

**PUNCHING SHEAR PERFORMANCE OF
STEEL FIBER-REINFORCED RECYCLED
AGGREGATE CONCRETE SLABS
STRENGTHENED WITH NEAR SURFACE
MOUNTED COMPOSITES**

BY

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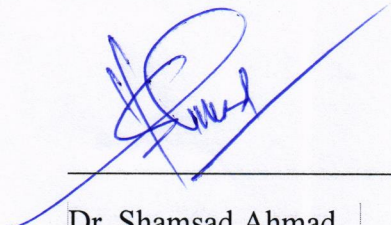
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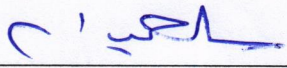
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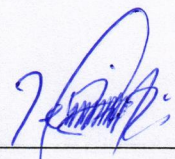
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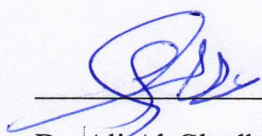
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To my family

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ABSTRACT

Full Name : Maan Abdulmajeed Lardhi
Thesis Title : Punching Shear Performance of Steel Fiber-Reinforced Recycled Aggregate Concrete Slabs Strengthened with Near Surface Mounted Composites
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This thesis aims to evaluate the punching shear performance of steel fiber-reinforced recycled aggregate concrete (RAC) column-slab connection strengthened by near-surface-mounted (NSM) fiber-reinforced polymer (FRP) bars. 5 specimens are tested using a specially designed frame with 4 mixtures across two types NSM FRP bars. Simultaneous replacement of recycled coarse aggregate (RCA) with natural coarse aggregate (NCA) (0% and 100%) are combined with each two steel fiber contents (0% and 1%) in the mixtures, using Carbon FRP bars and Glass FRP bars as the strengthening NSM FRPs. The experimental results of the test are analyzed and compared with the analytical results. The NSM FRP-concrete bond behavior is evaluated for several mixture variations shared with the previous test. 9 mixtures are designed containing 0%, 30%, 60%, and 100% RCA replacement for each 0%, 0.5%, and 1% steel fibers of specimen's volume. The NSM FRP-concrete bond behavior of GFRP bars, CFRP bars, and CFRP strips is investigated by various direct pull-out tests using the Universal Debonding Test Apparatus. For the apparatus' abilities of gripping laminates bars in different configurations, single-shear, double-shear, beam-bend, and tests on edge distance bond tests are performed. The compressive strength of the steel fiber-reinforced RAC,

punching shear capacity, maximum slab deflection, and strain distribution of different components of specimens are discussed. Also, the bond-slip curves, and maximum bond capacity for bond tests are discussed. The validity of the Universal Debonding Test Apparatus is discussed.

ملخص الرسالة

الاسم الكامل: معن عبدالمجيد لرضي

عنوان الرسالة: دراسة أسلوب البلاطات الخرسانية المكونة من الركام المعاد تدويره والمقواة بألياف الفولاذ وأسياخ الألياف البوليمرية الموضوعة بالحشو السطحي عند تحمل الضغط للقوى المركزة

التخصص: الهندسة المدنية

تاريخ الدرجة العلمية: ديسمبر 2022

تهدف هذه الرسالة إلى تقييم سلوك الرابطة الخرسانية مع أسياخ الألياف البوليمرية الموضوعة بالحشو السطحي لثلاثة أنواع من الاسياخ من خلال العديد من اختبارات السحب المباشر: الفردي ، المزدوج ، والانحناء ، ومسافة الحافة الخرسانية غير الكافية. بعد اختبار السحب الفردي، يتم تقييم سلوك الرابطة الخرسانية أسياخ الألياف البوليمرية الموضوعة بالحشو السطحي للعديد من الخلطات الخرسانية. متغيرات هذا البحث هي ألياف الفولاذ ونسبة دمج الركام المعاد تدويره. يحتوي 12 خليطاً على كميات 0% ، 30% ، 60% ، و 100% من الركام المعاد تدويره لكل 0% ، 0.5% ، و 1% ألياف فولاذية من حجم العينة. بعد النظر في الرابطة الخرسانية مع أسياخ الألياف البوليمرية الموضوعة بالحشو السطحي، يتم تقييم سلوك أسياخ الألياف البوليمرية الموضوعة بالحشو السطحي على بلاطات خرسانية مكونة من الركام المعاد تدويره والمقوى بالألياف الفولاذية. المتغيرات الثلاثة لهذه الدراسة هي محتوى ألياف الصلب ، واستبدال الركام المعاد تدويره ، ونوع أسياخ الألياف البوليمرية. تم تحليل النتائج التجريبية للدراسة ومقارنتها بالنتائج التحليلية. تمت مناقشة صلاحية جهاز اختبار إزالة الروابط الخرسانية الموحد. كما تمت مناقشة القوة القصوى للروابط. تمت مناقشة قوة الضغط الخاصة بالخرسانة المكونة من الركام المعاد تدويره والمقوى بالألياف الفولاذية ، وقدرة تحمل الضغط للقوى المركزة ، وانحراف البلاطات الخرسانية ، وتوزيع التمدد لأسياخ الألياف البوليمرية.

CHAPTER 1

INTRODUCTION

1.1 General

The massive use of Natural Coarse Aggregate (NCA) in concrete production is one of the reasons that obstruct the natural resources preservation [1–3]. As structures demolition is a periodic point in civilized societies for future enhancements, demolition waste has a potential to be piled up and left unused forming Construction Demolition Waste (CDW). CDW can be recycled and crushed producing Recycled Coarse Aggregate (RCA) as its development has been an interesting topic among researchers [4–14]. Regarding environmental benefits, 65% of greenhouse gas emissions could be saved by using RCA in concrete mixtures with a reduced consumption of 58% in non-renewable energy resources [15]. Other studies concentrating on RCA's environmental benefits affirmed these findings [16,17]. Different behaviors between NCA and RCA in concrete were exhibited, and the old adhered cement mortar presented in the surface of the original NCA was the major cause for the different behaviors [18–21].

Given the environmentally friendly nature of RCA concrete (RAC), its use in different infrastructural constructions require thorough investigation, especially that the material is weaker than NCA concrete (NAC). Although flat slabs are advantageous in

any structural systems for its light appearance serving architects views, one of their critical failure modes is punching shear. Therefore, this behavior calls for systematic study if constructed using RAC. There are traditional methods to increase the punching shear resistance published previously [22,23]. However, there are limitations to these methods.

Since these traditional strengthening methods proved promising results even in structures using NCA, RCA-incorporated structures deserve the attention to be investigated when strengthened. As their strength could reach 4 times higher than steel strength, Fiber-Reinforced Polymers (FRP) offer a potential solution. There are two main methods for strengthening structural concrete elements with FRP. They are the Externally bonded reinforcement (EBR) and Near-surface-mounted (NSM). EBR is used more common than NSM for strengthening concrete structures. However, the early debonding of the FRP for the surface is the major drawback in EBR strengthening [24]. Some of the many pros of NSM strengthening over EBR strengthening are protection against impacts, a stronger bond with concrete, and a higher fire resistance [25]. Also, it is proved that NSM strengthening increases the peak load more effectively than EBR strengthening [24]. NSM-strengthening serves as the better solution in which the NSM strengthening process involves the external insertion of rebars with covering, therefore, meeting the requirement of historical structures preservation to save their aesthetical properties. As many types of concrete structural elements strengthened by NSM FRP were studied [24,26–30], bond behavior is an essential factor that must be investigated to extract the highest benefit of NSM FRP strengthening.

Since the primary factor dictating the design of any FRP-strengthening application is bond strength, NSM FRP-concrete bond requires further investigations. By modifying current testing of steel reinforcements that imitate an NSM configuration, direct pull-out tests for NSM FRP strengthening were created [31]. For different concrete stress states, different testing sets are employed to simulate specific FRP NSM system component [32]. The beam-bend, double-shear, and single-shear tests are the ones that are most frequently used to examine the bond strength between FRP, adhesive, and concrete. Some of the initial studies especially focused on the topic of NSM bond utilizing various test methodologies seeking to imitate the FRP system's pull-out [33–35]. The various forms of pull-out tests may be classified, as shown in Fig. 1, into the following broad groups [36]: single/double-shear pulling test, single/double-shear pushing test, and beam-bending test. The cubic and prismatic blocks are commonly used in single-shear and double-shear tests, however in beam-bend test, a beam with a notch in the middle or two concrete blocks joined by a mechanical hinge are utilized, adopting the configuration for three or four point bending tests [37]. When bond tests using NSM-FRP strips/bars are conducted, the following failure types can be seen: failure through debonding at the adhesive-FRP interface, cohesive failure in the adhesive, failure at the adhesive-concrete interface, cohesive failure in the concrete, and FRP rupture [32].

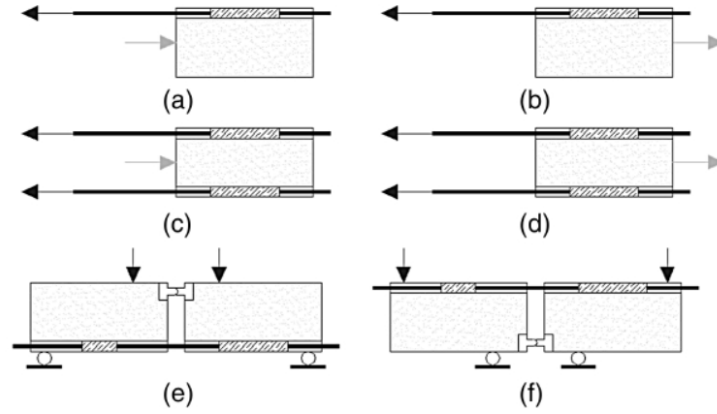


Figure 1: Setups for evaluating the NSM FRP bond behavior: (a) single-shear pushing test; (b) single-shear pulling test; (c) double-shear pushing test; (d) double-shear pulling test; and (e and f) beam-bend test. Reprinted from Bilotta et al. [38].

When the concrete edge distance is less than a threshold value, which is mostly based on the groove height and the concrete strength, the edge distance can have a substantial impact on NSM FRP bond [39]. Basically, a higher threshold value of edge distance results from a larger groove height or concrete strength [39].

As different bond test setups are proposed for every NSM bond test investigated in the literature, differences in results are expected. Therefore, the need for unifying test methods for every NSM bond test is high for the literature to possess comparable NSM bond test results between research labs globally. Ultimately, a universal method of testing for all NSM bond tests is favorable [40].

This thesis aims to evaluate punching shear performance of steel fiber-reinforced RAC column-slab connection strengthened by NSM FRP bars. Seven specimens are tested using a specially designed frame with 4 mixtures across two types NSM FRP bars. Simultaneous replacement of recycled coarse aggregate (RCA) with natural coarse aggregate (NCA) (0% and 100%) are combined with each two steel fiber contents (0%

and 1%) in the mixtures, using Carbon FRP bars and Glass FRP bars as the strengthening NSM FRPs. The experimental results of the test are analyzed and compared with the analytical results. The NSM FRP-concrete bond behavior is evaluated for several mixture variations shared with the previous test. 12 mixtures are designed containing 0%, 30%, 60%, and 100% RCA replacement for each 0%, 0.5%, and 1% steel fibers of specimen's volume. The NSM FRP-concrete bond behavior of GFRP bars, CFRP bars, and CFRP strips is investigated by various direct pull-out tests using the Universal Debonding Test Apparatus [40]. For the apparatus' abilities of gripping laminates bars in different configurations, single-shear, double-shear, beam-bend, and tests on edge distance bond tests are performed.

1.2 Significance of the Research

The use of RCA has been associated with environmental benefits by many researchers claiming that 65% of greenhouse gas emissions could be saved by using RCA in concrete mixtures with a reduced consumption of 58% in non-renewable energy resources [15–17]. For the punching shear behavior of steel fiber reinforced RAC slabs, not enough research work has been issued, to the best knowledge of the author, as well as the bond behavior of NSM FRP. This limits the exploitation of steel fiber-reinforced RAC, in general, and limiting the research progress on punching shear behavior of NSM strengthened RAC slabs. This research has the potential to convince the construction industry in Saudi Arabia or the gulf countries to make a major step towards sustainability and utilize the waste of demolition, hence the recycled aggregate, and ease the way for NSM FRP use in existing buildings. The thesis studies the punching shear behavior and

the bond behavior of NSM FRP in steel fiber RAC with varying steel fiber volumetric ratio, RCAs replacement percentages, and FRP types.

1.3 Objectives

The thesis intends to demonstrate the effects of FRP strengthening on the punching behavior of slab/column connection constructed from steel fiber RAC. The specific objectives of this thesis are to:

- Study the bond behavior of NSM FRP bars/strips in steel fiber RAC
- Evaluating NSM FRP-concrete bond by various pullout tests using a unified debonding tests apparatus
- Study the punching behavior of slab/column connection constructed from steel fiber RAC
- Compare the punching shear results with existing design codes and analytical models

1.4 Research Methodology

This thesis studies the effect of near-surface-mounted FRP (NSM FRP) strengthening of concrete in one of the most critical components of building structures; the column/slab connection. Recycled aggregates are constituents in the concrete used raising a problem for weakening this critical point of the structure. Looking for solutions, NSM FRP and steel fibers are two solutions applied to eliminate such problem.

The bond behavior of NSM FRP bars/strips and steel fiber RAC was studied based on the criteria of mentioned standard bond tests for NSM FRP. Single shear,

symmetrical double shear, asymmetrical double shear, and beam tests are performed. The punching behavior of column/slab connection constructed from steel fiber RAC was studied following established test methods. It is the simply supported slab with a loaded column in its center. The column was built with the slab to imitate real applications. The specimens are scaled down by a factor of 0.5. The work was conducted in the following two phases:

Phase I Bond Tests:

1. Single-shear bond test were performed on NSM FRP with circular cross-section in each mixture. The variables of this study are steel fibers content and recycled aggregate incorporation ratio; 9 mixtures contained 0%, 30%, 60%, and 100% RCA replacement for each 0%, 0.5%, and 1% steel fibers of specimen's volume. Linear Variable Differential Transducers (LVDT) were used to measure the bond slip and strain gauges were used to measure the strain were used to measure the FRPs strain.
2. Single-shear bond test, symmetric double-shear bond test, asymmetric double-shear bond test, beam-bend test, and tests on edge distance effect were performed on NSM FRP with circular cross-section in an ordinary concrete mixture with 2 samples for each test, circular GFRP and CFRP on one hand, and rectangular CFRP, on the other hand. The different NSM FRP tests were performed by the Universal Debonding Test Apparatus [40] for its capabilities to be mounted in compatible configurations. Linear Variable Differential Transducers (LVDT) were used to measure the bond slip and strain gauges were used to measure the strain were used to measure the FRPs strain.

3. Failure modes and ultimate loads of bond are discussed. NSM FRP strains are analyzed to study the local behavior of the bond. Finally, bond-Slip relations are investigated.

Phase II Punching Shear Tests on Steel Fiber-Reinforced Concrete Slabs Strengthened with NSM FRP:

1. Five slab specimens were casted (1500 mm x 1500 mm x 120 mm) with a column stub at the center (150 mm x 150 mm x 150 mm).
2. The specimens were strengthened by 4 NSM FRP bars with circular cross-section in tension side in each slab specimen.
3. The slab specimens were attached with strain gauges at the steel reinforcements, at the NSM FRP, and at the concrete surface (tension side) to capture the strain distribution.
4. Five LVDTs were used to capture the deflection of the slab.
5. The slab specimens were tested in the lab mounted on specially made steel frame representing simple supports with a 3000-kN hydraulic jack applying on the column stub resisted by a reaction frame.
6. The punching shear failure modes are discussed with the ultimate load, followed by an analysis on the punching radius. The maximum deflection of the specimens is evaluated to account for specimens' serviceability. NSM FRP, steel reinforcement, and concrete strains are evaluated to account for the local behavior of the specimens. Flexural stiffness results are calculated.

7. Ultimate Loads are compared with those obtained from Finite-Element Analysis (FEM), and calculated from design codes (ACI, fib, and Eurocode) and the analytical model (Critical Shear Crack Theory).

CHAPTER 2

LITERATURE REVIEW

2.1 Punching Shear Behavior of NSM FRP-Strengthened Slab-Column Connections

An experimental study was performed to evaluate the efficiency of NSM method in strengthening slab-column connection in the tension side against punching shear failure using Carbon-FRP (CFRP) bars [29]. Using NSM CFRP bars of 10 mm diameter with different locations and arrangements, the specimens were strengthened. With a thickness of 130 mm, the slab specimens' dimensions were 1100 x 1100 mm and a column stub of 150 x 150 mm that was casted at the center of the slabs monolithically to imitate real application. The specimens were reinforced with high tensile steel meshes of 10 mm in diameter at the bottom and top. The column stubs were reinforced by four 12 mm steel bars and 8 mm mild steel stirrups every 100 mm. The grooves were formed by 25 x 25 mm wooden sticks put under the molds before casting concrete to install the required NSM-CFRP bars. The use of wooden sticks to make the grooves is widely considered in NSM method publications [27,28,41]. 30 mm cover was kept at the bottom of the slab and 15 mm cover was reserved at the top of the slab. Different layouts were proposed in installing the CFRP bars. Two specimens had orthogonal square configurations and the other two had skewed square configurations as shown in Fig. 2. After casting, the grooves were filled halfway with the structural adhesive Sikadur-330 and inserted the CFRP bars to fill the sides first. After that, a second layer was applied to

fill the grooves completely and levelled the surfaces. A rise of 40-56% in the ultimate punching capacity was resulted compared to the controls specimens. The stiffness in the uncracked stage was increased by 57-99%. A higher punching capacity was resulted when CFRP bars, in square shape, were installed at a distance equal to the slab depth than when CFRP bars were installed at a distance equal to half the slab depth. The skewed configuration showed a slight improvement in the stiffness and punching capacity compared to orthogonal configuration. In punching shear mode, all the specimens failed.

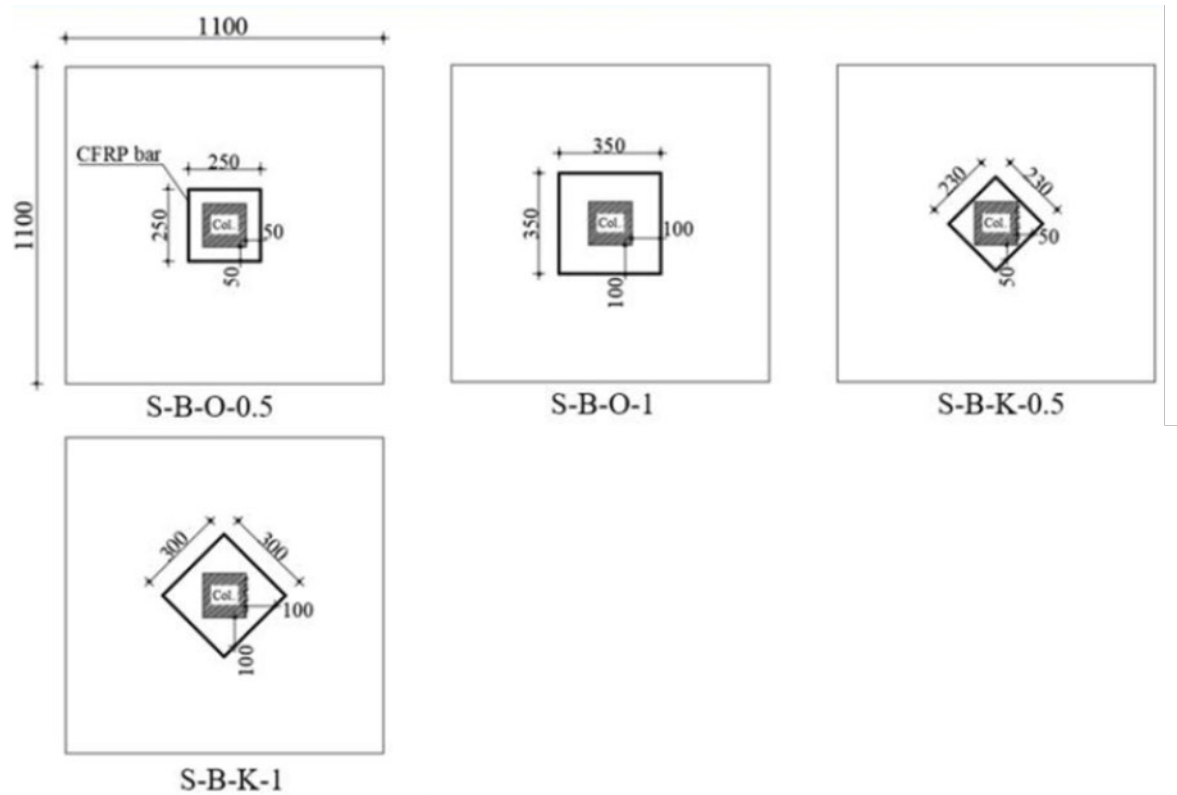


Figure 2: The four configurations of CFRP bars installations. Reprinted from Abdul-Kareem [29]

In addition, An experimental study to examine the efficiency of the NSM method in strengthening column-slab connection against punching shear failure was conducted by varying the configuration of the strengthening NSM-CFRP bars and the NSM-CFRP bar diameter [30]. Claiming they were examples of a real flat slab scaled down by a factor of

0.5, their specimens had the dimensions of 1500 x 1500 x 120 mm and a column head with the dimensions of 150 x 150 x 150 mm. The specimens were reinforced by a mesh of 15 high tensile strength steel bars at the tension side with a diameter of 8 mm and a mesh of 8 high tensile strength steel bars at the compression side with a diameter of 6 mm. The column head was reinforced by four 10 mm high tensile strength steel bars and three 6 mm stirrups. The NSM-CFRP bars had a tensile strength of 1800-2200 MPa and an elastic modulus of 140-150 GPa with a minimum rupture strain of 0.0129. The grooves were made by the wooden sticks technique as mentioned in the previous study. The grooves dimensions were proposed to be $1.5d_b \times 1.5d_b$, where d_b is the CFRP bar diameter. As shown in Fig. 3, development of dowel forced in the CFRP bars will be allowed in the two chosen layouts at the intersection point with the inclined shear crack. The test resulted in a classical failure of punching shear to all the specimens. Generally, the punching shear capacity of the slab-column joint increased by the use of NSM-CFRP bars. Additionally, the average strength gain for strengthening layouts 1 and 2 was 18% and 41%, respectively. Compared with the control specimens, the maximum flexural stiffness was recorded with layout 2 and a CFRP bar diameter of 10 mm, which showed an increase of 100%. A linear relationship for load-strain curves and load-displacement was maintained as a result of delaying the development of first crack in concrete because of strengthening.

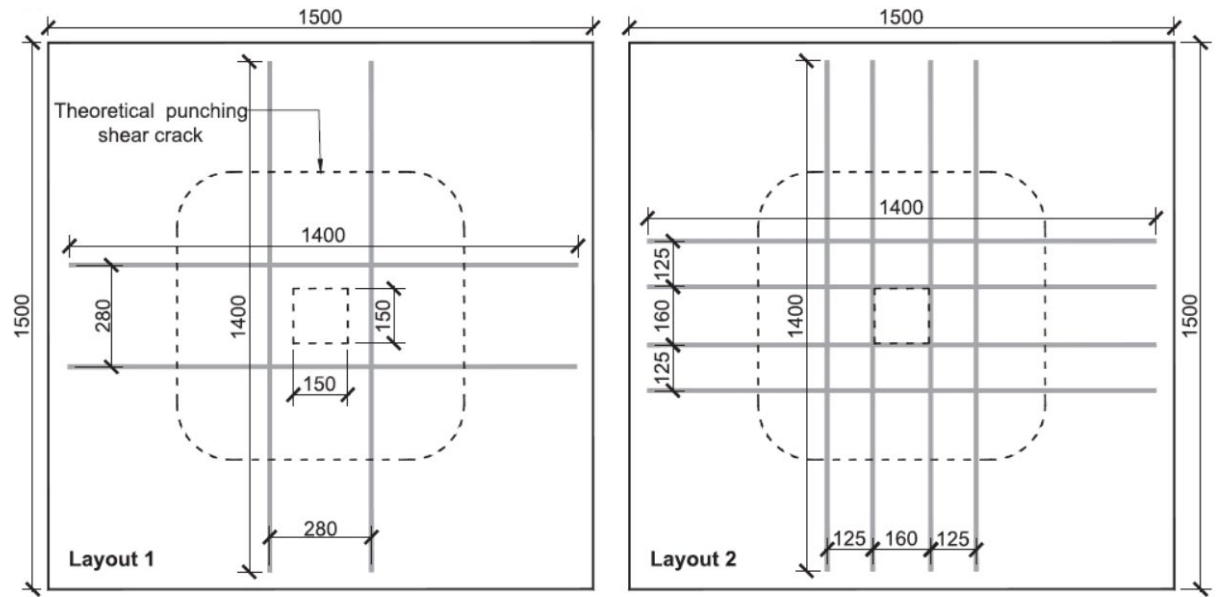


Figure 3: The two layouts of Akhundzada et al.'s test. Reprinted from Akhundzada et al. [30]

2.2 Recycled Aggregate Concrete

2.2.1 RCA Incorporation in Concrete

It is essential to demonstrate the mechanical properties of the recycled aggregate concrete (RAC). Regarding the compressive strength of RAC, many studies performed compressive strength test on RAC to assess the effect of RCA incorporation in concrete and predict the range of its strength [5,42–44]. The effect of RCA incorporation was negligible on RAC compressive strength and resulted in almost equal compressive strength [5]. However, an insignificant effect of RCA incorporation on RAC compressive strength was resulted in other studies with fluctuated results [43–45]. The crushing process of RCA is the reason behind this fluctuation and tertiary crushing process of RCA makes aggregates resemble the shapes of NCA with rounder surfaces [5].

Regarding Young's Modulus, RCA incorporation affects Young's modulus in RAC and it reduces Young's modulus since RCA have higher deformability than NCA, as a result of the adhered mortar [5]. The range of Young's modulus of RAC can be 10 – 30% [5,46,47].

Due to adherent mortar in RCA, RAC's overall permeability is greater than that of ordinary concrete based on the contribution of RCA permeability [5]. Therefore, RAC's water absorption is higher than water absorption of ordinary concrete. Full RCA incorporation can result in an increase of 42% in capillary absorptions and 27% in absorptions due to immersion [5]. Other studies showed less water absorption results of RAC due to different origins of RCA and type of crushing process [42,43]. Thus, high resemblance of RCA to NCA means low water absorption. A rise in abrasion resistance in RAC of 6% was reported [5,7]. As negligible as it can be, the abrasion resistance is controlled primarily by cement matrix wear rather than aggregate wear, which means the better the cement/aggregate adhesion is the better the wear resistance [48].

Many studies showed the effect of RCA incorporation on RAC's shrinkage, where RAC of full RCA incorporation had an increase of about 16% in a 91-day shrinkage [5,43,49]. This increase in long term shrinkage is explained to be instigated by the deformability of the adherent mortar of the RCA [5]. However, there is a decrease in the 7-day shrinkage of about 26% as a result of the raised water absorption, causing the RAC to cure internally, where water migrates from RCA to the new cement [5]. In addition, for full RCA incorporation, the range of increase in long term shrinkage of RAC is 10 – 100% [50]. Factors that varies shrinkage results of RAC are different ages of the source concrete and different mix design approaches [5].

Regarding RAC's durability, RCA incorporation produces marginal increase in the carbonation depth of RAC, where it is reported that the range of increase of full RCA incorporation is 1.2 – 2 mm [5,51]. RAC ingress more chloride compared to conventional concrete because RAC is more permeable, where the effect of RCA incorporation depends on the RCA quality rather than incorporation ratios [5].

2.2.2 RAC Punching Shear Behavior

This section focuses on RAC slab punching behavior and capacity from the few works that can be found in the literature. An experimental, analytical, and numerical study on the RCA incorporation effects on the punching shear of slab was conducted [52]. The RCA was produced with a maximum aggregate size of 22.4 mm from an ordinary concrete source where there were mortar and aggregate. The average compressive strength of the RAC was 42.8 ± 1.3 MPa at 28 days. For a full RCA incorporation, RAC slabs performed as much as the ordinary concrete for punching shear capacity, with a strength reduction of only 2%.

The punching behavior of RAC slabs was investigated to expand the knowledge of RAC punching shear failure [53]. Specifically, their objective was to analyze the practicability of using RCA produced from concrete waste with unidentified mechanical properties to consider RAC structural elements. A total of 15 slabs had been tested with 0%, 30%, 50%, 80%, and 100% RCA incorporation ratios. The water absorption of the obtained recycled aggregate was 7.54%. For steel reinforcement in slabs, high tensile steel bars of diameter 5 mm were used in a mesh for top and bottom reinforcement with 100 mm spacing. The slab specimen dimensions were 1100 x 1100 x 50 mm.

Nevertheless, the column part was not included in the specimen by making a column stub at the center. The column stub was replaced by putting a small steel plate between the hydraulic jack and the slab specimen with dimensions of 200 x 200 mm. Although the compressive strength of the RAC cubes on was decreasing with increasing RCA incorporation ratios, it was stated that the punching shear strength was comparable to that of natural aggregate concrete slabs. Also, for full RCA incorporation, the first crack load was comparable to that of natural aggregate concrete slabs. In addition, with increasing RCA incorporation ratios, the maximum deflections for each group increased.

When steel fibers were introduced to concrete, investigations were performed to analyze the behavior of NAC slab-column connection and enhance the shear resistance and the deformation capacity [54]. Also, a model had been developed to predict the punching resistance of centrally loaded steel fibers reinforced concrete slabs [55]. The behavior of punching shear of RAC slab with steel fibers was investigated [56]. The test was performed on 8 RAC slabs. RCA with a water absorption of 5.0%, clay content of 3.8%, and a crushing of 13.7% were used. With 50% and 100% RCA incorporation ratios, the steel fiber contents were 0%, 0.5%, and 100% in each mixture. Fig. 9 shows the details of their slab specimens. Without using a column stub, the applied load by the hydraulic jack on the slab specimen was on a separate concrete-filled square steel tubular short column. Fig. 10 shows the details of the loading setup. For full RCA incorporation ratio, RAC slabs with 0%, 0.5%, and 1% steel fiber content had punching shear capacities of 303.4 kN, 331.2 kN, and 350.2 kN, respectively. The NAC slab had a punching shear capacity of 320 kN. In addition, NAC slab's punching shear capacity is slightly superior to that of RAC's slab. 7% - 15% is the increase in punching shear capacity when steel

fibers are added. Slab's deformation ductility, and energy consumption is enhanced too. It was concluded that the addition of steel fibers in RAC slabs helps to convert the failure pattern from shear failure to bending-shear failure for RAC slabs under punching shear.

2.3 NSM FRP-Concrete Bond Strength

2.3.1 NSM FRP Single-Shear Bond Test

An experimental study was performed to analyze the bond behavior of NSM Carbon-FRP (CFRP) strips bonded to concrete elements under sustained loading [57]. Regarding the adhesive, the tensile behavior of the adhesive Sikadur 30 considered in the NSM CFRP-concrete joint through a sustained-loading testing was characterized in accordance with ISO 527-2 [58]. It resulted in an average tensile strength of 27.2 MPa and an average elastic modulus of 10.7 GPa. The five specimens of the short-term pull-out tests were blocks with 150 mm bonded length (L_b) and 10 mm groove width (t_g), 150 mm L_b and 7.5 mm t_g , 225 mm L_b and 10 mm t_g , 225 mm L_b and 7.5 t_g , and 300 mm L_b and 10 mm t_g . The groove widths were considered based on two minimum requirements: 10 mm thickness was to satisfy the ACI requirements [59] and 7.5 mm thickness was to satisfy the fib bulletin 90 requirements [60]. The specimen with 150 mm L_b and 10 t_g showed a maximum load of 41.9 kN, with 150 mm L_b and 7.5 mm t_g showed a maximum load of 45.1 kN, with 225 mm L_b and 10 mm t_g showed a maximum load of 53.3 kN, with 225 mm L_b and 7.5 mm t_g showed a maximum load of 58.6 kN, and with 300 mm L_b and 10 mm t_g showed a maximum load of 62.8 kN. All specimens with 10 mm t_g failed at the FRP-adhesive interface whereas all specimens with 7.5 mm t_g demonstrated a failure in concrete cohesion, as shown in Fig. 4. The different shear stiffness ($t_a \times G_a$, where t_a is the

adhesive thickness and G_a is the adhesive's shear modulus) of the adhesive was responsible for this difference in failure. The strain transmitted to the concrete is lower when the adhesive's shear stiffness is higher and vice versa.

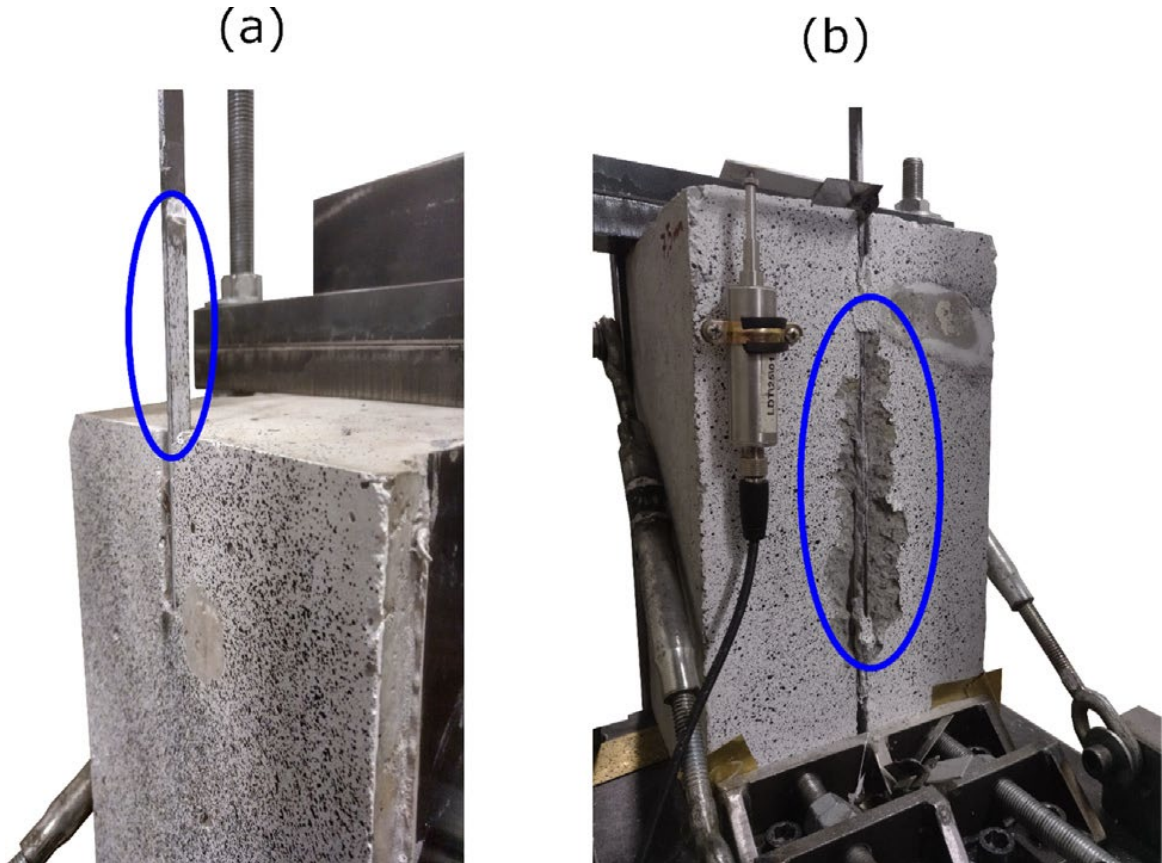


Figure 4: (a) FRP-adhesive interface failure and (b) concrete cohesive failure. Reprinted from Gomez et al. [57]

Another study was conducted to examine the behavior of NSM FRP-concrete bond by single-shear test [61]. Fig. 5 shows the setup of the single-shear test performed with several bonded lengths and FRP installation depths. Using MBrace Epoxikleber 220 as a structural adhesive. The bond failed at FRP-adhesive surface at a maximum load of 27.87 kN with 90 mm L_b and 0 mm installation depth. The FRP ruptured at a maximum load of 35.36 kN with 120 mm L_b and 0 mm installation depth. For installation depth equal to 6 mm, the FRP ruptured at a maximum load of 33.90 kN and 34.46 kN with 90

mm and 120 mm L_b , respectively. Also, for installation depth equal to 12 mm, the FRP ruptured at a maximum load of 35.63 kN and 37.92 kN with 90 mm and 120 mm L_b , respectively.

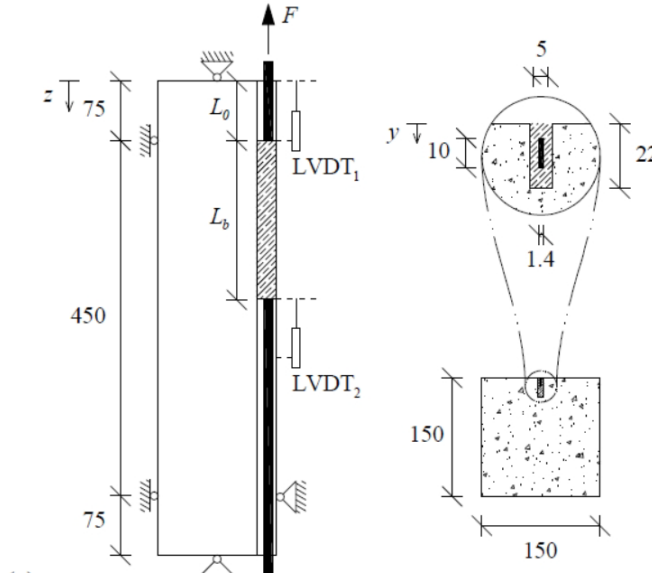


Figure 5: Single-shear test setup. Reprinted from Costa and Barros [61]

2.3.2 NSM FRP Double-Shear Bond Test

The double-shear test was evaluated in this study also [61]. Fig. 6 shows the setup of the test with one bonded length of 300 mm. Three specimens were tested and failed by concrete fracture with an average maximum load of 39.3 kN.

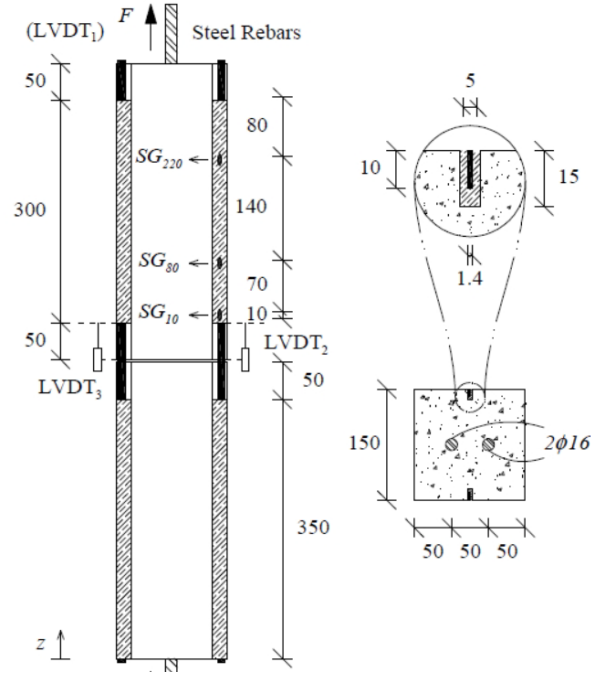


Figure 6: Double-shear test setup. Reprinted from Costa and Barros [61]

2.3.3 NSM FRP Beam Test

The beam test for NSM FRP-concrete bond was evaluated in this study [61]. Considering three bond lengths: 60 mm, 90 mm, and 120 mm for the beam bending test, the specimen's configuration is shown in Fig. 7. All failure modes demonstrated were debonding at the FRP-adhesive interface. An average maximum load of 18.7 kN, 23.9 kN, and 27.7 kN were resulted for beam specimens with 60 mm, and 90 mm, and 120 mm, respectively. Also, an average loaded-end slip of 0.43 mm, 0.79 mm, 2.65 mm were resulted for beam specimens with 60 mm, and 90 mm, and 120 mm, respectively.

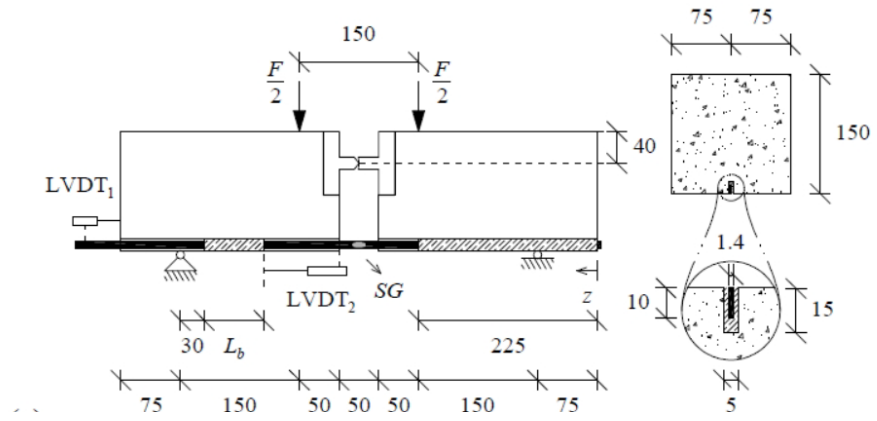


Figure 7: Beam bending test setup. Reprinted from Costa and Barros [61]

CHAPTER 3

EVALUATION OF NSM FRP-CONCRETE BOND

3.1 Evaluation of Bond Between NSM FRP and RAC

3.1.1 Materials

3.1.1.1 Cement and Fine Aggregate

Type I Portland cement (ASTM C150 [62]) with a specific gravity of 3.15 and a Blaine fineness of 4500 cm²/g was used in all the concrete mixtures. For fine aggregate, dune sand with a specific gravity of 2.53 was adopted.

3.1.1.2 Coarse Aggregate

Crushed natural limestone with a specific gravity of 2.6 was used as the natural coarse aggregate. The recycled concrete aggregate (RCA) was obtained from the Saudi Investment Recycling Company (SIRC) – a local source of construction and demolition waste (CDW) with state-of-the-art processing plants. The source of the CDW used to produce the RCA was pure structural concrete, excluding the finishing material waste. It underwent several processes to produce the RCA, including the tertiary crushing process, to guarantee acceptable results. This resulted in RCA with different diameters consisting of gravel with mainly 4 – 10 mm and 10 – 20 mm sizes. Fig. 8 (a – c) shows the CDW dump, the RCA after the crushing process, and the natural coarse aggregate (NCA), respectively.

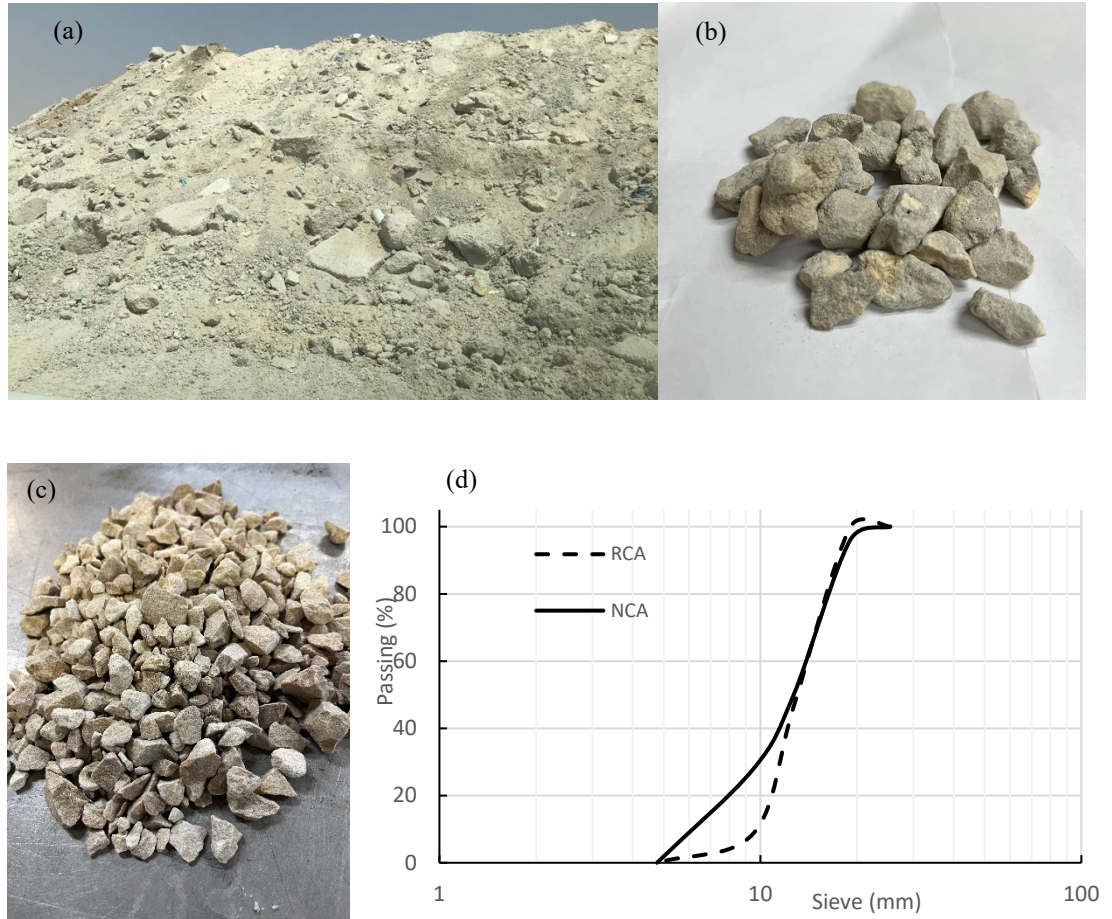


Figure 8: Dump/source of the construction and demolition waste (a), processed RCA (b), NCA (c), and particle size distribution of RCA and NCA (d)

The density and water absorption of the coarse aggregates were measured according to ASTM C127 [63] and the results are reported in Table 1. The RCA demonstrated lower density and a higher water absorption capacity than those of the NCA, which agrees with the findings in the literature. The particle-size distributions, shown in Fig. 8(d), for the RCA and NCA in this study were produced according to ASTM C33 [64].

Table 1 Material Properties

Aggregate			Concrete			
Type	Density (kg/m ³)	Water absorption (%)	Type	f_c (MPa)*		
NCA	2.6	1.9	RCA0-SF0-REF	39.4		
RCA	2.3	3.0	RCA0-SF0	14.8		
			RCA100-SF0	12.2		
Steel Fiber			RCA100-SF1	11.6		
Type	l_s (mm)	d_s (mm)				
Hooked-end	60	0.75				
FRP*						
Type	A_f (mm ²)	f_u (Mpa)	E_f (Gpa)	Rupture strain		
GFRP	71	910	50	0.0182		
Adhesive**						
Type	Density (kg/l)	f_{ac} (Mpa)	f_{at} (Mpa)	E_a (Gpa)	Interfacial tangential resistance (Mpa)	
					Concrete	Steel
Sikadur 30 LP	1.65	> 85	17	10	> 4	> 22

* Reported by the manufacturer [65]

** Reported by the manufacturer [66]

3.1.1.3 Concrete

Nine different concrete mixtures were prepared, with RCA's incorporation ratio, steel fiber content, and vibration method (manual tamping versus machine vibration to produce low- and normal-strength concretes, respectively) being the three variables studied. First group of the mixtures consisted of 0%, 30%, 60%, and 100% RCA replacement for each of 0% and 0.5% steel fiber content in terms of the specimen's

volume. All these mixes were vibrated using manual tamping that produced low-strength concretes. The use of steel fiber is made In some of the mixes to see if the low-quality concrete will benefit from some enhancement due to its incorporation. The second group of mixtures consisted of one having 100% RCA and 1% steel fiber (representing a manually vibrated low-strength concrete with higher steel fiber content), and another representing a normal-strength machine vibrated concrete produced using NCA containing no steel fibers. The latter, designated as RCA0-SF0-REF is used to compare its NSM FRP-concrete bond performance with that of the low-strength concrete mixes. Table 2 demonstrates the specimens' distribution with their mix proportioning. The density of cement, water, sand were 433 kg/m³, 210 kg/m³, and 630 kg/m³, respectively, for all mixtures. 'RCA' followed by a numerical value in the specimen's name represents the percentage of aggregate replacement using the recycled aggregate in a mixture, whereas 'SF', followed by a second numerical value, represents the steel fiber content in the mix. Representative compressive strength values for the upper and lower limits of the mixes were determined, as shown in Table 1.

Table 2 Concrete mix design/specimen distribution

Mix/Specimen	Steel fiber (%)	RCA (%)	NCA (kg/m3)	RCA (kg/m3)	Steel Fiber (kg/m3)
RCA0-SF0-REF	0	0	1173.0	0.0	0.0
RCA0-SF0	0	0	1173.0	0.0	0.0
RCA0-SF0.5	0.5	0	1173.0	0.0	39.5
RCA30-SF0	0	30	821.1	351.9	0.0
RCA30-SF0.5	0.5	30	821.1	351.9	39.5

RCA60-SF0	0	60	469.2	703.8	0.0
RCA100-SF0	0	100	0.0	1173.0	0.0
RCA100-SF0.5	0.5	100	0.0	1173.0	39.5
RCA100-SF1	1	100	0.0	1173.0	79.0

3.1.1.4 Fiber Reinforced Polymer Bars

As shown in Fig. 9(a), 10 mm diameter sand-coated glass FRP (GFRP) bars with spirally wound surfaces, for enhanced bond with the concrete, was used in this work. Table 1 reports the bar's cross-sectional area, A_f , tensile strength, f_u , modulus of elasticity, E_f , and the rupture strain. The values of the GFRP mechanical properties were provided by the manufacturer [65] based on the nominal cross-sectional area.

3.1.1.5

3.1.1.5 Steel Fibers

Locally manufactured 60 mm long hooked-end steel fibers – Fig. 9(b) – were used in the study, and their technical specifications are reported in Table 1 according to the manufacturer (Forcotech) [67]. The steel fibers of 0.75 mm diameter possessed tensile strength exceeding 1100 Mpa made from low carbon steel.



(a) Spirally wound surface GFRP bar with small degree of sand-coating



(b) Hooked end steel fibers

Figure 9: GFRP and steel fibers used in the study

3.1.2 Specimens Preparation

Concrete block specimens, each of dimensions 400 mm x 150 mm x 150 mm, were prepared in pre-designed formworks for producing eccentrically grooved specimens suitable for NSM FRP application in single shear bond test configuration with the bonded length fixed at 200 mm. Two block specimens for each of the nine mixtures were produced. The specimens were demolded after 24 hours of casting and submerged to cure in water for a period of 28 days. At the end of the curing period, the GFRP bar (350 mm long each) was embedded in each of the roughened specimen's groove with Sikadur 30-LP [66] as the structural adhesive. Fig. 10(a) shows a schematic representation of the so prepared NSM FRP-concrete bonded specimen configuration. A specially designed steel grip was attached at the unbonded end of the FRP bar to facilitate its gripping and prevent damage to it due to the high compressive stresses caused by the machine gripping jaws

during testing. An expansive mortar sandwiched between the FRP bar end and a hollow square steel grip was used in the attachment as schematically shown in Fig. 10(a).

3.1.3 Test Setup

The single-shear pull-out test of the NSM FRP-concrete bond was performed using universal testing machine loaded according to the test setup shown in Fig. 10(b). A specially designed clamp was attached at the bottom fixed grip of the testing machine to provide the base for the concrete. A T-beam was also designed and used to restrain the upper surface of the concrete block using four rods extending from the base at the fixed grip. A linear variable differential transducer (LVDT) was used to record the slip response during the test – Fig. 10 (a and b).

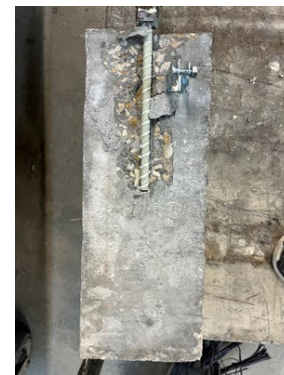
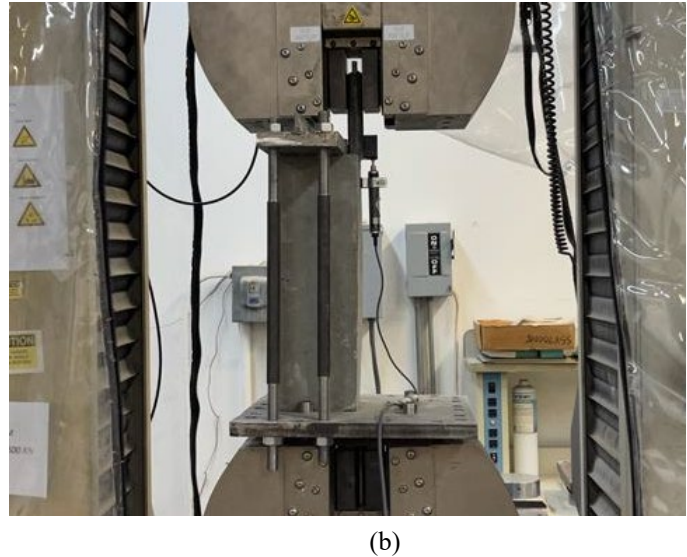
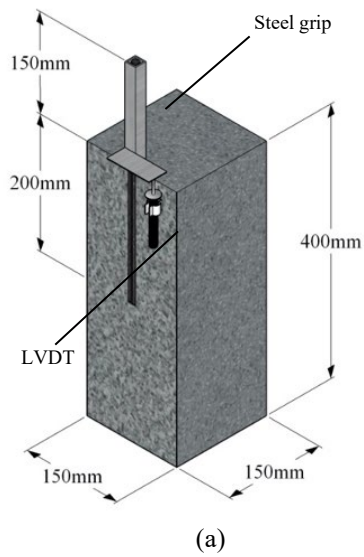


Figure 10: Direct pull-out single-shear NSM FRP-concrete bond test: (a) Specimen details (b) Test setup (c) Typical debonding failure in low-strength concrete [concrete-adhesive interface with degrees of concrete damage] (d) Typical debonding failure in normal-strength concrete [FRP-adhesive interface with partial substrate damage]

3.1.4 Gripping Technique

One of the main issues that faces researchers in NSM bond testing is the damage of the FRP end when gripped by the testing machine. Special methods have been proposed to overcome this issue, but questions over their economic feasibility are raised. In this experiment, a convenient method was used to overcome the gripping issue for a

better testing. An expansive mortar, a non-explosive demolition technology that is used to crack enormous stones in quarrying, is used. When holes are drilled in stones, a slurry of the expansive mortar is poured to be activated after an amount time and expanded to crack the stones. As shown in Fig. 10(a), the FRP bars/strips were prepared in a steel case to, also, increase the grip quality during the test, in which the clamps would not damage the FRP while gripping. After that, the grip was filled by the expansive mortar. After the mortar is cured after one day, the steel grip is holding the FRP with high shear resistance considered between the two elements as a result of the pressure applied on the wall of the grip by expanded mortar. Fig. 11 shows the typical casting of expansive mortar inside the steel grip.

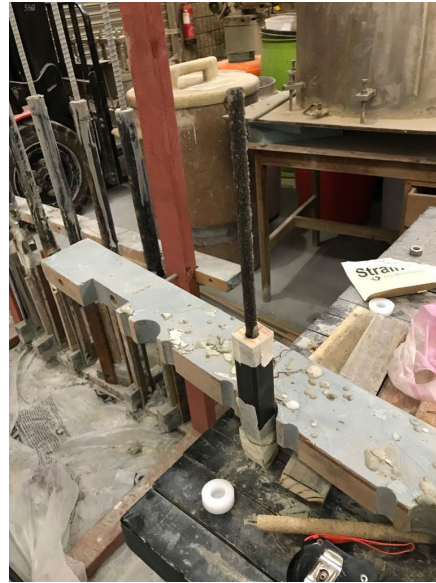


Figure 11: Casting of expansive mortar inside steel grip

To validate the use of the expansive mortar along with the steel grip, the system was tested in the servo-hydraulic machine for the same steel grip length and size on a CFRP bar. Fig. 12 shows a CFRP bar tested assembled with the expansive mortar and steel grip at the ends. The system failed at the mortar filing at an average ultimate load of

74 kN. Considering the same grip length, the system is allowed to the NSM bond tests performed as the system possessed a higher failure load than the expected loads from all the NSM bond tests.

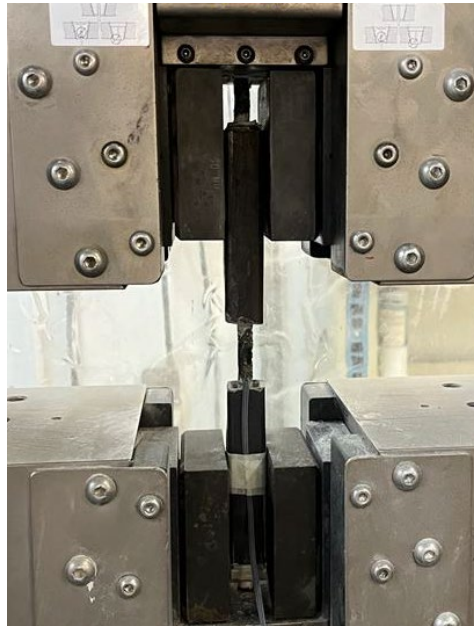


Figure 12: Testing the gripping technique

3.2 Assessment of NSM FRP-concrete Bond Behavior Using Universal Debonding Test Apparatus

3.2.1 Materials

3.2.1.1 Cement and Aggregate

Type I Portland cement (ASTM C150) with a specific gravity of 3.15 was used in every batch of concrete mixture. With a specific gravity of 2.53, dune sand was used into

all concrete compositions. Crushed natural limestone aggregates with a specific gravity of 2.6, a water absorption of 1.9, and a density of 2.6 kg/m³ were utilized as coarse aggregate.

3.2.1.2 Fiber-Reinforced Polymers

Three different types of FRP—Carbon FRP (CFRP) bars, Glass FRP (GFRP) bars, and CFRP strips—have been used in the experiment to evaluate the NSM bond using various FRP types and test methodologies. The 10 mm diameter CFRP and GFRP bars utilized in this experiment have sand-coated and spirally wound surfaces to provide stronger contact with the concrete. The CFRP strips are several rectangular strips glued together forming a square CFRP rod having an equivalent cross-sectional area to that of bars. Fig. 13 shows the three types of NSM FRP used in this experiment. Table 3 lists the mechanical properties of the different types of FRP provided by the manufacturer [65,68,69].

Table 3 Properties of FRP bars/strips [65,68,69]

FRP	$A_f(\text{mm}^2)$	f_u (MPa)	E_f (GPa)	Rupture Strain
Circular Glass FRP	75.8	910	50	0.0182
Circular Carbon FRP	75.8	2172	124	0.0175
Rectangular CFRP	~ 75.8	3100	170	0.0180



Figure 13: NSM FRP types: CFRP strip (left), GFRP bar (middle), and CFRP square bar (right)

3.2.2 Mix Design

All specimens were subjected to a one-mix design in order to evaluate the various bond test methods for a single concrete mixture. The concrete mixture was proportioned using the ACI absolute volume technique to attain a 40 MPa characteristic strength. The water-to-cement ratio associated with this non-entrained concrete characteristic strength is 0.38. The final mix proportioning, the average compressive strength (f_c), and the average tensile strength (f_{ct}) are demonstrated in Table 4.

Table 4: Mixture proportioning

Batch	Cement (kg/m ³)	Fine Aggregate (kg/m ³)	Coarse Aggregate (kg/m ³)	Water-to- Cement ratio	f_c (MPa)	f_{ct} (MPa)
1	418	702	1039	0.38	39.42	3.9

2	418	702	1039	0.38	37.61	3.6
3	418	702	1039	0.38	39.6	4.58

3.2.3 Specimens' Production of NSM Bond Tests

The dimensions of single-shear and double-shear test specimens were 400 mm x 150 mm x 150 mm – Fig. 14(a,b, and c), whereas beam-bend test specimens' dimensions were 200 mm x 150 mm x 150 mm – Fig. 14(d). The specimens were created in pre-designed formworks that included a wood stick in the middle of one of the surfaces for the single-shear and beam-bend test specimens, and two on the middle of two surfaces for symmetric double-shear test specimens to create an NSM groove following demolding, as shown in Fig. 15. After demolding, the NSM grooves of all test specimens were filled by the structural adhesive – Sikadur 30 LP. Then, the FRP bars/strips were immersed inside the fillings and the adhesive was flushed by a scraper, as shown in Fig. 16.

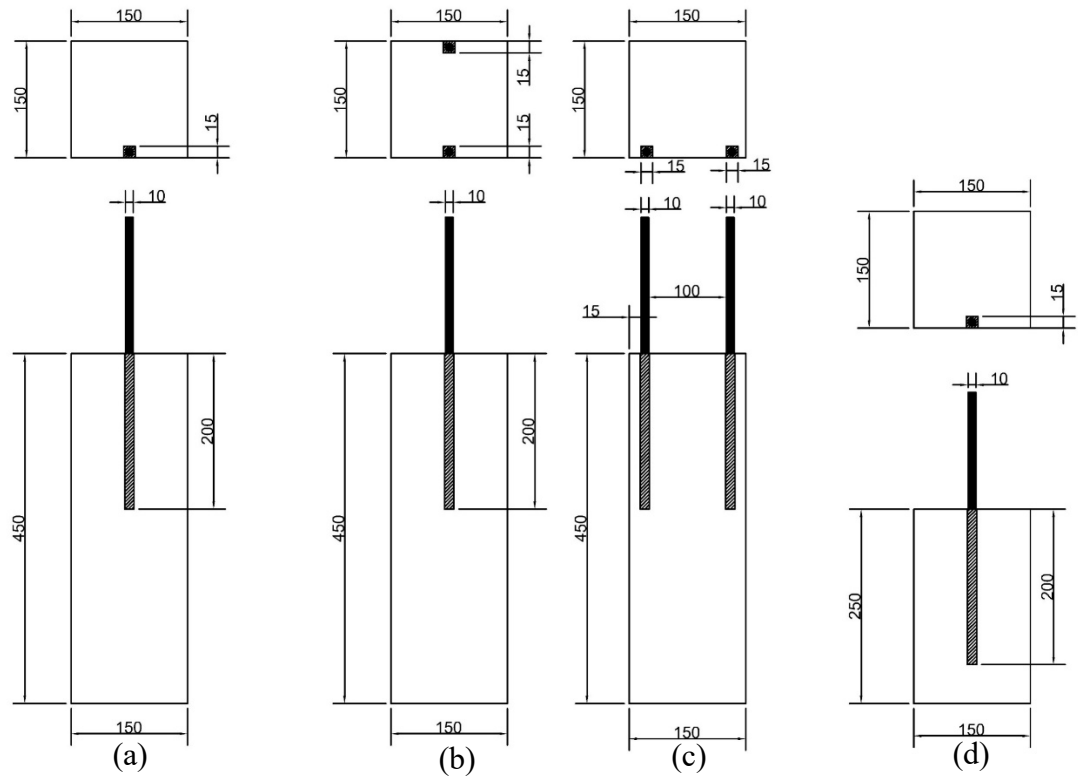


Figure 14: Details of single-shear (a), double-shear (b), asymmetric double-shear, and beam-bend (c) specimens



(a)



(b)



(c)

Figure 15: Formwork for single-shear (a) and double-shear (b), and asymmetric double-shear test specimens (c)



Figure 16: Installation of FRP bars/strips

The NSM grooves for all specimens possess the dimensions of $1.5d_b \times 1.5d_b$, where d_b is the diameter of the NSM FRP, according to the preferred NSM groove dimension in ACI PRC-440.2-17 [70]. The bonded length for all specimens is 200 mm, as shown in Fig. 3. Table 3 summarizes the distribution of the specimens. “S”, “D”, “B”, and “E” abbreviation stands for single-shear, double-shear, beam-bend, and edge-distance-effect single-shear, respectively. “G”, “C”, and “RC” stands for specimens with GFRP bars, CFRP bars, and CFRP strips, respectively.

Table 5 Specimens Distribution

Specimen	Test Method	FRP Type	Cross-section shape	FRP Area (mm²)	Quantity
S-G	Single-shear	Glass	Circle	78.5	2
S-C	Single-shear	Carbon	Circle	78.5	1
S-RC	Single-shear	Carbon	Rectangular	78.5	1
D-G	Double-	Glass	Circle	78.5	2

	shear				
D-C	Double-shear	Carbon	Circle	78.5	1
D-RC	Double-shear	Carbon	Rectangular	78.5	1
B-G	Beam-bend	Glass	Circle	78.5	2
B-C	Beam-bend	Carbon	Circle	78.5	2
DA-C	Double-shear (asymmetric)	Carbon	Circle	78.5	1
B-RC	Beam-bend	Carbon	Rectangular	78.5	2
E-G	Edge distance	Glass	Circle	78.5	4
E-C	Edge distance	Carbon	Circle	78.5	2
E-RC	Edge distance	Carbon	Rectangular	78.5	2

3.2.4 NSM Bond Test Methods

3.2.4.1 Universal Debonding Test Apparatus for FRP-concrete system

The test apparatus [40] used for this study takes into account the challenges to standardize the dissimilar NSM bond test methods. It can be utilized to examine the interfacial bonds between concrete and FRP by forcing a debonding process in which the apparatus acts as a clamp. To explore the bond behavior and debonding failures of NSM

FRP, a variety of testing sets, and methodologies are available. The test equipment specified in the current disclosure is constructed of three structural blocks: a primary structural block with a receiving slot, a secondary structural block, and an attachment mechanism that may be adjusted to perform various test types on an FRP-concrete bond test specimen. Single-shear testing, double-shear testing, mixed-mode testing, tension pull-off testing, and beam-bend testing all require the use of the primary structural block and the secondary structural block. Fig. 17 shows a schematic representation of the apparatus.

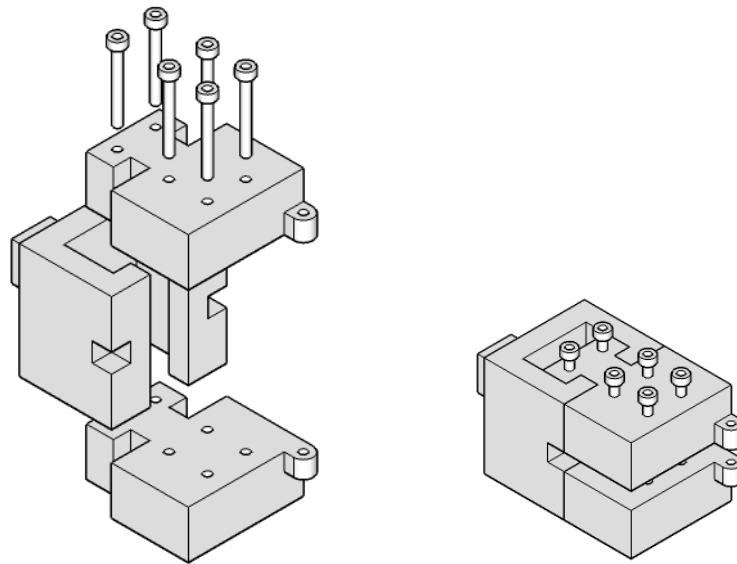


Figure 17: A schematic representation of the Universal Debonding Test Apparatus [40]

For a single-shear test configuration of the NSM FRP-concrete, the specimen with the FRP attached is installed in which the FRP is in the middle of the apparatus closure. Therefore, the attachment mechanism tightens the secondary block inside the closure and sandwiches the FRP, as shown in Fig. 20(a). To perform the test on the specimen, the concrete is fixed whereas the apparatus holding the free-end FRP is pulled by the servo-hydraulic machine.

For the double-shear test configuration, the specimen is installed by flushing the surfaces of the apparatus with the specimen's surfaces in which the symmetrical FRPs are sandwiched at the edges of the closure, as shown in Fig. 20(b). However, asymmetric double-shear test configuration manages the apparatus' closure by sandwiching the FRPs that are on the same surface of the concrete, as shown in Fig. 20(c). Once constructed, the assembly undergo the test by pulling similar to the single-shear test.

Moreover, for the beam-bend test, the specimen is installed by flushing the edges of the apparatus with the specimen in which the single FRP is sandwiched at the edges of the closure, as shown in Fig. 20(e). With the apparatus representing another connected concrete specimen, the whole setup is loaded as a beam.

3.2.5 Single-Shear Pull-out Test

The Universal Debonding Test Apparatus was used to conduct the single-shear pull-out test, and it was set up in accordance with Fig. 20(a). The free-end slip was recorded by the displacement transducers (LVDT) that are fixed on the concrete by U-clamps. A steel plate was attached to the steel case around the FRP to be projected for the

LVDT to reach. Strain gauges (SGs) are installed at every 90 mm along the bonded length. Fig. 18(a) and Fig. 19(a) show the instrumentation details of the specimen.

The concrete specimen was installed at a steel base fixed in the testing machine where a T-plate was mounted on the top surface of the specimen and tightened by rods attached from the steel base. The Universal Debonding Test Apparatus was installed to the moving part of the testing machine to be clamped to the FRP end. Fig. 20(a) shows the full setup of the single-shear test loaded at a rate of 0.1 mm/s.

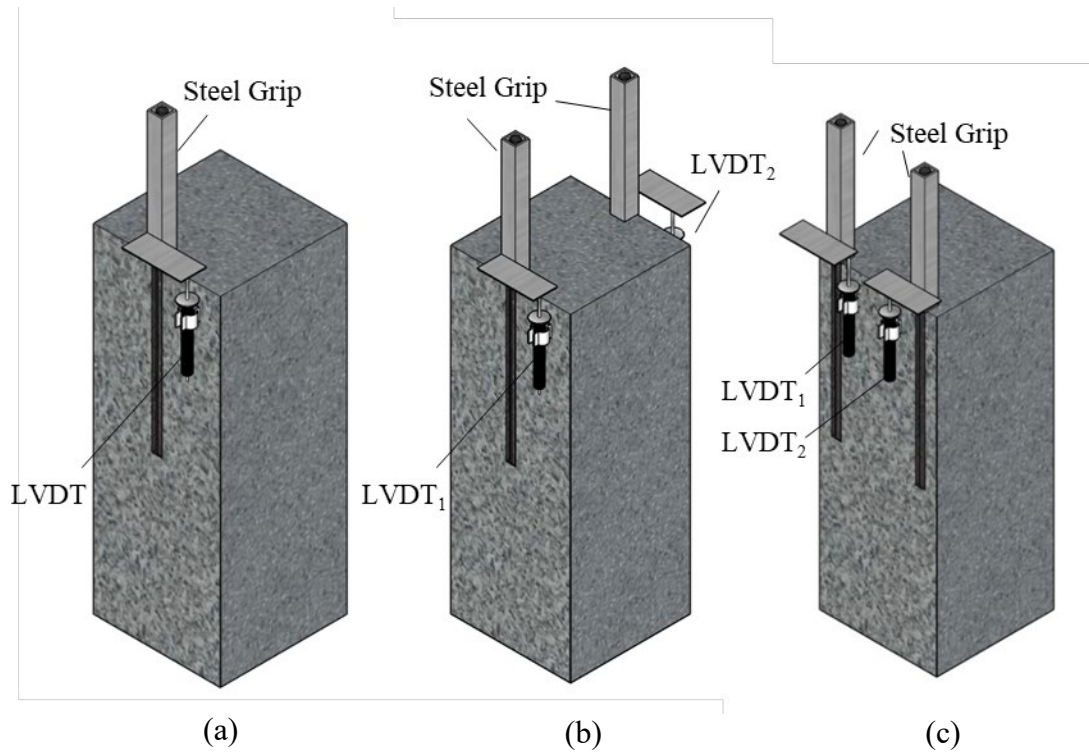


Figure 18: LVDT details of the single-shear test specimen before testing for single-shear (a), double-shear (b), and asymmetric double-shear (c) test

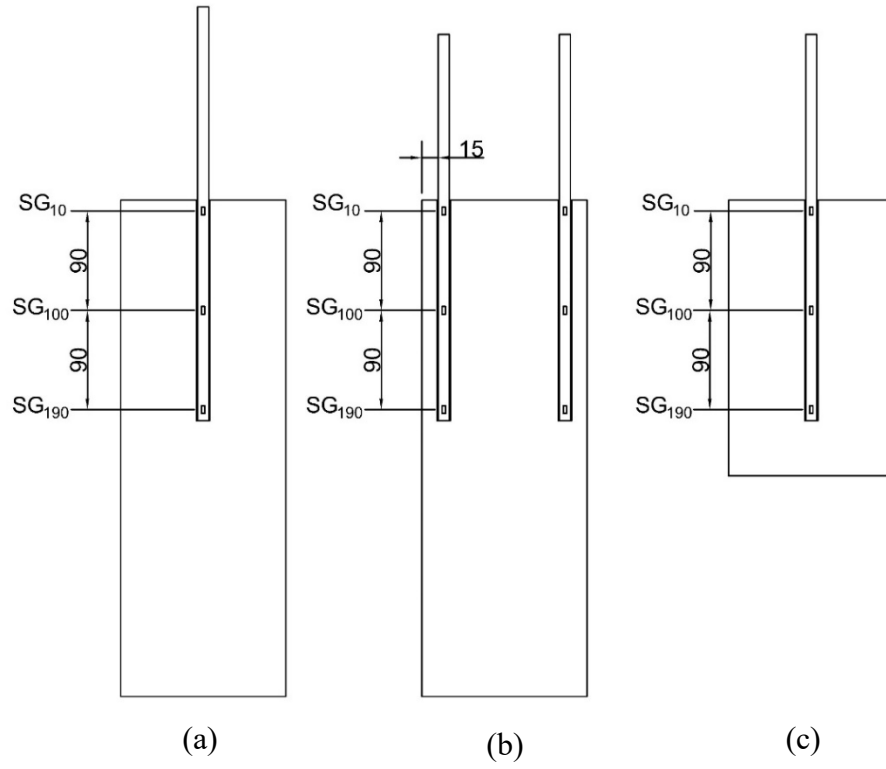
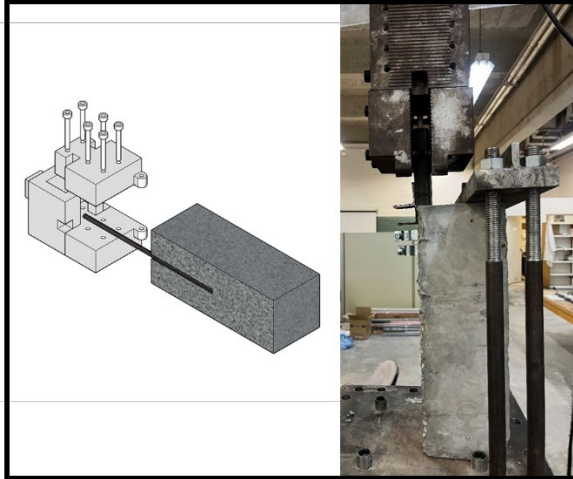
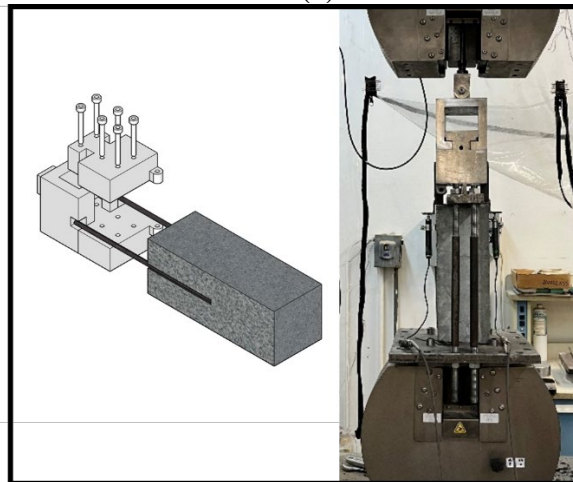


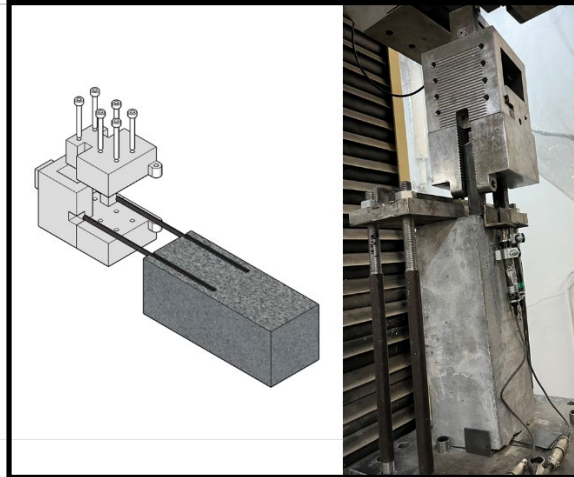
Figure 19: Strain gauges placement for single-shear and double-shear (a), asymmetric double-shear (b), and beam-bend (c) test



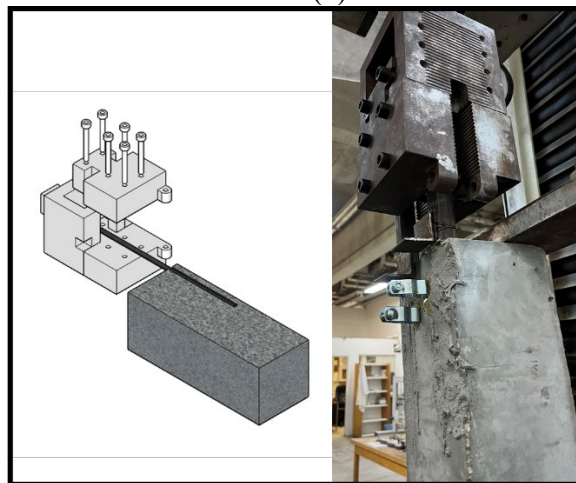
(a)



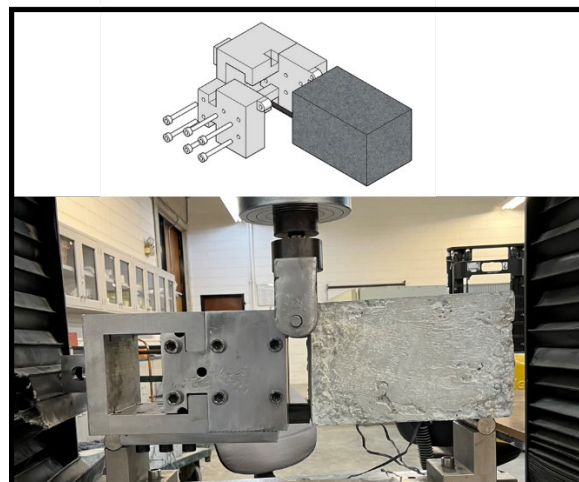
(b)



(c)



(d)



(e)

Figure 20: The setup of single-shear (a) double-shear (b) asymmetric double-shear (c) insufficient edge distance (d) beam bend (e) test

3.2.6 Double-Shear Pull-out Test

For the double-shear test, the specimen's arrangement followed the scheme of the single-shear test where the LVDTs are placed on the side of both grooves to reach the steel plates that are attached to the FRP's steel cases. Also, the SGs are located at a spacing of 90 mm along the bonded length starting from 10 mm of the loaded end. Fig. 18(b) and Fig. 19(a) demonstrate the specimen's instrumentation details.

The double-shear bond test appears fully in Fig. 20(b) before starting, with a loading rate of 0.1 mm/s. As the concrete was fixed into the steel base as followed by the single-shear test, the top surface of the specimen had the t-beam mounted between the FRPs.

3.2.7 Beam Test

The beam-bend test specimen, as depicted in Fig. 20(e), is assembled with the Universal Debonding Test Apparatus horizontally representing a beam. The arrangement of bending for this setup is a three-point bending. SGs were placed at a spacing of 90 mm from 10 mm of the loaded end, as shown in Fig. 19(c). Fig. 21 shows the assembly of the beam-bend test with the Universal Debonding Test Apparatus. The full setup of beam-bend testing is demonstrated in Fig. 20(e) where the two blocks were loaded at a rate of 0.5 mm/s.



Figure 21: Typical assembly of beam-bend test specimen with the apparatus

3.2.8 Asymmetric Double-Shear Test

Fig. 20(c) shows the setup for testing asymmetric double-shear test specimens using the Universal Debonding Test Apparatus. The LVDT placement details and the strain gauges installation are depicted in Fig. 18(c) and Fig. 19(b), respectively.

3.2.9 Decreased Edge Distance Single-Shear Test

The details of the specimen tested for an insufficient edge distance and its instrumentation details are similar to that of the asymmetric double-shear test shown in Fig. 18(c) and Fig. 19(b). However, the apparatus only clamps one FRP bar in this test. Therefore, this test studies the bond behavior of a single-shear specimen with a decreased concrete edge distance of 15 mm. For every concrete sample, two FRP bars/strips were installed to decrease the number of concrete samples cast. Thus, the test is performed two times for every concrete sample. Fig. 20(d) shows the setup using the Universal Debonding Test Apparatus to test the specimens for a decreased edge distance.

CHAPTER 4

EVALUATION OF PUNCHING SHEAR BEHAVIOR OF

STEEL FIBER-REINFORCED RAC SLAB

STRENGTHENED WITH NSM FRP

4.1 Mix Proportioning

All materials specified for this chapter were described in subsection 3.1.1. For the CFRP bar, its properties are described in subsection 3.2.1.2. Four different mix designs were performed by fixing the type of cement used. The reference concrete, named RAC0-S0, was used with NCA without steel fibers. Varying the type of aggregate and the steel fibers' incorporation by volume, the other three concrete mixes include RCA with 0% steel fibers, NCA FRC with 1% steel fibers, and RCA FRC with 1% steel fibers, referred to as RAC100-S0, RAC0-S1, and RAC100-S1, respectively. Using a total water/cement ratio of 0.5, the reference concrete was proportioned following ACI absolute volume method. To produce RAC mix designs, the NCA of 4 mm – 20 mm in the RAC0-S0 were replaced simultaneously with RCA by unit weight. Steel fibers were added as a percentage of the specimen's volume. Table 6 shows the mix design used for the four groups.

Table 6 Mix Proportioning

Mix	Cement (kg/m³)	Water (kg/m³)	NCA (kg/m³)	RCA (kg/m³)	Sand (kg/m³)	Steel Fiber (%)	Steel Fiber (kg/m³)
RAC0-S0	433	216	1173	0	630	0	0
RAC100-S0	433	216	0	1173	630	0	0
RAC0-S1	433	216	1173	0	630	1	79
RAC100-S1	433	216	0	1173	630	1	79

4.2 Test Specimens

4.2.1 Specimens Distribution

In this experiment, a total of five reinforced concrete slab-column specimens were produced for testing their punching shear resistance. Two FRP bar material types, namely, CFRP and GFRP, were used as variables in the NSM strengthening method. The five specimens include different permutations of the four mix designs presented in Section 4.1 and the FRP material type. Specimens were named after the mix design IDs followed by “G” or “C” to refer to the GFRP or CFRP bar strengthening, respectively. As a result, RAC0-S0-G, RAC100-S0-G, and RAC0-S1-G were each strengthened with GFRP bars, while RAC0-S1-C and RAC100-S1-C were each strengthened with CFRP bars.

4.2.2 Scaling

The specimens consisted of a square slab and column stub at the center of the slab to represent the behavior of the slab-column connection in real applications. The specimen size was scaled by 1/2 relative to the full-scale structure. Dimensions of the slab and the column stub were 1500 mm x 1500 mm x 120 mm and 150 mm x 150 mm x 150 mm, respectively, as shown in Fig. 22.

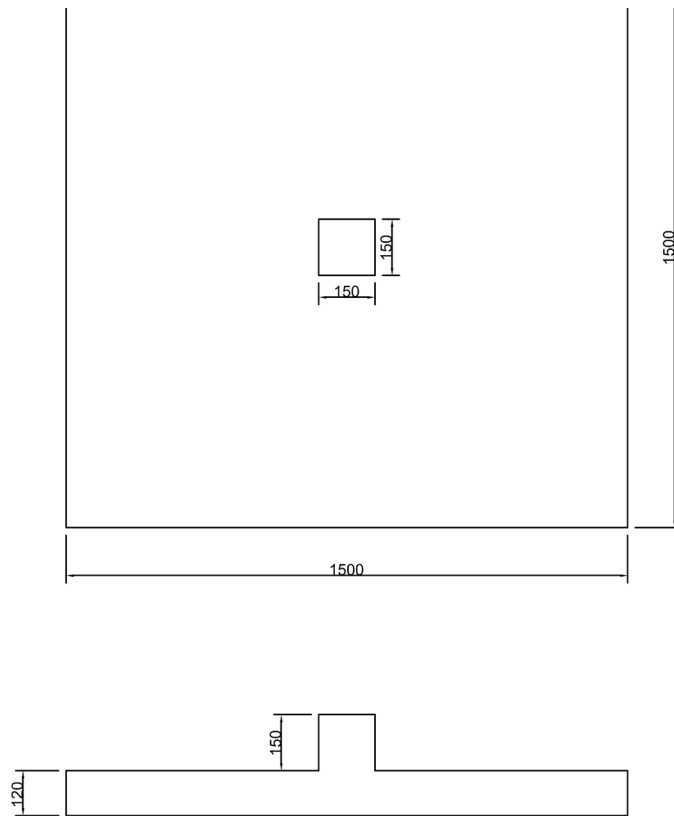


Figure 22: Specimen's Dimensions

4.2.3 Reinforcement Layout

Steel reinforcement was designed according to a real application example. The reinforcement having a characteristic strength of 500 MPa was installed in the specimen's tension and compression regions. Mesh for the tension reinforcement had a diameter of 8

mm and a spacing of 100 mm, whereas the compression region's reinforcement had a spacing of 200 mm and a diameter of 6 mm. Four steel bars with a diameter of 10 mm were installed for the column stub. Stirrups with a spacing of 90 mm and a diameter of 6 mm were used along the column stub. Fig. 23 shows the specimen's reinforcement layout with the placement of strain gages labelled SGs to denote steel rebar strain gages. A typical final appearance of the specimens fabricated by experienced workers at a construction site is shown in Fig. 24. The strain attached to the steel are also visible in the figure.

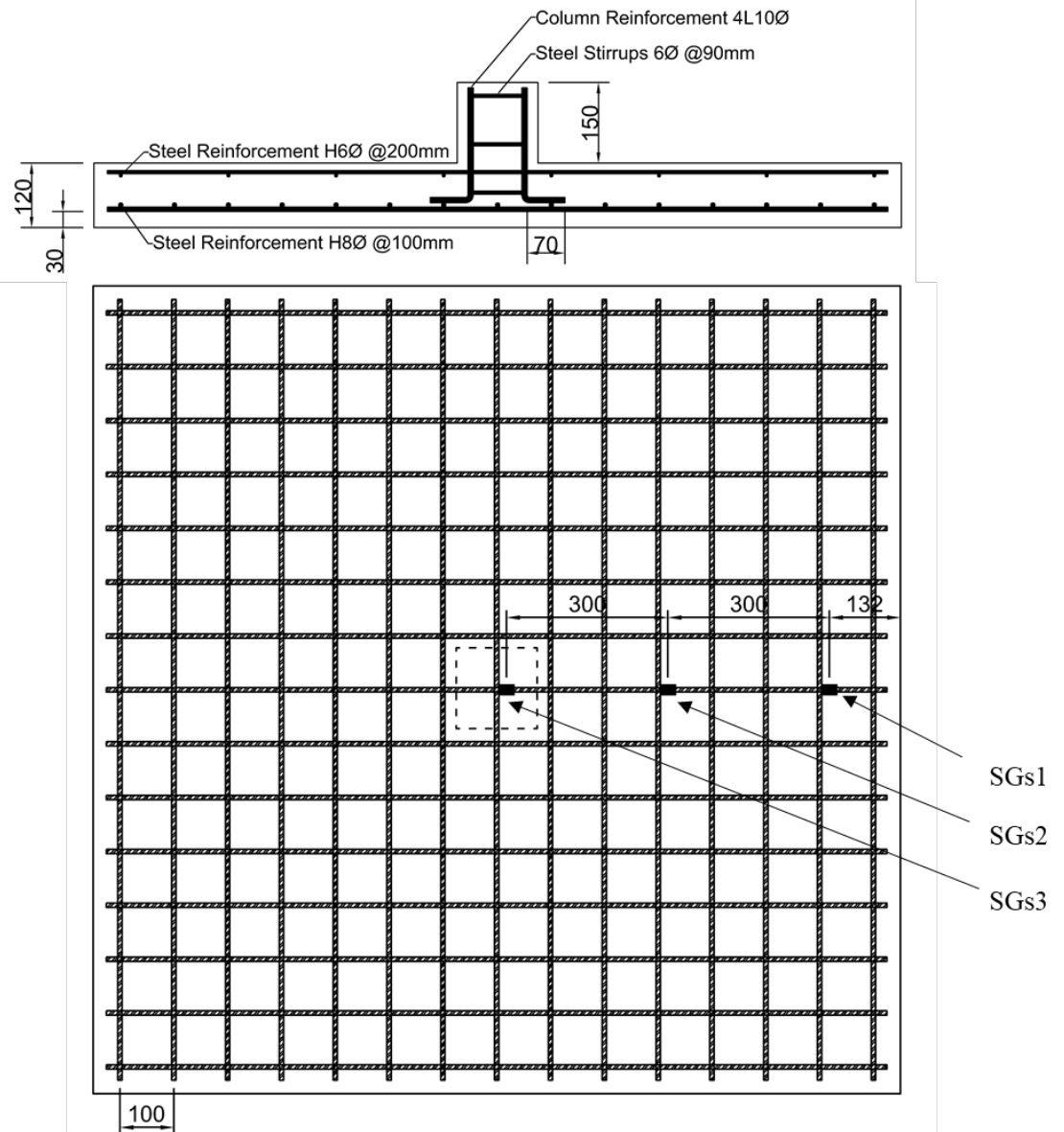


Figure 23: Steel reinforcement details



Figure 24: Typical distribution of the flexural steel reinforcement for the specimens

4.2.4 NSM Strengthening Layout

The NSM FRP strengthening layout for the slab-column specimen was achieved by placing wooden strips (meant to be removed after concrete hardening) so that they produce grooves with dimensions of $1.5d_b \times 1.5d_b$, where d_b is the NSM FRP bar diameter. Despite the use of wooden strips contrary to the use of concrete saw in creating grooves on existing building members after the concrete hardening, as it will be seen in the results (Section 6.2), the failure of all specimens happened with accompanying FRP-adhesive interface debonding rather than adhesive-concrete interfacial failure. This,

therefore, precludes the concern for weaker groove material for not using a concrete saw. As shown in Fig. 25, four FRP bars were used in the tension side of each specimen. The figure also illustrates the  strain gage location denoted as ‘SGf’ as a designation for FRP strain gage.

After curing the specimens for a period of 28 days at the construction site, they were hauled to the heavy structures laboratory [Fig. 26(a)] for testing. Fig. 26(c) shows the NSM FRP and the adhesive being laid out after cleaning the groove using a vacuum cleaner in Fig. 26(b). GEOTREE NSM adhesive was used to bond the FRP using a dual-cartridge dispenser for mixing the adhesive’s epoxy and resin followed by the application.

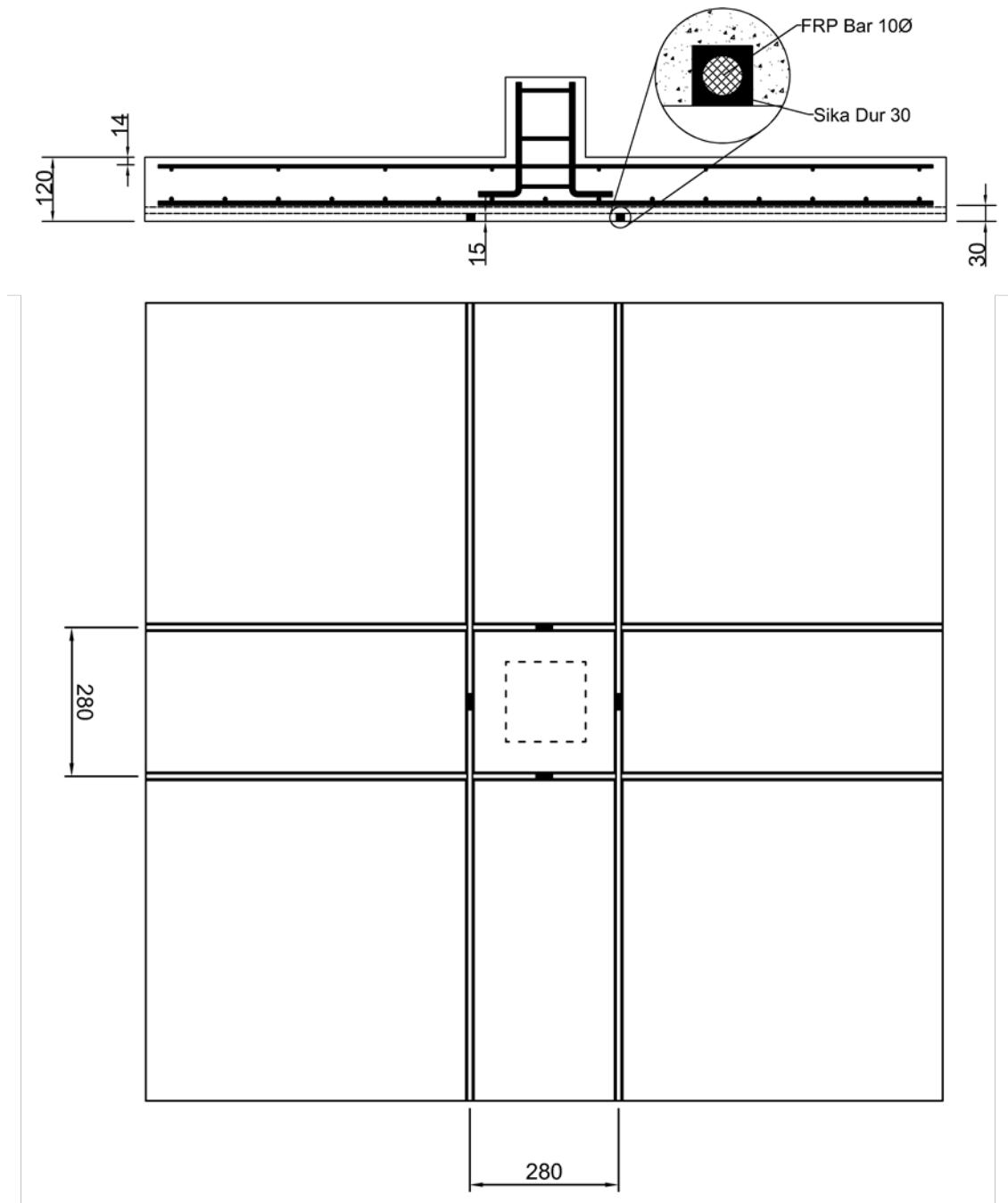
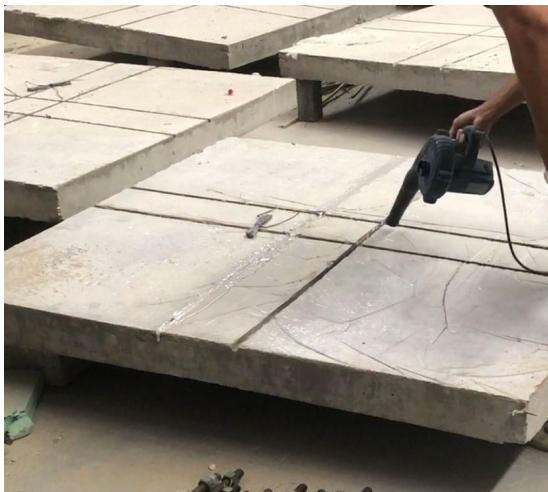


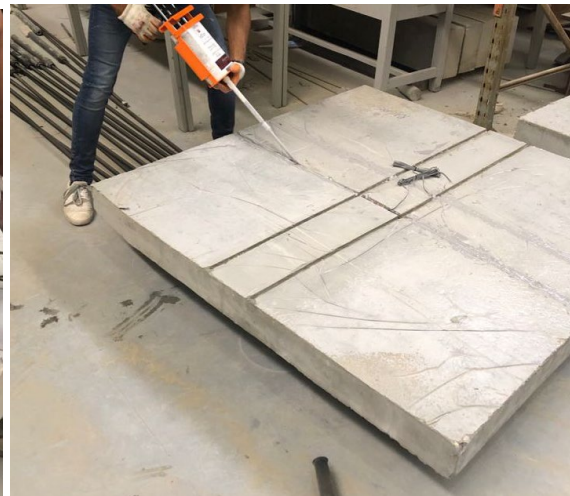
Figure 25: NSM FRP strengthening configuration and strain gage placement



(a) hauled specimens



(b) cleaning the NSM grooving



(c) laying the NSM FRP with adhesive

Figure 26: Specimens and NSM FRP strengthening

4.3 Testing

4.3.1 Compressive Strength Test

After 28 days of curing, compressive strength test was performed according to ASTM C39 [71] on cylindrical concrete specimens of dimensions 150 mm diameter and 300 mm height. A set of three cylinders was tested for each mix.

4.3.2 Punching Shear Test

This test was conducted on the five column-slab specimens to measure several features and attributes of the punching shear behavior. A special steel frame was manufactured to support the specimens against the loading on the column stub. The loading was applied by a hydraulic jack with a load cell mounted on it. Shown in Fig. 27, this setup is mounted under a steel reaction frame to resist the loading from the hydraulic jack. The specially manufactured steel frame was designed to withstand more than the expected ultimate punching shear load by the specimen.

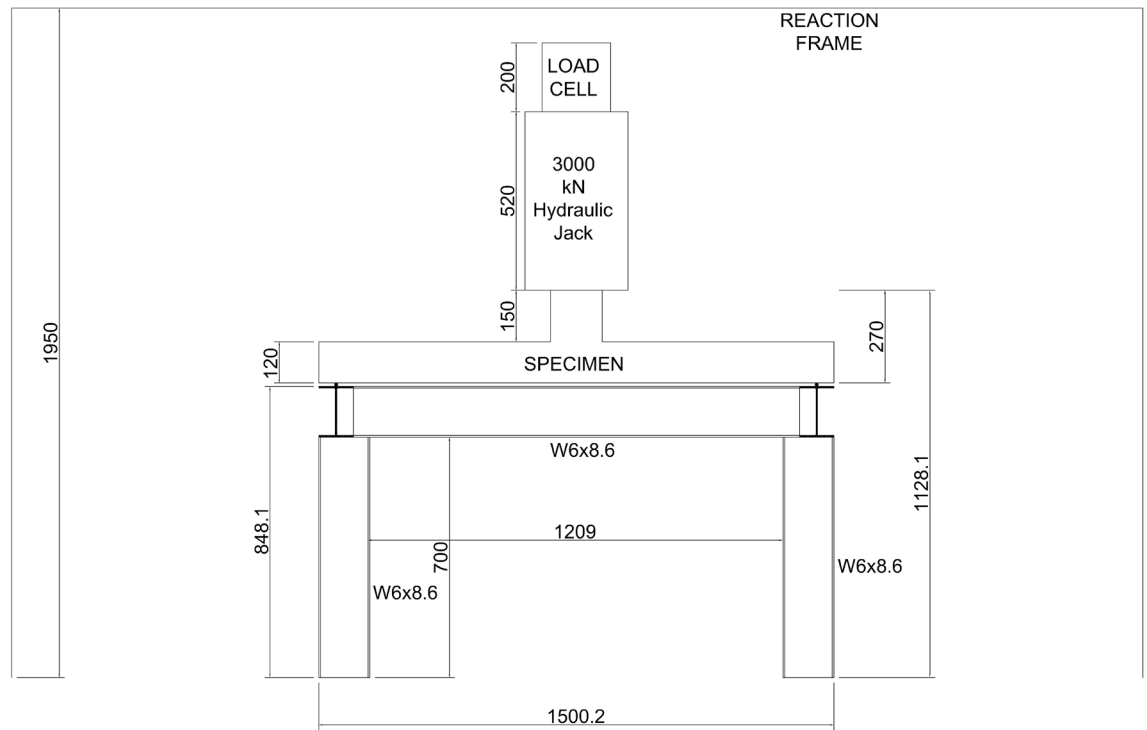


Figure 27: Diagram of the punching shear test setup

Five linear variable differential transducers (LVDT) placed at equal distances along the middle profile of the specimen were used to measure its deflection during the test. In addition, five concrete strain gauges (SGc1, SGc2, SGc3, SGc4, and SGc5) were attached to the bottom surface of the specimen in two lines to capture the concrete strain in different regions. Fig. 13 shows the distribution of the LVDTs and the concrete strain gages under the specimen. The full setup before testing appears in Fig. 14 with an external frame to hold the LVDTs.

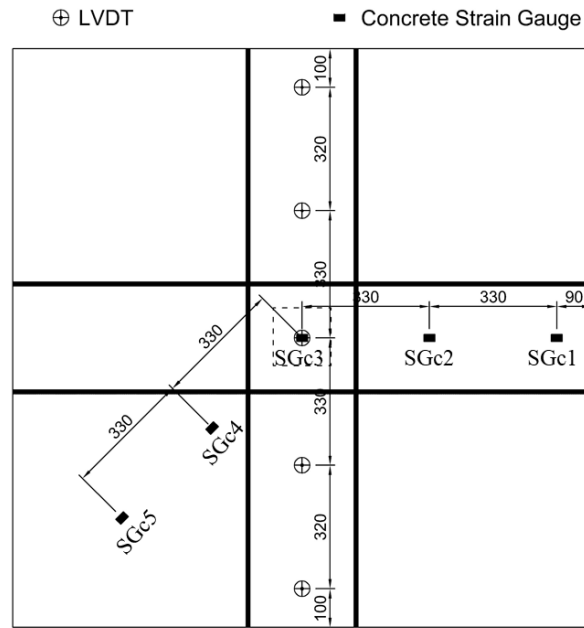
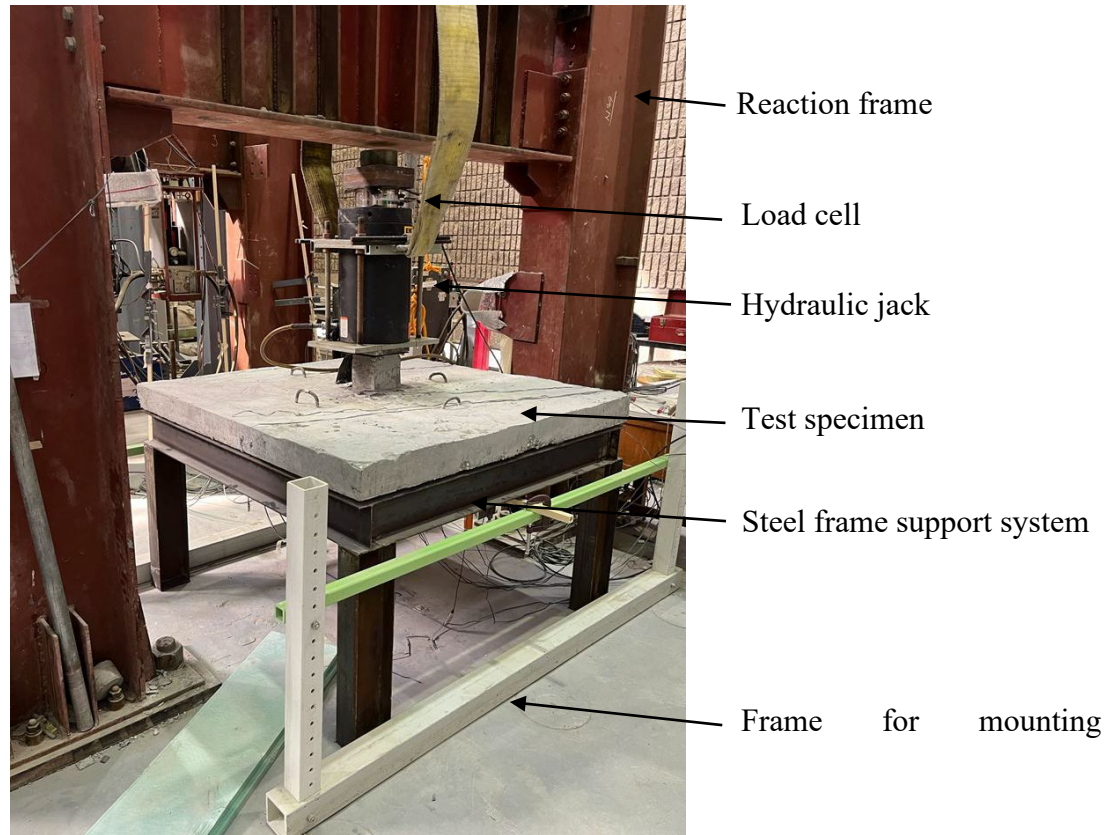


Figure 28: Concrete bottom surface instrumentation



(a)



(b)

Figure 29: Punching shear test setup (a) and LVDT arrangement (b)

CHAPTER 5

NSM FRP-CONCRETE BOND TEST RESULTS AND DISCUSSION

5.1 Bond Behavior Between NSM FRP and RAC Concrete

5.1.1 Failure Modes and Failure Loads

During the pull-out test, the NSM FRP-concrete bond behavior showed a successive balance of strength between concrete fracture and debonding. However, all specimens ultimately failed by debonding at the concrete-adhesive interface, with various concrete damage. Fig. 10(c) shows a typical failure across all the specimens, where, despite the round shape of the FRP bar, a rectangular shaped adhesive (that takes the shape of the initial groove) can be seen to adhere to it after the debonding.

Table 3 lists the average ultimate loads of the tested specimens. The table also shows a reference possessing good-quality concrete tested with the same NSM GFRP material as in the current work. Considering the low-strength concrete mixtures, the failure loads are alarmingly low compared to the normal-quality concrete bonded with the NSM GFRP. Among the poor-quality mixes, RCA60-SF0 exhibited the lowest ultimate loads, whereas RCA100-SF1 demonstrated the highest ultimate load.

Fig. 30(a) shows plots of the ultimate loads against the RCA incorporation ratio, emphasizing the fluctuations of failure loads when poor workmanship is considered. For a given percentage of RCA addition, the steel fiber content appears to indicate the expected pattern of failure load values (highest and lowest for specimens with 1% and 0% steel fiber contents, respectively). Among specimens containing 100% natural aggregate, the influence of the steel fiber addition can be seen to enhance the ultimate load, in which RCA0-SF0.5 had the highest failure load. For specimens incorporating 30% RCA of aggregate replacement, failure load increased marginally when steel fiber content increased. Also, the ultimate load can be seen to improve slightly as steel fibers addition increased among specimens with 60% RCA incorporation ratio. As for specimens with 100% RCA's incorporation ratio, steel fibers addition increased the failure load distinctly. As for the highest-performing specimen among the low-strength concretes, RCA100-SF1 saw a 44.5% reduction in ultimate load compared to a normal-strength of concrete. Compared with RCA0-SF0-REF, 44.5% - 61.7% reduction in the ultimate load when low strength concrete was used across the range of the studied variables.

Fig. 30(b) shows plots of the maximum loads against the steel fiber content. For all RCA0 and RCA100, increasing the steel fiber content increases the ultimate load. Specimens fully incorporating RCA have shown consistent reduced ultimate loads compared to RCA0. The maximum load was reduced by a 6% and 13% when the steel fiber content was 0.5% and 1%, respectively.

Table 7 The ultimate loads of this study with a result from a normal strength concrete of the same test

Specimen	Steel Fiber (%)	RCA Content (%)	P_u (kN)	Reduction (%)
RCA0-SF0-REF	0	0	34.3	
RCA0-SF0	0	0	14.4	58.3
RCA0-SF0.5	0.5	0	18.5	46.3
RCA30-SF0	0	30	15.1	56.2
RCA30-SF0.5	0.5	30	16.1	53.2
RCA60-SF0	0	60	13.2	61.7
RCA100-SF0	0	100	13.5	60.9
RCA100-SF0.5	0.5	100	16.1	53.4
RCA100-SF1	1	100	19.1	44.5

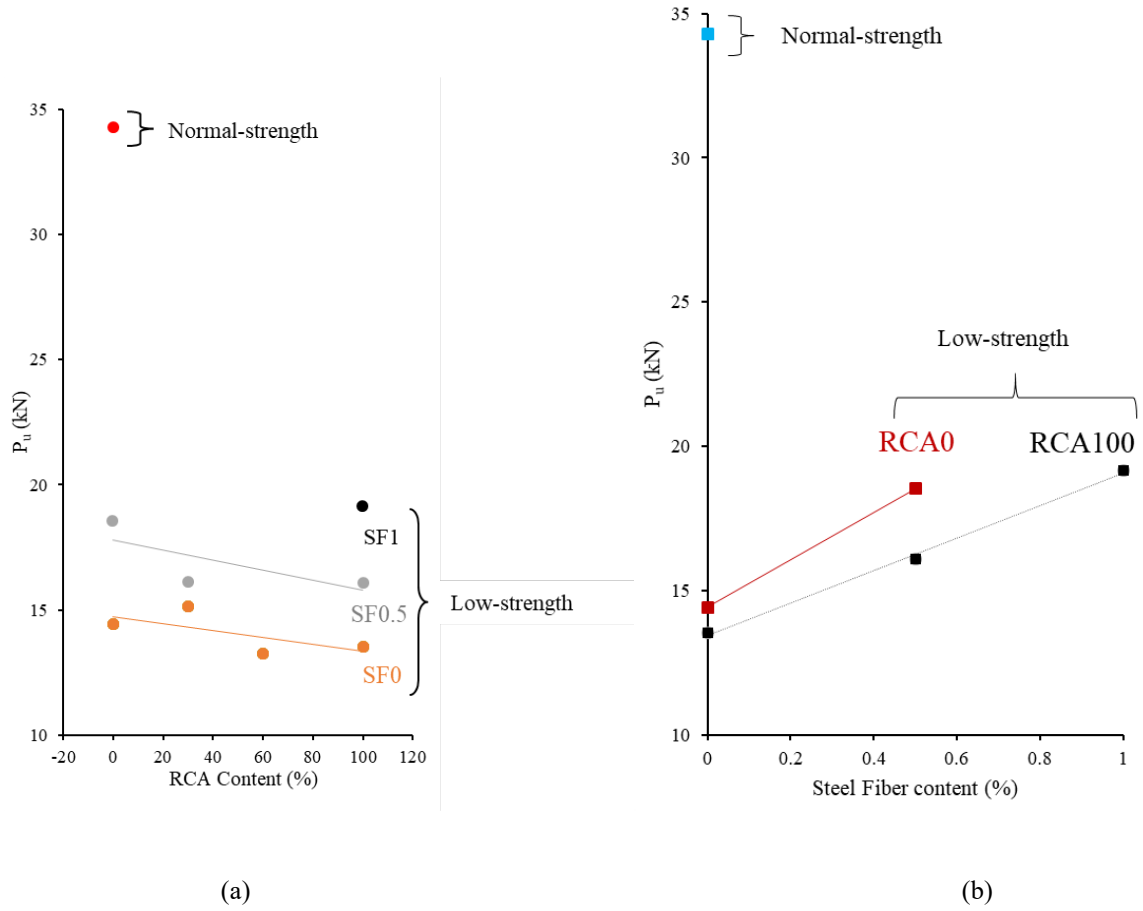


Figure 30: Ultimate bond failure load versus (a) RCA content (b) steel fiber content for the two extremes of RCA contents (0% and 100%) in the tested NSM GFRP-concrete connections

5.1.2 Load-Slip Relation

The load behavior versus the loaded end slip realized in the specimens produced from the nine different mixtures is demonstrated in Fig. 31. Fluctuations in the bond behavior is observed due to the artifact of the specimens having low-strength concrete. Steeper curve shown by RCA0-SF0-REF (Fig. 31(a)) than that of the low-strength concrete specimens [Figs. 31 (b – c)] directly relates to the concrete properties to the NSM FRP-concrete bond behavior. Except for RCA0-SF0.5 [Fig. 31(c)], all specimens failed at lower ultimate slip (slip corresponding to the ultimate loads) than the normal-

strength concrete. Comparing the behavior of the specimens made from NCA versus RCA, it can be seen that the loadslip curve for all specimens made from 100% NCA [RCA0-SF0-REF, RCA0-SF0, and RCA0-SF0.5 of Figs. 31 (a – c)] appear to have bilinear behavior – elastic response followed by initiation of inelastic behavior – prior to reaching their ultimate loads. This observed phenomenon is missing from the loadslip plots of all the specimens containing partial or full substitution of RCA in them, thereby indicating the weaker role of recycled aggregate in allowing for progressive damage of the NSM FRP-concrete connection prior to reaching the failure load.

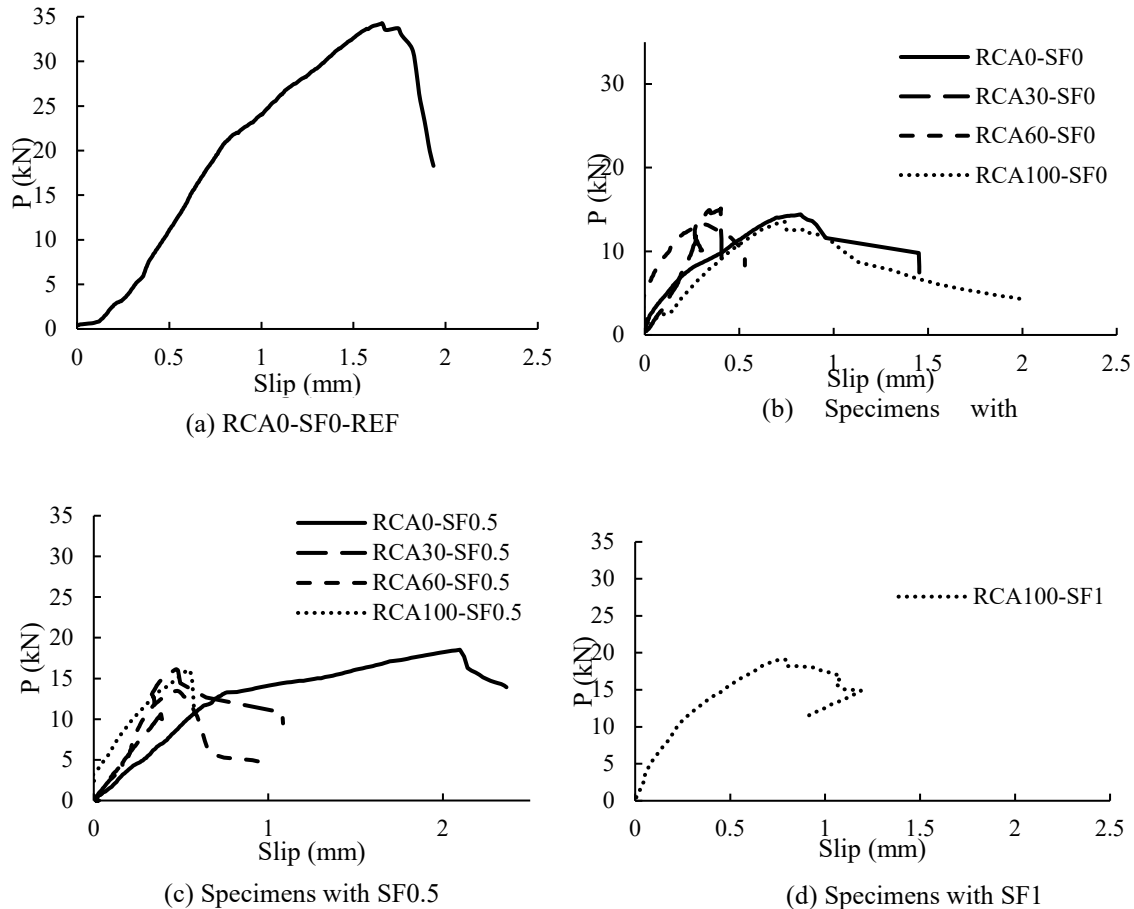


Figure 31: Load-slip behavior in the tested NSM GFRP-concrete bond along

5.1.3 Finite-Element Analysis

Since the above experimental program has aided one's understanding of the NSM FRP-concrete bond behavior for a limited number of low- and normal-strength concrete specimens, the present section adopts the finite element analysis to widen the spectrum of the study to include more ranges of the mixture's quality (both low- and high-strength materials) and other key parameters not covered in Section 3. This allows one to see different effects, especially given that design guides suggest a minimum concrete compressive strength for NSM FRP strengthening. The finite element model is first validated against the experimental observations of sections 3.4 and 3.5, followed by using the model to run a series of parametric studies using the bar diameter, concrete compressive strength, FRP modulus, and steel fiber content as variables.

5.1.3.1 Model Setup

Numerical model of the NSM FRP-concrete bond specimens was developed using the commercial program ANSYS [72]. As shown in Fig. 32, the grooved concrete block, bonded NSM glass FRP (GFRP), and the adhesive portion between the FRP and concrete were the essential components included in the model. For the test's boundary conditions, a specific area of the top surface of the block was only constrained from a translational movement in the y-axis representing the steel T-beam of the experimental test setup. The back bottom edge of the block was constrained from translational movement but allowed to rotate freely. Displacement-controlled boundary condition was applied at the FRP end to mimic the monotonously increasing loading. The problem is solved using the Newton-Raphson solution method with a convergence criterion of 0.05 [73].

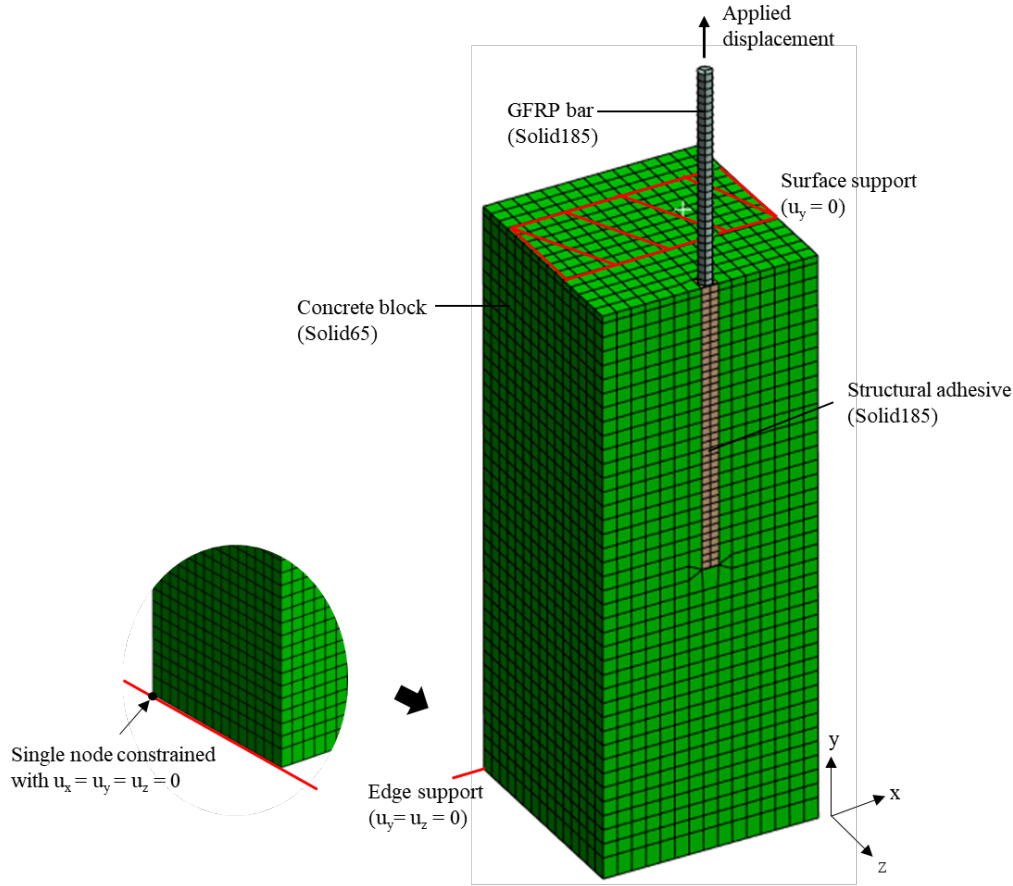


Figure 32: Finite element mesh of the test specimens

5.1.3.2 Element Types and Discretization

An eight-noded brick element, CPT216, having three translational degrees of freedoms (DOFs) per node and capable of defining the coupled plasticity-damage and nonlocal behavior was used to describe the concrete block. The bonded GFRP bar and the adhesive portion were modeled using Solid185 – a comparable brick element with eight nodes and three DOFs per node but lacking the cracking and crushing capabilities. In order to simulate the debonding failure, the GFRP bar-adhesive and adhesive-concrete interfacial responses had to be modeled independently. For the adhesive-concrete interface, a contact-target pair consisting of CONTA173 and TARGE170 elements were

used for the adhesive and concrete, respectively. In accordance with ANSYS's recommendations [72] for choosing contact and target surfaces, the contact element was constructed from four nodes and three translational nodal DOFs, and it was then superimposed on the adhesive. The GFRP bar-adhesive interface was also modeled using the contact-target elements pair following the aforementioned procedure, in which the GFRP surface was overlaid by a contact shell element using TARGE170. The Cohesive Zone Model (CZM) was adopted to define the interfacial behavior and potential debonding of the contact-target elements.

Cubic elements with maximum side length of 10 mm were used to mesh the concrete block while a 5-mm-side-length mesh of cubic elements was used to discretize the adhesive portion. Cylindrical elements were used to represent the GFRP bar with a maximum side length of 10 mm.

5.1.3.3 Fundamental Material Laws

Using the microplane model in ANSYS [72], the concrete was modeled based on the coupled plasticity-damage model demonstrated by Cicekli et al. [74]. Table 4 summarizes the parameters accounted for in this model. For the concrete's compressive strength, an experimentally measured value was employed. However, the Poisson's ratio and shear transfer coefficient were assumed to have values of 0.2 and 0.3, respectively, based on the literature recommendations [73,75]. Calculated values were used for the concrete tensile strength which includes the steel fibers' contribution. Both the GFRP bar and the Sikadur 30-LP [66] adhesive portion were modeled as isotropic elastic, using the mechanical characteristics provided by their manufacturers – Table 1.

5.1.3.4 Bond-Slip Modeling

Specifying the interfacial behavior of the GFRP-adhesive and concrete-adhesive interfaces, and simulating potential debonding are necessary for the CZM model and contact-target components. ANSYS [63] offers different CZM models. A mode-II (shear debonding) bond-slip CZM model was used to simulate debonding in the specimens for both the interfaces, as normal debonding was absent in the experimental tests conducted in Sections 5.1.1 and 5.1.2. The CZM model is schematically depicted in Fig. 33(a) consisting of the mode-II bilinear traction-separation law. The model includes a linear elastic portion until a shear, $\tau_{\max}$, stress is met along with a tangential slip, δ_t^* , as well as a softening line that terminates at the maximum tangential slip, δ_t^c .

Table 8 Concrete microplane model's parameters and their assumptions

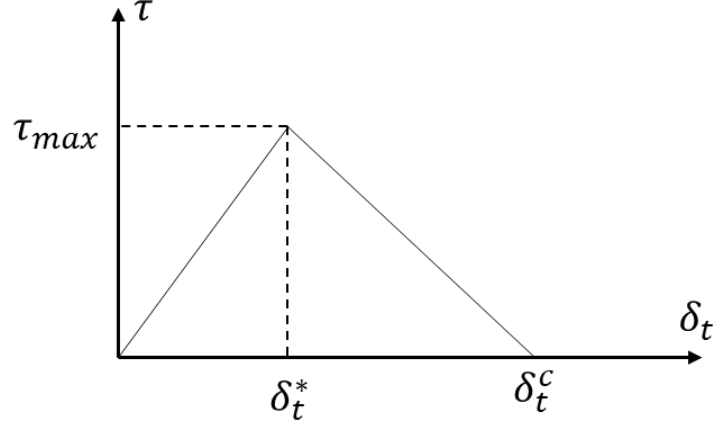
Parameter	Symbol	Assumption
Uniaxial Strength	f_c	Measured
Biaxial Strength	f_b	$1.15f_c$
Tensile Strength	f_t	$f_{ct} + f_f^1$
Yield/Compression Intersection	σ_v^c	$-0.67f_b$
Compression Cap Ratio	R	1-3
Hardening Constant	D	10000-500000
Tensile Cap Hardening	R_T	1
Tension Damage Threshold	γ_{t0}	0
Compression Damage Threshold	γ_{c0}	0.00001-0.0001
Compression Damage Evolution	β_c	1000-10000
Tension Damage Evolution	β_t	$1.5\beta_c$

¹ f_f is the tensile strength attained by steel fibers

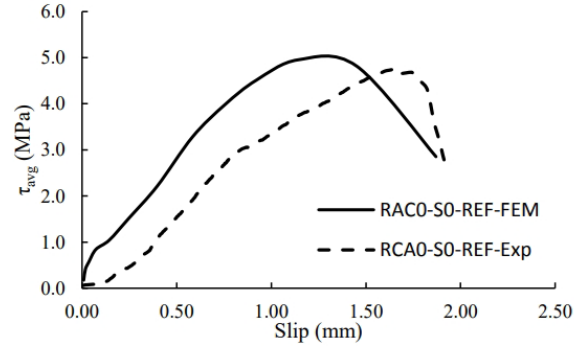
The Mode-II CZM model's interface separation is dependent on the shear behavior specified by Eq. (1) [72]:

$$\tau = K_t \delta_t (1 - d_m) \quad (1)$$

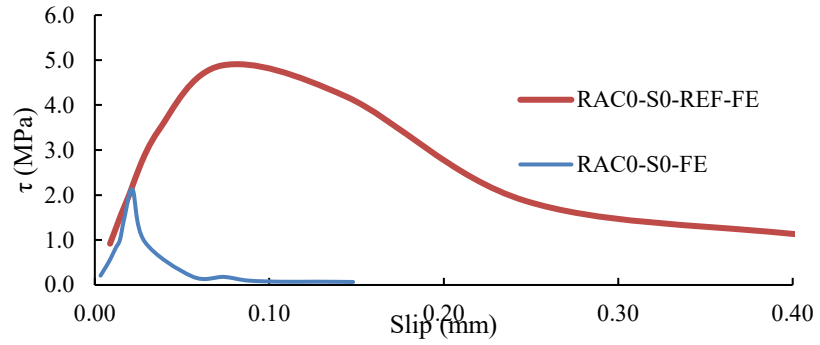
where, τ is the shear stress, δ_t is the tangential slip, and K_t is the tangential contact stiffness, which is automatically defined by ANSYS [72].



(a) Generic mode-II bond-slip relation for a GFRP-concrete interface used in the FEM simulation



(b) Experimental versus numerical of the average bond shear-slip behavior for the reference specimen



(c) Numerical bond shear-slip (not average) relations for normal versus low-strength concretes

Figure 33: Bond-slip behavior of the NSM FRP-concrete joint

5.1.3.5 Steel Fiber Contribution

As the present study has, as one of its studied parameters, the influence of steel fiber addition on the NSM FRP-concrete, the same needs to be considered in the model. As the mechanical properties of steel fiber-reinforced concrete (SFRC) are enhanced by the addition of steel fibers, one of the concrete properties that affects the NSM FRP-concrete bond is the tensile strength. Therefore, a steel fiber dependent analytical model – given by Eq. (2) [76] – for concrete tensile strength is adopted to enhance the performance of the FEM in capturing the mechanical response and behavior of the SFRC.

$$f_f = \frac{V_f l_f \tau_b}{2d_f} \left(1 - \frac{2w_{cr}}{l_f}\right)^2 \quad (2)$$

where, V_f is the steel fibers fraction of concrete's volume, l_f is the fiber's length, τ_b is the pull-out bond strength of the fiber-concrete, d_f is the diameter of the fiber, and w_{cr} is the crack width at failure. The steel fiber's bond strength and the crack width at failure were assumed based on the diverse embedment model (DEM) [77].

5.1.3.6 Model Validation and Parametric Studies

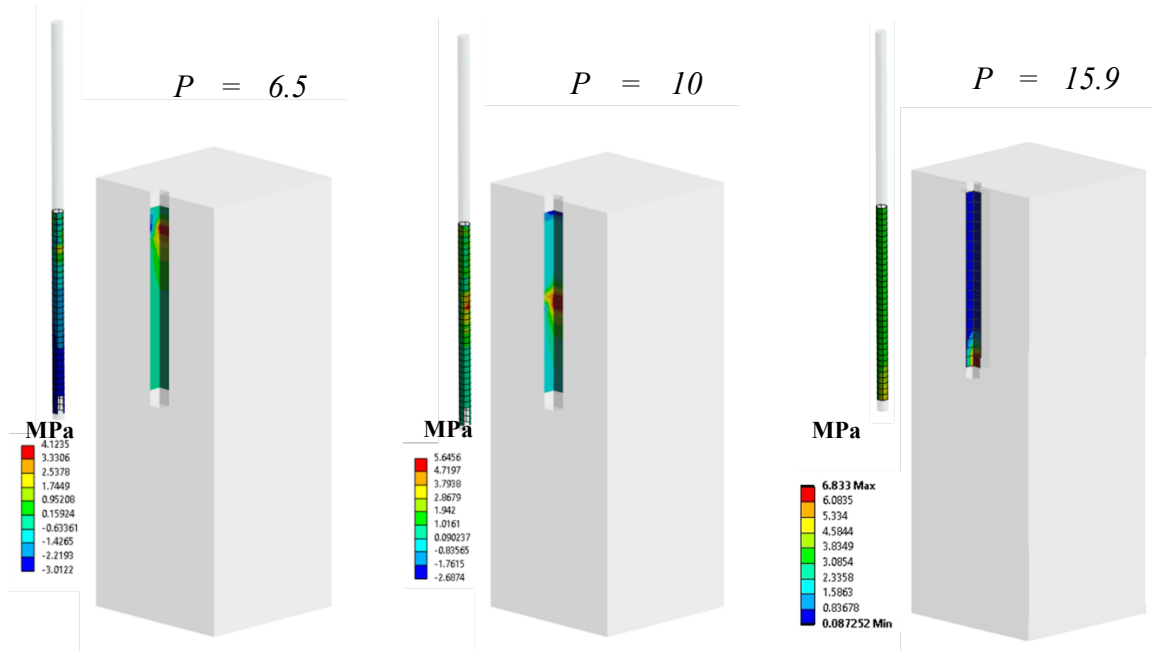
The postprocessed FEM results at the end of the simulation include the failure mode, debonding behavior, ultimate loads, bond strength, fracture energy, and bond-slip relation. Prior to delving into the parametric FEM analysis, the experimental results for the NSM FRP-concrete bond tests presented earlier were, first, used to validate the developed model. Because adhesive-concrete interfacial failure was the predominant failure mode realized, an average shear stress, τ_{avg} , was calculated by dividing the load by the bonded epoxy-concrete surface area (i.e., the three surfaces of the NSM groove). Fig. 33(b) shows the bond τ_{avg} -slip relation for Specimen RCA0-S0-REF of the experimental

test compared with the FEM prediction. Despite the slightly lower initial slope of the experimental curve, a close alignment between the two predictions is noticeable. The predicted slip corresponding to the ultimate load was 1.66 mm and 1.33 mm for the experimental and FEM results, respectively. Contrary to the experiment in which no strain gages were used to allow for computing the pointwise shear stress, τ , the numerical model allows computing both the τ_{avg} and τ . Fig. 33(c) shows the numerical bond τ -slip relation computed on the concrete groove surface for specimens RCA0-S0-REF and RCA0-S0-FEM which further highlights the detrimental effect of the low-strength concrete on the NSM FRP-concrete bond compared to the normal-strength behavior. RCA0-S0-FEM refers to FEM model of RCA0-S0-REF. The shear stress τ was computed at 30 mm from the loaded end for the two specimens. The highest peak, in each of the curves in Fig. 33(c), represent the debonding failure stress at that point. The normal-strength concrete shows a higher peak and larger area under the curve as well as higher ultimate slip.

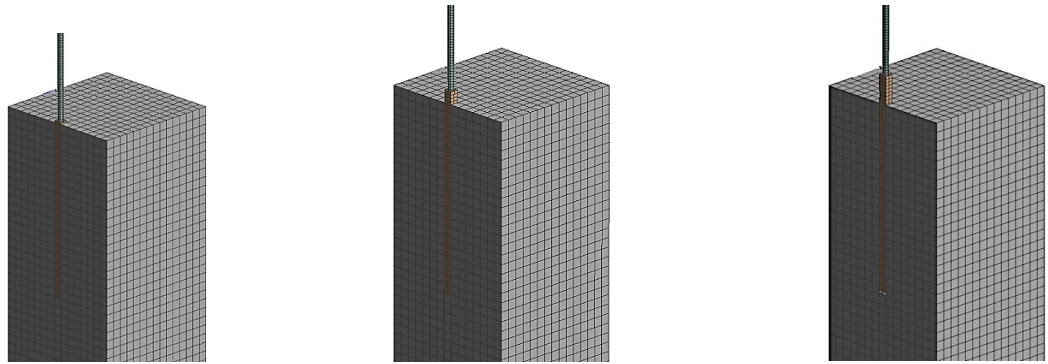
Similar to the experimental observation for the low-strength concrete, the dominant failure mode in the numerical model occurred in the adhesive-concrete interface with different degrees of concrete damage around the groove. Fig. 34 shows the typical shear stress propagation at three load steps of the FRP-adhesive interface and adhesive-concrete interface. RCA0-S0-FEM specimen was used for the illustration.

Comparisons between the experimental and FEM predictions of the ultimate load, P_u^{Exp} and P_u^{FEM} , maximum average bond strength, $\tau_{max,avg}^{Exp}$ and $\tau_{max,avg}^{FEM}$, and bond fracture energy, G^{Exp} and G^{FEM} , respectively, are shown in Table 9. The model results show acceptable agreement with the experimental prediction for the specimens whose

compressive strengths are available from the experiments. The ratio between the experimental and FEM predictions of the ultimate load, $\frac{P_u^{Exp}}{P_u^{FEM}}$, range between 0.90 and 1.03, with a mean of 0.96 for the four specimens including the reference sample. In addition to the quantitative agreement between the experimental and numerical predictions, the failure modes of the low-strength concretes – adhesive-concrete interfacial failure – also agree with one another [cf. Fig. 34(b) and Fig. 10(c)]. The positive impact of the crack bridging due to the steel fiber incorporation is portrayed by both the experimental and the FEM results of Table 9 – thanks to the analytical model incorporated by the latter in accordance with Eq. (2). It is expected that the lowest experimental load-to-model prediction ratio (0.90) of the steel fiber-reinforced specimen (RCA100-SF1) compared to the other specimens stems from the assumed values of τ_b and w_{cr} in Eq. (2) that were not available experimentally. However, the reported agreement between the experimental result of the affected specimen and its numerical prediction is still fairly good, thereby justifying the validity of utilizing the steel fiber model [67] – Eq. (2) – in the numerical framework.



(a)



(b)

Figure 34: FE simulation of Specimen RCA0-SF0-REF debonding failure: (a) shear stress of the concrete groove surface and the GFRP bar; and (b) exaggerated view (scale of 1:20) of debonding propagation at the concrete-adhesive interface.

Table 9 Experimental and numerical model validation results

Specimen	P_u^{Exp} (kN)	P_u^{FEM} (kN)	$\frac{P_u^{Exp}}{P_u^{FEM}}$	$\tau_{max,avg}^{Exp}$ (MPa)	$\tau_{max,avg}^{FEM}$ (MPa)	G^{Exp} (N/mm)	G^{FEM} (N/mm)
RCA0-SF0-REF	34.30	35.60	0.96	4.76	4.79	5.50	6.57

RCA0-SF0	14.41	15.90	0.91	2.00	2.15	2.91	1.57
RCA100-SF0	13.52	13.75	0.98	1.88	1.91	2.17	1.31
RCA100-SF1	19.15	21.25	0.90	2.66	2.95	2.34	2.48

Next, a parametric study was performed using the validated FE model by varying the following essential variables and their values:

- FRP bar diameter, d_b : 6, 8 and 12 mm
- FRP modulus, E_{frp} : 50, 80, and 120 GPa
- Concrete compressive strength, f_c : 20, 40, and 60 MPa
- Steel fiber content in terms of the concrete volume fraction: 0 %, 0.5 %, 1 %, and 2%.

The NSM groove dimensions follow the bar diameter as per the ACI [78] recommendation for creating NSM grooving ($1.5d_b \times 1.5d_b$). The numerical specimens' naming proceed as follows: PR- x - y - z - u , where x is the FRP modulus, y the concrete compressive strength, z the FRP bar diameter, and u the steel fiber content.

Table 6 lists the predicted ultimate load values of the parametric study model. For a given compressive strength or FRP modulus, the maximum load increases with increasing the bar diameter. This behavior becomes clearer when presented in Fig. 35(a) for a given concrete compressive strength. The figure also portrays the effect of increasing the concrete compressive strength (i.e., from low to high) which increases the maximum load. Increasing the compressive strength from 20 MPa to 40 MPa resulted in an increase in the bond failure load by 127%, 95%, and 71%, for the specimen with 6-

mm, 8 mm-, and 12-mm-diameter bars, respectively. Increasing the compressive strength from 40 MPa to 60 MPa also resulted in an increase in the ultimate load. For example, corresponding to this change in the compressive, an increase of 55% in the bond failure load for the 8-mm diameter bar specimen was observed. To aid one's judgement, the experimental results for the low- (12.2 MPa and 14.8 MPa) and normal-strength (39 MPa) concretes are superimposed on those of the parametric FE study in Fig. 35(a). The influence of FRP modulus on the bond failure load is presented in Fig. 35(b), where it can be observed that, similar to the effect of the compressive strength, increasing the FRP modulus increases the NSM FRP-concrete bond failure load. Finally, the increase in the maximum bond failure load with an increase in the steel fiber content is presented in Fig. 35(c). The rate of the increase, however, reduces at high steel fiber content. For example, an increase of 8% in the maximum load is obtained when the steel fiber content increased from 0.5% to 1%, whereas an increase of 5% in the maximum load is realized by increasing the steel fiber content from 1% to 2%.

Table 10 FE parametric study of the described variables

Specimen	Steel Fiber (%)	fc (MPa)	Concrete Strength Designation	Bar Diameter (mm)	FRP Modulus (GPa)	Ultimate Bond Failure Load (kN)
PR-50-20-6	0	20	Low	6	50	11.15
PR-50-20-8	0	20	Low	8	50	15.4
PR-50-20-12	0	20	Low	12	50	21.1
PR-50-40-6	0	40	Normal	6	50	27.7
PR-50-40-8	0	40	Normal	8	50	30.3

8

PR-50-40-	0	40	Normal	12	50	34.12
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12

PR-50-60-	0	60	High	6	50	42.04
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6

PR-50-60-	0	60	High	8	50	48.7
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8

PR-50-60-	0	60	High	12	50	51.2
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12

PR-80-40-	0	40	Normal	6	80	28.6
-----------	---	----	--------	---	----	------

6

PR-80-40-	0	40	Normal	8	80	32.8
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8

PR-80-40-	0	40	Normal	12	80	36.9
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12

PR-120-	0	40	Normal	6	120	30.05
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40-6

PR-120-	0	40	Normal	8	120	35.9
---------	---	----	--------	---	-----	------

40-8

PR-120-	0	40	Normal	12	120	38.5
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40-12

PR-80-40-	0.5	40	Normal	8	80	45.6
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8-0.5

PR-80-40-	1	40	Normal	8	80	49.56
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8-1

PR-80-40-	2	40	Normal	8	80	52.31
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8-2

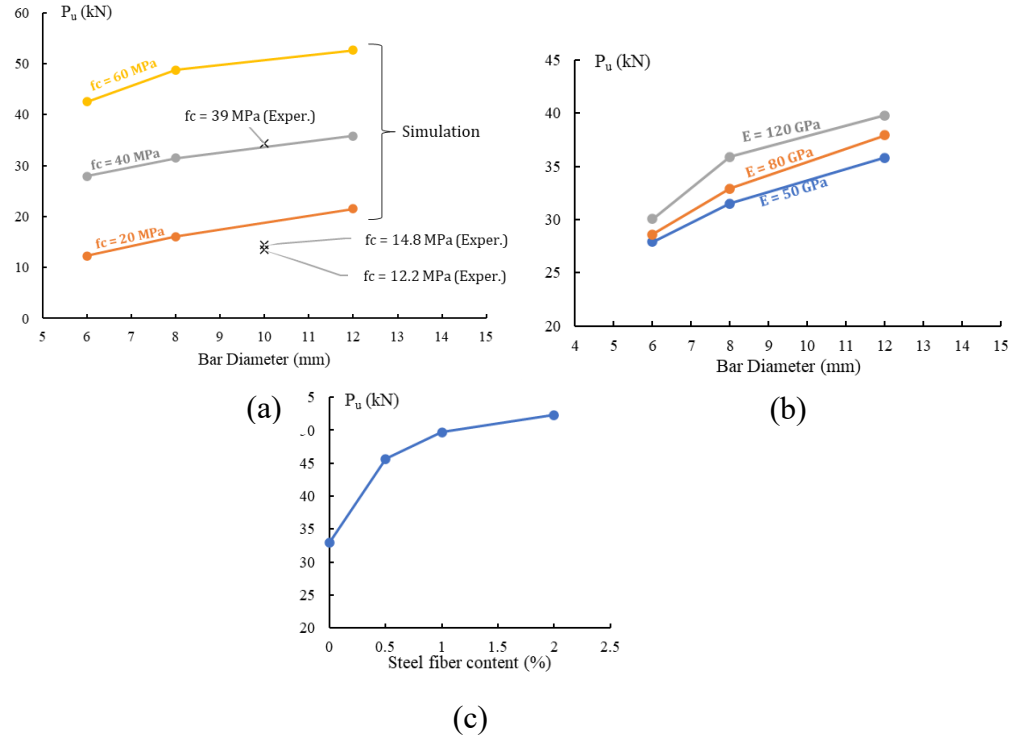


Figure 35: FE parametric study on the influence of essential variables on NSM FRP-concrete bond: (a) concrete compressive strength varied for a constant FRP modulus of 50 GPa; (b) FRP modulus varied for a constant concrete compressive strength of 40 MPa; and (c) steel fiber content varied for a constant FRP modulus of 80 GPa, a constant concrete compressive strength of 40 MPa, and a constant FRP bar diameter of 8 mm.

5.2 Assessment of NSM FRP- Concrete Bond Behavior Using Universal Debonding Test Apparatus

The experimentally reported failure modes (FMs) were divided into the following categories:

1. FRP rupture (FR)
2. Splitting of adhesive failure (SA)
3. Concrete-adhesive interface debonding (C/A-D)
4. FRP-adhesive interface debonding (F/A-D)
5. Wedge failure (WF)

Splitting at the concrete corner (CC)

5.2.1 Single-Shear Pull-out Test Results

5.2.1.1 Failure Modes and Ultimate Loads

In the performed single pull-out tests, failure modes were reported. Various types were observed. Table 11 demonstrates the failure modes and the average failure loads of each specimen tested via the single-shear pull-out test. Fig. 36 shows the typical failure modes observed on the specimens.

Table 11 Ultimate loads, failure modes, and predicted loads of each specimen. (F/A-D: FRP-adhesive interface debonding and WF: wedge failure)

Specimen	P_u (kN)	FM
S-G	17.8	F/A-D
S-C	17.5	F/A-D
S-RC	40.7	WF

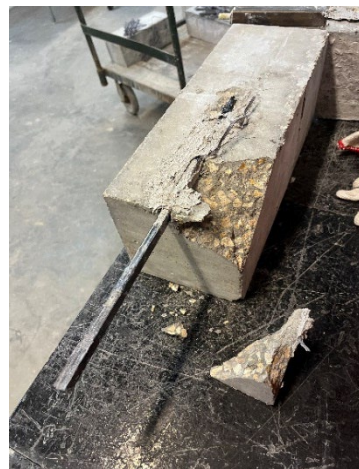
S-G specimen exhibited an ultimate load of 17.8 kN before failing for F/A-D. Also, an ultimate load of 17.5 kN was reported for the S-C specimen before failing for F/A-D. On the other hand, the S-R specimen demonstrated an ultimate load of 40.7 kN before failing for WF. This is noticed in the failure pattern between the two specimens shown in Fig. 36(a). S-RC had the highest ultimate load reported, which can be attributed to the larger bonded surface area, with 36% and 57% difference compared to S-G and S-C bond strength results, respectively.



(a) S-G



(b) S-C



(c) S-RC

Figure 36: Failure patterns of each specimen (S-G: circular GFRP, S-C: circular CFRP, and S-RC: rectangular CFRP)

5.2.1.2 Bond Strain Distribution

The measured strains for the various NSM systems evaluated at five load levels are presented in Fig. 37 to compare the strain field across the bond length at 10 mm, 90 mm, and 180 mm from the start of the bonded length. Strains along the bonded length were comparable amongst the specimens for the chosen load levels.

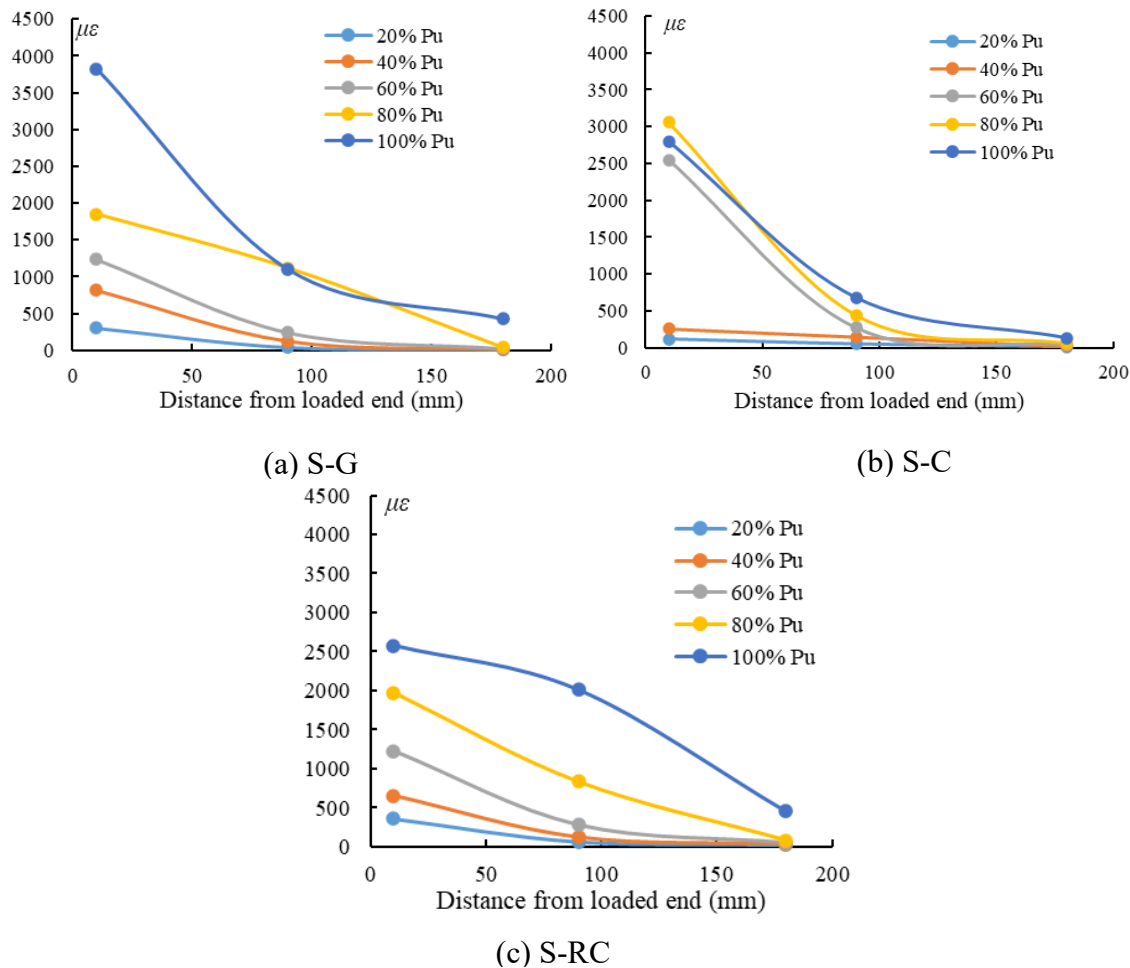


Figure 37: Bond strain distribution for each specimen

Ultimately, the S-G specimen exhibited the highest strain at the first point of bonded FRP at failure load. However, at the failure load, the S-RC specimen showed the highest strain in the middle of the bonded length. Regarding the S-G specimen, the strain increased dramatically before the failure. Therefore, only, the loaded end debonded at the

interface of FRP-adhesive allowing the GFRP to stretch compared to the remaining bonded length, which failed due to decreased bonded length after that. For S-C, as shown in Fig. 37(b), the strain increased drastically after half of the loading approximately, which indicates the debonding at the loaded end at the FRP-adhesive interface. Thus, an earlier failure of bond resulted due to decreased bonded length. S-RC specimen, along the bonded length, demonstrated a smooth behavior in strain, which indicates no debonding case occurred. This stronger bond is explained by the higher surface area of the rectangular CFRP compared to the circular CFRP bar.

5.2.1.3 Bond-Slip Relation

Utilizing experimentally observed strains, ε , along the FRP, equivalent shear stresses, τ , were obtained using the following equation and related to the interfacial slip, s :

$$\tau = E_f \cdot \frac{A_f}{p_f} \cdot \frac{\Delta \varepsilon_i}{\Delta x_i} \quad (2)$$

where E_f is the elastic modulus of FRP, A_f is the cross-sectional area of FRP bar/strip, p_f is the bar section perimeter, $\Delta \varepsilon_i$ is the strain difference, and Δx_i is the distance between the two considered SGs. The shear stress results were shown for the two segments of the bonded FRP to capture wider analysis. SEG 1 is the closest 90-mm-length to the loaded end and SEG 2 is the furthest 90-mm-length to the loaded end.

Fig. 38 illustrates the bond-slip relation of the specimens that were tested. Obviously, the plots showed variation in the equivalent shear stress between the two segments for all specimens. As the slip increases, the shear stress in SEG 1 peaked firstly.

Then, the shear stress in SEG 2 rose substantially when the shear stress in SEG 1 decreased. This behavior confirms that debonding starts from the loaded end point and continue along the bonded FRP. The shear stress of S-G and S-RC peaked at the latter segment of the FRP whereas the shear stress of S-C peaked at the FRP's first segment. The S-C specimen demonstrated the highest peak shear stress ($\tau_{max} = 10.9$ MPa). However, S-G specimen showed the lowest peak shear stress ($\tau_{max} = 3.1$ MPa) between the tested specimens. S-C failed at the lowest slip reading compared to other specimens.

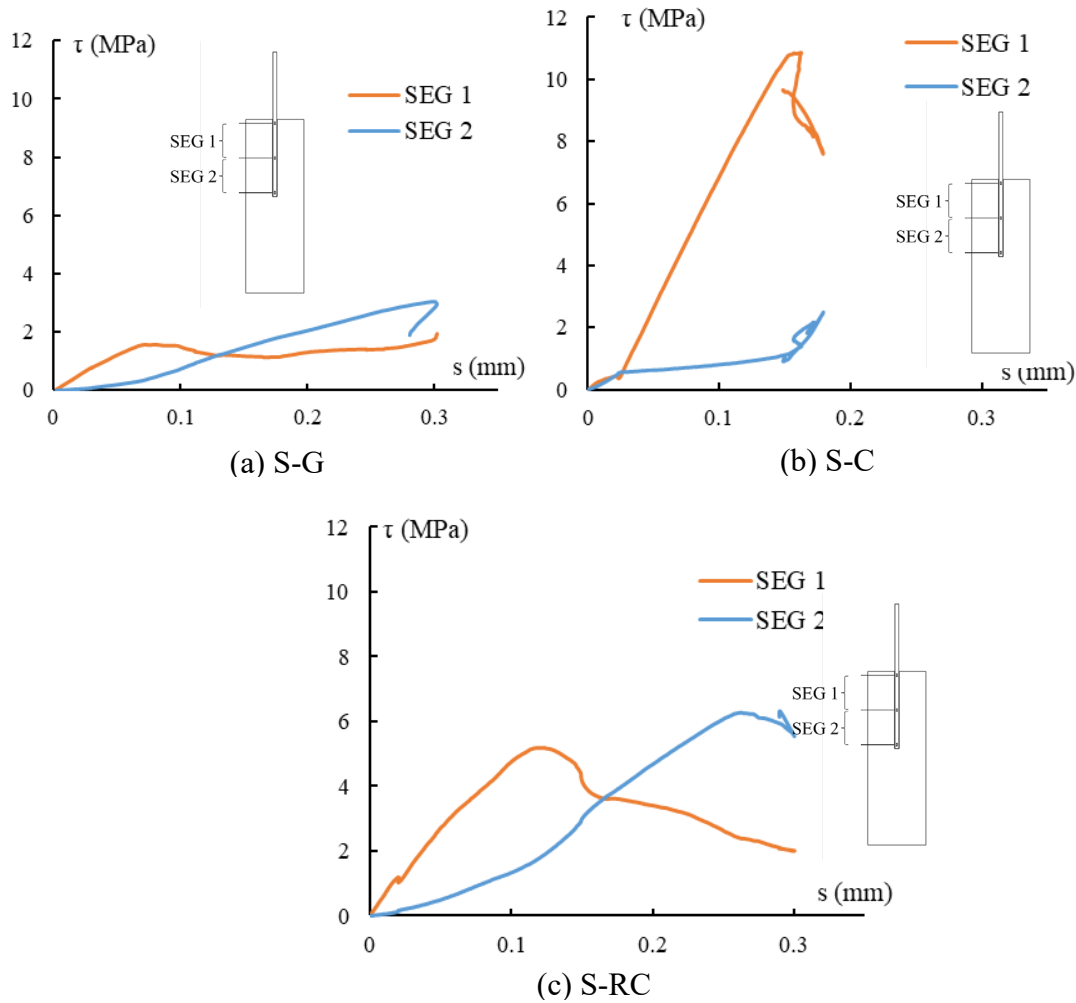


Figure 38: Bond-Slip relation for each specimen (S-G: circular GFRP, S-C: circular CFRP, and S-RC: rectangular CFRP)

5.2.2 Double-Shear Pull-out Test Results

5.2.2.1 Failure Modes and Ultimate Loads

In the double pull-out tests performed, failure modes were reported. Various types were observed. Table 12 demonstrates the failure modes and the average failure loads of each specimen tested via the symmetrical double-shear pull-out test. Fig. 39 shows the typical failure modes observed on the specimens.

Table 12: Ultimate loads, failure modes, and predicted loads of each specimen. (C/A-D: concrete-adhesive interface debonding and F/A-D: FRP-adhesive interface debonding)

Specimen	P_u (kN)	P_u (kN)	$P_{u,avg}$ (kN)	FM
D-G	46.2	54.7	50.5	F/A-D
D-C	57.6	-	57.6	F/A-D
D-RC	47.2	-	47.2	C/A-D



(a) D-G



(b) D-C

Figure 39: Failure Pattern of symmetrical double-shear stress (D-G: circular GFRP and D-C: circular CFRP)

The average ultimate load at which D-G specimens failed for F/A-D was 45.1 kN. Additionally, a 48.5 kN average ultimate load was recorded for the D-C specimen prior to failure for F/A-D. CFRP Circular cross-section NSM CFRP had the highest bond strength reported, with 7% difference compared to circular cross-section GFRP. Considering that both specimens failed with full plane, the effect of restraint method remained absent when Universal Debonding Test Apparatus FRP-Concrete System was used.

5.2.2.2 Bond Strain Distribution

Fig. 40 displays the raw strain readings for the various NSM systems that were tested using symmetrical double-shear test and evaluated at five load levels. The loaded end of both FRPs in D-C displayed the greatest strain reading at failure load, knowing that the strain reading via the debonded FRP was the highest ($\mu\epsilon = 4,828$). The midpoint of D-C's and D-G's FRP bonded length shared similar strain reading at the failure load whereas D-RC exhibited relatively lower strain reading for the two FRPs along the entire bonded length. It is indicated by the shape of graph, for the two specimens, that F/A-D occurred agreeing with previous results of single-shear tests. Testing symmetric double-shear specimens triggers the analysis of tensile balance. Notably, imbalance of symmetrical pulling is inevitable. Therefore, it can be observed from the strain reading along the bonded length of D-G that insignificant imbalance is present due to imperfect concrete casting. That endorses the capability of the apparatus to handling perfect specimens. On the other hand, due to imperfect specimen production, significant imbalance is present in testing D-C, where the debonded FRP's strain have deviated substantially compared to the strain reading displayed by the bonded FRP.

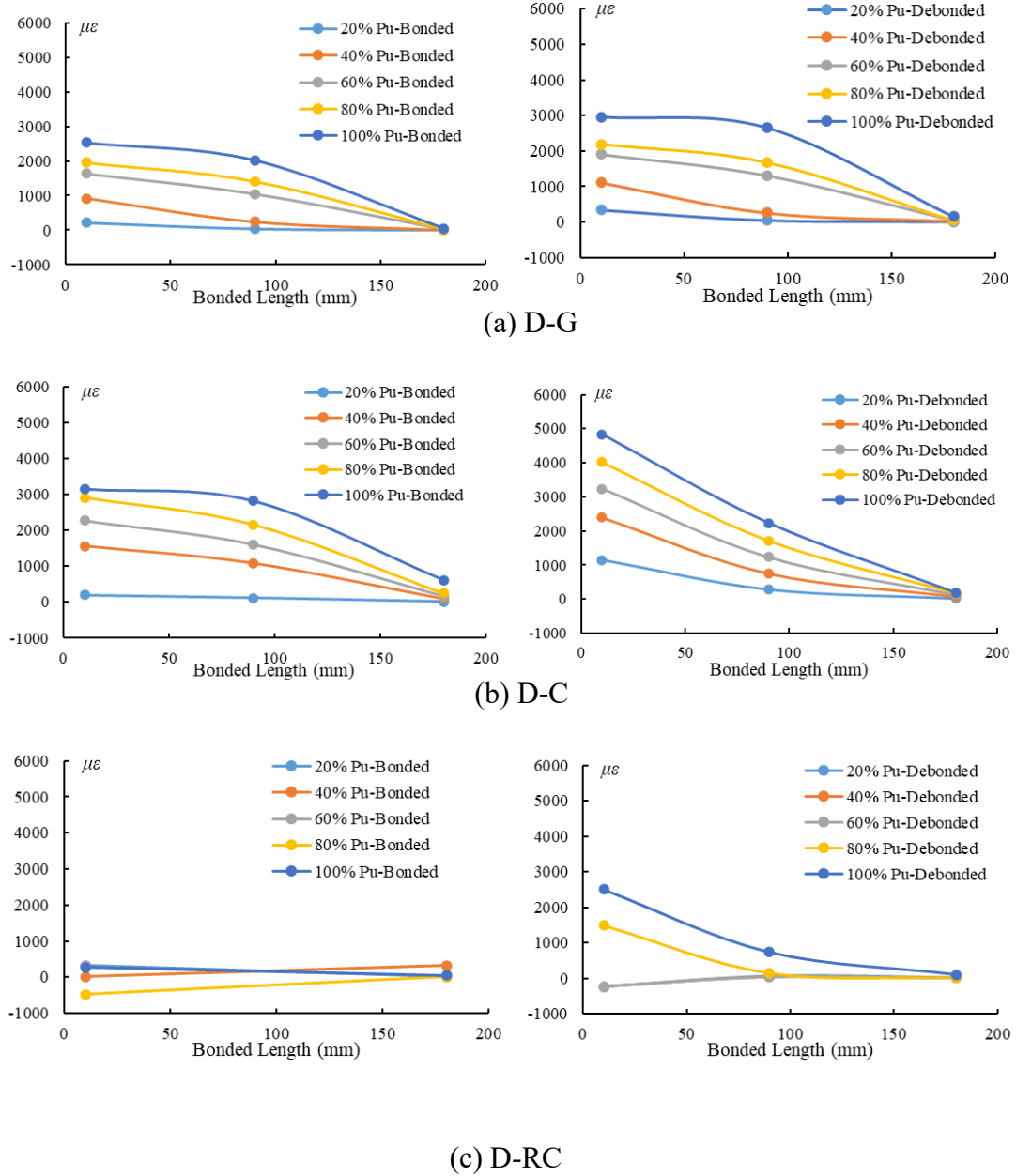


Figure 40: Bond strain distribution of symmetric double-shear bond test (D-G: circular GFRP, D-C: circular CFRP, and D-RC: rectangular CFRP)

5.2.2.3 Bond-slip Relation

Fig. 41 illustrates the bond-slip relation of the specimens that were tested. Similar to the single-shear test's bond-slip relation, the shear stress results were shown for the two segments of the bonded FRPs. D-RC misses strain readings of the intact FRP due to

malfunctioning during adhesive curing. D-C demonstrated the highest shear stress of the debonded FRP ($\tau_{\max} = 8.66$ MPa) at the second segment. Relatively, D-G failed at low slip readings with low shear stress. Except for D-RC, double-shear testing has triggered the latter segment of the bonded FRP earlier than the single-shear testing, which can give ideal results when perfect specimen balance is present. Eventually, this enhances the mistake mitigation in NSM testing field.

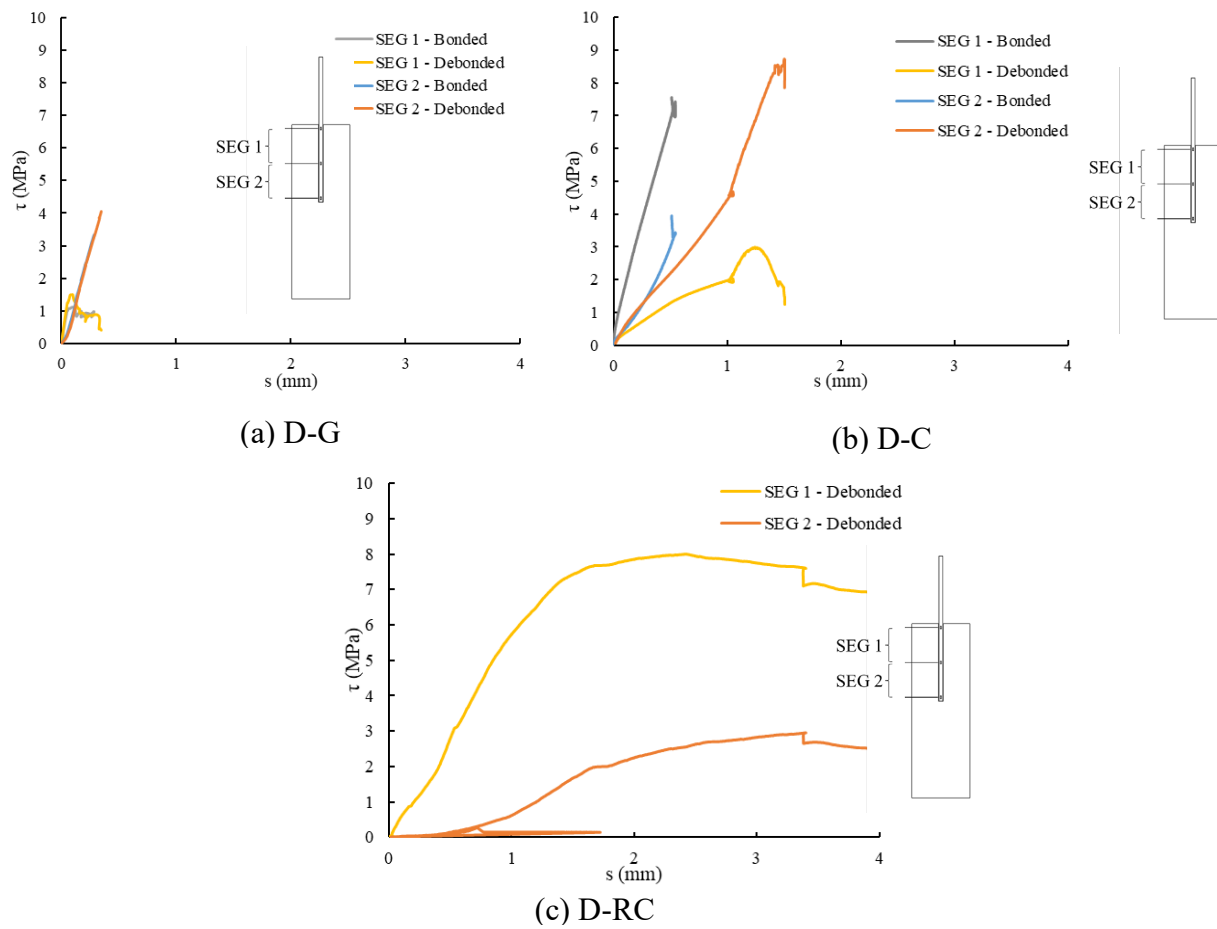


Figure 41: Bond-slip relation of symmetric double-shear bond test (D-G: circular GFRP, D-C: circular CFRP, and D-RC: rectangular CFRP)

5.2.3 Beam Test Results

5.2.3.1 Failure Modes and Ultimate Loads

In the beam tests performed, few of these failure modes were recorded. The most frequently observed mode was F/A-D. Table 13 demonstrates the failure modes and the average failure loads of each specimen tested via beam test. Fig. 42 shows the typical failure modes observed on the specimens.

Table 13: Ultimate loads, predicted loads, and failure modes of beam test specimens. (F/A-D: FRP-adhesive interface debonding, FR: FRP bending rupture)

Specimen	P_u (kN)	P_u (kN)	$P_{u,avg}$ (kN)	FM
B-G	13.508	14.109	13.8	F/A-D
B-C	31.144	30.752	30.9	F/A-D
B-RC	11.559	15.235	13.4	FR

The average ultimate load at which B-G specimens failed for F/A-D was 13.8 kN. Additionally, a 30.9 kN average ultimate load was recorded for the B-C specimen prior to failure for F/A-D. The B-RC specimen, on the other hand, showed an average ultimate load of 13.4 kN before failing for FR. B-C had the highest bond strength reported, with 55% and 56% difference compared to those of B-G and B-RC, respectively.

B-RC failed for FR due to the lack of continuous integrity across the cross-sectional area as the rotational effect in the beam test expressed bending impact of loaded end of the FRP causing the FRP to bend and locally failing the strip.



(a) B-G

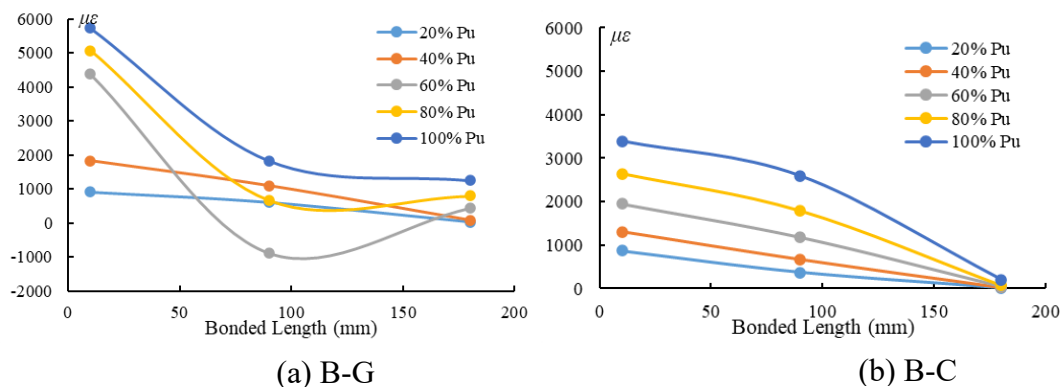


(b) B-C

Figure 42: Failure pattern of beam test specimens (B-G: circular GFRP and B-C: circular CFRP)

5.2.3.2 Bond Strain Distribution

In Fig. 43, the measured strains for the various NSM systems tested by beam test and assessed at five load levels are shown. At failure load, specimen B-G showed the maximum strain at the loaded end of bonded FRP. The midpoint of the bonded length on the B-C specimen, however, had the maximum strain at the failure load. After 40% of the maximum load, the strain in the B-G specimen drastically rose. As a result, the FRP's loaded end debonded, allowing the GFRP to stretch and flex causing the remaining bonded length to be subjected to moment. Therefore, the middle point of the bonded length showed sudden decrease in strain reading before increasing gradually till the ultimate load, as illustrated in Fig. 43(a). For B-C, along the bonded length, demonstrated a smooth behavior in strain reading, which indicates no debonding case occurred for F/A-D. Moreover, the behaviour of the B-C curve was similar to that of D-G and D-C (bonded), those which are considered true NSM FRP behavior. B-RC specimen, along the bonded length, demonstrated low strain reading before failing at an early ultimate load. The decreased strain readings at the loaded end at failure load confirm the unusual behavior occurred to the rectangular cross-section CFRP that was explained previously.



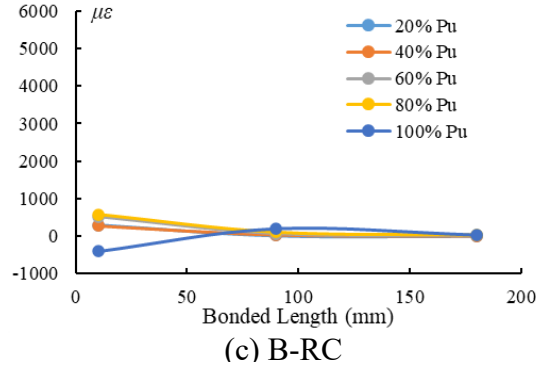


Figure 43: Bond strain distribution of beam test specimens (B-G: circular GFRP, B-C: circular CFRP, and B-RC: rectangular CFRP)

5.2.3.3 Bond-Slip Relation

Fig. 44 illustrates the bond-slip relation of the specimens that were tested. B-RC experienced low shear stress before failing by FRP bending rupture. B-G showed irregular plot agreeing with the disturbance occurred in the strain reading. However, it reached the highest shear stress ($\tau_{\max} = 8.9$ MPa) at the first segment which is converted from the note that the FRP bent near the strain gauge. B-C showed the highest shear stress ($\tau_{\max} = 7.8$ MPa) at the latter segment, Reportedly, the intermediate debonding of FRP-concrete interface in B-G occurred around a slip of 0.35 mm where the GFRP experienced a sudden increase in average shear stress. Regardless of the B-RC's result, B-C specimen failed at a lower ultimate slip ($s = 0.22$ mm) than B-G ($s = 0.41$ mm), which agrees with the other test methods involving circular GFRP and CFRP.

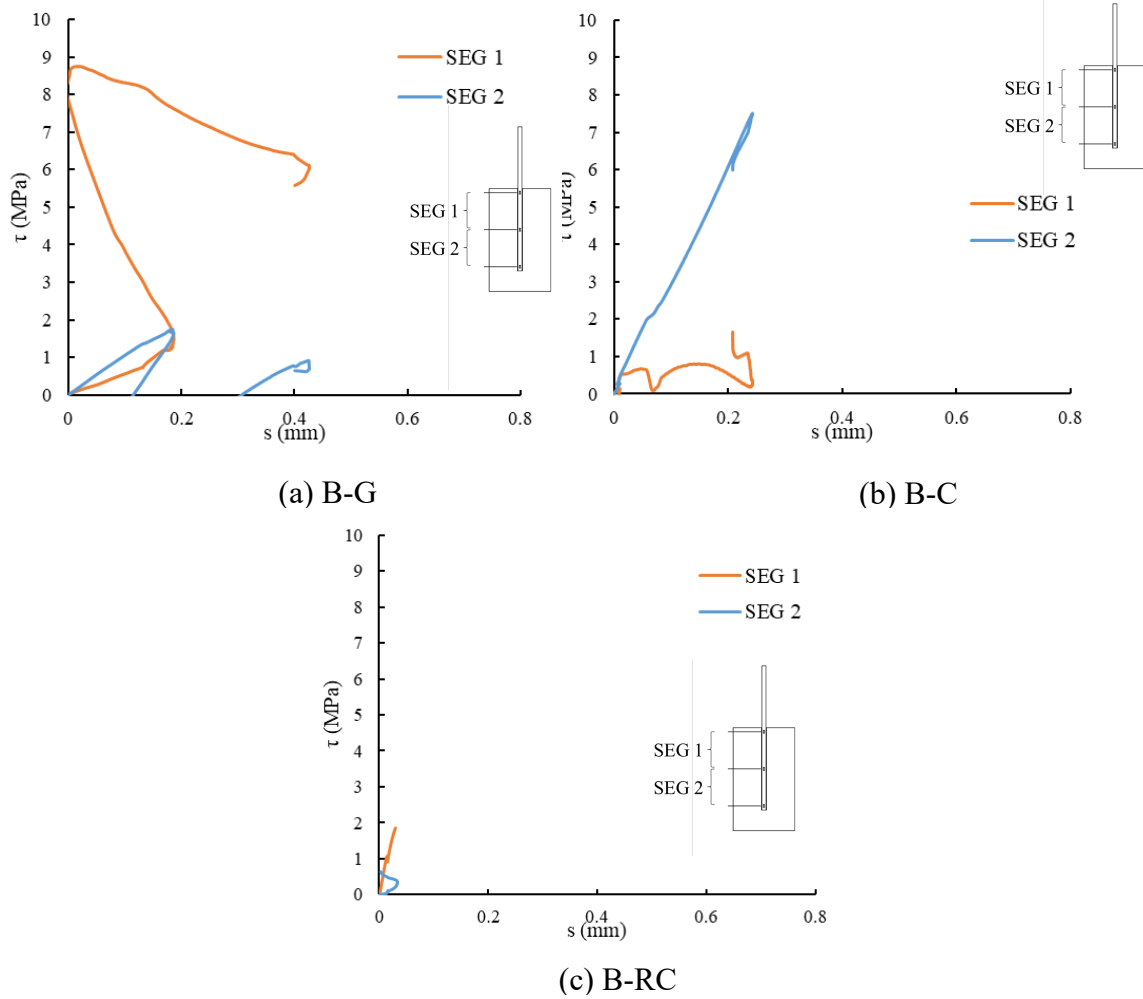


Figure 44: Bond-slip relationship of beam test specimens (B-G: circular GFRP, B-C: circular CFRP, and B-RC: rectangular CFRP)

5.2.4 Asymmetric Double-Shear Test

For this test, one specimen was tested as a conjugate of the double-shear test specimen D-C. Table 14 lists the failure mode and the ultimate strength of the specimen. The specimen failed under splitting of concrete corner with an ultimate load of 51.56 kN. Fig. 45 shows the failure pattern of the tested specimen. The failure mode shown is expected for this type of testing because an insufficient concrete edge distance was present. Compared to D-C specimen's average ultimate load, a decrease of 10% was

reported for performing an asymmetric double-shear bond test, which is attributed to insufficient concrete edge distance.

Table 14: Failure modes and ultimate loads of decreased edge distance NSM bond. (CC: Splitting at concrete corner)

Specimen	P_u (kN)	FM
DA-C	51.56	CC



Figure 45: Failure pattern of DA-C (DA-C: circular CFRP)

5.2.5 Insufficient Concrete Edge Distance Test

5.2.5.1 Failure Modes and Ultimate Loads

As this test studies the bond behavior under a decreased concrete edge distance, failure pattern of the four specimens of GFRP bars-concrete bond, two specimens of CFRP bars-concrete NSM bond, and two specimens of CFRP strips-concrete bond are shown in Fig. 46. Table 15 lists the failure modes and the failure strength of each specimen.

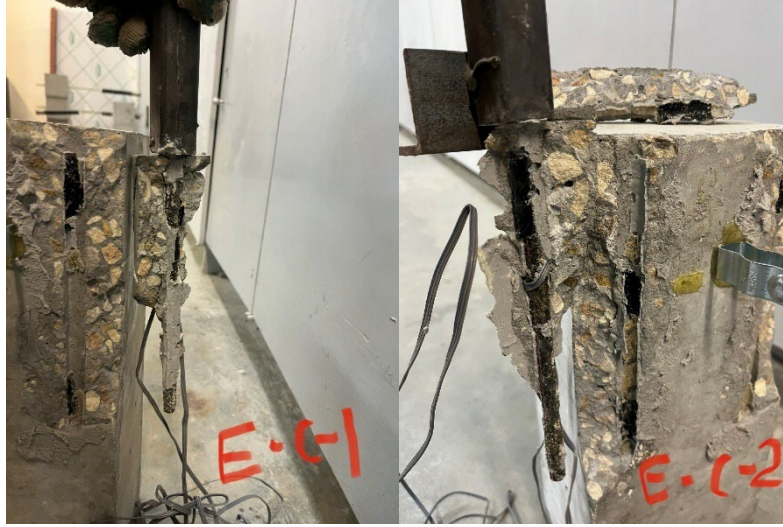
Table 15: Failure modes and ultimate loads of decreased edge distance NSM bond. (CC: Splitting at concrete corner and F/A-D: FRP-adhesive interface debonding)

Specimen	P_u (kN)	P_u (kN)	P_u (kN)	P_u (kN)	$P_{u,avg}$ (kN)	FM
E-G	8.9	5.8	13.9	17.9	11.6	CC, F/A-D
E-C	27.9	25.1	-	-	26.5	CC
E-R	25.1	21.3	-	-	23.2	CC

All specimens failed under splitting of concrete corner except for specimen E-G-3 that failed under FRP-adhesive interface debonding. Reportedly, CFRP bars possessed a better NSM bond than the rest of NSM FRP types with an average failure load of 26.5 kN. However, the second highest NSM bond was demonstrated by the CFRP strips with an average failure load of 23.2 kN. As expected, based on their mechanical properties and their degree of sand coating, GFRP bars showed the lowest NSM bond with an average ultimate load of 11.6 kN.



(a) E-G



(b) E-C



(c) E-RC

Figure 46: Failure pattern of single-shear test for decreased edge distance specimens (E-G: circular GFRP, E-C: circular CFRP, and E-RC: rectangular CFRP)

5.2.5.2 Bond Strain Distribution

The measured strains for the various NSM systems tested by beam test and assessed at five load levels are shown in Fig. 47. At failure load, E-C showed the highest strain reading than those of other FRP types. According to the Fig. 47(a), GFRP bars NSM bond were utilizing the middle of the bonded length more than other FRP types. The lowest strain reading was demonstrated by E-RC specimens despite failing at

relatively high ultimate loads. Rectangular CFRP tends to display low strain reading although its specimen showed a high ultimate load as seen in the previous tests.

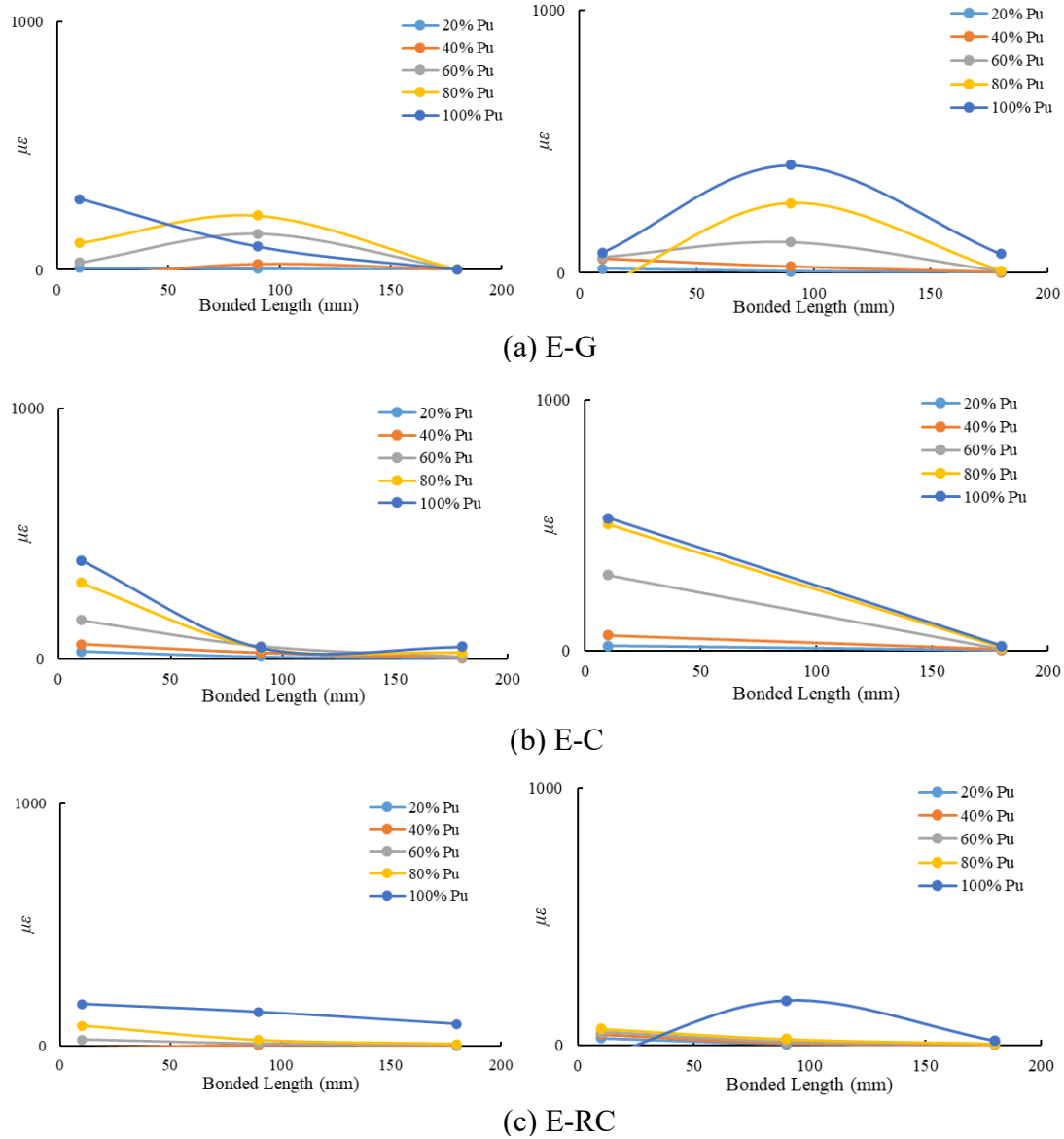


Figure 47: Bond strain distribution of single-shear test for decreased edge distance specimens (E-G: circular GFRP, E-C: circular CFRP, and E-RC: rectangular CFRP)

5.2.6 Influence of Test Methods Variation

Double-shear NSM bonds have increased the overall ultimate bond strength compared to the single-shear 184%, 229%, and 15% for GFRP bars, CFRP bars, and CFRP strips, respectively. Ideally, a double-shear test ensures a 100% increase over a

single-shear test. Therefore, single-shear test results demonstrated lower results than usual for this experiment. However, the single result of CFRP strips single-shear results could be taken into consideration.

Despite the low results of the single-shear test, double-shear test of NSM bonds offer more balance in strain distribution over the bonded length during the loading, according to Fig. 37 and Fig. 40. According to the beam-bend test strain distribution, rectangular cross-section NSM FRPs performed poorly under beam-bend test due to loss of integrity of NSM strips.

Considering the ultimate load of S-G and S-RC, the concrete edge distance of 15 mm resulted in lower ultimate load than that of the single-shear test specimens that possess a concrete edge distance of 70 mm. The decreased concrete edge distance reduced the ultimate load in the S-G specimen by 66%, whereas a 43% reduction in the S-RC specimen.

Compared to D-C's average ultimate load, a decrease of 10% was reported for performing an asymmetric double-shear bond test, which is attributed to insufficient concrete edge distance.

CHAPTER 6

NSM FRP-CONCRETE PUNCHING SHEAR RESULTS AND DISCUSSION

6.1 Compressive Strength Test Results

Table 16 shows the compressive strength test results for the four concrete mixtures after 28 days of curing.

Table 16 Compressive strength test results for the studies mixes

Specimen ID	Load (kN)	f'_c (MPa)	Average f'_c (MPa)
RAC0-S0-1	306.1	17.33	14.76
RAC0-S0-2 ^a	203.9	11.55	
RAC0-S0-3	271.9	15.40	
RAC0-S1-1	322.8	18.27	17.71
RAC0-S1-2 ^a	247.3	14.00	
RAC0-S1-3	368.0	20.84	
RAC100-S0-1	200.3	11.34	12.19
RAC100-S0-2	225.5	12.77	
RAC100-S0-3	220.0	12.46	
RAC100-S1-1	200.7	11.36	11.63
RAC100-S1-2	193.9	10.98	
RAC100-S1-3	221.3	12.53	

By virtue of the results in Table 16, the range of the concrete mixtures' compressive strength adopted in this investigation is from 11.63 MPa to 17.70 MPa, implying that three of the mixtures qualify as low-strength ($f'_c < 17$ MPa). The varying compressive strength exhibited by the mixtures stems from the combined effect of the

replacement of NCA by RCA and the addition of steel fiber content. In general, concrete mixtures using NCA result in higher compressive strengths. As expected, the concrete mixture with the highest compressive strength is the one containing NCA and steel fibers, exhibiting a compressive strength of 17.7 MPa. Mixes containing recycled aggregate yielded the lowest compressive strength. The two variants of RAC mixes resulted in approximately equal average values of their compressive strengths (12.19 MPa and 11.63 MPa), implying ineffective use of steel fibers in the RAC mix unlike the NAC mix that saw more noticeable enhancement (increase from 14.76 MPa to 17.71 MPa) in their compressive strength with the addition of steel fibers. Despite this inferior strength of the RAC mixes compared to those of the NAC, as it will be seen in the subsequent sections, the former yields better punching shear resistance than the latter when strengthened using NSM FRP scheme. Such observation may be attributed to the positive effect of high angularity in the RAC than in NCA, thereby resulting in more aggregate interlock contribution to the shear strength. In fact the literature [52] [53] also reports comparable performance between RAC and NAC under punching shear despite lower compressive strength of the RAC.

6.2 Punching Shear Failure Modes

The five tested slab-column connections experienced a traditional punching failure mode at the ultimate load. Fig. 48 demonstrates their bottom side cracking shapes of the shear failure cone that punches through the slab. In all specimens, insignificant cases of linear cracking in the radial direction from the slab edges to the center was observed. The circular shaped punching crack was not observed until the ultimate load

was reached. Fig. 49 shows the typical NSM debonding failure at the FRP-adhesive interface experienced by all the specimens, thereby eliminating the effect of pre-grooving during casting. Away from the column location, the NSM FRP bars debonded due to the displaced truncated slab, while the structural adhesive remained bonded to the concrete groove. The NSM FRP in the proximity of the column remained unexposed and bonded with the truncated slab.

An abrupt drop in the load was recorded after reaching the maximum punching shear capacity. The failure – separation of the truncated slab and column stub from the remaining slab – was sudden in all the specimens. Upon analyzing the FRP strain measurements in all the specimens, the FRP bars appeared strongly bonded to the concrete slab throughout the test. In addition, no rupture of the FRP bars was observed in all tests. The detachment of the FRP bars from the grooves at the FRP-adhesive interface only manifested after the truncation happened. Further discussion on the effect of cone size on the FRP debonding and the resulting influence on the ultimate punching shear capacity is analyzed.



(a) RAC0-S0-G



(b) RAC100-S0-G



(c) RAC0-S1-G



(d) RAC0-S1-C



(e) RAC100-S1-C

Figure 48: Failure shapes: bottom side cracking of the shear cone that punched through each slab specimen



Figure 49: Debonding of the NSM FRP bars

6.3 Ultimate Punching Shear Capacity

Fig. 50 presents the ultimate punching shear load of the slab-column specimens, their midspan deflection at the ultimate load, and their rotation calculated using the readings from the midspan LVDT and the one adjacent to it.

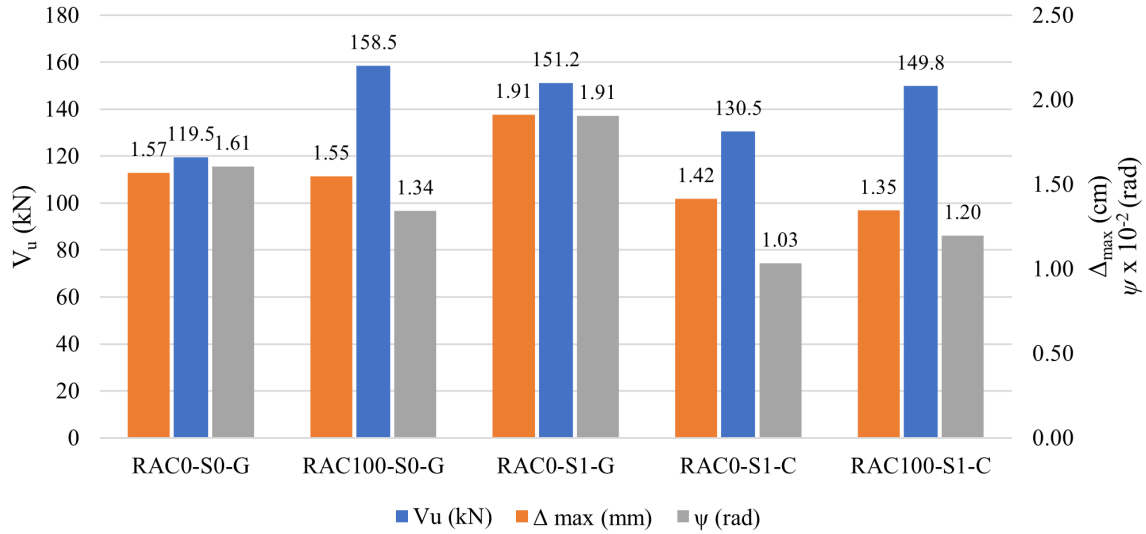


Figure 50: Ultimate load (V_u) and corresponding midspan deflection (Δ_{max}) and rotation (ψ) for each of the tested specimens

The highest (158.5 kN) and lowest (119.5 kN) ultimate punching shear capacity was realized by RAC100-S0-G and RAC0-S0-G, respectively. It is, therefore, evidenced that, among the GFRP bar strengthened specimens, the incorporation of RCA has succeeded in increasing the ultimate punching shear resistance in specimens without the addition of steel fibers. Similar observation can be made about the CFRP bar strengthened specimens, given that RCA's incorporation in RAC100-S1-C increased the ultimate punching shear load (159.8 kN) compared to the use of NCA in RAC0-S1-C that failed at 130.5 kN. Overall, specimens strengthened with GFRP outperformed those strengthened with CFRP. See, for example, RAC0-S1-G versus RAC0-S1-C, and RAC100-S0-G versus RAC100-S1-C despite the use of steel fiber in the RAC100-S1-C – Fig. 50. It can, therefore, be discerned that the use of steel fibers appears more effective in the specimens containing NCA given that RAC0-S1-G saw an improvement in capacity compared to RAC0-S0-G. With the change in FRP type, however, RAC100-S0-G and RAC100-S1-C indicate that the FRP material effect outweighed that of the use of

steel fiber. The observed superior performance of the GFRP strengthened specimens may be attributed to the suspected reason discussed in the next section.

The incorporation of RCA had an insignificant effect in reducing the maximum deflection – see RAC100-S0-G versus RAC0-S0-G and RAC100-S0-C versus RAC0-S0-C in Fig. 50. Connections strengthened with CFRP showed, generally, decreased rotation compared to those strengthened with GFRP. As expected, the use of steel fibers in the specimen RAC0-S0-G containing NAC have increased both the rotation and deflection over its control specimen RAC0-S0-G. Comparing RAC100-S0-G and RAC0-S1-G, it can be observed that, despite the incorporation of steel fibers in RAC0-S1-G, its punching shear capacity still appears lower than that of the specimen containing RAC without steel fibers (RAC100-S0-G). This could indicate the role the RAC played by virtue of its aggregate (RCA) geometric shapes and features discussed next.

In agreement with the remarkable RAC's performance realized above, existing studies [79–81] have also shown that RCA's incorporation did not have negative influence on the punching shear capacity of slabs despite the RCA's inferior mechanical properties. The increase in the punching shear strength realized by the RAC specimens is explainable based on the positive influence of better aggregate interlock in RCA compared to the more rounded NCA. In [82], it was demonstrated that the interface bonding between RCA and the new cement matrix was enhanced due to the increased interlock between the aggregate and cement matrix as well as the increased contact area. The micro-hardness values of the interfacial transition zones (ITZ) of RAC were higher than those of NAC, which was attributable to the denser microstructure associated with porosity refinement [82]. This finding is supported by the new cement paste penetrating

and filling the RCA's pores, in either the old limestone aggregate or the old adhered cement [83,84].

6.4 Effect Of Crack Pattern Size and FRP Type on The Ultimate Load

Fig. 51 summarizes the crack radii acquired from digitally reconstructed pattern by tracing the experimental crack shape of each specimen (Fig. 52) using a Computer-Aided Design (CAD) program. Upon establishing the coordinates, the maximum and minimum radii, r_{max} and r_{min} , respectively, of each shape was recorded. As Fig. 51 suggests, RCA incorporation decreased the average crack radius (r_{avg}) in the GFRP strengthened specimens (*cf.* RAC0-S0-G and RAC100-S0-G). However, an increase in the average crack radius was recorded with the use of RCA in the CFRP strengthened specimens (*cf.* RAC0-S1-C and RAC100-S1-C) which may be attributed to the stronger bond achieved by the CFRP bar than the GFRP bar in addition to the effect of the mix that is discussed in the later part of this section. For the same concrete material mix and FRP type, the influence of steel fibers resulted in reducing the crack radius (*cf.* RAC0-S0-G and RAC0-S1-G).



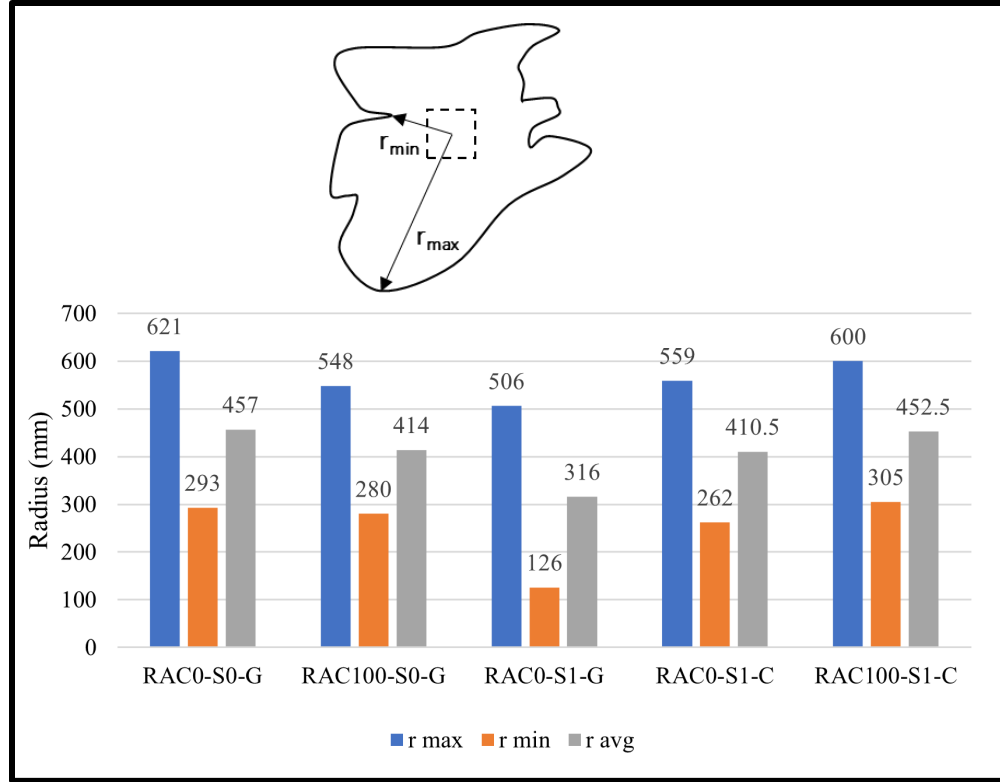
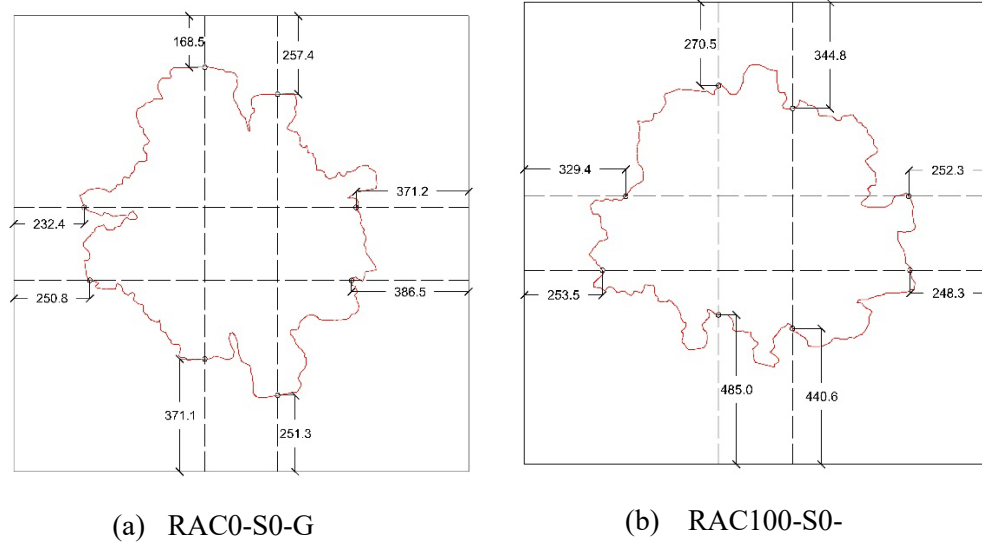
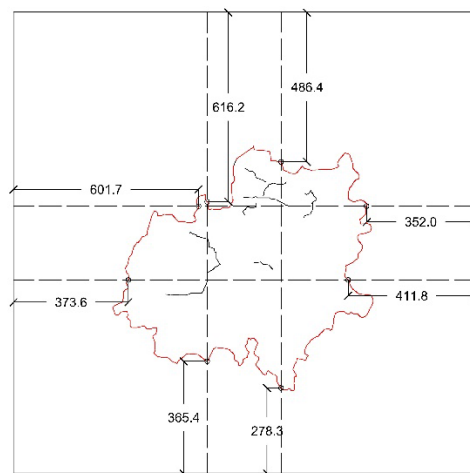


Figure 51: Crack radius of the specimen's failure cone with a schematic definition of the minimum and maximum radii of the crack

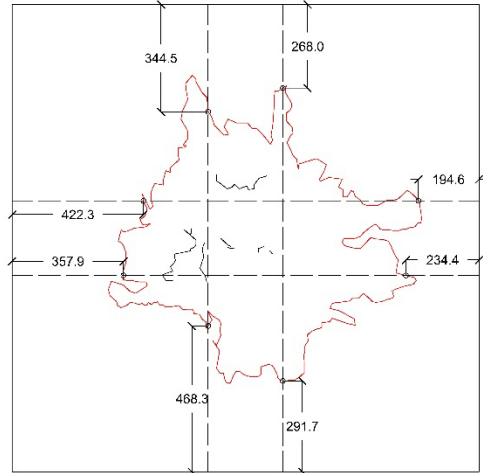
Fig. 53 demonstrates the FRP residual bonded length intersecting the shear crack perimeter, which serves as the effective bond length at the verge of specimens' failure. As shown in the figure, each specimen can be seen to have eight unequal residual bonded length values. Considering that at the verge of failure, for a given specimen, the least of these residual bonded lengths will debond faster than all the other seven residual bonded lengths, the smallest of the residual bonded length (also referred to as the critical residual bonded length) can play a significant role in governing the ultimate load in addition to other factors. For example, with no steel fibers, incorporating RCA in the GFRP-strengthened specimens saw a 47% increase in the critical residual bonded length (169 mm for RAC0-S0-G versus 248 mm for RAC100-S0-G) which corresponds to a 33% increase in the ultimate load (119.5 kN for RAC0-S0-G versus 158.5 kN for RAC100-S0-

G) Fig. 53. Similar observation can be made about RAC0-S1-G versus RAC0-S1-C and many other specimens (Table 5). An outlier to this observation, is the case of incorporating RCA in the CFRP-strengthened specimens containing steel fibers that saw a 13% decrease in the critical residual bonded length (195 mm for RAC0-S1-C versus 169 mm for RAC100-S1-C), yet the ultimate load increased by 15% (119.5 kN for RAC0-S1-C versus 158.5 kN for RAC100-S1-C), as Fig. 53 shows. This could be attributed to the stronger bond shown by the mix containing RCA and steel fiber as reported in Section 5.1, which confirmed that the combination of RCA and steel fiber exploited the effect of bridging cracks and enhanced aggregate angularity on the NSM FRP-concrete bond.

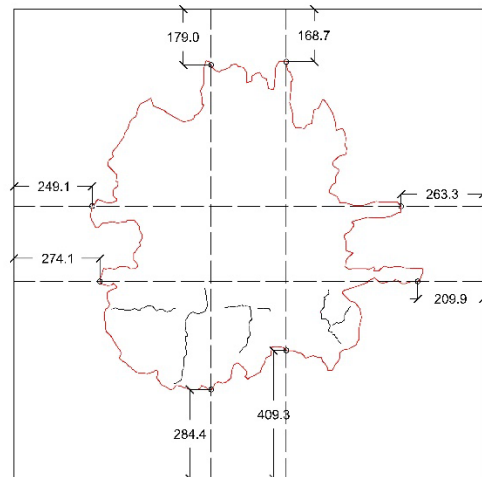




(c) RAC0-S1-G



(d) RAC0-S1-C



(e) RAC100-S1-C

Figure 52: Punching shear failure demonstrating the residual bonded length (in mm) of FRP bars. Values are indicated to scale relative to the slab layout

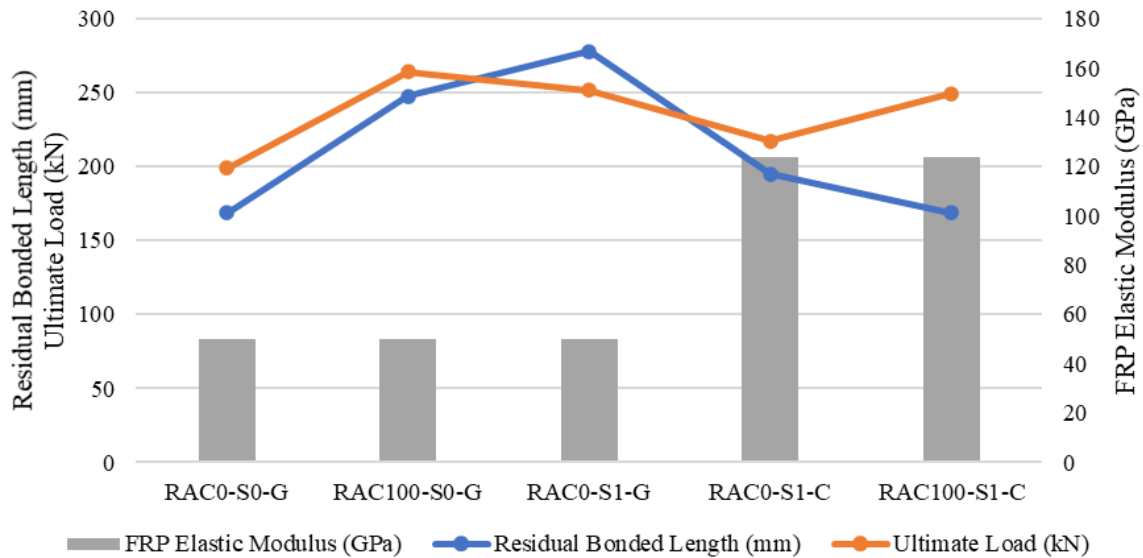


Figure 53: Summary of the critical residual bonded length of FRP, ultimate load, and FRP elastic modulus

As discussed previously that, for the same concrete mix, the GFRP-strengthened specimen (RAC0-S1-G) reported higher ultimate load than the CFRP-strengthened one (RAC0-S1-C) because the bonded length of RAC0-S1-C was lower. Fig. 48 explains the nature of the FRP bond failure experienced by all specimens. The FRP bar failed in debonding in a direction mainly orthogonal to the deep adhesive interface as illustrated in Fig. 54(a). Based on this debonding mechanism, role of the FRP bar elastic modulus becomes significant. With the punching behavior acting on the critical shear perimeter-FRP intersection region, the FRP tends to deform transversely – dowel action. Therefore, lower FRP elastic modulus would allow more of such deformation, thereby increasing the tendency of maintaining both sides of the FRP to remain bonded just prior to failure as demonstrated in Fig. 54(b). In other words, stiffer FRP material would tend to debond faster. As a result, the specimen strengthened with the higher modulus CFRP (Table 2) showed a lower ultimate load even though the CFRP possessed higher tensile strength

than the GFRP. However, it is noted that the decrease in ultimate failure load may be mitigated if the slab-column configuration allows higher critical residual bonded length of FRP than the one observed here. Between The positive influence of RCA may have been the reason behind only a negligible decrease in the ultimate strength between RAC0-S1-G and RAC100-S1-C, thereby offsetting the CFRP's elastic modulus effect in the latter. It can, therefore, be anticipated that the ultimate strength could be more noticeably higher for RAC0-S1-G than that of RAC100-S1-C if RCA was incorporated in the former. In addition, despite the use of steel fiber in RAC100-S1-C, the ultimate strength was higher in RAC100-S0-G where GFRP bar was used even though the latter mix contains no steel fiber – another example of the FRP elastic modulus effect. Table 5 summarizes the observations in the results of the tested specimens by changing the variables.

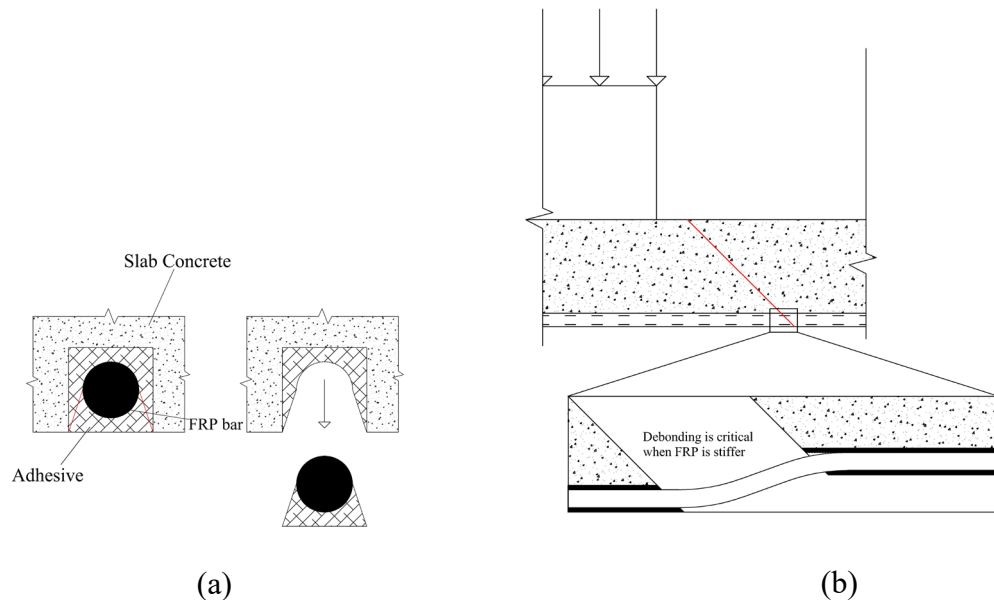


Figure 54: Schematics of the observed FRP debonding under punching failure (cf. Fig. 48): (a) adhesive-FRP debonding (b) debonding during punching is critical when the FRP is stiffer

Table 17 Summary of specimen performances regarding ultimate strength and critical residual bonded length under different variables

Specimens		Variable	Outcome		Expectation
Changing from	to		Vu	Critical residual bonded length	
RAC0-S0-G	RAC100-S0-G	RCA	↑	↑	
RAC0-S0-G	RAC0-S1-G	Steel fiber	↑	↑	
RAC0-S1-G	RAC0-S1-C	FRP type	↓	↓	
RAC0-S1-G	RAC100-S1-C	FRP type	↓	↓	Vu could be higher if 100% replacement of RCA was for the two specimens
RAC100-S0-G	RAC100-S1-C	FRP type	↓	↓	
↑ increase ↓ decrease					

6.5 Load-Deflection Response

Fig. 55 shows the load-midspan deflection behavior for all specimens. It can be observed that, in addition to having high shear capacity, RAC0-S1-G has also failed at a higher deflection than the other specimens. Such ductile behavior at ultimate loading in slabs is expected due to the incorporation of steel fibers, given that the fibers change the punching shear failure into shear-bending failure as proven by Xiao et al. [56]. However, regardless of aggregate, the remaining specimens incorporating steel fibers (RAC0-S1-C and RAC100-S1-C) were strengthened with CFRP, and they did not display such high deflection compared to RAC0-S1-G. This may be attributed to the effect of differences in modulus and bond strength issues between CFRP and GFRP as discussed in the

preceding section. All the tested specimens exhibited responses delineated by two major regimes based on the change in load-deflection slope (Fig. 55) signifying the initial and final flexural stiffnesses as discussed next.

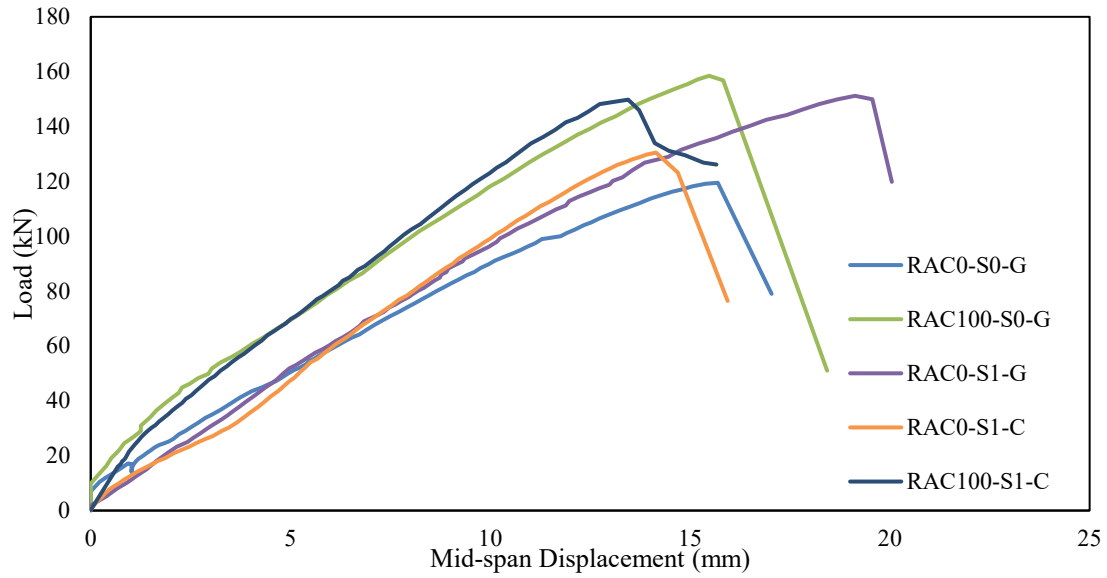


Figure 55: Load-deflection behavior of the tested specimens

6.6 Flexural Stiffness

Defining the flexural stiffness as the load divided by the mispan deflection, Fig. 56 shows the initial and final flexural stiffnesses of the tested specimens. RAC100-S0-G and RAC100-S1-C had the highest initial flexural stiffness of 16.7 kN/mm and 16.8 kN/mm, respectively, which indicated that RCA incorporation increases the initial flexural stiffness. In addition, RAC0-S1-C and RAC100-S1-C had the highest final flexural stiffness of 9.4 kN/mm and 9.9 kN/mm, respectively. As a result, it is noted that the FRP type of NSM strengthening is responsible for the final stage of flexural stiffness.

Specifically, using CFRP bars enhanced the final flexural stiffness more than the GFRP bars, despite the lower ultimate punching shear capacity of the CFRP-strengthened specimens realized and discussed in Section 6.2. The lowest decrease between the initial and final flexural stiffnesses occurred in RAC0-S1-C and RAC0-S1-G with values of 10% and 33%, respectively. Therefore, the use of steel fiber resulted in mitigating the decrease that occurs between the slab's initial and final stiffnesses, thereby having a likelihood of yielding little warning about the onset of punching shear failure.

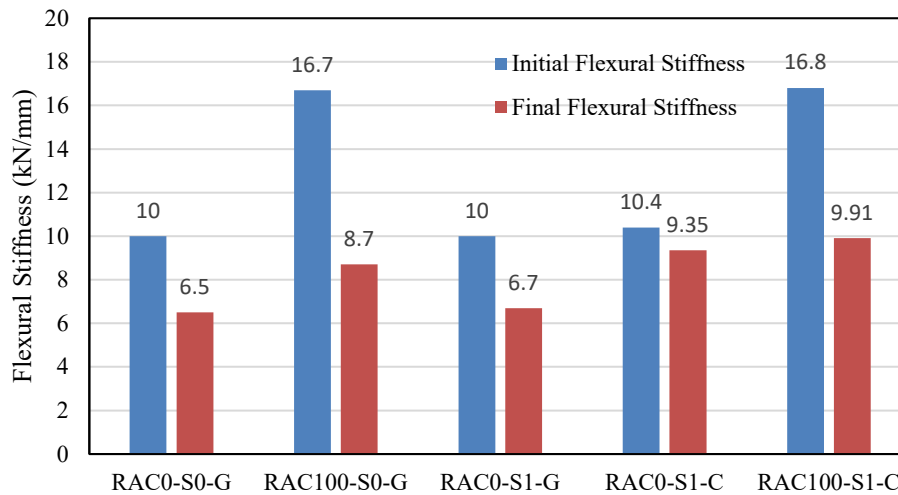


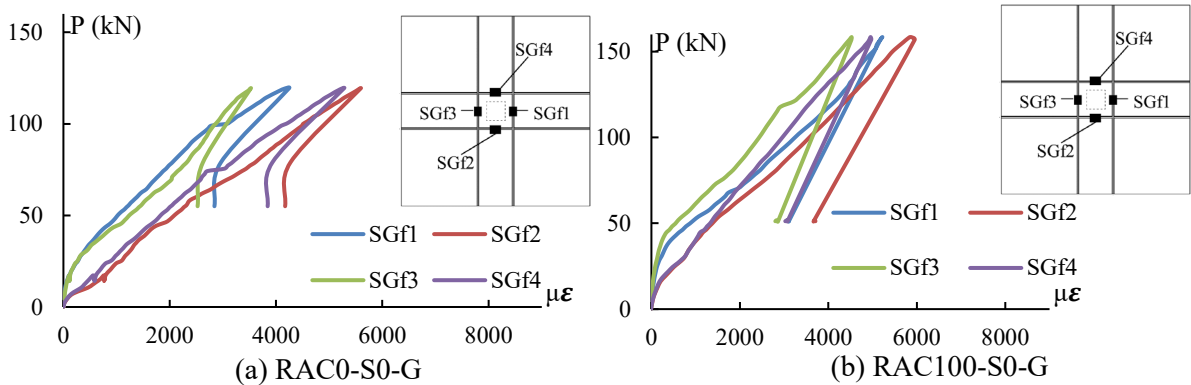
Figure 56: Flexural stiffness results of the tested specimens

6.7 FRP Strain

Fig. 57 illustrates the load-strain variations for the FRP bars used in strengthening the slab-column connections. Even though the concrete was of low quality, the bilinear relationship between the FRP bar strain and the monotonic applied load in all specimens indicates that all strengthening FRP bars had strong bond with the concrete during the

testing. The possibility of any given specimen failing under low FRP strain increases with the slope of the second bilinear section. Thus, RAC0-S1-C and RAC100-S1-C failed under lower strain utilization as shown in Fig. 57(d and e). RAC100-S1-C utilized only up to 21% ($\mu\epsilon = 4,497$) of its rupture strain ($\mu\epsilon = 17,500$), which appeared as the lowest strain utilization compared to those of other specimens. Specimens strengthened with GFRP bars failed with higher FRP strain utilization than those strengthened with CFRP, in which specimens RAC0-S0-G, RAC100-S0-G, and RAC0-S1-G reached up to 5,599 $\mu\epsilon$, 5,950 $\mu\epsilon$, and 7,642 $\mu\epsilon$, respectively, corresponding to 30%, 33%, and 42% of their rupture strains, respectively.

Incorporation of RCA resulted in different trends of strain at failure of the specimens. When GFRP bars were used, the incorporation of RCA increased the FRP strain at specimen's failure as evidenced in the strain results of RAC0-S0-G ($\mu\epsilon = 5,599$) and RAC100-S0-G ($\mu\epsilon = 5,950$) in Fig. 57. Similarly, the incorporation of RCA increased the FRP strain at specimen's failure when CFRP bars were used as portrayed by the strains of RAC0-S1-C ($\mu\epsilon = 4,014$) and RAC100-S1-C ($\mu\epsilon = 4,497$) in the same figure.



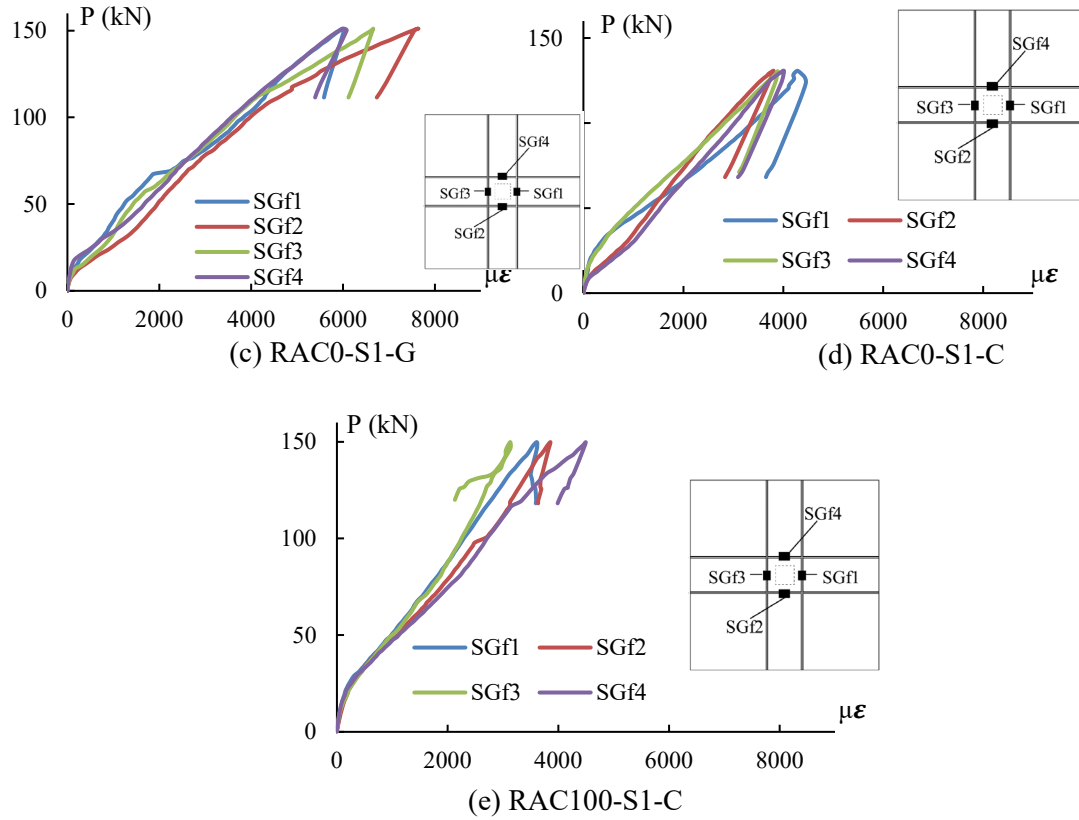


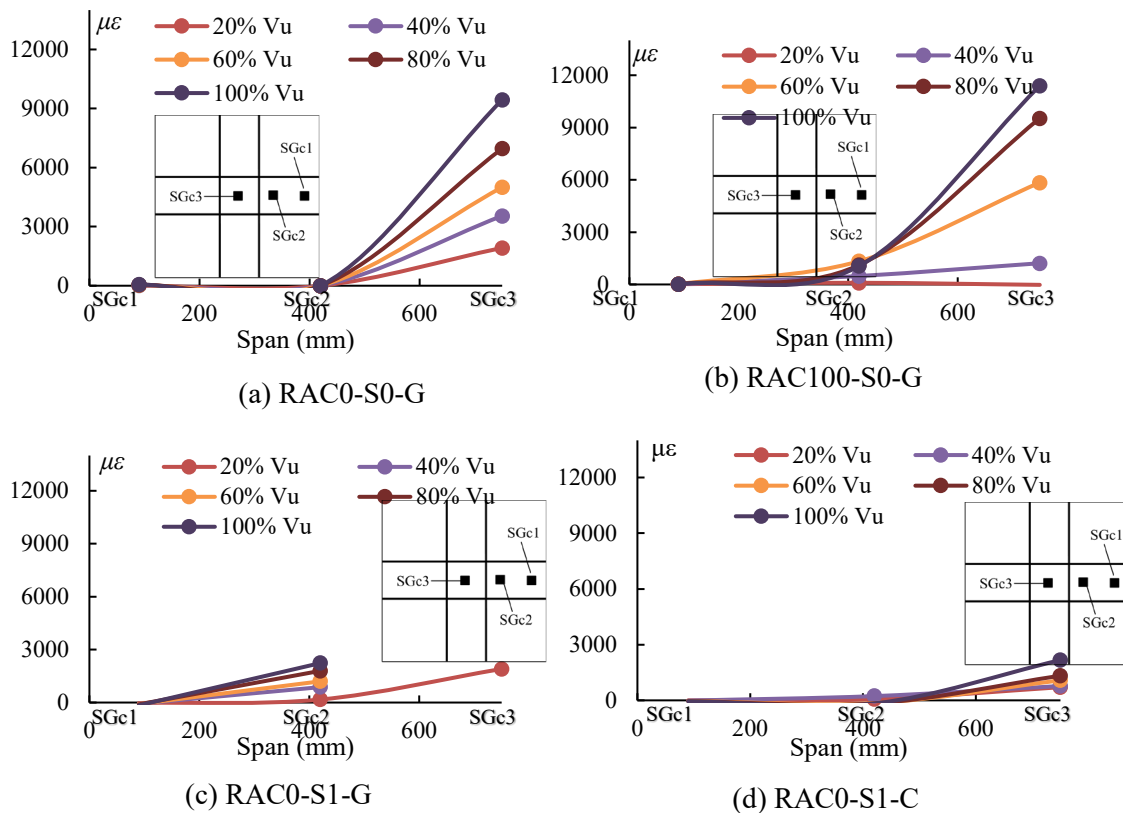
Figure 57: Load-FRP strain plots of tested specimens

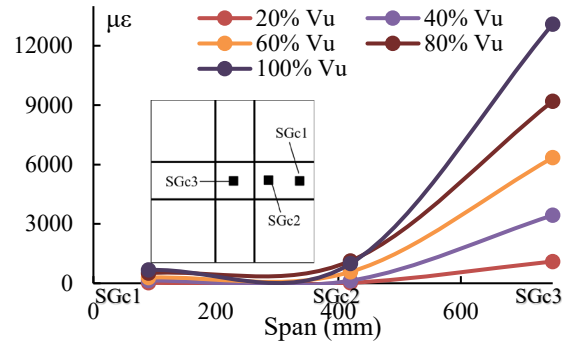
6.8 Concrete Strain

According to the arrangement of concrete strain gages, Fig. 58 shows the strain measurements of the concrete surface at the middle profile from the specimen's edge to the mid-span point while Fig. 59 illustrates the concrete surface strain measurements along the diagonal profile starting from the mid-span point. Among the GFRP strengthened specimens, RAC100-S0-G showed the highest concrete strain at failure ($\mu\epsilon = 11,400$) as shown in Fig. 58(b). Considering the CFRP strengthened specimens, however, RAC100-S1-C exhibited the highest concrete strain at failure ($\mu\epsilon = 13,092$) as shown in Fig. 58(e). It can be observed that RCA incorporation increased the maximum strain in the slab-column connections contrary to the deflection results where the

deflection of GFRP-strengthened specimens was not affected by the RCA incorporation while it only resulted in slight decrease in the deflection of CFRP-strengthened specimens.

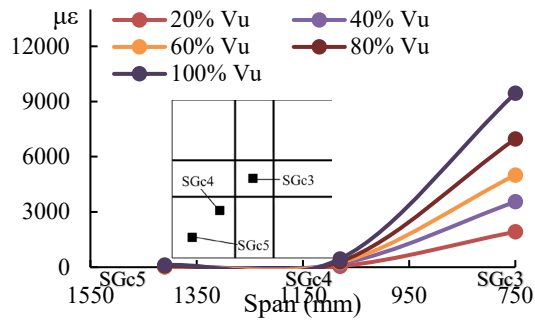
Regardless of the FRP type installed, when incorporated, RCA increased the resulting concrete strain. As shown in Fig. 58(a and b), the strain of the mid-span point (SGc3) increased by 21% by changing the coarse aggregate type from NCA in RAC0-S0-G to RCA in RAC100-S0-G. Also, a 503% increase in strain of the mid-span point (SGc3) when RCA was incorporated in RAC100-S1-C compared to RAC0-S1-C and as shown in Fig. 58(d and e). Although such extreme increase (503%) could be attributed to possible technical malfunction, yet, influence of the RCA incorporation is still significant. It can, therefore, be concluded that the tension surface of the specimens suffered higher stresses when RCA was incorporated.



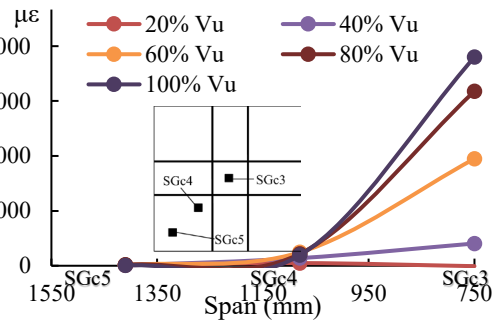


(e) RAC100-S1-C

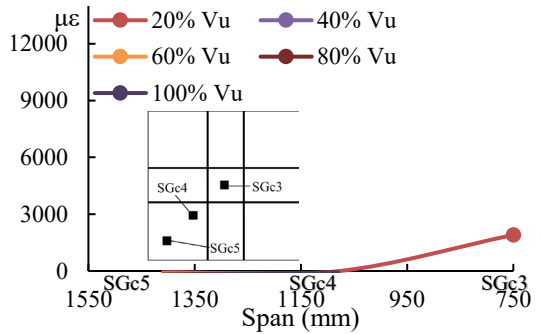
Figure 58: Concrete strain results along the mid-span point profile of the slab



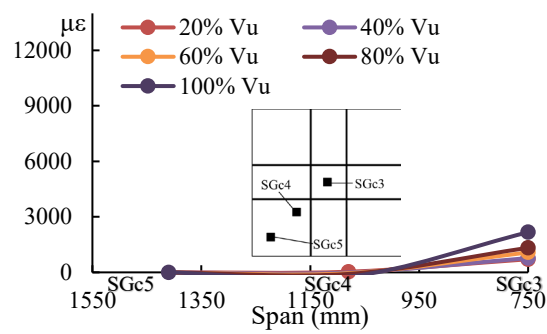
(a) RAC0-S0-G



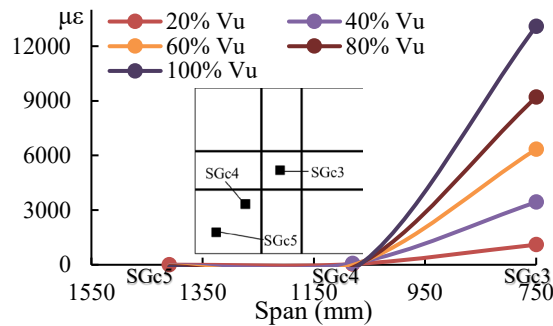
(b) RAC100-S0-G



(c) RAC0-S1-G



(d) RAC0-S1-C



(e) RAC100-S1-C

Figure 59: Concrete strain results along the diagonal profile of the slab

6.9 Steel Reinforcement Strain

Measurements of the steel rebar strain according to the arrangement shown in Fig. 8 aided in confirming that the yielding of tension rebars does not forego punching failure. Fig. 60 shows the strain results of the gages SGs1, SGs2, and SGs3 (*cf.* Fig. 8) for each specimen at different percentages of the ultimate punching shear load V_u . Steel rebar strain readings for Specimen RAC0-S1-C are not included in Fig. 60 due to their damage while casting the concrete. Due to the same reason, the strain gage SGs3 reading for RAC0-S0-G is missing [Fig. 60(a)]. As mentioned, the steel rebar material had a yield strength and Young modulus of 413 MPa and 200 GPa, respectively, its yield strain turns out to be $\approx 2,065 \mu\epsilon$. Hence, it can be observed that some yielding of the middle steel rebar of the GFRP strengthened specimens (RAC0-S0-G, RAC100-S0-G, and RAC0-S1-G) is evident based on the records of strain gages SGs2 and SGs3 [Fig. 60(a, b, and c)], whereas the corresponding bar for the CFRP strengthened specimen (RAC100-S1-C) did not indicate steel yielding at any point [Fig. 60(d)]. Although Fig. 60(a) is truncated due to the damage of strain gage SGs3, it can be observed that RAC100-S0-G would have yielded at SGs3 based on the reading of SGS2 $2,439 \mu\epsilon$ that would imply even higher strain value for SGs3 by virtue of their relative locations. Compared to RAC0-S0-G, the addition of steel fibers in RAC0-S1-G increased the strain measured at SGS2 – *cf.* Figs. 60(a) and 60(c).

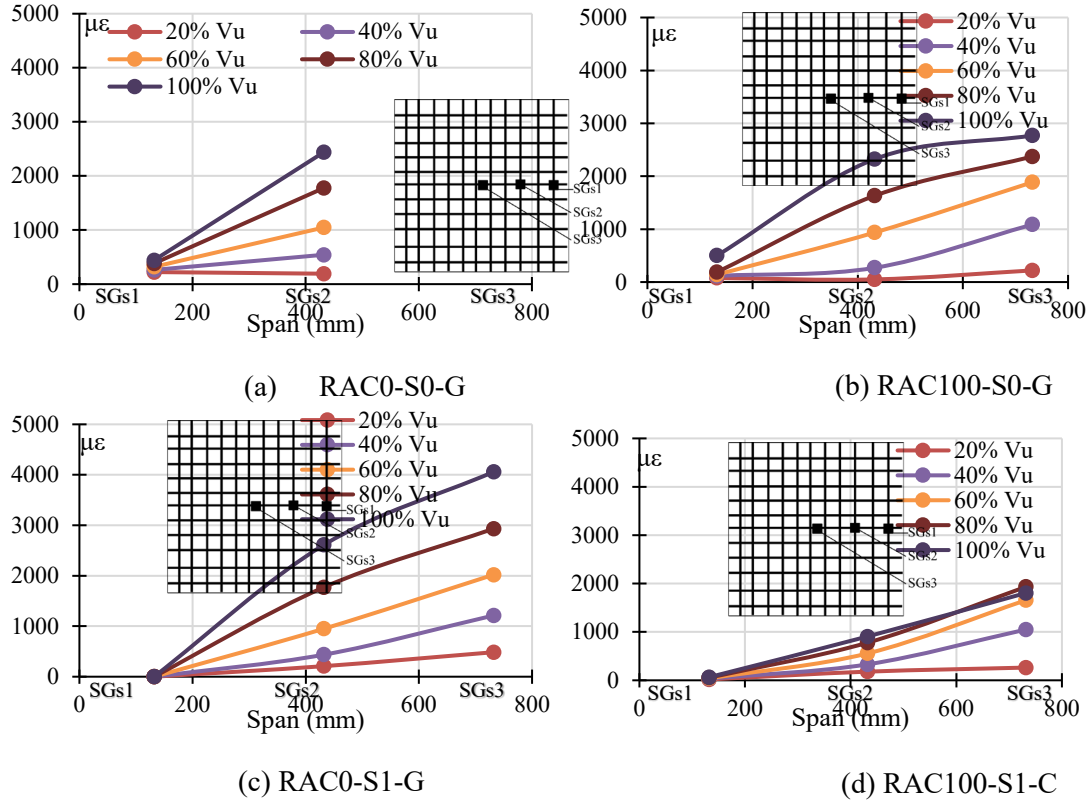


Figure 60: Measurement of steel reinforcement strains

6.10 Comparison With Normal Strength Concrete, Design Codes, And an Analytical Model

This section aims to assess the following:

- Performance of existing popular design codes and an analytical model in predicting the experimental punching shear capacities.
- Extent of punching shear capacity improvement due to the use of NSM FRP strengthening of the tested specimens compared to corresponding/conjugate specimens having no FRP strengthening.
- Extent of degradation in the punching shear capacity of unstrengthened low-strength (11.6 MPa – 17.7 MPa) concrete slab-column connection relative to that of unstrengthened connection made from a normal strength mix ($f'_c = 40$ MPa)

- Success of recovering the punching shear capacity of normal strength concrete via strengthening the low-strength concrete using the NSM FRP

Presented in Table 18, the three popular design codes, namely, ACI 318 [85], Eurocode [86], and *fib* Model Code 2010 [87] and one analytical model based on the Critical Shear Crack Theory (CSCT) [88] used in achieving the above objectives aided in augmenting the experimental results discussed in the previous section. In addition, numerical simulation of the unstrengthened slab-column connections made from the normal- and low-strength concretes was carried out using finite element method (Appendix A) to establish the most reliable among the predictions by the studied design codes (ACI 318 [85], Eurocode [86], and *fib* Model Code 2010 [87]) and the CSCT model [88] for the new cases having no experimental results. In summary, the newly added cases include four unstrengthened slab-column specimens made from the same RAC0-S0, RAC100-S0, RAC0-S1, and RAC100-S1 mixes used in the experiment of the previous sections and one unstrengthened specimen – labelled RAC0-S0-NS – possessing normal strength ($f'_c = 40$ MPa).

Table 18 Summary of the design code provisions and analytical model for punching shear of concrete

Design provision Codes	Punching shear capacity, V_c	Conditions, factors, and variables
ACI-318 [85]	Least of $\begin{cases} 0.17 \cdot \left(1 + \frac{2}{\beta}\right) \cdot \lambda \cdot \sqrt{f'_c} \cdot \mu \cdot d \\ 0.083 \cdot \left(\frac{\alpha_s \cdot d}{\mu} + 2\right) \cdot \lambda \cdot \sqrt{f'_c} \cdot \mu \cdot d \\ 0.33 \cdot \lambda \cdot \sqrt{f'_c} \cdot \mu \cdot d \end{cases}$	β: ratio of the long side to the short side of the column λ: a factor reflecting the different mechanical characteristics of lightweight concrete f'_c: 28-day concrete compressive strength α_s: a constant = 40 for interior columns d: slab effective depth μ: control punching shear perimeter

Eurocode 2 [86]	$0.18k(100\rho f_{ck})^{\frac{1}{3}} \cdot d \cdot \mu$	k: size factor ρ: flexural reinforcement ratio f_{ck}: characteristic 28-day concrete compressive strength d: slab effective depth μ: control punching shear perimeter
<i>fib</i> Model Code 2010 [87]	$k_{\psi} \frac{\sqrt{f_{ck}}}{\gamma_c} \mu d$	$k_{\psi} = \frac{1}{1.5 + 0.6 \cdot \psi \cdot d \cdot k_{dg}}$ ≤ 0.6 $k_{dg} = \frac{32}{(16 + d_g)}$ k_{ψ}: parameter that depends on the rotation of the slab γ_c: partial safety factor for concrete material properties f_{ck}: characteristic 28-day concrete compressive strength d: slab effective depth μ: control punching shear perimeter ψ: rotation of the slab around the column outside the critical perimeter k_{dg}: parameter that depends on the aggregate size d_g: maximum aggregate size
Model		
CSCT [88]	$k_b \left(100\rho f'_c \frac{d_{dg}}{r_s} \right)^{\frac{1}{3}} \cdot \mu \cdot d$	$V_c \leq 0.55 \cdot \mu \cdot d \cdot \sqrt{f'_c}$ $k_b = \sqrt{8a \frac{d}{\mu}} \geq 1$ $d_{dg} = d_{g0} + d_g$ k_b: coefficient accounting for the effective depth-to-control perimeter ratio as well as for the parameter a f'_c: 28-day concrete compressive

strength
 ρ : reinforcement ratio
 d : slab effective depth
 μ : control punching shear perimeter
 d_{dg} : reference crack roughness value
 r_s : distance from the center of the slab to the zero radial moment line
 a : constant taken as 8 for interior columns
 d_{g0} : reference roughness value of the crack (taken as 16 mm)
 d_g : maximum aggregate size

To consider the effect of steel fiber in the prediction models, the term s_f given by Eq. (3) [56] is multiplied by each code/analytical model given in Table 18.

$$s_f = \left(1 + \beta_b \cdot V_f \cdot \frac{l_f}{d_f} \right) \quad (3)$$

where, β_b is the influencing factor of the steel fibers, V_f the steel fiber volume fraction of the specimen, l_f the length of one steel fiber, and d_f the diameter of one steel fiber.

In addition to the above modification for steel fiber incorporation, it is also noted that the punching shear capacity models presented in Table 18 are meant for reinforced concrete slabs without the NSM FRP strengthening. As a result, they require some modifications to make them applicable to the investigated NSM FRP strengthened slabs. In the present study, the effective depth d and the reinforcement ratio ρ are modified based on the approach proposed by Chen and Li [89]. Hence, the following equivalent terms are substituted in each of the models presented in Table 18.

$$d_{eqv} = \frac{M_{nf}}{T_s + T_f} + \frac{a}{2} \quad (4)$$

$$\rho_{eqv} = \frac{T_s + T_f}{bd_{eqv}f_s} \quad (5)$$

where, M_{nf} is the internal resisting moment, T_s the tensile force in the steel reinforcement, T_f the tensile force in the NSM FRP bars, a the depth of the equivalent rectangular stress block, and b the unit width of the slab.

Table 19 summarizes the results of the predicted punching shear capacity of the steel fiber RAC slab-column connections of low-strength concrete production by the codes and the analytical model, and the ultimate punching shear capacities obtained from the experimental work, plotted in Fig. 61. Overall, ACI 318 [85] estimated conservative design values whereas Eurocode [86] estimated higher design values. *fib* Model Code 2010 [87] estimated relatively close punching shear capacities for specimens that do not incorporate RCA. The CSCT model [88] estimated agreeable design values compared to the experimental ultimate punching shear strength for specimens that did not incorporate RCA. Even though the experimental work produced low-strength concrete, the CSCT model [88] had better matching punching shear capacity results, regardless of RCA's incorporation, because the aggregate interlock effect is emphasized, reducing the effect of the compressive strength of concrete.

On the other hand, specimens incorporating RCA presented non-matching results compared to the CSCT model [88] which could be attributed to the additional effect of RCA, which is associated with dropping the positive RCA's influence of enhancing the punching shear capacities in the model that was reported previously and supported in the literature [82–84].

As shown in Table 19, CSCT [88] also achieved the closest punching shear capacities to the FEM results with a range of V_{FEM} -to- V_c ratio of 0.8 – 0.9. After benchmarking the FEM results of un-strengthened specimens, a 32% increase in the punching shear capacity was reported between RAC0-S0 (90 kN) and RAC0-S0-G (119 kN). In addition, the NSM GFRP strengthening enhanced the punching shear capacity by 86% between RAC100-S0 (85 kN) and RAC100-S0-G (158.5. It is important to note that RCA's effect on the punching shear behavior discussed previously was not considered in the FEM model. For the RAC0-S1 mix, the NSM GFRP and CFRP enhanced the punching shear by 50% and 29%. As RAC100-S1 was only strengthened by CFRP, RAC100-S1-C showed an increase of 78% in the punching shear capacity compared to RAC100-S1.

Table 19 Experimental, FEM, and calculated (code/model) punching shear capacities

Specimen	Experiment	FEM	Eurocode 2 [86]		<i>fib</i> MC 2010 [87]		ACI 318-14 [85]		CSCT [88]	
	V_u (kN)	V_u (kN)	$V_{c,cal}$ (kN)	$V_u/V_{c,cal}$	$V_{c,cal}$ (kN)	$V_u/V_{c,cal}$	$V_{c,cal}$ (kN)	$V_u/V_{c,cal}$	$V_{c,cal}$ (kN)	$V_u/V_{c,cal}$
Unstrengthened normal-strength specimen										
RAC0-S0-NS	-	164.3	242.3	0.7	200.7	0.8	79.1	2.1	153.4	1.1
Unstrengthened low-strength specimens										
RAC0-S0	-	90	173.5	0.5	121.7	0.7	36.9	2.4	109.9	0.8
RAC100-S0	-	85	163.1	0.5	121.1	0.7	33.6	2.7	103.3	0.9
RAC0-S1	-	101	240.0	0.4	133.5	0.7	52.6	1.7	116.9	0.8
RAC100-S1	-	84	208.5	0.4	118.1	0.8	42.6	2.1	101.5	0.9
NSM FRP strengthened low-strength specimens										
RAC0-S0-G	119.0	-	196.0	0.6	138.5	0.8	42.0	2.7	126.0	0.9
RAC100-S0-G	158.5	-	174.8	0.9	113.4	1.4	31.5	5.0	105.8	1.5
RAC0-S1-G	151.2	-	286.5	0.5	187.3	0.8	73.8	2.0	163.5	0.9
RAC0-S1-C	130.5	-	290.6	0.4	154.5	0.8	60.9	2.1	145.3	0.9
RAC100-S1-C	149.8	-	226.1	0.7	90.4	1.7	32.6	4.6	91.3	1.6

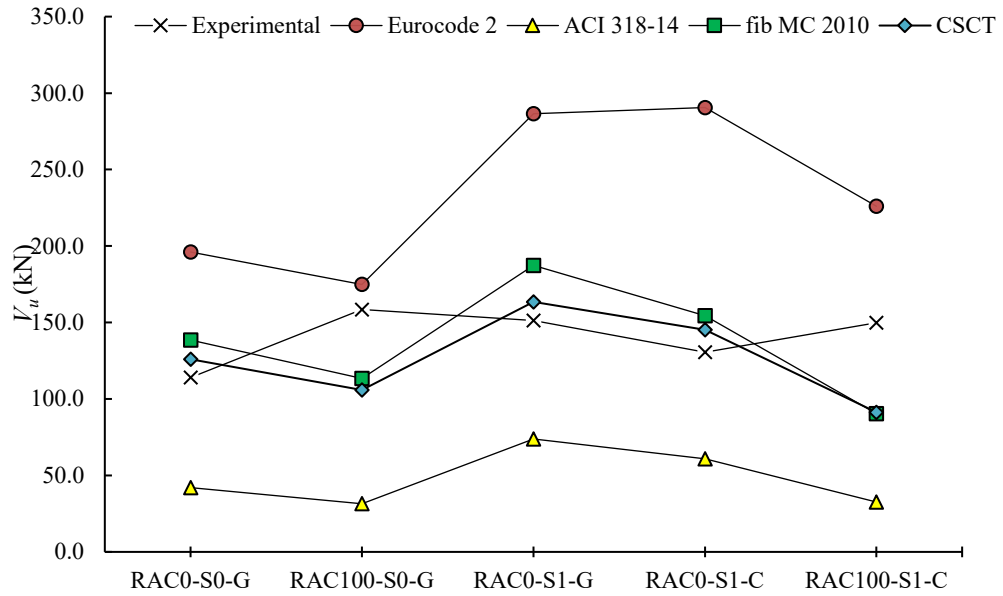


Figure 61: Experimental and predicted punching shear results

Generally, it can be concluded that low-strength concretes possessing inferior mechanical properties, including compressive strength, tend to provide needed punching shear capacities for retrofitting projects when low compressive strength is exhibited. Even though few codes limit the utilization of NSM FRP when low-strength concrete is encountered, the results from this paper promote the use of NSM FRP on low-strength concrete as it raises the capacity to that of a normally designed column-slab connection or higher.

It appears that, regardless of steel fiber content, the RAC slab-column connections provided an acceptable punching shear capacity for a specified scale of 0.5, which establishes the possibility for RCA to be incorporated in various applications for slab- column connections. The connection deflection's insensitivity towards RCA's incorporation supports the use of RCA in concrete applications where serviceability is a constraint.

CHAPTER 7

CONCLUSION AND RECOMMENDATION

7.1 Summary

This thesis experimentally investigated the bond behavior of NSM FRP strengthening in lowstrength steel fiber recycled aggregate concrete (RAC) mixtures. The variables included steel fiber content and recycled aggregate incorporation ratio, resulting in the mixes having 0%, 30%, 60%, and 100% RCA substitution for each of 0%, 0.5%, and 1% steel fiber per specimen volume. As the experimental program of the NSM FRP-concrete bond behavior was carried out for a limited number of low- and normal-strength concrete specimens, the study adopts the finite element analysis to widen the spectrum of the study to include more ranges of the mixture's quality (both low- and high-strength materials) and other key parameters not covered in the experiment. Also, Using a Universal Debonding Test Apparatus FRP-Concrete System, the work evaluated three different types of NSM bars and strips in single-shear, double-shear, and beam-bend pull-out tests to demonstrate the performance of a unified apparatus for diverse test techniques. Bond strength, bond-slip, and local behavior of bond were analyzed. Finally, this study conducted and critically analysed an experimental campaign on the behavior of NSM FRP strengthened steel fiber RAC slab-column connections against punching shear failure. The three variables studied include steel fiber addition, recycled aggregate replacement, and FRP bar material type – GFRP versus CFRP. Of the five punching shear tests conducted on low-strength concrete, three and two specimens were strengthened with GFRP and CFRP

bars, respectively, based on NSM approach. The three GFRP bar strengthened specimens differ in the type of concrete used, namely, ordinary concrete without steel fibers, ordinary concrete with steel fibers [the so-called fiber reinforced concrete (FRC)], and RAC without steel fibers. Among the two NSM CFRP strengthened specimens, one was made from ordinary concrete and the other from RAC, both with steel fibers, thereby resulting in recycled aggregate FRC. The study's experimental findings are examined along with findings from one analytical model and three design codes (ACI 318, Eurocode 2, and *fib*). The experiments' findings conclude the following inferences:

- 1) 44.5% to 61.7% reduction in the ultimate load was observed when low-strength concretes were used across the range of the studied variables.
- 2) Generally, steel fibers marginally improved the low-strength concrete-NSM FRP bond by reducing crack propagation during loading. A more significant influence of the steel fiber content was clearly observed in specimens with full RCA incorporation.
- 3) The best-performing low-strength concrete-NSM FRP specimen, in terms of ultimate bond failure load, had a decrease of 44% compared to the normal-strength concrete.
- 4) Depending on the scenario, the use of NSM FRP strengthening for low-strength concretes should be supplemented with other strengthening strategies to overcome the observed inadequacies reported herein.

- 5) A range of 0.9 – 1.03 experimental-to-FEM ratio of the maximum loads was reported considering the effect of the steel fiber in the FE model.
- 6) Based on the FE parametric study, the maximum load of the NSM FRP-concrete bond increases with the increase in GFRP bar diameter for a given GFRP modulus and concrete compressive strength. Moreover, increasing the steel fiber content increases the ultimate load of the bond.
- 7) The Universal Debonding Test Apparatus proved the validity of performing multiple NSM bond tests in a convenient way. The multiple types of NSM bond tests can be performed. Despite avoidance of beam-bend test in literature, the beam specimens performed well under the studied setup giving consistent results.
- 8) The double-shear bond test results have shown more balanced strain distribution compared to the single-shear test. This was achieved by the apparatus although imperfect concrete casting was present. Using this apparatus, in double-shear testing, the latter segment of the bonded FRP was triggered earlier compared to the single-shear testing, which can give ideal results when perfect specimen balance is present.
- 9) Overall, the circular CFRP bar showed the superior results in terms of load and slips at failure. In terms of equivalent shear stress, circular CFRP bar reached 10.9 MPa and 8.66 MPa for single-shear and double-shear test, respectively. Compared to the other types, the circular CFRP bar tend to fail at the first segment of FRP, near the loaded end.

- 10) By introducing a 15 mm edge distance, specimens with circular CFRP bar exhibited 56% and 12% higher ultimate load than specimens with circular GFRP and rectangular CFRP, respectively. Compared to single-shear test results, the 15 mm concrete edge distance reduced the ultimate load by more than 40% when a 40-MPa concrete is used with an equivalent FRP cross-section area of 78.5 mm^2 and groove dimension of $1.5d_b \times 1.5d_b$.
- 11) The incorporation of RCA enhanced the punching shear capacity of the NSM FRP strengthened flat slabs with or without steel fibers, where the GFRP strengthened specimen without steel fibers saw a 33% capacity increase (119.5 kN to 158.5 kN) while the CFRP strengthened specimen with steel fibers saw a 15% capacity increase (130.5 kN to 149.8 kN). The reasons behind such enhanced punching shear performance in the RAC specimens corroborates the literature findings of cases of RAC slab punching without NSM FRP strengthening.
- 12) RCA incorporation increases the initial flexural stiffness of the slab – 67% and 62% for the GFRP and CFRP strengthened specimens without and with steel fibers, respectively. The final flexural stiffness, however, got affected more by the change in FRP type, in which GFRP yielded a higher enhancement (34%) than the CFRP (6%).
- 13) Use of RAC in the NAC samples appear to have more significant improvement (33%) in the punching shear capacity than the use of steel fibers (with 27% improvement), at least for the concrete strength grades studied here.

- 14) Use of steel fibers in the NAC samples resulted in mitigating the decrease that occurs between the slab's initial and final stiffnesses – the decrease being 10% and 33% for the CFRP and GFRP strengthened specimens, respectively, thereby having a likelihood of yielding little warning about the onset of punching shear failure.
- 15) The CFRP strengthened specimen showed lower ultimate punching shear load (149.8 kN) than the GFRP strengthened specimens (with 158.5 kN) despite the use of steel fiber in the latter and not in the former and, moreover, although the CFRP bar possessed higher tensile strength than the GFRP bar. This may be attributed to the higher FRP's elastic modulus that tends to make it more rigid to not slightly deform at the formation of the punching failure crack (dowel action), thereby accelerating the FRP bar debonding normal to the FRP-adhesive interface.
- 16) The CFRP strengthened specimens exhibited decreased rotation values during punching than their GFRP strengthened counterparts. However, since the concrete mix used in the former contains steel fiber while that of the latter has no steel fiber, then the observed decreased rotation may be a compound effect of both the change in NSM bar type and concrete mix design.
- 17) The Critical Shear Crack Theory outperforms the three appraised design codes (ACI 318, Eurocode 2, and fib) in predicting the experimental observations for all the tested specimens irrespective of whether or not steel fibers or RCA are used in the concrete mix. However, although performing well in the specimens made with NAC,

the model itself has shown some slight deviations when applied to specimens made of RAC, a likely reason being the inherent unique geometric features of RCA along with its surface texture that makes it different from the NCA which existing models are based on.

18) Given that the present study focuses on testing and analyzing strengthened specimens made from low-strength (11.6 MPa – 17.7 MPa) concrete mixes so that the use of NSM FRP strengthening could revamp their performance under punching shear, finite element models of a normal strength concrete ($f'_c = 40$ MPa) and a further set of four low-strength (unstrengthened specimens) were developed. Their results validated by the codes/analytical still indicates the better performance of the Critical Shear Crack Theory than the codes. The so analyzed additional cases aided the understanding of the degree of success achieved in recovering the punching shear strength of low-strength concrete mixes strengthened with NSM FRP. For example, it was found that the use of RCA in low-strength concrete without and with NSM GFRP yielded a punching shear capacity of 90 kN and 158.5 kN, respectively – a reasonable recovery of the normal strength concrete slab capacity (164 kN based on the finite element prediction).

In general, strengthening the low-strength column-slab connection by NSM FRP is acceptable after reaching a level of normal reinforced concrete production of flat slabs. Further studies may investigate mechanical parameters to seek the factor that represents RCA on the behavior of concrete undergoing punching shear loading.

7.2 Future Work

Further studies may investigate:

- Mechanical parameters to seek the factor that contributes to the behavior of low-quality concrete on the ultimate punching shear.
- The behavior of RCA's interlock in concrete for shear-dependent states.

APPENDIX

To ensure reporting valid differences between un-strengthened specimens and the experimental strengthened specimens, a Finite Element Model is adopted to calculate the ultimate load of the punching shear test for control specimens that are not strengthened (RAC0-S0, RAC100-S0, RAC0-S1, and RAC0-S1) and an un-strengthened specimen possessing normal strength ($f'_c = 40$ MPa) using the commercial program ANSYS [72]. Fig. 62 shows the RAC0-S0's model with its essential components: concrete and steel reinforcement. For the test boundaries, a displacement boundary condition was applied on the top surface of the column stub and edge support representing simple support and only restraining the movement in the z-axis with free rotation. To prevent rigid movement, the two lines of support, shown in Fig. 62(b), that are aligned at the y-axis were restrained from movement in the y-axis. Also, the restraints of movement in the x-axis were applied on the two lines of support aligned at the x-axis. Regarding the element types, Solid65 was used to describe the concrete block which is a brick element with eight nodes and three translational degrees of freedom (DOFs) per node that was capable of cracking, crushing, and plasticity. Link180 element was used to describe the steel reinforcement. 10 mm-cubic elements were used to mesh the concrete block. Using Willem and Warnke's five-parameter concrete failure criterion, the concrete plasticity damage model adopted the compressive strength results shown in Table 3 and estimated parameters from existing literature [73,90]. The steel reinforcement was modeled as an isotropic elastic, using the mechanical characteristics as the

reinforcement used in the experiment. Fig. 63 shows the failed FE model of the control specimen with complete damage of concrete elements in the punching shear perimeter.

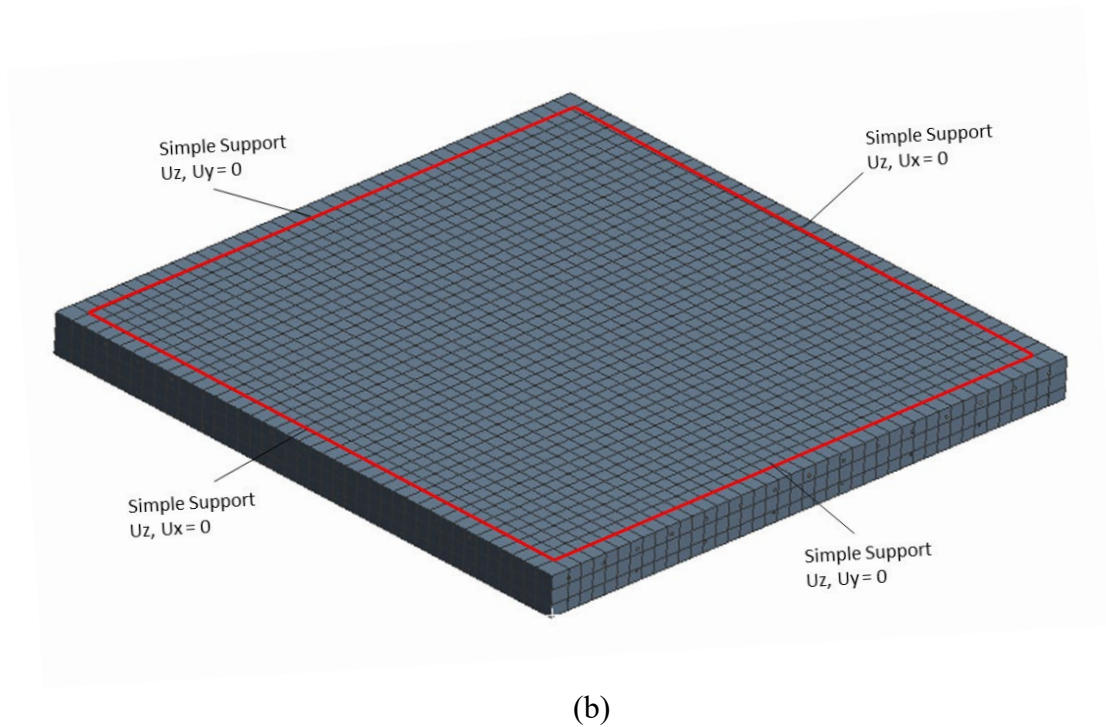
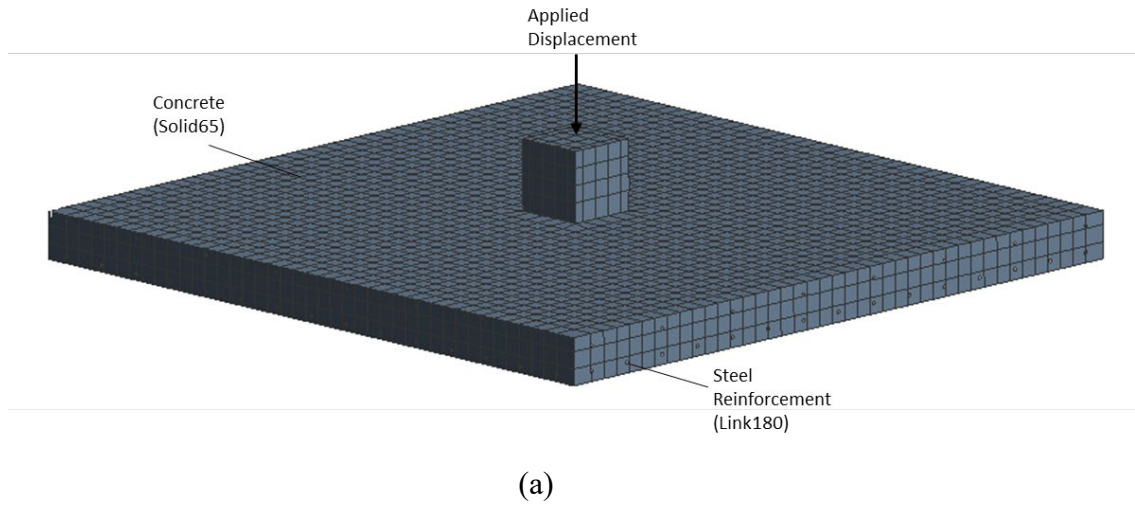


Figure 62 RAC0-S0's FE model: (a) top and (b) bottom

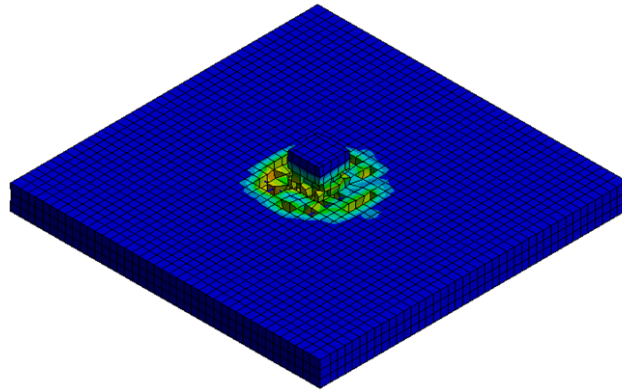
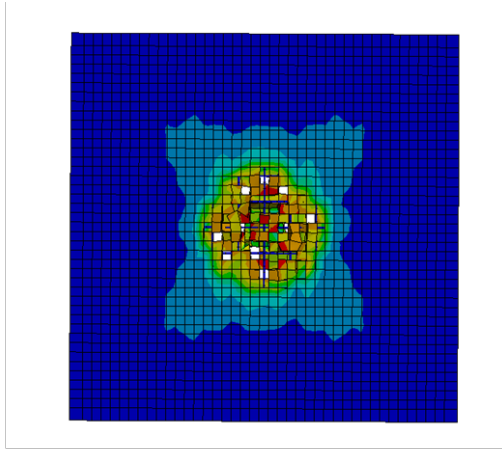


Figure 63 The damaged specimen of RAC0-S0 at failure

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EDUCATION

M.Sc. in Civil Engineering

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PUBLICATIONS

Lardhi, M. and Mukhtar, F. (2023) Radiation shielding performance of seawater-mixed concrete incorporating recycled coarse aggregate and steel slag. *Journal of Radiation Research and Applied Sciences*.

Lardhi, M. and Mukhtar, F. (2023) Experimental and FE analysis of NSM FRP bond in low- and normal-strength natural and recycled aggregate concretes. *Construction and Building Materials*.

Lardhi, M. and Mukhtar, F. (2023) Punching shear of recycled aggregate FRC slabs strengthened with NSM FRP bars. *Engineering Structures*. - Under Review

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RESEARCH INTERESTS

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- Recycled Materials in Concrete
- Structural Dynamics
- Fiber-reinforced Polymer Strengthening of Structural Elements
- Bio-inspired Materials in Structural Engineering