Geopolymer- and Cement-Based Fabric-Reinforced Matrix Composites for Shear Strengthening of Concrete Deep Beams: Laboratory Testing and Numerical Modeling

Nour Khir Allah, Tamer El-Maaddawy * (✉) and Hilal El-Hassan

Department of Civil and Environmental Engineering, College of Engineering, Al Ain Campus, United Arab Emirates University, Al Ain 15551, United Arab Emirates; 201770375@uaeu.ac.ae (N.K.A.); helhassan@uaeu.ac.ae (H.E.-H.)

* Correspondence: tamer.maaddawy@uaeu.ac.ae

Abstract: This paper examines the effectiveness of using carbon fabric-reinforced matrix (C-FRM) composites to improve the shear response of reinforced concrete (RC) deep beams. Ten RC deep beams were tested. Test variables included the presence of internal steel stirrups, number of C-FRM layers, angle of inclination of the second layer of C-FRM, and type of matrix. In the absence of minimum steel stirrups, the use of one layer of C-FRM with cementitious and geopolymeric matrices resulted in 95% and 77% shear strength gains, respectively. Increasing the number of C-FRM composites to two layers insignificantly increased the shear strength gain. Positioning the second layer of C-FRM in the vertical direction tended to be more effective than placing it in the horizontal direction. The gain in the shear capacity was less pronounced in the presence of steel stirrups, where a maximum shear strength gain of 18% was recorded. Numerical models were developed to predict the shear response of the tested beams. Outcomes of the numerical modeling were in good agreement with those obtained from the tests. The inclusion of a bond–slip law at the fabric–matrix interface insignificantly reduced the predicted shear strength. The ratios of the predicted-to-measured shear capacity of the models with and without the bond–slip law were, on average, 0.90 and 0.95, with corresponding standard deviations of 0.09 and 0.11, respectively.

Keywords: deep beams; shear; strengthening; carbon; fabrics; composites; cementitious; geopolymeric

1. Introduction

Shear strengthening of existing RC beams is frequently required in practical settings due to an increase in the applied live load or updated code design loads. Nonmetallic fabric-reinforced cementitious matrix (FRCM) composites consist of fabrics, such as carbon (C), glass (G), and poly(paraphenylene benzobisoxazole (PBO) embedded into a cementitious matrix [1]. The nonmetallic fabrics inhibit corrosion of the strengthening system, which makes them suitable for use in highly corrosive environments. Due to the use of cement-based matrices rather than epoxy, FRCM composites possess an improved heat resistance and better compatibility with the concrete substrate relative to those of the epoxy-based fiber-reinforced polymer (FRP) composites [1–3].

Shear strengthening with FRCM reduced the deflection at service loads, delayed yielding of stirrups, and reduced surface cracks, thus offering an improved shear resistance [2–9]. Failure of RC beams strengthened in shear with FRCM could be due to fabric slippage/rupture, local debonding of the composite layer, debonding at the fabric–matrix interface, or separation of the side concrete covers [2–9]. A shear compression mode of failure was also reported in some studies for RC beams heavily strengthened in shear [4–6]. The strength of the concrete and shear span-to-depth ratio (a/h) had an insignificant effect on the contribution of FRCM to the shear capacity [5,7]. The improvement in the shear strength caused by FRCM may be comparable to that caused by FRP composites [8–10].
The efficiency of FRCM shear strengthening decreased with an increase in the number of internal steel stirrups [9–11]. Increasing the number of FRCM layers resulted in minimal or a non-proportional additional gain in the shear capacity [4,6,7,11–13]. Anchoring of vertical FRCM strips into slits pre-cut on the surface of the concrete cover at the ends of the FRCM in shear-strengthened RC beams did not result in a full utilization of the tensile strength of the FRCM composites due to pull-out of the fabric from the slit or extraction of the entire anchoring from the top surface of the concrete [14]. In contrast, the use of fan-shaped textile-based anchors (also called spike anchors) bonded to the top end of vertical FRCM strips and anchored vertically into the flange dramatically improved the shear strength gain [15].

Reinforced concrete deep beams (i.e., beam with $a/h \leq 2$) are typically used as transfer girders in high-rise buildings. External loads in RC deep beams are transferred directly to the supports through concrete struts in the shear span (i.e., internal arch action effect), which makes their shear capacity sensitive to the splitting and compressive strengths of the diagonal strut [16]. The shear behavior of RC deep beams is affected by the presence of stirrups and type/configuration of FRCM composites [16–21]. Doubling the number of C-FRCM layers resulted in only a 21% additional increase in the shear capacity of concrete beams having $a/h$ of 1.7 without internal steel stirrups [17]. The shear strength gain reported for RC deep beams with stirrups was approximately 50% of that reported for similar beams without stirrups [16,18]. Azam et al. [16] reported shear strength gains in the range of 13 to 23% for RC deep beams without stirrups and 8 to 16% for those with internal stirrups. Other researchers reported that the presence of internal steel stirrups reduced the average shear strength gain in RC deep beams from 60 to 30% [18]. Continuous FRCM configuration was more effective than the discontinuous configuration in improving the shear capacity of RC deep beams [17–19]. Deep beams strengthened in shear with C–FRCM experienced smaller crack widths and higher shear capacity than those of their counterparts strengthened with PBO– or G–FRCM systems [17–20]. This behavior was attributed to the increased axial stiffness of C-FRCM composites relative to that of other types [17–20]. The inclusion of mechanical anchorage at the top end of the vertical FRCM strips had no impact on the shear capacity of strengthened RC deep beams [20]. Although the use of different end anchorage systems improved utilization of the fabric tensile strength, the fabric was not ruptured in any of the tested deep beams [21].

Despite the advantages offered by the inclusion of a cementitious matrix in FRCM composite systems, manufacturing of cement generates a significant amount of carbon dioxide. Few researchers investigated the possibility of using cement-free geopolymeric matrices instead of cementitious mortars to create a sustainable fabric-reinforced geopolymeric matrix (FRGM) strengthening solution [22–25]. A very good adhesion between geopolymers and metallic chords was reported in previous studies [23,24]. Composite layers of C-FRGM made with carbon fabrics and a geopolymeric matrix exhibited a bond behavior comparable to that of C-FRCM made with the same type of fabric but with a cementitious mortar [25]. Some researchers reported, however, a premature debonding mode of failure at the fabric–matrix interface for RC beams strengthened in flexure with C-FRGM composites [22,23]. No information is available in the literature on the structural performance of large-scale RC beams strengthened in shear with FRM composites involving nonmetallic fabrics and a geopolymeric matrix.

This research provides new knowledge on the shear behavior of RC deep beams strengthened with C-FRCM and C-FRGM composites. It offers new information on the potential use of geopolymer-based FRM composites as a sustainable structural solution for strengthening of RC structures. The work includes laboratory testing and FE modeling. The shear responses of RC deep beams, with and without internal stirrups, strengthened with C-FRGM were examined and compared to those of similar beams strengthened with C-FRCM. The effects of the number and orientation of C-FRCM layers on the shear response were investigated. The FE models developed in the current study employed realistic material laws that accounted for the nonlinear behavior of materials and the bond behavior
at the fabric–matrix interface. The FE models were capable of predicting the response of the tested beams with good accuracy; hence, they can be considered a valuable alternative to laboratory tests.

2. Experimental Program

The experimental study comprised the testing of ten RC beam deep beam specimens with $a/h$ of 1.6. Test parameters included the presence of internal steel stirrups, number of C-FRM composite layers, angle of inclination of the second layer of C-FRM, and type of matrix.

2.1. Test Matrix

The test matrix is presented in Table 1. The notation NS denotes no internal steel stirrups, while ST refers to presence of stirrups. The letters C and G refer to cementitious and geopolymeric matrices, respectively. The numbers 1 and 2 denote the number of C-FRM strengthening layers, 90 refers to the angle of inclination of the fabric in case of one or two layers in the vertical direction, and $0/90$ refers to the angle of inclination of the fabric in case of two layers, one in the vertical direction and one in the horizontal direction. The specimens were divided into four groups. The control group included two beams, one with internal steel stirrups and one without stirrups. The matrix used in strengthening of specimens in groups A and B was cement-based, whereas that used in specimens in group C was geopolymeric. Group A included three specimens without internal steel stirrups. The three specimens were strengthened in shear with C-FRCM (i.e., with a cementitious matrix). Specimens NS-C1-90 and NS-C2-90 were strengthened with one and two layers of C-FRCM, respectively, in the vertical direction. Specimen NS-C2-0/90 was strengthened with two layers of C-FRCM; one layer had a fabric aligned in the vertical direction, while that of the second layer was aligned in the horizontal direction. Group B consisted of three specimens with internal steel stirrups. The three specimens were strengthened in shear using the same C-FRCM schemes as those of their counterparts from Group A. Group C included two specimens strengthened in shear with one layer of C-FRGM (i.e., with a geopolymeric matrix) aligned in the vertical direction. One specimen had internal steel stirrups, whereas the other did not include stirrups. It should be noted that laboratory testing of large-scale RC structural elements, such as the deep beams of the current study, are costly, time-consuming, and labor intensive. As such, only one beam was tested for each configuration. Nevertheless, the numerical investigation included in the current study is meant to offset this limitation.

| Group | Matrix Type | Presence of Stirrups | C-FRM Layers | Angle of C-FRM | Designation |
|-------|-------------|----------------------|--------------|---------------|-------------|
| Control | -           | -                    | -            | -             | Control-NS  |
| A     | Cementitious| -                    | One layer    | 90-degree     | NS-C1-90    |
|       |             |                      | Two layers   | 90-degree     | NS-C2-90    |
|       |             |                      | Two layers   | 0/90-degree   | NS-C2-0/90  |
| B     | Cementitious| √                    | One layer    | 90-degree     | ST-C1-90    |
|       |             |                      | Two layers   | 90-degree     | ST-C2-90    |
|       |             |                      | Two layers   | 0/90-degree   | ST-C2-0/90  |
| C     | Geopolymeric| -                    | One layer    | 90-degree     | NS-G1-90    |
|       |             | √                    | One layer    | 90-degree     | ST-G1-90    |

Table 1. Test matrix.
2.2. Test Specimens

Details of the specimens with and without steel stirrups are shown in Figure 1a,b, respectively. The specimens were 3300 mm long, 150 mm wide, and 500 mm deep. The beams were designed to fail in shear without yielding of the flexural reinforcement. The longitudinal reinforcement in the tension side consisted of 4 bars with a diameter of 25 mm (4φ25) placed at a distance \( d = 450 \) mm from the compression face, with a tension steel ratio of \( \rho = 0.029 \). The compression reinforcement consisted of 2φ25 placed at a depth of 25 mm with a compression steel ratio of \( \rho' = 0.015 \). The internal shear reinforcement, when it existed, consisted of φ5 stirrups at a spacing of 80 mm in both vertical and horizontal directions, which corresponded to a shear reinforcement ratio of 0.0033 in each direction. This arrangement satisfies the provisions of the ACI 318-19 [26] for deep beams, which requires steel stirrups in each direction with a maximum spacing of \( d/5 = 90 \) mm and a minimum shear reinforcement ratio of 0.0025 in each direction. Three stirrups were provided in the free region outside the supports to improve the end anchorage resistance, hold the steel cage, and maintain positions of the steel reinforcing bars during casting. The steel reinforcing bars became bonded to the hardened concrete after casting and curing; therefore, their position inside the beam was not altered. The absence of steel stirrups within the beam span did not have any effect on the position of the steel reinforcing bars during testing. Test observations confirmed that the position of the steel reinforcing bars inside the beam during the structural test to failure was maintained.

![Figure 1. Details of test specimens (dimensions are in mm): (a) Specimens without steel stirrups; (b) Specimens with steel stirrups.](image)

2.3. Materials

Test specimens were constructed using a ready-mixed normal-strength concrete obtained from Readymix Abu Dhabi, Al Ain, UAE. The concrete mix proportions per cubic meter are given in Table 2. Ordinary Portland cement (OPC), obtained from Emirates Cement Factory, Al Ain, UAE, was used in the mixes. The water-cement ratio (w/c) of the concrete mix was 0.55. The coarse aggregates were a mix of 10 mm (33%) and 20 mm (67%) crushed aggregate, obtained from Al Buraimi Crusher, Al Ain, UAE. The fine aggre-
gates were a blend of dune sand (37%), obtained from Al Ain Municipality, Al Ain, UAE, and 5 mm crushed aggregate (63%), obtained from Stevin Rock, Ras Al Khaimah, UAE. Ten cylinders (150 × 300 mm) and five cubes (150 × 150 × 150 mm) were sampled from the concrete during casting. The admixture POZZOLITH® LD10, manufactured by BASF, Dubai, UAE, was added at the plant at a dosage of 1.5 L/m³ to improve the concrete workability and give a controlled retardation of initial set. Five cylinders were used to determine the concrete compressive strength, while the other five were used to determine the concrete splitting tensile strength. The five cubes were used to determine the concrete cube compressive strength. The cylinder and cube compressive strength tests of the concrete were conducted in accordance with the ASTM C39 [27] and BS 12390-3 [28], respectively. The splitting tensile strength test was carried out according to the ASTM C496 [29]. The average cylinder and cube compressive strengths of the concrete were 26.3 and 33.5 MPa, respectively, whereas the splitting tensile strength was, on average, 2.4 MPa. Deformed steel reinforcing bars with a diameter of 25 mm were used for the longitudinal reinforcement, whereas 5 mm diameter smooth bars were used as internal steel stirrups. Three samples were tested for each bar diameter under uniaxial tension. Tests of steel samples were conducted as per the BS 4449:2005 [30]. The average measured yield strengths for the 25 and 5 mm diameter bars were 539 and 505 MPa, respectively, whereas their respective ultimate tensile strengths were 649 and 543 MPa.

Table 2. Proportions of the concrete mix per cubic meter.

| Material | OPC | Coarse Aggregates | Fine Aggregates | Water | Admixture POZZOLITH® LD10 |
|----------|-----|-------------------|-----------------|-------|---------------------------|
|          | 300 kg | 700 kg | 350 kg | 600 kg | 350 kg | 165 L | 1.5 L |

The carbon fabric used in the current study, S&P ARMO-mesh L600, obtained from S&P Clever Reinforcement, Switzerland, consists of unidirectional carbon fiber strands and nonmetallic transverse holders (Figure 2). Properties of the fabrics provided by the manufacturer are given in Table 3 [31]. The measured width and thickness of one carbon fiber strand were approximately 5.0 and 0.54 mm, respectively, which corresponded to a cross-sectional area per unit length of 159 mm²/m. This result was consistent with that provided by the manufacturer (157 mm²/m) [31]. The website of the S&P Clever Reinforcement [31] indicates that an ISO 9001:2015 QS management system is consistently maintained at all production sites of the company, verifying that the development, production, and marketing of the reinforcing materials are in compliance with requirements of the ISO standard [32]. The cementitious mortar used in the C-FRCM strengthening was a polymer-modified mortar. The mortar was mixed as per the procedure provided by the manufacturer [31]. Mechanical properties tests of the mortar were conducted in accordance with the ASTM standards [33,34]. Based on the results of five replicate samples, the cementitious matrix had an average 28-day cube compressive strength of 42 MPa, cylinder compressive strength of 35 MPa, splitting tensile strength of 2.4 MPa, and Young’s modulus of 29 GPa, respectively. The geopolymeric matrix used in the C-FRGM strengthening included ground granulated blast furnace slag (GGBS), obtained from Emirate Cement, Al Ain, UAE, and class F fly ash, obtained from Ashitech International, Dubai, UAE, as binding materials, dune sand as fine aggregates, and an alkaline activator solution consisting of sodium silicate (SS) and sodium hydroxide (SH), obtained from Alrama Chemicals, Sharjah, UAE. Proportions and components of the geopolymeric matrix are given in Table 4. Based on the results of five replicate samples, the geopolymeric matrix had an average measured 28-day cube compressive strength of 43 MPa, cylinder compressive strength of 34 MPa, splitting tensile strength of 3.0 MPa, and Young’s modulus of 7 GPa.
Figure 2. Carbon fabrics: (a) Fabric net; (b) Close view showing the fiber strands.

Table 3. Fabric properties data from [31].

| Property                              | Value     |
|---------------------------------------|-----------|
| Weight per unit area (g/m²)           | 281.000   |
| Tensile strength (MPa)                | 4300.000  |
| Modulus of elasticity (GPa)           | 240.000   |
| Elongation at break (%)               | 1.800     |
| Cross-sectional area (mm²/mm)         | 0.157     |
| Spacing between fabric bundles (mm)   | 17.000    |

Table 4. Geopolymeric matrix components and proportions.

| Component                  | Fly Ash | Slag | Dune Sand | Sodium Silicate (SS) | Sodium Hydroxide (SH) |
|----------------------------|---------|------|-----------|----------------------|-----------------------|
|                            | 362.5   | 362.5| 752.0     | 285.5                | 114.0                 |

2.4. C-FRM Strengthening Methodology

The carbon fabric strands of the fabric were too stiff to be wrapped around the cross section of the beam, therefore, the side-bonded FRM configuration was selected. The strengthening process included preparation of the concrete surface and application of the C-FRM composite layers. Typical steps of the C-FRM strengthening are summarized in Figure 3. The concrete surface was roughened using a high-pressurized water jet. The concrete surface was then left to dry prior to the application of the C-FRM composites. One layer of mortar with a thickness of approximately 4 mm was first applied to the roughened concrete surface. A carbon fabric sheet, precut to the desired length, was then placed on top of the mortar layer then fully impregnated in the mortar using hand pressure. A second layer of mortar, with a thickness of approximately 4 mm, was subsequently applied on top of the fabric. The C-FRM application process was repeated to apply an additional layer of C-FRM for the beams strengthened with two layers of C-FRM composites. The C-FRCM strengthening composite layer was cured for 28 days using periodically wetted burlap sheets. It should be noted that the beam zone above the support plates represents the extension of a supporting column encountered in practical setting. The critical section for shear in RC beams supported by columns is located away from the face of the support. As such, the strengthening layer(s) in each shear span adopted in the current study started from the face of the support plate and ended at the face of the load plate. Both numerical and experimental results verified that no premature failure occurred at the supporting zones. The shear failure occurred within the shear span in all beams. The behavior was predicted numerically and verified experimentally.
2.5. Test Set-Up and Instrumentation

The deep beam specimens rested on two steel pedestals that were 2900 mm apart, on center. The load was applied at the top surface of the beam on two points, 1300 mm apart, using two MTS 201.45 hydraulic actuators, manufactured by MTS Systems Corporation, Eden Prairie, MN, USA. The corresponding shear span had a length of 800 mm. The experiments were initially performed under a load control at a rate of 0.5 kN/sec. At about 85 to 90% of the theoretical load capacity, the loading scheme was changed to be displacement-controlled at a rate of 0.6 mm/min. Such a change in the loading scheme was done to prevent possible catastrophic failure of the beam at ultimate load. Two steel plates (150 × 150 × 20 mm) were placed at the load and support points to prevent the concentration of stresses. Strain gauges (SGs), Type 5LA-5-11, manufactured by Tokyo Measuring Instruments Laboratory, Tokyo, Japan, were bonded to the surface of the tension steel reinforcement at discrete locations, 180 mm apart, in the shear span to report the steel strain profile. For the beams with internal steel stirrups, two additional SGs were installed on two stirrups in the middle of the shear span; one SG was bonded to a horizontal stirrup, while the other one was bonded to a vertical stirrup. An additional SG was bonded to the vertical fabric roving in the middle of each shear span. Two load cells, Type CLF-NA 500 kN, manufactured by Tokyo Measuring Instruments Laboratory, Japan, were placed between the actuators and the load plates to record the applied load. The midspan deflection was measured using a linear variable differential transducer (LVDT), Type CDP-100, manufactured by Tokyo Measuring Instruments Laboratory, Japan. A data logger system, Type TDS-630, manufactured by Tokyo Measuring Instruments Laboratory, Japan, was used to record the readings of the measuring tools. A schematic showing the test setup is illustrated in Figure 4, whereas a test in progress is shown in Figure 5.

![Figure 4](image_url)
3. Experimental Results

3.1. Crack Pattern and Failure Mode

The crack patterns at failure of the unstrengthened specimens are shown in Figure 6. None of the beams experienced a flexural mode of failure. The strains in the tension steel were below the yielding strain as intended in the design stage (refer to steel strain data in Section 3.5). Specimen Control-NS that did not include steel stirrups exhibited a diagonal crack that initiated at the middle of each shear span and then propagated rapidly toward the support and load points. A splitting longitudinal crack at the level of the tension steel reinforcing bars developed in the region close to the support prior to failure. The beam failed in a shear-compression mode of failure when the diagonal crack penetrated into the compression zone of the west shear span, which caused concrete crushing at the tip of the crack, in addition to the formation of another splitting crack at the level of the compression steel reinforcement (Figure 6a). Other deep beams specimens failed by crushing of the diagonal strut in the shear span (i.e., a diagonal compression mode of failure). Crushing of the strut could occur at any location along the length of the strut in the diagonal direction. Specimen Control-ST, with stirrups exhibited a first shear crack in the middle of the shear span. Additional shear cracks developed in the diagonal direction as the load progressed. Further increase in load resulted in propagation of these cracks toward the support and load points along with formation of additional parallel cracks in the diagonal direction forming a diagonal strut. The beam eventually failed by crushing of the concrete along the diagonal strut (Figure 6b). Figure 7 shows the crack patterns of specimens in group A at failure. Specimen NS-C1-90 experienced a diagonal shear crack that developed in the middle of the shear span and then propagated toward the support and load points. The beam failed due to crushing of the concrete at the top part of the diagonal strut (i.e., diagonal compression mode of failure) (Figure 7a). Specimens NS-C2-90 and NS-C2-0/90 exhibited multiple cracks in the shear span during the test. They eventually failed due to crushing of the diagonal strut developed in the shear spans (Figure 7b,c). The crack patterns at failure of specimens in group B are presented in Figure 8. Specimen ST-C1-90 exhibited multiple shear cracks in the shear span (Figure 8a). Specimen ST-C2-90 with two layers of C-FRCM in the vertical direction experienced an increased number of shear cracks in the shear span (i.e., band of shear cracks) (Figure 8b). All specimens of this group failed by crushing of the diagonal strut in the shear span. Crushing of concrete was evident in the middle of the diagonal strut of specimens ST-C1-90 and ST-C2-90, whereas ST-C2-0/90 experienced concrete crushing at the top part of the diagonal strut in the shear span (Figure 8c). The crack patterns of specimens in group C at failure are shown in Figure 9. Specimens of this group, NS-G1-90 and ST-G1-90, with no and minimum internal steel stirrups, respectively, were strengthened C-FRGM. The cracks were not visible during testing, possibly because of the low Young’s modulus of the geopolymeric matrix that could have facilitated large deformation in the matrix and hindered the visibility of shear cracks on the surface of the matrix. Specimen NS-G1-90 failed suddenly due to crushing of the diagonal strut in the east shear span (Figure 9a). Specimen ST-G1-90 also experienced crushing of the diagonal strut in the middle of the west shear span (Figure 9b).
3.2. Shear Load–Deflection Response

Figure 10 shows the load–deflection responses of tested beams. A summary of the test results is given in Table 5. From Figure 10a, it can be seen that the shear load–deflection response of the unstrengthened beam Control-NS started with a linear relationship until the shear load reached approximately 75% of the shear capacity, where a small drop in load occurred due to the initiation of an inclined shear crack in the shear span. In the post-cracking stage, the deflection continued to increase but at a higher rate. Another drop in load occurred at 81% of the shear capacity due to the initiation of a splitting crack.
near the support. Then, the beam exhibited an increased deflection with an insignificant increase in load until failure took place. Specimen Control-ST exhibited a quasilinear shear load–deflection response until it failed at a midspan deflection that was approximately 1.7 times that of its counterpart Control-NS. Due to the presence of internal stirrups, the development and propagation of shear cracks in specimen Control-ST did not result in a significant change in the slope of the shear load–deflection response.

Figure 9. Specimens in group C at failure: (a) ST-C1-90; (b) ST-C2-90.

Figure 10. Experimental shear load–deflection response: (a) Control beams; (b) Group A; (c) Group B; (d) Group C.

Figure 10b shows the deflection response of specimens in group A that did not include internal shear reinforcement and strengthening with C-FRCM. The response of the control specimen, Control-NS, was included in the same figure. The pre-cracking stiffness of the strengthened specimens almost coincided with that of the control specimen. Strengthened specimens exhibited the first shear cracking at a shear load value that was 48 to 56% of the shear capacity. Initiation of shear cracks in the strengthened beams did not change the slope of the deflection response, thus, the deflection continued to increase almost linearly until the beams reached their shear capacity at a shear load that was approximately two times
that of Control-NS. Increasing the amount of C-FRCM had no effect on the stiffness, but marginally increased the shear and deflection capacities of the beams, irrespective of the angle of orientation of the second layer of C-FRCM.

Table 5. Test results.

| Group | Specimen  | Cracking Load | Shear Capacity | Deflection Capacity | $V_{cr}/V_{max}$ | Strength Gain (%) | Failure Mode       |
|-------|------------|---------------|----------------|---------------------|------------------|------------------|------------------|
|       |            | $V_{cr}$ (kN) | $V_{max}$ (kN) | $\Delta_{peak}$ (mm) |            |                  |                  |
| Control | Control-NS | 104           | 139            | 6.8                 | 0.75             | -                | Shear compression|
|        | Control-ST | 140           | 348            | 11.7                | 0.40             | -                | Strut crushing   |
| A      | NS-C1-90   | 130           | 271            | 8.5                 | 0.48             | 95               | Strut crushing   |
|        | NS-C2-90   | 150           | 290            | 8.9                 | 0.52             | 109              | Strut crushing   |
|        | NS-C2-0/90 | 160           | 288            | 9.7                 | 0.56             | 107              | Strut crushing   |
| B      | ST-C1-90   | 170           | 409            | 13.3                | 0.42             | 18               | Strut crushing   |
|        | ST-C2-90   | 150           | 411            | 11.7                | 0.36             | 18               | Strut crushing   |
|        | ST-C2-0/90 | 150           | 377            | 12.2                | 0.40             | 8                | Strut crushing   |
| C      | NS-G1-90   | 130           | 246            | 7.7                 | 0.53             | 77               | Strut crushing   |
|        | ST-G1-90   | 130           | 380            | 11.4                | 0.34             | 9                | Strut crushing   |

1 The strength gain of the strengthened beams with and without stirrups was calculated relative to the shear capacity of the control unstrengthened beams Control-ST and Control-NS, respectively.

Figure 10c shows the deflection responses of specimens in group B that had internal stirrups and had been strengthened with C-FRCM. The response of specimen Control-ST was included in the same figure. The stiffness of the specimens was insignificantly different. Shear cracks initiated in the strengthened specimens at an average shear load value of 40% of the shear capacity. Strengthened specimens failed at a midspan deflection that was insignificantly higher than that of the control specimen Control-ST.

Figure 10d shows the deflection response of specimens in group C that were strengthened with C-FRGM. The responses of specimens Control-NS and Control-ST are included in the figure for the intention of comparison. Although cracks of specimens in this group were not visible during testing, values of the shear cracking load could be determined from the stirrup and fabric strain data presented in Figures 11 and 12, respectively. The shear cracks developed in specimens NS-G1-90 and ST-G1-90 at load values of 34 and 42% of the shear capacity, respectively. Strengthened specimens exhibited a quasilinear response with an insignificant change in the slope of the shear load–deflection response. The response of NS-G1-90 outperformed that of Control-NS, whereas the response of ST-G1-90 was insignificantly different from that of Control-ST.

Figure 11. Typical stirrup strain response: (a) Control-ST; (b) ST-C1-90; (c) ST-G1-90.
Figure 12. Typical fabric strain response: (a) Specimens with different matrices; (b) Specimens with different amounts of fabrics.

3.3. Stirrup Strains

Typical stirrup strain responses of tested beams are shown in Figure 11. The strain in the horizontal stirrups of beam ST-G1-90 was not captured due to the malfunction of the SG. Results of two duplicate SGs bonded to the same vertical stirrup in specimen ST-G1-90 are reported. The stirrups were not strained until shear cracks initiated in the shear span. The strain increased almost linearly in the post-cracking phase until yielding of stirrups took place. The horizontal stirrups tended to exhibit slightly higher strains in the post-cracking phase than those of the vertical stirrups. The horizontal stirrups of the control beam Control-ST yielded at peak load. The strains in some of the stirrups of the strengthened beams continued to increase after yielding with an insignificant increase in load until failure occurred. Specimen ST-G1-90 with a geopolymeric matrix exhibited higher stirrup strains in the post-cracking stage than those of specimen ST-C1-90 with a cementitious mortar. The higher stirrup strains exhibited by ST-G1-90 can be ascribed to the reduced contribution of the C-FRGM to the shear resistance caused by the lower Young’s modulus of the geopolymeric matrix relative to that of the cementitious mortar.

3.4. Carbon Fabric Strains

Typical fabric strain responses of tested beams are presented in Figure 12. It is evident that the fabrics started to contribute to the shear resistance after the initiation of shear cracks. The fabric strains in specimens NS-C1-90 and NS-G1-90 without stirrups increased at a higher rate than those of their counterparts ST-C1-90 and ST-G1-90 with internal stirrups (Figure 12a). Similarly, specimen NS-C2-0/90 exhibited a higher rate of fabric strain increase than that of its counterpart ST-C2-0/90 (Figure 12b). This behavior signifies the load sharing between the carbon fabrics and the internal steel stirrups. Specimen NS-C1-90 with one layer of fabric exhibited higher strains than those of its counterpart NS-C2-0/90 with two layers of fabric (Figure 12b). However, the impact of increasing the number of C-FRCM layers on the strain response (ST-C1-90 and ST-C2-0/90) was not evident in the presence of internal steel stirrups.

3.5. Steel Strains

Figure 13 shows the measured steel strains in the shear span at four different loading stages of 25, 50, 75, and 100% of the shear capacity. Strain data of specimen NS-C2-90 are missing because the SGs malfunctioned before testing. The steel stain profile within the shear span was almost uniform during all stages of loading. The uniform strain profile verifies development of the arch action in the tested beams. None of the specimens experienced yielding of the tension steel as intended in the design stage.
4. Discussion

Studies on the shear behavior of RC deep beams strengthened with FRM composites are scarce in the literature [16–21]. Although these studies produced interesting findings and conclusions, the tested beams did not include internal steel stirrups [17,20] or the steel stirrups used were in the vertical direction only at a large spacing of 0.8 \( d \) [16,18,19,21], which was not in compliance with the minimum requirements of the ACI 318-19 code provisions [26] for RC deep beams. The application of FRM composites in practical setting would involve strengthening of RC deep beams having internal steel stirrups in compliance with international codes and standards. As such, half of the beams tested in the current study included internal stirrups satisfying the minimum requirements of the ACI 318 code provisions [26], whereas the other half did not include steel stirrups for the purpose of comparison.

The unstrengthened beam that did not include internal shear reinforcement, Control-NS, exhibited shear cracking at approximately 75% of the shear capacity (i.e., \( V_{cr}/V_{max} = 0.75 \)). The beam failed shortly after the initiation of shear cracks in a shear–compression mode of failure because of the absence of internal shear reinforcement. The inclusion of internal shear reinforcement changed the mode of failure to crushing of the concrete strut (i.e., diagonal compression), increased the shear cracking load, and improved the shear capacity. The presence of internal shear reinforcement also increased the difference between the cracking and ultimate load, and thus, reduced the ratio of \( V_{cr}/V_{max} \) to an average value of

![Figure 13. Steel strain profile: (a) Control-NS; (b) Control-ST; (c) NS-C2-90; (d) NS-C2-0/90; (e) ST-C1-90; (f) ST-C2-90; (g) ST-C2-0/90; (h) NS-G1-90; (i) ST-G1-90.](image-url)
The shear capacity of the specimen having internal stirrups, Control-ST, was 1.6 times that of its counterpart specimen without stirrups (Control-NS).

The results from group A indicate that external C-FRCM shear reinforcement in RC deep beams can play a role similar to that of the internal stirrups. Specimens in group A exhibited a reduced ratio of $V_{cr}/V_{max}$, higher shear capacity, and higher deflection capacity than those of their counterpart control specimens (Control-NS). Also, shear strengthening with C-FRCM changed the mode of failure to a diagonal compression mode of failure (i.e., crushing of the diagonal concrete strut). Specimen NS-C1-90, with one layer of C-FRCM, exhibited a shear strength gain of 95%. These results are consistent with those reported in the literature for RC deep beams without internal steel stirrups having $a/h$ of 1.7, where a 77 to 100% increase in the shear capacity was recorded due to strengthening with two layers of C-FRCM composite [17,20]. Increasing the number of C-FRCM layers insignificantly increased the shear capacity. The shear capacity of specimens NS-C2-90 and NS-C2-0/90, with two layers of C-FRCM was, on average, 8% higher than that of specimen NS-C1-90. The angle of orientation of the second layer of C-FRCM had almost no effect on the shear capacity of the specimens without internal stirrups. These results verify other findings on the effect of the number of FRM layers on the shear strength gain, where a non-proportional increase in the shear capacity is anticipated [4,6,7,11–13]. The results are also in line with those reported for strengthened RC deep beams, where doubling the number of C-FRCM layers in the absence of internal steel stirrups increased the shear capacity by only 21% [17].

The results for group B indicate that the effectiveness of C-FRCM to improve the shear capacity of RC deep beams was affected by the presence of internal shear reinforcement. The gain in shear capacity was less pronounced in the presence of internal steel stirrups. The reduced efficiency of FRM shear strengthening in the presence of internal steel stirrups is in line with other findings reported in the literature for strengthened RC deep beams [16,18,21]. Only 18% shear strength gain was recorded due to shear strengthening with one layer of C-FRCM aligned in the vertical direction in the presence of internal stirrups. Increasing the amount of C-FRCM did not result in an additional shear strength gain in the presence of internal shear reinforcement. This behavior implies that beams of this group were over-reinforced for shear, and the diagonal strut could have reached its maximum capacity. Positioning the second layer of carbon fabric in the horizontal direction (i.e., at an angle of inclination of $0^\circ$) tended to be less effective than placing it in the vertical direction (i.e., at an angle of inclination of $90^\circ$).

The results for group C demonstrate the viability of using a geopolymeric matrix as a sustainable alternative to commercial cementitious mortar. Specimen NS-G1-90 strengthened with one layer of C-FRGM (i.e., with a geopolymeric matrix) experienced a 77% shear strength gain relative to that of its counterpart control specimen (Control-NS). The shear capacity of specimen NS-G1-90 was only 9% lower than that of its counterpart strengthened with C-FRCM (i.e., with a cementitious matrix). The effectiveness of the shear strengthening system involving the geopolymeric matrix was reduced in the presence of internal stirrups, similar to the behavior of their counterpart specimens strengthened with C-FRCM (i.e., with a cementitious matrix). Specimen ST-G1-90 failed by crushing of the diagonal strut at a shear capacity 9% higher than that of the control specimen (Control-ST). The shear capacity of specimen ST-G1-90 was only 7% lower than that of its counterpart ST-C1-90 strengthened with C-FRCM (i.e., with a cementitious matrix). Although the feasibility and effectiveness of using geopolymer-based FRM composites for shear strengthening of RC deep beams was not investigated in previous studies, the significant improvement in the shear capacity due to strengthening in the absence of internal stirrups and the reduced shear strength gain recorded for the beams with internal steel stirrups are consistent with other findings published in the literature for cement-based FRM composites [16–21].

The minor reduction in the shear strength gain caused by using a geopolymeric matrix rather than a cementitious matrix may be considered satisfactory given the environmental benefits associated with the widespread use of geopolymers as an alternative to cement-based matrices in FRM composites. Such benefits include the recycling of industrial
by-products as geopolymeric binders, reduced water consumption necessary for curing of cementitious mortars, and reduced production of cement with associated energy savings, preservation of natural resources, and mitigation of carbon dioxide emissions.

5. Finite Element Modeling

Available codes and standards, including the ACI 549.4R-20 [1], do not provide a closed-form solution for performance prediction of RC deep beams strengthened with FRM composites. Conventional analytical approaches that calculate the FRM contribution to the shear capacity in a way analogous to that used for internal steel stirrups, such as that offered by the ACI 549.4R-20 [1], are not valid for strengthened RC beams with a shear compression/diagonal compression mode of failure [3]. In contrast, FE modeling offers numerical solutions for such cases. As such, three-dimensional (3D) FE models were developed to simulate the response of the tested beams. The software ATENA 3D [35] was used in the numerical analysis.

5.1. Constitutive Laws of Material

The uniaxial constitutive laws of the concrete and mortar in compression and tension start by a linear part having a slope equal to the materials’ modulus of elasticity. The compressive hardening and softening laws are shown in Figure 14a,b, respectively, whereas the tensile softening law is shown in Figure 14c. The strain hardening phase in compression starts at a stress value of $f'_{	ext{co}} = 2f_t$, where $f_t$ = uniaxial tensile strength, and ends at a peak stress of $f'_c$ and a corresponding plastic strain of $\varepsilon_{cp}$ having a default value of 0.001. The compressive stress ($\sigma_c$) in the post-peak stage decreases linearly with an increase in the displacement ($w_c$) throughout the length scale ($L_c$). The value of ($w_c$) at any plastic strain ($\varepsilon_p$) equals ($\varepsilon_p - \varepsilon_{cp}$)($L_c$). The displacement at zero stress ($w_d$) has a default value of 0.5 mm [16]. The tensile softening law starts when the tensile stress ($\sigma_t$) reaches a peak value of $f_t$ where the tension decreases exponentially, with an increase in the crack opening displacement ($w_t$) calculated from the fracturing strain ($\varepsilon_{cf}$) times the crack band length ($L_t$). The value of the crack opening at a complete release stress ($w_{tc}$) is determined automatically by the software based on the fracture energy of the material ($G_f$). Key input parameters of the concrete and mortars used are provided in Tables 6–8. The tension stiffening effect in heavily reinforced concrete beams is accounted for using a limiting minimum value of a reduced tensile strength on the tensile softening branch. In such cases, the tensile strength cannot drop below a value equal to the product $C_{ts}f_t$, where $C_{ts}$ = tension stiffness coefficient taken as 0.4 in the current study for the beams with internal and/or external shear reinforcement. The stress–strain response of the steel reinforcing bars was assumed as linear–elastic with a post-yield strain hardening (Figure 15a), where $E_s$ = steel Young’s modulus, $f_y$ = yield strength, $f_u$ = ultimate strength, and $E_{sh}$ = post-yield modulus assumed as 0.01 $E_s$. The stress–strain relationship of the carbon fabrics was modeled as linear–elastic (Figure 15b), where $f_{fu}$ = ultimate strength of the fabric, $\varepsilon_{fu}$ = ultimate strain of the fabric, and $E_f$ = Young’s modulus of the fabric. The steel reinforcing bars were assumed to be perfectly bonded to the concrete. A linear–elastic response was assumed for the steel plates at the support and loading points. Materials properties reported earlier were used as input data in the FE analysis.
Figure 14. Constitutive laws of concrete and mortar: (a) compressive hardening; (b) compressive softening; (c) tensile softening.

Table 6. Key input parameters of the concrete.

| Parameter | Description                      | Value              |
|-----------|----------------------------------|--------------------|
| $f_{cu}$  | Cube compressive strength        | $-33.50$ MPa       |
| $f'_c$    | Cylinder compressive strength    | $-26.30$ MPa       |
| $f_t$     | Tensile strength                 | $2.40$ MPa         |
| $E_c$     | Elastic modulus                  | $2.41 \times 10^4$ MPa |
| $\mu$     | Poisson’s ratio                  | $0.20$             |
| $G_f$     | Specific fracture energy         | $6.23 \times 10^{-5}$ MN/m |
| $w_d$     | Critical compressive displacement| $-5.00 \times 10^{-4}$ m |
| $\varepsilon_{cp}$ | Plastic strain at compressive strength | $-8.97 \times 10^{-4}$ |

Table 7. Key input parameters of the cementitious matrix.

| Parameter | Description                      | Value              |
|-----------|----------------------------------|--------------------|
| $f_{cu}$  | Cube compressive strength        | $-42.0$ MPa        |
| $f'_c$    | Cylinder compressive strength    | $-35.3$ MPa        |
| $f_t$     | Tensile strength                 | $2.40$ MPa         |
| $E_c$     | Elastic modulus                  | $2.89 \times 10^4$ MPa |
| $\mu$     | Poisson’s ratio                  | $0.20$             |
| $G_f$     | Specific fracture energy         | $7.25 \times 10^{-5}$ MN/m |
| $w_d$     | Critical compressive displacement| $-5.00 \times 10^{-4}$ m |
| $\varepsilon_{cp}$ | Plastic strain at compressive strength | $-1.03 \times 10^{-3}$ |

Table 8. Key input parameters of the geopolymeric matrix.

| Parameter | Description                      | Value              |
|-----------|----------------------------------|--------------------|
| $f_{cu}$  | Cube compressive strength        | $-43$ MPa          |
| $f'_c$    | Cylinder compressive strength    | $-34$ MPa          |
| $f_t$     | Tensile strength                 | $3.00$ MPa         |
| $E_c$     | Elastic modulus                  | $0.70 \times 10^4$ MPa |
| $\mu$     | Poisson’s ratio                  | $0.20$             |
| $G_f$     | Specific fracture energy         | $7.36 \times 10^{-5}$ MN/m |
| $w_d$     | Critical compressive displacement| $-5.00 \times 10^{-4}$ m |
| $\varepsilon_{cp}$ | Plastic strain at compressive strength | $-1.05 \times 10^{-3}$ |
5.2. Bond–Slip Law at Fabric–Matrix Interface

The bond–slip laws adopted at the carbon fabric–matrix interface for both types of matrices are shown in Figure 16. These bond constitutive laws were developed in an earlier study by Abu Obaida et al. [25]. The key parameters of the bond slip laws including the maximum shear stress, $\tau_m$, the slip at maximum shear stress, $s_m$, the maximum slip at zero stress, $s_{\text{max}}$, and the fracture energy, $G_f$, are summarized in Table 9 [25]. The FE models of the tested beams were also analyzed assuming a perfect bond condition at the matrix–concrete interface for the purpose of comparison. A perfect bond condition was assumed between the matrix and concrete substrate. This assumption was verified experimentally since none of the tested beams failed by interfacial debonding at the matrix–concrete interface.

Figure 16. Bond–slip models at fabric–matrix interface data from [25].

Table 9. Key bond–slip model data from [25].

| Bond Model          | $\tau_m$ (MPa) | $s_m$ (mm) | $s_{\text{max}}$ (mm) | $G_f$ (N/mm) |
|---------------------|----------------|------------|------------------------|--------------|
| Cementitious matrix | 1.22           | 0.11       | 1.00                   | 0.40         |
| Geopolymeric matrix | 1.21           | 0.22       | 2.00                   | 0.77         |

5.3. Types of Element

Solid 3D macro-elements were used to model the concrete, mortars, and steel plates. The steel bars and carbon fiber strands were modeled as one-dimensional elements embedded into the macro-elements of the concrete and mortar, respectively. One-quarter of the beam was modeled to optimize the processing time. The software manual recommends having a minimum of 4 to 6 elements in the shortest dimension of the member to warrant solution convergence, while minimizing the computational time [35]. The shortest dimension of the beam of the current study is 150 mm. As such, adopting 4 to 6 elements in
the shortest dimension corresponds to a mesh size in the range of 25 to 37.5 mm. A mesh sensitivity analysis was conducted to investigate the possibility of reducing the mesh size further so that more representative numerical results may be obtained. As such, all models were, initially, processed with mesh sizes of 25 and 20 mm, noting that software did not operate when a smaller mesh size of 15 mm was adopted. Results of the mesh sensitivity analysis are shown in Table 10. The ratio \( \frac{V_{20}}{V_{25}} \) was in the range of 0.94 to 1.04 with an average of 0.99 and standard deviation of 0.04. The insignificant difference between the predicted shear capacities of the models with mesh sizes of 20 and 25 mm indicates the stabilization of numerical results. As such, the smallest possible mesh size of 20 mm was adopted in the analysis.

Table 10. Results of the mesh sensitivity analysis.

| Beam Model        | Predicted Shear Capacity 1 (kN) | Ratio \( \frac{V_{20}}{V_{25}} \) |
|-------------------|---------------------------------|----------------------------------|
|                   | \( V_{25} \) | \( V_{20} \) |                                                |
| Control-NS        | 119       | 124       | 1.04                                           |
| Control-ST        | 288       | 270       | 0.94                                           |
| NS-C1-90          | 270       | 283       | 1.05                                           |
| NS-C2-90          | 304       | 297       | 0.98                                           |
| NS-C2-0/90        | 281       | 295       | 1.05                                           |
| ST-C1-90          | 339       | 329       | 0.97                                           |
| ST-C2-90          | 357       | 359       | 1.01                                           |
| ST-C2-0/90        | 359       | 341       | 0.95                                           |
| NS-G1-90          | 273       | 269       | 0.99                                           |
| ST-G1-90          | 323       | 313       | 0.97                                           |
| **Average**       |           |           | 0.99                                           |
| **Standard Deviation** |           |           | 0.04                                           |

1 \( V_{25} \) and \( V_{20} \) are the shear capacities of the model with mesh sizes of 25 and 20 mm, respectively.

5.4. **Loading and Boundary Conditions**

The boundary conditions established for the quarter model are shown in Figure 17. The support plate was restricted from movement in the Y (transverse) and Z (vertical) directions. Surface supports were used to prevent surfaces at the planes of symmetry from movement in a direction normal to the other symmetrical part of the beam. The loading scheme was displacement-controlled at a prescribed displacement of 0.1 mm/step applied at the midpoint of the top surface of the load plate. The standard Newton–Raphson iterative solution approach was adopted in the FE analysis.

![Figure 17. Boundary conditions.](image-url)
6. Results of Finite Element Modeling
6.1. Predicted Deflection Response

Figure 18 shows the predicted shear load–deflection responses of the deep beam models. Predicted responses of the FRM-strengthened models shown in the figure correspond to the models with the bond–slip law at the fabric–matrix interface. The predicted response of the unstrengthened beam models indicates that the presence of internal stirrups had no effect on the rate of increase of the beam deflection but significantly increased the shear capacity. This outcome is in agreement with that obtained from the tests. The predicted shear load–deflection responses of beam models of groups A and B indicate that the number of C-FRCM layers and angle of orientation of the second FRCM layer had no significant effect on the rate of increase of the midspan deflection. This finding is in alignment with those obtained from the experiments. The predicted response of the beam models in group C confirms the validity of using geopolymers as a matrix in the strengthening system, which was also verified experimentally. The numerical and experimental shear load–deflection responses are compared in Figure 19. The rate of increase of the midspan deflection predicted numerically is in agreement with that obtained from the experiment. The FE deep beam models tended, however, to fail at a load value lower than that obtained from the experiment, which reduced the predicted deflection at failure relative to that determined experimentally. This behavior is more pronounced in the results of the unstrengthened models and those of the FRM-strengthened models having internal steel stirrups. A comparison between the predicted and measured shear capacities is presented in the following section.

Figure 18. Predicted shear load–deflection response: (a) Control beams; (b) Group A; (c) Group B; (d) Group C.
6.2. Comparative Analysis between Predicted and Measured Shear Capacities

A comparison between the shear capacities predicted numerically, with and without a bond–slip law, and those measured experimentally are given in Table 11. The numerical results of beam models in group A indicate that the use of one layer of C-FRCM increased the shear capacity of the models without internal stirrups approximately 2.2-fold. This finding is in agreement with that obtained from the experiments. Additionally, the numerical results show that doubling the number of C-FRCM layers resulted in a 4 to 8% increase in the shear capacity. This finding is in-line with the corresponding experimental results which show an increase of approximately 7% due to doubling the number of C-FRCM layers. Similarly, the shear capacity of the counterpart beam models NS-C2-90
and NS-C2-0/90 predicted numerically was insignificantly different. The experimental test results verify the insignificant effect of the angle of orientation of the second layer of C-FRCM on the shear capacity. The numerical results of specimens in group B indicate that shear strengthening of RC deep beams having internal stirrups with one layer of C-FRCM resulted in approximately 20 to 22% gain in the shear capacity. The shear strength gain obtained from the tests was 18%. The increase in the shear capacity predicted by the beam model with the bond–slip law at the fabric–matrix interface (20%) was closer to that obtained from the test (18%). The negligible effect of increasing the number of C-FRCM layers on the shear capacity of specimens of this group was predicted numerically and verified experimentally. The predicted shear capacity of model ST-C2-0/90 was 3 to 5% lower than that of its counterpart ST-C2-90. The experimental results for this group verify the insignificant reduction in the shear capacity caused by changing the angle of orientation of the second layer of C-FRCM from 0 to 90°. The numerical results of the beam models in group C (NS-G1-90 and ST-G1-90) and the counterpart specimens from other groups (NS-C1-90 and ST-C1-90) indicate that the use of geopolymers as a matrix instead of a cementitious commercial mortar reduced the shear capacity by 5 to 6%. This finding was verified experimentally since corresponding experimental test results showed a 7 to 9% reduction in the shear capacity due to the use of the geopolymeric matrix in the strengthening system rather than the cementitious mortar. The agreement between the outcomes of the numerical analysis and those obtained from the experiments verifies the validity of the FE models.

Table 11. Comparison between predicted and measured shear capacities.

| Group | Specimen   | Experimental | Numerical | V_{FE}/V_{Exp} |
|-------|------------|--------------|-----------|---------------|
|       |            | V_{Exp} (kN) | V_{FE} (kN) |               |
|       |            |              | Perfect Bond | With Bond–Slip Law |
|       |            |              | Perfect Bond | With Bond–Slip Law |
| Control | Control-NS | 139          | 124        | N.A.          | 0.89 | N.A. |
|        | Control-ST | 348          | 270        | N.A.          | 0.78 | N.A. |
| A      | NS-C1-90   | 271          | 283        | 261           | 1.04 | 0.96 |
|        | NS-C2-90   | 290          | 297        | 283           | 1.02 | 0.98 |
|        | NS-C2-0/90 | 288          | 295        | 273           | 1.02 | 0.95 |
| B      | ST-C1-90   | 409          | 329        | 325           | 0.80 | 0.79 |
|        | ST-C2-90   | 411          | 359        | 345           | 0.87 | 0.84 |
|        | ST-C2-0/90 | 377          | 341        | 336           | 0.90 | 0.89 |
| C      | NS-G1-90   | 246          | 269        | 250           | 1.09 | 1.02 |
|        | ST-G1-90   | 380          | 313        | 305           | 0.82 | 0.80 |

V_{Exp} = Experimental shear capacity. V_{FE} = Shear capacity predicted numerically.

The numerical results from the strengthened specimens indicate that the inclusion of the bond–slip law at the fabric–matrix interface slightly decreased the predicted shear capacity. The ratios of the predicted-to-measured shear capacity of the models with and without the bond–slip law were, on average, 0.90 and 0.95 with corresponding standard deviations of 0.09 and 0.11, respectively. In the absence of internal stirrups, the shear capacity of the beam models with the bond–slip law was, on average, 7% lower than that of the beam models with a perfect bond connection at the fabric–matrix interface. The effect of the inclusion of the bond–slip law in the analysis was less pronounced in the presence of internal stirrups where only a 2% average shear strength reduction was recorded relative to the capacity of the beam models with perfect bond at the fabric–matrix interface.
6.3. Comparative Analysis between Predicted and Observed Crack Patterns

The numerical crack patterns at failure are compared to those of the experimental tests in Figure 20. In agreement with the experimental observation, the beam model that did not include internal stirrups, Control-NS, exhibited a single shear crack in the shear span connected to a longitudinal splitting crack at the level of the tension steel reinforcing bars in the region close to the support. The numerical models with internal stirrups and/or external C-FRCM layers exhibited multiplate shear cracks in the shear span and concrete crushing at multiple locations along the diagonal strut. The change in the mode of failure predicted numerically due to the presence of internal and/or external shear reinforcement is in agreement with the experimental findings.

Figure 20. Cont.
7. Conclusions

The effectiveness of C-FRCM and C-FRGM shear strengthening composite systems in improving the shear behavior of RC deep beams with \(a/h\) of 1.6 was investigated. The study comprised laboratory testing and FE modeling. The main conclusions of the work are as follows:

- The control specimen without internal shear reinforcement failed shortly after the initiation of shear cracks in a shear–compression mode of failure. The unstrengthened specimen having internal shear reinforcement failed in a diagonal–compression mode of failure. All strengthened specimens failed due to crushing of the diagonal strut in the shear span (i.e., diagonal compression mode of failure). Such a mode of failure is typically reported for strengthened RC deep beam elements [1,16].

- External shear strengthening with C-FRCM played a role similar to that of the internal steel stirrups. In the absence of internal stirrups, one layer of C-FRCM increased the shear capacity approximately two-fold. The significant improvement in the shear capacity caused by C-FRCM in the absence of internal steel stirrups is in line with other data reported previously for RC beams with \(a/h\) of 1.7, where a shear strength gain in the range of 77 to 100% was recorded [17,20].

- The use of two C-FRCM layers instead of one layer or changing the angle of orientation of the second layer had a negligible effect on the shear strength gain. This conclusion is consistent with other published data, indicating that doubling the number of C-FRCM increases the shear capacity by only 21% [17].

- The shear strength gain caused by the application of C-FRCM composites was less significant in the presence of internal stirrups. One layer of C-FRCM increased the shear capacity of the specimen with internal stirrups by 18%. A further increase in the number of C-FRCM layers did not result in an additional increase in the shear capacity. Positioning the second layer of carbon fabric in the horizontal direction tended to be less effective than placing it in the vertical direction. The reduced gain in the shear
capacity in the presence of internal stirrups is in line with other findings reported in the literature for RC deep beams, where a shear strength gain of 16% was reported at $a/h$ of 1.25 [16] and an average strength gain of 19% was reported at $a/h$ of 2 [21].

- Test results confirm the feasibility of using a cement-free geopolymeric matrix rather than the commercial cementitious mortar to develop a sustainable C-FRGM strengthening solution. One layer of C-FRGM resulted in 77 and 9% shear strength gains in the absence and presence of internal stirrups, respectively. The shear capacity of the specimens strengthened with C-FRGM (i.e., with a geopolymeric matrix) was 7 to 9% lower than that of their counterparts strengthened with C-FRCM (i.e., with a cementitious matrix). This is a new finding since no information is available in the literature on the behavior of RC deep beams strengthened in shear with geopolymer-based FRM composites. Nevertheless, the significant improvement in the shear capacity of RC deep beams due to strengthening in the absence of internal stirrups and the reduced strength gain recorded in the presence of steel stirrups are consistent with other findings published in the literature for cement-based FRM composites [16–21].

- The 3D numerical models developed in the current study are capable of predicting the nonlinear shear behavior of the tested beams with good accuracy. The inclusion of a bond–slip law at the fabric–matrix interface slightly reduced the contribution of the carbon fabrics to the shear capacity; hence, this resulted in more conservative shear strength predictions. Although the rate of increase of the midspan deflection predicted numerically was insignificantly different from that obtained from the experimental tests, the FE models tended to fail at a lower load, which resulted in a reduced deflection at failure relative to that measured experimentally. This behavior was less pronounced in the prediction of the FRM-strengthened models without internal steel stirrups. The reduced contribution of the carbon fabrics to the shear capacity due to the inclusion of the bond–slip law between the fabric and the matrix was less pronounced in the presence of internal stirrups.

**Author Contributions:** Conceptualization, T.E.-M. and H.E.-H.; methodology, T.E.-M. and H.E.-H.; software, N.K.A. and T.E.-M.; validation, N.K.A. and T.E.-M.; formal analysis, N.K.A., T.E.-M. and H.E.-H.; investigation, N.K.A., T.E.-M. and H.E.-H.; resources, T.E.-M.; data curation, N.K.A. and T.E.-M.; writing—original draft preparation, N.K.A. and T.E.-M.; writing—review and editing, N.K.A., T.E.-M. and H.E.-H.; visualization, N.K.A. and T.E.-M.; supervision, N.K.A., T.E.-M. and H.E.-H.; project administration, T.E.-M.; funding acquisition, T.E.-M. All authors have read and agreed to the published version of the manuscript.

**Funding:** This research was funded by UAE University, grant number 12N004.

**Institutional Review Board Statement:** Not applicable.

**Informed Consent Statement:** Not applicable.

**Data Availability Statement:** The data presented in this study are available on request from the corresponding author. The data are not publicly available due to privacy issues.

**Conflicts of Interest:** The authors declare no conflict of interest.

**References**

1. ACI Committee 549. Guide to Design and Construction of Externally Bonded Fabric-Reinforced and Steel-Reinforced Grout Systems for Repair and Strengthening of Concrete Structures; American Concrete Institute: Farmington Hills, MI, USA, 2020.

2. Awani, O.; El-Maaddawy, T.; Ismail, N. Fabric-reinforced cementitious matrix: A promising strengthening technique for concrete structures. Constr. Build. Mater. 2017, 132, 94–111. [CrossRef]

3. Koutas, L.; Tetta, Z.; Bournas, D.; Triantafillou, T. Strengthening of concrete structures with textile reinforced mortars: State-of-the-art review. J. Compos. Constr. 2019, 23, 03118001. [CrossRef]

4. Al-Salloum, Y.; Elsanadedy, H.; Alsayed, S.; Iqbal, R. Experimental and numerical study for the shear strengthening of reinforced concrete beams using textile-reinforced mortar. J. Compos. Constr. 2012, 16, 74–90. [CrossRef]

5. Azam, R.; Soudki, K. FRCM strengthening of shear-critical RC beams. J. Compos. Constr. 2014, 18, 04014012. [CrossRef]

6. Loreto, G.; Babaeidarabad, S.; Leardini, L.; Nanni, A. RC beams shear-strengthened with fabric-reinforced-cementitious-matrix (FRCM) composite. Int. J. Adv. Struct. Eng. 2015, 7, 341–352. [CrossRef]
7. Tetta, Z.; Koutas, L.; Bournas, D. Shear strengthening of concrete members with textile-reinforced mortar (TRM): Effect of shear span-to-depth ratio, material and amount of external reinforcement. *Compos. Part B* 2018, 137, 184–201. [CrossRef]
8. Tetta, Z.; Koutas, L.; Bournas, D. Textile-reinforced mortar (TRM) versus fibre-reinforced polymers (FRP) in shear strengthening of concrete beams. *Compos. Part B* 2015, 77, 338–348. [CrossRef]
9. Gonzalez-Liberos, G.; Sneed, L.; D’Antino, T.; Pellegrino, C. Behavior of RC beams strengthened in shear with FRP and FRCM composites. *Eng. Struct.* 2017, 150, 830–842. [CrossRef]
10. Azam, R.; Soudki, K.; West, J.; Noël, M. Behavior of shear-critical RC beams strengthened with CFRCM. *J. Compos. Constr.* 2018, 22, 04017046. [CrossRef]
11. Awani, O.; El-Maaddawy, T.; El Refai, A. Numerical simulation and experimental testing of concrete beams strengthened in shear with fabric-reinforced cementitious matrix. *J. Compos. Constr.* 2016, 20, 04016056. [CrossRef]
12. Triantafillou, T.; Papanicolaou, C. Shear strengthening of reinforced concrete members with textile reinforced mortar (TRM) jackets. *Mater. Struct.* 2006, 39, 93–103. [CrossRef]
13. Aljazaeri1, Z.; Myers, J. Strengthening of reinforced-concrete beams in shear with a fabric-reinforced cementitious matrix. *J. Compos. Constr.* 2017, 21, 04017041. [CrossRef]
14. Trapko, T.; Urbanska, D.; Kaminski, M. Shear strengthening of reinforced concrete beams with PBO-FRCM composites. *Compos. Part B* 2015, 80, 63–72. [CrossRef]
15. Tetta, Z.; Koutas, L.; Bournas, D. Shear strengthening of full-scale RC T-beams using textile-reinforced mortar and textile-based anchors. *Compos. Part B* 2016, 95, 225–239. [CrossRef]
16. Azam, R.; Soudki, K.; West, J.; Noël, M. Shear strengthening of RC deep beams with cement-based composites. *Eng. Struct.* 2018, 172, 929–937. [CrossRef]
17. Wakjira, T.; Ebead, U. Hybrid NSE/EB technique for shear strengthening of reinforced concrete beams using FRCM: Experimental study. *Constr. Build. Mater.* 2018, 164, 164–177. [CrossRef]
18. Wakjira, T.; Ebead, U. FRCM/internal transverse shear reinforcement interaction in shear strengthened RC beams. *Compos. Struct.* 2018, 201, 326–339. [CrossRef]
19. Wakjira, T.; Ebead, U. Internal transverse shear reinforcement configuration effect of EB/NSE-FRCM shear strengthening of RC deep beams. *Compos. Part B* 2019, 166, 758–772. [CrossRef]
20. Younis, A.; Ebead, U.; Shrestha, K. Different FRCM systems for shear-strengthening of reinforced concrete beams. *Constr. Build. Mater.* 2017, 153, 514–526. [CrossRef]
21. Marcinczak, D.; Trapko, T.; Musial, M. Shear strengthening of reinforced concrete beams with PBO-FRCM composites with anchorage. *Compos. Part B* 2019, 158, 149–161. [CrossRef]
22. Vasconcelos, E.; Fernandes, S.; de Aguiar, J.B.; Pacheco-Torgal, F. Concrete retrofitting using metakaolin geopolymers mortars and CFRP. *Constr. Build. Mater.* 2011, 25, 3213–3221. [CrossRef]
23. Menna, C.; Asprone, D.; Ferone, C.; Colangelo, F.; Balsamo, A.; Prota, A.; Cioffi, R.; Manfredi, G. Use of geopolymers for composite external reinforcement of RC members. *Compos. B Eng.* 2013, 45, 1657–1676. [CrossRef]
24. Bencardino, F.; Condello, A.; Ashour, A. Single-lap shear bond tests on steel reinforced geopolymeric Matrix-concrete joints. *Compos. B Eng.* 2017, 110, 62–71. [CrossRef]
25. Abu Obaida, F.; El-Maaddawy, T.; El-Hassan, H. Bond behavior of carbon fabric-reinforced matrix composites: Geopolymeric matrix versus cementitious mortar. *Buildings* 2021, 11, 207. [CrossRef]
26. ACI 318-19; Building Code Requirements for Structural Concrete—Commentary on Building Code Requirements for Structural Concrete (ACI 318R-19). American Concrete Institute (ACI): Farmington Hills, MI, USA, 2019.
27. ASTM C39/C39M-20; Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens. ASTM International: West Conshohocken, PA, USA, 2020.
28. BS EN 12390-3; Testing Hardened Concrete—Compressive Strength of Test Specimens. British Standards: London, UK, 2009.
29. ASTM C496/C496M-17; Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens. ASTM International: West Conshohocken, PA, USA, 2017.
30. BS 4449:2005 +A2:2009; Steel for the Reinforcement of Concrete—Weldable Reinforcing Steel–Bar, Coil and Decoiled—Product Specification. British Standards: London, UK, 2009.
31. S&P—A Simpson Strong-Tie Company. S&P ARMO-Mesh Reinforcement Made from Carbon Fiber. Available online: https://www.sp-reinforcement.ch/en-CH (accessed on 10 December 2019).
32. ISO—International Organization for Standardization. Available online: https://www.iso.org/home.html (accessed on 7 March 2022).
33. ASTM C109/C109M-20; Standard Test Method for Compressive Strength of Hydraulic Cement Mortars (Using 2-in. or [50 mm] Cube Specimens). ASTM International: West Conshohocken, PA, USA, 2020.
34. ASTM C469/C469M-14; Standard Test Method for Static Modulus of Elasticity and Poisson’s Ratio of Concrete in Compression. ASTM International: West Conshohocken, PA, USA, 2014.
35. ATENA [Computer Software], n.d. Cervenka Consulting s.r.o., Prague, Czech Republic. Available online: https://www.cervenka.cz/products/atena/ (accessed on 7 March 2022).