Effect of Tie Bar Volume on the Seismic Performance of Polypropylene Fiber Reinforced Columns based on Hybrid Loading Experiments

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It has been shown based on previous research that damage of RC bridge columns can be mitigated and ductility capacity can be enhanced by implementing polypropylene fiber-reinforced cement composites (PFRC) at the plastic hinge. The bridging action of polypropylene fibers which can mitigate crack propagation and widening of cracks has similar action with the lateral confinement by tie bars. Thus, it is likely that use of PFRC can reduce amount of tie bars required for ductility capacity enhancement. This paper shows a feasibility study for reducing an amount of tie bars by implementing PFRC at the plastic hinge. Based on a hybrid loading experiment and a nonlinear dynamic response analysis, it is shown that it is feasible to reduce an amount of tie bars by half by using PFRC though failure of the column slightly increases.

Key Words : high performance material, polypropylene fiber, seismic design, ductility capacity, hybrid loading experiment, bridge column

1. INTRODUCTION

High-performance fiber reinforced cement composites (HPFRCC) are materials that exhibit multiple fine cracks upon loading in tension which leads to improvement in toughness, fatigue resistance, and deformation capacity. A type of HPFRCC that is now being widely used for structural and retrofit applications is called engineered cementitious composites (ECC)1). It has tensile strain capacity of about 0.03-0.05 resulting from the formation of closely spaced micro cracks due to the bridging action of fibers2) 3) . It has low elastic stiffness compared to concrete due to the absence of coarse aggregates and larger strain at peak compressive strength.

Polypropylene fiber-reinforced cement composites (PFRC) is a type of ECC4). Application of PFRC to bridge columns at the plastic hinge was investigated based on bilateral cyclic loading to a 1.68 m high, 0.4 m by 0.4 m square cantilever column5). Other two columns, each using steel fiber-reinforced concrete (SFRC) and regular high strength concrete (RC), were also investigated. The column using PFRC had superior performance than the other columns due to the substantial mitigation of cover and core concrete damage, longitudinal bar buckling, and deformation of tie bars at the plastic hinge region resulting from the high tensile strain capacity of PFRC which delays the propagation and widening of cracks and the high compression strain capacity which avoids loss of integrity of cover concrete by crushing and spalling. Thus the use of PFRC is promising for developing "damage free bridge columns" which are free from repair even after a major event.

The use of ECC with polyvinyl alcohol (PVA) fibers for repairing a scaled column reinforced with shape-memory alloys (SMA) subjected to shake-table excitations were explored by Saiidi and Wang6). The ECC-repaired column exhibited only minor damage even under high-amplitude motions.
Saiidi et al. then implemented these materials at the plastic hinge of scaled columns subjected to unilateral cyclic loading\textsuperscript{7}). Use of PVA-ECC substantially reduced damage and use of SMA bars reduced residual displacements. HPFRCs have also been used for seismic retrofit applications such as concrete jacket by Kosa et al.\textsuperscript{8}). The deformation capacity and energy absorption capacity of structural members were significantly improved.

The effectiveness of PFRC was also investigated based on shaking table experiments on a full-size bridge column (C1-6 column) subjected to near-field ground motions\textsuperscript{9}). The use of PFRC substantially reduced the apparent damage which can allow the bridge to be serviceable. PFRC did not have the brittle compression failure of regular reinforced concrete under repeated large inelastic deformation due to the bridging mechanism of fibers. Cyclic loading experiments on 6/35 scaled model of C1-6 was also conducted to study the effectiveness of PFRC\textsuperscript{10}).

Traditionally, the lateral confinement of core concrete and buckling-mitigation of longitudinal bars is achieved by providing sufficient amount of tie bars at the plastic hinge. However, past investigations suggested that an amount of tie bars could be reduced if PFRC was used instead of normal concrete due to the bridging action of PFRC\textsuperscript{9), 10}). Since tie bars have to be set densely in reinforced concrete columns at the plastic hinge, it is beneficial if congestion of reinforcements can be mitigated by the use of PFRC.

Based on the above concepts, two scaled model columns were constructed for investigating the effect of reducing the amount of ties. The volumetric tie reinforcement ratio $\rho_s$ was set as 1.51 % and 0.76 % in two models. The specimen with $\rho_s$ of 1.51 % and 0.76 % is called hereinafter PFRC-1 and PFRC-2 models, respectively. Hybrid loading experiment and nonlinear dynamic response analysis were conducted imposing two lateral ground accelerations recorded at JR Takatori Station during the 1995 Kobe, Japan earthquake.

2. MODEL COLUMNS

Fig. 1 shows the configuration of C1-6 cantilever type column designed to have flexure dominated failure\textsuperscript{9}). It is a 7.5 m tall, 1.8 m by 1.8 m square column designed in accordance with the 2002 JRA code\textsuperscript{11)} assuming moderate soil condition under the Type II design ground motion (near-field ground
motion) as shown in Fig. 2. PFRC was used at a part of the column with a depth of 2.7 m above the column base and a part of the footing with a depth of 0.6 m below the column base to minimize the volume of PFRC. The 2.7 m depth of PFRC is three times the estimated plastic hinge length of one-half the column width corresponding to 0.9 m to avoid failure at the PFRC-concrete interface. Regular concrete was used in the other parts of the column. The design compressive strength of PFRC was 40 MPa. The mixture consisted of cement mortar, fine aggregates with maximum grain size of 30 mm, water, and 3% volume of polypropylene fibers. Fibrillated polypropylene fibers with diameter of 42.6 μm, length of 12 mm, tensile strength of 482 MPa, Young's modulus of 5 GPa, and density of 0.91 kg/m³ were used. Superplasticizers were added to improve the workability of the mix.

Concrete with design compressive strength of 30 MPa was used and the actual average 28-day cylinder compressive strength was 41 MPa. The longitudinal and tie bars had nominal yield strength of 345 MPa (SD345). The actual yield strength, tensile strength and elastic modulus of longitudinal bars was 386 MPa, 577 MPa, and 197 GPa, respectively. Eighty 35 mm diameter deformed longitudinal bars were set in two layers corresponding to a reinforcement ratio of 2.47%. Deformed 22-mm diameter ties with 135 degree bent hooks lap spliced with 40 times the bar diameter were provided. Outer ties were spaced at 150 mm and inner ties were spaced at 300 mm throughout the column. Cross-ties with 135 degree bent hooks at 150 mm spacing were provided, as shown in Fig. 1 to increase confinement of the square ties. Volumetric tie reinforcement ratio within 2.7 m from the column base was 1.72%. The actual yield strength, tensile strength and elastic modulus of tie bars was 396 MPa, 590 MPa, and 192 GPa, respectively.

Based on 2002 Design Specifications, the flexural capacity of C1-6 column was 2547 kN, and the yield and ultimate displacements were 0.042 m and 0.30 m, respectively. The fundamental natural period was about 0.6 s at the small response range.

Two specimens were designed by scaling down C1-6 column based on a geometrical scale of 6/35 as shown in Fig. 3. They were 1.63 m tall models with a 300 mm square section. Eighty 6 mm diameter deformed longitudinal bars were set in two layers corresponding to a reinforcement ratio of 2.61%. Deformed 4-mm diameter ties with 135 degree bent hooks lap spliced with 40 times the bar diameter were provided. Outer ties were spaced at 26 mm and inner ties were spaced at 52 mm throughout the column in PFRC-1, while in PFRC-2, they were spaced at double of the spaces of PFRC-1. Four cross-ties with 135 degree bent hooks were provided at 26 mm and 52 mm spacing in PFRC-1 and PFRC-2, respectively, similar to C1-6 column. Volumetric tie reinforcement ratio within 450 mm from the column base was 1.51%. PFRC with design compressive strength of 45 MPa and the same mixture with that of C1-6 were used within 450 mm from the base. Though PFRC was used at a part of the footing with a depth of 0.6 m below the column base in C1-6, PFRC was not used in the PFRC-1 and PFRC-2. Normal concrete with design strength of 30 MPa was used in the rest parts. The actual yield strength, tensile strength and elastic modulus were 369 MPa, 557 MPa, and 182 GPa, respectively in longitudinal bars, and 380 MPa, 519 MPa, and 188 GPa, respectively in ties and cross ties.

Loading experiments were conducted using three dynamic actuators in Tokyo Institute of Technology. To identify four faces of the model, they are called hereinafter N, S, E and W faces as shown in Fig. 3. Two lateral actuators were set at 1790 mm from the column base (loading point) on N and E faces.
vertical force at the top of models, strains of longitudinal bars, ties and cross ties at the plastic hinge were measured by a digital data acquisition system. Axial strains of two cross ties directing in the NS and EW directions were measured at a height of 119 mm from the column base at 1) center, 2) 50 mm south from the north edge and 50 mm north from the south edge of the cross bar in the NS direction, and 3) 50 mm north from the south edge and 50 mm south from the north edge of the cross bar in the EW direction. They are represented such as C (cross tie)-NS (NS direction)-N (near north edge), C (center) and S (near south edge) as shown in Fig. 3(c).

3. HYBRID LOADING EXPERIMENTS

Hybrid loading experiment, which is also known as pseudo-dynamic experiment, was conducted in this investigation. Under a constant axial force of 86.4 kN in the column axial direction, the bilateral near-field ground accelerations recorded at JR Takatori Station during the 1995 Kobe earthquake (refer to Fig. 4) were imposed to the models assuming that a model column and a part of superstructure supported by the column were idealized as a SDOF system. The initial natural period $T$ of the two models was assumed 0.7 s which is generally the case for a bridge supported by a short column. The yielding stiffness $k_y$ was set as 5,000 kN/m according to previous research\(^{10}\). Therefore the lumped mass of the SDOF system $m$ was assumed 62.1 t from

$$m = \frac{k_y T^2}{4\pi^2} \quad (1)$$

in which $T$ is the assumed natural period and $k_y$ is the assumed yielding stiffness.

Damping ratio was assumed 0.05 throughout the experiments. Intensity of JR Takatori ground accelerations was varied in 3 levels so that the model columns behave virtually in the elastic range, slightly plastic range, and extremely plastic range. Since the hysteresis between flexural moment at the base vs. lateral displacement at the loading point was obtained from a previous study\(^{10}\) as shown in Fig. 5, the target response displacement of the models at the loading point was set as 20 mm during the 1st excitation, 55 mm during the 2nd excitation, and 75 mm during the 3rd excitation.

Consequently, assuming that the natural period of two models was 0.7 s, the intensities of ground acceleration during the 1st and 2nd excitations were set as 10 % and 20 % of the original record from the
response displacement of JR Takatori ground acceleration as shown in Fig. 6. Assuming that the natural period of the models extend to nearly 1.0 s due to flexural deterioration, the intensity of ground acceleration was set 25 % of the original during the 3rd excitation.

4. PROGRESS OF DAMAGE

1) PFRC-1 Column

Photo 1 shows the progress of damage in PFRC-1 after the 1st, 2nd and 3rd excitations were completed. Since the principal response direction of the lateral response of both PFRC-1 and PFRC-2 was nearly 20 degree from the E direction, the damage at the NE and the SW corners where the most significant damage occurred are shown here. The principle response direction is defined here as the direction from origin to the point where combined

two lateral response displacements was the maximum. It is seen in Photo 1 that almost no cracks were developed during the 1st excitation. Several horizontal cracks were developed during the 2nd excitation, and the base of PFRC column started to separate from the footing surface. They further progressed during the 3rd excitation.

Several important features are identified for the progress of damage. The first is that damage of the PFRC-1 was insignificant compared to damage of regular reinforced concrete columns. Almost no spalling off of covering concrete occurred even the peak response displacement reached 8.2 % drift in the principal response direction during the 3rd excitation, as which will be shown later. It is considered that 3 % drift is sufficient for a target drift of a regular reinforced concrete column in practice, if 3 % drift can be reliably assured for a column. Thus the 3rd excitation which resulted in 8.2 % drift was an extreme condition showing that stable response can be assured under 3 % response displacement.

The second is that a vertical crack occurred at the NE corner during the 2nd and 3rd excitations. The
Table 1  Peak response displacements (mm) and peak drift

<table>
<thead>
<tr>
<th>Excitations</th>
<th>PFRC-1</th>
<th></th>
<th>PFRC-2</th>
<th></th>
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<tr>
<td></td>
<td>NS</td>
<td>EW</td>
<td>Combined</td>
<td>NS</td>
</tr>
<tr>
<td>1st</td>
<td>9.5 (0.7 %)</td>
<td>16.7 (1.2 %)</td>
<td>18.3 (1.3 %)</td>
<td>9.9 (0.7 %)</td>
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<tr>
<td>2nd</td>
<td>33.2 (2.4 %)</td>
<td>51.2 (3.7 %)</td>
<td>59.7 (4.4 %)</td>
<td>32.8 (2.4 %)</td>
</tr>
<tr>
<td>3rd</td>
<td>55.4 (4.0 %)</td>
<td>98.2 (7.2 %)</td>
<td>112.2 (8.2 %)</td>
<td>56.1 (4.1 %)</td>
</tr>
</tbody>
</table>

Table 2  Residual displacement (mm) and residual drift

<table>
<thead>
<tr>
<th>Excitations</th>
<th>PFRC-1</th>
<th></th>
<th>PFRC-2</th>
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<tbody>
<tr>
<td></td>
<td>NS</td>
<td>EW</td>
<td>Combined</td>
<td>NS</td>
</tr>
<tr>
<td>1st</td>
<td>0.6 (0.0 %)</td>
<td>1.6 (0.1 %)</td>
<td>1.7 (0.1 %)</td>
<td>0.8 (0.1 %)</td>
</tr>
<tr>
<td>2nd</td>
<td>7.9 (0.6 %)</td>
<td>11.4 (0.8 %)</td>
<td>13.5 (1.0 %)</td>
<td>7.4 (0.5 %)</td>
</tr>
<tr>
<td>3rd</td>
<td>15.2 (1.1 %)</td>
<td>63.9 (4.6 %)</td>
<td>61.1 (4.5 %)</td>
<td>12.3 (0.9 %)</td>
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</tbody>
</table>

Fig. 8  Response displacement at loading point (PFRC-2)

Photo 2  Damage of PFRC-2 after 3 excitations
vertical cracks of covering concrete also occurred in C1-6 although it is not generally developed in regular reinforced concrete columns. The vertical cracks were developed due to strut action of covering concrete when the column was subjected to flexure compression at the NE corner. Since the covering concrete did not crush and spalled off as is the case of regular reinforced concrete columns, the shell consisting of covering concrete resulted in the strutting action. It should be noted that in regular reinforced concrete columns, such vertical cracks of covering concrete do not occur because covering concrete spall off at a much earlier stage.

The third is that no longitudinal reinforcements ruptured during the entire excitations. Local buckling of longitudinal bars was constrained by covering concrete (PFRC). Note that rupture of longitudinal bars did not occur in PFRC-2.

Fig. 7 shows the response lateral displacements at the loading point in the NS and EW directions during three excitations. The peak displacements were quite close to the target displacement. Table 1 summarizes the peak response displacements in the NS, EW and principal response directions. The peak response displacement was 112.2 mm (8.2 % drift).

It should be noted that the response displacement increased extensively during the 3rd excitation even though the intensity of ground acceleration increased only by 25 % compared to that of the 2nd excitation. This clearly shows that nonlinear response of a column sharply increases due to only 25 % increase of ground acceleration intensity once the column enters in the plastic range. Note also that residual drift was large during the 3rd excitation as shown in Table 2. Since the principal response direction was about 20 degree from the E direction leading to predominant failure at NE and SW corners, the residual displacement was larger in the N or E direction.

2) PFRC-2 Column

Photo 2 shows the progress of damage of PFRC-2. During the 1st and 2nd excitations, damage of PFRC-2 was in the same level with the damage of PFRC-1. However, during the 3rd excitation, spalling of covering concrete was slightly more extensive in PFRC-2 than PFRC-1. Thus, 50 % reduction of ties and cross ties at the plastic hinge is feasible without much deterioration of the seismic performance of PFRC-2 during the 1st and 2nd
excitations. This implies that 50% reduction of ties and cross ties at the plastic hinge is feasible if 3% is the target ductility drift. However, the effect of reducing the tie volume by half was observed once PFRC-2 undergoes in the strong hysteretic response under the 3rd excitation.

Fig. 8 shows the response displacements of PFRC-2 at the loading point. The general trend of the response was similar to that of PFRC-1. However as shown in Table 1, the peak response displacements are generally larger in PFRC-2 than PFRC-1. In particular, both the peak response displacements and peak residual response displacements during the 3rd excitation are larger in PFRC-2 than PFRC-1. This is obviously related to the more extensive damage of PFRC-2 at the plastic hinge shown above.

Fig. 9 compares hystereses of moment at the column base vs. lateral displacement at the loading point in the principal response directions for PFRC-1 and PFRC-2 under three excitations. As shown above, the hystereses are almost identical between PFRC-1 and PFRC-2 during the 1st and 2nd excitations. However, the hysteresis is larger in PFRC-2 than PFRC-1 near the peak response in the N or E direction.

5. BAR STRAINS

(1) Longitudinal bars

Fig. 10 shows axial strains of longitudinal bars at 208 mm from the base at SW corner. During the 1st excitation, the peak strain of the longitudinal bars was 2,000 \( \mu \)s, same as the yield strain (2,000 \( \mu \)) and 13,000 \( \mu \) in tension during the 1st and 2nd excitations, respectively. The strains are almost identical between PFRC-1 and PFRC-2. During the 3rd excitation, the peak strains were over 20,000 \( \mu \) with the strain of PFRC-2 slightly larger than that of PFRC-1. This resulted from slightly larger damage at column base in PFRC-2 than PFRC-1.

Figs. 11 and 12 show distribution of the peak axial strains of both outer and inner longitudinal bars at SW and NE corners for PFRC-1 and PFRC-2, respectively. Since the strains were out of the instrumentation range of the sensors during the 3rd excitations, they are not shown here. It is seen from Figs. 11 and 12 that bar strains are nearly identical between PFRC-1 and PFRC-2 during the 1st and 2nd excitations. Obviously bar strains are larger in the outer bars than the inner bars. During the 1st excitation,
longitudinal bars started to yield at three locations between the base and 208 mm from the base. During the 2nd excitation, the peak bar strains exceeded 15,000 \( \mu \) at NE corner at the base, though the strains 208 mm below the footing surface were still in the elastic range.

(2) Tie bars

Fig. 13 shows tie bar strains at the center of E face at 119 mm height from the base. Since among the tie bar
strains measured at the center of N, E, S and W faces and SW corner, the strain of the tie bar at the center of E face was the largest, and they are shown here. Both strains of the outer and inner ties during the three excitations are shown here. It is obvious that strain is larger in the outer tie than the inner tie which shows that the outer tie is more important than the inner tie for the lateral confinement. In PFRC-1, tie strains are always less than the yield strain during the 3rd excitation. On the other hand, in PFRC-2, the tie strains are similar with those of PFRC-1 during the 1st excitation. However they are nearly two to three times larger than those of PFRC-1 during the 3rd excitation.

The peak strain reached 3,700 \( \mu \) at the outer tie. Thus the outer tie yielded during the 3rd excitation though the inner tie was still in the elastic range. This resulted from more significant damage of core concrete in PFRC-2 than PFRC-1, leading to more extensive cracking of covering concrete in PFRC-2.

(3) Cross ties

Fig. 14 shows strains of a cross tie at 119 mm height from the base in the EW direction. Strains at the center and 50 mm west from the east edge (it is called East side hereinafter) of a cross bar are compared for the 3 excitations. Cross tie strains was larger at the center than East side. This is reasonable from the anticipated deformation mode of a cross tie which is subjected to tension due to volumetric extension of core concrete under flexural compression. At the center, the cross bar strain was less than 100 \( \mu \) during the 1st excitation, and it increased to over 700 \( \mu \) and 1,400 \( \mu \) during the 2nd and 3rd excitation, respectively. Compared to the cross bar strain of PFRC-1, the cross bar strain of PFRC-2 was nearly the same during the 1st excitation, but it became larger than the cross bar strain of PFRC-1 during the 2nd and 3rd excitations. Together with the increase of tie bar strains shown above, the increase of cross bar strain in PRFC-2 than PFRC-1 was resulted in deterioration of core concrete leading to decrease of the lateral confinement effect.

6. FIBER ELEMENT ANALYSIS

To analytically correlate the experimental results of PFRC-1 column, it was idealized by a discrete analytical model as shown in Fig. 15. The plastic hinge of the column was idealized by three 2 dimensional fiber elements as shown in Fig. 16 and the rest of the column was idealized by elastic beam elements. The pull-out effect of longitudinal bars from the footing (PLB-effect)\(^9\) was idealized by a set of rotational springs with stiffness as shown in Table 3. The rotational stiffness was determined so that computed natural periods of the column are in good agreement with the predominant period of the

![Fig. 15 Idealization of column and PLB-effect](image1)

![Fig. 16 Division of fiber elements in plastic hinge region](image2)

<table>
<thead>
<tr>
<th></th>
<th>1st excitation</th>
<th>2nd excitation</th>
<th>3rd excitation</th>
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<tbody>
<tr>
<td>NS</td>
<td>31,000 kNm/rad</td>
<td>18,000 kNm/rad</td>
<td>8,000 kNm/rad</td>
</tr>
<tr>
<td>EW</td>
<td>30,000 kNm/rad</td>
<td>17,000 kNm/rad</td>
<td>8,000 kNm/rad</td>
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![Fig. 17 Hysteretic model of PFRC developed by Yamada et al.\(^{14}\)](image3)
Fig. 18 Correlation of response displacement in the principal response direction

Fig. 19 Correlation of moment at the base vs. lateral force at the loading point hysteresis

Fig. 20 Computed stress vs. strain hysteresis for the 1st excitation
Fig. 21 Computed stress vs. strain hysteresis for the 2nd excitation

Fig. 22 Computed stress vs. strain hysteresis for the 3rd excitation
response of PFRC-1.

The stress-strain hysteresis of the column body outside of the plastic hinge region was idealized by a standard confinement model\(^1\), \(^2\), \(^13\), while PFRC at the plastic hinge was idealized as shown in Fig. 17\(^14\), \(^15\). This model considers an interaction of compression and tension hysterese, and loading, unloading and reloading path of PFRC. However since this model does not consider the effect of tie bars on the lateral confinement by PFRC, the computed result represents the response of PFRC-1 assuming that there were no ties and cross ties at the plastic hinge.

Fig. 18 shows correlation of analytical response displacements with the experimental results. Response displacements in the principal response direction are shown here. The analytical correlation for the response displacement for the 1st excitation is very well, and reasonably well except residual drift for the 2nd excitation. It is noted that prediction of residual drift is very sensitive to the post peak stiffness of the column\(^16\). Analytical correlation for the 3rd excitation further deteriorated since the computed response deviated from the experimental response due to residual drift. It should be noted that damage of PFRC-1 was extensive during the 3rd excitation, thus it is difficult to have a good correlation by analysis.

Fig. 19 shows an analytical correlation for the experimental hystereses of moment at the column base vs. lateral displacement at the column top. The hysterese in the principal response direction is shown here. Correlation of the hysteresis for the 1st and 2nd excitations is very well and reasonably well, respectively. It is noted that the computed restoring moment is in a good agreement with the experimental values for the 1st and 2nd excitations. Correlation of the hysteresis for the 3rd excitation is not sufficient reflecting the discrepancy between the experimental and computed response displacement (refer to Fig. 18 (c)).

Figs. 20, 21 and 22 show computed stress vs. strain hystereses of covering concrete, core concrete and a longitudinal bar at the base at the SW corner for the 1st, 2nd and 3rd excitations, respectively. The hysterese of covering concrete and core concrete at the extreme fiber are shown here. During the 1st excitation, stresses of the covering concrete and the core concrete have not yet reached the peak stress \(\sigma_{pc}\) and \(\sigma_{ce}\), and the outer and inner longitudinal bars are elastic. However during the 2nd excitation, not only covering concrete but also core concrete at the extreme fiber exceeded the peak stress \(\sigma_{pc}\) and \(\sigma_{ce}\) and deteriorated to the constant stress zone. Both the outer and inner longitudinal bars yielded with the peak strain of about 2,000 \(\mu\). During the 3rd excitation, stresses of both the covering concrete and core concrete at the extreme fiber at SW corner settled down to the constant plastic stress. The outer and inner longitudinal bars extensively entered into nonlinear range with the peak strain over 8 % in compression.

7. CONCLUSIONS

In this study, feasibility of reducing the amount of tie bars by implementing polypropylene fiber reinforced cement was investigated for a 6/35 scaled model of C1-6 (PFRC-1) and a 6/35 scaled model which reduced an amount of ties and cross ties by half (PFRC-2) based on a hybrid loading experiment and a nonlinear dynamic response analysis. Based on the results presented herein, the following conclusions may be deduced.

1) There was no distinct difference in the progress of damage and response displacements between PFRC-1 and PFRC-2 during the 1st and 2nd excitations where the peak column response in the principal response direction was 1.3 % drift and 4.2 % drift. However during the 3rd excitation where the peak response displacement in the principal response direction was 9 % drift, spalling off of covering concrete at the plastic hinge and residual drift were more significant in PFRC-2 than PFRC-1.

2) The above results were developed due to larger strains of longitudinal bars, and ties and cross bars in PFRC-2 than PFRC-1 during the 3rd excitation.

3) Analytical correlation for PFRC-1 shows reasonably well agreement with the experimental results for the 1st and 2nd excitation though the accuracy of correlation deteriorated due to poor prediction for residual drift for the 3rd excitation. It is found from the analysis that the core concrete at the extreme fiber was still in the increasing range during the 1st excitation, however the core concrete strain exceeded the strain corresponding to the peak stress and dropped to the constant strain range during the 2nd and 3rd excitations, respectively. The computed result well match with the progress of damage and strains of longitudinal bars and ties and cross ties.

4) Considering that 3 % drift is sufficient for a target drift of a regular reinforced concrete column in practice if 3 % drift can be reliably assured, the 3rd excitation which resulted in 8.2 % drift was an extreme condition. Thus, 50 % reduction of ties and cross ties at the plastic hinge is feasible without much deterioration of the seismic performance of PFRC-1 if the target ductility drift is 3 %.
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