REPLACEMENT OF CONVENTIONAL STEEL STIRRUPS BY INTERNAL REINFORCING CFRP GRIDS IN SHEAR OF CONCRETE BEAMS

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This study proposes an innovative idea of shear reinforcement in concrete beam by replacing the conventional steel stirrups with CFRP grids at the shear span inside concrete cover. The compatibility between the internal grids and the surrounded concrete was investigated first in the elementary test. Flexural performances of two concrete elements reinforced by different spacing of internal grids were compared and discussed. After that, a concrete beam test was conducted with a total of eight RC beams varied in the number of grid strips in the shear span and the spacing of reinforcing grids. The experimental results showed that the shear capacity and the shear carried by internal CFRP grids significantly improved with a higher amount of grid strips; however, the increasing ratio of shear capacity was not proportional to the shear reinforcement ratio. Moreover, the location of internal grid in the shear span was an important part of the shear-resisting effectiveness in this study since the more number of grid strips located near the center of the shear span was much more effective to resist the propagation of diagonal shear crack than the grids at the edge. Finally, a model for evaluating shear carried by internal CFRP grids was introduced. Validity of the proposed model was explained by comparing with the experimental results and also other sets of experimental results.

Key Words: CFRP grid, internal shear reinforcement, shear capacity, shear carried by FRPs

1. INTRODUCTION

Carbon Fiber Reinforced Polymer (CFRP) is one of the most well-known materials in concrete engineering nowadays due to its many benefits, such as high strength, high durability, ease of application, and aesthetic appearance. Various types of CFRP, such as sheets, strands, rods and plates, have been produced based on purposes of use (types and shapes of structure, required strength, application method) and cost effectiveness. At the moment, one of the popular techniques of concrete beams with CFRP is to use CFRPs as a strengthening material in the shape of CFRP sheets. In this technique, CFRP sheets are attached to the surface of existing concrete members, and the third component like epoxy resin is then applied to make a good bond between CFRP sheets and the concrete. A great property of carbon fibers in term of high tensile strength is really effective to enhance structural performance of concrete beams both in flexure and shear¹, ². However, this technique still has some drawbacks due to the weaknesses of epoxy resin, including low fire resistance, degradation under UV light, and low compatibility with the substrate material³. Such problems can reduce the ability of epoxy resin to transfer stress to the
concrete, leading to an unexpected damage to the members. Additionally, the epoxy can be a potential hazard to workers during the application and the toxic fumes can be released in a fire event\(^4\).

To avoid using epoxy resin, a new method called Textile Reinforced Concrete (TRC) has been developed. This system comprises a cementitious matrix and high-performance structural fibers such as alkali-resistance (AR) glass fibers or carbon fibers in the form of textile fabric\(^5\). Remarkable improvement in structural performance of RC beams due to the TRC system has been verified by many researchers\(^5\)-\(^8\). In addition, this system shows some interesting profits due to the textile shape. According to Brückner et al.\(^9\), the single fibers in the textile can be positioned in almost any direction and nearly perfectly adopted to the orientation of the applied load. Moreover, since the diameter of the textile reinforcement is normally one or two times lower than the necessary diameter of steel bars, it is possible to develop very thin concrete elements.

In order to develop a new challenge of CFRP for concrete beams while retaining the advantages of textile reinforcements, this study presents a new alternative shear improvement by using another type of textile reinforcement such as CFRP grid as a replacement for conventional steel stirrups at the shear span inside concrete beams. With this new method, the defects of the application of CFRP due to the fire and UV light degradation can be eliminated since the grids are covered by concrete from the beginning without additional cementitious matrix or epoxy resin. Moreover, the grid type of CFRP can provide a better anchorage owing to its lattice points, compared with the fabric type of TRC system, which requires an overlap area of the fabric to ensure a good anchorage during an installation.

The main objective of this study is to investigate the shear characteristic of concrete beams reinforced with internal CFRP grids. The contents of this paper can be classified into two parts. The compatibility between the reinforcing grids and concrete was studied first in the elementary test. After that, eight concrete beams with and without CFRP grids in the shear span were tested. The parameters included the number of CFRP grid strips in the shear span and the spacing of grids which varied between 50 mm and 100 mm. The experimental results in terms of shear capacities, load-displacement relationships, crack patterns, and shear-resisting mechanism were presented and discussed. Lastly, a model for evaluating the shear carried by internal CFRP grids in concrete beams was introduced. The validity of the proposed model was considered by comparing the calculated results with experimental results and also with a set of experimental data from Kim et al.\(^{10}\).

2. ELEMENTARY TEST

(1) Outline of elementary test
Before heading to the shear test of CFRP grid-reinforced concrete beams, it is important to guarantee at the beginning that the bond between the selected reinforcing CFRP grids and the designed concrete is good enough to resist the force effectively, without a slip of internal grid from the surrounded concrete. In order to do that, a bending test of concrete elements with the same type of CFRP grids as used in the shear test has been conducted, instead of testing by a direct pull-out test.

The actual tensile strength of the reinforcing grids obtained from the bending test in this elementary test will be compared with their ultimate strength taken from the uni-axial test. This comparison is to estimate the performance of CFRP grids when they are used as reinforcing materials inside concrete members.

(2) Specimen details in elementary test
Two concrete elements reinforced with internal CFRP grids in flexure were prepared in this test. The two specimens have a total length (\(l\)) of 1100 mm, width (\(b\)) of 250 mm, and height (\(h\)) of 100 mm. The shear span-to-effective-depth ratio (\(a/d\)) is 4.8 and both of them are designed to fail in flexure as shown in Fig. 1.

The difference between these two concrete specimens is the amount of internal reinforcing grids. One specimen, named C100, was reinforced by two strips of 100 mm-spacing CFRP grid and another specimen, named C50, was reinforced by four strips of 50
mm-spacing CFRP grid as shown in Fig. 2. Hence, the flexural capacity of C50 was expected to be about double of that of C100.

(3) Materials used in elementary test

a) Concrete

The self-compacting concrete (SCC) was selected in this study because a high workability and consistency of matrix are guaranteed so that the fresh concrete can be fully penetrated into the grid without segregation, ensuring the good bond between fiber and matrix 11). As shown in Table 1, the mix proportions consists of high-early strength cement, limestone powder, fine aggregates, coarse aggregates, viscosity improver and superplasticizer (high-performance air entrained water-reducing agent). The concrete is designed with an average seven-day age strength of 35 N/mm².

b) CFRP grids

The two orthogonal CFRP grids have the same nominal thickness for one strip of 2.64 mm and width for one strip of 5 mm. Thus, the cross-sectional area for one strip of the grids is about 13.2 mm² both in 100 mm-spacing grid and 50 mm-spacing grid as can be seen in Fig. 3. According to JSCE-E 531-2010 12), the uni-axial test for tensile properties of the CFRP grid used in the elementary test was conducted as shown in Fig. 4. The results from the uni-axial tensile test of five samples for each case showed that the average ultimate tensile strength and Young’s modulus of the reinforcing CFRP grids were 1934 N/mm² and 110 kN/mm² in the case of the 100 mm-spacing grid, and 1914 N/mm² and 118 kN/mm² in the case of the 50 mm-spacing grid. In addition, the rupture strain of both two-sized grids varied between 1.43% to 1.72%.

(4) Loading method and measurements

The failure location of specimens was designed to happen in a uniform bending region; thus, the four-point bending test with spacing between the two loading point of 200 mm was performed.

Measurement items included displacement transducers, electrical strain gauges, and π-gauges. Four displacement transducers were set up at the mid-span and supports of specimens. The 2 mm gauge length strain gauges were attached on the internal CFRP grids and the 30 mm gauge length types were attached at the top surface of concrete. All the gauges were attached at the mid-span. The 50 mm π-gauges were attached at the bottom of the specimens to check the starting of flexural crack. Moreover, the cracks on the side surface were recorded by digital cameras during the test.

(5) Test results of elementary test

a) Load–displacement relationships, flexural capacities and failure modes

The relationships of the applied load and displacement for C100 and C50 are presented in Fig. 5. The first crack of C100 and C50 was observed when the load reached 6.2 kN and 9.1 kN, respectively. Then, the load dropped a bit and increased again due to the internal reinforcing grids. As a smaller number of grid strips in C100, a wider first crack was obtained when compared with the first crack of C50 and that led to a larger reduction of load after the first drop as shown in the green line in Figs. 5 and 6.

Table 1 Mix proportions of self-compacting concrete.

<table>
<thead>
<tr>
<th>Maximum size of coarse aggregate (mm)</th>
<th>Water cement ratio (%)</th>
<th>Sand aggregate ratio (%)</th>
<th>Unit weight (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Water</td>
<td>Cement</td>
<td>Limestone powder</td>
</tr>
<tr>
<td>15</td>
<td>60</td>
<td>175</td>
<td>292</td>
</tr>
<tr>
<td>50</td>
<td>45</td>
<td>292</td>
<td>249</td>
</tr>
</tbody>
</table>
Some new cracks were generated after the first crack, causing a swing of the curves in both C100 and C50. Note that an extension of the internal grids also started after the first crack at the same load level as can be seen in Fig. 6. After that, the load gradually increased with a propagation of flexural cracks at the mid-span of the specimens until the peak load. The flexure failure due to the total rupture of the grid was evidenced in C100 when the load reached 15.7 kN, leading to a collapse of the specimen as shown in Fig. 7.

A good reinforcing efficiency of the internal grid was obtained in C50 where even the total rupture of the grid was not found. This was confirmed from the load-strain curve of C50 in Fig. 6 which exhibited that the strain of the grid had already reached its limit (1.43% to 1.72%). The peak load of C50 was 33.0 kN, which was about double that of C100 as expected, and the failure mode was considered as shear failure since a severe shear crack was observed at the peak as seen in Fig. 7.

b) Actual tensile strength of internal CFRP grids

The mechanical properties of concrete in both C100 and C50 were identical in which the compressive strength ($f_{c}'$) was 43.0 N/mm², the tensile strength ($f_t$) was 2.63 N/mm², and Young’s modulus ($E_c$) was 32.4 kN/mm². To calculate the actual tensile strength ($f_p$) of the internal CFRP grids from the elementary test, Eq. (1) was considered.

$$f_p = \frac{M}{A_f \cdot z}$$  \hspace{1cm} (1)

where $M$ is the bending moment at the peak load; $A_f$ is the total cross-sectional area of CFRP grids in the specimens (26.4 mm² in C100 and 52.8 mm² in C50); and $z$ is the moment arm length which can be defined as explained in Fig. 8.

The results of the actual tensile strength from Eq. (1) was 1457 N/mm² in C100 and 1560 N/mm² in C50. The comparison of the tensile strength of CFRP grids from the bending test and its maximum values from the uni-axial test (1934 N/mm² in C100 and 1914 N/mm² in C50) revealed that the difference was 24.7% in C100 and 18.5% in C50. Even though the calculation from Eq. (1) was based on the assumption
that the internal grids had reached their maximum tensile strength, it was remarkable to note that the internal grid in C50 was still considered by this equation even if the failure mode was shear failure. Although a rupture of the grids did not occur in C50, the tensile strength of CFRP grid was very close to its maximum at the time the specimen had failed. This was evidenced by the strain of CFRP grid which was very close to its ultimate, and by the peak load of C50 which was about two times that of C100 as mentioned before. This corresponded to the amount of CFRP grid in C50 which was two times that of C100. In addition, the 18.5% difference in the case of C50 was also lower than that of 24.7% in the case of C100.

The deviation of the tensile strength was considered to have happened mainly because of the localized failure. In the bending test, the failure of CFRP grids occurred in only one fracture section; however, a dispersed failure was found in all tested grids in the uni-axial test as seen in Fig. 9. In summary, the results from this elementary test demonstrates that the proposed internal grid system is practically productive to enhance the ability of concrete elements and there is no bond problem between the selected CFRP grids and the surrounding concrete. Therefore, this internal reinforcing system by the same type of CFRP grids can reasonably apply to the shear test of CFRP grid-reinforced concrete beams in further studies.

3. TEST PROGRAMS OF SPECIMENS IN CONCRETE BEAM TEST

(1) Specimen details and cases
A total of eight RC beams were prepared and tested in this study. One beam was the control beam (CON) in which no reinforcement was provided in the shear span of this specimen. The other seven RC beams were reinforced in shear in different numbers and spacing of internal grids as shown in Fig. 10. The reinforced specimens were divided into two groups according to the spacing of CFRP grids in the shear span. Specimens were named according to spacing of CFRP grid and number of CFRP grid strips. In Group I, the beams were reinforced with the CFRP grid with spacing of 100 mm and the number of grid strips was increased proportionally from three strips in C100-3 to five and seven strips in C100-5 and C100-7, respectively. Similarly, the number of CFRP grid strips of specimens in Group II were varied from two strips in C50-2 to six, ten, and fourteen strips in C50-6, C50-10, and C50-14, respectively. However, the spacing of CFRP grid in Group II was changed to 50 mm.

Deformed high-strength steel bars with the reinforcement ratio of 3.04% were provided as tensile reinforcements in all tested beams to prevent flexure failure and a number of steel stirrups were fully arranged in the untested shear span to control the failure side of the specimens as seen in Fig. 10. All specimens have a total length \( l \) of 1800 mm, width \( b \) of 200 mm, height \( h \) of 300 mm, length of shear span \( a \) of 700 mm and shear span effective depth ratio \( a/d \) of 2.8. In addition, specimens in this study were designed to fail in shear in all cases.

(2) Materials used in concrete beam test
The mix proportion of concrete and the reinforcing CFRP grids used in this RC beam test had the same properties as used in the elementary test. The designed compressive strength of seven-day curing age of concrete was also 35 N/mm². The reinforcing grids were attached to the steel reinforcements in advance by using the binding wires as commonly used in steel stirrups. Since the spacing of reinforcing grids had already been set, it became very comfortable for the grid attachment.

Three deformed high-strength reinforcing bars with 25.4-mm nominal diameter \( (A_s = 507.7 \text{ mm}^2) \) and 1187 N/mm² yield strength were arranged as tensile reinforcing bars. Two deformed bars with 9.53-mm nominal diameter \( (A_s = 71.3 \text{ mm}^2) \) and 345 N/mm² yield strength were provided as compression bars. In the untested shear span, 12.7-mm nominal diameter \( (A_s = 126.7 \text{ mm}^2) \) with yield strength of 405 N/mm² deformed bars were arranged with spacing of 100 mm. In addition, round bars with 6-mm diameter \( (A_s = 28.3 \text{ mm}^2) \) and yield strength of 309 N/mm² were provided to prevent failure at the point load in all cases as displayed in Fig. 10.
(3) Loading method

Figure 11 shows the environment during the loading test of C50-6. A four-point load was set up with the load generated from a 2000-kN loading machine. To ensure a sufficient anchorage of tensile bars, the anchorage plates and nuts were equipped at the end of the bars as seen in Fig. 11. Teflon sheets with grease were placed on the roller supports to prevent the friction in horizontal direction and a load distribution beam was placed below the applied loading point to reduce the stress concentration.

(4) Measurements

The displacement of specimens was measured by attaching four displacement transducers at the mid-span and supports. Three 50 mm \( \pi \)-gauges were attached to the bottom of specimens to investigate the starting point of flexure crack. Electrical strain gauges with 2 mm gauge length were attached to all internal CFRP grids, tensile bars, and also compression bars at the location as shown in Fig. 10. Strain gauges with 30 mm gauge length were attached on a top fiber at the mid-span to measure the strain of concrete at the peak load. In addition, digital cameras were set to record the propagation of cracks on the side surface throughout the loading test.

4. TEST RESULTS AND DISCUSSION IN CONCRETE BEAM TEST

(1) Shear capacity

a) Evaluation method of shear carried by internal CFRP grid from the experiments

In the conventional way to determine the shear capacity of normal RC beams, a total shear capacity can be separated into two components, namely the shear carried by concrete \( V_c \) and the shear carried by steel stirrups \( V_s \). The term \( V_c \) has actually included the effects of the shear in the compression zone, the vertical component of the aggregate interlock, and the dowel action in its calculation already\(^{13}\).

Similarly, the total shear capacity of the reinforced beams observed from the experiment \( V_{\text{exp}} \) in this study is also a summation of the shear carried by concrete \( V_c \) and the shear carried by internal CFRP grids \( V_g \) as expressed in Eq. (2). The \( V_c \) of each reinforced specimens can be referred from \( V_c \) of the control beam \( V_{c,\text{con}} \). However, it is readjusted as
shown in Eq. (3) because of the inequality of compressive strength of concrete between the control specimen \(f'_{c,\text{con}}\) and the others. That inequality is proportional to the third root of the compressive strength of concrete according to the equation of RC beams without shear reinforcement proposed by Niwa et al.\(^{14}\).

\[
V_{\text{exp}} = V_{c} + V_{g} 
\]  
\[
V_{c} = f'_{c,\text{con}} \left(\frac{f_{\text{c,exp}}}{f'_{c,\text{con}}}\right)^{1/3} 
\]  

As a consequence, the evaluation method of shear carried by internal CFRP grids from the experiments \(V_{\text{gexp}}\) is presented as follows:

\[
V_{\text{gexp}} = V_{\text{exp}} - V_{c} 
\]  

b) Shear capacity and shear carried by internal CFRP grid from the experiments

Table 2 lists the summary of experimental results and Fig. 12 clarifies the augmentation of shear capacity due to the internal CFRP grids. The values of \(V_{c}\) in each specimens was slightly different since the variation of compressive strength of concrete in each specimen was not far from that of the CON beam.

It can be said that the total shear capacity and shear carried by internal CFRP grids in each group significantly increased with an addition of the number of CFRP grid strips in the shear span; however, such increment of capacities was not proportional to the amount of the reinforcing grid strips. In Group I, the total shear capacities of C100-3, C100-5, and C100-7 were higher than that of the CON beam by 49.5%, 76.7%, and 79.7%, respectively. Besides, the shear carried by internal CFRP grid increased by 59.5% in C100-5 and 66.2% in C100-7 when compared with \(V_{\text{gexp}}\) of C100-3.

Similarly, the total shear capacities of C50-2, C50-6, C50-10, and C50-14 in Group II were higher than that of the CON beam by 71.7%, 97.6%, 109.5%, and 114.7%, respectively. Comparing the increase in shear carried by internal CFRP grid in this group showed that the \(V_{\text{gexp}}\) of C50-6, C50-10, and C50-14 was higher than that of C50-2 by 40.8%, 56.7%, and 61.1%, respectively.

The results of shear capacities implied that the effectiveness of concrete beam reinforced with internal CFRP grids tended to be small when a greater amount of reinforcing grids was provided. This fact was obviously found when comparing the \(V_{\text{gexp}}\) of C100-5 and C100-7 because the difference in \(V_{\text{gexp}}\) was only 4.2% even when the number of grid strips in C100-5 was lower than that of C100-7 by about 40%. Again, the difference in \(V_{\text{gexp}}\) of C50-10 and C50-14 was only 2.8%, while the difference of number of grid strips was also 40%.

Furthermore, the comparison of shear carried by CFRP grid between the 50-mm spacing grid and the 100-mm spacing grid revealed that a higher shear-reinforcing efficiency of internal grids was obtained from specimens with a higher number of grid strips near the center of shear span. From Fig. 12, it was evident that the capacities of C50-2 was significantly larger than those of C100-3 even when the number of grid strips in shear span was smaller and this was confirmed again when comparing the capacities between C50-6 and C100-7. The reason for this phenomenon will be explained again in the next section of this paper.

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**Table 2 Summary of concrete beam test results.**

<table>
<thead>
<tr>
<th>Specimen designation</th>
<th>Mechanical properties of concrete</th>
<th>(V_{\text{exp}}) (kN)</th>
<th>(V_{c}) (kN)</th>
<th>(V_{\text{gexp}}) (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CON</td>
<td>(41.2)</td>
<td>(30.0)</td>
<td>(77.8)</td>
<td>(77.8)</td>
</tr>
<tr>
<td>C100-3</td>
<td>(40.5)</td>
<td>(29.7)</td>
<td>(116.3)</td>
<td>(77.5)</td>
</tr>
<tr>
<td>C100-5</td>
<td>(37.6)</td>
<td>(30.0)</td>
<td>(137.5)</td>
<td>(75.5)</td>
</tr>
<tr>
<td>C100-7</td>
<td>(37.2)</td>
<td>(28.7)</td>
<td>(139.8)</td>
<td>(75.3)</td>
</tr>
<tr>
<td>C50-2</td>
<td>(39.5)</td>
<td>(26.1)</td>
<td>(133.6)</td>
<td>(76.8)</td>
</tr>
<tr>
<td>C50-6</td>
<td>(35.0)</td>
<td>(30.6)</td>
<td>(153.7)</td>
<td>(73.7)</td>
</tr>
<tr>
<td>C50-10</td>
<td>(35.3)</td>
<td>(29.8)</td>
<td>(163.0)</td>
<td>(73.9)</td>
</tr>
<tr>
<td>C50-14</td>
<td>(37.5)</td>
<td>(29.5)</td>
<td>(167.0)</td>
<td>(75.5)</td>
</tr>
</tbody>
</table>

\(f'_{c}\) = compressive strength, \(E_{c}\) = Young’s modulus, \(V_{\text{exp}}\) = total shear capacity from experiment, \(V_{c}\) = shear carried by concrete, \(V_{\text{gexp}}\) = shear carried by internal CFRP grids from experiment.
(2) Load-displacement relationships and crack patterns

The relationships between the applied load and displacement of specimens in Groups I and II are presented in Figs. 13 and 14. The main diagonal crack (red line) and crack patterns of all specimens are illustrated in Fig. 15. Besides, the location of the internal reinforcing grids is also indicated by the blue dashed line in the shear span of all reinforced specimens in this figure.

The diagrams in Figs. 13 and 14 show that the shapes of curves for all specimens except that of the CON are basically similar. The first flexural crack of all specimens initiates when the load level is about 25-40 kN. Then, the loads increase steadily until they reach 150-170 kN. At this level, the initiation of diagonal shear crack is observed around the middle height of the shear span and this leads to the peak load of the control beam at the load level of 155.7 kN. The failure mode of the CON beam is the diagonal tension failure, which is a typical failure type of concrete beams without shear reinforcement as shown in Fig. 15(a).

In the case of the beams reinforced with CFRP grids, the loads still increase after the appearance of the diagonal shear crack due to the internal CFRP grids. However, a reduction in stiffness can be seen in Figs. 13 and 14. After that, the loads gradually decrease.
develop with a small extension in displacement and the crack width of the main shear crack becomes clearer and bigger in this stage.

Finally, the loads of all specimens reach their peaks when the main diagonal shear crack extends to connect between the loading point and supports. The compressive strains of concrete from the gauges at the top fiber get to their limiting strain and crushing of concrete is clearly observed in some specimens at the location near the loading point as the examples in Figs. 15(b), 15(e), and 15(g). Therefore, the failure mode of all reinforced specimens is considered the shear compression failure. In addition, the longitudinal reinforcing bars and the steel stirrups in untested shear span do not yield in all cases since the actual tensile strain of the bars obtained from the strain gauges do not reach their yielding strain.

(3) Shear resisting mechanism in concrete beam test

The location of strain gauges according to the number is illustrated in Fig. 16. The relationships between the applied load and actual tensile strain of each strip of internal CFRP grid from C100-5 and C50-10 are presented in Figs. 17 and 18. Since the shear-resisting mechanisms of all reinforced specimens are quite similar, the graph of C100-5 is considered as a representative for the beams with 100-mm spacing grid and the graph of C50-10 is considered as a representative for the case of the 50-mm spacing grid.

At the beginning state of the loading, all beams behave in the linear manner prior to the occurrence of flexural crack and followed by a diagonal shear crack. After that, the internal CFRP grids start to resist the shear load. The evidence of this action can be described by the first inclination of the graphs in Figs. 17 and 18 at the load level around 150-170 kN, which correspond to the peak load of the control beam. This assumption is similar to the utilization of steel stirrups in the conventional RC beams.
The propagation of the main diagonal shear crack in the next stage is clearly observed in Figs. 17 and 18 where the reinforcing grids in the shear span do not equally resist the shear force. While the grids located at about the center of shear span are hugely elongated, the grids located near the supports are not elongated significantly. Because of this, the shear carried by the internal CFRP grid does not increase significantly even if the number of grid strips is increased as found in the comparison of $V_{gexp}$ between C100-5 and C100-7, or C50-10 and C50-14.

Since the initiation of shear crack of RC beams having shear span effective depth ratio ($a/d$) more than 2.5 usually occurs at about the center of a shear span, the more number of grid strips at the center of a shear span, the more effective it is in resisting a widening and growing of the shear crack. This is the reason why the shear-resisting performance of specimens with the 50-mm spacing grid is better than with the 100-mm spacing grid. $V_{gexp}$ of C50-2 becomes close to $V_{gexp}$ of C100-5 because the two grid strips of C50-2 are extremely elongated and the actual tensile strain obtained from the gauges located near the center of the shear span (position 3 in C100 and position 5 or 6 in C50 according to Fig. 16) from C50-2 shows the maximum value among all specimens in this study with the tensile strain of $9684 \times 10^{-6}$ (about 60% of the ultimate strain of the grids) as shown in Fig. 19. From this point of view, the location of internal CFRP grid in the shear span is very important in concrete beams reinforced with internal CFRP grids. The influence of the location of reinforcement on the shear capacity of concrete beams corresponds to the results from the previous study done by the authors in which the strengthening mesh that is not used at the center of the shear span exhibits a much lower performance than the mesh located at the center despite the same amount of mesh provided.

Although the internal CFRP grids are able to enhance the shear capacity of concrete beams by resisting the propagation of the diagonal shear crack, they cannot yield like conventional stirrups. At the critical point, the grids no longer restrained an expansion of the crack since it became very big (more than 1 mm) and very easy to be seen by the naked eye. The loads of reinforced specimens reached its peak when the main diagonal shear crack extended to the loading point, or the concrete in compression zone above the diagonal crack was crushed.

Even if CFRP grids are commonly known as brittle materials, it is important to note that a total rupture of the reinforcing grids and a sudden collapse of the beams are not observed in all specimens during and after the test including C50-2 in which a very small number of grids is provided. This is confirmed again by opening concrete cover and investigating the inside grids after the loading test as the example in Fig. 20. One of the possibilities to describe this incident is the deficiency of the grid’s nodes in the vertical direction. According to Dutta et al.\(^\text{16}\), the force transfer in a composite grid is concentrated at the intersecting grid lines (nodes) while the surface on the grid ribs is essentially no load transfer. Therefore, the number of nodes between the upper and lower longitudinal bars in C50-2 might not be adequate to provide a deserving anchorage, which could lead to a rupture of the reinforcing grid. However, since the explanation here is only a supposition, further studies about the bond between concrete and FRP grids are needed since the information is now still limited.

5. MODELING OF SHEAR CARRIED BY INTERNAL CFRP GRIDS

(1) Calculation method of shear carried by internal CFRP grid

In order to model the equation for evaluating shear carried by internal CFRP grid based on the experi-
mental results of specimens in this study, the reinforcing ability of each grid strip in the shear span is considered one by one. The distributions of shear load on each internal grid strips at the peak load are shown in Fig. 21. It can be seen from the figure that the grid strips located near the center of the shear span are elongated the most as mentioned before in Section 4(3). In order to model the equation, the distributions from Fig. 21 are rearranged as shown in Fig. 22. Based on the rearrangement, the shear carried by internal grid can be expressed as shown in Eq. (5).

\[
V_{\text{gcal}} = \sum_{i=1}^{N_g} \left\{ \frac{1}{N_g^{0.6}} \cdot \left( 1 - \frac{x_i}{z \cdot \cot \alpha / 2} \right) \right\} \cdot A_f \cdot f_{fu} \tag{5}
\]

where \(V_{gcal}\) is the calculated value of shear carried by internal CFRP grids (kN); \(N_g\) is the number of internal grid strips; \(x_i\) is the distance from the center of shear span to the concerned grid strip (mm); \(z\) is the distance between the horizontal compression and tension members (mm) \([z = (7/8)d]\); \(\alpha\) is the angle of shear load distribution on internal grid for each \(N_g\) \([\alpha = 43N_g^{0.5}]\); \(A_f\) is the total cross-sectional area of CFRP grid for one strip (mm\(^2\)) \([A_f = 2 \cdot t_f \cdot l_f]\) where \(t_f\) and \(l_f\) are thickness and width for one strip of the grid; and \(f_{fu}\) is the ultimate tensile strength of CFRP grid (N/mm\(^2\)).

From Eq. (5), it is explained that the reinforcing efficiency of the internal grid strip located at the center of shear span \((x_i = 0)\) is the highest one and the efficiency becomes lower as \(x_i\) with linear relationship increases as shown in Fig. 22. Finally, the efficiency is equal to zero when \(x_i\) is extended to be as large as \(z \cdot \cot \alpha / 2\).

To specify the magnitude of the triangles for each specimen in Fig. 22, the relationships between the total number of grid strip \((N_g)\) and the strain proportion at the center of shear span in each case are plotted as shown in Fig. 23. Besides, the relationships between the total number of grid strip \((N_g)\) and the angle of shear load distribution \((\alpha)\) are also plotted as shown in Fig. 24. By using the least-squares procedure, power regression models are obtained as shown in the first term of Eq. (5) and also shown in the calculation of \(\alpha\).

The new proposed model for shear carried by internal CFRP grid is based on the assumption that the anchorage between the internal grid and the surrounded concrete is absolutely sufficient. Therefore, the strain proportion at the center of shear span in Fig. 23 can be equal to one when there is only one grid strip in the shear span \((N_g = 1)\). The power regressions obtained from Figs. 23 and 24 demonstrate...
the reasonable tendencies that the strain proportion at the center of shear span and the angle of distribution should be close to zero when the number of grid strips is significantly increased. However, the proposed model at this stage limited the number of grid strips from 2 to 14 strips since it is developed only from the experimental results in this study.

(2) Comparison of experimental values and calculated values

The summary of the shear carried by internal CFRP grids from the experiments as derived by Eq. (4) and from the proposed model as calculated by Eq. (5) is presented in Table 3. As a comparison between the experimental values and the calculated values, a moderate accuracy is obtained with the average (avg.) of 0.95 and the coefficient of variation (C.V.) of 13.3% as shown in Fig. 25. However, the experimental value of C100-3 seems to be comparatively low when compared with the calculation from Eq. (5).

This deviation is considered to have happened due to an insufficient node points in the C100-3 specimen. As mentioned before, the force transfer in a grid is concentrated mainly at the nodes. From that point, the horizontal components of internal grid does not directly resist the shear load; however, they provide the node points that are necessary for the anchorage between the FRP grids and concrete. For this reason, the only three nodes in the vertical direction of C100-3 may not be sufficient to generate shear capacity to be at the level as it should be resulting in the 30% reduction of shear capacity compared with the calculation from the proposed model.

Moreover, the insufficient node point in C100-3 also causes a different load-displacement curve when compared with other reinforced specimens as can be seen in Fig. 13. For C100-3, the diagonal crack propagated to the loading point very fast since the anchorage between the grid and concrete was quite low. However, after reaching the peak load, the concrete below the loading point was still able to carry a bit shear load. This resulted in the post peak behavior in this specimen.

(3) Validity of the proposed equation with other experimental results

Due to the many advantages offered by FRPs, the researches on FRP bars used as flexural reinforcement have been performed extensively, as well as the FRP sheets attached externally to the surface of concrete beams in terms of strengthening. However, the number of papers focusing on FRP reinforcement as a substitute for steel stirrups is still limited and the model to estimate the shear carried by internal reinforcing FRP in shear still need improvement.

One of the papers focusing on the same aspect as in this study has been done by Kim et al. In that paper, nine concrete beam specimens reinforced with the FRP plate material with opening were investigated. The parameters included the type of FRP, shape of FRP shear reinforcement, and the amount of the FRP reinforcement.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$A_f$ (mm$^2$)</th>
<th>$f_{u}$ (N/ mm$^2$)</th>
<th>$N_g$ (strips)</th>
<th>$\alpha$</th>
<th>$V_{g_{\text{Cal}}}$ (kN)</th>
<th>$V_{g_{\text{Exp}}}$ (kN)</th>
<th>$V_{g_{\text{Exp}}}/V_{g_{\text{Cal}}}$</th>
<th>$R$</th>
<th>$K_{\text{JSCE}}$</th>
<th>$V_{d_{\text{f}}}$ (kN)</th>
<th>$V_{f_{\text{Exp}}}/V_{f_{\text{Cal}}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>C100-3</td>
<td>79.2</td>
<td>1934</td>
<td>3</td>
<td>24.83</td>
<td>56.9</td>
<td>38.8</td>
<td>0.68</td>
<td>700</td>
<td>5.66</td>
<td>110</td>
<td>40.5</td>
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<tr>
<td>C100-5</td>
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<td>1934</td>
<td>5</td>
<td>19.23</td>
<td>60.0</td>
<td>61.9</td>
<td>1.03</td>
<td>700</td>
<td>9.43</td>
<td>110</td>
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<tr>
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<td>16.25</td>
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<td>64.5</td>
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<td>700</td>
<td>13.20</td>
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<td>37.2</td>
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<tr>
<td>C50-2</td>
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<td>56.8</td>
<td>0.98</td>
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<td>3.77</td>
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<tr>
<td>C50-6</td>
<td>158.4</td>
<td>1914</td>
<td>6</td>
<td>17.56</td>
<td>81.0</td>
<td>80.0</td>
<td>0.99</td>
<td>700</td>
<td>11.31</td>
<td>118</td>
<td>35.0</td>
</tr>
<tr>
<td>C50-10</td>
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<td>1914</td>
<td>10</td>
<td>13.60</td>
<td>91.8</td>
<td>89.0</td>
<td>0.97</td>
<td>700</td>
<td>18.86</td>
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<td>C50-14</td>
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<td>1914</td>
<td>14</td>
<td>11.49</td>
<td>97.8</td>
<td>91.5</td>
<td>0.93</td>
<td>700</td>
<td>26.40</td>
<td>118</td>
<td>37.5</td>
</tr>
</tbody>
</table>

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The average compressive strength measured at 28 days in that paper was 44.6 MPa. Two layers of 10 deformed steel bars with a diameter of 25 mm were used as longitudinal reinforcement with their tensile strength and modulus of elasticity of 500 MPa and 200 GPa, respectively. Aramid fiber-reinforced polymer (AFRP), CFRP, and GFRP were used in the manufacturing of the FRP shear-reinforced plates. All concrete specimens had the same size with the section of 350x420 mm, the shear span of 813 mm, the effective depth of 342 mm, and the \( a/d \) ratio of 2.4.

Table 4 shows the results of estimating the shear strength of concrete with FRP shear reinforcement \( (V_{cal}) \) using the shear carried by concrete from JSCE as shown in Eq. (6) and calculating the shear carried by FRP shear reinforcement by the proposed equation in Eq. (5). However, the parallelogram shape in that paper was not included in this comparison since Eq. (5) did not concern the effect of the inclined grid strip. Also, the number of grid strip \( (N_g) \) shown in Table 4 counted only the strips inside the shear span.

\[
V_c = 0.2\left(\frac{f'_c}{\rho_s}\right)^{\frac{1}{2}}\left(\rho_f\right)^{\frac{1}{2}}\left(\frac{1000}{d}\right)^{\frac{1}{2}}\left(0.75 + \frac{1.4}{a/d}\right) b_w d \tag{6}
\]

The comparison in Table 4 indicates that the empirical model gives fairly conservative values to the experimental data from Kim et al. with an accuracy of 1.11 on the average and the coefficient of variation \( (C.V.) \) of 6.7%. However, it is noted that the estimation of shear strength by Eq. (6) gives a higher accuracy than the equations from ACI 318-11 which have been used for calculating shear strength of specimens in that paper. By using the ACI equations, the ratio of \( V_{exp}/V_{cal} \) is more than 1.30 on the average.

(4) Comparison of the effectiveness between the proposed internal grid system and other systems for concrete beams with FRPs in shear

In this section, the shear reinforcing effectiveness of the internal grid system from this study is compared with two existing systems for concrete beams with FRPs in shear. One is the beams reinforced internally with FRP rods and another is the beams strengthened externally by FRP sheets. Both of them are now recommended in the guideline books of the Japan Society of Civil Engineers (JSCE)\(^{(19,20)}\).

As shown in Eq. (7), the shear carried by FRP rods is estimated by the 45\(^\circ\) truss analogy method similar to the calculation for conventional steel stirrups. However, the strain in FRP rods in Eq. (7) is not the ultimate strain of the material and the value has been limited as evaluated from Eq. (8).

\[
V_f = A_f \cdot E_f \cdot \varepsilon_f \cdot z / s_f \tag{7}
\]

\[
\varepsilon_f = \sqrt{\frac{h}{300}} - \left[ f'_c \cdot \frac{\rho_s \cdot E_s}{\rho_f \cdot E_f} \right] \cdot 10^{-4} \tag{8}
\]

where \( V_f \) is the FRP transverse reinforcement shear resistance (kN); \( A_f \) is the total cross-sectional area of FRP shear reinforcement (mm\(^2\)); \( \varepsilon_f \) is the strain in the FRP shear reinforcement; \( z \) is the distance between the horizontal compression and tension members (mm) \( [z = (7/8)d] \); \( s_f \) is the spacing of shear reinforcement (mm); \( h \) is the height of the member (mm); \( f'_c \) is the compressive strength of concrete (N/mm\(^2\)); \( \rho_s \) is the longitudinal steel reinforcement ratio \( [\rho_s = A_s/(b_w \cdot d)] \); \( E_s \) is the modulus of elasticity of longitudinal steel reinforcement (kN/mm\(^2\)); \( \rho_f \) is the FRP shear reinforcement ratio \( [\rho_f = A_f/(b_w \cdot s_f)] \); and \( E_f \) is the modulus of elasticity of FRP shear reinforcement (kN/mm\(^2\)).

By using Eq. (7) to evaluate the shear capacity of internal CFRP grid in this study, the results can be seen as shown in Table 3 with the average with the experimental results of 2.91. The comparison obviously indicates that the evaluation method for FRP rods is too conservative for evaluating the shear capacity of internal CFRP grids. Moreover, from this point of view, the reinforcing effectiveness of the internal FRP grid might be better than that of the internal FRP rods. Nevertheless, the efficiency of the

<table>
<thead>
<tr>
<th>Specimen</th>
<th>( t_f ) (mm)</th>
<th>( t_l ) (mm)</th>
<th>( d_f ) (mm(^3))</th>
<th>( f'_c ) (N/mm(^2))</th>
<th>( f'_f ) (N/mm(^2))</th>
<th>( \rho_f )</th>
<th>( V_c ) (kN)</th>
<th>( N_g ) (strips)</th>
<th>( z ) (mm)</th>
<th>( \alpha )</th>
<th>( V_{exp} ) (kN)</th>
<th>( V_{cal} ) (kN)</th>
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<tr>
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<td>259.3</td>
<td>285.4</td>
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<td>480</td>
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<td>259.3</td>
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<td>0.0423</td>
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<td>3</td>
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<td>24.83</td>
<td>267.9</td>
<td>300.0</td>
<td>1.12</td>
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<tr>
<td>CB-2</td>
<td>1.0</td>
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<td>24.83</td>
<td>267.5</td>
<td>339.6</td>
<td>1.27</td>
</tr>
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</table>
internal grid system might be inferior to the internal FRP rod system when considering the confining effect. Unlike the flat shape of FRP grids in this study, conventional steel stirrups and internal FRP rods have always been bent to enclose concrete core resulting in a better confining effect in a seismic event. To solve this, 3-D cages of FRP grid that are analogous to the steel reinforcement cages may be a good choice for the development of concrete beams reinforced with FRP grids in shear. This type of material is produced by the FRP grid called as NEFMAC in the market\(^2\).

Apart from the internal FRP reinforcement, the effectiveness of the internal grid system is also compared with the externally bonded FRP sheet system which is well-known for retrofitting deteriorated concrete structures. JSCE has proposed the equation for estimating the shear capacity of the rated concrete structures. JSCE has proposed the system which is well-known for retrofitting deteriorated concrete structures. JSCE has proposed the equation for estimating the shear capacity of the rated concrete structures. JSCE has proposed the system which is well-known for retrofitting deteriorated concrete structures. JSCE has proposed the equation for estimating the shear capacity of the rated concrete structures.

\[ K_{JSCE} = 1.68 - 0.67R \]  
\[ R = \left( \rho_f \cdot E_f \right)^{1/3} \left( \frac{f_{fu}}{f''c} \right)^{1/3} ; 0.5 \leq R \leq 2.0 \]  

where \( K_{JSCE} \) is the shear reinforcing efficiency of continuous fiber sheets; \( A_f \) is the total cross-sectional area of continuous fiber sheets in space \( s_f \) (mm\(^2\)); \( s_f \) is the spacing of continuous fiber sheet (mm); \( f_{fu} \) is the tensile strength of continuous fiber sheet (N/mm\(^2\)); \( E_f \) is the modulus of elasticity of continuous fiber sheet (kN/mm\(^2\)); \( f''c \) is the compressive strength of concrete (N/mm\(^2\)); \( \alpha_f \) is the angle formed by continuous fiber sheet about the member axis; and \( z \) is the distance between the horizontal compression and tension members (mm) \([z = (7/8)\alpha_f] \).

It can be seen from Eq. (9) that the composition of equation from JSCE is also not far from the 45º truss analogy method. However, instead of limiting the strain value of FRP, the ultimate tensile strength of FRP sheet has been multiplied by the reinforcing efficiency \( K_{JSCE} \) which can be assessed by Eq. (10). It is assumed that the same size of RC beams is externally bonded by CFRP sheets with the same reinforcement ratio, the same tensile property of FRP and the same compressive strength of concrete as the experiments in this study. Thus, the \( K_{JSCE} \) from Eq. (10) and the shear carried by FRP from Eq. (9) can be obtained as presented in Table 3.

The comparison implies that the reinforcing efficiency of the internal grid seems to be much smaller than that of the external FRP sheet when a number of FRPs is provided in the shear span as found in C100-7 or C50-14. However, the efficiency from the two systems has a little difference when RC beams are reinforced with a limited amount of FRPs near the center of the shear span as observed in C100-5 and C50-6. Moreover, the internal system becomes much more effective than the external system in C50-2 where the specimen is reinforced by a very small amount of FRPs at the center of the shear span.

However, it should be noted that both of the equations recommended by JSCE are based on the assumption that the reinforcing/strengthening FRPs have been provided for the whole shear span of concrete beams and the shear capacity obtained from each FRP strip is assumed to be equaled. The influence of the location of FRP in the shear span as studied in this paper has not been concerned in their equations yet. Hence, it is quite unfair to compare the effectiveness of each FRP from each system except those FRPs in specimens C100-7 and C50-14 in which the reinforcing grids have been provided for the whole shear span.

6. CONCLUSIONS

The new reinforcing system of concrete beams by internal CFRP grid in flexure and shear was studied. Based on the experimental and calculated investigations, the following conclusions can be drawn:

1) The new alternative reinforcement by internal CFRP grid system was confirmed to enhance the performance of concrete elements both in flexure and shear. The reinforcing grids started their tensile-resisting ability after the occurrence of flexure crack in the flexure test and after the diagonal shear crack in the shear test.

2) From the elementary test, since a good reinforcing efficiency of the internal grids was obtained and the actual tensile strength of the grids was not far from its ultimate strength, it is reasonable to apply this system for reinforcing concrete beams.

3) In the shear reinforcing system, the location of internal grid in the shear span of concrete beams is very important. The more number of grid strips at the center of the shear span, the more effective it is in resisting the propagation of the diagonal shear crack, resulting in a higher shear capacity of the beams. In fact, the shear resisting effectiveness of the grids became smaller when the grid strips were fully provided for the whole shear span.

4) With the same spacing of CFRP grids, a greater amount of the internal reinforcing grids in the
shear span of concrete beams can lead to a higher shear carried by internal grid \( (V_g) \) and also a higher total shear capacity \( (V_{exp}) \). However, the increment of \( V_g \) is not proportional to the number of grid strips because each strip of the grids does not equally resist the shear force.

5) A model considering the influence of the location of CFRP grid in the shear span is reasonable to evaluate the shear capacity of the internal CFRP grid. The average of the values between experimental results and calculated results was 0.95 and the coefficient of variation was 13.3%.

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