beams using different bolt models with adhesive bonding configurations. The results obtained were compared to the unstrengthened specimens (CB-Specimens) based on the load versus deflection curves shown in Figure 4-48. It was observed that the curves approximately demonstrate similar profiles in which both BEM10H and BEM12H specimens exhibited higher load capacities with relatively similar ductility when compared to CB. This was quantified by computing the percentage increase for different states, similar to the previous sec-

tions. Table 4-15 shows the load increase, in the ultimate and failure states, between the BEM10H and BEM12H specimens. The maximum percentage increase in the ultimate state took place for BEM10H-2 at 35%, whereas the maximum percentage increase in the failure state took place for BEM10H-2 and BEM12H-2 at 39%, as shown in Table 4-
74
Percentage Increase in Load (%)
15. Due to the small variation in the values, it is difficult to pinpoint which anchorage technique is superior. Therefore, the CV was evaluated for the load increase in the ultimate and failure states, and were reported as 7.21% and 13.1%, which reinforces this conclusion.

Table 4-16 shows the percentage load increase when crushing and yielding of both steel reinforcement and AA plate took place in both BEM10H and BEM12H. For example, the maximum load increase at which crushing occurred was for both BEM12H-2 at 35%, while the maximum load increase at which yielding in the steel reinforcement and the AA plate occurred was for BEM12H-1 at 41% and 15%, respectively. Although the maximum percentage increase in terms of mechanical response occurred for the BEM12H specimens, it is still not sufficient enough to conclude whether BEM12H specimens are superior to the BEM10H ones. Figure 4-51 graphically summarizes the load percentage increase for each limit state, where maintaining the same spacing while changing from HST3 M10 bolts to HST3 M12 bolts, regardless of the presence of epoxy, slightly affected the strength enhancement in the RC beams.

100
80
60
CB
40 BEM10H-1
BEM10H-2
20 BEM12H-1
BEM12H-2
0
0 10 20 30 40
Deflection (mm)

Figure 4-48: Load versus deflection curves for different bolt models at low-uniform spacing.

4.2.4. Effect of different bolt models at high-uniform spacing with bonding. This section aims at describing the effect of varying the torque magnitudes, at high spacing, on the structural response of strengthened RC beams using different bolt mod-

75

Load (kN)

Table 4-15: Strength increase in the ultimate and failure states for different bolt models at low-uniform spacing.

Beam ID	Ultimate State		Failure State	
	Pult(kN)	Increase (%)	Pfail(kN)	Increase (%)
CB	64.2	-	57.7	-
BEM10H-1	80.5	25	76.5	33
BEM10H-2	86.5	35	80	39
BEM12H-1	82.2	28	78.3	36
BEM12H-2	86.2	34	80.3	39

Table 4-16: Strength increase at concrete crushing and yielding for different bolt models at low-uniform spacing.

Beam ID	Pconc,cr(kN)	Increase (%)	Psteel,y(kN)	Increase (%)	PAA,y(kN)	Increase (%)
CB	63.6	-	53.2	-	-	-
BEM10H-1	80.32	26	72.8	37	80.3	11
BEM10H-2	84.7	33	67.5	27	80.1	11
BEM12H-1	82.1	29	74.9	41	81.2	12
BEM12H-2	86.1	35	65.9	24	80.6	12

150
100
ULS
FLS
CC
YS
50 YAA
0
H-
1 -2 1 2
0 0H H
- H-
M1 M1 M1
2 12
BE BE
M
BE BE
Beam ID

Figure 4-49: Bar plot of load comparisons.

els with adhesive bonding configurations. The results obtained were compared to the unstrengthened specimens (CB-Specimens) based on the load versus deflection curve shown in Figure 4-50. It was observed that, in terms of strength performance, the curves approximately demonstrate similar profiles; in which BEM10L and BEM12L specimens exhibited higher load capacities with relatively similar ductility when compared to CB. This was quantified by computing the percentage increase for different states, similar to the previous sections. Table 4-17 shows the load increase, in the ulti-

76

Percentage Increase in Load (%)

mate and failure states, between the BEM10H and BEM12H specimens. The maximum percentage increase in the ultimate and failure state took place for BEM12L-2 at 35% and 38%, respectively, as shown in Table 4-17. However, due to the small variation in the values, it is difficult to pinpoint which anchorage technique is superior. Therefore the CV was evaluated for the load increase in the ultimate and failure states, and were reported as 19.5% and 13.3%, which reinforces the consistency of the results.

Table 4-18 shows the percentage load increase when crushing and yielding of both steel reinforcement and AA plate took place in both BEM10H and BEM12H. For example, the maximum load increase at which crushing occurred was for both BEM12L-2 and BEM12L-2 at 34%, while the maximum load increase at which yielding in the steel reinforcement and the AA plate occurred was for BEM12L-2 at 29% and 13%, respectively. In general, it is observed that the maximum load increase in which crushing of concrete and yielding of both internal and external reinforcements occurred in the specimen. However, this does not warrant a sufficient conclusion about which anchorage configuration is superior to the other. Figure 4-51 graphically summarizes the load percentage increase for each limit state, where it can be observed that the BEM10L and BEM12L specimens, regardless of the presence of epoxy, did not impact the strength enhancements as much as the torque magnitudes.

100
80
60
40 CB
BEM10L-1
BEM10L-2
20 BEM12L-1
BEM12L-2
0
0 10 20 30 40
Deflection (mm)

Figure 4-50: Load versus deflection curves for different bolt models at high-uniform spacing.

77

Load (kN)

Table 4-17: Strength increase in the ultimate and failure states for different bolt models at high-uniform spacing.

Beam ID	Ultimate State		Failure State	
	Pult(kN)	Increase (%)	Pfail(kN)	Increase (%)
CB	64.2	-	57.7	-
BEM10L-1	80	25	76.2	32
BEM10L-2	85.8	34	78	35
BEM12L-1	78.1	22	72.7	26
BEM12L-2	86.4	35	79.6	38

Table 4-18: Strength increase at concrete crushing and yielding for different bolt models at high-uniform spacing.

Beam ID	Pconc,cr(kN)	Increase (%)	Psteel,y(kN)	Increase (%)	PAA,y(kN)	Increase (%)
CB	63.6	-	53.2	-	-	-
BEM10L-1	79.8	25	N.A	N.A	69.6	-4
BEM10L-2	85	34	67.5	27	80.1	11
BEM12L-1	77.4	22	61.3	15	75.7	5
BEM12L-2	85.4	34	68.8	29	81.8	13

150
ULS
FLS
CC
YS
100
YAA
50
0
-1 -2 -1L -
2
10 10
L 2L1 12
L
EM EM EM MB B B BE
Beam ID

Figure 4-51: Bar plot of load comparisons.

4.2.5. Effect of different bolt models at high-uniform spacing without bonding. This section aims at describing the effect of varying the torque magnitudes, at high-uniform spacing, on the structural response of strengthened RC beams by means of alternating between two bolt models (HST3 M10 and HST3 M12) without using epoxy. The results obtained were compared to the unstrengthened specimens (CB-Specimens) based on the load versus deflection curve shown in Figure 4-52. It was observed that, in

Percentage Increase in Load (%)

terms of strength performance, the curves approximately demonstrated similar profiles; in which BM10H and BEM12H specimens exhibited higher load capacities with relatively similar or larger ductility when compared to CB. This was quantified by computing the percentage increase for different states, similar to the previous sections. Table 4-19 shows the load increase, in the ultimate and failure states, between the BEM10E and BEM12E specimens. The maximum percentage increase in the ultimate state took place for BM12H-2 at 34%, whereas the failure state took place for BM12H-1 at 24%, as shown in Table 4-19. However, due to the small variation in the values, it is difficult to pinpoint which anchorage technique is superior. Therefore, the CV was evaluated for the load increase in the ultimate and failure states, and were reported as 2.33% and 2.34%, which reinforces the consistency of the results.

Table 4-20 shows the percentage load increase when crushing and yielding of both steel reinforcement and AA plate took place in both BM10H and BM12H. For example, the maximum load increase at which crushing occurred was for both BM12H-1 and BM12H-2 at 31%, while the maximum load increase at which yielding, in the steel reinforcement and the AA plate, occurred was for BM12H-1 and BM12H-2 at 22% and 11%, respectively. In general, it is observed that the maximum load increase in which crushing of concrete and yielding of both internal and external reinforcements occurred in the specimen. However, this does not warrant a sufficient conclusion about which anchorage configuration is superior to the other. Figure 4-53 graphically summarizes the load percentage increase for each limit state, where it can be observed that the BM12H specimens demonstrated higher strength enhancements than the BM10H specimens. This supports the phenomenon that describes the effect large torque magnitudes possess in utilizing the high frictional shear strength between the concrete and plates.

Table 4-19: Strength increase in the ultimate and failure states for different bolt models at low-uniform spacing without bonding.

Beam ID	Ultimate State		Failure State	
	Pult(kN)	Increase (%)	Pfail(kN)	Increase (%)

CB	64.2	-	57.7	-
BM10H-1	74.4	16	73.5	14
BM10H-2	79.5	24	74.1	15
BM12H-1	83.5	30	79.5	24
BM12H-2	83.8	31	78.9	23

79
100
80
60
CB
40 CBE
BM10H-1
BM10H-2
20
BM12H-1
BM12H-2
0
0 10 20 30 40
Deflection (mm)

Figure 4-52: Load versus deflection curves for different bolt models at low-uniform spacing without bonding.

Table 4-20: Strength increase at concrete crushing and yielding for different bolt models at low-uniform spacing without bonding.

Beam ID	Pconc,cr(kN)	Increase (%)	Psteel,y(kN)	Increase (%)	PAA,y(kN)	Increase (%)
CB	63.6	-	53.2	-	-	-
BM10H-1	74.1	17%	56.5	6%	72.7	1%
BM10H-2	79	24%	59.9	13%	N.A	N.A
BM12H-1	83.2	31%	64.9	22%	81.7	13%
BM12H-2	83.4	31%	57.8	9%	83	15%

120
ULS
100 FLS
CC
80 YS
YA
60
40
20
0
-1H H-
2 -1H H-
2
M1
0 0 2 2
B BM
1
BM
1 1
BM
Beam ID

Figure 4-53: Bar plot of load comparisons for BM10H and BM12H.

Ductility Index
Load (kN)

4.2.6. Effect of different bolt models at the beams' ends with bonding. This section aims at describing the effect of varying the torque magnitudes, at the ends of the beams, on the structural response of strengthened RC beams using different bolt models with adhesive bonding configurations. The results obtained were compared to the unstrengthened specimens (CB-Specimens) based on the load versus deflection curve shown in Figure 4-54. It was observed that, in terms of strength performance, the curves approximately demonstrate similar profiles; in which BEM10E and BEM12E specimens exhibited higher load capacities with relatively similar or larger ductility when compared to CB. This was quantified by computing the percentage increase for different states, similar to the previous sections. Table 4-21 shows the load increase, in the ultimate and failure states, between the BEM10E and BEM12E specimens. The maximum percentage increase in the ultimate state took place for BEM12E at 34%, whereas the failure state took place form BEM10E at 29%, as shown in Table 4-21. However, due to the small variation in the values, it is difficult to pinpoint which anchorage technique is superior. In this section, the CV was not calculated due to the small number of specimens for this comparison.

Table 4-22 shows the percentage load increase when crushing and yielding of both steel reinforcement and AA plate took place in both BEM10E and BEM12E. For example, the maximum load increase at which crushing occurred was for both BEM12E at 34%, while the maximum load increase at which yielding in the steel reinforcement and the AA plate occurred was for BEM10E at 28% and 11%, respectively. In general, it is observed that the maximum load increase in which crushing of concrete and yielding of both internal and external reinforcements occurred in the specimen. However, this does not warrant a sufficient conclusion about which anchorage configuration is superior to the other. Figure 4-55 graphically summarizes the load percentage increase for each limit state, where it can be observed that the incorporation of HST3 M12, at the beam’s ends, performed slightly better than the bean with HST3 M10 bolted at its ends (BEM10E).

81

100
80
60
40 CB
CBE
20 BEM10E
BEM12E
0
0 10 20 30 40 50
Deflection (mm)

Figure 4-54: Load versus deflection curves for different bolt models at edge of beams.

Table 4-21: Strength increase in the ultimate and failure states for different bolt models at high-non-uniform spacing.

Beam ID	Ultimate State		Failure State	
	Pult(kN)	Increase (%)	Pfail(kN)	Increase (%)
CB	64.2	-	57.7	-
BEM10E	82.6	29	74.3	29
BEM12E	86.1	34	72.1	25

Table 4-22: Strength increase at concrete crushing and yielding for different bolt models at ends of beams.

Beam ID	Pconc,cr(kN)	Increase (%)	Psteel,y(kN)	Increase (%)	PAA,y(kN)	Increase (%)
CB	63.6	-	53.2	-	-	-
BEM10E	80.1	26	68.3	28	80.2	11
BEM12E	85.7	35	67.4	27	78.9	9

4.3. Failure Modes

Another characteristic structural engineers observe is the failure mode of the structural member after ultimate load conditions were achieved. In this project, the failure modes were characterized as: (a) end-debonding/delamination (ED); (b) intermediate debonding/delamination (ID); (c) Plate Rupture (PR); (d) Flexural Failure (FF). In the previous sections, these failure modes were addressed by analyzing the strain measurements in the elements of interest by means of load versus strain curves. However, in this section, the failure modes were identified by means of observing the crack patterns at failure or at a 15% drop in the ultimate load, in cases of FF.

82

Load (kN)
140
120
100
ULS

FLS
80 CC
YS
60 YAA
40
20
0
E E
10 12
BE
M EMB
Beam ID

Figure 4-55: Bar plot of load comparisons.

4.3.1. CB-Specimens. The CB-Specimens group was used as a benchmark to measure the differences between EBR systems versus retrofit systems that are anchored using expansion anchors with/without adhesive bonding (epoxy). The group consisted of an unstrengthened RC beam (CB), which was designed to fail in flexure, and a strengthened RC beam by externally bonding an AA plate to its soffit (CBE), which was designed to fail by debonding/delamination. Figure 4-56 shows the failure modes after the CB-Specimens group failed; whereby CB failed typically in FF, as shown in Figure 4-56a, and CBE failed by ED, as shown in Figure 4-56b.

(a) FF for CB (b) ED failure for CBE

Figure 4-56: Failure modes of CB-Specimens.

Percentage Increase in Load (%)

4.3.2. M10-Specimens. The M10-Specimens group was composed of strength- ened RC beams that were varied by alternating between the spacing and position of HST3 M10 bolts with/without the addition of epoxy. Both BM10H speciemns (BM10H- 1 and BM10H-2) failed by PR while demonstrating obvious cracking patterns, as shown in Figure 4-57; indicating concrete crushing combined with steel yielding. Similarly, the BEM10H and BEM10L specimens also failed by PR while exhibiting crack patterns, as shown in Figure 4-58 and Figure 4-59, whereas BEM10E failed by ID, as shown in Figure 4-60. As a result, all of the strengthened specimens in M10- Specimens group ex- hibited obvious cracking behavior, which indicated the large deflection imposed on the specimen. These physical evidences are in close agreement with the mechanical prop- erties provided by the strain measurements, in the previous sections; whereby the order at which the elements reached their own capacities were expressed based on the time recorded by the element's strain gauge. All M10-Specimens exhibited steel yielding followed by concrete crushing until failure was reached.

4.3.3. M12-Specimens. The M12-Specimens group was also composed of strengthened RC beams that were varied by alternating between the spacing and position. However, these specimens were bolted using HST3 M10 bolts with/without the addition of epoxy. Both BM12H speciemns (BM12H-1 and BM12H-2) failed by PR while demonstrating obvious cracking patterns, as shown in Figure 4-61; indicating concrete crushing combined with steel yielding. Similarly, the BEM12H and BEM12L specimens also failed by PR while exhibiting crack patterns, as shown in Figure 4-62 and Figure 4-63, whereas BEM12E failed by ID, as shown in Figure 4-64. As a result, all of the strengthened specimens in M10-Specimens group exhibited obvious cracking behavior, which indicated the large deflection imposed on the specimen. These physical evidences are in close agreement with the mechanical properties provided by the strain measurements, in the previous sections; whereby the order at which the elements reached their own capacities were expressed based on the time recorded by the element's strain gauge.
All M12-Specimens exhibited steel yielding followed by concrete crushing until failure was reached. Table 4-23 summarizes the failure modes presented in this section where the failure modes were listed in order of.

(a) Side view of BM10H-1's crack pattern (b) PR failure for BM10H-1
(c) Side view of BM10H-2's crack pattern (d) PR failure for BM10H-2
Figure 4-57: Failure modes of BM10H specimens.

Table 4-23: Summary of ultimate load and failure modes.

Beam ID	P_{ult} (kN)	$P_{ult}/P_{ult,CB}$	Failure Modes[1]
CB	64.2	-	SY, CC
CBE	84.4	1.31	SY, CC, ED
BEM10L-1	80	1.25	SY, CC, PR
BEM10L-2	85.8	1.34	SY, CC, PR
BEM10H-1	80.5	1.25	SY, CC, PR
BEM10H-2	86.5	1.35	SY, CC, PR
BM10H-1	74.4	1.16	SY, CC, PR
BM10H-2	79.5	1.24	SY, CC, PR
BEM10E	82.6	1.29	SY, CC, ID
BEM12L-1	78.1	1.22	SY, CC, PR
BEM12L-2	86.4	1.35	SY, CC, PR
BEM12H-1	82.2	1.28	SY, CC, PR
BEM12H-2	86.2	1.34	SY, CC, PR
BM12H-1	83.5	1.30	SY, CC, PR

BM12H-2 83.8 1.31 SY, CC, PR
BEM12E 86.1 1.34 SY, CC, ID
1 S.Y: Steel yielding; C.C: Concrete Crushing
85
(a) Side view of BEM10H-1’s crack pattern (b) PR failure for BEM10H-1
(c) Side view of BEM10H-2’s crack pattern (d) PR failure for BEM10H-2
Figure 4-58: Failure modes of BEM10H specimens.

4.4. Ductility Index

In addition to monitoring the ultimate loading capacity of each specimen, the strength enhancement was characterized by measuring the ductility of each beam. This was performed by evaluating the ductility index of each specimen in which two equations were used for this purpose; namely, the ratio of the deflection at ultimate load to the deflection at yield and the deflection at failure load to the deflection at yield. However, in this project, there are two yield points experienced by the strengthened beams - mainly deflection in which steel yielded and deflection at which the AA plate yielded.
The modulus of elasticity in the AA plate is far less than the modulus of elasticity in the steel reinforcement; whereby, the flexural stiffness of the RC beams’ is mainly originating from their steel reinforcements. Therefore, the deflection at steel yielding was used when computing ductility index. Equation 2 and Equation 3 were used to compute
86
(a) Side view of BEM10L-1’s crack pattern (b) PR failure for BEM10L-1
(c) Side view of BEM10L-2’s crack pattern (d) PR failure for BEM10L-2
Figure 4-59: Failure modes of BEM10L specimens.
(a) Side view of BEM10E’s crack pattern (b) ID failure for BEM10E
Figure 4-60: Failure modes of BEM10E.
87
(a) Side view of BM12H-1’s crack pattern (b) PR failure for BM12H-1
(c) Side view of BM12H-2’s crack pattern (d) PR failure for BM12H-2
Figure 4-61: Failure modes of BM12H specimens.

the ductility indices, where $\mu_{\Delta,ult}$ and $\mu_{\Delta,fail}$ represent the ductility index at ultimate and failure conditions, respectively. Table 4-24 summarizes the ductility index at failure and ultimate conditions, respectively, for each specimen. It can be observed that most of the specimens exhibited positive increase in ductility in which the negative values are almost within a 10% margin.
The ductility achieved by the M10-Specimens surpassed the ductility of the CBE specimen; whereby the CBE exhibited the lowest ductility index as shown in Figure 4-

65. Furthermore, the BEM10H specimens experienced larger ductility index values during failure than both the BEM10L and BEM10E specimens. This extra deformation allowed the specimens in the M10-Specimens group to delay the loading process and distribute the internal forces to each element; concrete, steel and AA plate. Similarly,

(a) Side view of BEM12H-1's crack pattern (b) PR failure for BEM12H-1

(c) Side view of BEM12H-2's crack pattern (d) PR failure for BEM12H-2

Figure 4-62: Failure modes of BEM12H specimens.

the ductility index values for strengthened specimens in the M12-Specimens group surpassed that of the CBE specimen, as shown in Figure 4-66. Some of the specimens in the M12-Specimens group managed to exhibit more ductility than the un-strengthened RC beam (CB); like BEM12H-2 and BEM12E. This extra deformation could be the result of the larger magnitude imposed by the HST3 M12 bolts; however, further testing is required to warrant such a conclusion.

$$\mu_{\Delta,ult} = \frac{\delta_{ult}}{\delta_{steel,y}} \quad (2)$$

$$\mu_{\Delta,fail} = \frac{\delta_{fail}}{\delta_{steel,y}} \quad (3)$$

(a) Side view of BEM12L-1's crack pattern (b) PR failure for BEM12L-1

(c) Side view of BEM12L-2's crack pattern (d) PR failure for BEM12L-2

Figure 4-63: Failure modes of BEM12L specimens.

(a) Side view of BEM12E's crack pattern (b) ID failure for BEM12E

Figure 4-64: Failure modes of BEM12E.

Table 4-24: Summary of deflections and ductility indices for each specimen.

Beam ID	δ_{ult} (mm)	δ_{fail} (mm)	$\delta_{steel,y}$ (mm)1	$\mu_{\Delta,ult}$	% Change CBE (%)	$\mu_{\Delta,fail}$	% Change CBE (%)
CB	17.4	30.5	6.77	2.57	-	4.51	-
CBE	14.3	19.94	6.62	2.16	0	3.01	0
BEM10L-1	16	25	N.A	N.A	N.A	N.A	N.A
BEM10L-2	17.1	24.4	7.06	2.42	12.04	3.46	15
BEM10H-1	15.6	29.4	8.23	1.9	-12.04	3.57	18.6
BEM10H-2	18.5	26.5	7.1	2.61	20.8	3.73	23.9
BM10H-1	16.4	32.7	6.9	2.38	10.2	4.74	57.5
BM10H-2	22	25.9	7.45	2.95	36.6	3.48	15.6
BEM10E	14.9	24.8	7.09	2.1	-2.78	3.5	16.3
BEM12L-1	16.1	24.6	6.81	2.36	9.26	3.61	19.9
BEM12L-2	16.5	25	6.69	2.47	14.4	3.74	24.3
BEM12H-1	18.3	23.6	8.96	2.04	-5.56	2.63	-12.6
BEM12H-2	19.1	31.3	6.9	2.77	28.2	4.54	50.8
BM12H-1	19.5	26.1	7.17	2.72	25.9	3.64	20.9
BM12H-2	20.1	28.7	5.95	3.38	56.5	4.82	60.1
BEM12E	21	41	7.4	2.84	31.5	5.54	84.1

1 N.A: Not available due to damage in equipment
8
,ult
,fail
6
4
2
0
CB BE -2 1 2 1 2C 0L H
- H- - - 0
E
1 10 10 10
H
10
H 1
EM E
M
B BE
M EMB B
M BM B
Beam ID
Figure 4-65: Bar plot of ductility index for M10-Specimens group.
8 ,ult
,fail
6
4
2
0
CB BE -1L L-
2
H-
1 -2 1 2 E
C 12 12 H H
- H- 12
M M M1
2
M1
2 12 12 EM
BE BE BE BE B
M BM B
Beam ID
Figure 4-66: Bar plot of ductility index for M12-Specimens group

Ductility Index Ductility Index

Chapter 5. Nonlinear Finite Element Modelling

An important step required to gain a stronger understanding on how structural members behave, is to reproduce the experimental results using a commercial finite element (FE) software. This chapter aims at developing 3D nonlinear FE models using Mechanical ANSYS APDL [41]. By adopting approaches conducted in previous studies

[82,83], an accurate model can be developed with the nonlinear properties of the tested specimens coupled with a simplified geometry. Afterwards, the load-stiffness response plots were extracted from the FE models and compared with the experimental results to help validate the FE models. These validated FE models were used to generate contour plots that express the stress and strain propagation in each individual element. Finally, a comprehensive summary of the results was developed to conclude this approach.

5.1. Geometry of FE Models

The FE models were developed to accurately resemble the geometric configuration and dimensions of the tested specimens. Owing to the symmetry in the cross section and span, as shown in Figure 3-7 - Figure 3-9, a quarter of the model was created by restraining longitudinal and transverse translations of the beams, as shown in Figure 5-67. This helped simplify the analysis and reduce excessive computation time periods. The steel reinforcing bars were modelled using 3D spar elements, whereas the rest of the elements were modelled using 3D solid elements. Further emphasis regarding the element description will be discussed in the following sections.

Axis of Symmetry
Rollers Rollers
240 mm
920 mm 62.5 mm

Figure 5-67: Quarter model of RC beam

5.2. Element Types and Material Properties

In general, a total of six elements were used to model the specimens in this project: (a) SOLID65 for concrete; (b) SOLID185 for loading and supporting plates,

epoxy and AA plates; (c) LINK180 for steel reinforcements; (d) INTER205 for interfacial cohesive bond between AA plate and concrete; (E) CONTA174 and TARGE170 for simulating hard contact between AA plate and concrete without epoxy. The properties and characteristics of each element will be discussed in the following subsections.

5.2.1. SOLID65. The concrete beam was modelled using SOLID65 elements to simulate cracking and compression when subjected to bending [84, 85]. The SOLID65 element, shown in Figure 5-68, consists of eight nodes having three degrees of freedom, per node, in which translations are permitted in the nodal x, y, and z directions. The assumptions and restrictions of the element are listed below [41]:

- Cracking is enabled in each orthogonal directions at the element's integration points.
- Upon the occurrence of concrete cracks, the elements are re-structured such that the material properties are tweaked to simulate hardening and softening behaviors in concrete.
- The concrete material is assumed to be initially isotropic.
- The reinforcements embedded within the concrete are assumed to be smeared throughout its elements.

Figure 5-68: Geometry of SOLID65 element [41].

In order to simulate the nonlinear effect of concrete, the tensile stresses and compressive stresses, in concrete, were incorporated using previously derived constitutive models. This granted the concrete model the capabilities of exhibiting both strain hardening and softening during each loading frame; thereby allowing the concrete elements to dissipate energy and experience stiffness decay after reaching their maximum tensile and compressive stresses [41]. In this study, the Hognestad Parabola was implemented to incorporate concrete compression and the William and Wranke model was implemented to simulate tensile cracking [84, 85].

Equation 4 was used to build the compressive stress-strain profile of the concrete elements, where f_c is the compressive stress, f'_{cc} is the average cylinder compressive strength from Table 3-4 (taken as 37 MPa), ε_c is the concrete strain ratio, $\varepsilon_{co} = \frac{2f'_{cc}}{E_c}$ is the strain corresponding to f'_{cc}, ε_{cu} is the crushing strain (taken as 0.0038 [82]) and E_c is the modulus of elasticity in concrete. The Young's Modulus of Elasticity in concrete (E_c) was evaluated using the ACI 318-14 standard [75] as shown in Equation 5. Figure 5-69a shows the compressive stress response that was employed within the SOLID65 elements.

$$f_c = f'_{cc}\left[\frac{2\varepsilon_c}{\varepsilon_{co}} - \left(\frac{\varepsilon_c}{\varepsilon_{co}}\right)^2\right], \text{ for } 0 < \varepsilon_c \leq \varepsilon_{cu} \quad (4)$$

$$E_c = 4700\sqrt{f'_c} \quad (5)$$

The tensile behavior of concrete was modelled using five strength parameters imposed by the William and Wranke model [85]: uniaxial tensile strength ft = 0.62 f ′cc , uniaxial compressive strength (f ′cc), biaxial compressive strength (fcb), compressive strength for a state of biaxial compression superimposed on hydrostatic stress state (f1), and uniaxial compression superimposed on hydrostatic stress state (f2). The uniaxial compressive strength was obtained from experimental testing, and the last three parameters (fcb, f1 and f2) were taken as their default values, 1.2f ′ , 1.45f ′ and 1.725f ′cc cc cc, respectively. In addition, the open and closed shear coefficients were employed to suc-

cessfully measure the amount of energy dissipated when the SOLID65 elements begin to crack. Their values were taken as 0.2 and 0.5, respectively [82, 83, 86].

The tensile stress-strain relationship was constructed using Equation 6, where Figure 5-69b shows the tensile stress response of the SOLID65 elements. The tensile response in concrete was modeled as linear-elastic until the concrete tensile strength (ft) was reached. Afterwards, a relaxation in the tensile stress is exhibited by a 40% drop in the concrete tensile stress, followed by an inversely linear decay until a tensile stress value of zero was reached at a strain value greater than or equal equal to 6 times the strain value corresponding to maximum concretes tensile strength (εto) [41, 83].

Both constitutive models represent an idealized form of concrete, in both compression and tension, such that these adopted models were capable of approximating the non-linearity of concrete while accelerating convergence during the analysis.

ft, if 0< εto < εt

ft = 0.6ft, if εto = εt (6)

−0.6ft6 − , if εt < εto ≤ 6εt

40 4

f ′c ft

30 3

40% Sudden Relaxation

20 2

10 1

6εt
εt
0 0
0 1 2 εco 3 εcu 4 0 2 4 6 8
Strain (mm/mm) ×10-3 Strain (mm/mm) 10-4
(a) Compressive stress strain curve (b) Tensile stress-strain curve

Figure 5-69: Idealized stress-strain curves for concrete.

5.2.2. LINK180. The steel reinforcing bars were modelled as LINK180 3D spar elements to simulate the flexural response of the internal reinforcement. The LINK180

Stress (MPa)
Stress (MPa)

element functions as a two-node uniaxial tension/compression member with three degrees of freedom, per node, in which translations are permitted in the three directions(x, y, and z). The limitation and restrictions imposed by LINK180 element are [41]:

1. The spar element resembles a truss bar that is axially loaded, at its ends, and consists of both section and material property definitions.
2. The displacement shape function implies a uniform stress in the spar.

The material definitions that were used to model the mechanical behavior of the steel reinforcements were linear and nonlinear definitions. The linear properties were defined an isotropic definition with a Young’s Modulus of 200 GPa and a poisson ratio of 0.3. The nonlinear properties were defined using a Kinematic Hardening Plasticity definition that incorporated a Bilinear approximation of the tensile stress-strain curve of the steel reinforcement; thus, yielding an elastic perfectly plastic curve, based on the von misses yield criteria [41], as shown in Figure 5-71. The yield stress was obtained from the tensile test in which an average value of 550 MPa was used.

Figure 5-70: Geometry of LINK180 element [41].

5.2.3. SOLID185. The loading and supporting plates, AA plates, bolts and epoxy were modelled as SOLID185 due to its plasticity and stress stiffening capabilities. The SOLID185 element is defined as an 8 node solid with three degrees of freedom, per node, in which translation is permitted in all three directions (x, y, and z), as shown in Figure 5-72. The loading and supporting plates were modelled using only an elastic-isotropic properties in which the Young’s modulus of elasticity was 200 GPa

and the poisson ratio was 0.3. This allowed the plates to demonstrate rigidity during
96
600
500
400
300
200
100
0
0 0.002 0.004 0.006 0.008 0.01
Strain (mm/mm)
Figure 5-71: ElasticPerfectly plastic stress-strain curve
loading. However, the AA plate and bolts were modelled using both an elastic-isotropic property and a Kinematic Hardening Plasticity with a Bi-linear definition. The Young's modulus of elasticity and poisson ratio for the AA plate was 50000 MPa and 0.33, respectively, whereas the modulus of elasticity and poisson ratio for the bolts were 200 GPa and 0.3, respectively. In addition, the yield stresses of the AA plates and bolts were 150 and 800 MPa, respectively, as shown in Figure 5-73. Finally, the Young's modulus of elasticity and poisson ratio of the epoxy was taken as 10000 MPa and 0.35, similar to a study conducted by Abu-Obeidah et. al [82].

5.2.4. INTER205. One of the difficulties researchers face, when modelling strengthened RC members, is the simulation of the adhesive interface that links the composite material and its adjacent host during the analysis. Several numerical studies were conducted in FE applications focused primarily on retrofit applications [82, 86] in which fracture or delamination, along the composite material, played a major role in limiting the stiffness and ductility of the strengthened structure. In this study, the epoxy was modelled using INTER205 elements where a cohesive zone material (CZM) model was employed to incorporate this interfacial bond [41]. INTER205 is a shell element consisting of eight nodes defined with three degrees of freedom, per node, in
97
Stress (MPa)
Figure 5-72: Geometry of SOLID185 element [41].
200 1000
800
150
600
100
400

50 200
0
0
0 0.002 0.004 0.006 0.008 0.01 0 0.005 0.01
Strain (mm/mm) Strain (mm/mm)
(a) AA plate (b) Bolts
Figure 5-73: Idealized tensile stress-strain curves.

which translation is permitted in all three directions (x, y, and z directions), as shown in Figure 5-74.

The CZM model is a function of the traction and the slip between the strengthening material and its host. It requires a bond stress-slip model which induces fracture mechanisms, within the INTER205 elements, leading to softening followed by a release

98

Stress (MPa)
Stress (MPa)

Figure 5-74: Geometry of INTER205 element [41].

of fracture energy release from within these elements. In this project, the bond stress-slip relationship presented by Lu et al. [87] was adopted using Equation 7, in which a curve was generated, as shown in Figure 5-75. The curve demonstrates an increase in shear stress with the increase in slip, whereby hardening is exhibited until a maximum shear stress is reached, s0. Afterwards, the curve shows an exponential decay simulating the instantaneous loss in shear strength combined with simultaneous debonding of the elements until a failure slip is reached. The failure slip was assumed to equal four times the value of s0.

□□□□□□ √τ smax s (,) if s≤ s0
τ =□□□□□
0
(7)
−α SS −1
τ 0maxe , if s > s0

where
√
=√
√√ b
2.25− f
β bcw b
1.25+ fbc
$\tau_{max} = 1.5\beta_w f_t$

99

$s_0 = 0.0195\beta_w^2 f_t$
√
$G_f = 0.308\beta_w^2 \sqrt{f_t}$
α = 1Gf 2

τmaxS
–
0 3

where β_w is the width ratio factor, b_f = width of aluminum, b_c = is the width of the concrete (mm), τ_{max} is the maximum local bond shear stress (MPa), so is the local slip corresponding to τ_{max} (mm), G_f is the interfacial fracture energy (MPa), s is the local slip within the interface (mm), α is a factor that depends on interfacial fracture energy, shear stress, and slip at τ_{max} plate (mm).

6
τmax
5

4

3
2

1
S0 4S0
0
0 0.05 0.1 0.15 0.2 0.25 0.3 0.35 0.40
Slip (mm)

Figure 5-75: Bond stress-slip model at the interface between the aluminum and concrete elements.

5.2.5. TARGE170 and CONTA174. The BM10H and BM12H specimens were plated by fixing AA plates to RC members using different bolt models; thereby, mechanically fastening the plate onto a hard surface without the presence of any additional adhesive compound. This type of interaction was assumed to be in the form of a hard-contact with the presence of friction between the two surfaces of the adjacent structural solid elements. In ANSYS, this was simulated by assigning CONTA174 elements to the surfaces of the concrete elements and TARGE170 elements to the surfaces of the Aluminum elements. The CONTA174 elements are responsible for simulating both contact

Shear Stress (MPa)

and sliding interactions between the TARGE170 elements, as shown in Figure 5-76. The target-contact interface was defined using a pair-base contact argument in which both elements were assumed to behave in a flexbile-flexible contact with a coefficient of friction value of 0.3.

Figure 5-76: TARGE170 and CONTA174 surfaces [41].

5.3. Convergence Criterion

During displacement controlled loading, ANSYS automatically treats each user-defined displacement as a unit-step to evaluate the nodal stresses and strains within the element. The numerical solver used to help the model achieve convergence was the Newton-Raphson method where the solver iteratively reduces the time-step until a solution is found. Afterwards, it iterates to the next step where the numerical solver begins evaluating the problem until convergence is achieved. However, ANSYS requires a user-defined convergence tolerance to abide by; typically, this value would range between 0.05-0.2 [83]. Therefore, in this study, the force convergence tolerance limit value was 0.1.

5.4. Failure Criteria

The analysis of each model was stopped based on the type of failure exhibited. In this study, the failure modes demonstrated during testing were: concrete crushing (CC), plate rupture (PR), end-debonding (ED), and intermediate-crack debonding (IC). These failure modes can be numerically distinguished by monitoring the stress and strain propagation of each element. During the tests, all of the beams exhibited crushing prior their unique failure modes. This helped define a criteria for which the analysis should stop until crushing and one of the three latter imposed failure modes. These three failure modes were detected based on:

1. Third principal strain contour in the concrete in which the range of crushing was between 0.003-0.0043.
2. First principal stress contour in the AA plates at 150 MPa, as defined in Figure 5-71.
3. Shear stress contour in the epoxy in which debonding occured at shear stress values of 5.8-6.1 MPa.

5.5. Modelling Techniques

As mentioned previously, the FE models were simulated by taking a quarter of the model and restraining any translation normal to the longitudinal and transverse symmetry plane. All elements were meshed such that the nodes between each unique element coincides with one another. This granted ideal load transfer across the nodes and greatly reduced computation complexity. An FE model of the unstrengthened specimen, CB, was modelled such that the concrete, steel reinforcement, and plates were meshed and merged together as shown in Figure 5-77 and Figure 5-78. The mesh size was selected based on the aspect ratio of the element (≤ 2) while satisfying the nodal coincidence between each adjacent elements. Therefore, the longitudinal and vertical lengths were meshed at 10 mm per segment while the transverse length was meshed at 5 mm per segment, as shown in Figure 5-77. The structural integrity of the FE model resembles that of a statically determinant beam; such that the external load coming from the plate will induce an equal and opposite reaction force on the support. Afterwards,

the simulation of the other strengthened beams consisted of this particular FE model combined with other 3D solid elements, some of which included: the epoxy, bolt and contact-target elements.

Transverse Restraints
Longitudinal Restraints
Supporting Plate with
Vertical Restraints
Concrete Beam

Figure 5-77: FE meshed geometry of CB specimen.

Top Steel Bar (†8 mm)
Stirrups (†8 mm)
Bottom Steel Bar (†10 mm)

Figure 5-78: FE meshed geometry of steel reinforcement.

The block volume command was used to model the epoxy, with a thickness of 2 mm, and the AA plate, with a thickness of 3 mm, whereby careful steps were taken to successfully simulate debonding/delamination between the AA plate and the concrete surface. This was accomplished by meshing the epoxy into PLATE185 elements while splitting the elements into top and bottom epoxy layers, as shown in Figure 6-146. INTER205 shell elements were assigned between the two adjacent layers using the CZMESH command whereby the CZM model, discussed in the previous section, was incorporated within the assigned elements as shown in Figure 6-146. It is worth mentioning that the addition of INTER205 shell elements, between the epoxy layers, did not provide any additional stiffness or contribute to an increase in the moment-arm within the section, due to having no definite thickness.

Top Epoxy Layer Bottom Epoxy Layer
INTER205 Shell Elements

Figure 5-79: Top and bottom meshed layers of epoxy.

Since the thickness of the AA plate is relatively small compared to the size of the elements, the elements were segmented in the longitudinal and transverse directions, as shown in Figure 5-80. Afterwards the nodes residing on the top epoxy layer was merged onto the nodes of the bottom concrete surface, while the the nodes residing on the bottom epoxy layer was merged onto the nodes on the top surface of the AA plate. This helped relieve any computation issues regarding the adhesive interaction between the epoxy layers and their adjacent solid elements while focusing mainly on the cohesive definitions in the center of the epoxy layers.

(a) Front view (b) 3D Isometric view

Figure 5-80: AA Plate Meshed Elements.

On the other hand, the M10-Specimens and M12-Specimens groups were modelled differently due to the presence of bolts with/without epoxy. The bolts were modelled as semi-circular cylinders, for the bolts away from the center of the beam, and quarter-circular cylinder, for the bolts exactly in the middle of the beam span, because of the symmetry in the longitudinal and transverse planes, as shown in Figure 5-81.

For the specimens that were composed of bolts without epoxy (BM10H and BM12H), they were modelled using contact-target elements such that the shear strength induced by the friction between the surfaces of the AA plate and concrete would contribute to the analysis of the FE models. Figure 5-82 shows the CONTA174 and TARGE170 assignments on the adjacent 3-D solid elements. Since the INTER205 elements require a certain type of mesh, involving the CZM argument, the remaining specimens that were plated using both bolts and epoxy were unable to be simulated due to the presence of the PLATE185 elements (bolts) within the zone of the INTER205 elements. Therefore, a 20 mm gap was left such that the bolts were able to be merged within the concrete and AA plate while not interfering with the CZM command assigned within the INTER205 shell elements. Figure 5-83 shows the different modelling approaches taken when constructing the plated specimens; whereby the epoxy (shown in purple), the AA plate (shown in blue) and the bolts (shown in red) were created such that the locations of the bolts and the epoxy-gaps resembled the geometry of the tested specimens.

5.6. Load Versus Deflection Curves

Several studies were focused on developing FE models, consisting of idealized material model definitions, whereby load versus deflection curves were generated to be compared and validated with those of the experiment [60, 83, 86]. In this section, the

Semi-Circular Cylinders
Quarter-Circular Cylinder

Figure 5-81: Bolt elements shapes for a strengthened specimen with high number of bolts.

TARGE170 Elements (AA Plate)
CONTA174 Elements (Concrete)

Figure 5-82: CONTA174 and TARGE170 element assignments.

load versus deflection curves produced by ANSYS were plotted with the experimental results to gain insight on how accurate the FE models are in simulating the flexural be-

Bolt Locations
Epoxy
AA Plate

(a) BEM10H/BEM12H

Bolt Locations
Epoxy
AA Plate

(b) BEM10L/BEM12L

Bolt Locations
Epoxy
AA Plate

(c) BEM10E/BEM12E

Figure 5-83: Epoxy and Bolt modelling for Plated Specimens.

havior of the tested specimens. It is worth mentioning that the softening stage of the experimental results were cropped out due to the nonlinear elastic definition imposed on the SOLID65 elements. Therefore, the predicted and experimental load versus deflection curves, for each specimen, were plotted until the maximum load was achieved.

5.6.1. CB-Specimens. The load and deflection values were obtained from the FE models, which resembled the beams in the CB-Specimens group, and were compared to the results obtained from the experiment. Figure 5-84 shows the load versus deflection curves of the unstrengthened RC beam, CB, and the strengthened beam that was externally bonded with an AA plate, CBE. In general, both curves demonstrate small deviations between curves of the FE models and the curves of the experiments. However, it was observed that both of the FE-produced curves demonstrated higher stiffness, slopes, than that of the experiment; whereby the cause of this phenomenon originated from the idealized assumption that the internal reinforcements are completely bonded to the nodes of the concrete elements. This prevented any bond-slip action between the bars and the concrete; thus, forcing the steel bars to fully dissipate energy in the form of axial deformations coupled with bending rather than the combination of both with the induced shear action resulting from the bar-slip. Moreover, the peak load and corresponding deflection values demonstrated by the FE models for the CB and CBE specimens were 60.2 kN at 17 mm and 84 kN at 13.8 mm, respectively, whereas peak load and deflection values obtained from the experiment for the CB and CBE specimens were 64.2 kN at 17.4 mm and 84.4 kN at 14.3 mm, respectively.

5.6.2. M10-Specimens. Similarly, the load versus deflection curves were generated using the FE models, resembling the beams in the M10-Specimens group, and were plotted against the curves measured during the test. Figure 5-85 shows the load versus deflection curves for the plated specimens fixed with HST3 M10 bolts in which the varying parameters were the addition of epoxy, the bolt spacing and layout. As a result, the FE models termed BEM10H, BEM10L, BM10H and BEM10E were capable of predicting peak load and corresponding deflection values of 80.9 kN at 15.1 mm, 82.7 kN at 16 mm, 72.8 kN at 16.2 mm and 83.8 kN at 13.9 mm, respectively, while the same experimental specimens yielded peak load and deflection values of 83.5 kN at 15.5 mm,

80 100
80
60
60
40
CB Exp 40
CB FE CBE Exp
20 CBE FE
20
0 0
0 5 10 15 20 0 5 10 15
Deflection (mm) Deflection (mm)
(a) CB (b) CBE

Figure 5-84: Load versus deflection curves between experimental and FE results for CB-Specimens.

82.9 kN at 16.5 mm, 76.9 kN at 17.1 mm and 82.6 kN at 14.9 mm, respectively. It is worth mentioning that most of the plated specimens within the M10-Specimens group were replicated twice; therefore, the load and deflection values were averaged and used as a benchmark to compare against the results predicted by the FE model.

5.6.3. M12-Specimens. Finally, the load versus deflection curves were generated using the FE models, resembling the beams in the M12-Specimens group, and were plotted against the curves measured during the test. Figure 5-86 shows the load versus deflection curves for the plated specimens fixed with HST3 M12 bolts in which the varying parameters were the addition of epoxy, the bolt spacing and layout. As a result, the FE models termed BEM12H, BEM12L, BM12H and BEM12E were capable of predicting peak load and corresponding deflection values of 82.2 kN at 17.7 mm, 82.6 kN at 15.6 mm, 79.6 kN at 19.9 mm and 88.9 kN at 19.7 mm, respectively, while the same experimental specimens yielded peak load and deflection values of 84.2 kN at

18.7 mm, 82.2 kN at 16.3 mm, 83.6 kN at 19.9 mm and 86.1 kN at 21 mm, respectively.

Similar to the previous section, the mean values of the load and deflection results used as a benchmark to compare against the results predicted by the FE model.

5.6.4. Summary of results. The FE models were capable of predicting the load and deflection values within a reasonable range of the experiment. Table 5-25 shows a table that outlines the peak load and corresponding deflection values for the specimen

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Load (kN)
Load (kN)
100 100
80 80
60 60
BEM10H-1 Exp BEM10L-1 Exp
40 BEM10H-2 Exp 40 BEM10L-2 Exp
BEM10H FE BEM10L FE
20 20
0 0
0 5 10 15 20 25 0 5 10 15 20 25
Deflection (mm) Deflection (mm)
(a) BEM10H (b) BEM10L
100 100
80 80
60 60
BM10H-1 Exp BEM10E Exp
40 BM10H-2 Exp 40 BEM10E FE
BM10H FE
20 20
0 0
0 5 10 15 20 25 0 5 10 15 20 25
Deflection (mm) Deflection (mm)
(c) BM10H (d) BEM10E

Figure 5-85: Load versus deflection curves between experimental and FE results for M10-Specimens.

and corresponding FE model. It was observed that the FE software was capable of estimating load and deflection for most of the specimens, of which, the absolute percentage difference in the predictions were below 5%.

5.7. Predicted Contour Plots and Cracking Patterns

After validating the stiffness response of the previous FE models, contour plots demonstrating the stress, strain and concrete crack propagation were generated and discussed during this section. The contour plots that were selected were the 1st Principal stress, 3rd Principal strain, shear stress on the X–Z plane, and deflection plots. These

contour plots were used to assess the flexural behavior and ductility of each beam. The
110
Load (kN) Load (kN)
Load (kN) Load (kN)
100 100
80 80
60 60
BEM12H-1 Exp
BEM12L-1 Exp
BEM12H-2 Exp
40 BEM12H FE 40 BEM12L-2 Exp
BEM12L FE
20 20
0 0
0 5 10 15 20 25 0 5 10 15 20 25
Deflection (mm) Deflection (mm)
(a) BEM12H (b) BEM12L
100 100
80 80
60 60
BM12H-1 Exp BEM12E Exp
40 BM12H-2 Exp 40 BEM12E FE
BM12H FE
20 20
0 0
0 5 10 15 20 25 0 5 10 15 20 25
Deflection (mm) Deflection (mm)
(c) BM12H (d) BEM12E

Figure 5-86: Load versus deflection curves between experimental and FE results for M12-Specimens.

Table 5-25: Comparison between the FE predicted and experimental measured results

Specimen	FE Model	Failure Load (kN)		% Difference	Maximum Deflection (mm)		% Difference
		Experimental	FE		Experimental	FE	
CB	FE CB	64.2	60.2	6.23	17.4	17	2.30
CBE	FE CBE	84.4	84	0.47	14.3	13.8	3.50
BEM10H	FE BEM10H	83.5	80.9	3.11	15.5	15.1	2.58
BEM10L	FE BEM10L	82.9	82.7	0.24	16.5	16	3.03
BM10H	FE BM10H	76.9	72.8	5.33	17.1	16.2	5.26
BEM10E	FE BEM10E	82.6	83.8	1.45	14.9	13.9	6.71
BEM12H	FE BEM12H	84.2	82.2	2.38	18.7	17.7	5.35
BEM12L	FE BEM12L	82.2	82.6	0.49	16.3	15.6	4.29
BM12H	FE BM12H	83.6	79.6	4.78	19.9	19.9	0.00
BEM12E	FE BEM12E	86.1	88.9	3.25	21	19.7	6.19

111
Load (kN) Load (kN)
Load (kN) Load (kN)

concrete crack patterns were reproduced by switching on the cracking capabilities of the SOLID65 elements and allowing the concrete elements to exhibit crack/crushing in the form of colored outlines. In general, ANSYS is capable of demonstrating the first, second and third cracks such that the order in which the cracks were induced followed a certain color scheme. The predicted cracks are located at the elements’ integration points; whereby the first crack was denoted with a red circle outline, the second crack with a green outline, and the third crack with a blue outline [41]. Owing to the large number of figures that were generated during this analysis, three of the presented FE models were selected within this section, whereas the rest of the contour plots were appended at the end of the paper—labelled Appendix A through Appendix E.

5.7.1. CB. The un-strengthened RC beam model, CB, was selected as a reference model—to study the effects of implementing bolts and epoxy on the stress propagation and crack patterns. Figure 6-100 shows the predicted nodal deflection contour plots where it can be observed that the boundary conditions and loads, imposed on the model, are helping the model simulate bending. Moreover, the compression behavior of the concrete elements was assessed using the 3rd principal strain contour plot, as shown in Figure 5-88, in which the FE model roughly achieved the ultimate cylindrical compressive strength at an average strain of -0.00335 at only the top 10–15 mm concrete layer, whereas the rest of the layers exhibited minimum tensile strains. The observed maximum compressive and minimum tensile stress values justify the constitutive models used during model development [84,85]. Refer to Appendix A through Appendix D to view the contour plots for specimen CBE.

Figure 5-89 shows the predicted crack pattern of CB where the beam exhibited sequential cracking during the FE analysis. It was observed that most of the initiated cracks were represented as first cracks, colored red, where shear and flexural cracks coupled with crushing were plotted. This phenomenon is typical for RC beams that are designed against shear; hence, fail in flexure. Afterwards, the second crack, colored green, developed but less frequently followed by a third crack, colored blue. Refer to Appendix E to view the crack pattern of specimen CBE.

1NODAL SOLUTION

-17.0011

-14.9724
-12.9437
-10.9151
-8.88638
-6.85771
MX
-4.82904
MN
-2.80037
-.771696
1.25698
Figure 5-87: Predicted nodal vertical deflection for CB model.

-.006034
-.005363
-.004693
-.004023
MX
-.003352
-.002682
MN
-.002011
-.001341
1 -.670E-03
CRACKS AND CRUSHING
STEP=.12492E-09
SUB =24
TIME=14.1988
Figure 5-88: Predicted nodal third principal strain for CB model.

Figure 5-89: Predicted crack patterns of the CB model.

5.7.2. BM10H. One of the specimens from the M10-Specimens group was selected, BM10H, to observe the effect of using only bolts to fix the AA plate to the RC beam. Figure 5-90 shows the predicted nodal deflection contour plots. Figure 5-91 shows the 1st principal stress contour plot of the bolted AA plate, where the maximum tensile stress occurred on the bolt at the end of the plate, 571.8 MPa, in which the bolt was resisting the tendency of the AA plate from shearing and debonding/delaminating off the concrete surface. In addition, the portion of the plate that is between the first 3 bolts, at mid-span of the beam, was subjected to a tensile stress concentration of 175.7

MPa. This gave room to support the justification mentioned in the experimental results section in which the implementation of a large number of bolts, without using epoxy,

limited the contribution of the plate to only 200 mm, which is the distance between the
first and third bolt located at mid-span of the model. Afterwards, a 37.5% stress decay
was exhibited at the rest of the bolts until the plate began resisting compressive stress
1values of -22.3 MPa, near the last three bolts.
-21.536
-18.9711
-16.4062
-13.8414
-11.2765
-8.71159
MX
-6.14671
-3.58183 MN
-1.01695
1.54794
Figure 5-90: Predicted nodal vertical deflection for BM10H model.
1

-22.301
43.7075 MMNX
109.716
175.725
241.733
307.742
373.75
439.759
505.767
Stress Concentration at Bolt Locations (Leff). 571.776
Figure 5-91: Predicted nodal first principal stress for BM10H model.

Figure 5-92 shows the shear stress contour plot in the X-Z plane for the bolts
at the end of the plate. The reason for monitoring the shear stress propagation of the
bolts located at the end of the plate is due to shearing-effect induced by the debond-
ing/delamination phenomenon. It was observed that large shear stress was subjected
at the last three bolts in which the maximum shear stress, 87.04 MPa, occurred at the
third bolt from the edge of the plate, as shown in Figure 5-92. Since premature fail-
ure was avoided, the ultimate cylindrical compressive strength of the concrete elements
was achieved by reaching an average strain value of 0.00466, as shown in Figure 5-93.
Figure 5-94 shows the predicted crack pattern of BM10H where the beam exhibited
sequential cracking, similar to the CB model. Therefore, the incorporation of bolts as

an anchorage system allowed the beam to gain stiffness and ductility while still demon-
strating crushing, similar to a typical RC beam. Refer to Appendix A through Appendix
E to view the contour plots and crack patterns of the rest of the plated specimens in the
M10-Specimens group.
1
-188.647
-158.015
-127.382 End of Plate.
-96.7499
-66.1175
-35.485 MX
-4.85258
25.7799
56.4123
87.0447 MN

Figure 5-92: Predicted nodal shear stress for BM10H model.

5.7.3. BEM12E. The last FE model selected was the BEM12E model from
the M12-Specimens group. Figure 5-95 shows the predicted nodal deflection contour
plots where it can be observed that the model demonstrated an intermediate debonding

1
-.041943
-.037283
-.032622
-.027962
-.023302
-.018641
MX
-.013981
MN
-.009321
1 -.00466
CRACKS AND CRUSHING
STEP=.23705E-08 MAR 23 2019
SUB =24 20:06:29
TIME=32.757
Figure 5-93: Predicted nodal third principal strain for BM10H model.

Figure 5-94: Predicted crack patterns of the BM10H model.
failure, similar to the experimental failure mode. Furthermore, the 1st principal stress

contour plot of the bolted AA plate was extracted as shown in Figure 5-96 where the

maximum tensile stress occurred throughout most of the moment region, approximately
425 mm from the mid-span of the beam, followed by a decay in the tensile stress until
the edge of the plate. Moreover, the bolts in this model were subjected to less tensile
stress than that of the BM10H model due to the tensile stress distribution along the
surface area of both the epoxy and the bolts, respectively.
Figure 5-97 shows the shear stress in the X-Z plane of the bolts where the
maximum shear stress also took place in the bolt at a value of 54.5 MPa, 37.4% less
than the shear stress in the bolts of the BM10H model. Also, the maximum shear stress
only occurred at the first bolt from the edge of the plate, unlike the BM10H model.
Therefore, the presence of epoxy reduced the shearing effect in the bolts. The concrete
elements also reached the ultimate cylindrical compressive stress in which the average
strain reached a value of -0.00596 when observing the 3rd principal strain contour plot
shown in Figure 5-98. Finally, the crack pattern of the BEM12E model was generated,
as shown in Figure 5-94, where the beam exhibited sequential cracking, similar to the
116
1
-19.2328
-16.9347
-14.6366
-12.3385
-10.0404
-7.74233
MX
-5.44423
-3.14613 MN
-.848037 End-Debonding action
1.45006 1
-19.2328
-16.9347
-14.6366
-12.3385
-1 0. 04 04
-7.74233
MN
-5.44423
-3.14613
1 -.848037
Figure 5-95: Predicted nodal vertica1.l450d06eflection for BEM12E model.

.198047
17.804

35.41
53.0159
MN
70.6219
MX
88.2278
105.834
123.44
141.046
Tensile Stress Contribution (L
158.652 eff).
Figure 5-96: Predicted nodal first principal stress for BEM12E model.
CB model. Refer to Appendix A through Appendix E to view the contour plots and
crack patterns for the rest of the specimens in the M12-Specimens group.
117
1
-108.484
-90.3735
MXMN End of Plate.
-72.2628
-54.152
-36.0413
-17.9306
.18011
18.2908
36.4015
54.5123

1 Figure 5-97: Predicted nodal shear stress for BEM12E model.
-.026808
-.023829
-.020851
-.017872
-.014893
-.011915 MX
-.008936
MN
-.005957
1 -.002979
.172E-10
Figure 5-98: Predicted nodal third principal strain for BEM12E model.

Figure 5-99: Predicted crack patterns of the BEM12E model.
5.7.4. Summary of results. A general summary that includes the contribution
of the plates (L), shear stress in bol eff,T ts (τ max,bolts) and failure modes, as shown in
118

Table 5-26. It can be observed, from the presented cases, that the specimens with edge anchors have proven to utilize most of their AA plate’s length during bending by treating their anchors as longitudinal restraints and preventing the plate from buckling. This phenomenon granted the specimens the ability to maintain a constant lever arm throughout the beams’ spans and resist the large and constant moment regions. The maximum shear stress concentration occurring within the bolts were mitigated by increasing diameter and number of the bolts while bonding by means of using adhesives, epoxy, when anchoring the plate. However, it was observed that the bonding agent has proven to be more effective in reducing the shear stress accumulation in the bolts than alternating the size and number of the bolts. This was observed when viewing specimens BM10H and BM12H where the maximum shear stress occurring within the bolts were 87 MPa, respectively. Therefore, by combining both bolts and epoxy, the structural engineer can make use of the AA plates’ mechanical properties when performing external strengthening applications and obtain a section that is stiffer and more ductile than its previous state. In addition, the failure modes were reproduced by observing the predicted stress and strain contour plots provided by ANSYS. CBE, BEM10E and BEM12E exhibited CC plus ED and ID by reaching the third principal and shear stresses within the previously established range. The rest of the models exhibited CC and PR failure modes since the first principal stress and third principal strain were within the range discussed the failure criteria section.

Table 5-26: Summary of stress contour plots.

FE Model	Effect Length (mm)	Shear Stress on Bolt (Mpa)	Maximum Shear Stress (MPa)	Third Principal Strain (mm/mm)	First Principal Stress (MPa)	Failure Mode
CBE	250	N/A	123.8	-0.00244	5.95	ED
BEM10H	100	15.2	242.5	0.00391	3.45	CC+PR
BEM10L	200	18.8	261.5	-0.00430	3.31	CC+PR
BM10H	200	87.04	241.3	-0.00466	-	CC+PR
BEM10E	425	17.2	141.3	-0.00432	6.1	CC+ID
BEM12H	100	10.4	229.8	-0.00444	4.12	CC+PR
BEM12L	200	16.13	259.4	-0.00423	4.36	CC+PR
BM12H	200	87	188	-0.00439	-	CC+PR
BEM12E	425	36	158.6	-0.00465	6.04	CC+ID

Chapter 6. Conclusion

This study investigated the effect of bolting and/or bonding of AA plates to the soffits of RC beams on their stiffness, ductility and failure modes. Several experimental parameters were considered; namely, bolt size, embedment depth and spacing. Test results which include: strain in concrete, steel and AA plate with load and deflection values that were recorded during testing. In addition, failure modes were captured. The study was divided into two phases; namely, the experimental part, which was conducted on the prepared specimens, and the FE part, which primarily focused on developing and validating FE models using the experimental results.

From the experimental investigation, the following conclusions can be drawn:

1. A bi-linear profile was observed in the plated specimens—due to the presence of two different reinforcing materials; steel and the AA plate.
2. The incorporation of AA plates in flexural strengthening applications has proven to increase the loading capacity when compared to the un-strengthened beam.
3. Using bolts has effectively negated any premature failure modes and promoted ductility, of which, some specimens demonstrated larger ductility than that of a typical RC beam.
4. The effect of increasing torque magnitudes when fixing epoxy-bonded plates to RC beams granted the beams ideal load transfer such that the plated specimens exhibited stiffer behavior during the test.
5. Larger strength enhancements were demonstrated for the specimens which included both epoxy and bolts.
6. End-plate anchorage granted the plated specimens larger ductility than plated specimens that were uniformly anchored along the span of the beams.
7. The plated specimens that were anchored uniformly, despite the presence of epoxy, have exhibited plate rupture, whereas end-plated specimens demonstrated intermediate plate-debonding.

From the FE models, the following conclusions can be drawn:

1. The FE models produced results that correlated well with the results obtained from the experiment.
2. FE modelling could be executed as an alternative to experimental work in which unique and complicated strengthening configurations can be designed and pro-

posed to the clients.
3. The peak load and deflection value predicted by the FE models showed mini-
mal percentage differences, less than 10%, when compared to the peak load and
deflection values obtained from the experiment.
4. Bond-slip models can be employed into the FE environment such that debonding-
related failures can be reproduced and simulated for externally strengthened spec-
imens.
5. The number and size of anchors is inversely proportional to the shear stress con-
centration within the bolts.
6. The implementation of bolting uniformly along the length of the plate reduces the
effective length in which the tensile stress acts along.
7. End-plate bolting combined with epoxy-bonding demonstrated the most efficient
section in terms of both stiffness and ductility.
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Reinforced Con-

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Curved Soffit RC Beams,” Advances in Structural Engineering, vol. 8, no.
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1 Appendix A: Predicted Nodel Vertical Deflection Plots for FE Models

NODAL SOLUTION
-17.0011
-14.9724
-12.9437
-10.9151
-8.88638
-6.85771
MX
-4.82904
MN
-2.80037
-.771696

1.25698
Figure 6-100: Predicted nodal vertical deflection for CB model.

1NODAL SOLUTION
MAR 23 2019
17:19:15
-18.2191
-16.0467
-13.8742
-11.7018
-9.52936
-7.35692
MX
-5.18448
-3.01204 MN
-.839599
Debonding action
1.33284
1
-18.2191
-16.0467
-13.8742
-1 1.7 01 8-9.52936
-7.35692
-5.18448
-3.01204
-.839599
1.33284

Figure 6-101: Nodal vertical deflection and debonding action for CBE model.
128
1
-36.0044
-31.7207
-27.4369
-23.1531
-18.8693
-14.5856
MX
-10.3018
-6.01803 MN
-1.73426
2.5495
1 Figure 6-102: Predicted nodal vertical deflection for BEM10H model.

-15.9652
-14.0582
-12.1511
-10.2441
-8.33701
-6.42995
MX
-4.52289

-2.61583 MN
-.708772
1.19829
1 Figure 6-103: Predicted nodal vertical deflection for BEM10L model.

-21.536
-18.9711
-16.4062
-13.8414
-11.2765
-8.71159
MX
-6.14671
-3.58183 MN
-1.01695
1.54794
Figure 6-104: Predicted nodal vertical deflection for BM10H model.

129
1

-14.9947
-13.2047
-11.4146
-9.62462
-7.83459
-6.04456
MX
-4.25453
-2.4645 MN
-.674469
End-Debonding Action
1.11556
1
-14.9947
-13.2047
-11.4146
-9.62462
-7.83459
-6.0-
4.2 445545 63
-2.4645
-.674469
1.11556
MN

Figure 6-105: Predicted nodal vertical deflection for BEM10E model.
1NODAL SOLUTION
-18.7055
-16.4727
-14.2399
-12.0071
-9.7743
-7.5415

MX
-5.30871
-3.07592 MN
-.843134
1.38966
Figure 6-106: Predicted nodal vertical deflection for BEM12H model.

130
1
-15.6703
-13.7985
-11.9267
-10.0549
-8.18306
-6.31125
MX
-4.43943
-2.56762 MN
-.695807
1.17601
Figure 6-107: Predicted nodal vertical deflection for BEM12L model.

1
-18.8871
-16.6339
-14.3808
-12.1277
-9.87456
-7.62143
MX
-5.3683
-3.11518 MN
-.862049
1.39108
Figure 6-108: Predicted nodal vertical deflection for BM12H model.

131
1
-19.2328
-16.9347
-14.6366
-12.3385
-10.0404
-7.74233
MX
-5.44423
-3.14613 MN
-.848037 End-Debonding action
1.45006 1
-19.2328
-16.9347
-14.6366
-12.3385

-1 0. 04 04

-7.74233
MN
-5.44423
-3.14613
-.848037
Figure 6-109: Predicted nodal vertic1a.45l006deflection for BEM12E model.

1 Appendix B: Predicted Nodal First Principal Stress Plots for FE Models
2.15879 MN
15.6778
29.1968
42.7158
56.2348
69.7538
83.2728
96.7918
MX
110.311
Tensile Stress Contribution.
123.83

F i g u r e 6 - 1 1 0 : P r e d i c t e d n o d a l
fi r s t p r i n c i p a l s t r e s s f o r C B E model.
1
.151591
27.084
54.0164
80.9488 MN
107.881
134.814
161.746
188.678
215.611
242.543
Stress Concentration at Bolt Locations (Leff). MX
Figure 6-111: Predicted nodal first principal stress for BEM10H model.

1
-2.08176
27.1799
56.4415
85.7031
114.965
144.226
173.488
202.75 MN
232.011
261.273
MX
Tensile Stress Contribution (Leff).
Figure 6-112: Predicted nodal first principal stress for BEM10L model.
1
-22.301

43.7075 MMNX
109.716
175.725
241.733
307.742
373.75
439.759
505.767
Stress Concentration at Bolt Locations (Leff). 571.776
Figure 6-113: Predicted nodal first principal stress for BM10H model.
134
1
.489059
19.0298
37.5705
56.1113
MN
74.652
MX
93.1927
111.733
130.274
148.815
Tensile Stress Contribution (Leff).
167.356
Figure 6-114: Predicted nodal first principal stress for BEM10E model.
1
-.706491
24.9029
50.5123
76.1217
101.731
127.34
152.95
178.559 MN
204.169
229.778
Stress Concentration at Bolt Locations (Leff). MX
Figure 6-115: Predicted nodal first principal stress for BEM12H model.
135
1
-.568747
28.3222
MN
57.2131
86.104
114.995
143.886
172.777
201.668
230.559
259.449
MX
Stress Concentration at Bolt Locations (Leff).

Figure 6-116: Predicted nodal first principal stress for BEM12L model.
1
-17.1999
34.3048 MX
MN
85.8096
137.314
188.819
240.324
291.828
343.333
394.838
446.343 Stress Concentration at Bolt Locations (Leff).
Figure 6-117: Predicted nodal first principal stress for BM12H model.

1
.198047
17.804
35.41
53.0159
MN
70.6219
MX
88.2278
105.834
123.44
141.046
Tensile Stress Contribution (L
158.652 eff).
Figure 6-118: Predicted nodal first principal stress for BEM12E model.

Appendix C: Predicted Nodal Shear Stress Plots in Bolts
1
-221.51
-174.17
End of Plate.
-126.829
-79.4891
-32.1489
15.1913
62.5316
109.872
157.212
204.552

Figure 6-119: Predicted nodal shear stress plots in bolts for BEM10H model.

1
-255.801
End of Plate
-200.878
-145.955
-91.0321

-36.109
18.814
73.7371
128.66
183.583
238.506 MX
Figure 6-120: Predicted nodal shear stress plots in bolts for BEM10L model.

138
1
-188.647
-158.015
-127.382 End of Plate.
-96.7499
-66.1175
-35.485 MX
-4.85258
25.7799
56.4123
87.0447 MN

Figure 6-121: Predicted nodal shear stress plots in bolts for BM10H model.

1
MN
-116.814
-94.4724
-72.131
End of Plate.
-49.7896
-27.4482
-5.10681
17.2346
39.576
61.9174
MX
84.2588

Figure 6-122: Predicted nodal shear stress plots in bolts for BEM10E model.

139
1
-195.007
-153.949
End of Plate.
-112.89
-71.8318
-30.7734
10.285
51.3434
92.4018

133.46
174.519

Figure 6-123: Predicted nodal shear stress plots in bolts for BEM12H model.

1
-254.658
-200.499
-146.339 End of Plate.
-92.1799
-38.0204
16.1391
70.2985
124.458
178.617
232.777
MX

F ig u r e 6 - 1 2 4 : P r e d ic t e d n o d a l s h e a r s t r e s s p l o t s i n b o l t s for BEM12L model.
140
1
-188.647
End of Plate
-158.015
-127.382
-96.7499
MX
-66.1175
-35.485
-4.85258
MN
25.7799
56.4123
87.0447
Figure 6-125: Predicted nodal shear stress plots in bolts for BM12H model.

1
-108.484
-90.3735
MXMN End of Plate.
-72.2628
-54.152
-36.0413
-17.9306
.18011
18.2908
36.4015
54.5123

Figure 6-126: Predicted nodal shear stress plots in bolts for BEM12E model.

Appendix D: Predicted Nodal Third Principal Strain for FE Models

-.006034
-.005363
-.004693
-.004023
MX
-.003352
-.002682
MN
-.002011
-.001341
-.670E-03
.292E-09

Figure 6-127: Predicted nodal third principal strain for CB model.

1

-.0044
-.003911
-.003422
-.002933 MN
-.002444
-.001955
MX
-.001467
-.978E-03
-.489E-03
.301E-08

Figure 6-128: Predicted nodal third principal strain for CBE model.

1
-.020845
-.018529
-.016213
-.013897
-.011581
-.009265
MX
-.006948
-.004632 MN
-.002316
.264E-08

Figure 6-129: Predicted nodal third principal strain for BEM10H model.

-.0155
-.013778
-.012056

-.010333
-.008611

-.006889 MX
-.005167
MN
-.003444
-.001722
.283E-08
1
Figure 6-130: Predicted nodal third principal strain for BEM10L model.

-.041943
-.037283
-.032622
-.027962
-.023302
-.018641
MX
-.013981
MN
-.009321
-.00466
.275E-08
Figure 6-131: Predicted nodal third principal strain for BM10H model.

1
-.015553
-.013825
-.012097
-.010369
-.00864
-.006912
MX
-.005184
MN
-.003456
-.001728
.280E-08
Figure 6-132: Predicted nodal third principal strain for BEM10E model.

-.019998
-.017776
-.015554
-.013332
-.01111
-.008888
MX
-.006666
-.004444 MN
-.002222
.281E-08
1 Figure 6-133: Predicted nodal third principal strain for BEM12H model.

-.038062
-.033833
-.029604
-.025375
-.021146
-.016916
MX
-.012687
MN
-.008458
-.004229
.260E-08
Figure 6-134: Predicted nodal third principal strain for BEM12L model.

1
-.026348
-.02342
-.020493
-.017565
-.014638
-.01171
MX
-.008783
MN
-.005855
-.002928
.237E-08
Figure 6-135: Predicted nodal third principal strain for BM12H model.

1
-.026808
-.023829
-.020851
-.017872
-.014893
-.011915 MX
-.008936
MN
-.005957
-.002979
.172E-10
Figure 6-136: Predicted nodal third principal strain for BEM12E model.

1 Appendix E: Predicted Crack Pattern for FE Models
CRACKS AND CRUSHING
STEP=14
SUB =24 The appendix could include copies of surveys, experimental results, software
TIME=14.1988

programs, data and other supporting information.

1
CRACKS AND CRUSHING
STEP=30 Figure 6-137: Predicted crack patterns of the CB model. MAR 23 2019
SUB =24 20:02:28
TIME=14.8512

1
Figure 6-138: Predicted crack patterns of the CBE model.

Figure 6-139: Predicted crack patterns of the BEM10H model.

1
1
CRACKS AND CRUSHING
STEP=30 MAR 23 201 9
SUB =24 20:06:29
TIME=32.757
Figure 6-140: Predicted crack patterns of the BEM10L model.

1

Figure 6-141: Predicted crack patterns of the BM10H model.

1

Figure 6-142: Predicted crack patterns of the BEM10E model.

1

Figure 6-143: Predicted crack patterns of the BEM12H model.

Figure 6-144: Predicted crack patterns of the BEM12L model.

1

Figure 6-145: Predicted crack patterns of the BM12H model.

1

Figure 6-146: Predicted crack patterns of the BEM12E model.

Vita

Omar Raed Abuodeh was born in 1994, in Pocatello, Idaho, in the United States of America. In 2001, he moved to the United Arab Emirates where he studied for 12 years in Dubai International School and graduated fifth amongst his classmates. Afterwards, Mr. Abuodeh pursued a Bachelors degree in Civil and Environmental Engineering from the University of Sharjah, where he partook in various academic competitions and resarch projects and graduated with a CGPA of 3.55 in June 2016.