Study of New RC Structures Using Ultra-High-Strength Fiber-Reinforced Concrete (UFC) — The Challenge of Applying 200 MPa UFC to Earthquake Resistant Building Structures

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Abstract

This paper describes the seismic behavior of new reinforced concrete (RC) building structures using ultra-high-strength fiber-reinforced concrete (UFC) with 200 MPa strength. A series of tests of columns and frames in UFC buildings subjected to seismic forces were conducted to obtain basic data of their behavior and to provide guides for design and construction. The test results are summarized as follows. 1) UFC, which is basically a brittle material, could be well confined with high-strength lateral reinforcements. 2) Stable behavior of columns could be obtained even under very high axial compression when they were well confined with high-strength lateral reinforcements. 3) Steel-fibers in UFC significantly enhanced the shear resistance of columns and frames. Analytical investigations indicated that the shear behavior of a column and a frame can be well evaluated by considering the contribution of steel fibers to the tensile resistance of UFC.

1. Introduction

Practical use of concrete with the specified design strength of 100 N/mm² for high-rise condominiums of reinforced concrete in Japan began in 1995. However, higher strength concrete is required to meet demand for taller buildings, wider living spaces and smaller size members. The announcement of the planned completion of a 59-story condominium using concrete with the specified design strength of 150N/mm² by August 2008 (Takenaka Corporation 2007) is but one example of the current trend toward the use of concrete with higher design strength for building structures.

Ultra-high-strength fiber-reinforced concrete (UFC), which is said to be able to realize compressive strength of up to 800 N/mm², may be one of the promising materials that can satisfy such demand; however, it has not yet been applied to buildings due to a lack of fundamental data and guides for design and construction. According to a survey of the Japan Concrete Institute (JCI), UFC with a specified design strength of 150 N/mm² or higher has been used for several small-scale bridge structures since 2002 to extend the span length and reduce the size of the web section of girders (JCI 2006). The Japan Society of Civil Engineers (JSCE) has established guidelines for the design and construction of structures using UFC (JSCE 2004). It should be noted that the JSCE cautions against the use of UFC with deformed bars for bridge structures as the shrinkage of UFC may cause cracks resulting from the restriction of the deformed bars.

Toward the development of new reinforced concrete structures that utilize UFC with the strength of 200 N/mm² (abbreviated to “200MPa-UFC”) and their practical use for buildings, a series of tests have been conducted at Hiroshima University since 2001 to investigate the seismic behavior of columns and frames in UFC buildings. First, UFC cylinders were tested in 2001 to obtain fundamental data on the compressive characteristics of UFC (Obata 2002). Then, rectangular columns were tested under uniaxial compression forces in 2002 (Kitakaze 2003) and under cyclic lateral forces in 2003 (Joko 2004). Further, interior beam-column subassemblages (partial frame) using 200MPa-UFC for both columns and beams were tested in 2004 (Joko 2005). The bending test of UFC prisms was added in 2006 to investigate the tensile resistance of UFC (Yamana 2007) since it was found that the contribution of steel fibers to the tensile resistance of UFC has a significant effect on the ductility and the shear resistance of columns and beam-column subassemblages (Joko 2005).

UFC consists of fine aggregates with a grain diameter of 2.5 mm or less, premixed powder (cement, silica and reactive micro-powder), water, superplasticizer and steel fibers (Fig. 1). It is essential that it be free of coarse aggregates. Steel fibers with a diameter of 0.10 to 0.20 mm and length of 10 to 20 mm are added to provide ductility. UFC must be subjected to steam curing to realize the designed high strength. UFC with compressive strength of 200 N/mm² requires 48 hours of steam curing at 90 degrees Celsius.

This paper describes the mechanical properties of UFC and the seismic behavior of columns and frames of UFC, as detailed below, based on the results of the above-mentioned tests.

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2. Compressive characteristic of UFC

2.1 Test outline

A uniaxial compression test of UFC cylinders was conducted to investigate the stress-strain relationship of UFC, which is basically a very brittle material like rock. The test cylinders were subjected to monotonically increasing axial forces up to failure. The post-peak loading was controlled to be very slow by a very small increment of axial displacement of the cylinder reading feed-back signals from displacement transducers.

The test cylinders, listed in Table 1, were 100 mm in diameter and 200 mm in height. The test variables were 1) compressive strength (120, 160, 200 N/mm²) and 2) type of contained fibers (non-fiber, 2.0 in volumetric percentage (vol.%) of steel-fiber and 3.0 vol.% of organic (PVA)-fiber). The steel fiber was 0.2 mm in diameter and 15 mm long while the PVA fiber was 0.3 mm in diameter and 15 mm long. The tensile strengths of the steel fiber and PVA fiber were 3000 N/mm² and 880 N/mm², respectively. The PVA fiber was used in this test for reference. All the test cylinders were subjected to steam curing in which the period and the temperature were determined in accordance with the design compressive strength (Table 1).

2.2 Test results and discussion

Figure 2 shows the test cylinder failure pattern. All the NF (non-fiber) cylinders showed explosive fracture as shown in Fig. 2(c). The fiber-including cylinders showed mostly diagonal sliding failure (Fig. 2(b)) or vertical splitting failure (Fig. 2(a)). Obviously, the contained fibers prevented explosive fracture or delayed crushing or splitting of the concrete.

Table 2 shows the test results. Each test result value in this table is the average value of the specimens with the same test variables. The stress-strain relationships of steel fiber-reinforced cylinders are shown in Fig. 3. It can be seen that 1) the loading curves became closer to straight lines with increases in compressive strength, 2) the strength significantly dropped immediately after the peak and 3) test cylinders still could carry axial stress 48 hours at 90 degrees C for 200 N/mm² strength.
after the sudden drop in strength. The observed elastic modulus is shown in Figure 4, where it is compared with the existing design equations. The observed elastic moduli of steel fiber-reinforced cylinders are larger than those of non-fiber and organic fiber-reinforced cylinders. The average observed elastic modulus is very close to the values calculated by the AIJ equation (AIJ 1999).

The relationship between the compressive strength of cylinder $\sigma_B$ and strain $\varepsilon_m$ at $\sigma_B$ is shown in Figure 5. Although the scatter is not small, the strain obviously increases with increases in $\sigma_B$. The equations by Fafitis and Shah and by Popovics, shown below, express the upper and lower boundaries, respectively, of the test results.

Fafitis and Shah’s equation

$$\varepsilon_m = 1950 + 14.6\sigma_B$$  \hspace{1cm} (1)

Popovics’ equation

$$\varepsilon_m = 763\sqrt[3]{\sigma_B}$$  \hspace{1cm} (2)

### 2.3 Evaluation of stress-strain relationship

A comparison of observed stress-strain relationships with existing equations (Obata 2002) indicated that 1) Muguruma’s equation, shown below as Equation 3, best fit the observed loading curves regardless of whether fiber was included and 2) no equations could evaluate the observed two-step post-peak curves, and therefore, it was considered necessary to propose equations to match the observed post-peak curves.

The following equations are proposed to match the observed stress-strain curves (Obata 2002).

**Loading zone**

$$\sigma = E\varepsilon + (E_c - E_m)(\varepsilon / \varepsilon_m)^2$$  \hspace{1cm} (3)

$$E = 33.5 (\gamma(2.4)(\varepsilon/60)^{1/3}$$  \hspace{1cm} (4)

**Post-peak zone (1): First curve**

$$\sigma = \sigma_B - 0.155(\varepsilon - \varepsilon_m)$$  \hspace{1cm} (5)

$$\varepsilon_m = 4270 \text{ (}\mu\text{)}$$  \hspace{1cm} (6)

**Post-peak zone (2): Second curve**

$$\sigma = f_0' \cdot \varepsilon - 0.0103(\varepsilon - \varepsilon_m')$$  \hspace{1cm} (7)

where $f_0' = 0.65 \sigma_B$ and $\varepsilon_m' = \text{strain at the stress } f_0'$.

Figure 6 shows the calculated and observed stress-strain relationships. Equation 3 was found to well predict the stress-strain relation in the loading zone. Equations 5 to 7 also well predicted post-peak behavior.

### Table 2 Test results.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Number of Specimens</th>
<th>Compressive Strength $\sigma_B$ (N/mm²)</th>
<th>Elastic Modulus $E_c$ (kN/mm²)</th>
<th>Poisson’s Ratio</th>
<th>Strain at $\sigma_B$ ($\mu$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>200FM2</td>
<td>12</td>
<td>190</td>
<td>50.5</td>
<td>0.203</td>
<td>4010</td>
</tr>
<tr>
<td>160FM2</td>
<td>3</td>
<td>168</td>
<td>48.9</td>
<td>0.193</td>
<td>4220</td>
</tr>
<tr>
<td>120FM2</td>
<td>3</td>
<td>139</td>
<td>46.6</td>
<td>0.187</td>
<td>3850</td>
</tr>
<tr>
<td>160FO3</td>
<td>4</td>
<td>159</td>
<td>43.3</td>
<td>0.182</td>
<td>3900</td>
</tr>
<tr>
<td>120FO3</td>
<td>3</td>
<td>141</td>
<td>43.6</td>
<td>0.200</td>
<td>3930</td>
</tr>
<tr>
<td>200NF</td>
<td>8</td>
<td>184</td>
<td>50.7</td>
<td>0.203</td>
<td>4100</td>
</tr>
<tr>
<td>160NF</td>
<td>3</td>
<td>171</td>
<td>47.3</td>
<td>0.194</td>
<td>4160</td>
</tr>
</tbody>
</table>

Fig. 3 Stress-strain curves (observed).

Fig. 4 Elastic modulus.
3. Tensile characteristics of UFC

3.1 Test outline

A bending test of UFC prisms was conducted to investigate the flexural tensile characteristics of UFC. The test prism was subjected to uniform bending moment at its mid-span (Fig. 7). The tensile strength and the ultimate tensile strain of UFC were examined based on the observed moment-curvature relationships. The influences of the amount of steel fiber, the compressive strength of UFC and the direction of casting fiber containing concrete on the tensile strength and the ultimate strain were examined.

The test specimens listed in Table 3 measured 100 mm by 100 mm in cross-section and 400 mm in length. The test variables were 1) specified design strength of UFC (120, 160, 200 N/mm²), 2) amount of steel fibers (non-fiber, 1.0, 2.0 and 3.0 vol.%) and 3) the direction of casting concrete (vertical casting and horizontal casting). The total number of test specimens was fifty four. The compressive strengths of the test cylinders are listed in Table 4.

The non-fiber specimens and 1.0 % steel fiber specimens failed immediately after crack initiation, while the specimens with 2.0% and 3.0 % steel fiber increased in strength even after crack initiation and gradually lost strength in the post-peak zone. Thus steel fiber was found to have a remarkable boosting effect on strength and ductility in the test specimens. The test results are listed in Table 5 in terms of tensile strength and ultimate tensile strain, which were evaluated referring to Shimizu’s method (Shimizu 2006). The tensile strength and ultimate strain values listed in this table were the characteristic values in the idealized rigid-plastic relation of tensile stress and tensile strain obtained from the observed...
moment-curvature relationship. Each test result value in the table is the average value of six specimens. As indicated in the table, the tensile strength increased with increases in the amount of steel fibers and the compressive strength. Horizontally cast specimens showed 5% higher tensile strength compared to vertically cast specimens.

Three specimens.

Fig. 8 Failure patterns and moment-curvature relationships.

3.3 Evaluation of tensile characteristics of UFC

The tensile characteristic of UFC was evaluated using the compressive strength of standard cylinder \( \sigma_B \). It is assumed that the 2.0% inclusion of steel fiber is the standard in practice. The relationship between the tensile strength of UFC and the amount of steel fiber is shown in Fig. 9. In this figure, the tensile strength is normalized by the strength for 2% inclusion. The tensile strength is proportional to the amount of steel fiber (Fig. 9(a)) and the relation is expressed with Equation 8. The increase in tensile strength with increases in the compressive strength of standard cylinder is shown in Fig. 9(b), and the relation is expressed with Equation 9. The tensile ultimate strain is proportional to the amount of steel fiber (Fig. 10(a)), a relation that is expressed with Equation 10. However, the strain is virtually constant against the compressive strength, \( \sigma_B \), as shown in Fig. 10(b).

\[
f(V_f) = 0.25V_f + 0.54 \quad (8)
\]

\[
\sigma_T = \alpha \sigma_B^{0.44} f(V_f) \quad (9)
\]

\[
\varepsilon_u = 0.15V_f + 0.185 \quad (10)
\]

where \( f(V_f) \) is the steel fiber effect coefficient, \( V_f \) is the amount of steel fiber (vol. %), \( \sigma_T \) is the tensile strength, \( \varepsilon_u \) is the ultimate strain and \( \alpha \) is the coefficient relating to the direction of casting UFC (1.00 for vertical casting and 1.05 for horizontal casting).

Table 5 Test results.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Maximum Moment ( M_{\text{max}} ) (kNm)</th>
<th>Curvature at ( M_{\text{max}} ) (( \mu \text{mm} ))</th>
<th>Tensile Strength ( \sigma_T ) (N/mm(^2))</th>
<th>Ultimate Tensile Strain ( \varepsilon_u ) (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>120FM2-V</td>
<td>3.43</td>
<td>54.3</td>
<td>8.13</td>
<td>4.20</td>
</tr>
<tr>
<td>160FM2-V</td>
<td>3.90</td>
<td>67.7</td>
<td>9.15</td>
<td>5.40</td>
</tr>
<tr>
<td>200FM0-V</td>
<td>3.41</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>200FM1-H</td>
<td>3.96</td>
<td>85.2</td>
<td>9.12</td>
<td>7.38</td>
</tr>
<tr>
<td>200FM2-H</td>
<td>4.53</td>
<td>74.4</td>
<td>10.87</td>
<td>5.63</td>
</tr>
<tr>
<td>200FM3-H</td>
<td>5.29</td>
<td>87.1</td>
<td>13.47</td>
<td>5.50</td>
</tr>
<tr>
<td>200FM1-V</td>
<td>3.43</td>
<td>45.2</td>
<td>8.25</td>
<td>3.57</td>
</tr>
<tr>
<td>200FM2-V</td>
<td>4.22</td>
<td>61.4</td>
<td>10.06</td>
<td>4.65</td>
</tr>
<tr>
<td>200FM3-V</td>
<td>5.69</td>
<td>86.7</td>
<td>13.34</td>
<td>6.60</td>
</tr>
</tbody>
</table>

Fig. 9 Tensile strength.

Fig. 10 Ultimate tensile strain.
4. Behavior of UFC columns under uniaxial compression

4.1 Test outline

A uniaxial compression test of UFC columns was conducted to find adequate methods to confine 200MPa-UFC, which is basically a very brittle material like rock. The columns were subjected to monotonically increasing axial forces up to failure. Nine rectangular section columns were constructed for the test (Fig. 11 and Table 6). The section of the columns was 200 mm by 200 mm and longitudinal reinforcements were 12-D10 with the specified yield strength of 685 N/mm². The test variables were 1) compressive strength of UFC (120, 160, 200 N/mm²), 2) amount of steel fibers (0, 2.0 vol.%), 3) diameter of lateral reinforcement (6.0, 7.1 mm) and 4) spacing of lateral reinforcement (35, 45, 55 mm). The specified yield strengths of high- and ultra-high-strength lateral reinforcements were 700 and 1400 N/mm², respectively. The amount of lateral reinforcement in terms of the ratio of pwσy to the compressive strength σB of UFC ranged from 0.0055 to 0.023 (Table 6) (pw is the lateral reinforcement ratio and σy is the yield strength of lateral reinforcement).

4.2 Test results and discussion

The stress-strain relationships of core concrete of the test sections are shown in Fig. 12, which also shows the stress-strain relationships of standard cylinders. As indicated in this figure, the compressive strength of core concrete was significantly higher than the standard cylinder strength except in the case of columns without steel fibers (NF columns). In the NF columns, the enhanced strength was very small. The compressive strength of core concrete fcc was enhanced with increases in the amount of lateral reinforcement pwσy / σB (Fig. 13). The ratio of the enhanced strength fcc to the standard cylinder strength σB was expressed with the following empirical equation.

\[ f_{cc} / \sigma_B = 0.92 \left( p_{ww} \sigma_y / \sigma_B \right)^{0.5} + 1.0 \] (11)

The compression ductility, which is expressed in terms of the ratio of the strain of column at the maximum stress εcm to the strain of the standard cylinder at its compressive strength εm, was enhanced with increases in the amount of lateral reinforcement, as shown in Fig. 14. The compression ductility εcm/εm is expressed with the following empirical equation.

\[ \varepsilon_{cm} / \varepsilon_m = 99.6 \left( p_{ww} \sigma_y / \sigma_B \right)^{2} + 1.0 \] (12)

<table>
<thead>
<tr>
<th>Specimen</th>
<th>σB (N/mm²)</th>
<th>Contained fibers (vol. %)</th>
<th>Lateral reinforcement (mm)</th>
<th>pw</th>
<th>σy (N/mm²)</th>
<th>pwσy</th>
<th>σB</th>
<th>B</th>
</tr>
</thead>
<tbody>
<tr>
<td>200FM2-35</td>
<td>222</td>
<td>Steel fiber (2.0)</td>
<td>35 2.3</td>
<td>1400</td>
<td>0.145</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>200FM2-45</td>
<td>181</td>
<td>Steel fiber (2.0)</td>
<td>45 1.8</td>
<td>1400</td>
<td>0.113</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>200FM2-55</td>
<td>159</td>
<td>Non-fiber</td>
<td>55 1.5</td>
<td>700</td>
<td>0.092</td>
<td>0.071</td>
<td></td>
<td></td>
</tr>
<tr>
<td>160FM2-35</td>
<td>200FM2-45</td>
<td>35 1.8</td>
<td>45 1.4</td>
<td>700</td>
<td>0.081</td>
<td>0.055</td>
<td></td>
<td></td>
</tr>
<tr>
<td>160FM2-45</td>
<td>200FM2-55</td>
<td>45 1.4</td>
<td>45 1.4</td>
<td>700</td>
<td>0.063</td>
<td>0.081</td>
<td></td>
<td></td>
</tr>
<tr>
<td>200NF-35</td>
<td>213</td>
<td>Non-fiber</td>
<td>35 2.3</td>
<td>1400</td>
<td>0.152</td>
<td>0.118</td>
<td></td>
<td></td>
</tr>
<tr>
<td>200NF-45</td>
<td></td>
<td></td>
<td>45 1.8</td>
<td>1400</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

σB : Compressive strength of UFC (N/mm²)

pw : Lateral reinforcement ratio (%)

σy : Yield strength of lateral reinforcement (N/mm²)

Fig. 12 Stress-strain relationship of core concrete.
5. Behavior of UFC columns under cyclic lateral loading

5.1 Test outline

A cyclic lateral loading test of columns of 200MPa-UFC was conducted to obtain basic data on their seismic behaviors because such data has not yet been reported. The columns were subjected to cyclic increasing lateral forces up to failure. Six rectangular section columns were constructed for the test. The cross section of the columns was identical to that of the columns for the uniaxial compression test described above (Table 7 and Fig. 15). The section of the test columns was 200 mm by 200 mm and the shear span ratio was 2.5 (Fig. 15).

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Axial stress ratio $\eta_0 = \sigma_0 / \sigma_B$</th>
<th>UFC $\sigma_0$ (N/mm²)</th>
<th>Lateral reinforcement</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>06FM23</td>
<td>0.6</td>
<td>218</td>
<td>35</td>
<td>F-C-Bu</td>
</tr>
<tr>
<td>06FM16</td>
<td>0.3</td>
<td>207</td>
<td>75</td>
<td>F-C-S-W</td>
</tr>
<tr>
<td>03NF16</td>
<td>0.6</td>
<td>199</td>
<td>50</td>
<td>F-C-Bu</td>
</tr>
<tr>
<td>03FM16</td>
<td>0.3</td>
<td>202</td>
<td>75</td>
<td>F-C-S-W</td>
</tr>
<tr>
<td>06FM05</td>
<td>0.6</td>
<td>214</td>
<td>75</td>
<td>F-C-Bu</td>
</tr>
<tr>
<td>03FM05</td>
<td>0.3</td>
<td>207</td>
<td>75</td>
<td>F-C-S-W</td>
</tr>
</tbody>
</table>

$\sigma_0$: axial stress, $\sigma_B$: compressive strength
$p_{ww}$: reinforcement ratio, $F$: main bar yielding, $C$: crush of concrete
$Bu$: main bar buckling, $S$: shear failure, $W$: rupture of hoop

Table 7 Test specimens.
(Cyclic lateral loading test of UFC columns)

Fig. 13 Enhanced strength of core concrete.

Fig. 14 Enhanced ultimate strain of core concrete.

06FM23  06FM16  03FM16  03NF16  06FM05  03FM05
Hoop space 35mm
$\rho_{ww}\sigma_0 / \sigma_B = 0.150$
Hoop space 50mm
$\rho_{ww}\sigma_0 / \sigma_B = 0.105$
Hoop space 75mm
$\rho_{ww}\sigma_0 / \sigma_B = 0.035$

Fig. 15 Test columns (Cyclic lateral loading test of UFC columns).
Longitudinal reinforcements were 12-D10 with the specified yield strength of 685 N/mm². The test variables were 1) axial stress ratio (0.3 and 0.6), 2) amount of steel fibers (0 and 2.0 vol.%) and 3) amount of lateral reinforcement ($p_{wB}/\sigma_y = 0.037-0.160$). The test specimens are listed in Table 7.

5.2 Test results and discussion
The lateral force vs. displacement relationship of each column is shown in Fig. 16 and the failure patterns of all the specimens are shown in Fig. 17. There were two types of failure mode. One is the type in which longitudinal reinforcements yielded in compression, concrete crushed and longitudinal reinforcements buckled (F-C-Bu: all the columns except 03FM05). The other was the type in which the shear failure followed by the rupture of lateral reinforcement took place after the crushing of concrete (F-C-S-W: 03FM05).

The moment vs. axial force relationship is shown in Fig. 18. The observed maximum strengths of all the columns except 03NF16 were much larger than the calculated flexural strength. The enhancement of the
strength is considered to be caused by confinement with lateral reinforcements. The ratio of the observed strength to the calculated flexural strength of the columns with the axial stress ratio of 0.6 became closer to 1.0 as shown in Fig. 19 when the effect of confinement with lateral reinforcement was evaluated based on Equation 11. The ultimate strength of the column without steel fibers could be evaluated based on the flexural strength of the core section.

The effects of the amount of both lateral reinforcement and steel fibers on the ductility of columns are shown in Figs. 20 and 21, respectively. The ultimate displacement, defined as the displacement at 80% the maximum strength after experiencing the maximum strength, significantly increased with increases in the amount of lateral reinforcement $\frac{p_{ww}\sigma_y}{\sigma_B}$ for the case where $\eta_0 = 0.6$, as shown in Fig. 20. In the case of low axial force level ($\eta_0 = 0.3$), both the strength and the ductility were enhanced by the inclusion of steel fibers, as shown in Fig. 21.

5.3 Evaluation of flexural behavior of UFC columns
The relationship between lateral force and story drift of each UFC column was investigated using fiber-model analysis of the column section (Murakami 2007). In the analysis, the previously discussed tensile characteristic of UFC and the confinement to UFC by lateral reinforcements were considered. The analytical model of confined UFC is shown in Fig. 22 and the constitutive equations are shown below.

**Compression Side**

\[
\sigma = E_{UFC} \varepsilon \quad (0 \leq \varepsilon \leq \varepsilon_c^*) \quad (13)
\]

\[
\sigma = \frac{f_c}{\varepsilon_{UFC}} \left(\varepsilon - \varepsilon_c^*\right) + f_c^* \quad (\varepsilon_c^* \leq \varepsilon \leq \varepsilon_{UFC}) \quad (14)
\]

\[
\sigma = \frac{f_c}{6} (\varepsilon - 6) \quad \left(\varepsilon_{UFC} \leq \varepsilon \leq 6\right) \quad (15)
\]

\[
UFC \varepsilon_c = (1.8 \left(\frac{P_{core}\sigma_y}{f_c}\right) + 1) f_c \quad (16)
\]

\[
UFC \varepsilon_c = (5 \left(\frac{P_{core}\sigma_y}{f_c}\right) + 1) \varepsilon_c \quad (17)
\]
where \( c_{UFC} \sigma_b \) is the compressive strength of UFC standard cylinder and \( V_f \) is the amount of steel fiber (vol.%).

The calculated lateral force vs. story drift relationships are shown in Fig. 24 for the cases with a low axial force column (03FM16) and a high axial force column (06FM16). The curvature distribution along the column height was assumed as shown in Fig. 23. Shear stiffness was assumed to be elastic. As shown in Fig. 24, generally the calculation result agreed well with the test result in the case of the low axial force column, though the flexural strength was slightly underestimated. Consideration of the tensile resistance of UFC slightly pushed up the flexural strength. In the case of the high axial force column, the flexural strength was more underestimated while the tensile resistance of UFC did not contribute to the flexural strength. The shape factor, the ratio of the strength of concrete in the column to the standard cylinder strength, of 0.85 for the compressive strength of UFC might have resulted in further underestimation of flexural strength.

6. Behavior of interior beam-column subassemblages under cyclic lateral loading

6.1 Test outline

In order to obtain fundamental data on the seismic behavior of beam-column subassemblages (partial frame) using 200MPa-UFC, a cyclic lateral loading test of interior beam-column subassemblages was conducted. Four interior beam-column subassemblages half the real scale were constructed for the test. The dimensions and arrangement of the reinforcing bars and concrete are listed in Tables 9 and 10. The test variables were 1) volume of steel fibers (0 and 2.0 vol.%) and 2) development length

Fig. 22 Analytical model of confined UFC.

Fig. 23 Assumed curvature distribution.

Fig. 24 Flexural displacement (comparison with test results).
of longitudinal reinforcement of the beam in the joint panel (14d and 17d; d: bar diameter). Every test specimen was designed so that both the shear failure of the joint panel and the flexural failure of the beam might take place. The shear strength of the joint panel was estimated by extending the following equation proposed by the Architectural Institute of Japan for ordinary reinforced concrete buildings (AIJ 1999).

\[ V_{ju} = 0.8\sigma_u^{0.7} b_j D_j \]  

(23)

where, \( \sigma_u \) is the compressive strength of concrete (N/mm²), \( b_j \) is the effective width of joint panel (average of the widths of beam and column) and \( D_j \) is the depth of the column.

6.2 Test results and discussion

The relationship between story shear force and story drift index of the specimens is shown in Fig. 26. Cracks and failure patterns are shown in Fig. 27. The failure process
was identical in all the specimens, e.g., 1) longitudinal reinforcements of the beam yielded in tension, 2) lateral reinforcements in the joint panel yielded and 3) the joint panel failed in shear compression of concrete. Thus both the flexural failure of the beams and the shear failure of the joint panel took place as designed. The ultimate story drift index was 3% in the J17 series specimens and 8% or larger in the J14 series specimens.

The envelopes of hysteresis curves of story shear forces or joint panel shear stress are shown in Figs. 28 and 29. In all the specimens, the observed maximum strength was much larger than the calculated flexural strength or the calculated shear strength of the joint panel. The observed strength was 1.5 times larger in J14FM specimens and 1.8 times larger in J17FM specimens than the calculated flexural strength. This indicates that both the flexural strength of the beams and the shear strength of the joint panel were significantly enhanced with the inclusion of steel fibers.

The observed joint panel shear stresses are plotted in Fig. 30 together with the results of other tests (Kimura 1989, Nakazawa 2001, Torii 2003, Iwaoka 2003 and Maruta 2004). AIJ Equation 23 is shown in Fig. 30. Equation 23 can be seen to overestimate the strength of other tests, although the strengths of non-fiber specimens in this test can be estimated with the equation. The strength of fiber-reinforced specimens is much larger than the strength calculated with Equation 23. The effect of the steel fibers on the shear strength of the joint panel should be evaluated.

### Table 9 Mechanical properties of reinforcing bars.

<table>
<thead>
<tr>
<th>Reinforcing bar</th>
<th>Diameter</th>
<th>Yield point</th>
<th>Strain</th>
<th>Tensile strength</th>
<th>Breaking elongation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Standard</td>
<td></td>
<td>Strength</td>
<td>Strain</td>
<td>Tensile strength</td>
<td>(%)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(N/mm²)</td>
<td>(µ)</td>
<td>(N/mm²)</td>
<td></td>
</tr>
<tr>
<td>SD980</td>
<td>D29</td>
<td>1049</td>
<td>1075</td>
<td>12.2</td>
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</tr>
<tr>
<td>SD685</td>
<td>D25</td>
<td>707</td>
<td>4786</td>
<td>946</td>
<td>11.4</td>
</tr>
<tr>
<td>SD785</td>
<td>D8</td>
<td>1057</td>
<td>7039</td>
<td>1194</td>
<td>8.9</td>
</tr>
</tbody>
</table>

### Table 10 Properties of UFC.

<table>
<thead>
<tr>
<th></th>
<th>Compressive strength (N/mm²)</th>
<th>Elastic modulus (kN/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column(NF)</td>
<td>182</td>
<td>50.1</td>
</tr>
<tr>
<td>Beam(NF)</td>
<td>190</td>
<td>51.4</td>
</tr>
<tr>
<td>Column(FM)</td>
<td>205</td>
<td>53.1</td>
</tr>
<tr>
<td>Beam(FM)</td>
<td>211</td>
<td>55.0</td>
</tr>
</tbody>
</table>

Fig. 26 Story shear vs. story drift.
the additional shear strength of the joint panel equivalent to the tensile strength of UFC. The additional shear strength, then, may be added to Equation 23 as follows.

$$\tau_p = 0.8\sigma_B^{0.7} + \sigma_T$$  \hspace{1cm} (24)

The tensile strength $\sigma_T$ of the UFC of the test specimen is calculated as follows using Equations 8-10 and the compressive strength $\sigma_B$ of standard cylinder as 208 N/mm² (average of beams and columns).

$$f(V_f) = 0.25V_f + 0.54 = 1.04$$  \hspace{1cm} (25)

$$\sigma_T = \sigma_B^{0.44} f(V_f) = 208^{0.44} \times 1.04 = 10.9 \text{ N/mm}^2$$  \hspace{1cm} (26)

$$\tau_p = 0.8\sigma_B^{0.7} + \sigma_T = 0.8\sigma_B^{0.7} + 10.9$$  \hspace{1cm} (27)

Equation 27, shown in Fig. 30, agrees well with the test results.

7. Concluding remarks

The results of a series of tests to put 200MPa-UFC to practical use for earthquake resistant buildings and subsequent analytical investigations are summarized as follows.

1) UFC, which is basically a very brittle material like rock, can be well confined with high- or ultra-high-strength lateral reinforcements available in the Japanese market. The compressive characteristics of UFC are evaluated using Equations 3-7 while the tensile characteristics are evaluated using Equations...
2) Stable seismic behavior of columns can be obtained even under very high axial compression when they are confined with high- or ultra-high-strength lateral reinforcements. The flexural behavior of UFC columns is well evaluated when taking the tensile resistance of UFC and confinement by lateral reinforcements into consideration and using the analytical model shown in Fig. 22.

3) Both shear failure of the joint panel and flexural failure of beams can concurrently take place. Steel fibers significantly contribute to enhance the shear strength of the joint panel. The contribution of steel fibers is evaluated by Equation 24. The shear strength of the joint panel without steel fibers can be evaluated by AIJ equation 23 for ordinary concrete.

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References


