Diagonal reinforced coupling (DRC) beams can provide excellent strength, stiffness and energy dissipation capacities. However, fabricating DRC beams at construction sites is difficult due to reinforcement congestion and interference among the reinforcements due to the complex arrangement required by current design provisions. The objective of this study is to simplify the reinforcement details of DRC beams by reducing the transverse reinforcement around the beam perimeter that is required for the confinement of DRC beams. This is achieved by using high-performance fiber-reinforced cement composites (HPFRCC) for DRC beams. Experiments are conducted using six specimens under reversed cyclic loads to evaluate the hysteretic behaviours of the HPFRCC DRC beams. The test results show that the use of HPFRCC in DRC beams is very effective, providing a confinement effect with transverse reinforcement. HPFRCC DRC beams with half the transverse reinforcement required by ACI 318-11 provide almost identical energy dissipation capacities as concrete DRC beams detailed according to ACI 318-11.

Notation

- $A_{ct}$: cross-sectional area of coupling beam
- $A_{rd}$: area of reinforcement in each group of diagonal bars
- $b_w$: beam web width
- $d_b$: nominal diameter of bar
- $F$: force
- $F_{\text{max}}$: maximum force
- $F_{\text{min}}$: minimum force
- $f_c'$: compressive strength of concrete
- $h$: height of beam
- $k_1$, $k_2$: first and second cycle stiffnesses
- $L_a$: length of beam
- $V_f$: volumetric ratio of fibres
- $V_{\text{sh}}$: maximum shear force obtained from hysteretic curve
- $\Delta$: drift
- $\Delta_{\text{max}}$: maximum drift
- $\Delta_{\text{min}}$: minimum drift
- $\theta$: drift ratio obtained from hysteretic curve
- $\theta_f$: drift ratio at failure obtained from hysteretic curve
- $\theta_{\text{sh}}$: maximum drift ratio obtained from hysteretic curve
- $\theta_y$: yield drift ratio obtained from hysteretic curve

Introduction

Shear walls are commonly used as structural walls to resist lateral forces such as winds and earthquakes. Two shear walls can be coupled by coupling beams, leading to a more efficient and cost-effective structural system than shear walls without coupling beams (Lee and Kim, 2013). During earthquakes, coupling beams may experience repeated large shear deformations. If coupling beams in a coupled wall system avoid brittle failure, most of the energy caused by an earthquake can be dissipated by the coupling beams. Therefore, the seismic performance of coupled shear wall systems strongly depends on the energy dissipation capacity of the coupling beams (Taranath, 2010).

Paulay and Binney (1974) developed a coupling beam with diagonal reinforcement. Previous experimental research studies (Barney et al., 1980; Paulay and Binney, 1974; Tassios et al., 1996) reported that diagonal reinforced coupling (DRC) concrete beams provided energy dissipation capacities significantly larger than conventionally reinforced coupling beams, while they retained their strength and stiffness during cyclic loading with large displacement amplitudes.

ACI 318-11 (ACI, 2011) section 21.9.6 provides two confinement options for DRC beams. For the first option, each diagonal element consists of a cage of longitudinal and transverse reinforcement (Figure 1(a)). At least four reinforcements are required for each diagonal element. A simpler transverse reinforcement was introduced in ACI 318-11 (ACI, 2011) section 21.9.7.4 (d). As seen in Figure 1(b), transverse reinforcement for the second confinement option is only
placed around the beam perimeter rather than around the diagonal elements.

Even though the use of the second confinement option makes the reinforcement detail simpler, construction difficulties due to reinforcement congestion and interference still exist. In particular, construction difficulty becomes more serious when the shear stress on coupling beams is large and the beam is slender. Harries et al. (2005) reported that placing reinforcement in coupling beams becomes practically impossible as the beam shear stress approaches $\frac{0.5}{\sqrt{F_c}}$.

Research studies (Fortney et al., 2008; Galano and Vignoli, 2005) were conducted to develop simple reinforcement details for coupling beams. Canbolat et al. (2005) developed high-performance cementitious composite (HPFRCC) coupling beams to reduce reinforcement in coupling beams. Kuang and Bączkowski (2009) conducted experimental tests of large-scale, steel-fibre-reinforced concrete (SFRC) coupling beams. They demonstrated that HPFRCC coupling beams provided superior damage tolerance and stiffness retention capacities. It was reported that steel fibre in concrete can significantly increase the shear strength of structural concrete (Casanova et al., 1994; Lim et al., 1987). Many studies were conducted to investigate the effect of the fibres on the mechanical behaviour of concrete (Aslani and Nejadi, 2013; Tadepalli et al., 2014; Yan et al., 2013). To evaluate the cyclic performance of a coupled wall system containing HPFRCC coupling beams, Lequesne (2011) tested a four-storey coupled wall specimen with HPFRCC coupling beams. However, to date, no research has been conducted investigating the confinement effect of HPFRCC in coupling beams detailed according to ACI 318-11, section 21.9.7.4 (ACI, 2011).

In this study, experiments were conducted on DRC beams detailed according to the second confinement option specified in ACI 318-11, section 21.9.7.4(d). Six DRC beam specimens were constructed with beam aspect ratios ($L_n/h$) of 2.0 and 3.5, and they were subjected to reversed cyclic loads, where $L_n$ is the beam length and $h$ is the overall height of the beam. Naish et al. (2009) reported that residential and commercial buildings have slender coupling beams with large aspect ratios ($L_n/h$) ranging from 2.4 to 3.3. Among the six specimens, two DRC concrete specimens satisfying the requirement of the reinforcement details specified in ACI 318-11, section 21.9.7.4(d) were used as control specimens, and four HPFRCC DRC specimens were fabricated – of which two had half the transverse reinforcement used in control specimens and two had no transverse reinforcement. Although the contribution of fibres could be directly measured by comparing the test results of DRC specimens with and without fibres, which had half the transverse reinforcement used in control specimens, the DRC specimens without fibre were not made because of funding and time limitations. From the experimental results, the hysteretic behaviour of the HPFRCC DRC beams was evaluated.

**Experimental programme**

*Test specimens and test set-up*

Experiments were conducted using six half-scale diagonal reinforced coupling (DRC) beam specimens subjected to reversed cyclic loads. The main test variables were the application of HPFRCC, the amount of lateral reinforcement around the beam perimeter, and the beam aspect ratios (2.0 and 3.5).

Among the six specimens, two were control concrete DRC specimens (RC-2.0 and RC-3.5) without fibres, with aspect ratios of 2.0 and 3.5, respectively, which satisfy the second confinement option specified in ACI 318, section 21.9.7.4(d) (Figure 1(b)). The FRC-0-2.0 and FRC-0-3.5 HPFRCC DRC beam specimens had the same diagonal reinforcement as the control specimens, but they did not have any lateral

![Figure 1. Two confinement options for DRC beams specified in ACI 318-11: (a) first confinement option; (b) second confinement option](image-url)
reinforcement (no confinement reinforcement) around the beam perimeter and they were made using HPFRCC instead of normal concrete. From the test results of these specimens, the contribution of HPFRCC on the DRC beams could be demonstrated. The FRC-0·5–2·0 and FRC-0·5–3·5 specimens are HPFRCC DRC beams, which had the same diagonal reinforcement as the control specimens with half of the lateral reinforcement used in the control specimens. It is noted that the purpose of this study was to observe the contribution of fibres to the cyclic performance of DRC beams. To estimate the amount of the contribution by the fibres accurately, more specimens should be made with a wide range of test variables. Polyvinyl acetate (PVA) fibres at volume fraction \( V_f \) of 2% were used for the HPFRCC DRC beam specimens. The material mix and test results for HPFRCC will be discussed in a later section. The amount of diagonal reinforcement was determined to make the maximum shear stress of the beam equal to \( 0.5\sqrt{f'_c} \) (MPa). Table 1 summarises the dimensions and reinforcement details of the specimens. Figure 2 shows the reinforcement arrangement of the DRC specimens.

Quality control for members using HPFRCC at construction sites is more difficult than for concrete members. Thus, precast DRC beam specimens were made and tested. Shear keys were provided at the interface between the beam and stub. Concrete with a compressive strength of 60 MPa was used to construct the stubs and sufficient reinforcement was placed in the stubs to avoid failure before the DRC beam specimens failed.

Figure 3 shows the test set-up. A specimen was placed vertically and a loading frame was installed. The stubs at both ends of the coupling beam specimens were fixed to the strong floor and loading frame. Quasi-static reversed cyclic loading was applied to the specimen by an actuator through the loading frame. Two cycles were applied for each drift loading amplitude, ranging from 0·25% to 12%, as shown in Figure 3. To achieve a zero moment (inflection point) at the beam mid-span, the actuator was installed at the left-hand vertical girder (A) to make the line of force meet the mid-span of the beam (Figure 3). The right-hand vertical steel girder (B) was made for balancing the gravity load against the left-hand vertical girder (A). Four rollers were installed on two strong columns (C and D) to prevent rotation of the top of the beam specimen. Two strong columns were anchored to the strong floor. Thus, only horizontal displacement was introduced through the test set-up, as shown in Figure 3. Stopper blocks were placed at the top and bottom stubs of the beam specimens to prevent slip between the beam, loading frame and strong floor.

Lateral loads were measured by a load cell installed in the actuator. The lateral displacement was measured by linear variable differential transformers (LVDTs). As shown in Figure 4, vertical and diagonal LVDTs were used to estimate

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Width: mm</th>
<th>Height: mm</th>
<th>Span: mm</th>
<th>Aspect ratio: ( L_n/h )</th>
<th>Angle, ( \alpha ): degrees</th>
<th>Diagonal bars</th>
<th>Longitudinal bars</th>
<th>Transverse bars</th>
</tr>
</thead>
<tbody>
<tr>
<td>RC-2·0</td>
<td>250</td>
<td>525</td>
<td>1050</td>
<td>2·0</td>
<td>20·4</td>
<td>22·2</td>
<td>8</td>
<td>8</td>
</tr>
<tr>
<td>FC-0·5–2·0</td>
<td>250</td>
<td>525</td>
<td>1050</td>
<td>2·0</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>FC-0·5–3·5</td>
<td>250</td>
<td>300</td>
<td>1050</td>
<td>3·5</td>
<td>8·9</td>
<td>25·4</td>
<td>8</td>
<td>8</td>
</tr>
<tr>
<td>RC-3·5</td>
<td>250</td>
<td>300</td>
<td>1050</td>
<td>3·5</td>
<td>8·9</td>
<td>35</td>
<td>8</td>
<td>8</td>
</tr>
<tr>
<td>FC-0·5–3·5</td>
<td>250</td>
<td>300</td>
<td>1050</td>
<td>3·5</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>X denotes no reinforcement.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 1. Summary of test specimens
the deformations associated with flexure and shear forces. Strain gauges were installed at various locations of the diagonal and transverse reinforcement, as shown in Figure 2.

**Material tests**

To investigate the material properties of HPFRCC and normal concrete, compressive and tensile tests were conducted. PVA fibres were used in HPFRCC. The properties of the PVA fibre are summarised in Table 2. The volume ratio \( V_f \) of the fibres in HPFRCC was 2%. Trial mixes for HPFRCC having a compressive strength \( f_c' \) of 40 MPa were made and compressive strength tests were conducted, from which the best mix was identified, as summarised in Table 3. Three specimens for normal concrete and HPFRCC were made and tested. The diameter and height of the specimens were 100 mm and 200 mm, respectively. Figure 5(a) shows the stress–strain curves obtained from the compressive tests. The compressive strengths of the HPFRCC and concrete specimens exceeded 40 MPa. The HPFRCC specimens failed at 65% larger strains than the concrete specimens. However, the secant elastic modulus of the...
HPFRCC specimens was 40% less than that of the concrete specimens.

To evaluate the tensile performance of HPFRCC, a direct tensile test was conducted using three dog-bone type specimens, which had sectional dimensions of 25 mm × 50 mm and a gauge length of 200 mm, as shown in Figure 5(b). The test results are summarised in Table 4. Figure 5(b) shows the stress-strain curve obtained from the tests. The HPFRCC specimens behaved in a very ductile manner prior to failure.
Multiple cracks were detected on the specimen surface due to the bridging effect of the fibres. In the final stage of loading, the width of one particular crack of the specimen increased due to the loss of the bridging effect of the fibres and loss of its tensile strength. The maximum strain was 2·5%, and strain hardening was observed, which is an advantageous property of HPFRCC (Naaman and Reinhardt, 1996).

In the coupling beam specimens, reinforcement with diameters of 13 mm (D13), 22 mm (D22) and 25 mm (D25) was used. The mechanical properties obtained from the tensile tests for reinforcement are summarised in Table 5.

Analysis of the test results
Hysteretic curves
Figure 6 shows the hysteretic curves obtained from the test, relating shear force and drift ratio. The abscissa of the graph is the drift ratio ($\theta$), which was calculated as the lateral drift of the coupling beam divided by the beam span length, and the ordinate is the shear force normalised by $\sqrt{f'_c A_{ct}}$, where $A_{ct}$ is the cross-sectional area of the coupling beam. In Table 6, the important test results are summarised, including the yield drift ratio ($\theta_y$), maximum shear force ($V_u$), maximum drift ratio ($\theta_u$), and drift ratio at failure ($\theta_f$) obtained from the hysteretic curves. The yield and maximum drift ratios were determined according to the method used by Pan and Moehle (1989). The maximum drift ratio, $\theta_f$, is defined as the drift ratio when the shear strength decreases by 20%.

Table 2. PVA fibre properties

<table>
<thead>
<tr>
<th>Tensile strength: MPa</th>
<th>Elastic modulus: GPa</th>
<th>Diameter: mm</th>
<th>Length: mm</th>
<th>Volume fraction, $V_f,$ %</th>
</tr>
</thead>
<tbody>
<tr>
<td>1600</td>
<td>25</td>
<td>0·039</td>
<td>12</td>
<td>2·0</td>
</tr>
</tbody>
</table>

Table 3. HPFRCC mixture proportion (kg/m³)

<table>
<thead>
<tr>
<th>Cement</th>
<th>Fly ash</th>
<th>Silica fume</th>
<th>Water</th>
<th>Filler: Calcium carbonate</th>
<th>Superplasticiser</th>
</tr>
</thead>
<tbody>
<tr>
<td>489</td>
<td>374·9</td>
<td>32·6</td>
<td>366·8</td>
<td>684·6</td>
<td>3·3</td>
</tr>
</tbody>
</table>

Table 4. Compressive strength of concrete and HPFRCC

<table>
<thead>
<tr>
<th>Materials</th>
<th>Compressive strength, $f'_c,$ MPa</th>
<th>Maximum compressive strain, $\varepsilon_{cu},$ %</th>
<th>Direct tensile stress, MPa</th>
<th>Maximum tensile strain, %</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>44</td>
<td>0·23</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>HPFRCC</td>
<td>41</td>
<td>0·46</td>
<td>4·3</td>
<td>2·5</td>
</tr>
</tbody>
</table>

Figure 5. Result of material test: (a) compression test of concrete and HPFRCC; (b) direct tensile test of HPFRCC
The control concrete DRC specimens RC-2·0 and RC-3·5 with aspect ratios of 2·0 and 3·5, respectively, detailed according to the second confinement option for DRC concrete beams specified in ACI 318-11 (ACI, 2011), showed stable hysteretic behaviour (Figures 6(a) and 6(d)). The strength and stiffness of these two specimens did not deteriorate until the drift ratios reached 5% and 7%, respectively. Their drift ratios, \( \theta_u \) and \( \theta_y \), were 5·2% and 7·0% (RC-2·0), and 9·8% and 10·0% (RC-3·5), respectively.

Figures 6(b) and 6(e) show the hysteretic curves of the FC-0·5–2·0 and FC-0·5–3·5 HPFRCC DRC beam specimens with aspect ratios of 2·0 and 3·5, respectively. Note that these specimens had the same amount of diagonal reinforcement as the control specimens, but did not have lateral reinforcement (Figures 2(b) and 2(e)). They showed stable hysteretic behaviour before a sudden strength drop occurred at a drift ratio of 4%. The drift ratios, \( \theta_u \) and \( \theta_y \), were 4·0% and 5·0% (FC-0–2·0), and 4·1% and 5·0% (FC-0–3·5), respectively. For these specimens, \( \theta_u \) and \( \theta_y \) did not vary with respect to aspect ratio. Even though these specimens failed earlier than the corresponding control specimens, HPFRCC in the FC-0–2·0 and FC-0–3·5 specimens provided a confinement effect until the drift ratio reached 4·0%. However, it is noted that the load capacities of fibre-only beams were considerably lower than those of reinforced concrete DRC beams with shear reinforcement. For example, the load capacity of RC-2·0 was 1087 kN, whereas that of FC-0–2·0 was 775 kN. This indicates that the whole transverse reinforcement in DRC beams could not be replaced solely by fibres.

The FC-0·5–2·0 and FC-0·5–3·5 HPFRCC DRC specimens having half the lateral reinforcement used in the control specimens had almost identical hysteretic behaviour to the control specimens, as shown in Figures 6(c) and 6(f). They demonstrated hysteretic behaviour as stable as the control specimens. Their drift ratios, \( \theta_u \) and \( \theta_y \), were 5·9% and 7·0% (FC-0·5–2·0), and 9·9% and 10·0% (FC-0·5–3·5), respectively. The drift ratios, \( \theta_u \) of these specimens were slightly higher than the corresponding control specimens, which can be attributed to the contribution of the fibres. Thus, DRC beams with half the transverse reinforcement and fibres had almost identical

![Hysteretic curves](image-url)

**Figure 6.** Hysteretic curves: : (a) RC-2·0; (b) FC-0–2·0; (c) FC-0·5–2·0; (d) RC-3·5; (e) FC-0–3·5; (f) FC-0·5–3·5
strength and drift capacity to reinforced concrete DRC beams designed according to ACI 318-11 (Figure 1(b)).

Figures 7(a) and 7(b) show the $V_u$ and $\theta_u$ values of the specimens. HPFRCC DRC beam specimens with half the lateral reinforcement used in the control specimens provided $V_u$ and $\theta_u$ values as large as those of the corresponding control specimens. The contribution of fibres in DRC beams could be evaluated more accurately by comparing the test results of fibre-reinforced specimens FC-0·5–2·0 and FC-0·5–3·5 with half the transverse reinforcement used in control specimens, and the specimens with half the transverse reinforcement and no fibres. However, specimens having half the transverse reinforcement and no fibres were not made in this experimental programme, owing to the funding limit as mentioned earlier. Nevertheless, the contribution of fibres was clearly observed in HPFRCC specimens FC-0–2·0 and FC-0–3·5, which did not have transverse reinforcement. Even though they did not provide $V_u$ and $\theta_u$ values as large as those of the corresponding control specimens, their drift capacities were substantial, and were larger than 4%. With an increase in the aspect ratio, $V_u$ decreased and $\theta_u$ increased except for the HPFRCC specimen without lateral reinforcement.

Cracking pattern and failure

Figure 8 shows the crack distributions of the specimens with an aspect ratio of 2·0 at a drift ratio ($\theta$) of 2·0% and 5%, and at failure. In the control concrete DRC specimen, RC-2·0, the horizontal crack was first detected near the interface between the beam and stub at a drift ratio of 0·25%. With an increase in the loading amplitude, horizontal cracks near the interface were transformed into diagonal cracks. At a drift ratio of 1%,

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$V_u$: kN</th>
<th>$\theta_u$: %</th>
<th>$V_u$: kN</th>
<th>$\theta_u$: %</th>
</tr>
</thead>
<tbody>
<tr>
<td>RC-2·0 (+)</td>
<td>1004</td>
<td>2·0</td>
<td>1087</td>
<td>6·2</td>
</tr>
<tr>
<td>RC-2·0 (–)</td>
<td>1086</td>
<td>2·0</td>
<td>1117</td>
<td>5·2</td>
</tr>
<tr>
<td>FC-0–2·0 (+)</td>
<td>739</td>
<td>1·4</td>
<td>775</td>
<td>4·7</td>
</tr>
<tr>
<td>FC-0–2·0 (–)</td>
<td>743</td>
<td>1·4</td>
<td>909</td>
<td>4·0</td>
</tr>
<tr>
<td>FC-0·5–2·0 (+)</td>
<td>981</td>
<td>2·0</td>
<td>1073</td>
<td>5·9</td>
</tr>
<tr>
<td>FC-0·5–2·0 (–)</td>
<td>1078</td>
<td>2·0</td>
<td>1163</td>
<td>6·1</td>
</tr>
<tr>
<td>RC-3·5 (+)</td>
<td>437</td>
<td>2·0</td>
<td>507</td>
<td>9·8</td>
</tr>
<tr>
<td>RC-3·5 (–)</td>
<td>469</td>
<td>2·0</td>
<td>504</td>
<td>9·9</td>
</tr>
<tr>
<td>FC-0–3·5 (+)</td>
<td>430</td>
<td>2·0</td>
<td>437</td>
<td>5·0</td>
</tr>
<tr>
<td>FC-0–3·5 (–)</td>
<td>441</td>
<td>1·9</td>
<td>452</td>
<td>4·1</td>
</tr>
<tr>
<td>FC-0·5–3·5 (+)</td>
<td>466</td>
<td>2·3</td>
<td>484</td>
<td>9·9</td>
</tr>
<tr>
<td>FC-0·5–3·5 (–)</td>
<td>531</td>
<td>2·5</td>
<td>562</td>
<td>10·0</td>
</tr>
</tbody>
</table>

Table 6. Summary of experimental test results
many diagonal cracks were formed over the entire beam surface. When the drift ratios were 2·0% and 5·0%, the diagonal crack widths were as large as 1·5 mm and 4·8 mm, respectively. The diagonal reinforcement yielded at a drift ratio of 2%. At a drift ratio of 7·0%, the cover concrete of the specimen at the beam top fell off (Figure 8(c)) and diagonal bars were fractured. Considering that the specimen did not lose its strength until the diagonal bars fractured, the second

Figure 8. Crack progression and failures of specimens having $L_n/h = 2·0$: (a) 2·0% drift; (b) 5·0% drift; (c) at failure (7·0%); (d) 2·0% drift; (e) 4·0% drift; (f) at failure (5·0%); (g) 2·0% drift; (h) 5·0% drift; (i) at failure (7·0%)
confinement option specified in ACI 318-11 provided a satisfactory confinement effect for the concrete DRC beams.

HPFRCC DRC specimen FC-0.5-2.0 with half the lateral reinforcement used in the control specimen showed similar crack patterns at the initial loading stages. As the loading amplitude increased, more cracks were formed and distributed on the beam surface (Figures 9(g)-9(i)) compared to the control specimen. At drift ratios of 2.0% and 5.0%, the largest crack widths were 0.5 mm and 1.4 mm, respectively, which are

![Figure 9](image-url)
much smaller than those of the control specimen, and can be attributed to the bridging effect of the fibres. At a drift ratio of 7%, diagonal bars were fractured but no concrete cover fell off, unlike in the control specimen.

In the FC-0–2·0 HPFRCC specimen without lateral reinforcement, an initial horizontal crack was detected near the interface between the beam and stub at a drift ratio of 0·25%, and diagonal cracks were detected near the beam mid-span at a drift ratio of 0·5%. At a drift ratio of 1·5%, diagonal cracks passed through the entire beam surface. At drift ratios of 2·0% and 4·0%, the crack widths were as large as 3·4 mm and 15 mm (Figures 8(d) and 8(e)), respectively. Finally, the specimen lost strength at a drift ratio of 5·0% (Figure 8(f)). No fracture occurred in the diagonal bars, but yielding occurred in the bars. This indicates that the application of HPFRCC for DRC beams without lateral reinforcement provides some amount of a confinement effect, but it is not as large as that provided by the RC-2·0 and FC-0·5–2·0 specimens. Thus, HPFRCC alone cannot replace the entire amount of lateral reinforcement for DRC beams, even though it provides a significant amount of confinement.

The crack patterns of the specimens with an aspect ratio of 3·5 were similar to those of the specimens with an aspect ratio of 2·0 (Figure 9). However, more flexural cracks were detected on the beams with an aspect ratio of 3·5. For the RC-3·5 control specimen, at a drift ratio of 10% the cover concrete near the beam ends severely fell off (Figure 9(c)), while the diagonal bars and lateral reinforcement were simultaneously fractured. Subsequently, the specimen lost its strength. The FC-0·5–3·5 HPFRCC specimen also failed due to fracture of the diagonal bars at a drift ratio of 10% (Figure 9(i)) and more cracks were detected in it than in the control specimen. The FC-0·5–3·5 HPFRCC specimen failed due to large cracks passing through the entire beam at a drift ratio of 5% (Figure 9(f)), while no fracture was detected on the diagonal bars.

In summary, fibres in DRC beams made some contribution to the confinement, which was observed from the test results of fibre-only DRC beams. Even though specimens FC-0–2·0 and FC-0–3·5 were fibre-only DRC beams, they had drift capacities greater than 4%, a drift limit for special moment frame members. Despite the large compression force in the fibre-only DRC beams induced by diagonal reinforcement during the test, the beams did not split in early loading stages. At the stage of failure, the diagonal reinforcement in the beams experienced yielding, but no fracture. This indicated that fibres in the DRC beams made some contribution to the confinement, but the contribution of fibres was not as excellent as that of transverse reinforcement in the control DRC beams.

Cyclic strength, stiffness deterioration and energy dissipation capacities
To estimate the cyclic strength deterioration of all specimens, envelope curves were extracted from the hysteretic curves shown in Figure 6, as plotted in Figure 10. Irrespective of the aspect ratio, the control specimens and HPFRCC specimens with half the lateral reinforcement required for the control specimens had similar cyclic strengths and stiffness deterioration characteristics. After their maximum strength, the...
specimens having an aspect ratio of 2 experienced a more severe cyclic strength drop than the specimens with an aspect ratio of 3.5 (Figures 10(a) and 10(b)). The HPFRCC specimens without lateral reinforcement were not as strong as the corresponding control specimens. The strength drop in these specimens started at an earlier loading stage than in the control specimens. Even though fibre-only DRC specimens had lower load capacity than control specimens with transverse reinforcement, the load capacity of the fibre-only beam exceeded the capacity calculated using the strength equation specified in ACI 318-11 (ACI, 2011). The load capacity of FC-0-2.0 was 775 kN, whereas the strength calculated using the equation in ACI 318-11 (ACI, 2011) was 634 kN. For specimen FC-0-3.5, tested and calculated strengths were 437 kN and 382 kN, respectively.

The cyclic stiffness deterioration of all specimens is plotted in Figure 11. The stiffness was determined by connecting the peak positive and negative displacements, as shown in Figure 11(a). Similarly to the strength deterioration results, the

![Figure 11. Normalised peak-to-peak stiffness: (a) \( L_n/h = 2.0 \); (b) \( L_n/h = 3.5 \)](image)

![Figure 12. Cumulative energy dissipation: (a) \( L_n/h = 2.0 \); (b) \( L_n/h = 3.5 \)](image)
HPFRCC specimens with half the lateral reinforcement used in the control specimens had almost identical cyclic stiffness deterioration characteristics to the corresponding control specimens, which may be attributed to the stiffness retention effect provided by the fibres. The HPFRCC specimens without lateral reinforcement experienced more rapid cyclic stiffness deterioration than the corresponding control specimens.

Figure 12 shows the cumulative dissipated energy at each drift ratio. A similar observation to the cyclic stiffness and strength deterioration results was made. The HPFRCC specimens with half the lateral reinforcement had almost identical energy dissipation capacities to the corresponding control specimens, whereas the HPFRCC specimens without lateral reinforcement had much lower energy dissipation capacities than the other specimens.

**Summary and conclusions**

In this study, an evaluation was made of the cyclic performance of HPFRCC DRC beams with less transverse reinforcement than required by the second option of confinement in ACI 318-11 (ACI, 2011). The experiments were conducted using six specimens. The conclusions of the test results are as follows.

(a) The concrete DRC beam specimens detailed according to the second confinement option in ACI 318-11 (ACI, 2011) provided excellent cyclic performance. The specimens with aspect ratios of 2:0 and 3:5 failed at large drift ratios of 7% and 10%, respectively.

(b) The HPFRCC DRC beam specimens without lateral reinforcement had a drift capacity of 4% irrespective of the beam aspect ratio. This is attributed to the confinement effect of HPFRCC. However, HPFRCC alone without lateral reinforcement cannot provide as much of the confinement effect as provided by the control specimens. The specimens lost their strength prior to fracture of the diagonal bars. Thus, HPFRCC alone provided a significant amount of confinement effect, but it cannot replace the entire lateral reinforcement for DRC beams.

(c) The HPFRCC specimens with half the lateral reinforcement used in the control specimens provided almost identical drifts, strengths and energy dissipation capacities as the control specimens. These specimens lost their strength when the diagonal bars fractured as observed in the control specimens. The cyclic deterioration characteristics of their strength and stiffness, as well as energy dissipation capacities, were also similar to those of the control specimens. Thus, the use of HPFRCC in DRC beams may significantly reduce the amount of lateral reinforcement without a loss of cyclic performance. To draw a more complete conclusion, more tests need to be conducted.

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