Investigation of punching shear performance in concrete slabs reinforced with GFRP and synthetic fibers: An experimental study

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1. Introduction

In specific circumstances, particularly within corrosive environments, glass fiber-reinforced polymer (GFRP) has gained popularity as an alternative to conventional steel reinforcement. The use of GFRP reinforcement can be beneficial as the production of steel involves high environmental and economic costs, and steel is known to have a limited lifespan due to corrosion [1]. Increased attention has been drawn to GFRP due to recent progress in material engineering (and as an effective alternative to steel), manufacturing methods, the introduction of innovative materials, and the emergence of new technologies [2]. GFRP is relatively inexpensive and widely available compared to other types of fiber-reinforced polymer (FRP) materials [3]. Furthermore, due to its relatively recent emergence as a manufacturing technology in comparison to steel, GFRP has witnessed a reduction in its economic and environmental costs over recent years, rendering it progressively more competitive in terms of pricing. Consequently, there has been a notable surge in the practical utilization of GFRP reinforcement in various real-world scenarios. Although GFRP may be regarded as a viable replacement for steel reinforcement, the substantial inherent distinctions in their physical and mechanical characteristics caused GFRP-RC to behave differently at both the member and structural levels [4–6].

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The primary distinction lies in the fact that an elastoplastic behavior is demonstrated by steel, whereas GFRP maintains linearity in the elastic region until it experiences failure [7]. Also, the modulus of elasticity of GFRP is approximately one-third to one-fourth that of steel. Therefore, it is necessary to consider these behavioral differences in the design of concrete elements reinforced with GFRP. The approach involves adapting design provisions that were created for steel-reinforced concrete structures to accommodate the variations arising from the substantial disparities in material properties inherent to GFRP [8–10].

Due to the lower elastic modulus of GFRP material, GFRP-RC flexural elements typically experience greater deformation than their steel-RC counterparts, resulting in design mostly being dominated by the serviceability provisions such as deflections and crack widths, in place of ultimate limit state conditions [11]. To address this issue and satisfy the codes’ design requirements, the thickness of the flexural members and/or the ratio of the reinforcement should be increased [12]; however, this increases the structural weight and construction cost. More innovative and effective approaches and techniques need to be developed to ensure the proper employment of GFRP materials while meeting both strength and serviceability requirements.

Shear failure can occur in the flexural members depending on the level of moment and shear force. Compared to steel-RC flexural members, GFRP-RC members are more susceptible to shear failure [13,14] and their reinforcement is less significant in the transmission of shear loads because of several reasons (a) due to the reduced modulus of elasticity of the GFRP reinforcement, wider cracks are generated, resulting in a decreased interaction between the crack walls, thereby decreasing the load-transferring mechanism via aggregate interlocking [15,16], (b) less shear resistance is provided by the concrete in compression in GFRP-RC slab compared to steel-RC slab with the same reinforcement ratio [17], (c) the tensile capacity of the shear stirrups are reduced significantly compared to straight bars due to the reduced strength at the bent region [18], and (d) the dowel effect of the longitudinal reinforcement in GFRP bars is small [15,17,19], primarily because the shear capacity of the rebar in the transverse direction, which is conferred by the resin, is relatively limited. In concrete slabs, shear reinforcement is not commonly used. Due to the low strength and stiffness of GFRP bars across the cross-section, if shear failure occurs in the GFRP-RC slabs, it would be more abrupt and catastrophic than in the steel-RC counterparts. The negative effect of shear cracks on the longitudinal reinforcements and the flexural load-carrying capacity of the members is particularly detrimental in areas where both shear and flexural stress are present, such as around supports.

The punching shear strength of GFRP-RC slabs is influenced by factors such as; slab thickness, column cross-section size and shape, and the slab reinforcement ratio [20,21]. Specifically, increasing the slab flexural reinforcement ratio leads to higher punching shear capacity and secant stiffness in high-strength concrete slabs due to an increase in the neutral axis depth. Shear studs have been found to enhance both the level of moment and shear force. Compared to steel-RC flexural members, GFRP-RC slabs, it would be more abrupt and catastrophic than in the steel-RC counterparts. The negative effect of shear cracks on the longitudinal reinforcements and the flexural load-carrying capacity of the members is particularly detrimental in areas where both shear and flexural stress are present, such as around supports.

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Table 1

<table>
<thead>
<tr>
<th>Mix</th>
<th>Fiber content (%)</th>
<th>Cement (kg/m³)</th>
<th>14 mm stone (kg/m³)</th>
<th>Sand (kg/m³)</th>
<th>Water (kg/m³)</th>
<th>Superplasticizer (kg/m³)</th>
<th>Average compressive strength (MPa)</th>
<th>Average flexural tensile strength (MPa)</th>
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<tr>
<td>No Fiber</td>
<td>0</td>
<td>300</td>
<td>1044</td>
<td>841</td>
<td>210</td>
<td>1.2</td>
<td>30.2</td>
<td>3.05</td>
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<tr>
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<td>1044</td>
<td>841</td>
<td>210</td>
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<td>210</td>
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<td>28.7</td>
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Table 2

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<tr>
<td>Length (mm)</td>
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<td>47</td>
<td>8</td>
</tr>
<tr>
<td>Thickness (µm)</td>
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<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Diameter (µm)</td>
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<tr>
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<td>1600</td>
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<tr>
<td>Young’s Modulus (GPa)</td>
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</tr>
<tr>
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<td>225</td>
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<tr>
<td>Water Absorption</td>
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<td>Nil</td>
<td>&gt; 1% by weight</td>
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</table>

Table 3

<table>
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<tr>
<th>Property</th>
<th>PPS</th>
<th>PPL</th>
<th>PVA</th>
</tr>
</thead>
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<tr>
<td>Melting point °C</td>
<td>160</td>
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<tr>
<td>Water Absorption</td>
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<td>Nil</td>
<td>&gt; 1% by weight</td>
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</table>

AS1012.9 [34], to evaluate the compressive properties of the mixes. Additionally, three beam specimens of 300 × 100 × 100 mm were prepared and tested under four-point bending according to AS1012.11 [35] to determine the flexural strength properties of each mix. To exclude the influence of varying curing on the strength of concrete, the cylinders and beam specimens were subjected to identical curing conditions as those applied to the slabs and were subsequently tested concurrently with the slab testing. The stress-strain response of the specimens with no fibers. As shown in Fig. 1(a), a more gradual decrease in the strength of the FRC was observed after the peak point, indicating the positive influence of fibers in improving the ductility of concrete mixes. Additionally, as shown in Fig. 1(b), while the flexural tensile strength of the specimens with plain concrete and PVA fibers decreased to about zero beyond the peak point, a more gradual strength reduction was observed for the concrete made with both short and long PP fibers. This can be attributed to the gradual pull-out of the fibers in the cracked regions of the beam specimens, while in the case of PVA specimens, fracture of fibers rather than pull-out governed their post-peak behavior resulting in no post-peak capacity.

The GFRP bars (ϕ13) employed in this investigation were produced through a pultrusion and underwent sand coating to boost their adhesive potency. Steel bars were found to adhere to class N (normal ductility) standards as per the Australian standard [10], exhibiting a 500 MPa yield strength with nominal diameters of 12 mm. Table 3 provides the properties of equivalent GFRP reinforcements (tested according to CSA-S807 [36]) along with the nominal properties for the steel bars.

2.2. Test specimens

This study involved testing of 1/3 scale 11 two-way slabs under a punching load applied at the center of the slabs. All slabs were square with a dimension of 1000 × 1000 mm and a thickness of 80 mm. A clear concrete cover of 20 mm was maintained for the main reinforcements. The test program was designed to investigate the effect of different fiber types, and reinforcement types/ratios on the performance of two-way slabs. One layer of reinforcement mesh was used in all the slabs with no shear reinforcement. The specimens were grouped into three test sets based on the slab reinforcement. The first set of slabs was cast using steel reinforcement. The second and third sets examined the performance of slabs reinforced with GFRP with a reinforcement ratio of 1.0% and 1.5%, respectively. Each set was designed to investigate the performance of slabs with different types of fibers. The specimens were labeled in Table 4 to indicate the type of mix and reinforcement used. Three types of reinforcement, labeled as “S”, “F1”, and “F2”, were used to identify the type/ratio of reinforcement in the slabs. Specimens labeled “S” were reinforced with ϕ12 @ 240 spacing steel bars in both directions, resulting in a longitudinal reinforcement ratio of around 0.9%. The slabs labeled “F1” and “F2” were reinforced with ϕ13 @ 240 mm and ϕ13 @ 160 mm spacing at both, corresponding to a reinforcement ratio...
of 1.0% and 1.5%, respectively. The type of fiber used was also indicated in the specimen labels; “N” indicating plain concrete with no fiber, “PVA” indicating FRC mixes with PVA fibers, “PPS” indicating FRC mixes with PP short fibers, and “PPL” indicating FRC mixes with PP long fibers.

2.3. Specimen preparation and test setup

The process of preparing the slab specimens is shown in Fig. 2. Square wooden forms were constructed, and silicone was applied to seal the edges in order to prevent any water leakage. The steel and GFRP reinforcement meshes were assembled using plastic zip ties and placed in the forms, as shown in Fig. 2. These meshes were secured in their desired location using 20 mm spacers to form the concrete cover. In order to quantify the strain that emerged in the reinforcement during testing, strain gauges with a gauge length of 20 mm were affixed to the central longitudinal bar in each direction at the mid-span of the slab. Reinforcement meshes were affixed with lifting lugs, facilitating the convenient maneuvering and handling of slabs via the laboratory crane. Careful vibration was applied to the concrete, with particular attention to the FRC slabs, to achieve thorough compaction and eliminate any trapped air. The slabs were covered with plastic covers for 24 h to minimize temperature and shrinkage cracking. After being removed from the forms, the slabs were kept in the ambient laboratory environment before testing. The cylindrical and beam samples were cast at the same time as the corresponding slabs and cured under the same conditions. Before testing, the slabs were coated in white paint and imprinted with gridlines to facilitate the tracking of crack positions and orientations throughout the testing process.

The punching shear test setup for slabs was designed as depicted in Fig. 3 and Fig. 4. After about two months of being cured, the slabs were positioned upside down and loaded from the bottom face to effectively monitor the formation and spread of cracks that occurred on the top surface during testing. Small 50 × 50 mm hunches were made in the corners of the slabs to facilitate the setting up of the slabs as shown in Fig. 4. The top reaction frame consisted of four rectangular hollow sections welded tougher to form a square. The top frame was bolted down to the strong floor using four Dywidag bars (38 mm in diameter). To provide line-load support and avoid moment transmission at the edge of the supports and also to reduce friction, roller condition was simulated by using an assembly of a central 20 mm diameter steel bar sandwiched between two 6 mm thick steel plates, which were placed between the loading frame and the slab specimens as shown in Fig. 4. Rubber pads having a thickness of 5 mm were placed as a filler underneath the top frame to allow an even distribution of the reaction force and avoid premature failure due to stress localization. Linear Variable Differential Transformers (LVDTs) were placed at the center and edges of the slabs to record their deflection during the test, as shown in Fig. 4. The slabs were subjected to force-controlled monotonic loading using a 650-kN hydraulic jack until failure occurred. The loading was applied to the bottom of the slabs through a 100 × 100 × 16 mm steel plate. The loaded area of 100 × 100 mm represents 1% of the slab area. A 15 mm thick rubber pad was used in between the loading plate and slab to provide uniform distribution of the load.

3. Discussion of results

3.1. Patterns of developing crack and modes of failure

The general crack development pattern in all specimens was almost the same. The first observed crack began to propagate at the center of the slabs in two perpendicular directions and parallel to the slab edges. The cracking load for specimens without fiber and with PP fibers was about 14–20 kN, while the cracking load for specimens with PVA fibers was found to be greater and ranged between 20 and 27 kN. With increasing the applied load, radial inclined cracks started to propagate from the point of load application towards the slab corners. The width of developed cracks was found to be dependent on both slab reinforcement and fiber type. The crack width at applied loads of 35 kN and 50 kN was
recorded to gain insights into the behavior of the test specimens. For Set I with steel reinforcement, the measured crack widths at an applied load of 35 kN were 1.0 mm, 0.8 mm, and 0.8 mm for specimens without fibers, with PVA fibers, and with PPS fibers, respectively; while it reached 1.5 mm, 1.0 mm, and 1.2 mm, respectively up on increasing the applied load to 50 kN. The crack width for Set II specimens with GFRP reinforcement was found to be larger than that of steel reinforcement. For Set II with a reinforcement ratio equal to 1.0%, the observed crack

Fig. 3. Testing apparatus in plan and section views (dimensions in mm).

Fig. 4. Test setup.
widths at a load equal to 35 kN were 2.5 mm, 0.4 mm, 0.6 mm, and 0.8 mm for the case with no fiber, PVA, PPS, and PPL fibers, respectively; while it increased to 10 mm, 2.5 mm, 3.5 mm, and 5 mm, respectively with increasing the applied load to 50 kN. The increase of the GFRP reinforcement ratio to 1.5% in Set III controlled the width of developed cracks. The measured crack widths at a load equal to 35 kN was 1.0 mm, 0.3 mm, 1.0 mm, and 0.9 mm in the case with no fibers, PVA, PPS, and PPL fibers, respectively and it was increased to 3.5 mm, 1.3 mm, 1.8 mm and 3.2 mm, respectively with increasing the load to 50 kN. However, the number of developed cracks in Set III was greater than that of Set I and Set II. The results indicated that the slab with PPS fibers exhibited a slower increase in crack width compared to that with PPL fibers. These results are consistent with the crackling sound heard during the test and the low tensile strength of PPL fibres. Interestingly, the crackling sound was noticed in the slab with PPS fibers at an earlier load than in the slab with PPL fibers. This explains the higher efficiency of short fibers PPS compared to longer fibers PPL in bridging the cracks and controlling the widening of developed cracks. Meanwhile, a sudden loud bang was heard in the slab with PVA fibers at failure. A closer inspection of the slabs revealed the rupture of PVA fibers, causing the loud bang, whereas PP fibers were pulled out of the concrete over a longer period.

The final crack pattern observed for each specimen of the three sets is shown in Fig. 5. As shown, the provision of fibers led to a reduction in the width of developed cracks. Moreover, the use of PVA fiber proved its high efficiency in limiting the width and number of developed cracks as well as increasing the cracking load of the slabs compared to other specimens. As shown in Fig. 5, following the formation of radial cracks, circumferential cracks were observed at about 200–300 mm from the slab center. These cracks formed when the specimens peaked at the capacity, which resulted in the formation of a punching cone around the point of load application and consequently ultimate failure of the specimens. The inspection of failure surfaces after the test confirms that all specimens failed due to punching shear failure.

3.2. Load-deflection behavior

Fig. 6 shows the obtained load-deflection curves for the specimens of the three test sets. The deflection was measured using LVDT located at the top of each slab. For each test set, the type of fibers significantly affected the deformation capacity, stiffness, and strength of the specimens. The load-deflection behavior for both steel and GFRP-reinforced specimens showed a sharp drop in the load-carrying capacity after reaching the peak load indicating a brittle shear failure of the specimens. As shown, all slabs displayed stiffness degradation as the initial cracking was developed. This is consistent with the crack observations made earlier. From the point of initial cracking to the failure load at a large deflection-to-span ratio of $\Delta/L = 4–6\%$, the stiffness of the GFRP-RC
slabs remained relatively constant. In contrast, significant alterations in stiffness were observed in the steel-reinforced concrete (steel-RC) slabs at notably lower deflection levels, approximately within the range of 21–23 mm (corresponding to a Δ/L of 1.2–1.4%). This observation suggests the yielding of (longitudinal) steel and the development of plastic hinges. Set II with a GFRP reinforcement ratio of 1.0% exhibited a more pronounced improvement in slab capacity and stiffness, as well as improved deformation capacity compared to the other two sets. The influence of PVA fibers on slab stiffness was particularly noticeable in Set III with GFRP reinforcement, while the stiffness improvement and deformation capacity due to the provision of fibers were limited for Set I with steel reinforcement compared to GFRP in the other two sets.

Table 5 presents the key results from the tests including the cracking loads, the maximum loads, the corresponding deflections, and the deflection at the failure load (20% strength drop). The results show that the specimens exhibited a sharp drop of strength after reaching the maximum strength. In general, there is little variation between the ultimate deflection and the deflection at maximum load, indicating a sudden brittle punching shear failure. However, the efficiency of the provision of PVA and PPS fibers in improving the slab’s deformation capacity was more for steel-RC slabs. In Set I with steel reinforcement, the ultimate deflection increased from 26.7 mm to 32.2 mm and 36.7 mm, corresponding to 20% and 37% improvements, respectively, with the provision of PVA and PPS fibers. Conversely, the presence of fibers had a minor impact on the deformation capacity of GFRP-RC slabs.

Analytical calculations were carried out to estimate the flexural capacity of the slab and check for the occurrence of flexural failure. The common yield line analysis method was employed (see Fig. 7a) [32]. For square slabs under concentrated loads, the peak load P equals 8 M, where M represents the flexural capacity of the slab obtained from the equilibrium of internal forces from cross-section analysis (see Fig. 7b) [32]. The flexural capacity of each slab, considering the reinforcement ratio, reinforcement type, and concrete strength, is listed in Table 5. A comparison between the obtained failure load and the flexural capacity of the slab indicates that none of the slabs had reached their flexural strength capacity, confirming no flexural failure occurred.

### 3.3. Effect of fiber type

In Fiber-Reinforced Concrete (FRC), shear forces are transmitted across the failure surface through five mechanisms: aggregate interlock, dowel action, compression ring, residual tensile stresses carried by the matrix, and fibers bridging the failure crack [37]. The addition of fibers can potentially improve both the compression and tensile properties of concrete. When subjected to tensile forces, concrete demonstrates distinct stress-strain and stress-crack opening relationships, influenced by both the concrete matrix and fiber presence. While the matrix’s contribution diminishes rapidly as cracks widen, fibers exhibit two mechanisms: initially bridging the crack, then decreasing in effectiveness due to fiber pull-out or rupture as the crack widens further [37]. Factors such as fiber volume, geometry, and the bond between the matrix and fibers greatly affect the fibers’ effectiveness.

The test results indicate that the inclusion of fibers impacts the inclination of the punching shear cone, cracking load, and ultimate capacity. The influence on each parameter varies depending on the fiber type. The inclination angle of the punching shear cone can be estimated by calculating the tangent of the slab thickness divided by the distances between the column edge and the failure surface, as shown in Fig. 5 [38, 39]. Table 5 provides the estimated inclination angles for all slabs. Generally, values for slabs with fiber additions (21–31.7°) surpass those without fibers (16.4–24.7°). Particularly, with short-length fibers like PPS and PVA, the inclination angle is higher than that with long fibers. This aligns with observations in [38] where the inclusion of short fibers reduced the need for contribution from concrete shear, resulting in a smaller punching cone and higher inclination angles [40].

In terms of cracking load, the effect of adding PPS or PVA fibers...
increased the cracking load from 17 kN in the steel-RC slab without fibres to 20 kN. In GFRP-RC slabs, the influence of adding PVA fibres showed a significant impact in increasing the cracking load. Meanwhile, the effect of PP fibres had minimal effect on the cracking load of GFRP-RC slabs. A similar observation was also reported previously where the addition of fibres had almost no effect on the cracking load of FRP-RC slabs [38].

The addition of PVA fibre showed a superior effect in terms of capacity in all sets of slabs. The reason is that the majority of PVA fibres were ruptured without pulling out from concrete which confirms the better contribution of PVA to the punching shear mechanism. Unlike PVA, the majority of PPS fibres were not ruptured, and instead pulled out from concrete making them less effective in terms of strength enhancement. The pull-out of fibres was confirmed by the formed holes in concrete in the concrete seen upon failure inspection. The pull-out of fibres became increased for a slab with PPL which gives evidence of the inefficiency of these fibres in bridging the cracks to reach higher strengths.

The maximum capacity of the slabs in Set I, which were reinforced with steel reinforcement, without fibers, with PVA fibers, and PPS fibers, were 78.9 kN, 90.0 kN, and 89.4 kN, respectively. The capacity of the slabs was increased by 14% with the use of both PVA and PPS fibers, and the stiffness of the slabs was approximately the same for both types of fibers (see Fig. 6). The comparable capacities align well with the similar cracking loads of both slabs.

The difference in slab’s behavior with different fiber types was more noticeable in Set II, where the slabs were reinforced with GFRP. In Set II, with a GFRP-RC slab with a reinforcement ratio of 1.0%, the maximum capacity of the slabs was 55.8 kN, 84.2 kN, 70.5 kN, and 68.0 kN without fibers, with PVA fibers, PPS fibers, and PPL fibers, respectively. The addition of PVA fibers significantly increased the capacity of the slab by 51%, while PPS and PPL fibers resulted in an increased capacity of 27% and 22%, respectively, compared to the slab with no fibers. Additionally, the slab with PVA fibers exhibited a relatively stiffer behavior compared to those with PPS or PPL fibers.

In Set III, with a GFRP reinforcement ratio of 1.5%, the maximum capacity of the slabs without fibers, with PVA fibers, PPS fibers, and PPL fibers was 75.0 kN, 108.0 kN, 83.6 kN, and 74.1 kN, respectively. Similar to Set II, the use of PVA fibers resulted in the greatest improvement of the capacity by 44% compared to the specimens without fibers. The improvement of the capacity with PPS was limited to only 11%. Additionally, the maximum capacity for specimens with PPL fibers yielded a capacity approximately similar to that obtained for the slab without fibers. Overall, the results demonstrate the high efficiency of PVA fibers in improving the slab capacity compared to PPS fibers, while PPL was found to be less effective. It is worth mentioning that none of the tested slabs exceeds its flexural capacity given in Table 5 which confirms that all specimens failed due to punching shear failure.

### 3.4. Effect of reinforcement type

The behavior of steel and GFRP reinforcement is different due to their different characteristics. Steel is an elastoplastic material with a ductile failure, while GFRP is a linear elastic material with a brittle failure mode. Moreover, the modulus of elasticity of GFRP is significantly lower than that of steel. The results of Set I and Set II, with approximately similar reinforcement ratios (0.9% and 1.0%), describes the behavior of slabs with steel and GFRP reinforcement. The comparison between the load-deflection behavior of specimens with steel and GFRP reinforcement, as shown in Fig. 8, revealed that the initial stiffness was relatively the same for steel and GFRP-RC slabs. However, beyond the load of about 30 kN, specimens with GFRP reinforcement exhibited less stiff behavior and higher deformations due to the lower modulus of elasticity. Fig. 9 also shows a comparison between the maximum
The control specimen with GFRP reinforcement had a capacity of 78.9 kN and 55.8 kN using steel and GFRP reinforcement, respectively. The strength of the control specimen with GFRP reinforcement was lower than that with steel reinforcement by 43%. This is attributed to the low modulus of elasticity of GFRP, which also resulted in wider cracks. The same observation was made for the slabs with the fibers, where the capacity of GFRP-RC slabs was lower than that with steel reinforcement by 7% and 27% when using PVA and PPS fibers, respectively, compared to the corresponding slabs reinforced with steel. When the PVA fibers were employed in the steel-RC slab, the punching shear capacity increased from 78.9 kN to 90 kN with an increase of about 14%. However, when the same dosage of PVA fibers was used in
the GFRP-RC slab, the capacity increased from 55.8 kN to 84.2 (an increase of 51%). Moreover, with the use of the same dosage of PPS fibers, the capacity of the slab was increased by 13% and 26% for steel-RC slabs and GFRP-RC slabs, respectively. The results indicate the significantly higher impact of PVA fibers on improving the punching shear capacity of GFRP-RC slabs compared to that of steel-RC. As shown in Fig. 8, the inclusion of PVA fibers for GFRP-RC slabs showed high efficiency in improving the slab capacity, with the difference between the capacity of GFRP and steel slabs being only 6%. This can be attributed to the yielding of the steel reinforcement while the PVA fibers demonstrated high efficiency in controlling the wide cracks associated with GFRP reinforcement and hence significantly improved the load capacity of GFRP-RC slabs.

3.5. Effect of reinforcement ratio

Fig. 10 compares different load-deflection responses of specimens with different GFRP reinforcement ratios (1.0% and 1.5%) when using different types of fibers. The initial stiffness of all specimens was almost the same, beyond which the stiffness degradation was observed which was more pronounced in the slabs with 1.0% GFRP reinforcement. When the reinforcement ratio increased from 1.0% to 1.5% for slabs without fibers, the capacity increased by 35%, while the improvement was 28%, 19%, and 9% for PVA, PPS, and PLL fibers, respectively. The results demonstrate that the type of fiber used influences the contribution of the GFRP reinforcement ratio to the slab punching shear capacity. The influence of the reinforcement ratio was more pronounced for slabs without fibers (see Fig. 11-a) Furthermore, Fig. 11-b shows that while the reinforcement ratio for steel-RC slabs was 0.9%, the reinforcement ratio in the GFRP reinforced slabs needs to be increased to 1.5% to provide a similar capacity in case of no fiber content or PPS fibers. Keeping the GFRP reinforcement ratio at 1.5% and using PVA fibers resulted in a 22% higher capacity in that slab compared to the steel-RC slab with 0.9% reinforcement. From the above results, it can be concluded that the use of PVA fibers can substantially enhance the punching shear capacity of GFRP-RC slabs.

4. Punching shear strength prediction

The capacity of the test slabs was compared to the calculated punching shear strength using different standards and previously developed models. The punching shear capacity of steel-RC slabs was compared with the ACI 318–19 code [26], the British Standards BS 8110 [41], and Model Code MC2010 [42]. These three predictive models are commonly used in design codes to predict the punching shear capacity of steel-RC slabs. Nonetheless, the influence of fibers on the punching shear...
strength of the slabs is not considered by these models. The punching shear capacity according to ACI 318–19 [26] is obtained by Eq. (1), which considers the influence of the location of the column through $a_i$ factor, taken as 40 for interior columns, and the effect of the column geometry $\beta$, defined as the ratio of the long dimension to the short dimension of the column.

$$ V_c = \min \left\{ \begin{array}{c} 0.33 \lambda_c \sqrt{f_c} \\ 0.083 \left( 2 + \frac{4}{\beta_i} \right) \lambda_i \sqrt{f_i} \\ 0.083 \left( 2 + \frac{a_i d}{b_i} \right) \lambda_i \sqrt{f_i} \end{array} \right\} $$

where $\lambda_c$ is a size effect factor $= \sqrt{\frac{2}{1 + \frac{d}{D}}}$, and $b_i$ is the punching shear perimeter measured at a distance equal to $d/2$ from the column face. The punching shear strength according to BS8810 [41] is obtained as

$$ V_c = 0.27 \left( 100 \rho_f \right)^{1/3} (f_{cu})^{1/3} \left( \frac{400}{\rho_f} \right)^{1/4} b_{ci,1.5d} d $$

where $\rho_f$ is the slab reinforcement ratio, $f_{cu}$ is the cube’s compressive strength, which can be taken equal to 1.25$f_{c'}$ [43], while $b_{ci,1.5d}$ is the punching shear perimeter measured at a distance equal to 1.5$d$ from the column face.

Comparing Eqs. (1) and (2) show that unlike ACI 318–19, BS 8110 code that takes into consideration the influence of the slab reinforcement ratio $\rho_f$, the effect of column location and column geometry is not included in BS 8110.

The punching shear capacity of steel-RC slabs according to MC2010 can be obtained using Eq. (3).

$$ V_c = k_v \sqrt{f_c b_i d} \tag{3} $$

where $k_v$ is a factor accounts for shear crack width and is given by Eq. (4).

$$ k_v = \frac{1}{1.5 + 0.9 \psi dk_{ag}} \geq 0.75 \tag{4} $$

where $\psi$ accounts for the slab rotation, and $k_{ag}$ accounts for the aggregate interlocking by considering the maximum aggregate size $d_a$ and is estimated as $k_{ag} = \frac{d_a}{100} > 0.6$.

A comparison is provided in Table 6 between the punching shear strength for steel-RC slabs as predicted and observed experimentally based on the standards ACI 318–19 and BS 8110. The results show that ACI 318–19 provides conservative prediction and underestimates the punching shear capacities by 30% to 42%. The mean value and standard deviation of the ratio between the experimental and predicted strength are 1.37 and 0.05, respectively. It is evident that ACI 318–19 provided conservative estimates for both the control reinforced concrete slab and fiber reinforced slabs. When fibers were added to the slabs, the conservatism was further increased as the code equations were developed for conventional concrete, not FRC. Using BS 8110 equations, the mean value and standard deviation of the ratio between experimental and predicted strength are 1.0 and 0.04, respectively. Although similar to ACI 318–19, BS 8110 does not account for the contribution of fibers, the results demonstrate a better accuracy of BS 8110 in predicting the punching shear strength of slabs, which can be due to the incorporation of the effect of the slab reinforcement ratio. The results according to MC2010 provided the most accurate results for slabs without fibers due to effective consideration of shear crack width and aggregate interlocking. However, the estimation showed lower prediction values since this model does not account for the effect of fibers in controlling the shear crack width. Generally, MC2010 equations lead to the mean value and standard deviation of the ratio between experimental and predicted strength are 1.07 and 0.04, respectively. According to these results, while the ACI 318–19 equation provides a very conservative prediction and MC2010 accurately predicts the capacity of the slab without fibers, the current equation of BS8110 results in a reasonable estimation of the punching shear capacity and can be adopted for steel-RC slabs reinforced with synthetic fibers.

The previous formulas used for predicting the punching shear capacity of steel-RC slabs cannot be directly applied to GFRP-RC slabs due to the different behavior of these two types of reinforcement. Various standards including American Standards ACI 440.1R-15 [18] and ACI 440.11–22 [12], as well as the Japan Society of Civil Engineering Standards JSCE [44], offer different formulas for evaluating the punching shear capacity of GFRP slabs. The punching shear strength of GFRP-RC slabs according to ACI 440.1R-15 [18] is determined using Eq. (5), which is a modified version of the equation provided in ACI 318–19 [26] by the inclusion of a factor $k_{fr}$ to reflect the effect of the axial force.

Table 6

<table>
<thead>
<tr>
<th>Slab label</th>
<th>Test load (kN)</th>
<th>Predicted load (kN)</th>
<th>Test/Prediction</th>
<th>Standard deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>SN</td>
<td>78.9</td>
<td>60.8</td>
<td>84.6</td>
<td>77.6</td>
</tr>
<tr>
<td>SPVA</td>
<td>90.0</td>
<td>65.1</td>
<td>88.5</td>
<td>83.2</td>
</tr>
<tr>
<td>SPDS</td>
<td>89.4</td>
<td>63.0</td>
<td>86.5</td>
<td>80.4</td>
</tr>
<tr>
<td>Mean</td>
<td>1.37</td>
<td>1.00</td>
<td>1.07</td>
<td></td>
</tr>
</tbody>
</table>

Fig. 11. Comparison of slab capacities having different reinforcement ratio.
stiffness of GFRP reinforcement.

\[ V_c = 0.8k_r \sqrt{f_d b_{0.5} d} \]  

(5)

where; \( k_r = \sqrt{2\rho_f \nu_f + (\rho_f \eta_f)^2 - \rho_f \nu_f \eta_f} = E_f/E_s, E_f \) is the modulus of elasticity of GFRP, \( E_s \) is the concrete modulus of elasticity, and \( \rho_f \) is the reinforcement ratio.

The most recent version of ACI 440.11–22 [12] further modified Eq. (5) as shown in Eq. (6),

\[ V_c = 0.83 \lambda_c \sqrt{f_d} \ b_{0.5} d \geq 0.13 \lambda_c \sqrt{f_d} \ b_{0.5} d \]  

(6)

These modifications are to account for the size effect through \( \lambda_c \) and to provide an absolute lower bound for the punching shear capacity.

Similar to ACI 318–19 [26], none of the equations of the ACI codes, Eq. (3) and Eq. (4), account for the slab reinforcement ratio. Moreover, they do not consider the influence of column dimension and location.

The punching shear capacity according to JSCE [44] is calculated using Eq. (7), which incorporates several parameters, including the reinforcement ratio and the axial stiffness of GFRP.

\[ V_c = \frac{\beta_p \rho_f f_{d,ac} b_{0.5} d}{f_d} \]  

(7)

where; \( \beta_p \) represents the size effect, \( \beta_p = ^{1/4} (1000/d)^1/4 \leq 1.5 \), \( \beta_p \) account for the axial stiffness \( \beta_p = ^{1/3} (100 \rho_f E_f/E_s)^{1/3} \leq 1.5 \), and \( \beta_k = 1 + 1/(1 + 0.25 s/d) f_{d,ac} = 0.2 \sqrt{f_d} \leq 1.2 \text{MPa} \).

Previous studies have proposed alternative equations for estimating the punching shear strength of GFRP-RC slabs. El-Ghandour et al. [45] proposed modifications to the ACI 318–19 [26] equation by introducing a factor \( (E_f/E_s)^{1/3} \) to account for the axial stiffness of GFRP, as described by Eq. (8).

\[ V_c = 0.33 \sqrt{f_d} (E_f/E_s)^{1/3} b_{0.5} d \]  

(8)

Ospina et al. [46] proposed Eq. (9) considering the effect of axial stiffness through \( \sqrt{f_d} \). 

\[ V_c = 2.77 (\rho_f f_d)^{1/3} \sqrt{E_f/E_s} b_{0.5} d \]  

(9)

Unlike the aforementioned models and standards [12,18,44,45], Eq. (9) takes into account the influence of the slab reinforcement ratio \( \rho_f \). Similarly, to account for the axial stiffness of GFRP reinforcement, Matthys and Taerwe [47] proposed a modified version of the equation given by BS 8110, as shown in Eq. (10).

\[ V_c = 1.36 \left[ (100 \rho_f)^{1/3} f_d \right]^{1/3} \left( \frac{E_f}{E_s} \right)^{1/3} \left( \frac{1}{d} \right)^{1/4} b_{0.5} d \]  

(10)

In this equation, the effect of the axial stiffness of GFRP reinforcement is reflected through the factor \( \left( \frac{E_f}{E_s} \right)^{1/3} \).

The comparison of the experimental findings with the predictions of different models is summarized in Table 7, and the variation of the results is illustrated in Fig. 12. Among the models, the predicted strengths according to ACI 440.1R-15 [18] and ACI 440.11–22 [12] were found to be the most conservative. The mean value for the ratio between the experimental and predicted strengths according to these two standards was 2.34 and 2.25, respectively. Similarly, the predicted strengths based on the models developed by El-Ghandour et al. [45] and Ospina et al. [46] showed an overestimation with a mean test to predicted strengths ratios equal to 1.85 and 2.06, respectively. The JSCE [44] and ACI models (12) produced more accurate results, with a mean value and standard deviation for the ratio between the test and predicted strength of 1.4 and 0.17, respectively. The JSCE [44] formula considers various parameters such as the axial stiffness of GFRP, reinforcement ratio, and size effect, which contribute to higher accuracy compared to previous models. Furthermore, the prediction according to Matthys and Taerwe [47] was found to be the most accurate compared to the test results. The ratio between the test and predicted strengths had a mean value of 1.18 and a standard deviation of 0.16. These results highlight the high efficiency of the model developed by Matthys and Taerwe [47], which considers the axial stiffness of GFRP, slab reinforcement ratio, and size effect. This model was primarily developed based on the principles outlined in BS 8110 [41], which has shown their accuracy in estimating the punching shear strength.

![Fig. 12. Comparison between the accuracy of different methods.](image-url)
strength of steel-RC slabs.

The utilization of existing equations from various design codes and predictive models is precluded for the estimation of punching shear strength in FRC-GFRP slabs due to their tendency to yield excessively conservative outcomes. For a better estimation of punching shear strength in FRC-GFRP slabs, it may be necessary to modify the existing models to consider the influence of synthetic fibers on punching shear capacity to account for the increase in capacity of the FRC material compared to plain concrete. Having considered this contribution, the models can be adjusted to account for the enhanced performance provided by the synthetic fibers in GFRP-RC slabs when estimating their punching shear strength.

5. Conclusions

This study presents an experimental study on the punching shear behavior of two-way slabs reinforced with GFRP and steel rebar with the incorporation of synthetic fibers. A total of 11 slabs were tested with different types and ratios of reinforcing materials, as well as fiber types. The results from this study were also compared with predictions according to different standards and models. In light of the findings presented in this investigation, the following conclusions are derived:

1. The crack behavior of two-way slabs is governed by the type of fibers used, which led to controlling the crack width compared to that without fibers. Specifically, with the use of PVA fibers, the cracking load was increased, and the number and width of cracks were significantly controlled compared to other types of fibers.

2. The inclusion of PVA fibers significantly improved the punching shear capacity of two-way slabs by 14% for steel-RC slabs and by 51% and 44% for GFRP-RC slabs with reinforcement ratios equal to 1.0% and 1.5%, respectively, while the PP fibers were found to be less effective.

3. GFRP-RC slabs exhibited less stiffness and more deformation with wider cracks compared to identical steel-RC slabs with similar reinforcement ratios. This was associated with a reduction of the capacity of GFRP-RC slabs by 43% and 27% for slabs without fibers and with PP fibers, respectively, while PVA fibers limited this reduction to 7%.

4. Increasing the GFRP reinforcement ratio from 1.0% to 1.5% controlled the widths and distribution of cracks as well as improved the punching shear capacity by 35%, 28%, 19%, and 9% for the case with no fibers, with PVA fibers, with PPS fibers, and with PPL fibers, respectively.

5. The punching shear capacity for steel-RC slabs with a reinforcement ratio of 0.9% was approximately obtained with a GFRP reinforcement ratio of 1.5% for slabs without fibers and with PPS fibers, whereas GFRP-RC slabs with PVA fibers showed a higher capacity by 22% compared to their counterparts with steel reinforcement.

6. A comparison of the test results with the predicted punching shear capacity according to different standards and models showed that the test specimens achieved greater capacity. Matthys and Taeer model gave the best prediction due to better consideration of the axial stiffness of GFPP and flexural reinforcement ratio. However, the ACI 440.11-22 and ACI 440.1R-15 standards yielded the most conservative results.

As reported in the paper, the use of discrete fibers in GFRP-RC slabs offers significant advantages in terms of crack control, ductility, and load-carrying capacity, ultimately enhancing the structural performance and serviceability of the slab under punching shear conditions. The contribution of fibers showed a significant improvement to the less effective punching shear resistance mechanism for GFRP-RC slabs. Such improvements enable the wide application of this non-corrosive reinforcement alternative. Further studies are needed to better understand the effect of fiber type, dosage, and other parameters on the punching shear performance of slabs.

CRediT authorship contribution statement


Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Data availability

Data will be made available on request.

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